Appendix D: Geotechnical Investigation

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GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL DEVELOPMENT

South of Agua Mansa Road, East of Former Cartier Lane
Colton, California
for
Lake Creek Industrial

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

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A California Corporation

March 12, 2019

Lake Creek Industrial
1302 Brittany Cross Road
Santa Ana, California 92705

Attention: Mr. Michael Johnson

Project No.: **18G212-2**

Subject: **Geotechnical Investigation**
Proposed Industrial Development
South of Agua Mansa Road East of Former Cartier Lane
Colton, California

Gentlemen:

In accordance with your request, we have conducted a geotechnical investigation and liquefaction evaluation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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TABLE OF CONTENTS

1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

Geotechnical Design Considerations

- The near-surface soils encountered at the ground surface at the boring locations consist of artificial fill materials, disturbed native alluvium and loose, undisturbed native alluvial soils. Most of the borings encountered disturbed alluvium at the ground surface, extending to depths of 2½ to 4½± feet. Artificial fill soils were encountered at the ground surface at one boring which was drilled the edge of an existing detention basin, extending to a depth of 8± feet. Borings performed by others for a previous study reported artificial fill soils extending to depths of 3 to 12± feet below the existing site grades. The remaining borings encountered loose native alluvium at the ground surface and most of the encountered loose alluvium beneath the disturbed soils and artificial fill, extending to depths of 6 to 10± feet. The disturbed alluvium, loose native soils, and artificial fill materials are generally underlain by medium dense to very dense alluvium, extending to the maximum depth explored of 50± feet.
- Fills of variable depths will be required throughout the site in order to facilitate the proposed development. Based on preliminary grading information, we expect that fills of 2 to 22± feet will be necessary to achieve the proposed site grades. Generally, fills of 2 to 12± feet are expected to be necessary throughout the majority of the site. Fills greater than 12 feet in depth are expected to be required in the existing detention pond areas in the southern-central portion of the site.
- The imported fill soils are expected to consist of crushed rock and soil materials mined from the CalPortland site located north of the subject site on the north side of Agua Mansa Road. These materials are predominately granular in composition, well-graded, and possess a maximum particle size of 6-inches.
- Remedial grading is recommended to remove the disturbed alluvium, artificial fill soils, and a portion of the loose near-surface alluvium from the proposed building areas and replace these soils as compacted structural fill.
- Our site-specific liquefaction evaluation identified liquefiable soils at two of the three 50±-foot deep borings.
- The potential liquefaction-induced settlements range from about 0.5 inches to 2.6± inches at Boring Nos. B-5 and B-7. No liquefiable soils were identified at Boring No. B-1. Differential settlements of up to 2± inches are expected to occur over spans of 100 feet, indicating a maximum angular distortion of approximately 0.0017 inches per inch.
- Based on the estimated magnitude of the differential settlements, the proposed structures may be supported on shallow foundations. Additional design considerations related to the potentially liquefiable soils are presented in this report.

Site Preparation

- Initial site preparation should include stripping of any surficial vegetation. Vegetation including grass and weed growth, trees, and any organic soils should be properly disposed of off-site.

Root masses associated with the trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils. Existing trash should be removed at the time of site stripping.

- The existing structures present on the subject site consist of telephone poles and metal towers in the southern and eastern portion so the site. Demolition of some of these telephone poles and towers may necessary in order to facilitate the proposed development.
- The existing soils within the proposed building areas should be overexcavated to a depth of 3 feet below existing grade and to a depth sufficient to remove disturbed alluvium and undocumented fills soils. Within the existing detention pond areas, undocumented fill soils extend to depths of 3 to 12± feet.
- Following evaluation of the subgrade by the geotechnical engineer, the exposed subgrade soils should be scarified, moisture conditioned to 0 to 4 percent above optimum, and recompacted. The resulting soils may be replaced as compacted structural fill.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,500 psf maximum allowable soil bearing pressure.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom) in strip footings due to the presence of potentially liquefiable soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slabs

- Conventional Slabs-on-Grade, 6 inches thick.
- Minimum reinforcement of the floor slab should consist of No. 3 bars at 16-inches on center in both directions, due to the presence of potentially liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer.
- Modulus of Subgrade Reaction: 150 lbs/in^3.

Pavements

ASPHALT PAVEMENTS (R = 40)					
Materials	**Thickness (inches)**				
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic (TI = 6.0)	Truck Traffic (TI = 7.0)	Truck Traffic (TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	4	6	7	8
Compacted Subgrade (90% minimum compaction)	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI =6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5½	5½	6½
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 18P417-3, dated December 13, 2018. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of the subject site, this investigation also included a site-specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located southeast of the former intersection of Agua Mansa Road and Cartier Lane in Colton, California. Cartier Lane has recently been demolished and redeveloped as a part of an adjacent industrial development. The site is bounded to the north by Agua Mansa Road, to the west by an existing industrial development, to the south by the Santa Ana River and a levee, and to the east by a water treatment plant and a single-family residence.

The subject site consists of several contiguous irregularly-shaped parcels, which total about 64 acres in size. The site is presently vacant and undeveloped with the exception of powerlines and telephone poles in the south and eastern portions of the site. The subject site is sparsely to heavily vegetated with grass and brush throughout. Some trees are present in the northeastern portion of the site.

Two irregularly-shaped basins are present in the south-central portion of the site. These basins were muddy and inaccessible to the drill rig at the time of subsurface exploration. The sides of the basins slope at inclinations of about 4h:1v to 5h:1v with heights of 5 to 10± feet.

Topographic information for the subject site was obtained from a survey prepared by JRN Civil Engineers, Inc. The survey indicates that the existing site grades range from a maximum elevation of 997± feet mean sea level (msl) along the eastern property line to minimum elevation of 971± feet msl in the southern portion of the detention basins located along the central portion of the southern property line. Generally, site topography in the east and central portions of the site slopes downward toward the basins. Site topography west of the basins slopes downward gently toward the southwest corner of the site.

3.2 Proposed Development

Our office was provided with two site plans for the proposed site development. Schemes 1 and 2, both indicate that the proposed site development will consist of two new industrial buildings constructed in the northern portion of the site along Agua Mansa Road with dock high doors located on the south sides of each building. Scheme 1 indicates that the western building will possess a footprint of 462,000± ft^2 and the eastern building will possess a footprint of 435,000± ft^2. Scheme 2 is similar to Scheme 1 except that Scheme 2 indicates that the eastern building will extend further to the east and possess a footprint of 447,000± ft^2. We expect that the buildings will be surrounded by asphaltic concrete pavements in the automobile parking and drive areas and Portland cement concrete in the truck court areas.

Detailed structural information is not currently available. It is assumed that the new buildings will be of concrete tilt-up construction, supported on conventional shallow foundation systems with concrete slab-on-grade floors. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 5 to 7 kips per linear foot, respectively.

Based on discussions with the client, we understand that 564,000± cubic yards of soil and crushed rock materials will be purchased from CalPortland, the present owner of the property, and imported to the subject site, in accordance with the terms of the sale agreement for the subject property. Grading plans for the proposed development are not available at the time of this report. However, a rough grading plan was prepared in 2015 for CalPortland, prepared by Otte-Berkeley Group, Inc. (OBG). This plan indicates that fills of 2 to 22± feet will be necessary to facilitate the previously planned rough grading. The deeper fills are indicated in the existing detention basin areas. Although this grading plan was not prepared for the currently proposed development, we understand that the proposed site grades may be similar to the pad grades depicted on the plan by OBG, based on a discussion with the project civil engineer. Therefore, at this time, we anticipate that fills on the order of 2 to 22± feet will be necessary in order to facilitate the proposed site grades. We understand that import of additional soil may be necessary, after the initial 564,000± cubic yards of soil have been placed, in order to achieve the proposed site grades. The proposed development is not expected to include any significant amounts of below-grade construction such as basements or crawl spaces.

3.3 Previous Studies

A previous geotechnical report was provided to our office by the client. The report is referenced as:

> Geotechnical Investigation, 75-Acre Site, South of Agua Mansa Road, Colton, California, prepared by CHJ Consultants, Inc. (CHJ), for CalPortland Company, dated April 8, 2016, CHJ Project No. 16113-3.

This report was prepared for a 75-acre overall site, which includes the 64-acre subject site. CHJ states in the introduction of the report that the purpose of the study was to "explore and evaluate the geotechnical engineering conditions at the subject site and to provide appropriate geotechnical engineering recommendations for preparation of the site before the placement of engineered fill for eventual site development." **The report lacks design recommendations for new structures and was not intended to be a design-level geotechnical investigation.** CHJ stated that "a site-specific geotechnical investigation should be performed to provide design-level recommendations for future proposed structures."

The subsurface exploration for the geotechnical investigation included a total of eight (8) hollow-stem auger borings, drilled to depths of 51½± feet, and sixteen (16) CPT soundings. All of the CPT soundings were advanced until refusal conditions were encountered in gravelly soils at depths of 4 to 38.9± feet below the existing ground surface.

CHJ identified native alluvium throughout the depths explored at most of the boring locations, with the exception of three borings which encountered soils classified as artificial fill or pond sediments at the ground surface. These three borings are located in the existing detention basins in the southern portion of the site and are identified as CHJ Boring Nos. B-3, B-4, and B-5 on the enclosed Boring Location Plan included as Plate 2, in Appendix A, of this report. Fill soils consisting of medium dense to dense fine sands and silty sands extend to depths of 3 to 12± feet at these boring locations.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of eight (8) borings, advanced to depths of 20 to 50± feet below currently existing site grades. All of the borings were logged during drilling by a member of our staff. In addition to these borings eight (8) borings and sixteen (16) CPTs were performed at the site for a previous geotechnical study, as discussed in the previous section of this report.

All borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. Samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

The approximate locations of the borings are indicated on the Boring Location Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B. The Boring and CPT logs for the previous study prepared by CHJ are included in Appendix F of this report.

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface at Boring No. B-5, extending to a depth of 8± feet below the existing site grade. The fill soils at this boring location consist of loose to medium dense fine sands with trace medium to coarse sand content, trace to little silt content, and trace fine gravel. These soils possess artificial debris, including plastic fragments, resulting in their classification as artificial fill.

Disturbed Alluvium

Soils classified as disturbed alluvium were encountered at the ground surface at most of the boring locations, extending to depths of 2½ to 4½± feet. These soils generally consist of loose silty sands, sandy silts, and fine sands. These soils possess a slightly disturbed appearance and loose relative densities but also resemble the near-surface native alluvium at the site.

Alluvium

Native alluvium was encountered at the ground surface at Boring Nos. B-2, B-4, and B-7 and beneath the disturbed alluvium or artificial fill soils at all of the remianing boring locations, extending to at least the maximum depth explored of 50± feet below existing site grades. The near-surface alluvial soils, within the upper 6½ to 10± feet, generally consist of loose silty fine sands, fine sandy silts, and fine sands. Boring B-7 encountered medium stiff to stiff clayey silts between depths of 0 and 3± feet. At greater depths, the alluvial soils generally consist of medium dense to very dense fine to medium sands, fine to coarse sands, silty fine sands, and fine sandy silts, and clayey sands with occasional very stiff to hard silty clay and sandy clay strata.

Groundwater

Groundwater was encountered during drilling at Boring Nos. B-1 and B-5, at a depths of 34± feet and 48½± feet, respectively. Boring No. B-7 was drilled to a depth of 50± feet and did not encounter free water during drilling. Based on the water level measurements performed during drilling and the moisture contents of the recovered soil samples, the static groundwater table varies throughout the site and is considered to have existed at depths between 34± feet and greater than 50 feet below existing site grades at the time of the subsurface exploration.

Research of historic high groundwater levels was performed as a part of the site-specific liquefaction evaluation. USGS Bulletin 1898 (Matti and Carson, 1991) indicates that the minimum historic depth to groundwater at the site is 20± feet. Additional research of available well data was performed using the *Cooperative Well Measurement Report,* published in Spring 2015 by the Western-Municipal Water District. The data included in this report includes well measurements taken between between January 3, 1994 and May 22, 2014. The historic high groundwater depth reported during this period was at an elevation of about 16.2 feet below the ground surface. For the purpose of the site-specific liquefaction evaluation, the historic high groundwater table is considered to be 16 feet below the ground surface.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. Field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-8 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

Representative bulk samples have been tested for their maximum dry densities and optimum moisture contents. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plates C-9 and C-10 in Appendix C of this report. This test is generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample.

The sample is initially remolded to 50± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

Sample Identification	**Expansion Index**	**Expansive Potential**
B-1 @ 0 to 5 feet	4	Very Low
B-1 @ 0 to 5 feet	18	Very Low

Grain Size Analysis

Limited grain size analyses have been performed on several selected samples, in accordance with ASTM D-1140. These samples were washed over a #200 sieve to determine the percentage of fine-grained material in each sample, which is defined as the material which passes the #200 sieve. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these laboratory tests are shown on the attached boring logs.

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on selected samples of various soil strata encountered at the site. This test is used to determine the Liquid Limit and Plastic Limit of the soil. The Plasticity Index is the difference between the two limits. Plasticity Index is a general indicator of the expansive potential of the soil, with higher numbers indicating higher expansive potential. Soils with a PI greater than 25 are considered to have a high plasticity, and a high expansion potential. Soils with a PI greater 18 are not considered to susceptible to liquefaction. The results of the Atterberg Limits testing are presented on the boring logs.

Soluble Sulfates

Representative samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	**Soluble Sulfates (%)**	**ACI Classification**
B-1 @ 0 to 5 feet	0.015	Not Applicable (S0)
B-4 @ 0 to 5 feet	0.001	Not Applicable (S0)

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations.

The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The recommendations are provided with the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to verify compliance with these recommendations. Maintaining Southern California Geotechnical, Inc., (SCG) as the geotechnical consultant from the beginning to the end of the project will provide continuity of services. The geotechnical engineering firm providing testing and observation services shall assume the responsibility of Geotechnical Engineer of Record.

The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structure should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

The 2016 California Building Code (CBC) was adopted by all municipalities within Southern California on January 1, 2017. The CBC provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

The 2016 CBC Seismic Design Parameters have been generated using U.S. Seismic Design Maps, a web-based software application developed by the United States Geological Survey. This software application, available at the USGS web site, calculates seismic design parameters in accordance with the 2016 CBC, utilizing a database of deterministic site accelerations at 0.01 degree intervals. The table below is a compilation of the data provided by the USGS application. A copy of the output generated from this program is included as Plate E-1 in Appendix E of this report. A copy of the Design Response Spectrum, as generated by the USGS application is also included in Appendix E. Based on this output, the following parameters may be utilized for the subject site:

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.864
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.824
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.864
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.237
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.242
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.824

*The 2016 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site coefficients are to be determined in accordance with Section 11.4.7 of ASCE 7-10. However, Section 20.3.1 of ASCE 7-10 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structure is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structure has a fundamental period greater than 0.5 seconds, a site specific seismic hazards analysis would be required and additional subsurface exploration would be necessary.

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2016 CBC. The peak ground acceleration (PGA_M) was determined in accordance with Section 11.8.3 of ASCE 7-10. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-10. The web-based software application SEAOC/OSHPD Seismic Design Maps Tool (described in the previous section) was used to determine PGA_M, based on ASCE 7-10 as the building code reference document. A portion of the program output is included as Plate E-1 in Appendix E of this report. As indicated on Plate E-1, the PGA_M for this site is 0.727g. An associated earthquake magnitude was obtained from the USGS Unified Hazard Tool, Interactive Deaggregation application available on the USGS website. The deaggregated modal magnitude is 8.1, based on the peak ground acceleration and Site Class D soils.

Liquefaction

Review of the San Bernardino County General Plan indicates that the subject site is not located within a zone of high liquefaction susceptibility. However, the site is located in close proximity to the Santa Ana River and data from a nearby well indicates historic high groundwater levels at elevations of approximately 16± feet below the ground surface. Therefore, the scope of this geotechnical investigation was expanded to include a site-specific liquefaction evaluation.

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is determined as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable. Additionally, in accordance with Special Publication 117A, clayey soils which do not meet the criteria for liquefiable soils defined by Bray and Sancio (2006), loose soils with a plasticity index (PI) less than 12 and moisture content greater than 85% of the liquid limit, are considered to be insusceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable.

As part of the liquefaction evaluation, Boring Nos. B-1, B-5, and B-7 were extended to depths of 50± feet. The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction analysis was performed for using data from the three 50±-foot-deep borings. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.727g for a magnitude 8.1 seismic event.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined using the equation for volumetric strain due to post-cyclic reconsolidation (Yoshimine et. al, 2006). This procedure uses an empirical relationship between the induced cyclic shear strain and the corrected N-value to determine the expected volumetric strain of saturated sands subjected to earthquake shaking. This analysis is also documented on the spreadsheets included in Appendix F.

Conclusions and Recommendations

The results of the liquefaction analysis have identified potentially liquefiable soils at the site. The potentially liquefiable soils were identified at Boring Nos. B-5 and B-7 between the depths of 16 and 27± feet. Soils that are located above the historic groundwater table (16± feet), or possess factors of safety in excess of 1.3 are considered non-liquefiable. Additionally, some of the soils encountered between depths of 27 and 37± feet at Boring No. B-5 are also considered non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the criteria of Bray and Sancio (2006). Settlement analyses were performed for each of the potentially liquefiable strata.

Based on the settlement analysis (also tabulated on the spreadsheets in Appendix F) total dynamic (liquefaction induced) settlements of 0.5± to 2.6± inches could be expected at Boring Nos. B-5 and B-7, respectively. No liquefiable soils were encountered at Boring No. B-1. Associated differential settlements are estimated to on the order of 2± inches. The estimated differential settlement could be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of approximately 0.0017 inches per inch. These settlements are considered to be within the structural tolerances of a typical building supported on a shallow foundation system.

Based on our understanding of the proposed development, it is considered feasible to support the proposed structures on shallow foundations. Shallow foundation systems can be designed to resist the effects of the anticipated differential settlements, to the extent that the structures would not catastrophically fail. Designing the proposed structures to remain completely undamaged during a seismic event that could occur once every 2475 years (the code-specified return period used in the liquefaction analysis) is not considered to be economically feasible. Based on this understanding, the use of shallow foundation systems is considered to be the most economical means of supporting the proposed structures.

In order to support the proposed structures on shallow foundations (such as spread footings) the structural engineer should verify that the structures would not catastrophically fail due to the predicted dynamic differential settlements. Any utility connections to the structures should be designed to withstand the estimated differential settlements. It should also be noted that minor to moderate repairs, including re-leveling, restoration of utility connections, repair of damaged drywall and stucco, etc., would likely be required after occurrence of the liquefaction-induced settlements.

The use of a shallow foundation system, as described in this report, is typical for buildings of this type, where they are underlain by the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the buildings proposed for this site will also be typical of similar buildings in the vicinity of this project. However, if the owner determines that this level of potential damage is not acceptable, other geotechnical and structural options are available, including the use of ground improvement, deep foundations, or a mat foundation.

6.2 Geotechnical Design Considerations

General

The near surface soils possess variable strengths and densities. Disturbed native alluvium was encountered at the ground surface at several of the boring locations, extending to depths of 2½ to 4½± feet. Artificial fill soils were encountered in one of the borings for this investigation, extending to a depth of 7½± feet and in three of the borings performed by CHJ in the existing detention pond area for the previous study (referenced in Section 3.3 of this report). The fill soils extended to depths of 3 to 12± feet at the CHJ boring locations. The near-surface native alluvium within the upper 6½ to 10± feet at the boring locations possesses variable strengths and loose to medium dense relative densities. The existing near-surface soils, in their present condition, are not considered suitable to support the foundation loads of the new structure. Based on these considerations, remedial grading is considered warranted to remove the disturbed alluvium, the artificial fill soils, and a portion of the variable strength alluvium from the proposed building areas and to replace these materials as compacted, structural fill soils.

Based on the current project information, we expect that fills of 2 to 22± feet will be necessary in order to achieve the proposed site grades. The greatest depths of fill will be necessary in the existing basin areas, which contain soils classified as in the previous study as pond sediment and artificial fill, extending to depths of 8 to 12± feet. **Greater depths of remedial grading are necessary in the existing basin areas in order to remove undocumented fill materials.**

Based on discussions with the client, the client will purchase about 564,000 cubic yards of potential fill materials from the seller of the property, Cal Portland, in accordance with the terms of the sale of the property. Based on discussions with representatives of CalPortland and the client, we understand that the potential import materials consist of soils and crushed rock materials mined from the CalPortland facility, located north of the subject site, on the north side of Agua Mansa Road. We observed the soil stockpile during a visit to the export site in October 2018. The stockpiled soil for potential import to the subject site appeared to be predominately granular in composition, possessing a well-graded grain size distribution.

As discussed in the previous section of this report, potentially liquefiable soils were identified at this site. The presence of the recommended layer of newly placed compacted structural fill above these liquefiable soils will help to reduce any surface manifestations that could occur as a result of liquefaction. The foundation design recommendations presented in the subsequent sections of this report also contain recommendations to provide additional rigidity in order to reduce the potential effects of differential settlement that could occur as a result of liquefaction.

Settlement

The recommended remedial grading will remove the undocumented fill soils, the disturbed soils, and a portion of the loose alluvial soils from the proposed building areas and replace these materials as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation generally possess favorable consolidation characteristics and will not be subject to significant load increases from the foundations of the new structures.

Provided that the recommended remedial grading is completed, the post-construction static settlements of the proposed structures are expected to be within tolerable limits.

Expansion

Laboratory testing performed on representative samples of the near surface soil indicates that these materials are very low expansive (EI = 4 and 18). Therefore, no design considerations related to expansive soils are considered warranted for this site. Additionally, we understand that the subject site will be covered with imported fill materials consisting of crushed rock and soil to thicknesses of 2 to 22± feet. It is recommended that the imported fill materials possess a very low expansion index (EI <20).

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils to correspond to Class S0 with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building areas.

Shrinkage/Subsidence

Removal and recompaction of the existing fill soils and near-surface alluvium is estimated to result in an average shrinkage of 12 to 16 percent. It should be noted that this potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1 feet.

These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be dependent on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

Grading and foundation plans for the currently proposed development were not available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

Initial site stripping should include removal of any surficial vegetation. This should include any weeds, grasses, shrubs, and trees. Root masses associated with the trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered. Any trash should also be cleared from the site prior to site grading.

Existing structures on the subject site consist of wooden telephone poles and metal towers supporting power lines. Demolition of some or all of the towers may be necessary to facilitate the proposed development. The foundation systems for these towers are unknown to SCG. If the demolished structures are supported by deep foundations, the existing foundations should be cut off at an elevation of at least 2 feet below the planned depth of overexcavation, prior to the placement of any fill soils. Shallow foundations should be demolished in their entirety.

Treatment of Existing Soils: Building Pads

Remedial grading should be performed within the proposed building areas in order to remove the undocumented fill soils, the disturbed alluvium, and the upper portion of the loose native alluvial soils. Based on conditions encountered at the boring locations, overexcavations of 2½ to 4½± feet are expected to be necessary to remove the disturbed alluvium. Greater removals will be required in the areas of the existing detention ponds where artificial fill soils were encountered between depths of 3 and 12± feet for this investigation and the referenced previous study. At a minimum, the existing soils within the proposed building area are recommended to be overexcavated to a depth of at least 3 feet below the below existing grade. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building perimeter and foundations, and to an extent equal to the depth of fill below the new foundations. Therefore, the lateral limit of overexcavation will extend to a greater distance than 5 feet in areas where fills will be placed at greater depths. If the proposed structures incorporate any exterior columns (such as for a canopy or overhang) the overexcavation should also encompass these areas.

Based on conditions encountered at the exploratory boring locations, some zones of moist to very moist soils will be encountered at or near the base of the recommended overexcavation. Stabilization of the exposed overexcavation subgrade soils is expected to be necessary. Scarification and air drying of these materials may be sufficient to obtain a stable subgrade. However, if highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying, mechanical stabilization, usually consisting of coarse crushed stone or geotextile, could be necessary. In this event, the geotechnical engineer should be contacted for supplementary recommendations.

Following completion of the overexcavation, the subgrade soils within the building areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structures. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, or low density native soils are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, moisture treated to 0 to 4 percent above the optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed site walls and retaining walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pads. Within the retaining wall areas, the depth of overexcavation should also be sufficient to remove any undocumented fill or disturbed soils. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. Erection pad areas for tilt-up walls are considered to be part of the foundation system and should also be overexcavated. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Parking and Drive Areas

In general, overexcavation of the existing soils in the new parking areas is not considered warranted, with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading, based on economic considerations. However, portions of the proposed parking and drive areas will be constructed above the existing detention pond areas. The existing detention pond areas are presently underlain by undocumented fill soils. Additionally, these areas are expected to receive new fill soils with depths of 12 to 22± feet. Based on the presence of undocumented fill soils and the influence of deep proposed fills, excessive settlements and resultant cracking of new pavements may occur in the areas of the existing detention ponds. Therefore, some overexcavation is considered warranted in the proposed pavement areas located above the existing detention pond areas.

Generally, subgrade preparation in the new parking areas should initially consist of removal of all soils disturbed during stripping operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of undocumented fill soils and disturbed or loose alluvial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not completely mitigate the extent of loose alluvium in the parking areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill. **In the existing detention pond areas, we recommend overexcavating the existing soils in the parking and drive areas to a depth of at least 3± feet below existing site grades and to a depth sufficient to remove the undocumented fill soils in the manner recommended for the proposed building pad areas.**

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2016 CBC and the grading code of the city of Colton.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. **Fill soils placed at depths of 15 feet or more below the proposed site elevations should be compacted to 95 percent of the ASTM D-1557 maximum dry density.** Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of very low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by city of Colton. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

6.4 Construction Considerations

Excavation Considerations

The near surface soils generally consist of silty sands, sandy silts, well-graded sands, and occasional silty clays. These materials will likely be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

Some of the near surface soils possess appreciable silt and occasional clay content and may become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. In addition, based on their granular content, some of the on-site soils will also be susceptible to erosion. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

As discussed in Section 6.3 of this report, unstable subgrade soils may be encountered at the base of the overexcavation within the proposed building areas. The extent of subgrade stability problems on this site will largely depend on the time of year during which grading occurs. It is therefore recommended that grading be performed during a period or warm, dry weather, if possible. If construction is to occur during cool, wet weather, the development budget should include costs associated with stabilizing a pumping or yielding subgrade, as well as costs associated with imported a drier, less moisture sensitive structural fill material.

Groundwater

At the time of subsurface exploration, the static groundwater table at this site was considered to exist at depths of approximately 34± feet to greater than 50± feet. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pads will be underlain by imported structural fill soils underlain by a layer of overexcavated and recompacted fill and alluvial soils. Based on the proposed grading, we expect that the new structural fill soils are expected to extend to depths of at least 3 feet below proposed foundation bearing grade. The proposed import soils that will be acquired as a part of the sale of the property possess a predominately granular and well-graded composition. Based on this subsurface profile, the proposed structure may be supported on shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom) in strip footings, due to the presence of potentially liquefiable soils.
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements, as discussed in Section 6.1. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 0 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report. However, the likelihood of these two settlements combining is considered remote. The static settlements are expected to occur in a relatively short period of time after the building loads being applied to the foundations, during and immediately subsequent to construction. It should be noted that the projected potential dynamic settlement is related to a major seismic event and a conservative historic high groundwater level.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 3,000 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floors of the new structures may be constructed as a conventional slabs-on-grade supported on newly placed structural fills. We expect that the imported fill soils will consist of predominately granular well-graded crushed rock and soil materials. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: k = 150 psi/in.

- Minimum slab reinforcement: No. 3 bars at 16 inches on-center, in both directions, due to the liquefaction potential of the encountered soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab where such moisture sensitive floor coverings are expected. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

- Moisture condition the floor slab subgrade soils to 0 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement. Additional rigidity may be necessary for structural considerations.

6.7 Retaining Wall Design and Construction

Although not indicated on the site plan, the proposed development may require some small retaining walls (less than 3 to 5± feet in height) to facilitate the new site grades and the in dock-high areas of the building.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use imported, well-graded, predominately granular, soil and crushed rock materials. Based on the granular composition and gradation, these materials are expected to possess a friction angle of at least 30 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		**Soil Type** Imported Well-Graded Soil and Crushed Rock
Internal Friction Angle (ϕ)		30°
Unit Weight		120 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	40 lbs/ft^3
	Active Condition (2h:1v backfill)	64 lbs/ft^3
	At-Rest Condition (level backfill)	60 lbs/ft^3

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In accordance with the 2016 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 3 feet below the proposed bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site soils and imported fill soils may be used to backfill the retaining walls. **However, all backfill material placed within 3 feet of the back-wall face should have a particle size no greater than 3 inches.** The retaining wall backfill materials should be well graded.

It is recommended that a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls be used. If the drainage composite material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The drainage composite should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a one cubic foot gravel pocket surrounded by a suitable geotextile at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

6.8 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of imported structural fill, consisting of predominately granular, well graded crushed rock materials and soils with maximum grain sizes of 6 inches. Based on this gradation and composition, the imported fill soils are expected to possess good pavement support characteristics with estimated R-values of 40 to 60. The subsequent pavement design is based on a conservative R-value of 40. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20 year design life, assuming six operational traffic days per week.

Traffic Index	No. of Heavy Trucks per Day
4.0	0
5.0	1
6.0	3
7.0	11
8.0	35

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

ASPHALT PAVEMENTS (R = 40)					
Materials	**Thickness (inches)**				
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	4	6	7	8
Compacted Subgrade (90% minimum compaction)	12	12	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI =6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5½	5½	6½
Compacted Subgrade (95% minimum compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. Any reinforcement within the PCC pavements should be determined by the project structural engineer. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness.

7.0 GENERAL COMMENTS

This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

8.0 REFERENCES

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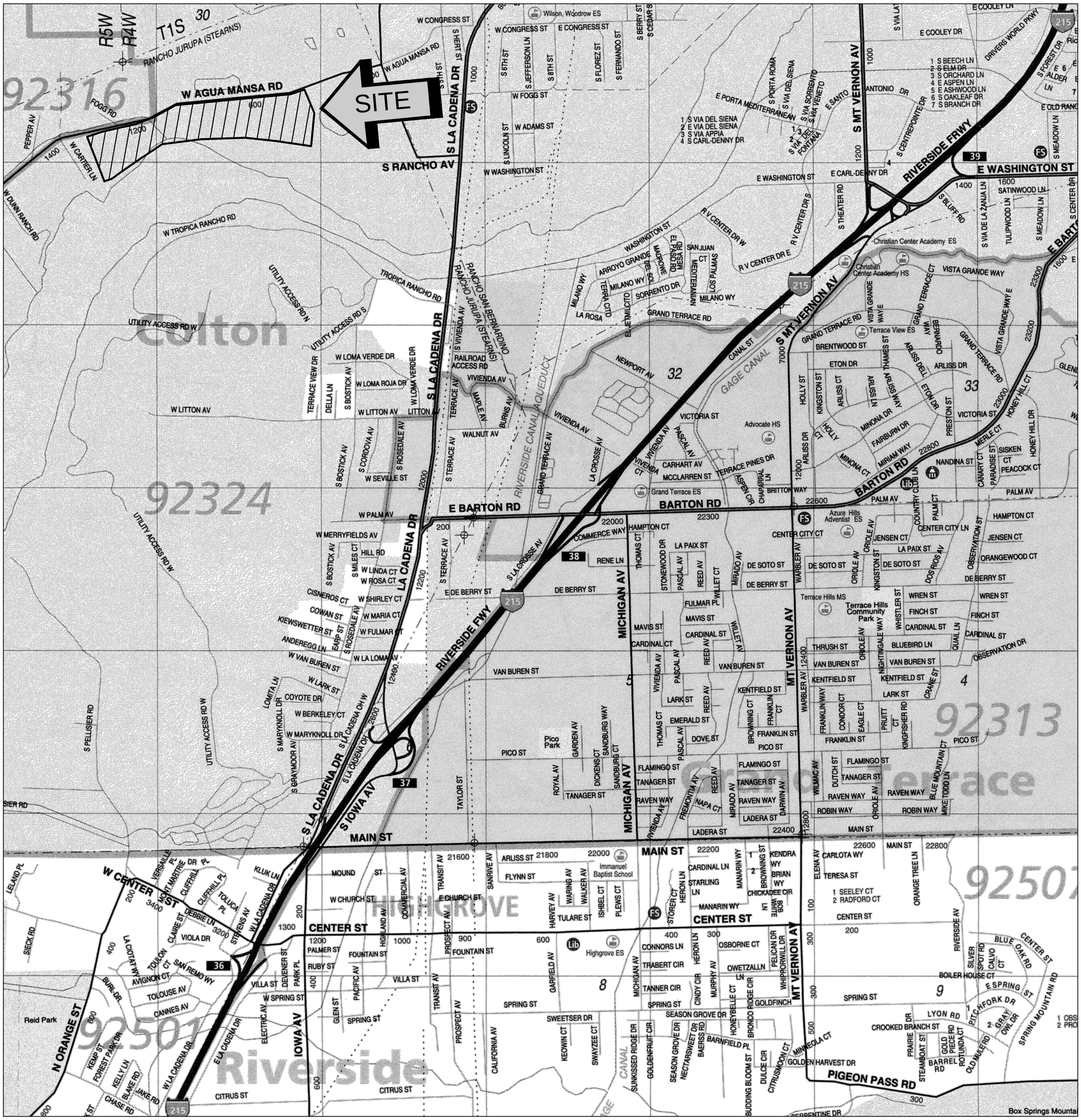
Tokimatsu K., and Seed, H. B., "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of the Geotechnical Engineering Division, American society of Civil Engineers, Volume 113, No. 8, August 1987, pp. 861-878.

Tokimatsu, K. and Yoshimi, Y., "*Empirical Correlations of Soil Liquefaction Based on SPT N-value and Fines Content,"* Seismological Research Letters, Eastern Section Seismological Society Of America, Volume 63, Number 1, p. 73.

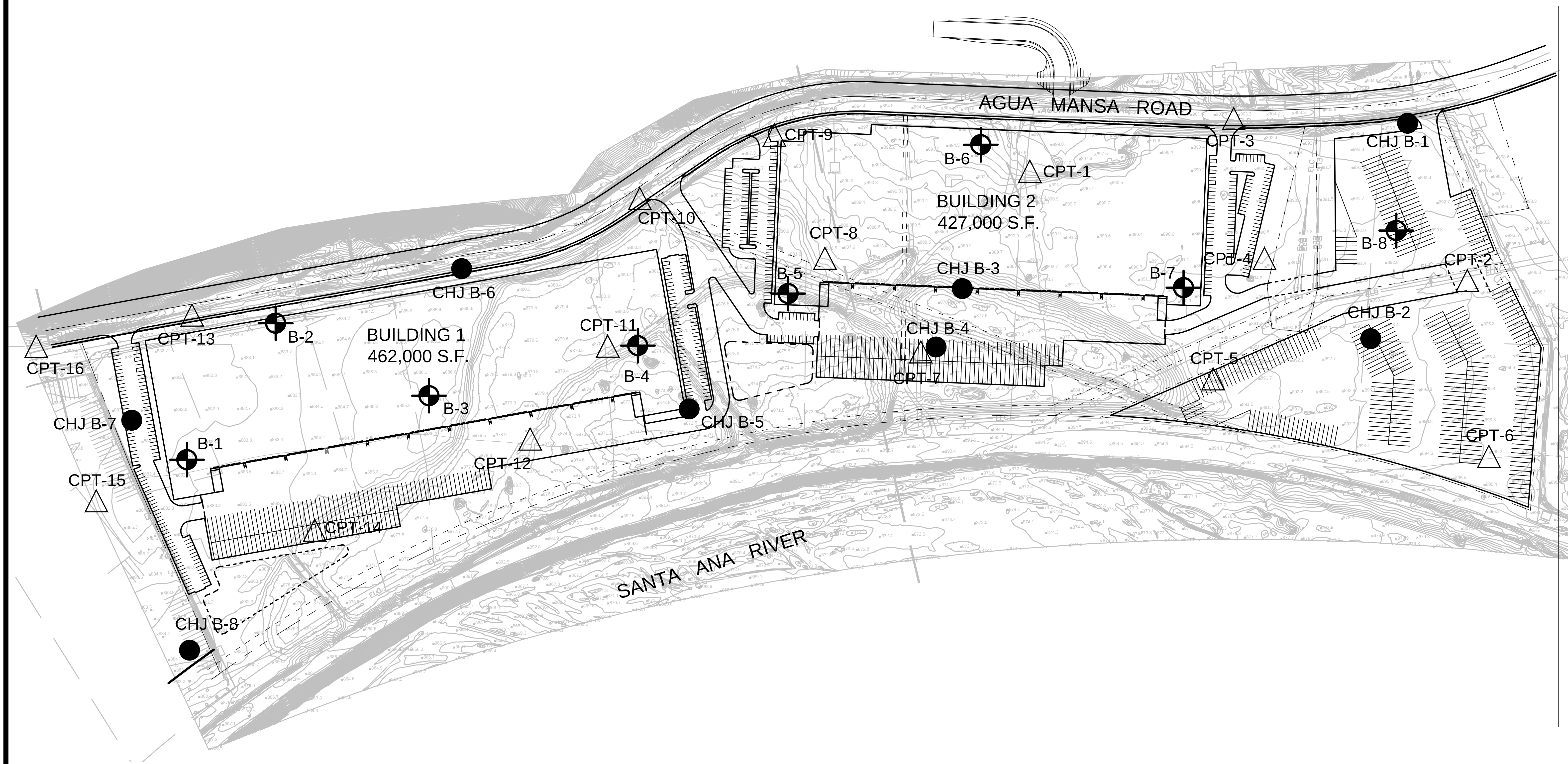
Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

APPENDIX A



SOURCE: SAN BERNARDINO COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP	
PROPOSED INDUSTRIAL DEVELOPMENT	
COLTON, CALIFORNIA	
SCALE: 1" = 2400'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: AL CHKD: GKM	
SCG PROJECT 18G212-2	
PLATE 1	



GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- PREVIOUS BORING LOCATION (CHJ PROJECT NO. 16113-3).
- PREVIOUS CPT LOCATION (CHJ PROJECT NO. 16113-3).

NORTH SoCalGeo

NOTE: BASE SITE PLAN PREPARED BY RGA..

BORING LOCATION PLAN

PROPOSED INDUSTRIAL DEVELOPMENT

COLTON, CALIFORNIA

SCALE: 1" = 240'

DRAWN: AL
CHKD: RGT

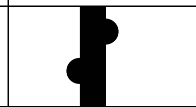
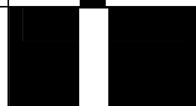
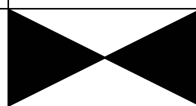
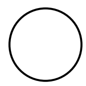
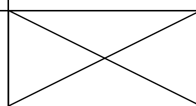
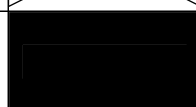
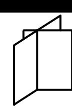
SCG PROJECT
18G212-2

PLATE 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

SAMPLE TYPE	GRAPHICAL SYMBOL	SAMPLE DESCRIPTION
AUGER		SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED)
CORE		ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK.
GRAB		SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED)
CS		CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED)
NSR		NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL.
SPT		STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED)
SH		SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED)
VANE		VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED.

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

BLOW COUNT: Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

POCKET PEN.: Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

GRAPHIC LOG: Graphic Soil Symbol as depicted on the following page.

DRY DENSITY: Dry density of an undisturbed or relatively undisturbed sample in lbs/ft^3.

MOISTURE CONTENT: Moisture content of a soil sample, expressed as a percentage of the dry weight.

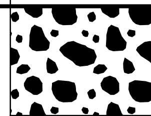
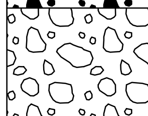
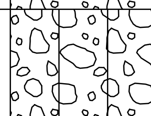
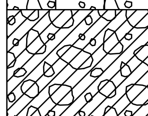
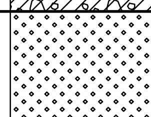
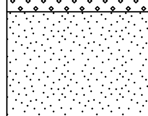
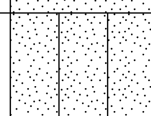
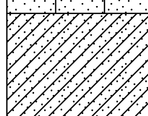
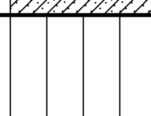
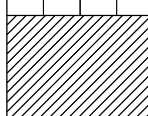
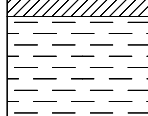
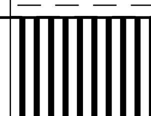
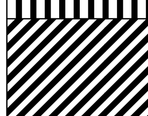
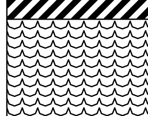
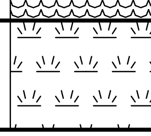
LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT: The moisture content above which a soil behaves as a plastic.

PASSING #200 SIEVE: The percentage of the sample finer than the #200 standard sieve.

UNCONFINED SHEAR: The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING NO. B-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 18G212-2	DRILLING DATE: 2/11/19	WATER DEPTH: 34 feet
PROJECT: Proposed Industrial Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Colton, California	LOGGED BY: Anthony Luna	READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 883 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	6			DISTURBED ALLUVIUM: Brown to Gray Brown Silty fine Sand to fine Sandy Silt, trace Iron oxide staining, loose-very moist		23					EI = 4 @ 0 to 5'
	X	10			ALLUVIUM: Brown Silty fine Sand, trace Clay, loose to medium dense-very moist		18					
5												
	X	7			Light Gray Brown fine Sandy Silt, trace calcareous nodules, loose-damp to moist		11			80		
	X	12			Light Brown to Light Gray Brown fine to medium Sand, trace coarse Sand, little Silt, medium dense to dense-dry to damp		1			9		
10												
	X	30			@ 13½ to 15, trace fine to coarse Gravel		2					
15												
					Light Gray Brown fine to coarse Sand, some fine Gravel, trace to little Silt, dense to very dense-dry to damp							
	X	48					2					
20												
	X	32					2					
25												
	X	66/11"					2					
30												
	X	70			@ 33½ to 35 feet, very moist to wet		5					

TBL 18G212-2.GPJ SOCALGEO.GDT 3/12/19

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 18G212-2
PROJECT: Proposed Industrial Development
LOCATION: Colton, California

DRILLING DATE: 2/11/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna

WATER DEPTH: 34 feet
CAVE DEPTH: 17 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Gray Brown fine to coarse Sand, some fine Gravel, trace to little Silt, dense to very dense-dry to damp							
					Light Brown fine to coarse Sand, trace fine Gravel, medium dense to dense-wet							
	X	30					12					
40												
	X	35					10					
45												
					Brown fine Sandy Silt, trace medium Sand, dense to very dense-wet							
	X	50					26					
50					Boring Terminated at 50'							

TBL 18G212-2.GPJ SOCALGEO.GDT 3/12/19

TEST BORING LOG

BORING NO. B-2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 18G212-2
PROJECT: Proposed Industrial Development
LOCATION: Colton, California

DRILLING DATE: 2/11/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna

WATER DEPTH: Dry
CAVE DEPTH: 11 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION — SURFACE ELEVATION: 883.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
		FIELD RESULTS				LABORATORY RESULTS						
		13			ALLUVIUM: Dark Gray Brown fine Sandy Silt, trace fine root fibers, trace Iron oxide staining, loose-very moist	86	25					
		11			Light Gray fine Sandy Silt, little Iron oxide staining, loose-very moist	70	27					
5		18			Light Gray Silty fine Sand to fine Sandy Silt, trace Iron oxide staining, medium dense-very moist	75	17					
		16			Light Gray fine to medium Sand, trace to little Silt, medium dense-dry to damp	101	2					
10		21			Light Gray fine Sand, little medium Sand, medium dense-dry	94	1					
					Light Gray Brown fine to coarse Sand, trace Silt, trace fine to coarse Gravel, medium dense to very dense-dry to damp							
15		50/5"										No Sample Recovered
20		29					3					
25		52			@ 23½ to 25 feet, some fine Gravel, very dense		2					
30		30					3					
					Boring Terminated at 30'							

BORING NO. B-3

JOB NO.: 18G212-2 | DRILLING DATE: 2/12/19 | WATER DEPTH: Dry
PROJECT: Proposed Industrial Development | DRILLING METHOD: Hollow Stem Auger | CAVE DEPTH: 14 feet
LOCATION: Colton, California | LOGGED BY: Anthony Luna | READING TAKEN: At Completion

SURFACE ELEVATION: 886 feet MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	6			DISTURBED ALLUVIUM: Light Gray Brown fine Sand, trace medium Sand, loose-damp		6					
	X	7			ALLUVIUM: Light Gray Brown fine to medium Sand, loose-damp		7					
5												
	X	16			Light Gray Brown fine to medium Sand, trace coarse Sand, trace Silt, medium dense-moist		8					
	X	38			Gray Brown fine to coarse Sand, little to some fine to coarse Gravel, trace Silt, medium dense to dense-damp		5					
10												
	X	16					3					
15												
	X	20					4					
20												

Boring Terminated at 20'

TEST BORING LOG **PLATE B-3**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 18G212-2	DRILLING DATE: 2/12/19	WATER DEPTH: Dry
PROJECT: Proposed Industrial Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 26 feet
LOCATION: Colton, California	LOGGED BY: Anthony Luna	READING TAKEN: At Completion

SURFACE ELEVATION: 878 feet MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Light Gray fine Sand, trace coarse Sand, trace medium Sand, medium dense-damp							
	X	17				101	5					
					Light Brown fine to medium Sand, trace coarse Sand, loose to medium dense-damp to moist							
	X	15				100	8					
5	X	12				106	3					
					Gray Brown Silty fine Sand, loose to medium dense-damp							
	X	21										No Sample Recovered
					Light Brown fine to coarse Sand, some fine Gravel, medium dense-dry to damp							
	X	17				113	2					
10												
					Light Gray Brown fine to medum Sand, little coarse Sand, dense-damp							
	X	34					3					
15												
					Red Brown to Brown Silty Clay, stiff to very stiff-very moist							
	X	25					21					
20												
	X	14					18					
25												
					Red Brown to Brown Clayey fine Sand to fine Sandy Clay, dense to hard-very moist							
	X	38					25					
30												
					Boring Terminated at 30'							

TBL 18G212-2.GPJ SOCALGEO.GDT 3/12/19

BORING NO. B-5

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 18G212-2	DRILLING DATE: 2/11/19	WATER DEPTH: 48.5 feet
PROJECT: Proposed Industrial Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 45 feet
LOCATION: Colton, California	LOGGED BY: Anthony Luna	READING TAKEN: At Completion

SURFACE ELEVATION: 880 feet MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	11		FILL: Light Brown fine Sand, trace medium to coarse Sand, trace fine Gravel, trace to little Silt, loose to medium dense-damp		4					
	X	14		@ 3½ to 7½ feet, trace Plastic fragments		3					
5											
	X	9				4			10		
	X	17		ALLUVIUM: Light Gray Brown fine to medium Sand, trace Silt, trace fine Gravel, medium dense-damp		4			5		
10											
				Light Gray Brown fine to coarse Sand, little to some fine Gravel, medium dense to dense-damp							
	X	32				3					
15											
	X	19				4			8		
20											
				Red Brown to Brown Silty Clay, some Iron oxide staining, very stiff-very moist							
	X	16	2.0			31	56	31	87		
25											
				Red Brown to Brown fine Sandy Clay, very stiff-moist							
	X	27	2.5			12					
30											
				Red Brown to Brown Silty Clay, trace to little fine Sand, stiff-very moist							
	X	14	1.5			22	35	21	79		

TBL 18G212-2.GPJ SOCALGEO.GDT 3/12/19

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 18G212-2
PROJECT: Proposed Industrial Development
LOCATION: Colton, California

DRILLING DATE: 2/11/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna

WATER DEPTH: 48.5 feet
CAVE DEPTH: 45 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Red Brown to Brown Silty Clay, trace to little fine Sand, stiff-very moist							
					Red Brown Clayey fine Sand, dense-moist							
	X	38					14					
40												
					Brown Silty Clay, trace fine Sand, trace Iron oxide staining, very stiff-moist							
	X	29	3.0				19					
45												
					Brown fine Sandy Silt, medium dense to dense-wet							
	X	30					26					
50												
					Boring Terminated at 50'							

TBL 18G212-2.GPJ SOCALGEO.GDT 3/12/19

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 18G212-2
PROJECT: Proposed Industrial Development
LOCATION: Colton, California

DRILLING DATE: 2/12/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 895 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					DISTURBED ALLUVIUM: Dark Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, loose to medium dense-damp							
		9				110	10					
		28				112	11					
5					ALLUVIUM: Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, loose to medium dense-damp @ 5 to 6 feet, slightly porous							
		17				105	7					
		15				114	6					
					Light Brown fine to coarse Sand, trace to little fine Gravel, loosse-dry to damp							
		11				115	3					
10												
		56			@ 13½ to 15 feet, some fine to coarse Gravel, very dense		2					
15												
					Light Gray Brown fine to coarse Sand, trace fine Gravel, dense-dry to damp							
		38					2					
20												
					Boring Terminated at 20'							

TBL 18G212-2.GPJ SOCALGEO.GDT 3/12/19

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 18G212-2
PROJECT: Proposed Industrial Development
LOCATION: Colton, California

DRILLING DATE: 2/11/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna

WATER DEPTH: Dry
CAVE DEPTH: 45 feet
READING TAKEN: At Completion

SURFACE ELEVATION: 891 feet MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Light Gray Clayey Silt, little fine Sand, trace Iron oxide staining, medium stiff to stiff-very moist							
	X	8					31					EI = 15 @ 0 to 5'
					Light Gray fine Sandy Silt, trace calcareous veining, trace Iron oxide staining, loose-damp to moist							
	X	9					14					
5												
	X	8					9			90		
					Light Gray fine to medium Sand, little coarse Sand, trace fine Gravel, medium dense-dry to damp							
	X	16					2			13		
10												
	X	14					2			5		
15												
	X	13			@ 18½ to 22 feet, 3" thick interbedded fine Sandy Silt lense		8			23		
20												
					Light Gray fine Sandy Silt, trace to little Clay, medium dense-very moist							
	X	11					22			52		
25												
					Light Gray fine to coarse Sand, trace fine Gravel, dense-dry to damp							
	X	48					2					
30												
					Gray Brown to Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, dense-very moist							
	X	36					17					

TBL 18G212-2.GPJ SOCALGEO.GDT 3/12/19

BORING NO. B-7

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 18G212-2	DRILLING DATE: 2/11/19	WATER DEPTH: Dry
PROJECT: Proposed Industrial Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 45 feet
LOCATION: Colton, California	LOGGED BY: Anthony Luna	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Gray Brown to Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, dense-very moist							
					Brown Silty Clay, hard-very moist							
	X	33	3.0				20					
40												
					Orange Brown Silty fine Sand to fine Sandy Silt, dense-moist							
	X	35					12					
45												
					Light Brown to Brown fine Sandy Clay, little Silt, very stiff-very moist							
	X	27	3.5				23					
50												
					Boring Terminated at 50'							

TBL 18G212-2.GPJ SOCALGEO.GDT 3/12/19

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

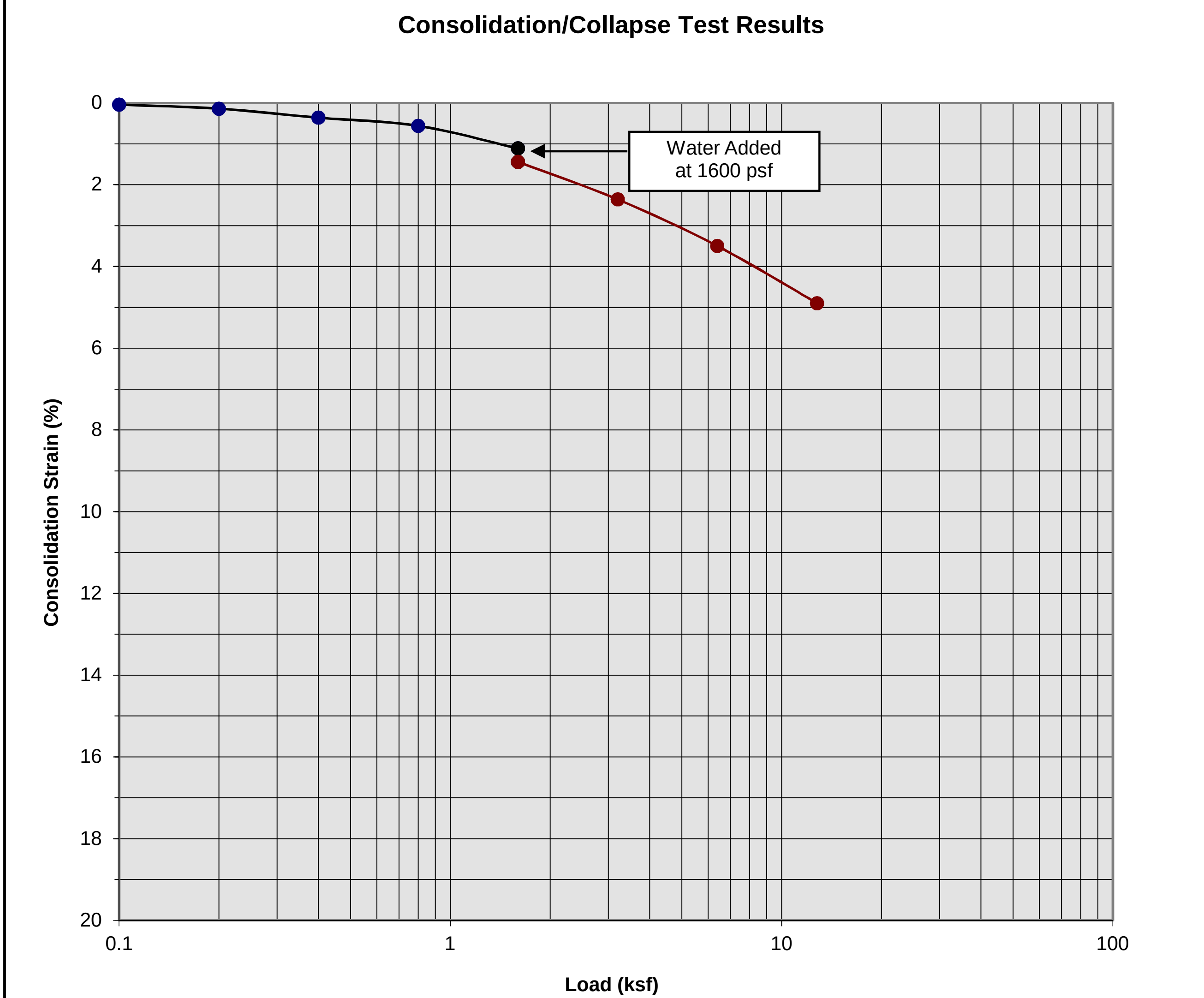
BORING NO. B-8

JOB NO.: 18G212-2 | DRILLING DATE: 2/12/19 | WATER DEPTH: Dry
PROJECT: Proposed Industrial Development | DRILLING METHOD: Hollow Stem Auger | CAVE DEPTH: 20 feet
LOCATION: Colton, California | LOGGED BY: Anthony Luna | READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (SURFACE ELEVATION: 892.5 feet MSL)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					DISTURBED ALLUVIUM: Dark Gray Brown fine to medium Sand, trace to little Silt, medium dense-very moist							
		23				108	14					
					ALLUVIUM: Light Gray fine to medium Sand, trace Iron oxide staining, loose-dry							
		10				105	2					
					Light Gray Brown Silty fine Sand, loose-damp to moist							
5		11				82	12					
		11				88	7					
		14				98	8					
10					Light Gray Brown fine to medium Sand, loose-damp to moist							
					Light Brown fine to coarse Sand, little to some fine Gravel, dense to very dense-dry							
		36					2					
15												
		53					2					
20												
					Brown fine Sandy Silt to Silty fine Sand, trace medium Sand, medium dense-very moist							
		20					20					
25												
					Light Gray Brown fine to medium Sand, trace coarse Sand, medium dense-dry							
		28					2					
30					Boring Terminated at 30'							

TBL 18G212-2.GPJ SOCALGEO.GDT 3/12/19

APPENDIX C



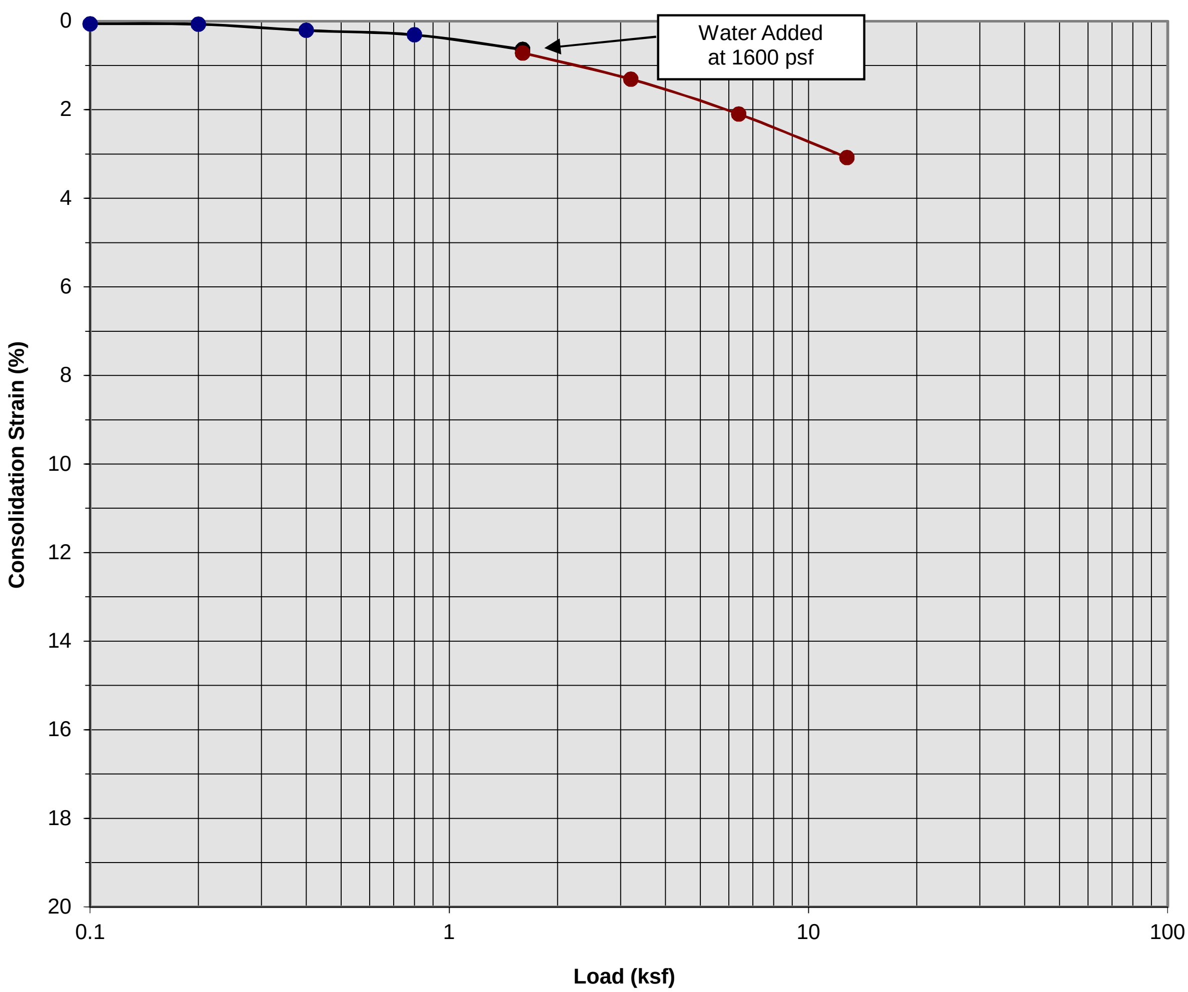
Classification: Light Gray fine Sand, trace coarse Sand, trace medium Sand

Boring Number:	B-4	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	1 to 2	Initial Dry Density (pcf)	101.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	107.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.33

Proposed Industrial Development
Colton, California
Project No. 18G212-2
PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

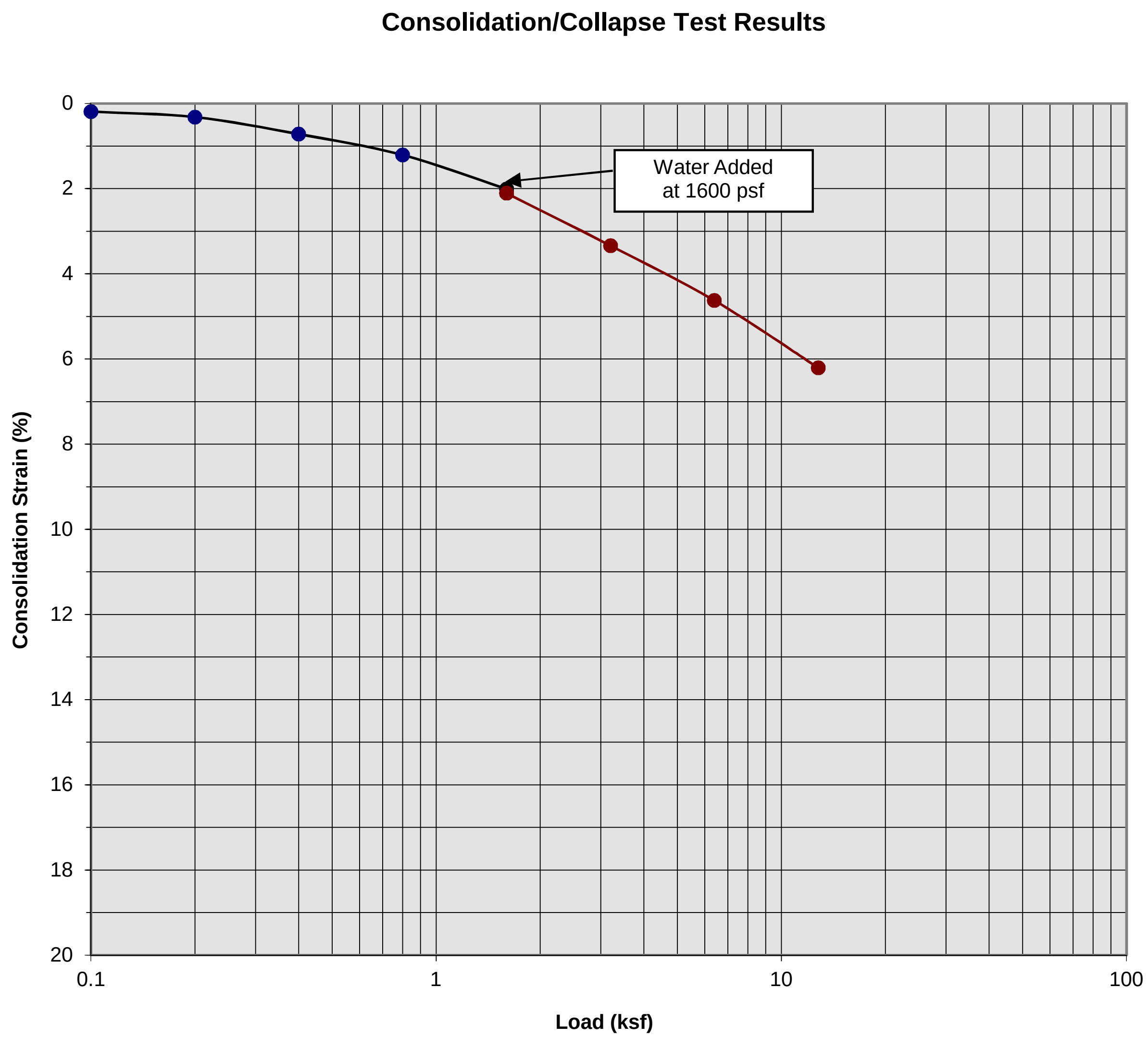


Classification: Light Brown fine to medium Sand, trace coarse Sand

Boring Number:	B-4	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	3 to 4	Initial Dry Density (pcf)	99.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	102.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.08

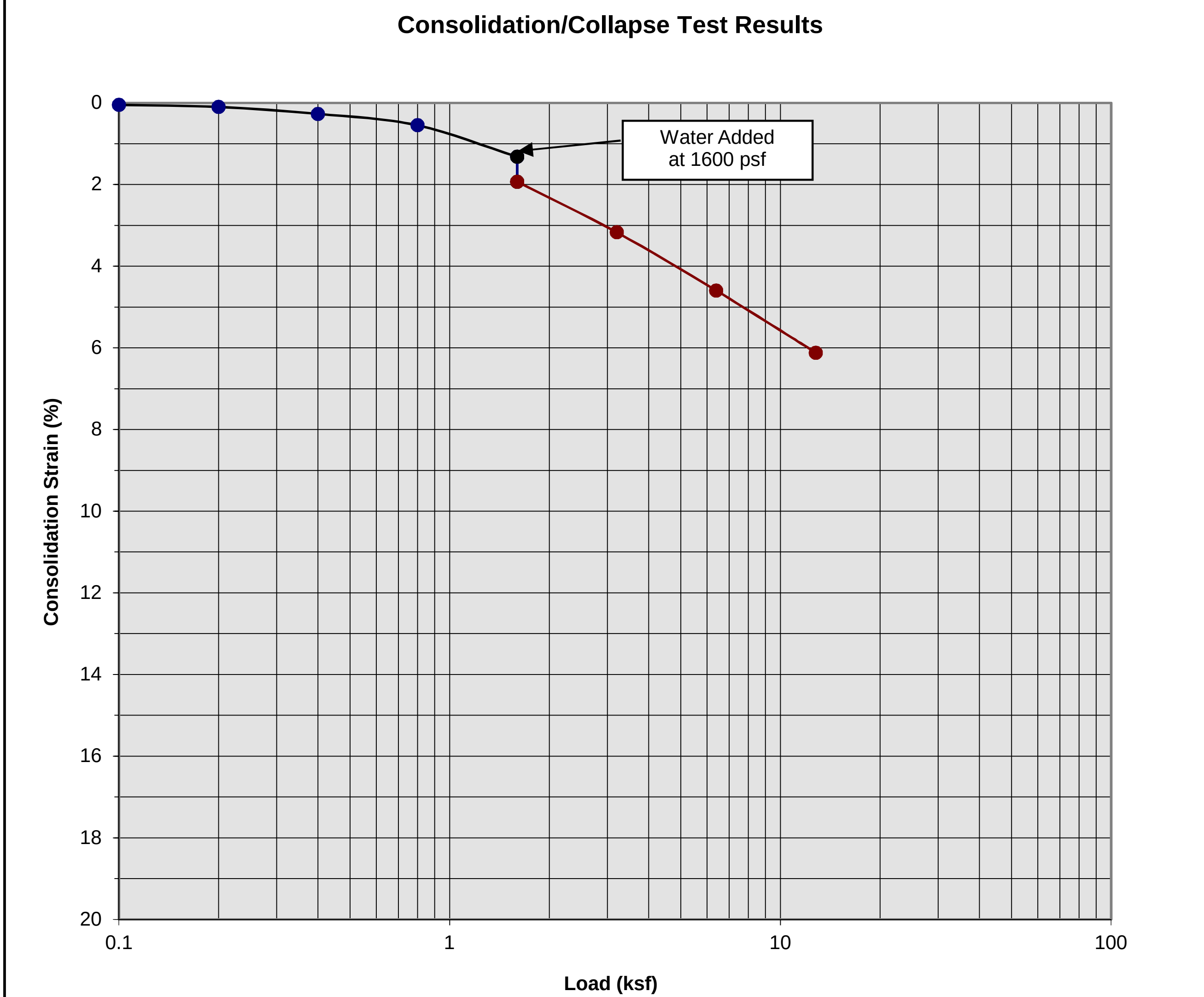
Proposed Industrial Development
Colton, California
Project No. 18G212-2
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Gray Brown Silty fine Sand

Boring Number:	B-4	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	25
Depth (ft)	5 to 6	Initial Dry Density (pcf)	105.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.10

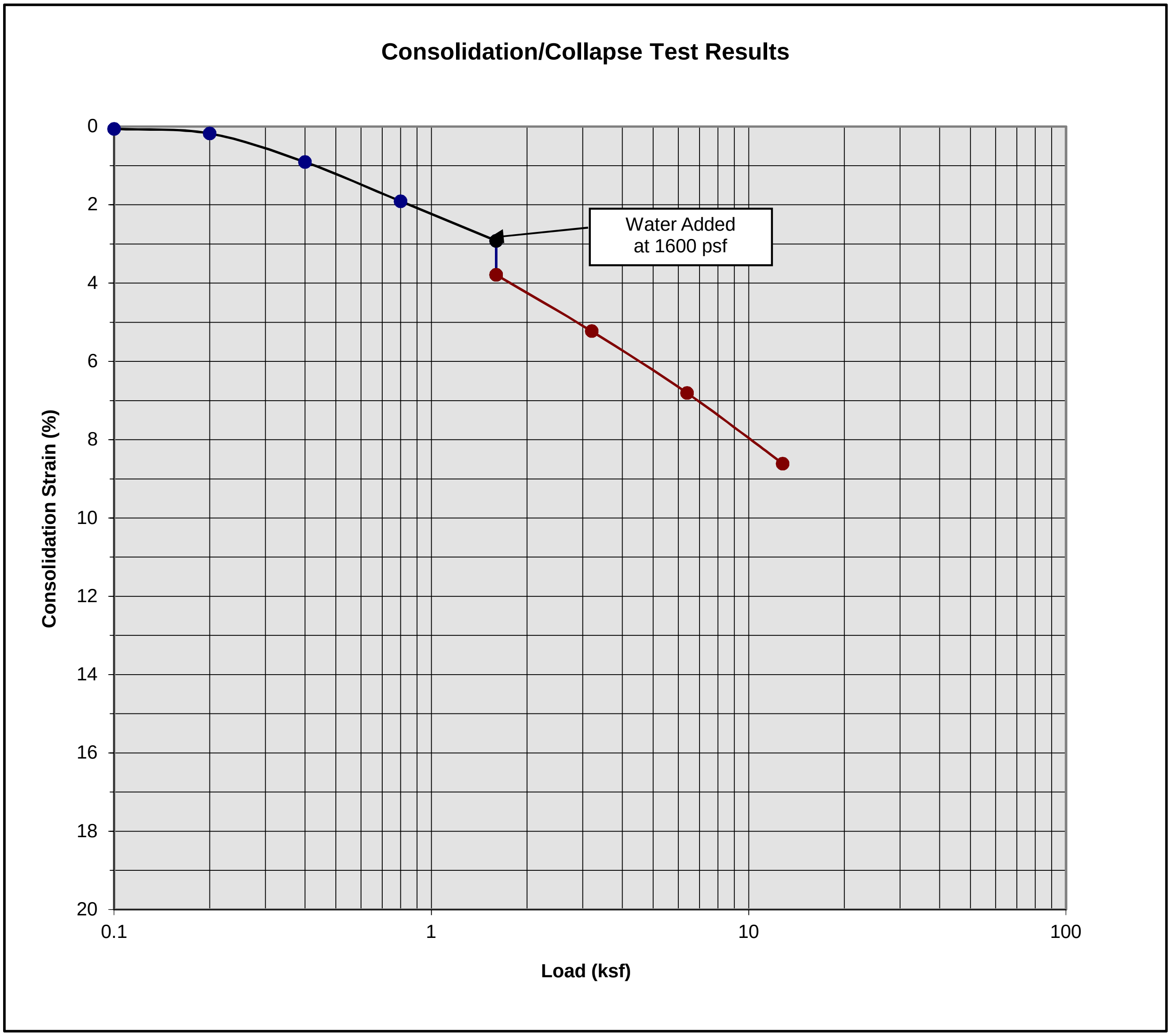


Classification: Light Brown fine to coarse Sand, some fine Gravel

Boring Number:	B-4	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	9 to 10	Initial Dry Density (pcf)	112.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.61

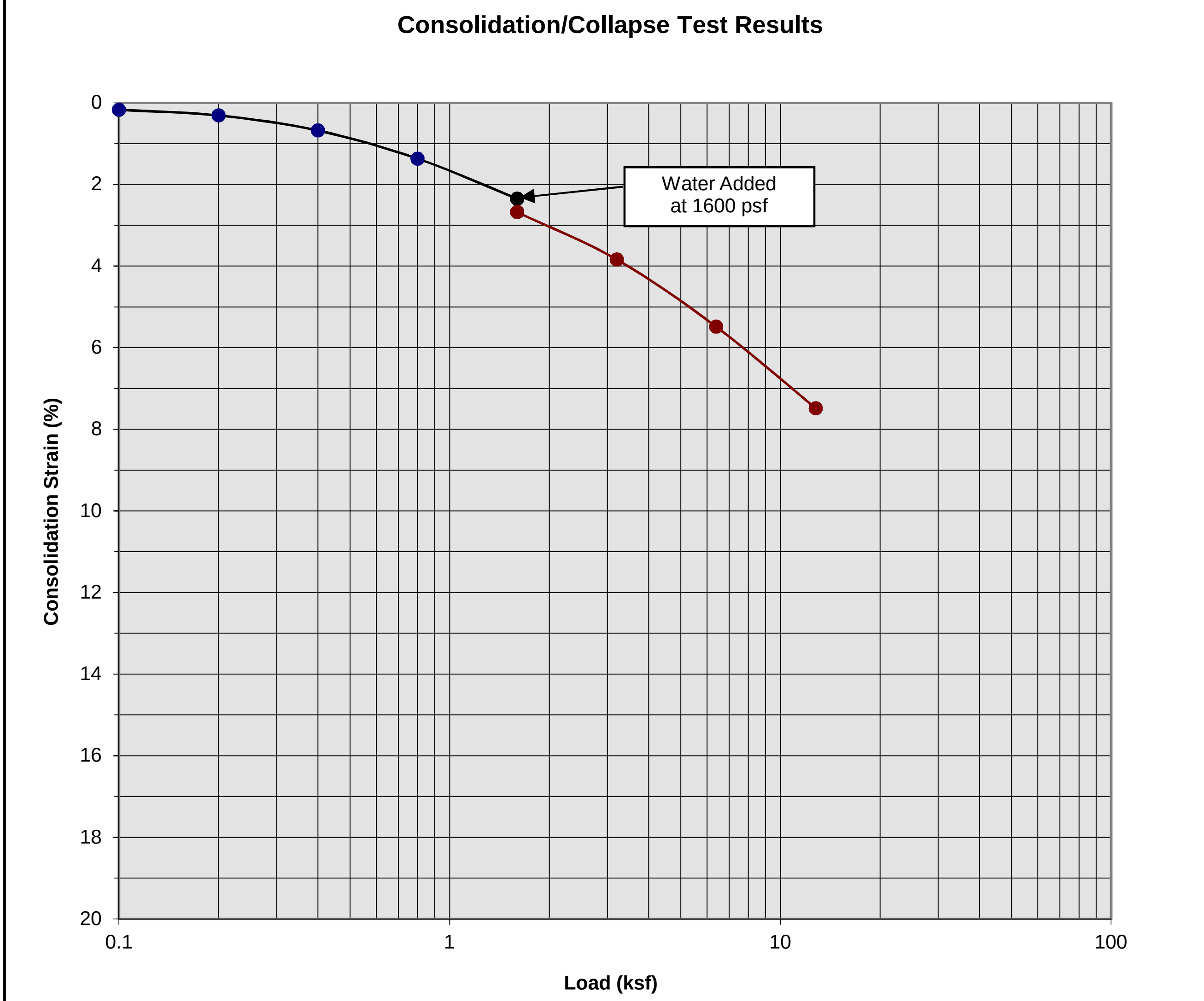
Proposed Industrial Development
Colton, California
Project No. 18G212-2
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Light Gray fine to medium Sand

Boring Number:	B-8	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	3 to 4	Initial Dry Density (pcf)	104.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.87

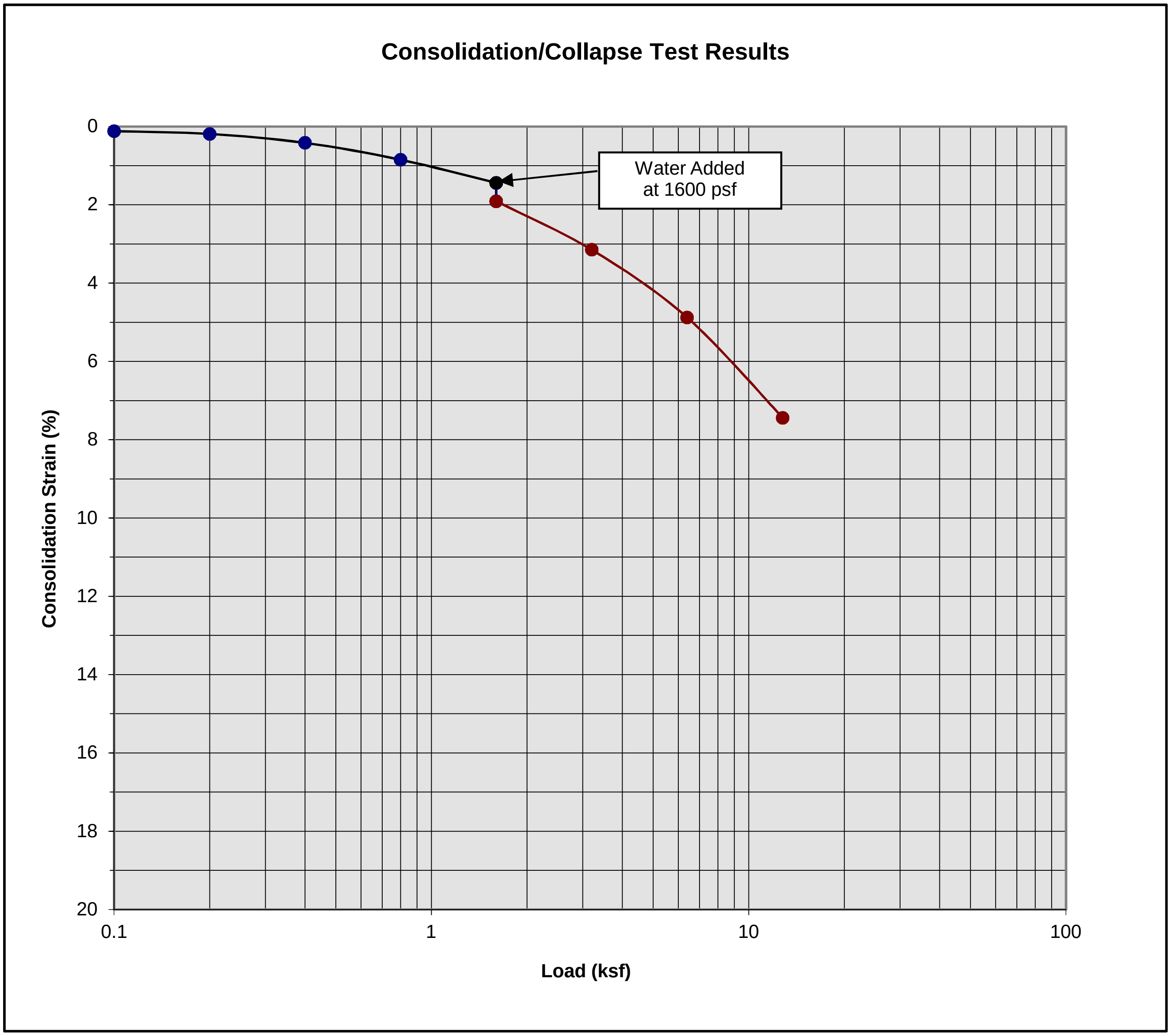


Classification: Light Gray Brown Silty fine Sand

Boring Number:	B-8	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	29
Depth (ft)	5 to 6	Initial Dry Density (pcf)	81.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	88.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.33

Proposed Industrial Development
Colton, California
Project No. 18G212-2
PLATE C- 6

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



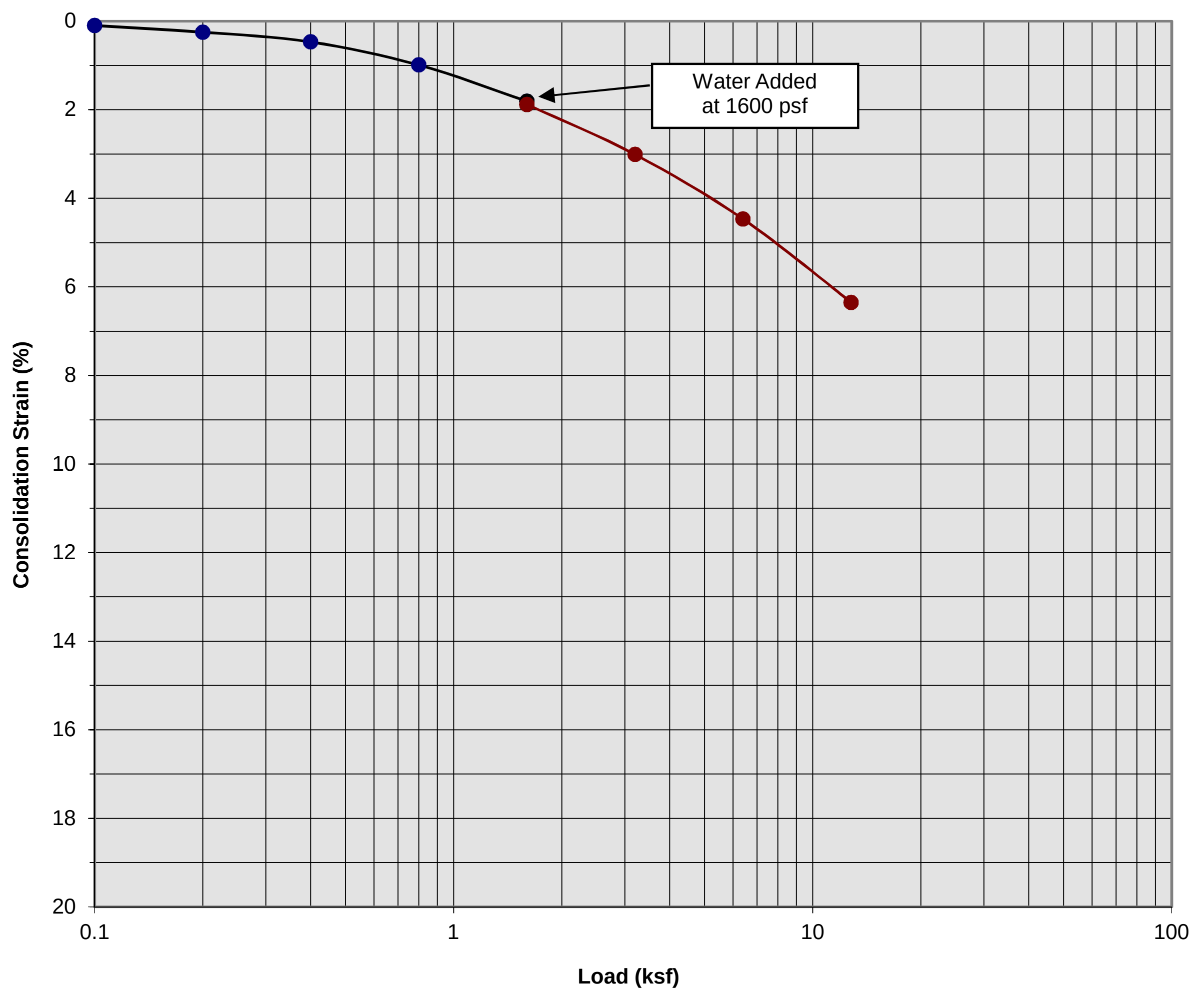
Classification: Light Gray Brown Silty fine Sand

Boring Number:	B-8	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	7 to 8	Initial Dry Density (pcf)	88.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	95.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.47

Proposed Industrial Development
Colton, California
Project No. 18G212-2
PLATE C- 7

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

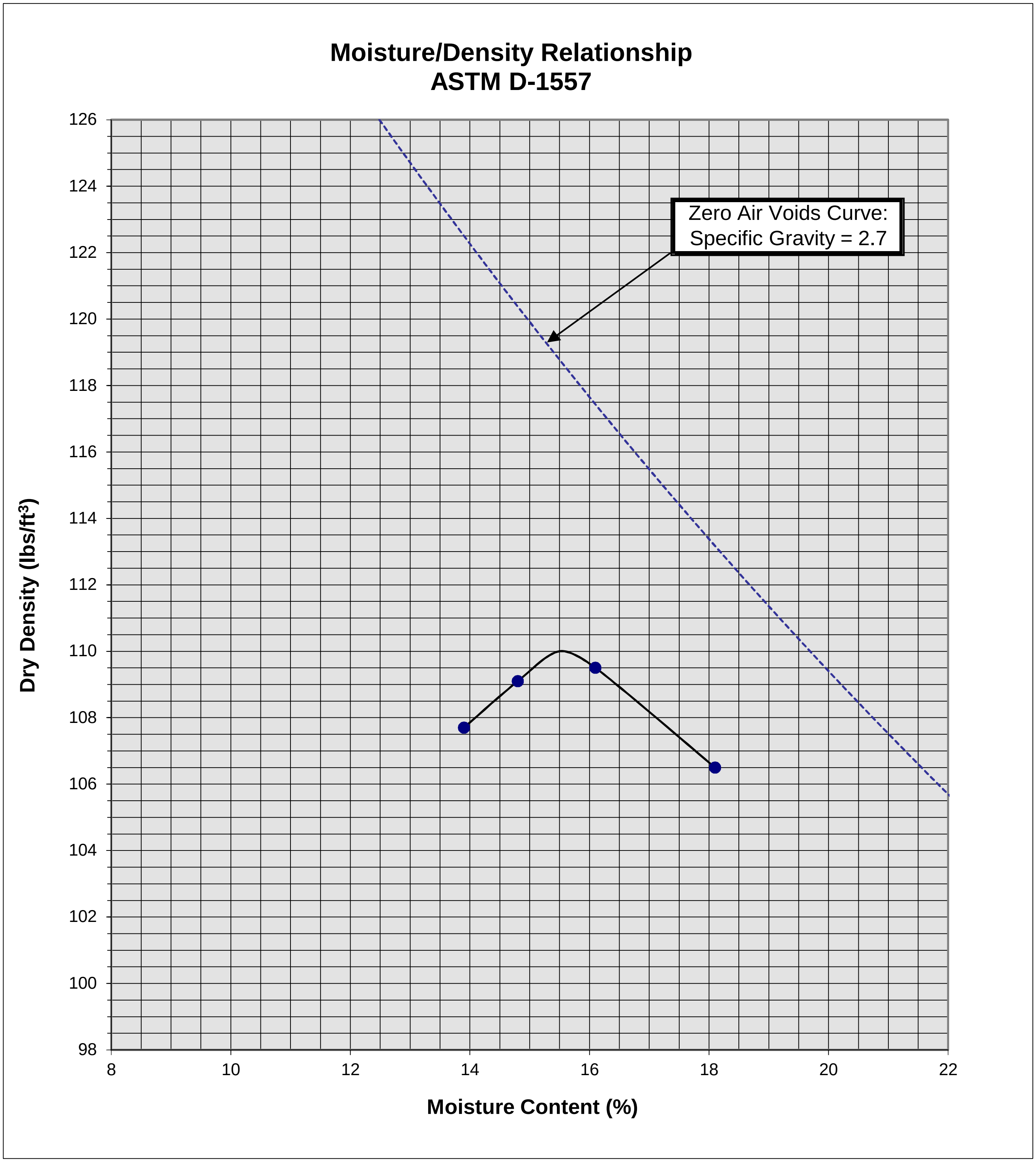


Classification: Light Gray Brown fine to medium Sand

Boring Number:	B-8	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	30
Depth (ft)	9 to 10	Initial Dry Density (pcf)	97.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	103.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.07

Proposed Industrial Development
Colton, California
Project No. 18G212-2
PLATE C- 8

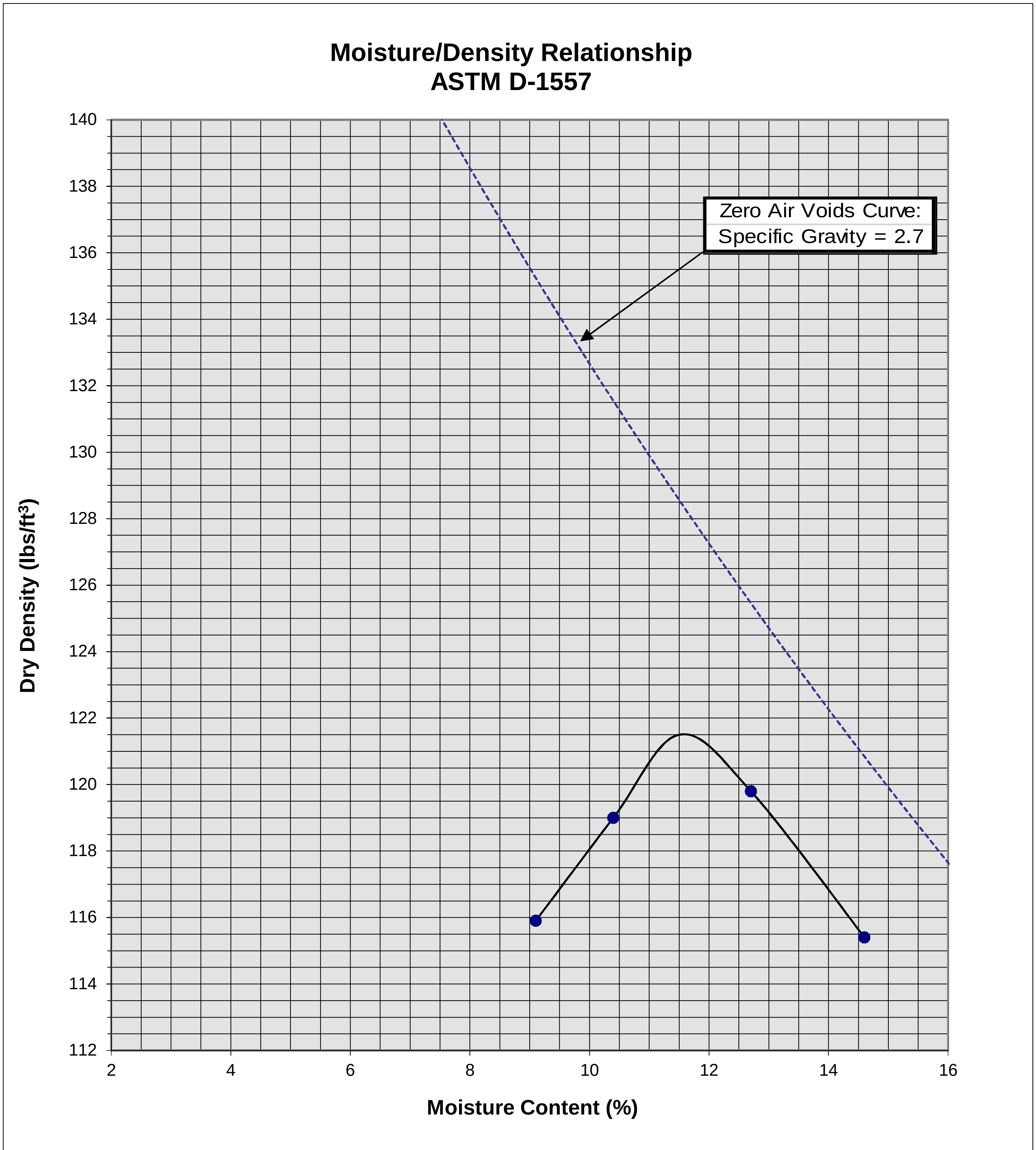
SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-1 @ 0 to 5'
Optimum Moisture (%)	15.5
Maximum Dry Density (pcf)	110
Soil Classification	Dark Gray Brown fine Sandy Silt

Proposed Industrial Development
Colton, California
Project No. 18G212-2
PLATE C-9

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number		B-8 @ 0 to 5'
Optimum Moisture (%)		11.5
Maximum Dry Density (pcf)		121.5
Soil Classification	Dark Gray Brown fine to medium Sand, trace to little Silt	

Proposed Industrial Development
Colton, California
Project No. 18G212-2
PLATE C-10

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.
- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.
- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.
- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.
- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.
- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.
- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

<u>Compacted Fills</u>

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as “contaminated,” and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:

  - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.

  - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

- Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.

- To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates D-6.

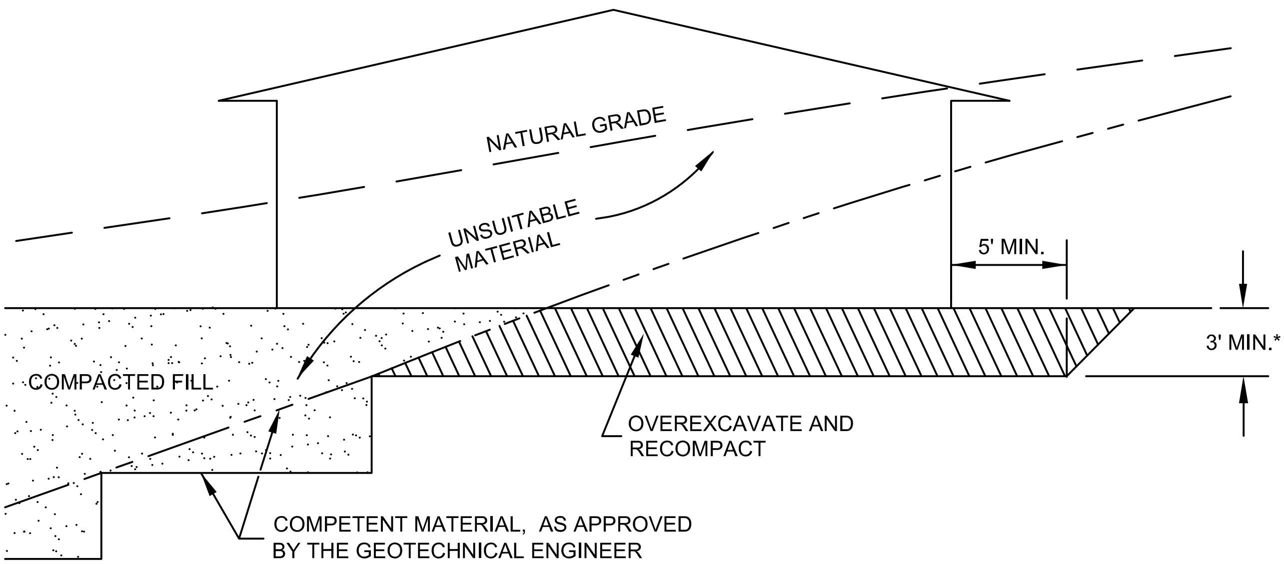
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

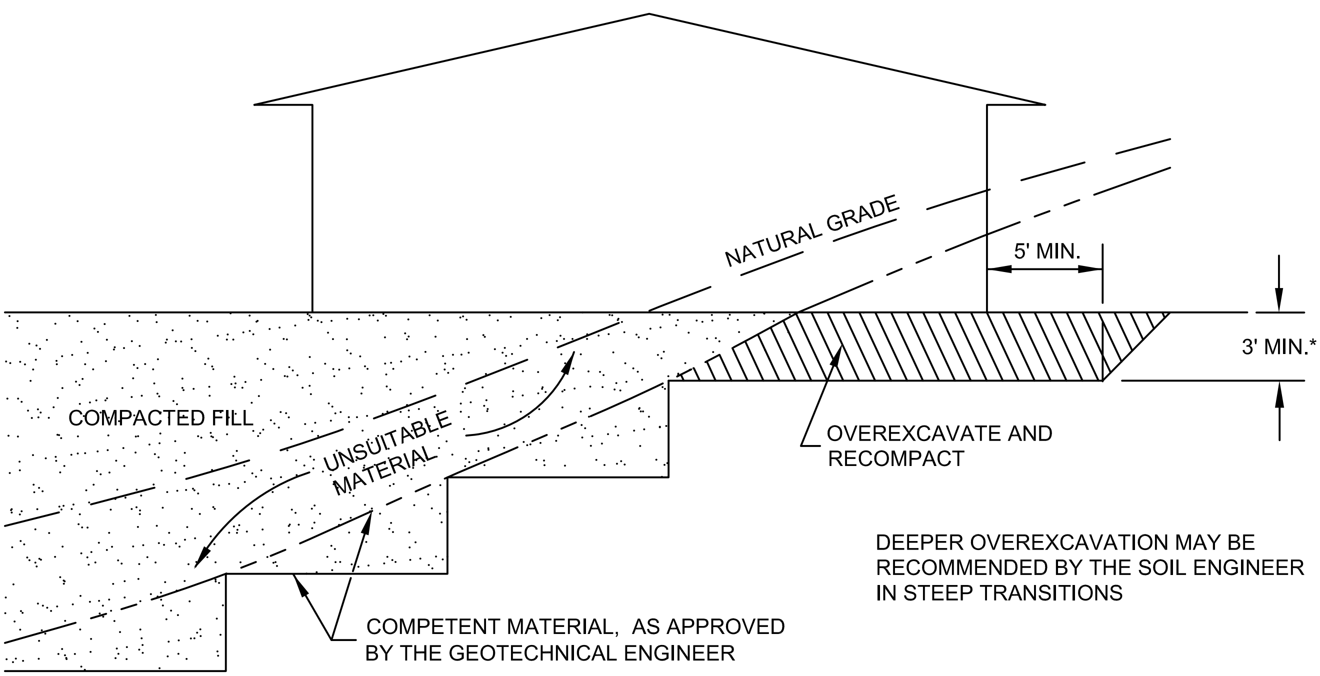
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

CUT LOT



CUT/FILL LOT (TRANSITION)



DEEPER OVEREXCAVATION MAY BE RECOMMENDED BY THE SOIL ENGINEER IN STEEP TRANSITIONS

*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL

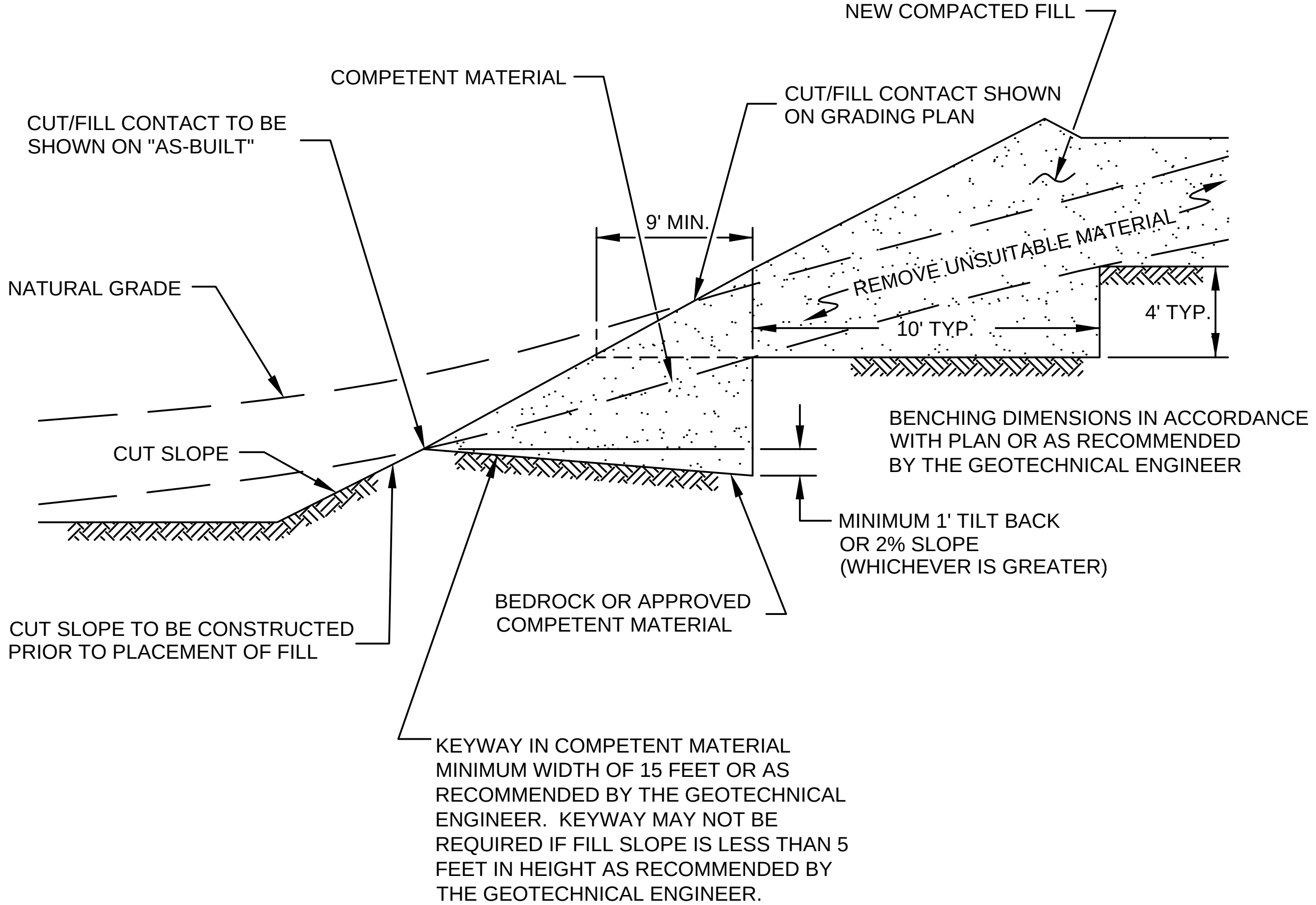
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

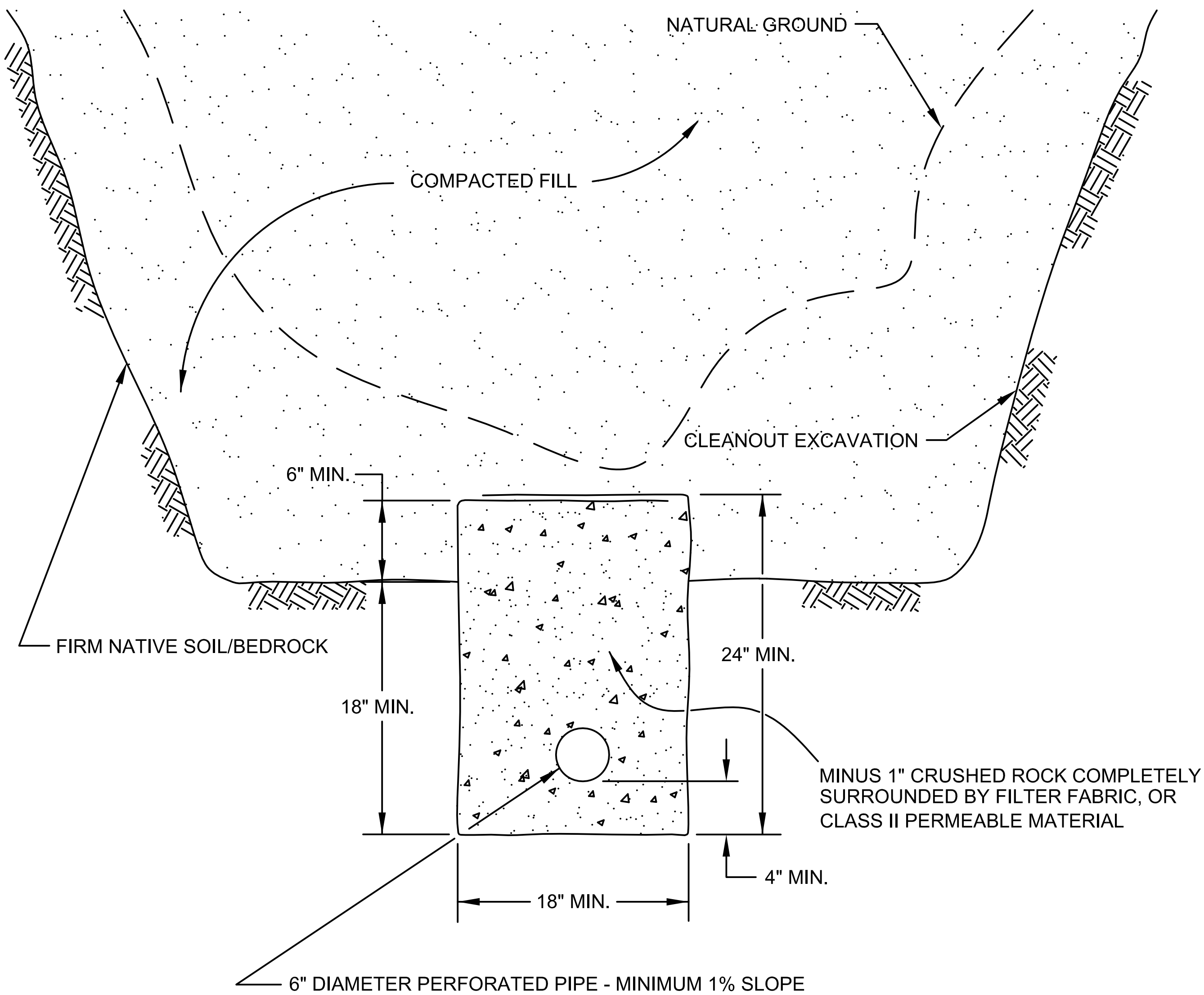
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SOUTHERN CALIFORNIA GEOTECHNICAL



PIPE MATERIAL	DEPTH OF FILL OVER SUBDRAIN
ADS (CORRUGATED POLETHYLENE)	8
TRANSITE UNDERDRAIN	20
PVC OR ABS: SDR 35	35
SDR 21	100

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

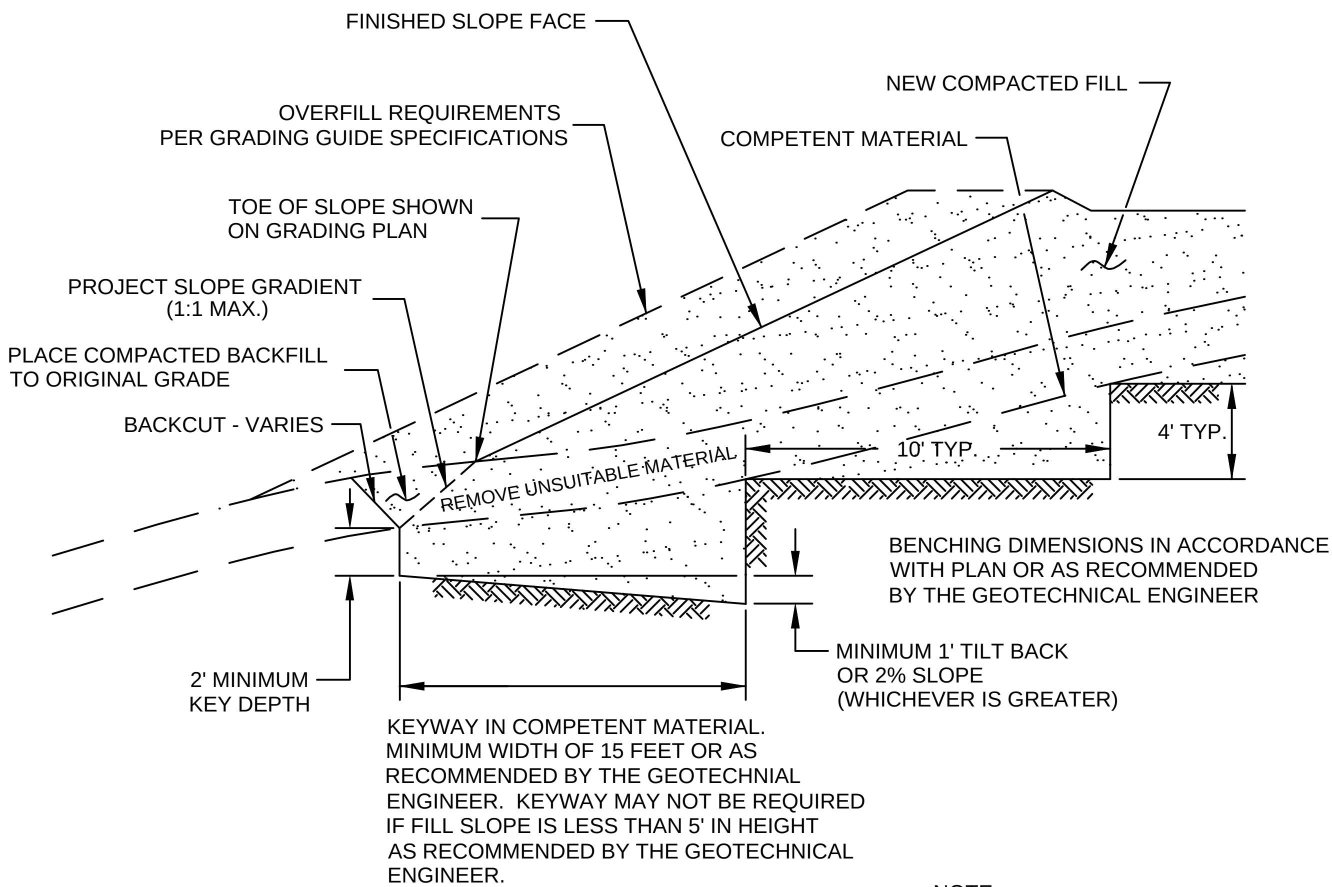
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



NOTE:
BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL

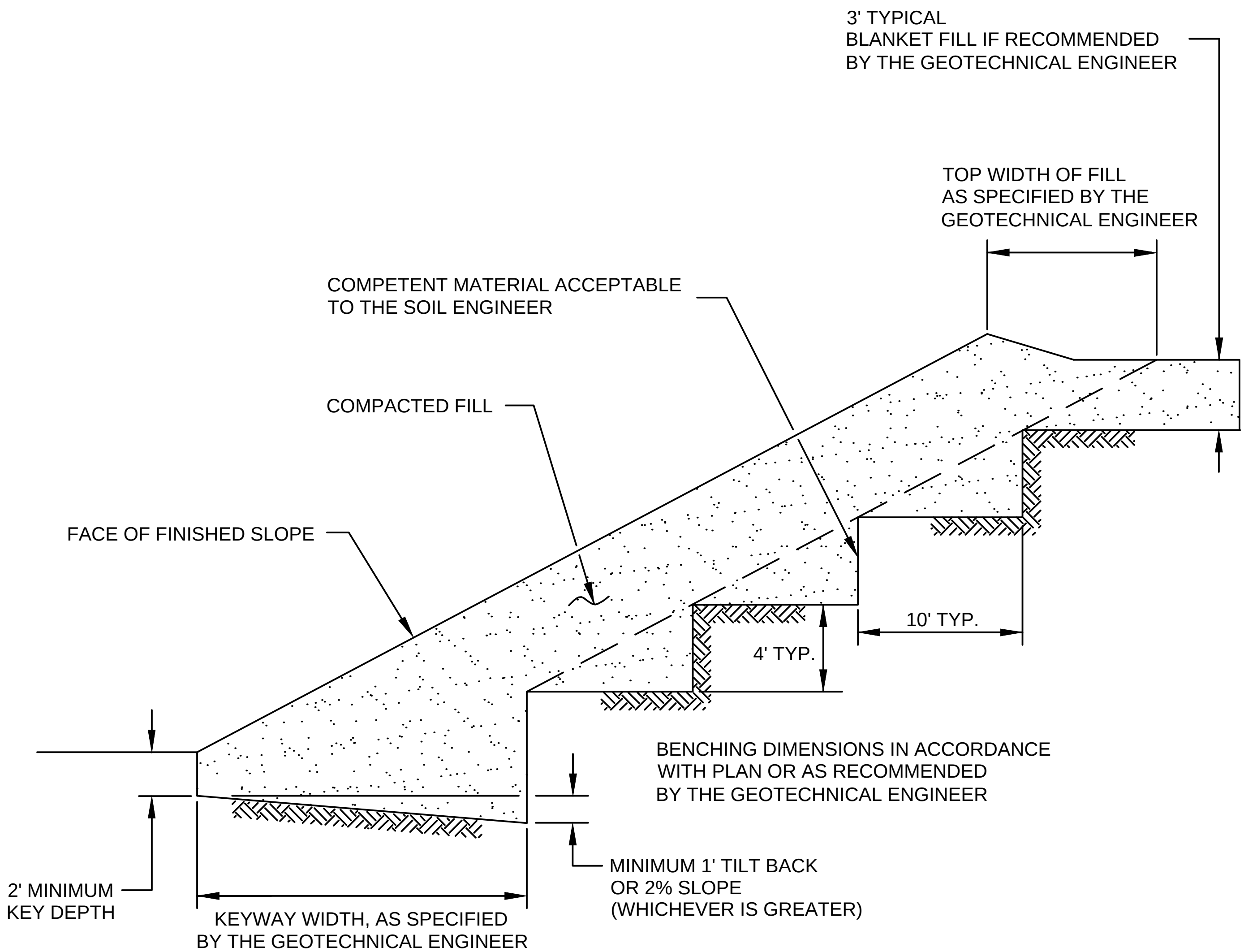
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

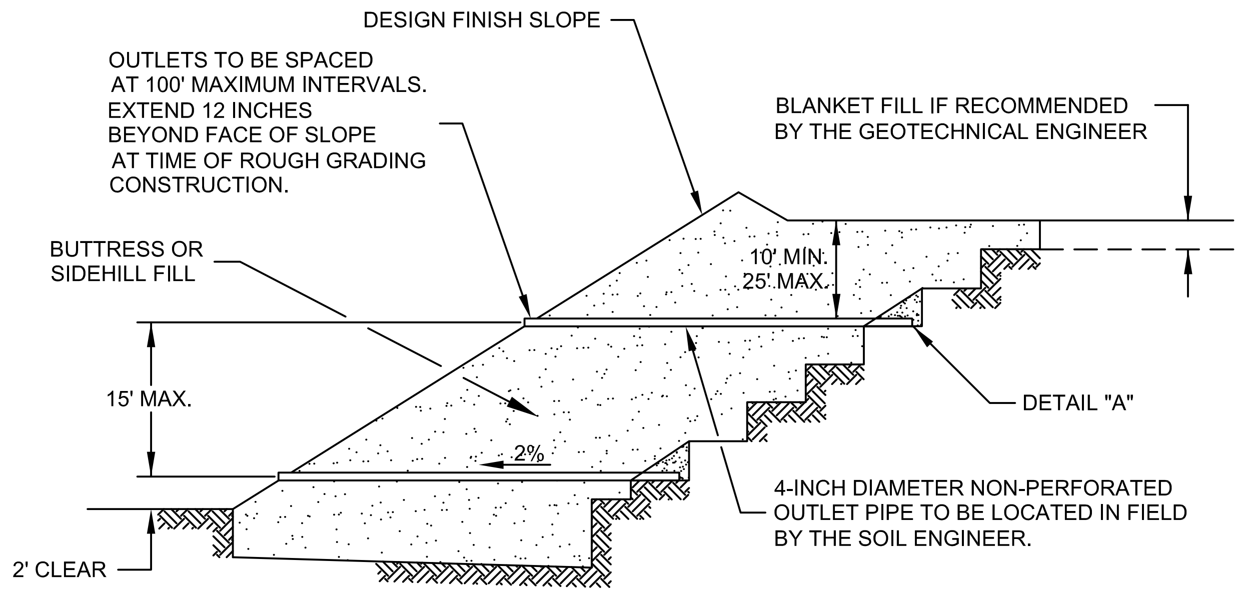
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

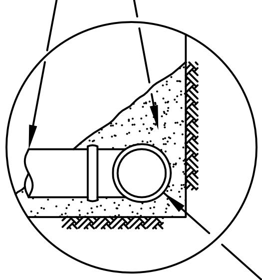
SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW



FILTER MATERIAL - MINIMUM OF FIVE CUBIC FEET PER FOOT OF PIPE. SEE ABOVE FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL FIVE CUBIC FEET OF GRAVEL PER FOOT OF PIPE MAY BE ENCASED IN FILTER FABRIC. SEE ABOVE FOR GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH A CRUSHING STRENGTH OF AT LEAST 1,000 POUNDS, WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

DETAIL "A"

NOTES:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

SLOPE FILL SUBDRAINS

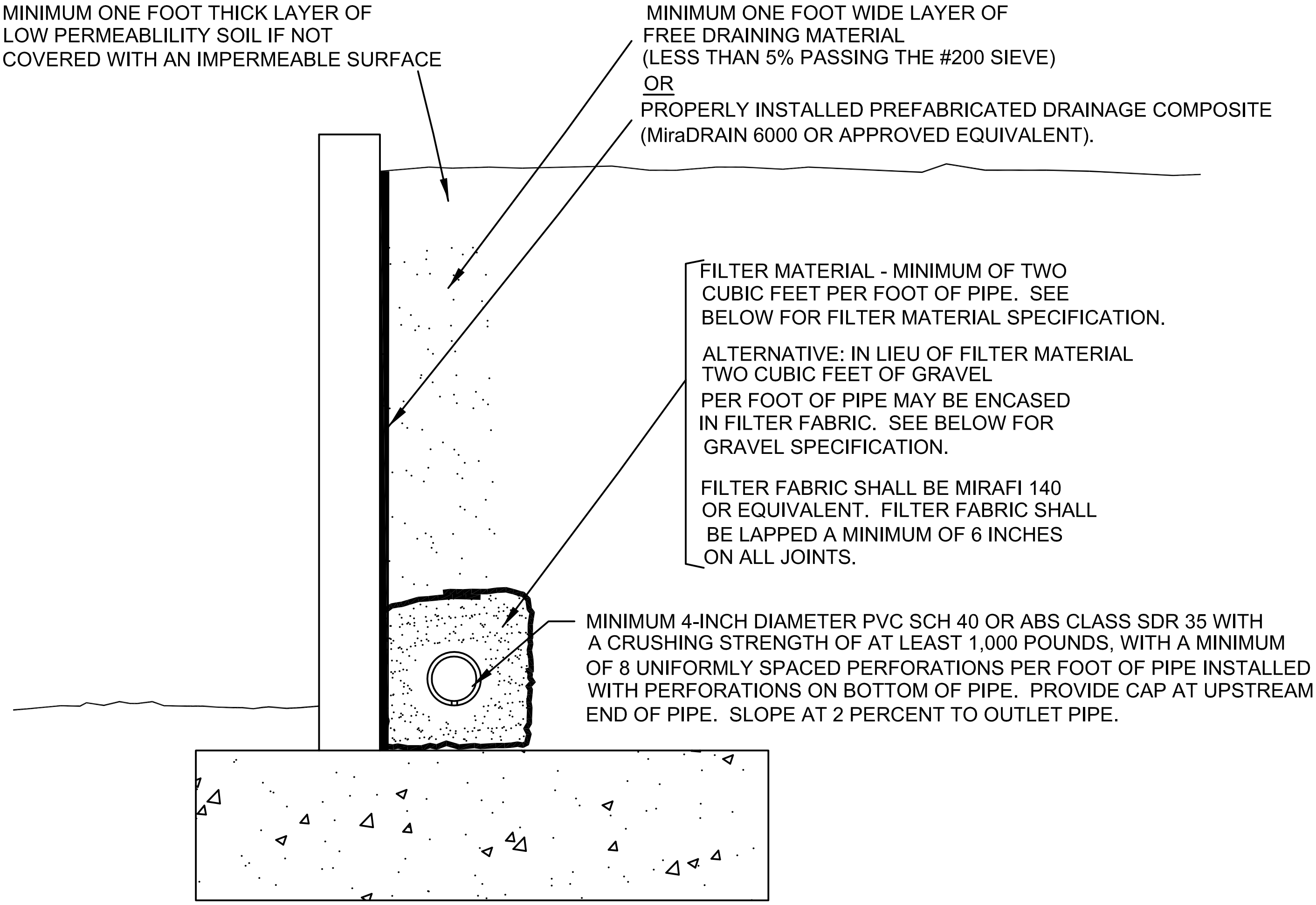
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	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" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

GRADING GUIDE SPECIFICATIONS

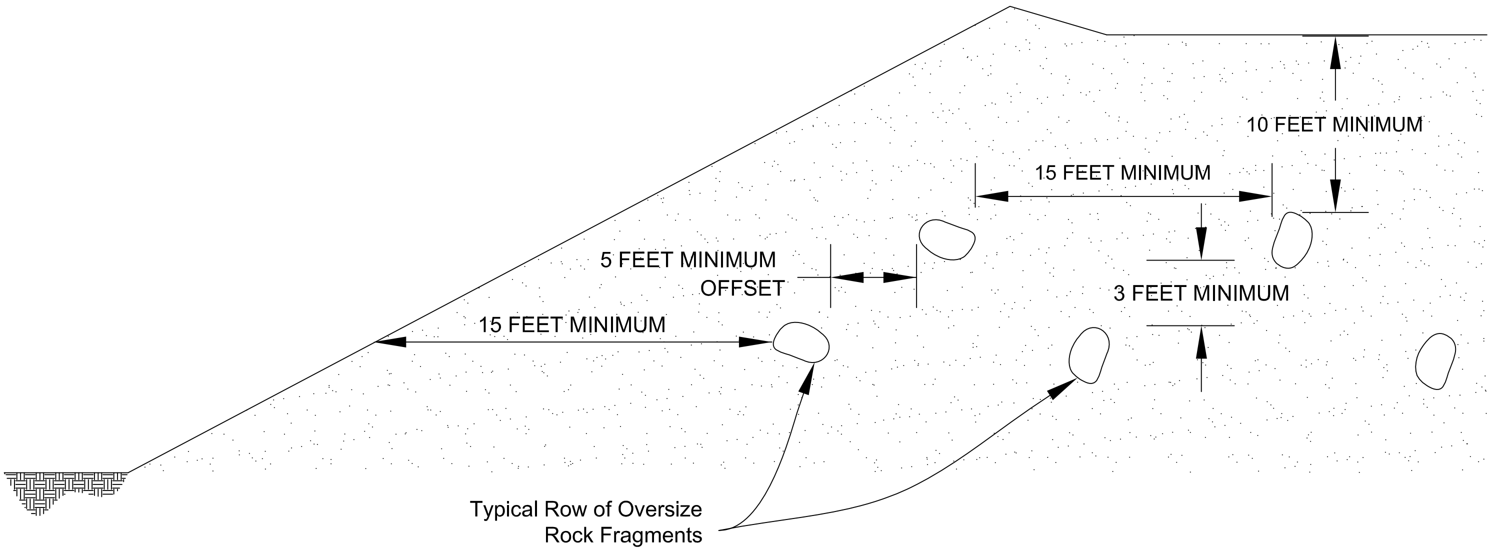
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

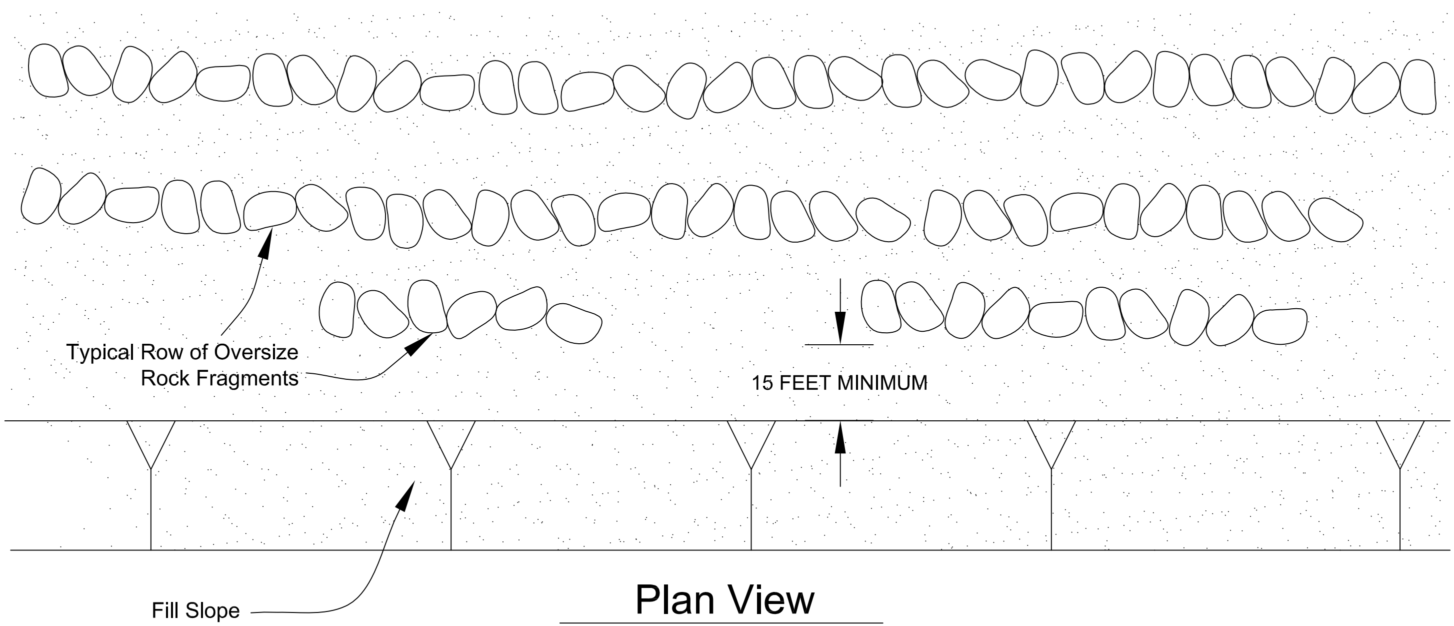
PLATE D-7

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

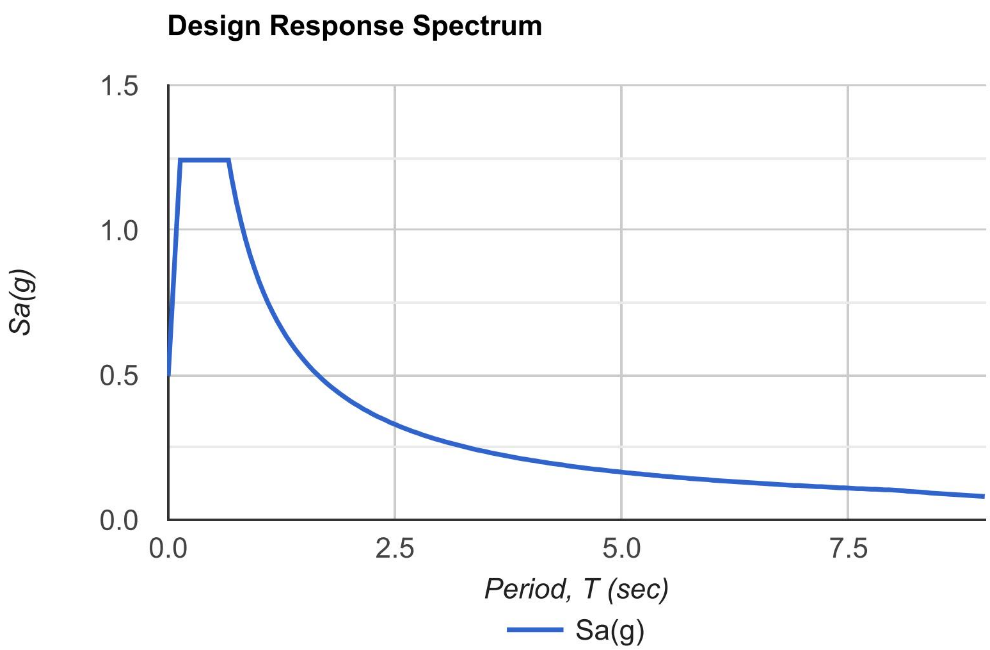
SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

Latitude, Longitude: 34.051940, -117.344519

Date	2/21/2019, 9:39:10 AM
Design Code Reference Document	ASCE7-10
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description	Type	Value	Description
S_S	1.864	MCE_R ground motion. (for 0.2 second period)	SDC	E	Seismic design category
S_1	0.824	MCE_R ground motion. (for 1.0s period)	F_a	1	Site amplification factor at 0.2 second
S_{MS}	1.864	Site-modified spectral acceleration value	F_v	1.5	Site amplification factor at 1.0 second
S_{M1}	1.237	Site-modified spectral acceleration value	PGA	0.727	MCE_G peak ground acceleration
S_{DS}	1.242	Numeric seismic design value at 0.2 second SA	F_{PGA}	1	Site amplification factor at PGA
S_{D1}	0.824	Numeric seismic design value at 1.0 second SA	PGA_M	0.727	Site modified peak ground acceleration



SOURCE: SEAOC/OSHPD Seismic Design Maps Tool
<https://seismicmaps.org/>

SEISMIC DESIGN PARAMETERS	
PROPOSED INDUSTRIAL DEVELOPMENT	
COLTON, CALIFORNIA	
DRAWN: AL CHKD: GKM	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
SCG PROJECT 18G212-2	
PLATE E-1	

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed Industrial Development	MCE_G Design Acceleration	0.727 (g)
Project Location	Colton, CA	Design Magnitude	8.1
Project Number	18G212-2	Historic High Depth to Groundwater	16 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	34 (ft)
		Borehole Diameter	6 (in)

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	KS	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=8.1)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	6	3		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	360	360	360	1.00	0.98	1.09	N/A	N/A	N/A	N/A	Above Water Table
7	6	8	7	7	120	80	1.3	1.05	1.12	1.53	0.75	12.3	17.8	840	840	840	1.00	0.92	1.1	N/A	N/A	N/A	N/A	Above Water Table
9.5	8	12	10	12	120	9	1.3	1.05	1.19	1.29	0.75	18.8	19.5	1200	1200	1200	0.99	0.91	1.07	N/A	N/A	N/A	N/A	Above Water Table
14.5	12	16	14	30	120		1.3	1.05	1.3	1.06	0.85	48.0	48.0	1680	1680	1680	0.98	0.78	1.07	N/A	N/A	N/A	N/A	Above Water Table
19.5	16	22	19	48	120		1.3	1.05	1.3	0.99	0.95	80.3	80.3	2280	2093	2280	0.97	0.78	1	2.00	1.56	0.50	3.11	Nonliquefiable
24.5	22	27	24.5	32	120		1.3	1.05	1.3	0.92	0.95	49.8	49.8	2940	2410	2940	0.96	0.78	0.96	2.00	1.49	0.55	2.70	Nonliquefiable
29.5	27	32	29.5	66	120		1.3	1.05	1.3	1.02	0.95	113.2	113.2	3540	2698	3540	0.95	0.78	0.93	2.00	1.44	0.59	2.45	Nonliquefiable
34.5	32	37	34.5	70	120		1.3	1.05	1.3	1.07	1	132.8	132.8	4140	2986	4109	0.93	0.78	0.9	2.00	1.40	0.61	2.28	Nonliquefiable
39.5	37	42	39.5	30	120		1.3	1.05	1.3	0.82	1	43.4	43.4	4740	3274	4397	0.92	0.78	0.87	2.00	1.35	0.63	2.15	Nonliquefiable
44.5	42	47	44.5	35	120		1.3	1.05	1.3	0.83	1	51.6	51.6	5340	3562	4685	0.90	0.78	0.84	2.00	1.31	0.64	2.05	Nonliquefiable
49.5	47	50	48.5	50	120		1.3	1.05	1.3	0.93	1	82.5	82.5	5820	3792	4915	0.89	0.78	0.83	2.00	1.29	0.65	1.99	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Industrial Development
Project Location	Colton, CA
Project Number	18G212-2
Engineer	DWN

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines cont	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	6	3	0.0	0.0	0.0	N/A	0.50	0.95	0.00	6.00		0.000		0.00	Above Water Table
7	6	8	7	12.3	5.5	17.8	N/A	0.20	0.63	0.00	2.00		0.000		0.00	**Above Water Table**
9.5	8	12	10	18.8	0.7	19.5	N/A	0.17	0.54	0.00	4.00		0.000		0.00	**Above Water Table**
14.5	12	16	14	48.0	0.0	48.0	N/A	0.00	-1.42	0.00	4.00		0.000		0.00	Above Water Table
19.5	16	22	19	80.3	0.0	80.3	3.11	0.00	-4.23	0.00	6.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	49.8	0.0	49.8	2.70	0.00	-1.57	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	113.2	0.0	113.2	2.45	0.00	-7.34	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	132.8	0.0	132.8	2.28	0.00	-9.28	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	43.4	0.0	43.4	2.15	0.00	-1.07	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	51.6	0.0	51.6	2.05	0.00	-1.72	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	82.5	0.0	82.5	1.99	0.00	-4.42	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				0.00	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed Industrial Development	MCE$_G$ Design Acceleration	0.727 (g)
Project Location	Colton, CA	Design Magnitude	8.1
Project Number	18G212-2	Historic High Depth to Groundwater	16 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-5		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	KS	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=8.1)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	6	3		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	360	360	360	1.00	0.98	1.09	N/A	N/A	N/A	N/A	Above Water Table
7	6	8	7	9	120	10	1.3	1.05	1.16	1.53	0.75	16.4	17.6	840	840	840	1.00	0.93	1.1	N/A	N/A	N/A	N/A	Above Water Table
9.5	8	12	10	17	120	5	1.3	1.05	1.28	1.24	0.75	27.6	27.6	1200	1200	1200	0.99	0.84	1.1	N/A	N/A	N/A	N/A	Above Water Table
14.5	12	16	14	32	120		1.3	1.05	1.3	1.06	0.85	51.0	51.0	1680	1680	1680	0.98	0.78	1.07	N/A	N/A	N/A	N/A	Above Water Table
19.5	16	22	19	19	120	8	1.3	1.05	1.3	0.97	0.95	31.2	31.6	2280	2093	2280	0.97	0.80	1	0.60	0.48	0.50	0.96	**Liquefiable**
24.5	22	27	24.5	16	120	87	1.3	1.05	1.22	0.88	0.95	22.4	28.0	2940	2410	2940	0.96	0.84	0.97	0.38	0.31	N/A	N/A	NonLiq: PI>18
29.5	27	32	29.5	27	120		1.3	1.05	1.3	0.85	0.95	38.9	38.9	3540	2698	3540	0.95	0.78	0.93	2.00	1.44	0.59	2.45	Nonliquefiable
34.5	32	37	34.5	14	120	79	1.3	1.05	1.17	0.75	1	16.8	22.4	4140	2986	4140	0.93	0.89	0.95	N/A	N/A	N/A	N/A	NonLiq: 12<PI<18, w<85%LL
39.5	37	42	39.5	38	120		1.3	1.05	1.3	0.85	1	57.3	57.3	4740	3274	4740	0.92	0.78	0.87	2.00	1.35	0.63	2.15	Nonliquefiable
44.5	42	47	44.5	39	120		1.3	1.05	1.3	0.83	1	57.4	57.4	5340	3562	5340	0.90	0.78	0.84	2.00	1.31	0.64	2.05	Nonliquefiable
49.5	47	50	48.5	30	120		1.3	1.05	1.3	0.74	1	39.2	39.2	5820	3792	5820	0.89	0.78	0.83	2.00	1.29	0.65	1.99	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Industrial Development
Project Location	Colton, CA
Project Number	18G212-2
Engineer	DWN

Boring No. B-5

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines cont	$(N_1)_{60\text{-CS}}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	6	3	0.0	0.0	0.0	N/A	0.50	0.95	0.00	6.00		0.000		0.00	Above Water Table
7	6	8	7	16.4	1.1	17.6	N/A	0.21	0.64	0.00	2.00		0.000		0.00	Above Water Table
9.5	8	12	10	27.6	0.0	27.6	N/A	0.06	0.07	0.00	4.00		0.000		0.00	Above Water Table
14.5	12	16	14	51.0	0.0	51.0	N/A	0.00	-1.67	0.00	4.00		0.000		0.00	Above Water Table
19.5	16	22	19	31.2	0.4	31.6	0.96	0.04	-0.19	0.04	6.00		0.007		0.51	**Liquefiable**
24.5	22	27	24.5	22.4	5.5	28.0	N/A	0.06	0.05	0.00	5.00		0.000		0.00	NonLiq: PI>18
29.5	27	32	29.5	38.9	0.0	38.9	2.45	0.01	-0.72	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	16.8	5.5	22.4	N/A	0.12	0.39	0.00	5.00		0.000		0.00	NonLiq: 12<PI<18, w<85%LL
39.5	37	42	39.5	57.3	0.0	57.3	2.15	0.00	-2.19	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	57.4	0.0	57.4	2.05	0.00	-2.20	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	39.2	0.0	39.2	1.99	0.01	-0.74	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				0.51	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed Industrial Development	MCE_G Design Acceleration	0.727 (g)
Project Location	Colton, CA	Design Magnitude	8.1
Project Number	18G212-2	Historic High Depth to Groundwater	16 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	48.5 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-7		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	KS	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=8.1)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	6	3		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	360	360	360	1.00	0.98	1.09	N/A	N/A	N/A	N/A	Above Water Table
7	6	8	7	8	120	91	1.3	1.05	1.14	1.51	0.75	14.1	19.6	840	840	840	1.00	0.91	1.1	0.20	N/A	N/A	N/A	Above Water Table
9.5	8	12	10	16	120	13	1.3	1.05	1.25	1.24	0.75	25.5	28.0	1200	1200	1200	0.99	0.84	1.1	0.38	N/A	N/A	N/A	Above Water Table
14.5	12	16	14	14	120	5	1.3	1.05	1.22	1.10	0.85	21.8	21.8	1680	1680	1680	0.98	0.89	1.03	0.23	N/A	N/A	N/A	Above Water Table
19.5	16	22	19	14	120	23	1.3	1.05	1.21	0.97	0.95	21.4	26.3	2280	2093	2280	0.97	0.85	1	0.32	0.28	0.50	0.55	**Liquefiable**
24.5	22	27	24.5	11	120	53	1.3	1.05	1.14	0.86	0.95	14.1	19.7	2940	2410	2940	0.96	0.91	0.98	0.20	0.18	0.55	0.33	**Liquefiable**
29.5	27	32	29.5	48	120		1.3	1.05	1.3	0.94	0.95	76.3	76.3	3540	2698	3540	0.95	0.78	0.93	2.00	1.44	0.59	2.45	Nonliquefiable
34.5	32	37	34.5	36	120		1.3	1.05	1.3	0.87	1	55.4	55.4	4140	2986	4140	0.93	0.78	0.9	2.00	1.40	0.61	2.28	Nonliquefiable
39.5	37	42	39.5	35	120		1.3	1.05	1.3	0.83	1	51.5	51.5	4740	3274	4740	0.92	0.78	0.87	2.00	1.35	0.63	2.15	Nonliquefiable
44.5	42	47	44.5	35	120		1.3	1.05	1.3	0.80	1	49.6	49.6	5340	3562	5340	0.90	0.78	0.84	2.00	1.31	0.64	2.05	Nonliquefiable
49.5	47	50	48.5	27	120	50	1.3	1.05	1.3	0.75	1	35.7	41.3	5820	3792	5820	0.89	0.78	0.83	2.00	1.29	0.65	1.99	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Industrial Development
Project Location	Colton, CA
Project Number	18G212-2
Engineer	DWN

Boring No. B-7

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines cont	$(N_1)_{60\text{-CS}}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	6	3	0.0	0.0	0.0	N/A	0.50	0.95	0.00	6.00		0.000		0.00	Above Water Table
7	6	8	7	14.1	5.5	19.6	N/A	0.17	0.54	0.00	2.00		0.000		0.00	Above Water Table
9.5	8	12	10	25.5	2.5	28.0	N/A	0.06	0.05	0.00	4.00		0.000		0.00	Above Water Table
14.5	12	16	14	21.8	0.0	21.8	N/A	0.13	0.42	0.00	4.00		0.000		0.00	Above Water Table
19.5	16	22	19	21.4	4.9	26.3	0.55	0.08	0.15	0.08	6.00		0.017		1.23	**Liquefiable**
24.5	22	27	24.5	14.1	5.6	19.7	0.33	0.16	0.53	0.16	5.00		0.023		1.40	**Liquefiable**
29.5	27	32	29.5	76.3	0.0	76.3	2.45	0.00	-3.86	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	55.4	0.0	55.4	2.28	0.00	-2.03	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	51.5	0.0	51.5	2.15	0.00	-1.71	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	49.6	0.0	49.6	2.05	0.00	-1.56	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	35.7	5.6	41.3	1.99	0.01	-0.91	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				2.63	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

APPENDIX G

EXPLORATORY BORING NO. 1

Date Drilled: 3/17/17 | Client: CalPortland

Equipment: CME75 Truck Rig | Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.

Surface Elevation(ft): 894 | Logged by: GA | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(ML) Sandy Silt, fine, with clay, brown	Native						
				X		3 4 4			Pass #200, SPT
					X		7.0		Cor., DS, MDC, SA
5				X		3 3 4			Pass #200, SPT
		(SP-SM) Sand, fine to coarse, with silt and gravel to 2", gray							
10			Auger Chatter	X		5 12 14			Pass #200, SPT
15		(SP) Sand, fine to coarse, with gravel to 1" and few silt, gray	Auger Chatter	X		7 10 13			Pass #200, SPT
20		(ML) Sandy Silt, fine, with clay, strong brown		X		9 17 12			Pass #200, SPT
25				X		11 12 9			Pass #200, SPT
			Added Water						
30				X		5 11 27			Pass #200, SPT

EXPLORATORY BORING NO. 1

Date Drilled: 3/17/17

Client: CalPortland

Equipment: CME75 Truck Rig

Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.

Surface Elevation(ft): 894

Logged by: GA

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine to coarse, with gravel to 1", yellowish brown	Added Water	X		35 38 28			Pass #200, SPT
40		(ML) Sandy Silt, fine, with clay, dark yellowish brown		X		6 10 30			Pass #200, SPT
45				X		12 23 17			Pass #200, SPT
50				X		7 8 13			Pass #200, SPT
		END OF BORING NO REFUSAL, NO BEDROCK NO CAVING, NO FILL NO GROUNDWATER							
55									
60									
65									

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 2

Date Drilled: 3/17/17 | Client: CalPortland

Equipment: CME75 Truck Rig | Driving Weight / Drop / Sampler Size: 140lbs./30in./3.25" O.D.

Surface Elevation(ft): 894 | Logged by: GA | Measured Depth to Water(ft): N/A

DEPTH (ft)	VISUAL CLASSIFICATION	REMARKS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0	(SP) Sand, fine to medium, greenish gray	Native						
			X		7 8 10	1.8	105	Pass #200, Ring
				X			2.4	Cor., DS, MDC, SA
5	(SP-SM) Sand, fine to coarse, with silt and gravel to 1", gray		X		3 6 9	1.6	Dist.	Pass #200, Ring
10	(SM) Silty Sand, fine to coarse, gray		X		4 14 25	2.0	Dist.	Pass #200, Ring
		Auger Chatter						
15	(SP) Sand, fine to coarse, with gravel to 3" and few silt, gray		X		12 19 8	1.5	122	Pass #200, Ring
20			X		12 29 39	2.1	116	Pass #200, Ring
25	(SM) Silty Sand, fine, with clay, dark grayish brown		X		11 16 18	15.2	118	Pass #200, Ring
		Auger Chatter						
30			X		17 13 33	13.9	121	Pass #200, Ring

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 2

Date Drilled: 3/17/17

Client: CalPortland

Equipment: CME75 Truck Rig

Driving Weight / Drop / Sampler Size: 140lbs./30in./3.25" O.D.

Surface Elevation(ft): 894

Logged by: GA

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine, with clay, dark grayish brown	Auger Chatter	X		17 35 50/5"	8.3	128	Pass #200, Ring
40				X		25 50	8.9	122	Pass #200, Ring
45			Gravel lenses	X		22 50/5"	10.3	130	Pass #200, Ring
50				X		26 50/5"	6.2	124	Pass #200, Ring
		END OF BORING							
		NO REFUSAL, NO BEDROCK NO CAVING, NO FILL NO GROUNDWATER							
55									
60									
65									

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 3

Date Drilled: 3/16/17
Client: CalPortland
Equipment: CME75 Truck Rig
Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.
Surface Elevation(ft): 876
Logged by: VJR
Measured Depth to Water(ft): 31.8

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine to medium, with gravel and few cobbles to 9", light grayish brown	Fill		X		4.8		Cor., DS, MDC, SA
				X		9 9 7	N.R.	N.R.	Pass #200, SPT
5				X		3 4 8			Pass #200, SPT
		(SM) Silty Sand, fine to coarse, with gravel to 3", yellowish brown	Native						
10				X		3 9 34			Pass #200, SPT
15		(SP-SM) Sand, fine to medium, with silt and few gravel to 1", yellowish brown		X		9 13 17			Pass #200, SPT
20				X		6 16 16			Pass #200, SPT
25				X		9 20 24			Pass #200, SPT
30				X		16 20 25			Pass #200, SPT
		(SM) Silty Sand, fine to medium with coarse, with gravel to 2", brown	Auger Chatter Groundwater						

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

CHJ

75-ACRE SITE
SOUTH OF AGUA MANSA ROAD, COLTON, CALIFORNIA

Job No. 16113-3

Enclosure B-3a

EXPLORATORY BORING NO. 3

Date Drilled: 3/16/17
Client: CalPortland
Equipment: CME75 Truck Rig
Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.
Surface Elevation(ft): 876
Logged by: VJR
Measured Depth to Water(ft): 31.8

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine to coarse, with gravel to 3", brown		X		22 32 36			Pass #200, SPT
40		(SP-SM) Sand, fine to coarse, with silt and gravel to 2", brown		X		15 24 28			SPT
45			Sand Plug	X		12 30 40			SPT
50		(SM) Silty Sand, fine to medium, brown		X		8 19 27			SPT
		END OF BORING NO REFUSAL, NO BEDROCK NO CAVING, FILL TO 7' GROUNDWATER ENCOUNTERED AT 31.75'							
55									
60									
65									

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 4

Date Drilled: 3/17/17
Client: CalPortland
Equipment: CME75 Truck Rig
Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.
Surface Elevation(ft): 883
Logged by: GA
Measured Depth to Water(ft): 37.0

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SP) Sand, fine to coarse, with gravel to 2" and few silt, grayish brown	Fill						
				X		4 5 7			Pass #200, SPT
					X		2.2		Cor., DS, MDC, SA
5				X		3 3 11			Pass #200, SPT
		(SP-SM) Sand, fine to medium, with silt, grayish brown							
10				X		2 2 8			Pass #200, SPT
			6" clay lens		X				
			Auger Chatter		X		4.0		
15				X		6 9 9			Pass #200, SPT
		(GP) Sandy Gravel, fine to coarse, with silt, gravel to 2", brown	Native						
20				X		8 20 30			Pass #200, SPT
25		(SM) Silty Sand, fine, grayish brown		X		8 14 18			Pass #200, SPT
30		(GP) Sandy Gravel, fine to coarse, with silt, gravel to 2", grayish brown		X		23 50			Pass #200, SPT
			Auger Chatter						

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 4

Date Drilled: 3/17/17

Client: CalPortland

Equipment: CME75 Truck Rig

Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.

Surface Elevation(ft): 883

Logged by: GA

Measured Depth to Water(ft): 37.0

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SP-SM) Sand, fine, with silt, grayish brown	▼ Groundwater	X		14 25 30			Pass #200, SPT
40		(SM) Silty Sand, fine to coarse, with clay and gravel to 1", brown		X		10 20 42			Pass #200, SPT
45				X		22 40 50/4"			Pass #200, SPT
			Auger Chatter						
50		(ML) Sandy Silt, fine, with clay, dark olive brown		X		8 16 42			Pass #200, SPT
		END OF BORING NO REFUSAL, NO BEDROCK NO CAVING, FILL TO 17' GROUNDWATER ENCOUNTERED AT 37'							
55									
60									
65									

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 5

Date Drilled: 3/17/17
Client: CalPortland
Equipment: CME75 Truck Rig
Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.
Surface Elevation(ft): 871
Logged by: GA
Measured Depth to Water(ft): 31.5

DEPTH (ft)	VISUAL CLASSIFICATION	REMARKS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
	Organic rich soil	Fill						
	(SP-SM) Sand, fine to coarse, with silt and gravel to 2", dark grayish brown		X		20 19 19			SPT
	(ML) Sandy Silt, fine to coarse, with gravel to 2", brown	Native		X		7.0		Cor., DS, MDC, SA
5			X		3 3 9			Pass #200, SPT
	(SP) Sand, fine to coarse, dark yellowish brown							
10		Auger Chatter	X		2 4 14			Pass #200, SPT
15	(SM) Silty Sand, fine to medium, with gravel to 1", dark yellowish brown		X		8 19 19			Pass #200, SPT
	(GP) Sandy Gravel, fine to coarse, with silt, gravel to 3", dark yellowish brown	Heavy Auger Chatter						
20	(SP-SM) Sand, fine to medium, with gravel to 1", dark yellowish brown		X		2 20 22			Pass #200, SPT
25			X		11 14 21			Pass #200, SPT
30	(ML) Sandy Silt, fine, with clay, dark olive brown	▼ Groundwater	X		6 8 11			Pass #200, SPT

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 5

Date Drilled: 3/17/17
Client: CalPortland
Equipment: CME75 Truck Rig
Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.
Surface Elevation(ft): 871
Logged by: GA
Measured Depth to Water(ft): 31.5

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(ML) Sandy Silt, fine, with clay, dark olive brown		X		6 6 9			Pass #200, SPT
40				X		5 9 19			Pass #200, SPT
45				X		9 11 10			Pass #200, SPT
			Auger Chatter						
50		(SP) Sand, fine to coarse, with gravel to 1/4", brown		X		17 28 44			SPT
		END OF BORING NO REFUSAL, NO BEDROCK NO CAVING, FILL TO 3' GROUNDWATER ENCOUNTERED AT 31.5'							
55									
60									
65									

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 6

Date Drilled: 3/16/17
Client: CalPortland
Equipment: CME75 Truck Rig
Driving Weight / Drop / Sampler Size: 140lbs./30in./3.25" O.D.
Surface Elevation(ft): 885
Logged by: VJR
Measured Depth to Water(ft): 42.3

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine, light yellowish brown	Native		X		5.0		Cor., DS, MDC, SA
				X		4 6 7	6.5	87	Pass #200, Ring
5				X		3 5 7	3.4	99	Consol., Pass #200, Ring
10		(SP-SM) Sand, fine, with silt, light grayish brown		X		8 17 23	1.2	102	Pass #200, Ring
					X		2.3		
15				X		5 11 20	3.3	101	Pass #200, Ring
20		(SP) Sand, fine to coarse, with silt and gravel to 2", brown		X		16 23 11	2.2	122	Pass #200, Ring
25			Auger Chatter	X		21 50/5"	N.R.	N.R.	Pass #200, Ring
30				X		8 30 45	6.2	Dist.	Pass #200, Ring

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 6

Date Drilled: 3/16/17
Client: CalPortland
Equipment: CME75 Truck Rig
Driving Weight / Drop / Sampler Size: 140lbs./30in./3.25" O.D.
Surface Elevation(ft): 885
Logged by: VJR
Measured Depth to Water(ft): 42.3

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SP) Sand, fine to coarse, with silt and gravel to 2", brown	Interbedded silt lenses	X		15 50/2"	16.3	101	Pass #200, Ring
40				X		14 30 50	12.6	114	Pass #200, Ring
		(SM) Silty Sand, fine, brown	Groundwater						
45				X		10 23 35	24.2	101	Pass #200, Ring
50				X		8 11 28	27.2	98	Pass #200, Ring
		END OF BORING NO REFUSAL, NO BEDROCK NO CAVING, NO FILL GROUNDWATER ENCOUNTERED AT 42.33'							
55									
60									
65									

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

CHJ

75-ACRE SITE
SOUTH OF AGUA MANSA ROAD, COLTON, CALIFORNIA

Job No. 16113-3

Enclosure B-6b

EXPLORATORY BORING NO. 7

Date Drilled: 3/16/17 | Client: CalPortland
Equipment: CME75 Truck Rig | Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.
Surface Elevation(ft): 883 | Logged by: VJR | Measured Depth to Water(ft): 40.0

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine, pale yellow	Native		X		5.6		Cor., DS, MDC, SA
				X		2 2 4			Pass #200, SPT
5		(SP-SM) Sand, fine, with silt, light gray		X		3 3 4			Pass #200, SPT
					X		1.8		
10		(SP-SM) Sand, fine to medium, with silt and gravel to 2", very pale brown	Auger Chatter	X		4 4 8			Pass #200, SPT
15		(SM) Silty Sand, fine to coarse, with gravel to 2", light yellowish brown	Interbedded silt and sand Auger Chatter	X		4 4 14			Pass #200, SPT
20		(SP) Sand, fine to coarse, with gravel to 3" and few silt, brown		X		12 16 30			Pass #200, SPT
25		(SP-SM) Sand, fine to medium, with silt and gravel to 2", light yellowish brown		X		13 19 25			Pass #200, SPT
30				X		14 30 35			Pass #200, SPT

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 7

Date Drilled: 3/16/17 | Client: CalPortland

Equipment: CME75 Truck Rig | Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.

Surface Elevation(ft): 883 | Logged by: VJR | Measured Depth to Water(ft): 40.0

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SP-SM) Sand, fine to medium, with silt and gravel to 2", light yellowish brown		X		13 39 40			Pass #200, SPT
40			Groundwater	X		8 12 9			Pass #200, SPT
		(ML) Sandy Silt, fine, brown			X		30.6		
45				X		3 4 6			Pass #200, SPT
50				X		8 14 23			Pass #200, SPT
		END OF BORING NO REFUSAL, NO BEDROCK NO CAVING, NO FILL GROUNDWATER ENCOUNTERED AT 40'							
55									
60									
65									

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 8

Date Drilled: 3/16/17 | Client: CalPortland

Equipment: CME75 Truck Rig | Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.

Surface Elevation(ft): 881 | Logged by: VJR | Measured Depth to Water(ft): 37.7

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(ML) Sandy Silt, fine, light brownish gray	Native		X		2.8		Cor., DS, MDC, SA Pass #200, SPT
				X		2 3 5			
5		(SP-SM) Sand, fine to medium, with silt, pale yellow		X		3 3 4			Pass #200, SPT
					X		2.4		
10		(SP) Sand, fine to medium, few silt, pale yellow		X		4 7 12			Pass #200, SPT
15		(GM) Silty Sandy Gravel, fine to medium, gravel to 2", pale yellow	3" clay lens	X	X	4 9 29	25.0		Pass #200, SPT
		(SP-SM) Sand, fine to medium, with silt and few gravel to 1", light browish olive							
20				X		9 24 30			Pass #200, SPT
25		(SM) Silty Sand, fine with medium, with gravel to 3", olive	Interbedded silt and sand	X		5 7 9			Pass #200, SPT
30			Auger Chatter	X		7 14 28			Pass #200, SPT

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

EXPLORATORY BORING NO. 8

Date Drilled: 3/16/17

Client: CalPortland

Equipment: CME75 Truck Rig

Driving Weight / Drop / Sampler Size: 140lbs./30in./2.0" O.D.

Surface Elevation(ft): 881

Logged by: VJR

Measured Depth to Water(ft): 37.7

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine with medium, with gravel to 3", olive		X		13 27 34			Pass #200, SPT
		(SM) Silty Sand, fine to coarse, with clay and gravel to 3", olive brown	▼ Groundwater						
40				X		16 16 18			Pass #200, SPT
			Auger Chatter						
45		(SP-SM) Sand, fine to coarse, with silt and gravel to 3", grayish brown		X		10 23 26			Pass #200, SPT
50				X		10 14 20			Pass #200, SPT
		END OF BORING NO REFUSAL, NO BEDROCK NO CAVING, NO FILL GROUNDWATER ENCOUNTERED AT 37.67'							
55									
60									
65									

10331-3 16113-3.GPJ CHJ.GDT 4/5/16

CHJ

75-ACRE SITE
SOUTH OF AGUA MANSA ROAD, COLTON, CALIFORNIA

Job No. 16113-3

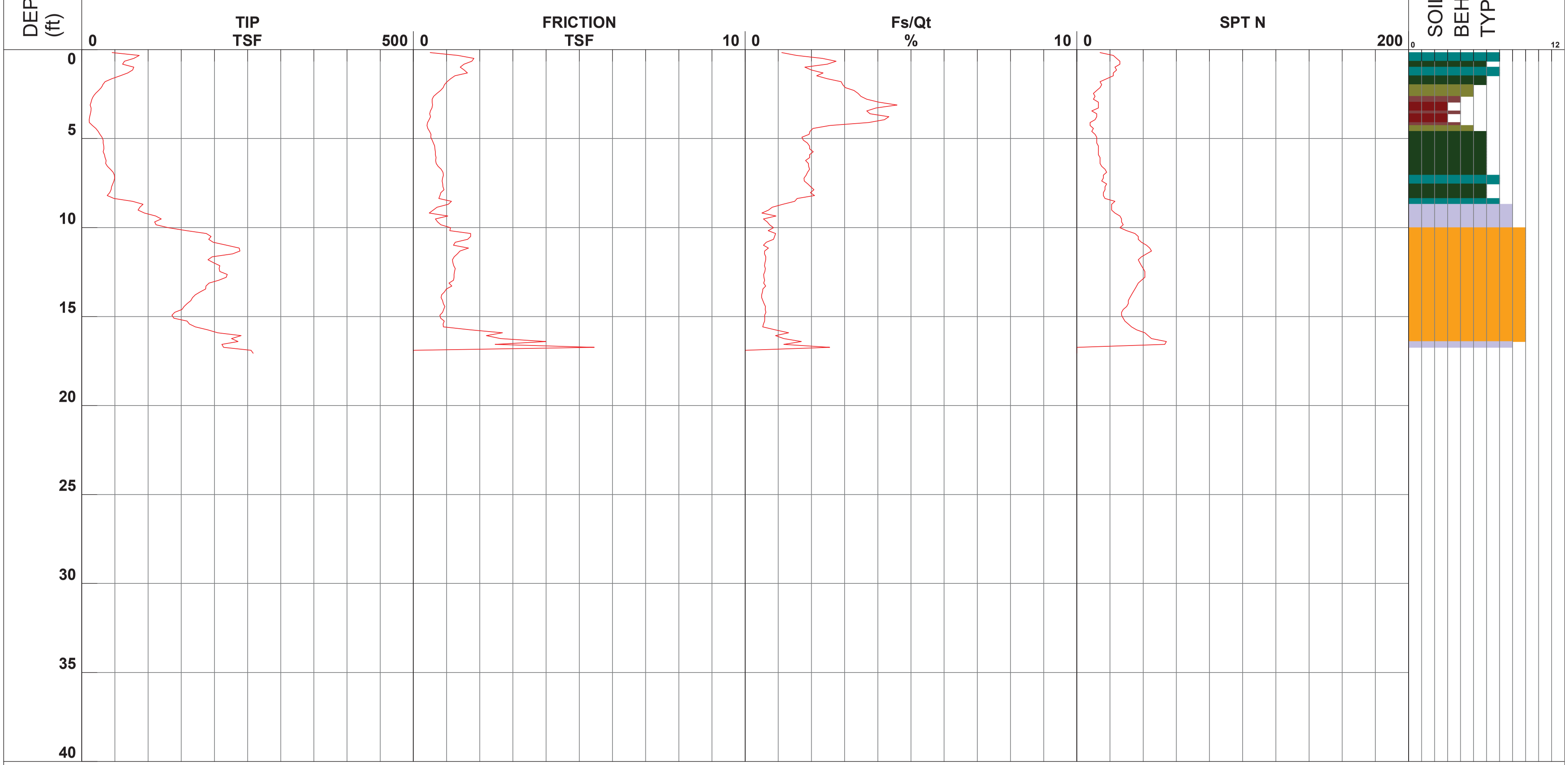
Enclosure B-8b

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(262).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-01	Date and Time	3/21/2016 8:34:34 AM	Maximum Depth	17.06 ft
EST GW Depth During Test	>17.06 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

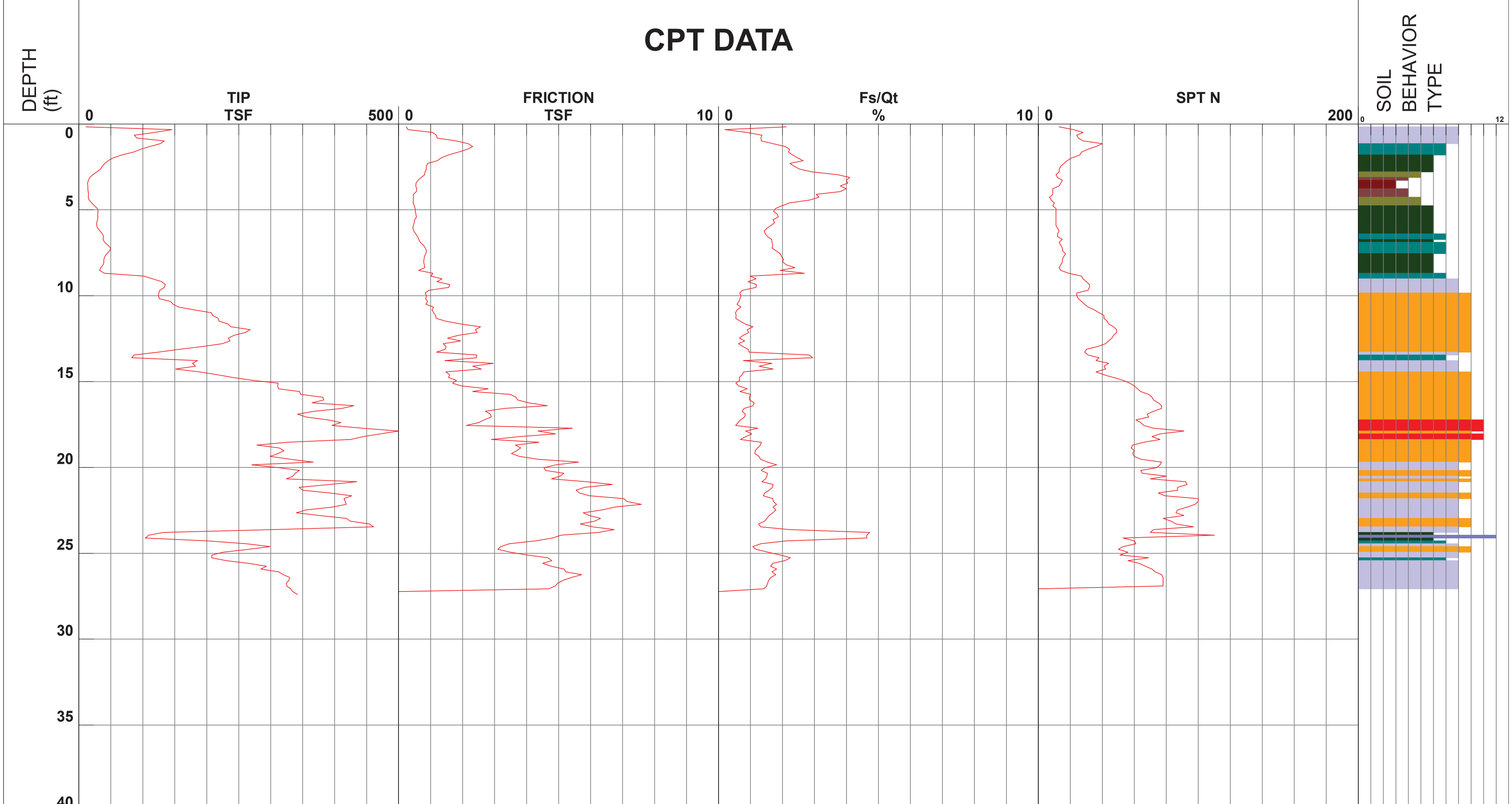
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(263).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-01A	Date and Time	3/21/2016 9:00:43 AM	Maximum Depth	27.39 ft
EST GW Depth During Test		>27.39 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

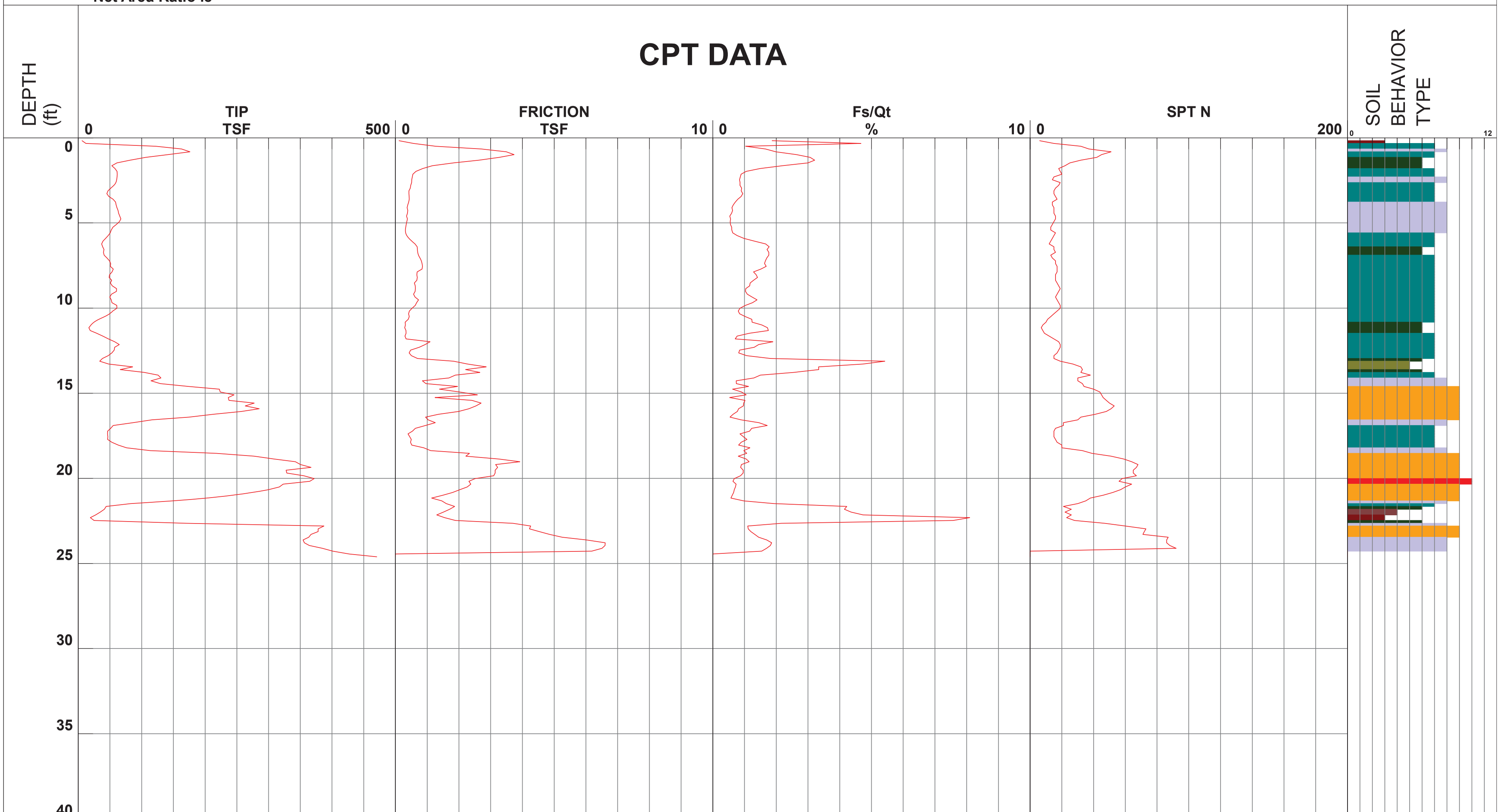
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(267).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-02	Date and Time	3/21/2016 12:12:10 PM	Maximum Depth	24.61 ft
EST GW Depth During Test	>24.61 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

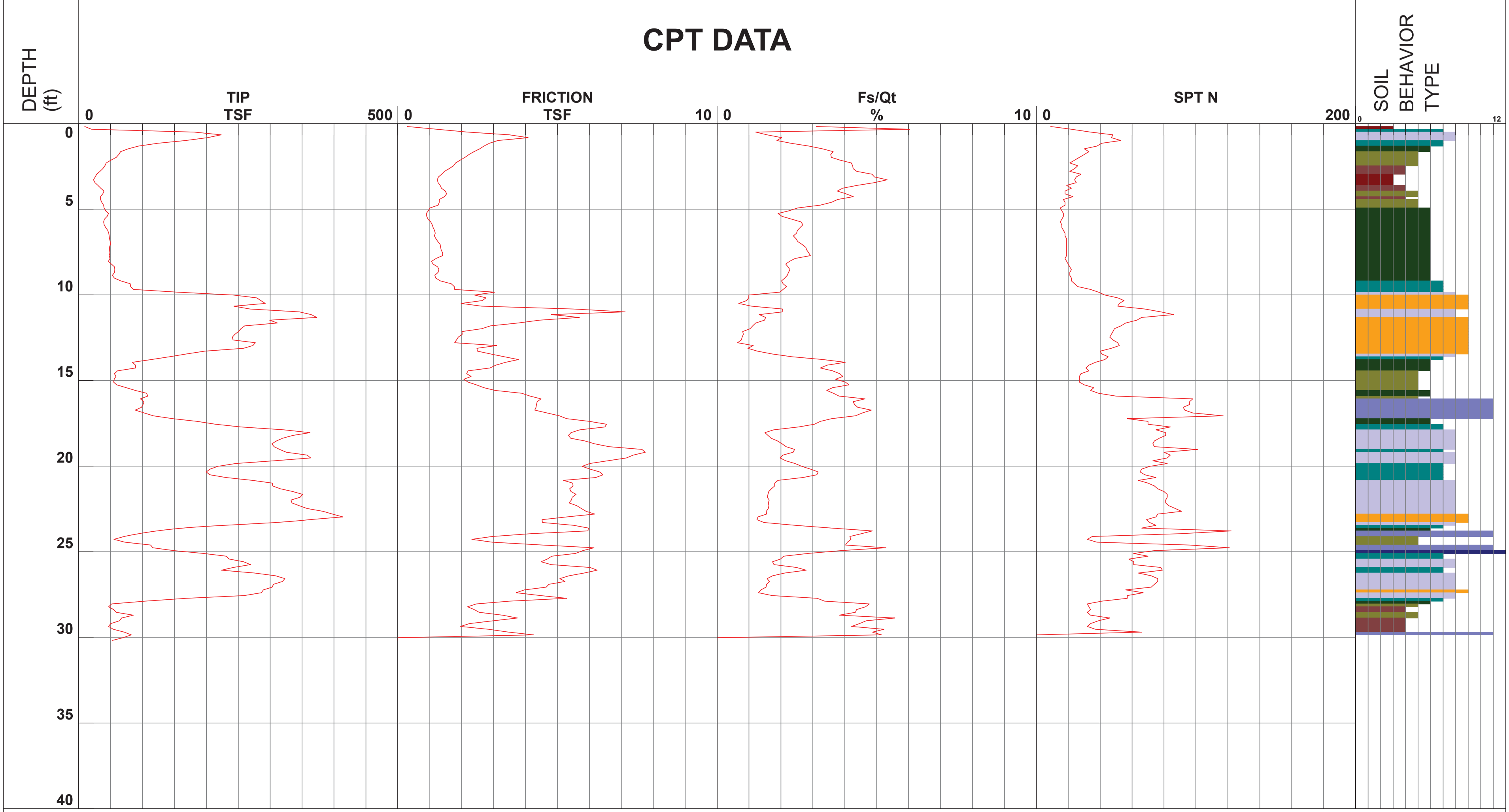
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(264).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-03	Date and Time	3/21/2016 9:52:27 AM	Maximum Depth	30.18 ft
EST GW Depth During Test		30.00 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

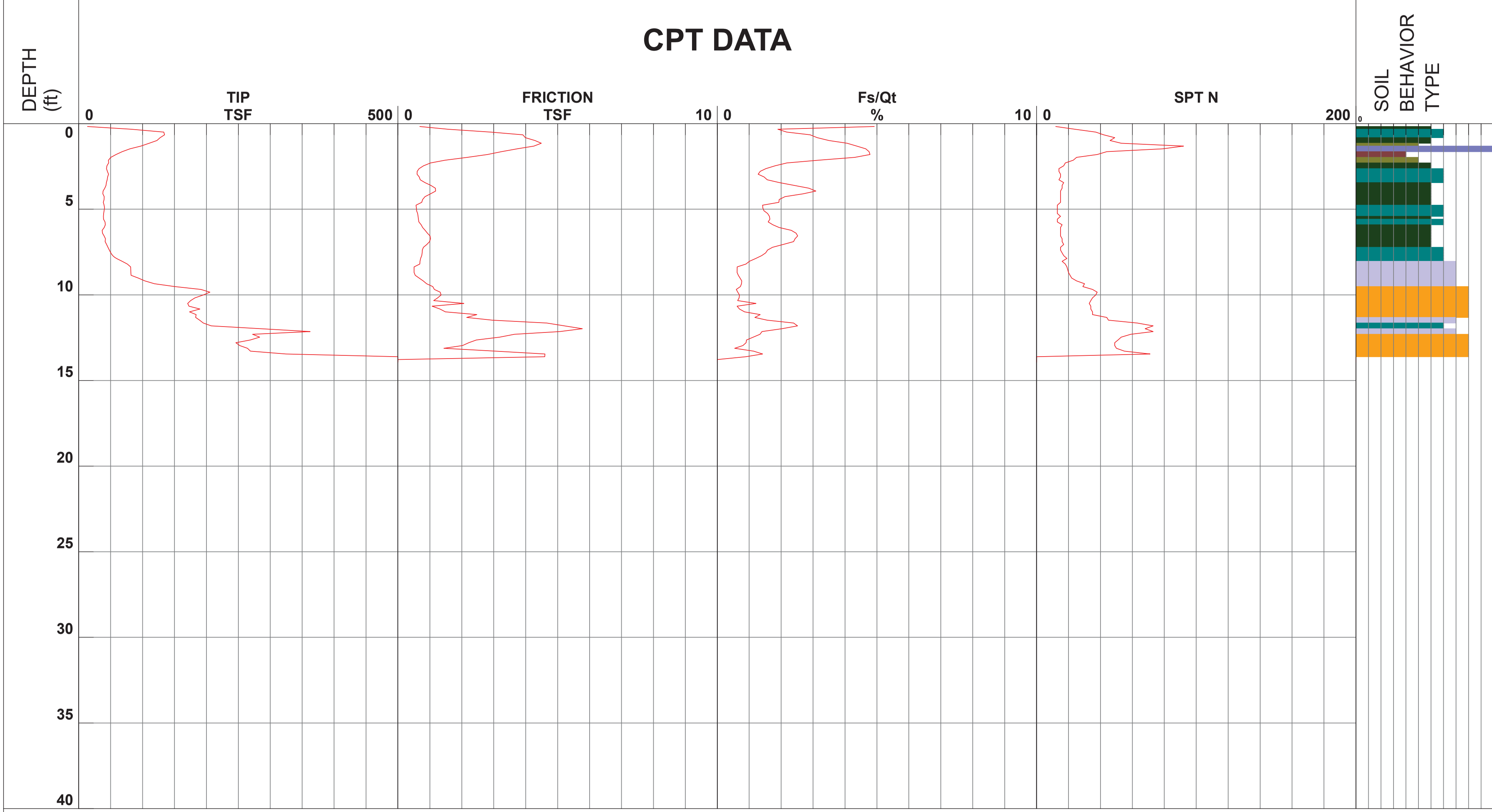
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(265).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-04	Date and Time	3/21/2016 10:28:56 AM	Maximum Depth	13.94 ft
EST GW Depth During Test	>13.94 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

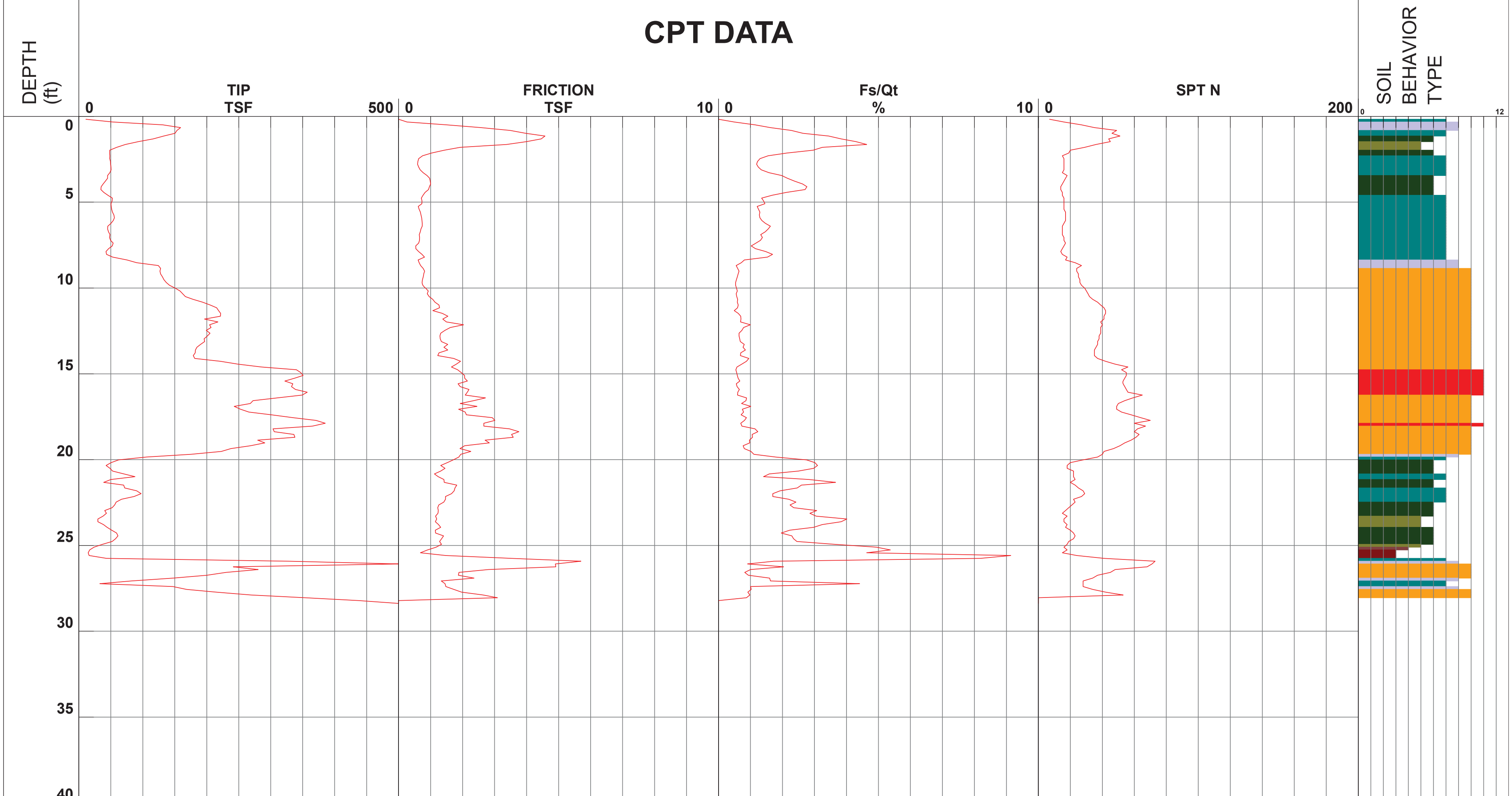
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(266).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-04A	Date and Time	3/21/2016 10:50:44 AM	Maximum Depth	28.38 ft
EST GW Depth During Test	>28.38 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

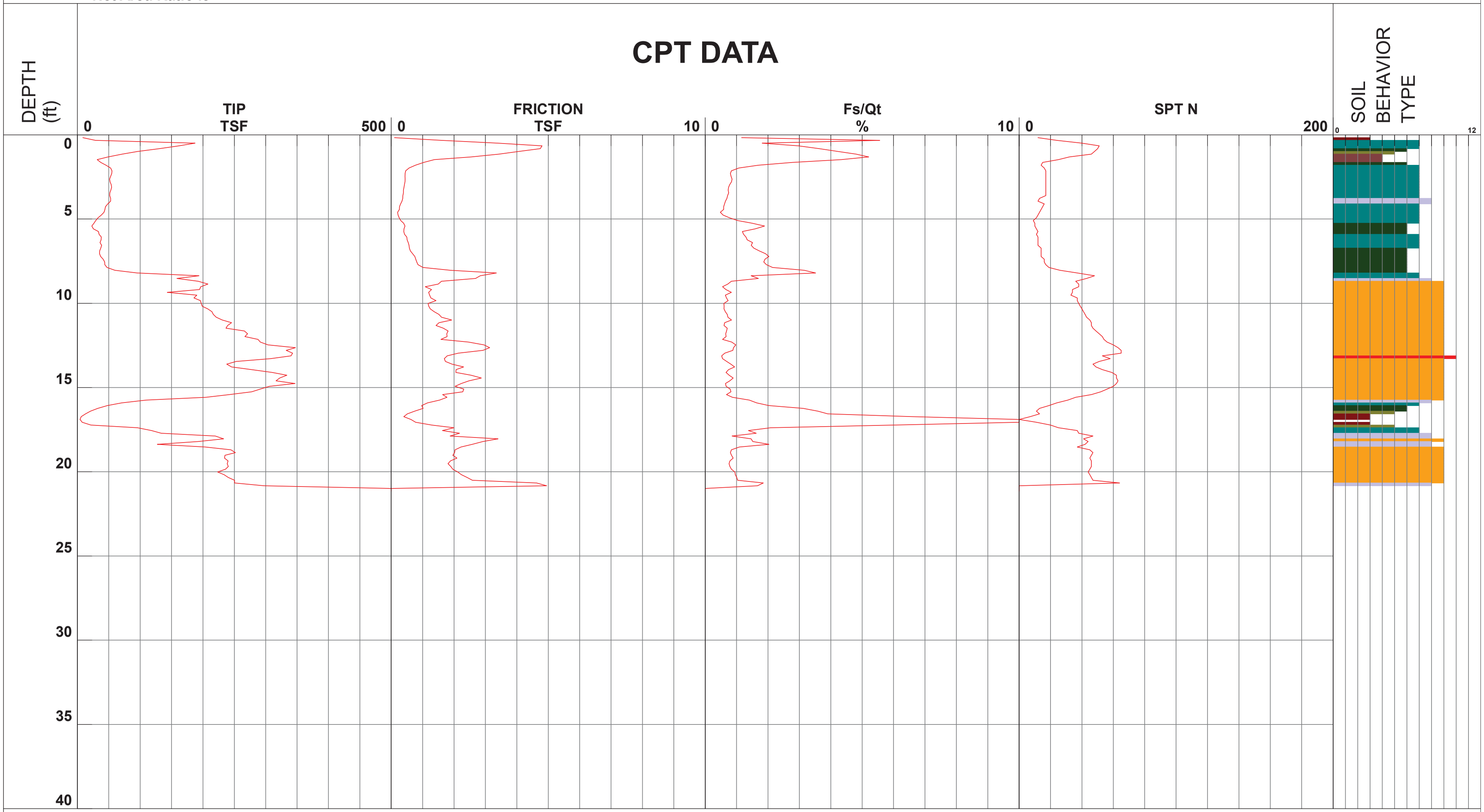
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(269).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-05	Date and Time	3/21/2016 1:17:48 PM	Maximum Depth	21.16 ft
EST GW Depth During Test	>21.16 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

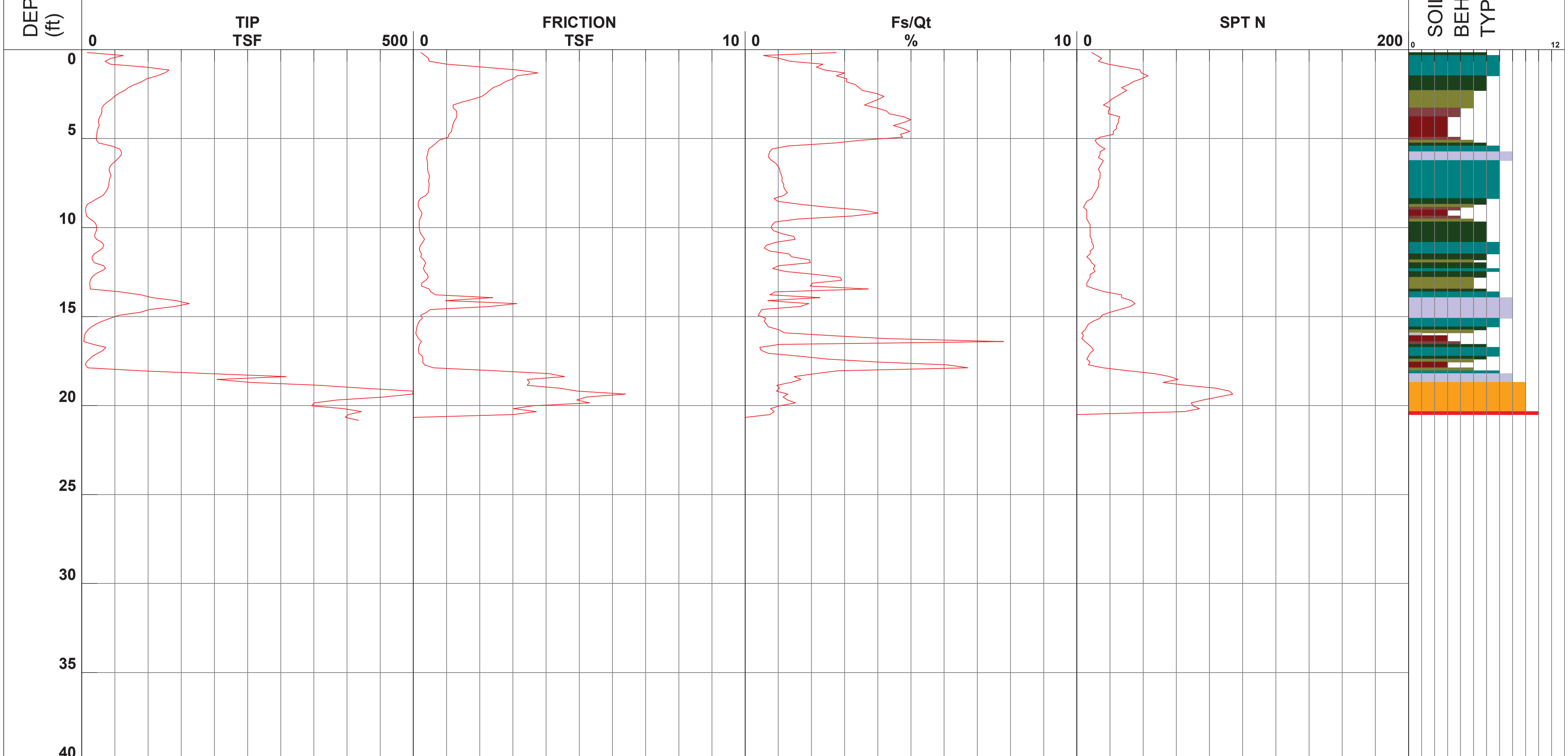
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(268).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-06	Date and Time	3/21/2016 12:43:50 PM	Maximum Depth	20.83 ft
EST GW Depth During Test		>20.83 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

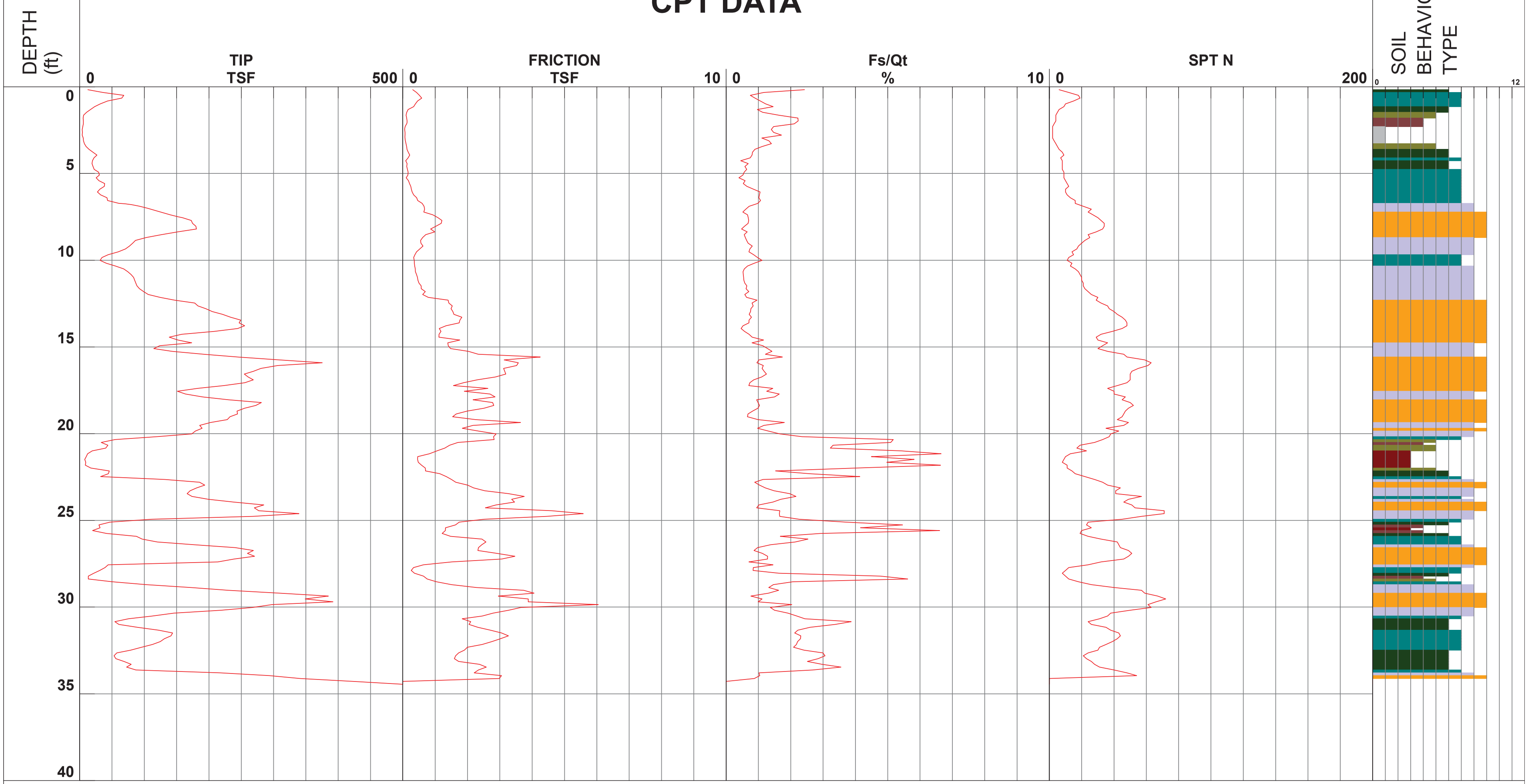
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(270).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-07	Date and Time	3/21/2016 2:09:16 PM	Maximum Depth	34.45 ft
EST GW Depth During Test		30.00 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

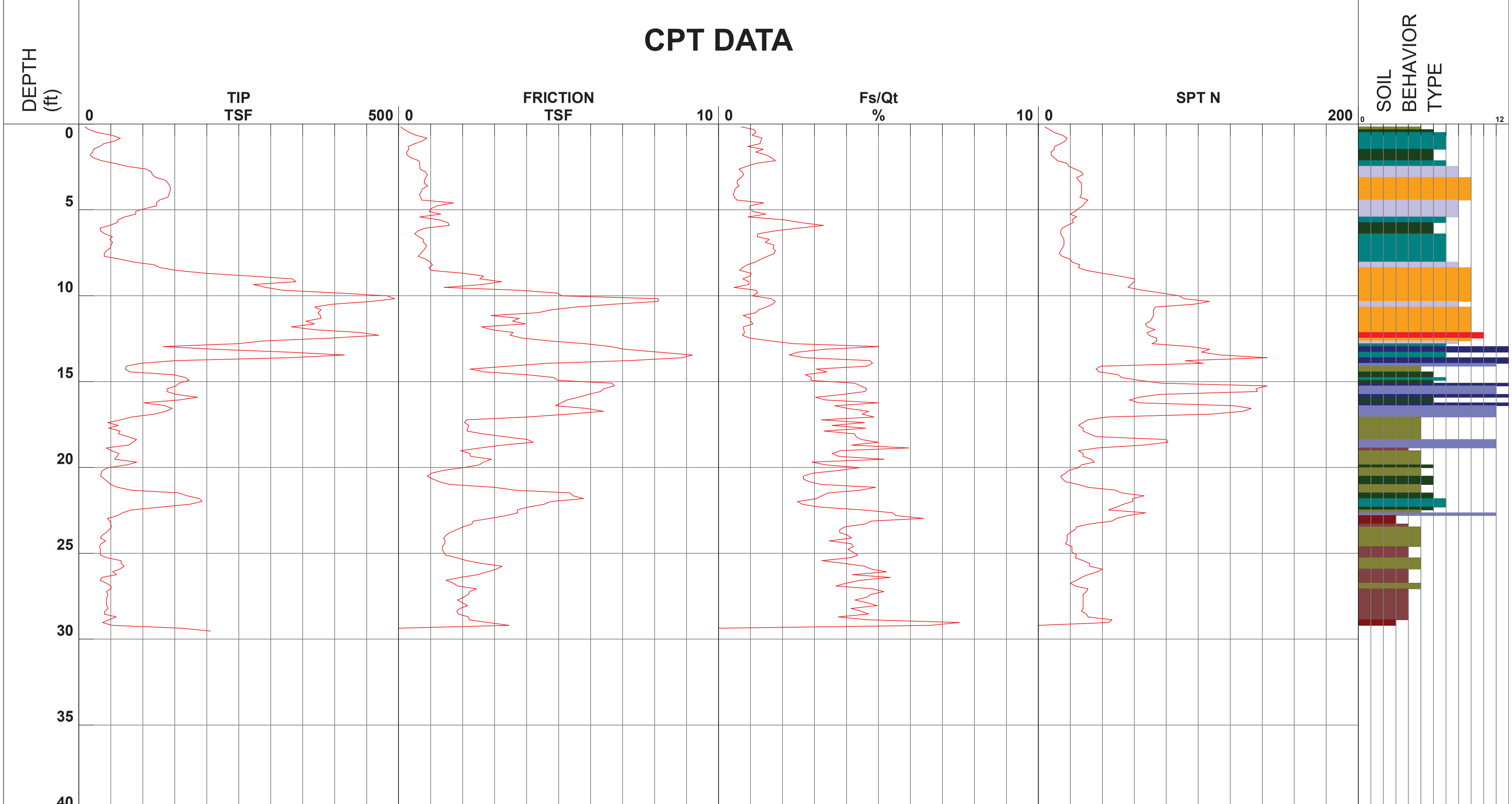
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(272).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-08	Date and Time	3/21/2016 3:31:12 PM	Maximum Depth	29.53 ft
EST GW Depth During Test		>29.53 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

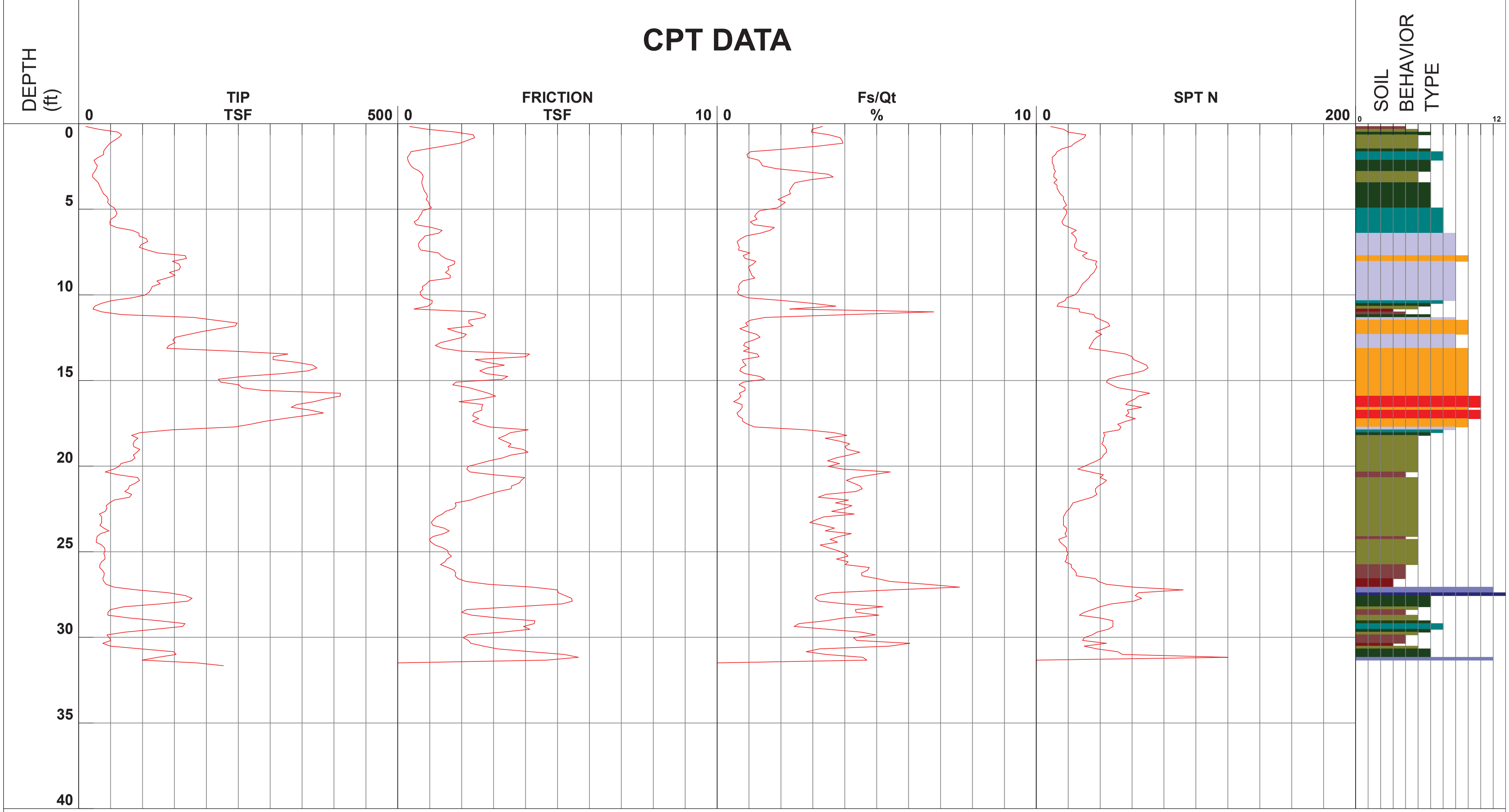
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(273).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-09	Date and Time	3/22/2016 8:06:44 AM	Maximum Depth	31.66 ft
EST GW Depth During Test	30.00 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

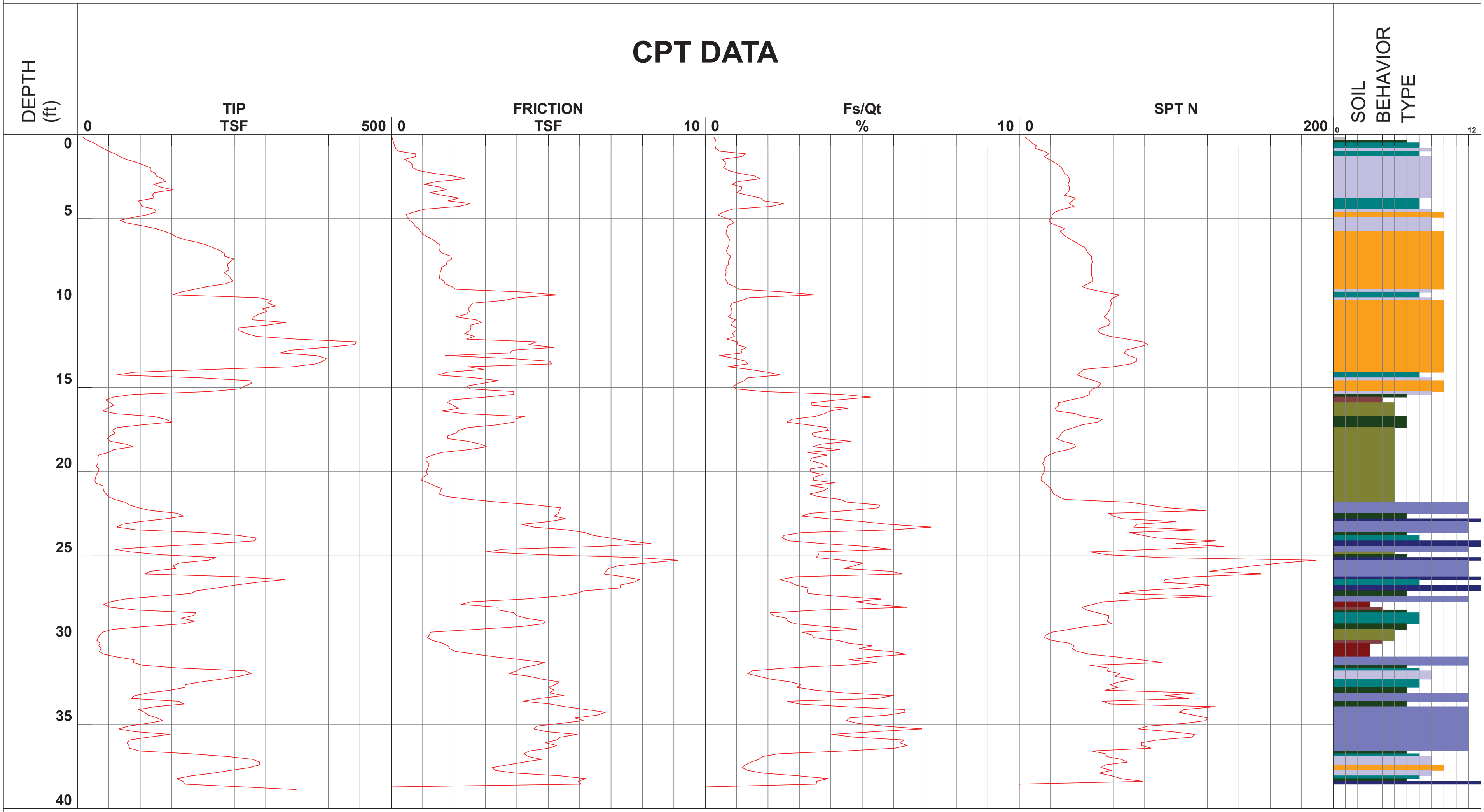
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(274).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-10	Date and Time	3/22/2016 8:44:11 AM	Maximum Depth	38.88 ft
EST GW Depth During Test		30.00 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

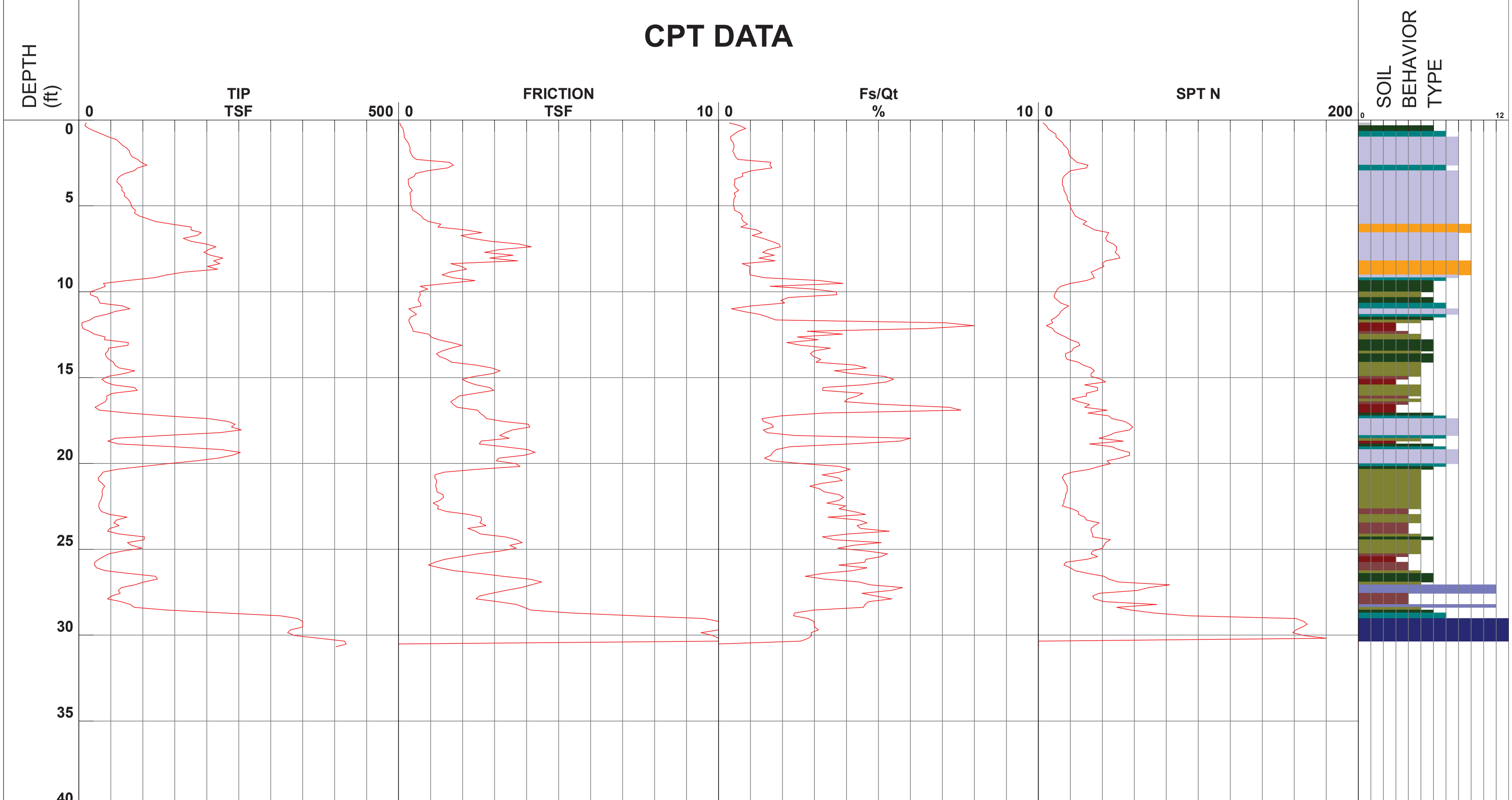
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(283).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-11	Date and Time	3/22/2016 3:45:02 PM	Maximum Depth	30.68 ft
EST GW Depth During Test		30.00 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

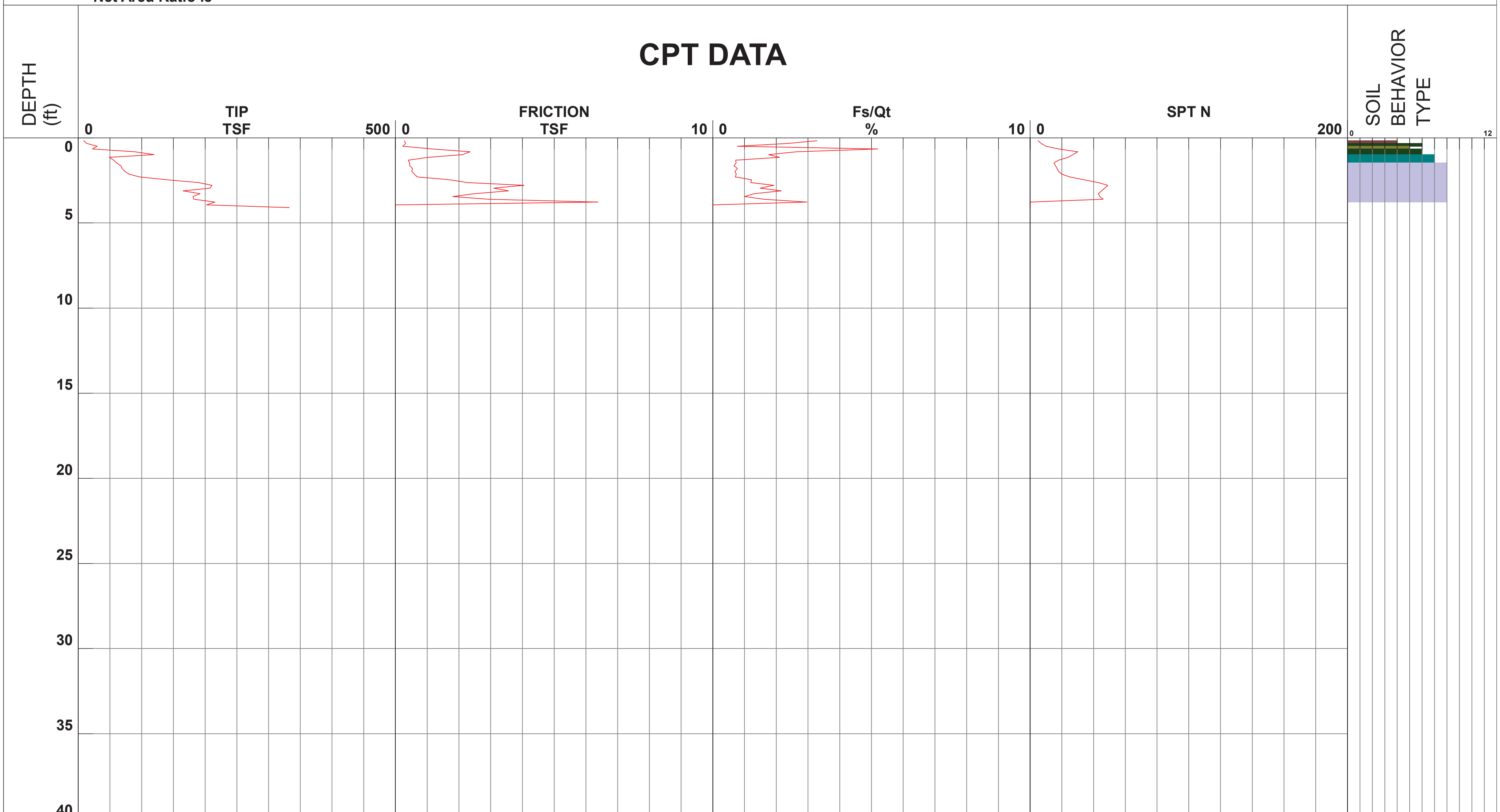
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(281).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-12	Date and Time	3/22/2016 3:04:47 PM	Maximum Depth	4.10 ft
EST GW Depth During Test	>4.10 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 (*)

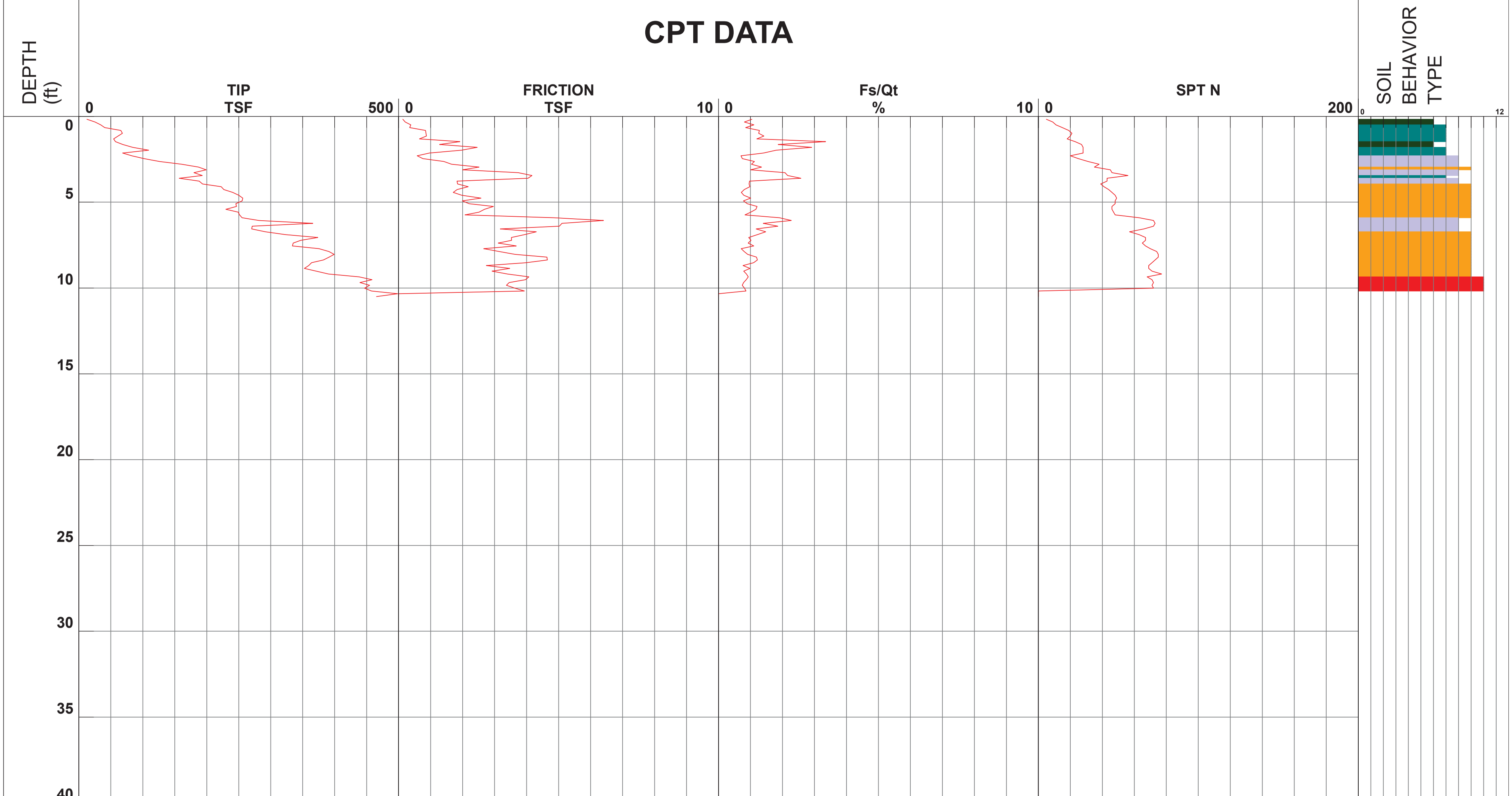
Cone Size 10cm squared

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

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(282).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-12A	Date and Time	3/22/2016 3:13:06 PM	Maximum Depth	10.50 ft
EST GW Depth During Test		>10.50 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

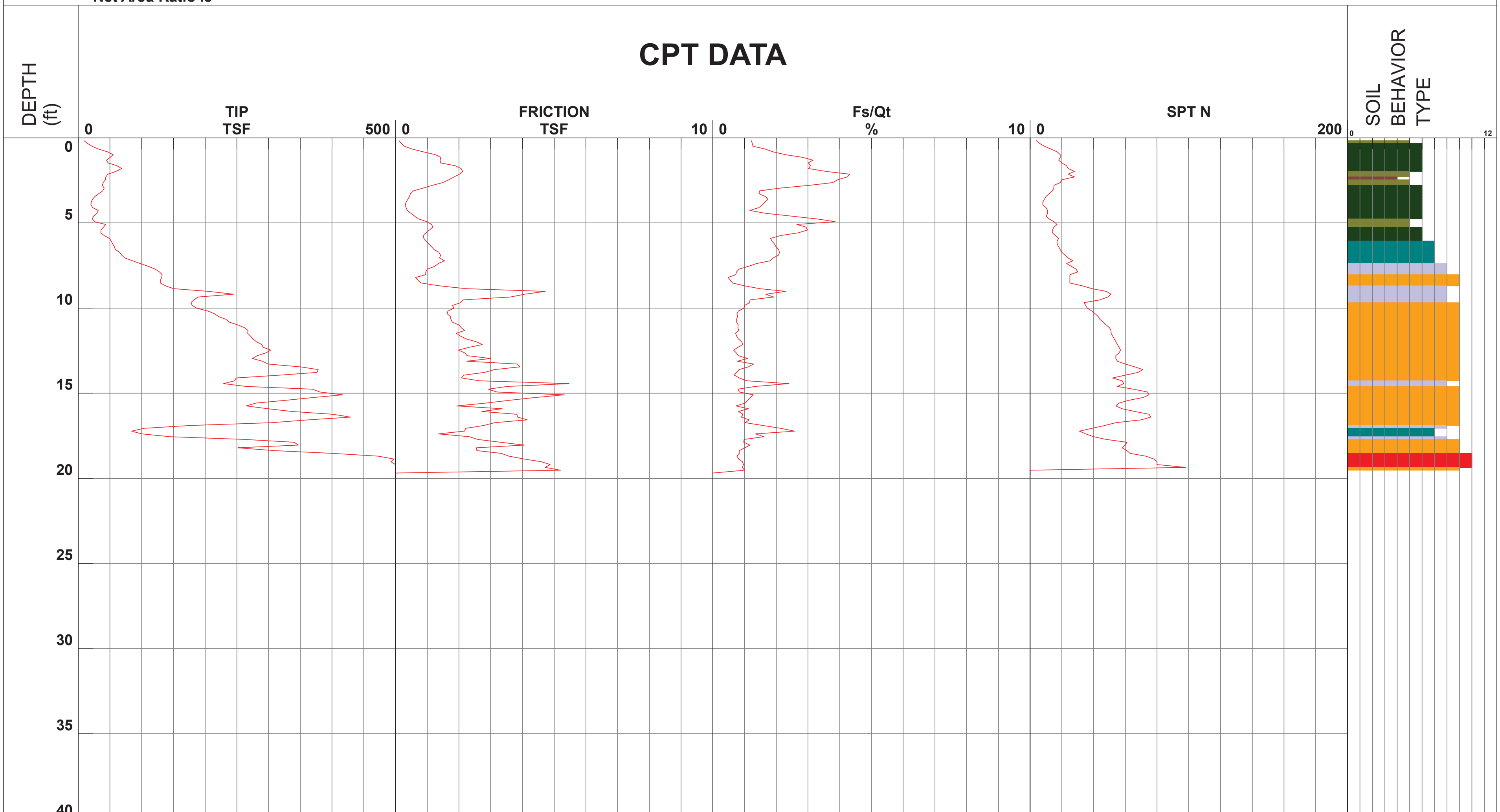
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(275).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-13	Date and Time	3/22/2016 10:08:54 AM	Maximum Depth	19.85 ft
EST GW Depth During Test	>19.85 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

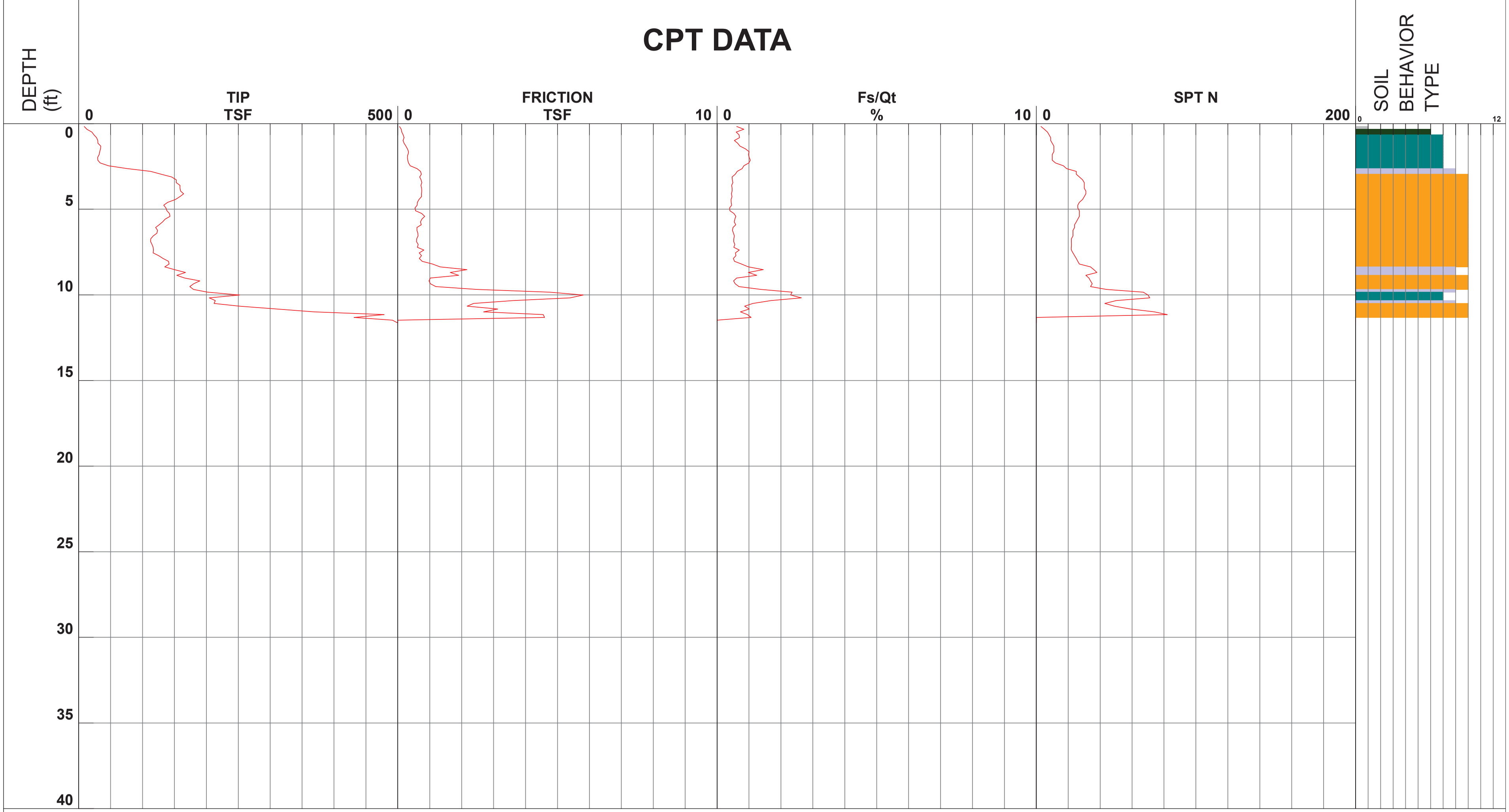
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(279).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-14	Date and Time	3/22/2016 1:53:10 PM	Maximum Depth	11.65 ft
EST GW Depth During Test	>11.65 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

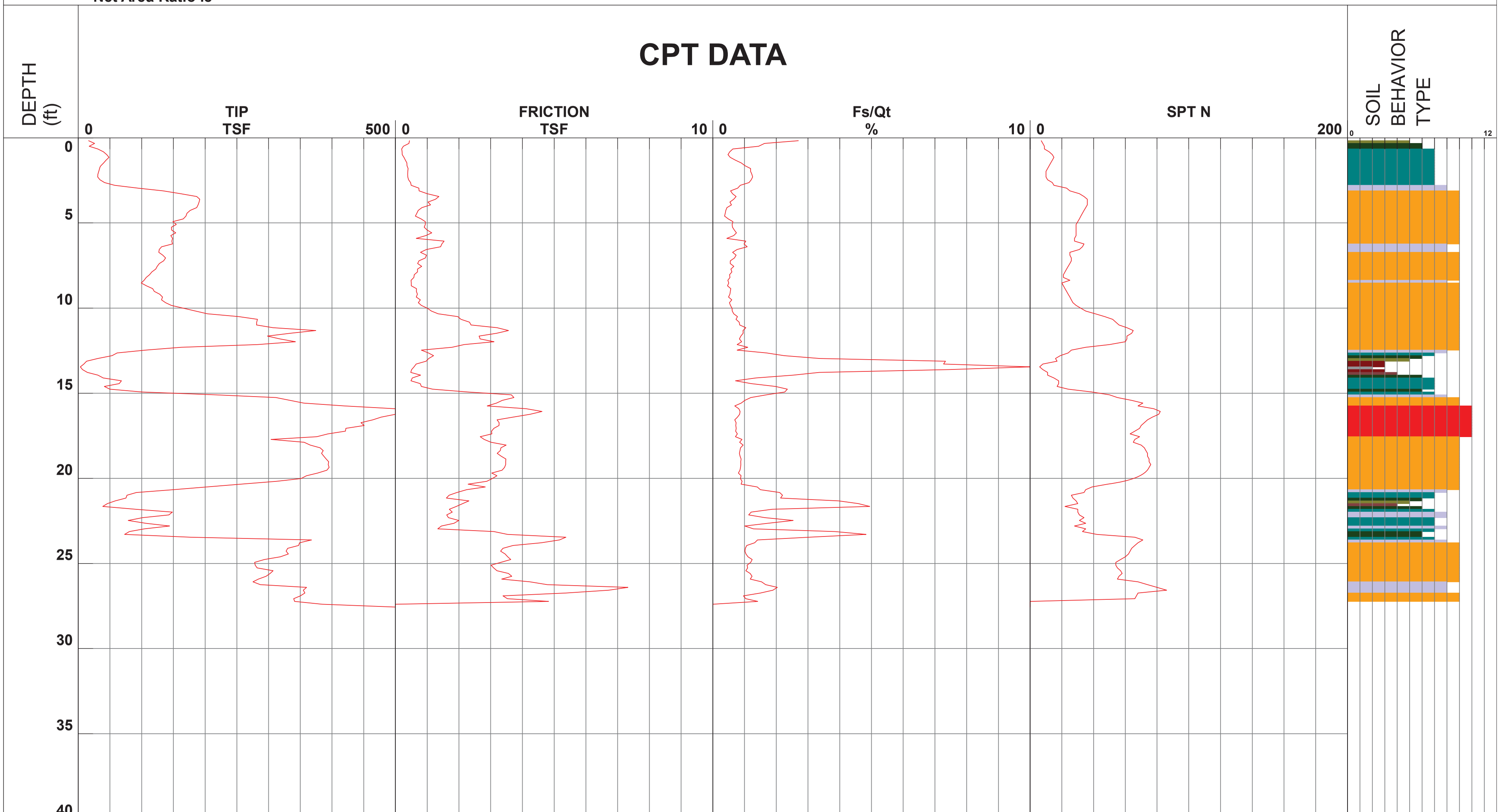
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(280).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-14A	Date and Time	3/22/2016 2:10:03 PM	Maximum Depth	27.56 ft
EST GW Depth During Test		>27.56 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

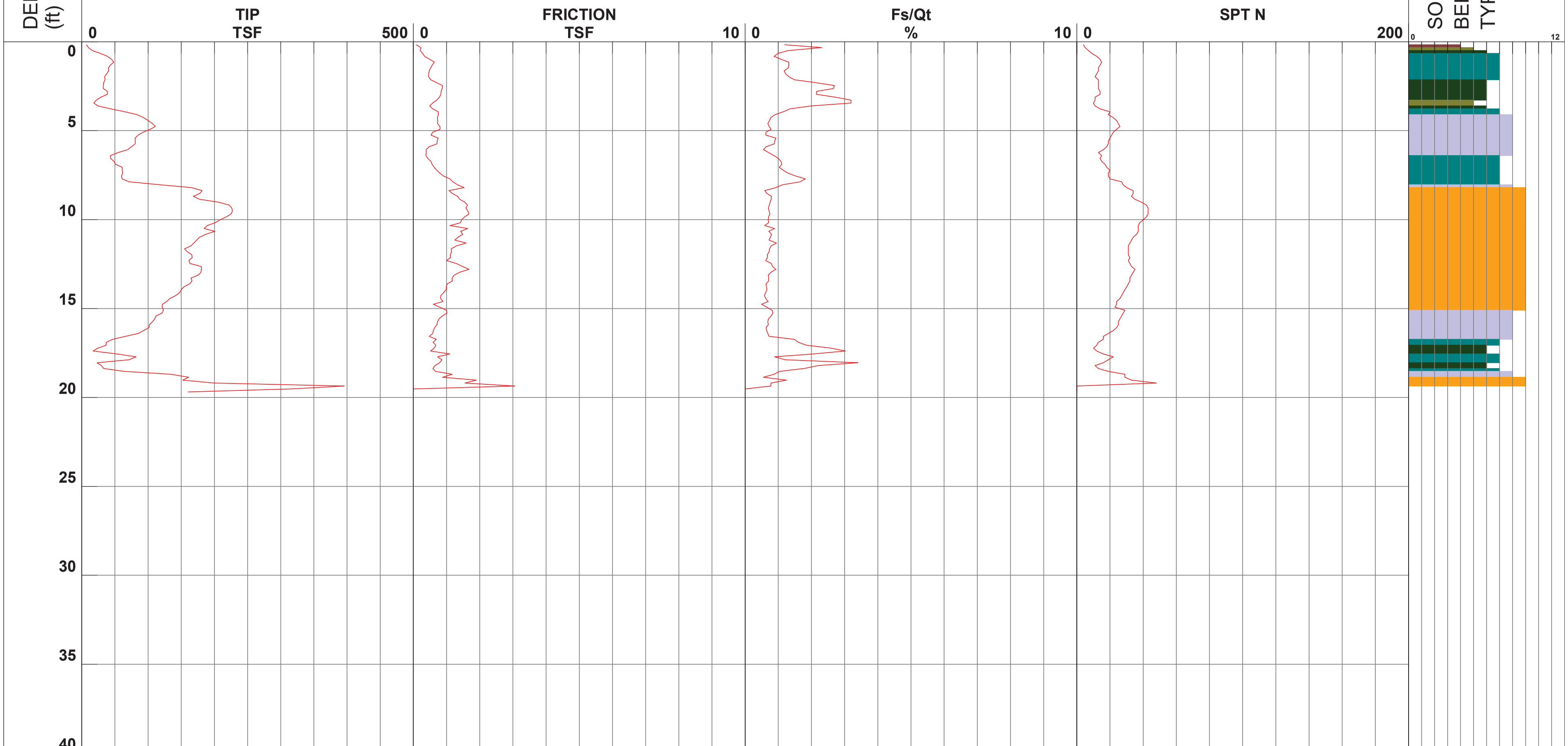
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(277).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-15	Date and Time	3/22/2016 12:26:51 PM	Maximum Depth	19.68 ft
EST GW Depth During Test	>19.68 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

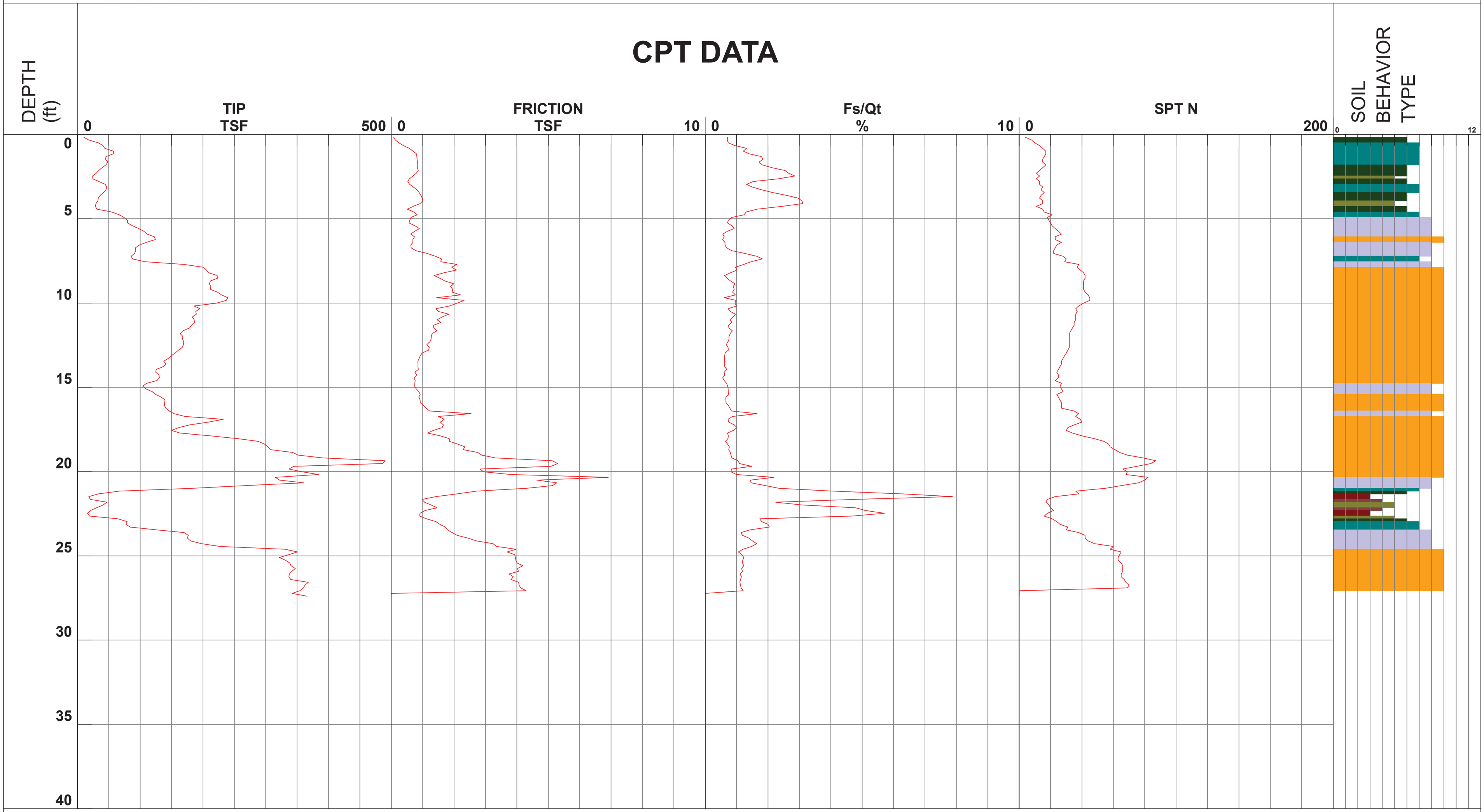
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(278).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-15A	Date and Time	3/22/2016 12:46:56 PM	Maximum Depth	27.39 ft
EST GW Depth During Test		>27.39 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

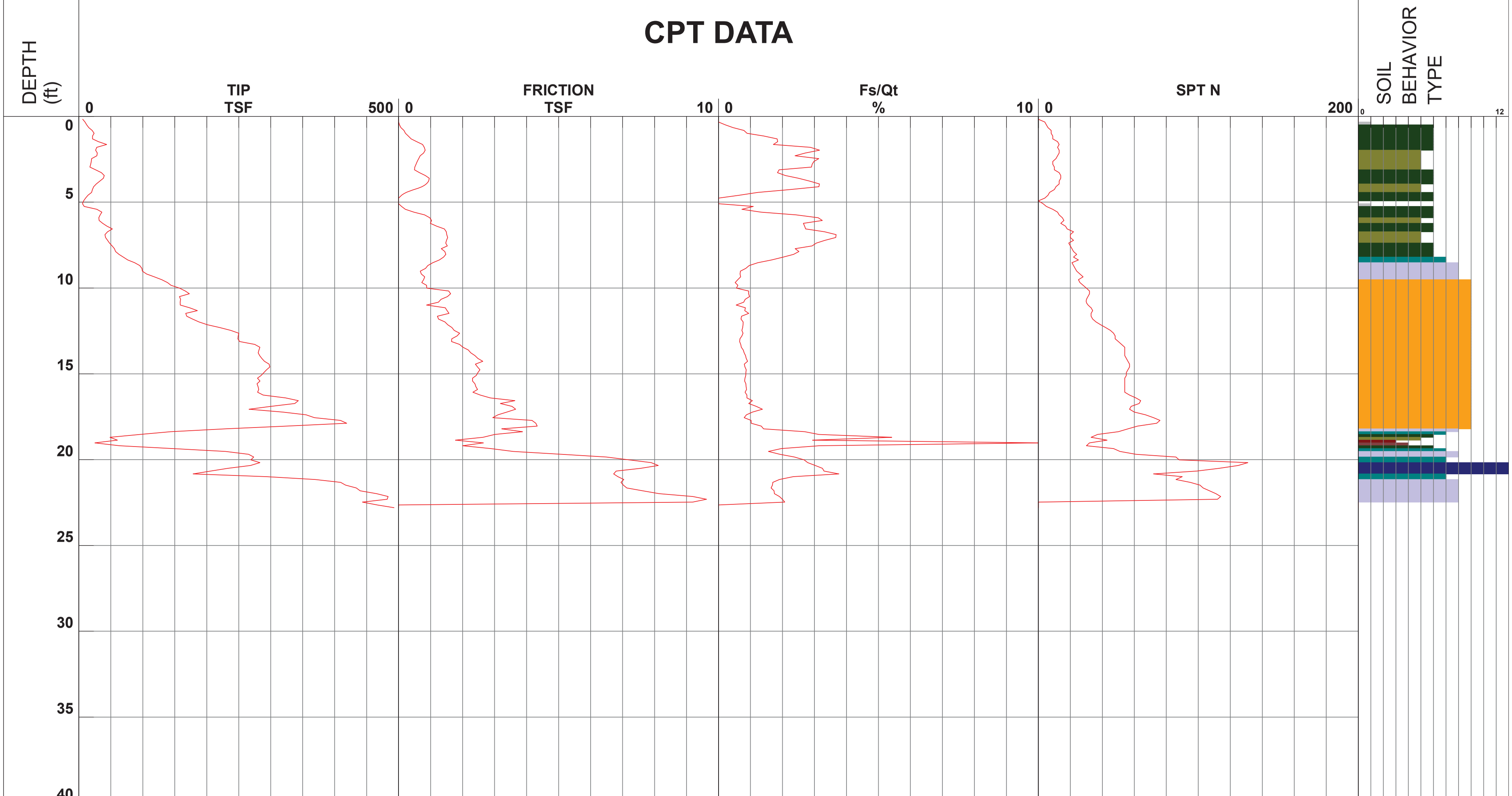
S*Soil behavior type and SPT based on data from UBC-1983

CHJ Inc

Project	Proposed 75-Acre Site	Operator	DG-RC	Filename	SDF(276).cpt
Job Number	16113-3	Cone Number	DDG1268	GPS	
Hole Number	CPT-16	Date and Time	3/22/2016 10:57:17 AM	Maximum Depth	22.80 ft
EST GW Depth During Test		>22.80 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

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