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Appendix 3.7A
Geotechnical Investigation Report

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CONFIDENTAL

June 10, 2016
Kleinfelder Project No. 20164522.001A

Mr. Paul Manzer, PE
PACLAND
11400 SE 8th St, Suite 345
Bellevue, WA 98004

Subject: Geotechnical Investigation Report
PACLAND Project 1926
San Jose, California

Dear Mr. Manzer:

Kleinfelder is pleased to present this report summarizing the results of our geotechnical engineering study for planning and design of the proposed development at the subject project site, located northwest of the Interstate 880 / Highway 237 interchange in San Jose, California.

The primary geotechnical design and construction related risks associated with the project are the presence of clay soils that are moisture sensitive, compressible, and potentially expansive, and potential soil liquefaction due to strong ground shaking from earthquakes. Recommendations to address these issues and other related key geotechnical considerations are contained herein. We anticipate providing additional recommendations and/or refining current recommendations as design of the project progresses. This may include, but not be limited to, addressing the following: a site response analysis to support seismic structural design; refined deep foundation recommendations based on a pile indicator program; and exploration and analyses to support design of storm water outfalls and proposed grading in the vicinity of the levee east of the site.

Kleinfelder appreciates the opportunity to provide continuing geotechnical engineering services on the project. If there are questions about the information presented in this report, please contact us at your convenience.

Respectfully submitted,

KLEINFELDER, INC.

Marcus Byers

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GEOTECHNICAL INVESTIGATION REPORT
PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA
KLEINFELDER PROJECT NO.: 20164522.001A

JUNE 10, 2016

A Report Prepared for:

Mr. Paul Manzer, PE
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GEOTECHNICAL INVESTIGATION REPORT
PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA

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June 10, 2016
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TABLE OF CONTENTS

FIGURES

APPENDIX A - FIELD EXPLORATION

GEOTECHNICAL INVESTIGATION REPORT
PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA

EXECUTIVE SUMMARY

This report summarizes Kleinfelder's geotechnical engineering study performed to support planning, design and construction of the proposed project. As currently planned, we understand the project includes four 90-foot tall, 4-story structures, each with an approximately 250,000 square foot footprint, as well as smaller ancillary structures including a new electrical substation, storm water facilities, and parking areas. We understand the project concept may evolve as design progresses and Kleinfelder should be provided with current site plan, grading, and structure loading information to evaluate whether revisions to our conclusions and recommendations are necessary.

The proposed project is feasible from a geotechnical perspective provided the recommendations presented in this report are incorporated into design and construction. However, since the project is still in the conceptual and planning stage, we anticipate providing additional recommendations and/or refining current recommendations as design of the project progresses. This may include, but not be limited to, addressing the following: a site response analysis to support seismic structural design; refined deep foundation recommendations based on a pile indicator program; and exploration and analyses to support design of storm water outfalls and proposed grading in the vicinity of the levee east of the site.

We prepared a summary of our early findings and preliminary recommendations to support preparation of permit documents. This information was presented in a letter titled "Preliminary Geotechnical Study Results, PACLAND Project 1926" dated April 27, 2016. This Geotechnical Engineering Report supersedes that April 27, 2016 letter.

Field exploration for this geotechnical study included a combination of soil borings and sampling, Cone Penetration Test (CPT) probes, Rho field and laboratory testing, and geotechnical laboratory testing. Appendices A through C present summary exploration logs and laboratory test results.

Based on our explorations, the near surface soils (upper 3 to 5 feet) consist of mostly granular materials including clayey sands, sands and gravels with variable clay content, and some sandy clays. Below these near surface soils, we encountered lean to fat clays extending to depths of about 20 to 25 feet. Deeper soils included increasingly dense/hard interbedded gravels, sands and clays to the full depth of our explorations (about 100 feet).

We estimate groundwater depths ranging from about 7 to 15 feet, through historical data indicates that groundwater may be as shallow as 5 feet. It is unlikely that groundwater will be encountered during site grading since the current plan includes mostly fills and only minor cuts. However, groundwater may be encountered in deeper excavations during construction, such as for utility trenching and grade beam or foundation excavations. Actual groundwater levels vary over time based on precipitation and other factors.

The primary geotechnical design and construction related risks associated with the project are the presence of clay subgrade soils that are moisture sensitive, compressible, and potentially expansive, and potential soil liquefaction due to strong ground shaking from earthquakes. Refer to Section 6, Key Geotechnical Considerations, for additional discussion of these and other items.

The following sections of this report discuss site and subsurface conditions, geologic setting and seismic hazards, design and construction considerations, conclusions and recommendations for earthwork grading, structural foundations, retaining walls, concrete slab on grade, pavement structural section designs, etc.

1 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed PACLAND 1926 Project. The approximately 65-acre site is located northwest of the Interstate 880 / Highway 237 interchange in San Jose, California, as shown on the Vicinity Map, Figure 1, and Site Plan, Figure 2.

1.1 PROJECT DESCRIPTION

While final site development plans have not yet been prepared, we understand site development may include up to four 90-foot tall, 4-story structures, each with an approximately 250,000 square-foot footprint, as well as smaller ancillary structures including a new electrical substation, storm water outfall, and parking. Details of structure loads are not available at this time; however, we understand that column loads for four-story structures may be up to 1200 kips. Site grading is anticipated to include fill placement to raise the current site grade by approximately 3 to 5 feet. Proposed grading contours and general site features are shown on Figure 3. The source and nature of the fill material to be imported to the site are unknown at this time.

1.2 PURPOSE AND SCOPE OF SERVICES

We performed our study in general accordance with our revised proposal dated March 3, 2016. The purpose of our geotechnical engineering study was to explore and evaluate the subsurface conditions at selected locations at the site, and to develop geotechnical conclusions and recommendations to support geotechnical aspects of project design and construction. We prepared a summary of our early findings and preliminary recommendations to support preparation of permit documents. This information was presented in a letter titled "Preliminary Geotechnical Study Results, PACLAND Project 1926" dated April 27, 2016. This Geotechnical Engineering Report supersedes that April 27, 2016 letter.

Kleinfelder's understanding of the project is based on a preliminary site plan (SJC-1000-MP_140, undated), which does not include structure loads or detailed site grading information. We assume that maximum fills of 5 feet and maximum cuts of 3 feet across the site. Our scope of services included the following:

- Field exploration including:
    - One 100-foot mud-rotary boring;
    - Three 50-foot mud-rotary borings;
    - Three 100-foot CPTs;
    - Three 50-foot CPTs;
    - Nine 20-foot CPTs;
    - Nine 20-foot hollow-stem auger borings; and,
    - Fifteen 5- to 10-foot hollow-stem auger borings (for Rho testing).
- Laboratory testing to evaluate relevant geotechnical engineering properties and corrosion potential of the site soils.
- Analyses of the field and laboratory data to develop conclusions and recommendations to guide the geotechnical design and construction of the project.
- Preparing geotechnical deliverables, including preliminary and design level reports.

Our scope of services did not include the assessment of site environmental characteristics, particularly those involving hazardous substances. Detailed site-specific response analysis for seismic design of the proposed structures in accordance with current California Building Code (CBC) provisions is also outside the scope of this current study. Based on the height of the proposed structures, we anticipate these services will be required to complete seismic structural design.

2 FIELD EXPLORATION

Prior to subsurface exploration, Kleinfelder marked exploration locations and notified the Underground Service Alert (USA) to provide utility clearance in the public right-of-way. We prepared a site-specific health and safety plan for the field exploration activities and the plan was discussed with the field crews prior to the start of field exploration work. Figure 2, following the text, presents the approximate exploration locations.

On March 21 and March 25, 2016 fifteen hollow-stem auger borings, labeled RHO-1 through RHO-15, were drilled to depths of about 5 to 10 feet below the existing ground surface for Rho testing purposes. Nine additional hollow-stem auger borings were drilled to depths of about 20 feet below the existing ground surface for soil sampling purposes. Borings were drilled by Exploration Geoservices, Inc. of San Jose, California. A B-53 drill rig using hollow-stem auger techniques was used to drill these borings.

On March 30, March 31, and April 1, 2016, three mud rotary borings, labeled MR-1, MR-3, and MR-4, were drilled to depths of about 50 feet below ground surface. One mud rotary boring, labeled MR-2, was drilled to a depth of about 100 feet below ground surface. Borings were drilled by Technicon Engineering Services, Inc. of Fresno, California. A CME-55 drill rig using hollow-stem auger and mud-rotary techniques was used to drill the borings.

On March 15, April 27, April 28, May 4 and May 5, 2016, fifteen cone penetrometer tests (CPTs) labeled CPT-1 through CPT-15 were advanced using a 25-ton CPT rig to depths of about 20 to 101 feet below the existing ground surface. Pore-pressure dissipation tests were performed at CPT-1, CPT-2, CPT-3 and CPT-6; seismic shear wave velocity measurements were collected at CPT-1, CPT-3 and CPT-6.

Explorations were located in the field by measuring distances from existing landmarks, and should be considered approximate. Horizontal coordinates and elevations of the borings were not surveyed.

Caving and borehole collapse conditions were encountered in some explorations between about 20 and 50 feet below ground surface.

A Kleinfelder professional prepared logs of the borings, visually classified the soils encountered according to the Unified Soil Classification System (presented on Figure A-1 in Appendix A), and obtained samples of the subsurface materials. Soil classifications made in the field from samples and auger cuttings were made in accordance with American Society for Testing and Materials (ASTM) Method D2488. These classifications were re-evaluated in the laboratory after further examination and testing in accordance with ASTM D2487. The undrained shear strengths of cohesive samples were estimated in the field using a hand-held penetrometer instrument. Sample classifications, blow counts, and other related information were recorded on the boring logs. The blow counts listed on the boring logs are raw values and have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency. Correction factors were applied to the raw sampler blow counts to estimate the sample apparent density noted on the boring logs and for engineering analyses.

Keys to the soil descriptions and symbols used on the boring logs are presented on Figures A-1 and A-2 in Appendix A. Logs of the borings are presented on Figures A-3 through A-15. CPT reports are presented in Appendix C.

Below the hand-auger depth, soil samples were collected from the borings at intervals of approximately 2½ feet to a depth of about 10 feet, and at approximate 5 foot depth intervals thereafter. Samples were obtained from the borings at selected depths by driving either a 2.5-inch inside diameter (I.D.) California sampler, or a 1.4-inch I.D. Standard Penetration Test (SPT) sampler driven 18 inches (unless otherwise noted) into undisturbed soil with a 140-pound automatic hammer free-falling a distance of 30 inches. The SPT sampler did not contain liners. The 2.5-inch I.D. California sampler contained stainless steel liners. The California sampler was in general conformance with ASTM D3550. The SPT sampler was in general conformance with ASTM D1586.

The apparent density and consistency terminology used in soil descriptions is based on field observations (see Figure A-2). Relatively undisturbed soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance and returned to Kleinfelder's laboratory for further examination and testing.

After the borings were completed they were backfilled with cement grout as required by State and local requirements. Drilling spoils from the mud rotary borings were contained in 55-gallon drums for subsequent testing and disposal by our subcontractor.

3 LABORATORY TESTING

Kleinfelder performed laboratory tests on selected samples recovered from the borings to evaluate their physical and engineering characteristics. The results of laboratory tests are presented on the summary boring logs in Appendix A and on test data sheets in Appendix B. The following laboratory tests were performed:

- Unit Weight (ASTM D2937)
- Moisture Content (ASTM D2216)
- Sieve Analysis (ASTM D1921)
- R-value (ASTM D2844)
- Plasticity Index (ASTM D4318)
- Triaxial Shear Strength (ASTM D2850)
- Compaction Testing – Modified Proctor (ASTM D1557)
- Rho Testing (ASTM D5334)
- Corrosion - Soluble Sulfate Content (ASTM D4327)
- Corrosion - Soluble Chloride Content (ASTM D4327)
- Corrosion - pH (ASTM D4972)
- Corrosion - Minimum Resistivity (ASTM G57)
- Corrosion - Redox (ASTM D1498)
- Corrosion - Sulfide (ASTM D4658)

Note that Kleinfelder does not practice corrosion engineering. The corrosivity test results in Appendix B are provided for information only, and should be evaluated by a qualified corrosion design engineer.

4 GEOLOGY AND SEISMICITY

4.1 REGIONAL GEOLOGY

The site is located on the eastern San Francisco Bay margin within the Coast Range geomorphic province of Northern California. This province is generally characterized by northwest trending mountain ranges and intervening valleys, which are a reflection of the dominant northwest structural trend of the bedrock formations and earthquake faults in the region. The regional area has undergone a complex geologic history of sedimentation, volcanism, folding, faulting, uplift and erosion, which continues to the present day, and has resulted in fractured and discontinuous stratigraphic sequences.

4.2 SITE GEOLOGY

The project site is located within an agricultural area in San Jose, California near the Coyote Creek and Los Esteros Electrical Power Substation. According to published geologic maps in the area, and our soil boring logs and previous CPT data for the adjacent electrical substation, the site is underlain by undifferentiated deposits of alluvium and marine deposits (locally referred to as Bay Muds). The alluvial deposits originated from the East Bay Hills, located a few miles to the east, and are generally composed of poorly consolidated and interlayered clays, silts, sands, and gravels. These soils may have been deposited by one of the several streams in the general area. The underlying older marine deposits consist primarily of clay and fine sand. This is generally consistent with soil encountered borings performed for this study.

4.3 FAULTING AND SEISMICITY

The site is located within the seismically active San Francisco Bay Area coastal region of California, and is subject to seismically-induced ground shaking from nearby and distant faults. Several faults have been mapped in the general site vicinity. The San Andreas fault zone is the boundary between two tectonic plates, the Pacific plate (west of the fault) and the North American plate (east of the fault). In the North Coast region of California, this movement is distributed across a complex system of predominantly strike-slip, right-lateral, parallel, and sub-parallel faults that include the San Andreas, Hayward, Calaveras and the Healdsburg/Rodgers Creek, among others.

Based on the information provided in Bryant and Hart (2007) and CGS (2000), the site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone where site-specific studies addressing the potential for surface fault rupture are required, and no known active faults traverse the site. Furthermore, the site is not located within a City or County designated hazard zone. At its closest point, the Hayward Fault is located approximately 2½ miles northeast of the site, the Calaveras Fault is located approximately 6½ miles northeast of the site, and the San Andreas Fault is located approximately 15 miles southwest of the site.

Moderate to major earthquakes generated on the San Andreas or the Hayward faults can be expected to cause strong ground shaking at the site. In addition, strong ground shaking can be expected from moderate to major earthquakes generated on other faults in the region.

4.4 GEOLOGIC HAZARDS

Geologic hazards reviewed for the project site include seismic shaking, flooding, liquefaction and lateral spreading. Conclusions regarding these hazards are provided below.

4.4.1 Seismic Shaking

Historically, the project site has been subject to intense seismic activity. The site will likely be subjected to at least one moderate to severe earthquake and associated seismic shaking and some ground settlement during the project lifetime. Some degree of structural damage due to strong seismic shaking should be expected. According to the recent Uniform California Earthquake Rupture Forecast, Version 3 by the Working Group on the California Earthquake Probabilities (Field, 2014), the likelihood of having earthquakes with magnitude greater than 6.7 in the San Francisco region is greater than 99% in the next 30 years (starting from 2014).

4.4.2 Liquefaction

It is generally accepted that the following four conditions must exist in order for liquefaction and possible associated effects from ground shaking to occur:

- It is generally accepted that the following four conditions must exist in order for liquefaction and possible associated effects from ground shaking to occur:
- The subsurface soils are in a relatively loose state
- The soils are saturated

- The soils have low plasticity
- Ground shaking is of sufficient intensity and duration to act as a triggering mechanism

After soil liquefies, dissipation of the excess pore pressures can produce volume changes within the liquefied soil layer, which can result in ground surface settlement that can be very detrimental to building structures and other infrastructure features.

Liquefaction analyses performed during our investigation program relied upon data from the CPTs and rotary wash borings that were performed at the project site. Blow counts recorded below the water table in hollow-stem auger borings are deemed unreliable for liquefaction analyses, and so were not used for those purposes.

In the past decade, several concentrated efforts have been undertaken to establish a uniform guideline for field-based simplified liquefaction analyses. Youd et al. (2001) published general guidelines for liquefaction analyses, which presented consensus of a task force committee. However, subsequent earthquakes provided additional data to researchers, especially for low plasticity clays and silts, which resulted in significant modifications to liquefaction evaluation methods, especially for soils with higher fines contents. Two of the most widely used new methods for SPT data have been presented by Cetin et al. (2004) and Idriss and Boulanger (2008). Liquefaction triggering analyses were performed using the methods proposed by Cetin et al. (2004) and Idriss and Boulanger (2008) and using the information obtained from the rotary wash borings advanced for the geotechnical investigation. Similarly, using the information obtained from the CPTs advanced for the geotechnical investigation, liquefaction triggering analyses were performed using the methods proposed by Moss et al. (2006) and Idriss and Boulanger (2008).

In order to perform liquefaction analyses, estimates of earthquake magnitude and peak ground acceleration (PGA_M) are needed. Using the USGS interactive deaggregation website (https://geohazards.usgs.gov/deaggint/2008/), the earthquake magnitude Mw = 6.9 was estimated and used in the liquefaction analysis. The PGA_M value for our analysis was calculated based on Equation 11.8-1 in Section 11.8.3 of the American Society of Civil Engineers (ASCE) 7-10. The PGA_M value for the site was calculated using US Seismic Design Maps application (http://earthquake.usgs.gov/designmaps/us/application.php) assuming a Site Class D. The calculated PGA_M value used in the liquefaction analysis is 0.584 g.

The evaluation of liquefaction in response to an earthquake is based on a comparison of a soil's resistance to liquefaction and the cyclic load or demand placed on the soil by the earthquake. A safety factor against liquefaction is commonly defined as the ratio of the cyclic shear stress required to cause liquefaction (cyclic resistance ratio, or CRR) to the equivalent cyclic shear stress induced by the earthquake (cyclic stress ratio, or CSR). Per our analysis methods, if the calculated safety factor against liquefaction (i.e., the ratio CRR/CSR) is less than 1.0 the soil is considered to be liquefiable for design purposes. Liquefaction-induced settlements were calculated using Cetin et al. (2009) procedure for the Cetin et al. (2004) and the Moss et.al (2006) methods, and using the Idriss and Boulanger (2008) methods.

Groundwater levels encountered in the exploratory borings ranged from about 12 feet to 22 feet and CPT dissipation test data indicated groundwater as shallow as 7 feet. We used a design groundwater depth of 5 feet in our calculations based on our review of historic groundwater data from adjacent Department of Water Resources wells.

Summary of liquefaction analysis results are presented in Table 4.4.2-1. The results are presented for each exploration location (where the boring/CPT was extended to at least 50 feet depth) from the current investigation. Liquefiable layers, identified from our screening analysis, are presented as depth intervals approximated from the existing ground surface. Corresponding calculated magnitudes of settlement for each exploration location are also presented. In calculating total liquefaction-induced settlements, we only considered top 45 to 50 feet of soils as deeper liquefaction would not manifest on the ground surface. Since the liquefaction is not consistent across the site, estimate of differential settlements could be taken as one-half the total settlement between adjacent supports from the nearest boring and/or CPT.

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Table 4.4.2-1

Summary of Liquefaction-Induced Settlement Analyses

Boring / CPT I.D.	Depths of Liquefiable Layer (ft.)	Primary Soil Type	$(N_1)_{60\text{-}cs}$ or $(q_{c1N})_{cs}$ [1]	$(N_1)_{60\text{-}cs\ for\ Sr}$ or $(q_{c1N})_{cs\ for\ Sr}$ [1]	Calculated Liquefaction-Induced Total Settlement (in.) [2]
MR-1	5 – 7.5	Silty Sand	26	23	2.5
	20 - 25	Sandy Silt	12	11	
MR-2	26 – 30	Poorly-Graded Sand	13	12	4.0
	35 – 40	Well-Graded Gravel with Silt and Sand	19	19	
	49.5 - 51	Clayey Gravel	10	8	
MR-3	40.5 – 41.5	Poorly-Graded Sand with Clay	13	12	< 0.5
MR-4	10 – 12.5	Clayey Sand	15	12	5.0
	12.5 – 15	Clayey Sand	13	9	
	20 – 23	Silty Sand	15	13	
	23 – 27.5	Silty Sand	17	14	
	48 - 50	Poorly-Graded Sand with Clay	11	11	
CPT-1	5 – 6	Silty Sand to Sandy Silt	88	107	6.0
	22 – 36	Silty Sand to Sandy Silt	83	103	
CPT-2	21 – 25	Silty Sand to Sandy Silt	96	108	2.0
CPT-3	31 – 33	Clean Sand to Silty Sand	106	117	1.0
CPT-4	5 – 6	Clean Sand to Silty Sand	85	96	7.5
	7.5 – 11.5	Clean Sand to Silty Sand	83	88	
	23.5 – 27.5	Clean Sand to Silty Sand	122	130	
	34 – 41.5	Silty Sand to Sandy Silt	111	124	
CPT-5	nl	nl	nl	nl	nl
CPT-6	5 – 7.5	Silty Sand to Sandy Silt	79	92	5.0
	12 – 14.5	Silty Sand to Sandy Silt	72	97	
	21 – 22.5	Silty Sand to Sandy Silt	66	85	
	26.5 – 30	Clean Sand to Silty Sand	107	112	

Note:
[1]Values are primarily based on the Idriss and Boulanger (2008) approach.
[2]Calculated settlements are primarily based on the Idriss and Boulanger (2008) approach.
nl = no liquefaction

Based on the results of our analysis, the potential of earthquake-induced liquefaction is significant in the eastern area of the project site. In general, loose to medium dense granular sandy soil deposits between depths of approximately 20 and 35 feet and between depths of approximately 60 to 75 feet are likely to liquefy during a design seismic event. Calculated settlements due to liquefiable soils between depths of approximately 60 to 75 feet are not likely to manifest at the ground surface. However, pile foundations should not be designed to tip in this liquefiable layer. The potential for earthquake-induced liquefaction is insignificant in the western area of the project site except in the vicinity of CPT-3 where up to one inch of settlement could be anticipated due to a layer of clean/silty sand about 2 feet thick which is likely to liquefy during a design seismic event.

4.4.3 Landslides and Seismically-Induced Slope Failures

The site is relatively flat and is not susceptible to landslides and seismically-induced slope failures.

4.4.4 Liquefaction-Induced Lateral Spreading

Liquefaction-induced lateral spreading is defined as the mostly horizontal movement of sloping ground (less than 5% surface slope) due to elevated pore pressures or liquefaction in underlying, saturated soils. Structures at the head of the slide are sometimes pulled apart while those at the toe are subjected to buckling or compression of the foundation soil. Linear infrastructure, such as utility lines and roadways, is particularly susceptible to damage in earthquake from lateral spreads at multiple locations (A. F. Rauch, 1997). Lateral spreading movements typically are greatest near a free-face (such as the bridge abutment embankment slope) and diminish with distance from the free-face.

According to our scope of services, we performed a preliminary screening investigation of lateral spreading potential at the project site. We utilized the Google Earth to determine the approximate ground elevation relief including in the vicinity of adjacent Coyote Creek. It is our finding that the elevation difference between the existing grade of the site and the creek bottom is about 7 feet in the southeast area and about 9 feet in the east and northeast area. Therefore, the height of the free-face slope ranges from about 7 feet in the southeast area to 9 feet in the east and northeast area along Coyote Creek. Our liquefaction triggering analyses as discussed above indicated that the occurrence of liquefaction is insignificant in the upper 20 feet (i.e., roughly 2 times the slope height) in the east and northeast area. However, the occurrence of liquefaction is significant in the upper 15 feet (i.e., roughly 2 times the slope height) in the southeast area. Therefore, based on the review of the topography at the project site using Google Earth and the results of our liquefaction triggering analysis, it is our opinion that the lateral spreading potential is low in the east and northeast area along the creek. The lateral spreading potential is high in the southeast area along the creek and a more comprehensive quantitative evaluation should be conducted.

4.4.5 Flooding

The Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map for the site effective February 19, 2014, indicates the site is located within Zone X. Zone X is described by FEMA as *areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood*. The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. No base flood elevation is indicated for the site. The governing agency should be consulted to establish the base flood elevation to be used for design.

4.4.6 Tsunami

A tsunami is a wave or series of waves generated by an earthquake, landslide, or volcanic eruption. The California Emergency Management Agency (CalEMA) in concert with the CGS and the University of Southern California have prepared tsunami inundation maps for emergency planning. According to the tsunami inundation map prepared for the site area, tsunami-generated waves will not reach the site area due to its distance from the San Francisco Bay and prominent water courses.

5 SITE AND SUBSURFACE CONDITIONS

5.1 SITE DESCRIPTION

The site generally consists of agricultural land with two houses located in the southern portion of the site. The site is bound to the west by the Los Esteros Electrical Power Substation and vacant farm land; to the south by Ranch Drive; to the east by Coyote Creek; and to the north by agricultural property. Site topography appears relatively flat, with a slight slope down to the north and about 10 feet of total vertical relief. A levee along Coyote Creek extends about 8 feet above the adjacent portion of the site.

5.2 SUBSURFACE CONDITIONS

5.2.1 Stratigraphy

The following description provides a general summary of the soil conditions encountered during this study. For more thorough descriptions of the actual conditions encountered at specific boring and CPT locations, refer to the boring logs in Appendix A and CPT logs in Appendix C.

Soils in the upper 3 to 5 feet of the site included predominately granular soils consisting of clayey sands, sands and gravels with variable clay content, and some sandy clays. Underlying these soils, our borings generally encountered lean to fat clays extending to depths of about 20 to 25 feet. These clayey soils were underlain by interbedded loose to medium dense gravels with sand, loose to medium dense sands with gravel, and low to medium plasticity sandy lean clays to a depth of approximately 80 feet bgs. Below this depth, dense, well-graded gravel with sand and clay and firm to very hard sandy lean clays were encountered and extended to the full depth of our explorations.

The borings and CPTs indicate a layer of granular soil approximately between 20 feet and 45 feet below the existing grade in the eastern area of the project site. However, this layer was not encountered in borings and CPTs in the western area. Therefore, two generalized soil profiles were developed for engineering calculations, one for the eastern area and the other for the western area. Figure 3 shows the approximate dividing line between these two profiles used in our analyses. The soil units encountered in our borings and CPTs appear to be consistent with geologic mapping of the area.

5.2.2 Groundwater

We inferred a groundwater depth during drilling of 12 feet bgs, or deeper, based on seepage into boreholes, wetting of the sampler, and/or sample moisture content. Due to the fine grained nature of the site soils, and corresponding low permeability, it is difficult to evaluate actual groundwater levels at the time of drilling. Pore pressure dissipation testing during Cone Penetration Testing indicated groundwater levels that ranged from about 7 to 12 feet bgs. We selected a design groundwater depth of 5 feet based on historic ground water levels from adjacent Department of Water Resources wells, and previous reports by others.

Groundwater conditions at the site will likely change due to variations in rainfall, tidal conditions, creek water levels, groundwater withdrawal or recharge, construction activities, well pumping and irrigation, or other factors not apparent at the time the study was performed.

5.2.3 Variations in Subsurface Conditions

Our interpretations of soil and groundwater conditions at the site are based on the conditions encountered in the borings drilled and CPTs advanced at the site, and our review of available geologic data. The geotechnical recommendations for design and construction that follow are based on those interpretations.

5.2.4 Coyote Creek Levee Conditions

The existing levees along the west bank of Coyote Creek, adjacent to the subject property, are owned by Santa Clara Valley Water District (SCVWD). These levees are located within Reach 2B of the Coyote Creek levee system, according to US Army Corps of Engineers and SCVWD's reach numbering system (SCVWD, 1984). The Coyote Creek levees adjacent to the project site are listed as "minimally acceptable" according to the National Levee Database (USACE, 2016) based on an inspection conducted in August, 2011.

In Reach 2B, which extends from about Station 535+00 to Station 566+00, the levees vary from 400 to 900 feet apart and from 4 to 7 feet in height above adjacent ground (USACE, 2010). Flood flows are split downstream of Highway 237 between the main channel and the overflow channel.

The overflow channel crosses the existing creek (cross over # 4) about 1,800 feet downstream of State Highway 237. In 1983, SCVWD prepared an Engineer's report on interim flood control improvements on lower Coyote Creek, in the vicinity of the project site. Flood control measures were constructed at various locations along Coyote Creek downstream of Highway 237 (SCVWD, 1984). Following additional planning in the late 1980s and early 1990s, the existing creek levees were removed and the channel banks lined with rock at the overflow crossings and levee construction was completed in 1992 by SCVWD (USACE, 2010).

During the summer of 2008, SCVWD undertook a project to improve and repair the Coyote Creek levees. The improvements consisted of recompaction of the soils along the face of the levees, clay slurry repairs (likely injected to fill rodent burrows), excavation and recompaction of specific sites on the levee, and repair of slumps on the levee. Detailed plans and drawings of these repairs were not found during Kleinfelder's data search, but may possibly be obtained from SCVWD at a future date as necessary.

No previous geotechnical investigation documents or levee design documents were obtained during our initial data collection efforts prior to publication of our April 2016 letter. Since publication of that letter, we have received significant additional documentation from SCVWD. Review and interpretation of that data will be handled in future phases of our project work for purposes of final design. Significant investigations into nearby levees to the north (including along Coyote Creek, downstream of the project site) have been conducted for the South Bay Salt Pond Restoration Project, and that data may be included in future data reviews. Portions of levees along Coyote Creek to the south, in downtown San Jose, are currently being analyzed by SCVWD for future maintenance and improvement.

6 KEY GEOTECHNICAL DESIGN CONSIDERATIONS

This section provides a summary of the key geotechnical considerations that must be addressed in project planning, design and construction and forms the basis for design recommendations presented in Section 7 and construction recommendations presented in Section 8. Since many of these issues are complex and interrelated, we recommend a meeting to discuss with the project team.

6.1 SITE SETTLEMENT IMPACTS TO FLOOR SLABS AND UTILITIES

Static and seismic settlements could affect slab-on-grade floors and utilities. Use of a pile-supported structural floor slab and flexible utility connections would effectively mitigate the risk of settlement-related damage to slabs and utilities. Alternatively, use of ground improvement methods and, depending on construction sequencing, a two to three month settlement period between fill placement and slab-on-grade floor construction would also effectively mitigate settlement risks. The following paragraphs discuss static and seismic settlement risks.

6.1.1 Static Settlement

As discussed in more detail in Section 7.3, site soils will settle under the proposed site fill and floor slab loads (static settlement). We estimate the site will settle about ¾ to 1¾ inch under the planned site fills of about 3 to 5 feet and that this settlement will require about 2 to 3 months to be substantially complete. Settlement due to fills can be mitigating by waiting 2 to 3 months following fill placement prior to pouring slabs.

We estimate that soils below floor slabs will settlement about ½ to ¾ inch as the slab loads are applied, which we understand exceeds the requested floor slab settlement tolerance of ¼ inch for this project. In our experience, floor slab settlements in excess of ½ inch present a significant risk of slab cracking, particularly when the structure is supported on piles and portions of the slabs will overlie pile caps. Reinforcing the slab with rebar can reduce, but not eliminate potential for cracking. A pile-supported structural floor slab or ground improvement methods can be used to mitigate floor slab settlement and potential cracking.

6.1.2 Seismic Settlement

As discussed in Section 7.2.3, we estimate potential liquefaction induced (seismic) settlement of nil up to 7½ inches during a strong earthquake. Differential settlement of the ground surface may be estimated as one-half of the total. However, slabs would likely hang up on pile caps resulting in potential differential settlement approaching 7½ inches in some locations. Under this scenario we would expect significant damage to slab-on-grade floors, equipment supported on the floors, and underlying utilities that do not have flexible connections. Use of a pile-supported structural slab would mitigate seismic settlement risks, as well as the above-mentioned static settlement risks.

If a pile-supported structural slab is used, it will still be necessary to construct utilities with flexible connections and/or hang utilities under the slab and structure. Utilities that are hung should be backfilled with flowable material such as pea rock, or designed so that the full weight of the utilities and overlying soils can be supported so they are not pulled away from supports when ground settlement occurs.

6.2 DEEP FOUNDATIONS

Based on the clay layers at the site, anticipated range of structural building loads, including column loads of up to approximately 1200 kips, and potential for liquefaction induced settlement, we recommend a deep foundation system. Spread footings would undergo excessive static and seismic settlements. Based on our local experience, we anticipate that driven, precast, prestressed concrete (PCC) piles will be the most economical pile type. We recommend conducting a pile indicator program to optimize foundation design. Section 6.8 presents addition discussion on feasible foundation types and design.

Various deep ground improvement approaches may be feasible alternatives to deep foundations. Kleinfelder can provide design recommendations for various ground improvement alternatives, if desired.

6.3 MOISTURE SENSITIVE AND EXPANSIVE SOILS

The on-site soils are moisture sensitive and will be difficult to traverse with construction equipment and to properly compact during or following wet weather. Construction planning should account for potential impacts due to wet weather conditions.

Some of the shallow site soils have low to medium expansion potential and may shrink or swell as a result of seasonal or human-induced soil moisture content changes. The planned import of 3 to 5 feet of fill to cap the site will largely mitigate the expansive soils provided imported material conforms to recommendations presented in Section 8.2 and an adequate cap of non-expansive soil is maintained as discussed in Section 7.4.

7 DESIGN RECOMMENDATIONS

7.1 SETBACK REQUIREMENTS

The City of San Jose currently has a Riparian Corridor Policy that requires a 50 to 100 foot setback from any established riparian corridor, which is defined as "any defined stream channel, including the area up to the bank full-flow line, as well as all riparian vegetation in contiguous adjacent uplands." Further definition and clarification may be found in the City's 1999 Riparian Corridor Policy Study. The City is currently considering revising this policy to require a minimum 100-foot setback in all but a limited number of instances, where no significant environmental impacts would occur. Accordingly, we recommend a 100-foot setback from the top of bank (the crest of the Coyote Creek Levees) be assumed for major development elements of this project. Any improvements within the 100-foot setback distance will require review by the City of San Jose and may require additional geotechnical evaluation.

7.2 SEISMIC DESIGN CONSIDERATIONS

7.2.1 Site Class

In developing seismic design criteria, the characteristics of the soils underlying the site are an important input parameter to evaluate the site response. As discussed in Section 4.4.2, some of the soil layers underlying the site may liquefy resulting in potentially excessive total settlements. Therefore, according to the 2013 California Building Code (CBC), the site should be classified as Soil Class F, which requires site response analysis.

However, ASCE 7-10 suggests that for a short period (less than ½ second) structure on a site with liquefiable soils, Soil Class D or E may be used instead of F to estimate design seismic loading on the structure. The selection of Soil Class D or E is based on the assessment of the site soil profile type assuming no liquefaction.

We understand the proposed buildings are 90-feet high and anticipate that the period of such structure(s) will likely be greater than ½ second. However, the structural engineer should confirm this assumption. For such structures, a site-specific site response analysis would be required. During our exploration program, we collected data to support a site-specific response analyses and can provide this service when the structural engineer confirms it is necessary and we receive authorization for additional Scope.

In the event the building period is less than ½ second and/or ground improvement methods are employed to minimize the consequences of potentially liquefiable soils at the site, the site may be classified as Site Class D according to Section 1613.3.2 of 2013 CBC and Table 20.3-1 of ASCE 7-10. Site Class D is defined as a soil profile consisting of stiff soil profile with a shear wave velocity between 600 ft/sec and 1,200 ft/sec, standard penetration test (SPT) blow counts (N-value) between 15 blows per foot and 50 blows per foot, or undrained shear strength between 1,000 psf and 2,000 psf in the top 100 feet.

7.2.2 Seismic Design – Ground Motion Parameters

The PACLAND 1926 site is located approximately at the following coordinates:

- Latitude: 37.426 degrees
- Longitude: -121.929 degrees

The Risk-Targeted Maximum Considered Earthquake (MCE_R) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1) were estimated using Section 1613.3 of the 2013 CBC and the U.S. Geological Survey (USGS) web based application (available at http://earthquake.usgs.gov/designmaps/us/application.php). The mapped acceleration values and associated soil amplification factors (F_a and F_v) based on the 2013 CBC and corresponding site modified (S_{MS} and S_{M1}) and design spectral accelerations (S_{DS} and S_{D1}) are presented in Table 6.3.2-1. It should be noted these factors are for Site Class D, which, as discussed above, may not be applicable for the larger proposed buildings at this site. Additional seismic analysis and refinement may be necessary for purposes of final design of the structures.

Table 7.2.2-1
CBC Seismic Design Parameters

Parameter	Value	Reference
S_S	1.520g	2013 CBC Section 1613A.3.1
S_1	0.601g	2013 CBC Section 1613A.3.1
Site Class	D	ASCE 7-10 Chapter 20
Seismic Design Category	D	2013 CBC Tables 1613A.3.5 (1) and (2)
F_a	1.000	2013 CBC Table 1613A.3.3(1)
F_v	1.500	2013 CBC Table 1613A.3.3(2)
S_{MS}	1.520g	2013 CBC Section 1613A.3.3
S_{M1}	0.902g	2013 CBC Section 1613A.3.3
S_{DS}	1.013g	2013 CBC Section 1613A.4.4
S_{D1}	0.601g	2013 CBC Section 1613A.4.4
PGA	0.584	ASCE 7-10 Figure 22-7
F_{PGA}	1.000	ASCE 7-10 Table 11.8-1
PGA_M	0.584g	ASCE 7-10 Section 11.8.3
C_{RS}	1.098	ASCE 7-10 Figure 22-17
C_{R1}	1.060	ASCE 7-10 Figure 22-18
T_L	12 sec.	ASCE 7-10, Figure 22-12

According to Section 1803.5.12 of the 2010 CBC, in the absence of a site-specific ground motion hazard analysis, the MCE geometric mean peak ground acceleration adjusted for Site Class effects (PGA_M) can be determined based on Equation 11.8-1 in Section 11.8.3 of ASCE 7-10. Therefore, based on the 2013 CBC, 0.584g should be used for the site PGA_M.

7.2.3 Seismic Settlement Due to Liquefaction

As described in Section 4.4.2, seismic total settlement estimates based on the results of data collection and liquefaction analyses computed from the CPTs and rotary wash borings range from nil to about 7½ inches. Differential settlement could be taken as one-half the total settlement from the nearest boring and/or CPT between adjacent supports and is estimated to be about 4 inches.

Based on the estimated liquefaction-potential ground surface and structural (foundation and floor) displacements due to ground movement from seismic liquefaction settlement hazards have been considered during this geotechnical investigation for design. The seismic-related liquefaction settlement issue influenced foundation type selection recommendations necessary to mitigate the liquefaction settlement hazard.

7.3 STATIC SETTLEMENT

A portion of the soils at the site are considered compressible and prone to a reduction in volume and corresponding settlement when subjected to loading, such as placement of fill and floor slab loads. Consolidation testing was not included in our Scope of Services. Therefore, we estimated soil compressibility characteristics based on published empirical relationships and data collected from field exploration and laboratory testing, as well as professional judgment. Two generalized soil profiles were developed for settlement calculations, one for the eastern portion of the site and one for the western portion of the site. The approximate dividing line between these two profiles is shown on Figure 3.

In general, a majority of the subsurface soils are considered normally consolidated to moderately over-consolidated. While the anticipated loads from placement of new fill to raise site grades will likely not exceed the pre-consolidation pressure, they will induce settlements that need to be accounted for in project design and construction. We estimated static settlement using conventional consolidation and elasticity theory methods using the computer program Settle3D, Version 3.0 (RocScience).

Estimated settlements due to loads imparted by 3 to 6½ feet of proposed fill range from approximately ¾ inch to 1¾ inch. Additional settlements imparted by an assumed 350 psf concrete floor slab load range from approximately ½ inch to ¾ inch, which we understand exceeds the requested floor slab settlement tolerance of ¼ inch for this project. Total settlements due to new fill loads plus concrete floor slabs range from approximately 1¼ inch to 2¼ inch.

With a pile-supported foundation system, settlements of the slab in excess of ½ inch will potentially cause slab cracking because portions of the slab overlying pile caps will not settle. This risk, combined with potential seismic settlement concerns, leads us to recommending a pile-supported slab to reduce potential damage following a seismic event.

If the Owner chooses not to mitigate seismic settlement of floor slabs by use of a structural slab or ground improvement, additional field exploration and subsequent laboratory testing to further refine consolidation parameters and estimates of static settlement is warranted. Although unlikely, such testing could demonstrate that settlement of slabs will be under ½ inch, depending on the results of the laboratory testing.

We recommend that fill be placed two to three months prior to construction of floor slabs. This would allow for the majority of fill-related settlement to occur prior to slab placement, reducing the overall anticipated deformation and potential for cracking of floor slabs.

7.4 EXPANSIVE SOILS

Based on laboratory testing results, the low to medium expansive soils encountered for some of the surficial subgrade soils at the site may shrink or swell as a result of seasonal or human-induced soil moisture content changes. Special measures will be needed where the proposed structures directly overlie these areas. To mitigate expansive soil behavior the building footprint limits should be underlain by at least 12 inches of non-expansive fill. Exterior flatwork areas and pavement areas should be underlain by at least 6 inches of non-expansive fill.

In general, based on the planned 3 to 5 feet of fill across the site, the most efficient way to mitigate the presence of expansive soils will be to import non-expansive fill and cap the potentially expansive soils. Imported fill should consist of non-expansive soil as discussed in Section 8.2. In areas where there will be less than 12 inches of non-expansive fill overlying potentially expansive fill, such as cuts for grade beams or other structural elements, it may be necessary to remove a portion of the existing site fills.

It is our recommendation that supplementary laboratory testing be performed on the subgrade soils after the completion of rough grading operations to evaluate the expansion potential of the exposed subgrade soils, and confirm or modify these recommendations.

7.5 FOUNDATION SUPPORT

7.5.1 General

This section presents foundation design recommendations for the proposed buildings with column loads on the order of 1,200 kips. While it is possible that shallow foundations could be used to support of static loads, significant over-excavation and/or ground improvement would be required to meet static settlement requirements. In addition, potential foundation displacements up to about 7½ inches due to settlement from liquefaction essentially precludes the use of a shallow foundation system. The project structural engineer should be consulted regarding this conclusion for the proposed buildings and structural systems.

For mitigation of potential effects on the building structures due liquefaction, we recommend either a "structural" deep foundation solution, or a "ground improvement" shallow foundation solution. Various ground improvement options, such as cement deep soil mixing (CDSM), "densification" methods, or other technically viable methods are not anticipated to be the preferred solution due various factors including the large building footprint areas and reported contaminants from past agricultural uses and practices.

It should be recognized that a "value engineering" (VE) study at a later date could potentially lead to other technically viable foundation options that may result in cost and/or perhaps schedule optimization. However, this type of VE evaluation is beyond the scope of our current geotechnical study and we believe the range of options provided in the following sections represent the best value in terms of cost and schedule. Potential options to consider in a VE study include:

- Controlled Modulus Columns
- Drilled displacement piles
- Aggregate piers
- Cast-in-drill-hole piles

7.5.2 Recommended Deep Foundations

A deep foundation system can mitigate the potential effects of liquefaction-induced settlement and reduce the risk of differential settlement between heavily loaded adjacent building columns. We also recommend structurally supporting the floor slab on deep foundations to mitigate static and liquefaction-induced settlement. Based on the predominantly clay subsurface conditions and anticipated range of structural building loads, including column loads of up to approximately 1200 kips, we recommend a deep foundation system using driven displacement piles. We recommend use of driven 14-inch-square, precast, prestressed concrete (PCC) pile foundations. Driven 14-inch square PCC piles are commonly used throughout the San Francisco Bay area, including sites in San Jose. These piles are generally cost-effective, installation methods have been well developed, are suitable for the site subsurface conditions, and have been used successfully on other commercial and industrial projects.

7.5.3 Axial Pile Capacity

Pile foundations should be designed to develop their axial compression capacity primarily in skin friction in the stiff clays and intermittent dense sand layers. Charts illustrating the allowable axial compressive and tensile capacities of 12-inch auger cast piles, 14 inch square PCC piles, and 14-inch steel pipe piles are presented on Figure 4 and Figure 5. The capacity curves apply for single piles for post-liquefaction soil conditions.

Consideration for a reduction in individual axial pile capacity to account for group effects usually is not necessary for piles with center-to-center spacing of three or more pile diameters or widths. Group effects on deep foundations are dependent on a number of factors, including soil properties, pile size and group configuration. Additional axial and lateral pile analysis will be required if deep foundations are to be considered with center-to-center spacing of less than three pile diameters or widths. Note that the curves on Figures 4 and 5 are for capacity based on soil conditions; the structural capacity of the piles may control and should be checked.

For an anticipated 1200-kip column load, a pile group layout of four 300-kip PPC piles, with driven pile tip depth on the order of up to approximately 75 feet below ground surface should suffice to develop an allowable axial compression capacity of 300 kips per pile (for dead loads plus live loads). A one-third increase to this value can be assumed for seismic and/or wind loading conditions. Based on the subsurface conditions encountered during our field exploration program, it may be difficult to advance driven piles beyond depths of 75 to 80 feet in dense granular soils.

The allowable uplift (tension) capacity will be developed solely by frictional resistance between the pile shaft and the surrounding soils. A chart illustrating the ultimate axial tension capacities of 12-inch auger cast piles, 14 inch square PCC piles, and 14-inch pipe piles is presented on Figure 5. The capacity curves apply for single piles. A one-third increase to this value can be assumed for seismic and/or wind loading conditions.

Pile embedment beneath finished grade should be evaluated during the final design phase as well as during a construction "indicator pile program" (discussed below) for the driven piles. Actual pile tip elevations and pile lengths may vary depending on driving conditions encountered during the indicator pile program.

Based on the presence of liquefiable soil layers beneath the site, we expect that seismic settlements up to several inches may occur as discussed previously in this report. For settlements of this magnitude, a drag load (negative skin friction) should be considered in addition to the long-term pile loads for checking the structural capacity of the piles. Liquefaction-induced settlements are likely to occur above the neutral plane of the pile, and thus will likely not have a large impact on the structural capacities of the piles. However, this design consideration will require further geotechnical analyses in collaboration with the structural engineering designer, and considering various aspects such as load demands on piles, dragloads, downdrag settlement and final pile design lengths.

The range of structural settlement magnitude for various load cases using a deep foundation system is premature to estimate at this time since information is not yet known regarding structural loading, column spacing, pile group sizes, and final pile lengths. Typical pile designs limit total settlement to less than about 1-inch. Differential column settlements could occur within the building as a result of variations in column loads and variable soil conditions. Typical maximum differential settlements between adjacent columns are on the order of ½ inch for long-term, total settlements, and up to about ¼ inch for immediate, post-construction settlements. These aspects of design will need to be addressed between the structural and geotechnical engineering members of the design team during final design.

7.5.4 Lateral Capacity for Deep Foundation Systems

Resistance to lateral loads will be provided by passive soil pressure against the driven pile and by the bending strength of the pile itself. When the preferred pile type is selected, with geotechnical input from Kleinfelder, the structural engineer can develop soil-structure interaction analytical models (such as with LPILE software), and the LPILE models can be analyzed for various load and structural fixity combinations to evaluate the lateral response of the driven pile foundations.

More detailed pile group lateral capacity analyses should be performed when the foundation loading and system geometry is more clearly defined. At such time when additional lateral loading analyses may be required for final design, Kleinfelder can perform LPILE and related calculations to assist in determining the shear, moments, and lateral displacement for piles and pile groups based on the final design loads and layouts. However, this type of analysis is beyond the scope of our current investigation.

Resistance to lateral loads can also be provided by passive soil pressure against pile caps and grade beams, and by soil frictional resistance against the sides of pile caps and grade beams.

For preliminary design purposes of pile caps and grade beams, the passive pressure should be calculated using equivalent fluid pressure value of 300 pounds per cubic foot (pcf) above groundwater table.

Friction along the sides of pile caps and grade beams may be used in combination with the passive resistance. The frictional resistance can be estimated by using a coefficient of friction of 0.35. The effective at-rest pressures normal to the sides of the structural elements should be used in estimating frictional resistance along the sides. We recommend using equivalent fluid weight of 60 pcf for the effective at-rest earth pressure in soils above the groundwater level.

The resistance from the upper 12 inches of pile caps and grade beam should be neglected in lateral resistance calculations unless the adjacent soil surface is covered by a permanent pavement or floor slab. However, the pressure distribution for any case should be calculated from the soil surface.

Friction along the bottom of pile-supported structures (grade beams, pile caps) should not be used in lateral resistance calculations because of the anticipated ground settlements that may occur over the life of the structure.

7.5.5 Construction Indicator Pile Program

Prior to construction, the pile driving contractor should submit a report of drivability study, using wave equation analyses, to confirm that the selected pile hammer, cushion, and cap block can be used to achieve the desired pile capacities without damage to the piles.

We recommend that prior to production pile driving, an Indicator Pile Program be undertaken to evaluate driving resistances and developed capacities across the site and obtain data for the selection of production pile lengths. We recommend that indicator pile driving be monitored with a pile driving analyzer (PDA) to evaluate soil resistance and driving criteria and the stresses in the pile during driving.

Several of the indicator piles should be re-struck after at least 48 hours following initial driving to evaluate setup or increase in capacity with time. During initial driving, skin friction typically will be relatively low due to disturbance and excess pore water pressures that build up but then dissipate after driving stops. If the observed setup is less than needed, it could be necessary to allow more time to pass, accept reduced pile capacities, or lengthen the piles.

The indicator pile driving program should be used to provide installation driving criteria for the production piles. Modifications to the pile design capacities may be required based on the results of the indicator pile program.

We recommend that the indicator pile program include at least 20 to 30 piles uniformly covering each of the building footprints. The actual number of recommended indicator piles will depend on the final configuration of the foundation system and the number of production piles. For planning purposes, it can be assumed that the indicator piles will be on the order of 80 to 90 feet long (i.e., about 10 feet longer than expected for production piles). If precast concrete indicator piles are to be used at production pile locations, and if additional reinforcing steel is placed in the upper portions of the piles for lateral load bending moments, then the reinforcing steel should be extended a minimum of 15 feet for the indicator piles in order to allow for variation and pile cut-off.

7.5.6 Pile Installation Criteria – Driven Precast or Pipe Piles

The piles should be driven using a hammer capable of developing at least 80,000 foot-pounds of rated energy. We expect that piles driven to about 30 to 40 blows per foot, assuming the hammer delivers at least 80 percent of the rated energy, can develop the allowable axial capacity. All driving criteria should be developed using the PDA results from the indicator pile program. The same size and type of hammer should be used for both indicator and production pile driving.

Predrilling, if used, should include predrill hole diameters of less than the width (12 to 14 inches) of the concrete or steel pipe piles. Predrilling criteria should be developed further during the indicator pile program.

7.6 SLAB ON GRADE AND EXTERIOR FLATWORK

It is our understanding that concrete slabs-on-grade for this project will consist of interior concrete floor slabs, and some exterior flatwork. Based on our laboratory testing and our experience with similar soils in the surrounding area, the near surface soils at the site have a low to medium expansion potential. We recommend that subgrade soils below interior concrete slabs-on-grade be replaced with "non-expansive" engineered fill, to a minimum depth of 12-inches below rough building pad grade elevation (or below the bottom of the crushed rock or gravel capillary break discussed further below). If overexcavation and removal of the expansive subgrade soil is undesirable, then a raised (built up) fill pad using imported non-expansive engineered fill should be considered.

Care should be taken to avoid drying of soils exposed in the floor slab and flatwork subgrades. Due to expansive soil concerns, proper moisture conditioning of the underlying subgrade soils and maintaining that moisture prior to construction of the concrete slabs is essential to the success of the project. The moisture content of the subgrade for foundation and slab support should be checked 24 hours before pouring concrete. Supporting soils should be in a moist condition prior to the placement of concrete to reduce the potential for volume changes in the soil. If moisture contents are less than 2 percent above optimum per ASTM D 1557, the foundation material should be sprayed with water 24 hours prior to pouring concrete to increase the moisture content until the moisture exceeds the above requirement for a depth of 6 inches beneath the foundation.

As a minimum, we recommend that the interior floor slab-on-grade should be underlain by a capillary moisture break consisting of at least 4 inches of clean, free-draining crushed rock or gravel graded such that 100 percent by weight will pass the 1-inch sieve, and less than 5 percent by weight will pass the No. 4 sieve. The exterior grading and site drainage will have an impact on the potential moisture beneath concrete slab-on-grades. In general, the elevation of exterior grades should not be higher than the elevation of the sand/gravel layer beneath the interior floor slab to help reduce water intrusion beneath the slab. Otherwise, waterproofing the slab and walls should be considered.

Subsurface moisture and moisture vapor naturally migrate upward through the soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. To reduce the impact of this subsurface moisture and the potential impact of future introduced moisture (such as landscape irrigation or precipitation) on moisture sensitive flooring, we recommend placing a vapor retarder below the slab. Vapor retarder often consists of visqueen or polyvinyl plastic sheeting at least 10 mil in thickness. Other proprietary systems such as Stego Wrap or equivalent may also be used.

It should be noted that although vapor barrier systems are currently the industry standard, this system might not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building and all elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction may have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect indoor air quality. We recommend consulting a specialist for details of vapor retarder design and installation.

A Modulus of Subgrade Reaction (k) value of 125 pounds per square inch/per inch of settlement can be used to design floor slabs and exterior flatwork supported on the "non-expansive" engineered fill subgrades. The structural engineer should design the slab thickness, reinforcing, and control joint spacing. However, a minimum floor slab thickness of 6 inches is recommended for interior floor slabs and exterior flatwork subject to light vehicle traffic. If reinforcement is included, special care should be taken to ensure it is placed and maintained at the height designed during the concrete pour.

Exterior slabs should be underlain by a minimum of 12 inches of "non-expansive" fill. Where exterior flatwork will be subjected to vehicle traffic, we recommend that the upper 6-inches of engineered fill be Caltrans Class 2 aggregate base compacted to a minimum relative compaction of 95 percent per ASTM D1557. It is recommended that all exterior concrete flatwork be cast free from adjacent footings or building slabs. This may be accomplished by using a strip of 30-pound felt divider material between the slab edges and adjacent structures.

7.7 RETAINING WALLS

We understand new low retaining walls may be used to accommodate grade changes, including at truck loading docks for some of the buildings. The maximum anticipated retaining wall height is about 5 feet. We assume the walls will be designed as restrained walls (not free to rotate) and that the retained backslope will be nearly level.

7.7.1 Lateral Earth Pressures

Retaining walls should be designed to resist lateral pressures caused by water, soil and external surface (surcharge) loads. The magnitude of the lateral pressures will depend on whether or not the wall will be allowed to move, the backfill type and the method of its placement, the magnitude of external loads (e.g., loads at the top of the retained slope), and drainage provisions. The following equivalent fluid earth pressures are recommended assuming wall heights of 5 feet or less, and fully drained backfill conditions:

Table 7.7.1-1
Lateral Earth Pressure Values for Retaining Wall Design

Earth Pressure Condition	Backfill Slope	Lateral Earth Pressure (pcf)
At-Rest	level	60

Where uniform surcharge loads are located within a lateral distance from constrained retaining walls equal to the wall height, 45 percent of the surcharge load should be applied uniformly over the entire height of the wall.

Ultimate passive pressure resistance will develop under lateral deflections of about 2 percent of the wall height. Passive resistance in the upper 12 inches should be neglected unless the area in front of the footing is protected from disturbance by concrete or pavement.

7.7.2 Wall Backfill and Drainage

To achieve the earth pressure and drainage conditions described above, the backfill adjacent to the wall should consist of granular soil, compacted according to the recommendations provided in Engineered Fill section of this report. Kleinfelder should review and approve the proposed backfill materials before they are used in construction. Overcompaction of wall backfill should be avoided because increased compaction effort can result in lateral pressures significantly greater than those used in design. We recommend that all backfill placed within 3 feet of the walls be compacted with hand-operated equipment.

The walls may be designed without hydrostatic pressures if they are fully drained. Wall drainage should consist of either a prefabricated drainage material or a layer of drain rock placed behind the wall that is connected to a suitable drainage location. Prefabricated drainage material (such as Miradrain or an approved alternative) may be used behind retaining walls. Prefabricated drainage material should be installed in accordance with the manufacturer's recommendations.

As an alternative to prefabricated drainage material, a drain rock layer may be used. The drain rock should be at least 12 inches thick and extend to within 2 feet of the ground surface. Four-inch diameter perforated plastic pipe should be installed (with perforations facing down) along the base of the walls on a 2-inch thick bed of drain rock. The pipe should be sloped to drain by gravity to a sump or other drainage facility. Drain pipe should be rigid-walled PVC or similar material that is capable of withstanding all applied loads.

Drain rock should conform to Caltrans specifications for Class 2 permeable material. Alternatively, clean, ½- to ¾-inch maximum size crushed rock or gravel could be used, provided it is encapsulated in a non-woven geotextile filter fabric, such as Mirafi 140N or an approved alternative. A 1-foot thick cap of clayey soil should be placed over the drain rock to inhibit surface water infiltration.

7.8 PAVEMENTS

We anticipate that parking lot and drive aisles will consist of flexible asphalt concrete (AC), and the loading locks and trash storage areas will utilize rigid (Portland cement concrete) pavement sections. Traffic Index (TI) design input parameters have not been provided to us. We have assumed TI values between 5 and 7 for the project. The appropriate TI should be selected by the civil engineering designer. In addition, since the nature of planned import fill is unknown at this time, the pavement design recommendations should be reviewed by Kleinfelder once the fill material is identified.

Based on Caltrans design methods and measured laboratory Resistance Value (R-value) test result of 10 for lean clay subgrade soils in boring HSA-1, the recommended pavement sections for TIs ranging between 5 and 7 are provided below. Each TI represents a different level of use. The owner or designer should determine which level of use best reflects the project, and select appropriate pavement sections. A TI of 5 is commonly used for automobile parking spaces. A TI of 6 is commonly used for automobile access lanes. A TI of 7 is commonly used for truck (delivery) access and truck parking aprons.

Pavement section parameters include AC and Caltrans Class II aggregate base (AB). The recommended pavement sections are provided below:

Table 7.8-1
Recommended AC Flexible Pavement Sections for Native Subgrade Soils
Design R-Value = 10

Traffic Index	PACLAND 1926 Project Site	
	AC (inches)	AB (inches)
5	4.0	7.0
6	5.0	9.0
7	5.0	13.0

We recommend if rigid reinforced Portland cement concrete (PCC) pavement sections are selected for use, that they consist of at least 8 inches of PCC over 12 inches of AB for locations with TI values less than 9, and 9.5 inches of PCC over 16 inches of AB for TI values between 9 up to 11.

Additional subgrade sampling and laboratory testing will be necessary during initial site earthwork grading to better characterize the subgrade R-values, particularly for imported engineered fill materials, and to refine pavement section designs if necessary.

Pavement subgrade should be moisture conditioned and compacted to a minimum of 95 percent relative density to within 2 percent of optimum moisture content. Subgrade preparation should extend at least 3 feet laterally beyond the face of curbs.

Parking areas should be sloped and drainage gradients maintained at 2% minimum to carry surface water from the site. Surface water ponding should not be allowed anywhere on site during or after construction.

7.9 CORROSION

Laboratory chloride concentration, sulfate concentration, pH, oxidation reduction potential, redox, sulfide and electrical resistivity tests were performed for a near surface soil sample. The results of the tests are attached and are summarized in Table 6.12-1. We recommend that similar corrosion potential laboratory testing should be completed on imported fill material.

Table 7.9-1
Soil Corrosion Laboratory Test Results

Boring and Depth	Material	Resistivity, ohm-cm		pH	Oxidation Reduction Potential, mV	Water-Soluble Ion Concentration, ppm		
		Saturated	In-Situ Moisture			Chloride	Sulfide	Sulfate
MR-3 at 3 feet	Silty Sand	2,100	2,900	8.06	+350	39	N.D.	52
HSA-1 at 3 feet		860	1,600	7.80	+360	68	N.D.	120

*N.D. - None Detected

Ferrous metal and concrete elements in contact with soil, whether part of a foundation or part of the supported structure, are subject to degradation due to corrosion or chemical attack. Therefore, buried ferrous metal and concrete elements should be designed to resist corrosion and degradation based on accepted practices.

Based on the "10-point" method developed by the American Water Works Association (AWWA) in standard AWWA C105/A21.5, the soils at the location of MR-3 and HSA-1 are moderately corrosive and highly corrosive to buried ferrous metal piping, cast iron pipes, or other objects made of these materials, respectively. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures.

The degradation of concrete or cement grout can be caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete, causing cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete or cement grout. The American Concrete Institute (ACI) in their publication "Guide to Durable Concrete" (ACI 201.2R-08) provides guidelines for this assessment. Sulfate tests indicated the samples collected from MR-3 and HSA-1 had concentrations of 52 ppm and 120 ppm, respectively. The results of these sulfate tests indicate the potential for deterioration of concrete is mild, no special requirements should be necessary for the concrete mix.

Concrete and the reinforcing steel within it are at risk of corrosion when exposed to water-soluble chloride in the soil or groundwater. Chloride tests indicated the samples collected from MR-3 and HSA-1 had concentrations of 39 ppm and 68 ppm, respectively. The project structural engineer should review this data to determine if remedial measures are necessary for the concrete reinforcing steel.

7.10 SURFACE DRAINAGE

It is important that drainage away from the improvements be provided and maintained to reduce ponding and/or saturation of the soils in the vicinity of foundations. The design should incorporate the basis for good drainage, including:

- Sufficient pad height to allow for proper relief from drainage courses.
- Defined drainage gradients away from the structures to points of conveyance, such as drainage swales and/or area drains and discharge pipe.
- A plan for long-term maintenance to address settlement issues and to correct ponding and erosion areas, if needed.

Maintenance personnel should maintain the established site drainage by not blocking or obstructing gradients away from foundations or structures

8 CONSTRUCTION RECOMMENDATIONS

8.1 SITE PREPARATION

8.1.1 General

It is anticipated that site grading can be performed with conventional grading equipment and techniques. Considering the existing site grades and conceptual grading plan currently being prepared by the project team, site earthwork is anticipated to be limited to 5 feet or less of fill placement to establish building pad grades and surrounding ground surface improvements (parking, driveways, flatwork, landscaping). All references to compaction, maximum density and optimum moisture content are based on ASTM D1557, Modified Proctor, unless otherwise noted.

8.1.2 Stripping and Demolition

Areas to receive fill and structures should be stripped of existing surface vegetation, organic topsoil, debris, existing pavements and other man-made features, and any other deleterious materials prior to over-excavation or placement of engineered fill. Existing vegetation and organic laden soils should be stripped to a minimum depth of 3 to 4 inches prior to performing earthwork at the site. Any stripped organic materials or debris should not be reused as engineered fill. Soft or loose areas may be encountered during construction that may require over-excavation. Unit prices for over-excavation and replacement with engineered fill should be obtained during bidding.

Stripping and removals should extend laterally a minimum of 5 feet beyond the perimeters of structures, concrete flatwork, and any other facilities supported on grade. The contractor should be responsible for developing and implementing an erosion control and storm water pollution prevention plan prior to disturbing any materials on-site.

8.1.3 Existing Utilities

Active or inactive utilities within the construction area should be protected, relocated, or abandoned as appropriate. All active utilities should be relocated outside of structure footprints. Inactive utility pipes should be removed or filled with sand/cement slurry and capped at both ends.

8.1.4 Disturbed Soil, Undocumented Fill, and Subsurface Obstructions

Initial site grading should include a reasonable search to locate soil disturbed by previous activity, undocumented fill soils, abandoned underground structures and/or existing utilities that may exist within the areas of construction. Any loose or disturbed soil, void spaces made by burrowing animals, or unsuitable fill should be over-excavated to expose firm conditions and/or native soil, as approved by Kleinfelder.

8.1.5 Scarification and Compaction

Following site stripping and any required grubbing and/or over-excavation, it is recommended areas to receive engineered fill be scarified to a depth of 12 inches, uniformly moisture conditioned, and recompacted. Sandy soils should be moisture-conditioned to at least optimum moisture content and clayey soils at least 3 percent above the optimum moisture content. Sandy soils should be compacted to at least 90 percent relative compaction and clayey soils at least 88 to 92 percent relative compaction, as determined by ASTM D1557. Due to some subgrade soils having medium expansion potential, the subgrade should not be allowed to dry out prior to the placement of engineered fill or the construction of pavement sections, concrete slabs-on-grade, and flatwork. In addition to scarifying and compacting, all subgrades should be proof-rolled or otherwise evaluated by a Kleinfelder representative prior to covering with engineered fill or aggregate base material.

8.2 ENGINEERED FILL

8.2.1 Materials

On-site or imported non-expansive soils that meet the criteria outlined below that are to be used for engineered fill should be uniformly moisture conditioned to at least the optimum moisture content, placed in maximum 8-inch thick loose horizontal lifts, and compacted to at least 90 percent relative compaction. Expansive on-site clay soils, if encountered, may be used within engineered fill provided they are blended to meet the requirements in Table 8.2.1-1, below.

The on-site soils are moisture sensitive and will be difficult to traverse with construction equipment and to properly compact during or following wet weather.

Environmental data provided by PACLAND's environmental consultant indicates pesticides are possibly present in the upper two feet of soils at the site. Due to these constituents, the environmental consultant provided a preliminary recommendation to 'cap' the site with clean soil to minimize contact with the pesticides and eliminate or strictly limit the need to remove the contaminated soil from the site. The presence of pesticides in soil does not adversely affect the geotechnical characteristics of the soil and no modifications of our geotechnical recommendations are necessary. We understand that areas of the site where minor cuts are planned may encounter pesticide laden soils. Safe handling of these materials should be observed during the cut and fill placement process as recommended by the client's environmental consultant.

In addition to the above requirements, specific requirements for blended or imported engineered fill and non-expansive fill, as well as applicable test procedures to evaluate material suitability are provided in Table 8.2.1-1.

Table 8.2.1-1
Engineered Fill Requirements

Fill Requirement		**Test Procedures**	
		ASTM[1]	**Caltrans**[2]
Gradation			
Sieve Size	**Percent Passing**		
3 inch	100	D422	202
¾ inch	70-100	D422	202
No. 200	10-40	D422	202
Plasticity			
Liquid Limit	**Plasticity Index**		
<40	<15	D4318	204
Organic Content			
No visible organics		---	---
Expansion Potential			---
20 or less		D4829	---
Soluble Sulfates			
Less than 2,000 ppm		---	417
Soluble Chloride			
Less than 300 ppm		---	422
Resistivity			
Greater than 2,000 ohm-cm		---	643

[1]*American Society for Testing and Materials Standards (latest edition)*
[2]*State of California, Department of Transportation, Standard Test Methods (latest edition)*

All imported fill materials to be used for engineered fill should be sampled and tested by Kleinfelder prior to being transported to the site. Highly pervious materials such as clean crushed stone or pea gravel are not recommended for use in engineered fill because they can permit transmission of water into the underlying materials. We recommend representative samples of imported materials proposed for use as engineered fill be submitted to Kleinfelder for testing and approval at least one week prior to the start of grading and import of this material.

In addition, we recommend that a laboratory corrosion test series (pH, resistivity, chlorides, and sulfates) be performed on all proposed import materials. The corrosivity of proposed import materials should be evaluated and should be no more corrosive than the on-site soils as indicated by the laboratory results presented in Appendix B.

8.2.2 Placement and Compaction Criteria

All fill to be placed below structures, pavements, or other site improvements should be considered engineered fill. Scarification and compaction criteria are outlined above in Section 6.5.5. All engineered fill should be uniformly moisture conditioned to at least the optimum moisture content, placed in horizontal lifts less than about 8 inches in loose thickness, and compacted to at least 90 percent relative compaction for fine-grained soils, and 95 percent for granular soils, as determined by ASTM D 1557.

Additional fill lifts should not be placed if the previous lift did not meet the required relative compaction or moisture content, or if soil conditions are not stable. Disking or blending may be required to uniformly moisture condition soils used for engineered fill. Ponding or jetting compaction methods should not be allowed.

All site preparation and fill placement should be observed by Kleinfelder. It is important that during the stripping and scarification processes, a representative of Kleinfelder be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during the geotechnical site exploration.

8.3 TEMPORARY EXCAVATIONS

8.3.1 General

All excavations should comply with applicable local, state, and federal safety regulations including the current Occupational Safety & Health Administration (OSHA) Excavation and Trench Safety

Standards. Construction site safety generally is the responsibility of the contractor, who is also solely responsible for the means, methods, and sequencing of construction operations.

8.3.2 Excavations and Slopes

Slope height, slope inclination, or excavation depths (including utility trench excavations) should not exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations). Such regulations are strictly enforced, and, if they are not followed, the Owner, Contractor, or earthwork and utility subcontractors could be liable for substantial penalties.

Underground utilities should be located above a 1H:1V plane projected downward from the bottoms of new footings to avoid undermining the footings during the excavation of the utility trench.

8.3.3 Trench Backfill

Trench backfill should be placed and compacted in accordance with recommendations provided in this report for engineered fill.

9 ADDITIONAL SERVICES

We anticipate that the ongoing design of this project will be an interactive process involving several members of the design team. We anticipate that our continued involvement during the design and construction phases of the project will be required for the following:

- Refining lateral spreading calculations and estimates, particularly in the southeast area of the site;
- Performing additional seismic hazard analyses, especially ground motion design criteria such as site-specific response spectra;
- Reviewing the pre-final project plans and specifications, including any revisions or modifications;
- Performing explorations and analyses to support outfall design and construction;
- Performing explorations and analyses to support swale design and construction near the levee;
- Observing and evaluating the site earthwork operations to confirm subgrade soils are suitable for construction of foundations, slabs-on-grade, pavements and placement of engineered fill;
- Evaluating engineered fill for the structure and other improvements as it is placed and compacted per the project specifications; and
- Observing and interpreting results of pile foundation installation and load "testing" programs in the pre-production phase, such as indicator and load test piles, and during installation of production piles.

Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations as construction progresses, as appropriate. We look forward to continuing our active participation in this exciting project.

10 LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by PACLAND, their consultants and partners for this project, the registered design professional in responsible charge, the owner, and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

Recommendations contained in this report are based on our field observations and subsurface explorations, laboratory tests, and our present knowledge of the proposed construction. It is possible that subsurface conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the Client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated building loads, and the design depths or locations of the foundations changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction. Furthermore, the contractor should be prepared to handle contamination conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

The work performed was based on project information provided by the Client. If the Client does not retain Kleinfelder to review any plans and specifications, including any revisions or modifications to the plans and specifications, Kleinfelder assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, Client must obtain written approval from Kleinfelder's engineer that such changes do not affect our recommendations. Failure to do so will vitiate Kleinfelder's recommendations.

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Bright People. Right Solutions.

Figures

FIGURES

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PROJECT NO. 20164522		VICINITY MAP	FIGURE
DRAWN BY:	JS		1
CHECKED BY:	MB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	
DATE:	March 2016		
REVISED:			PAGE: 1 of 1

PLOTTED: 4/27/2016 10:27 AM BY: jeff stewart

CAD FILE: G:\20164522\20164522_F1.dwg

800

Legend

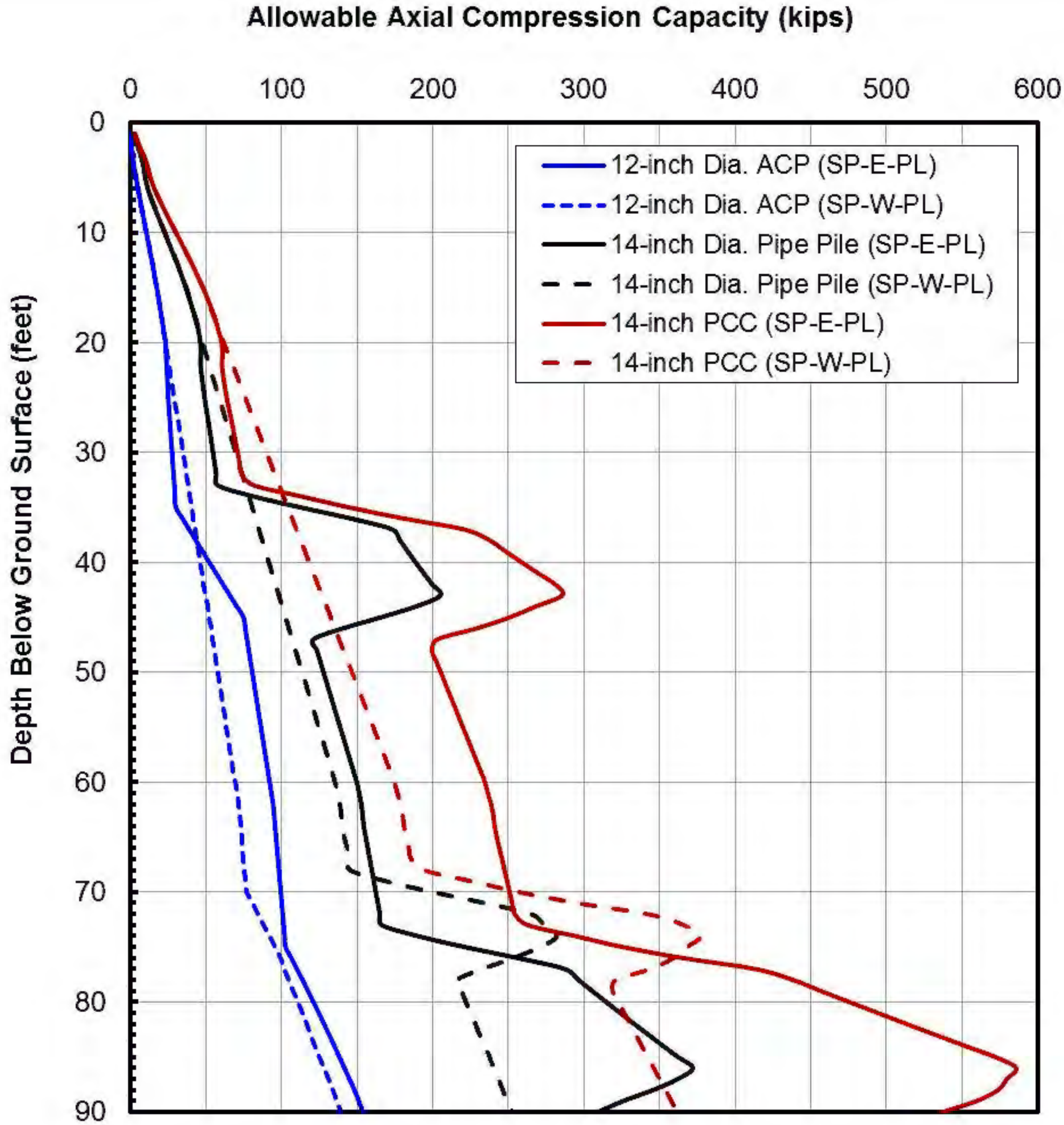
PROJECT NO. 20164522

REFERENCE: BASE MAP "C3.1 GRADING PLAN KEY MAP" PROVIDED BY PACLAND, ON APRIL 25, 2016

800

Legend

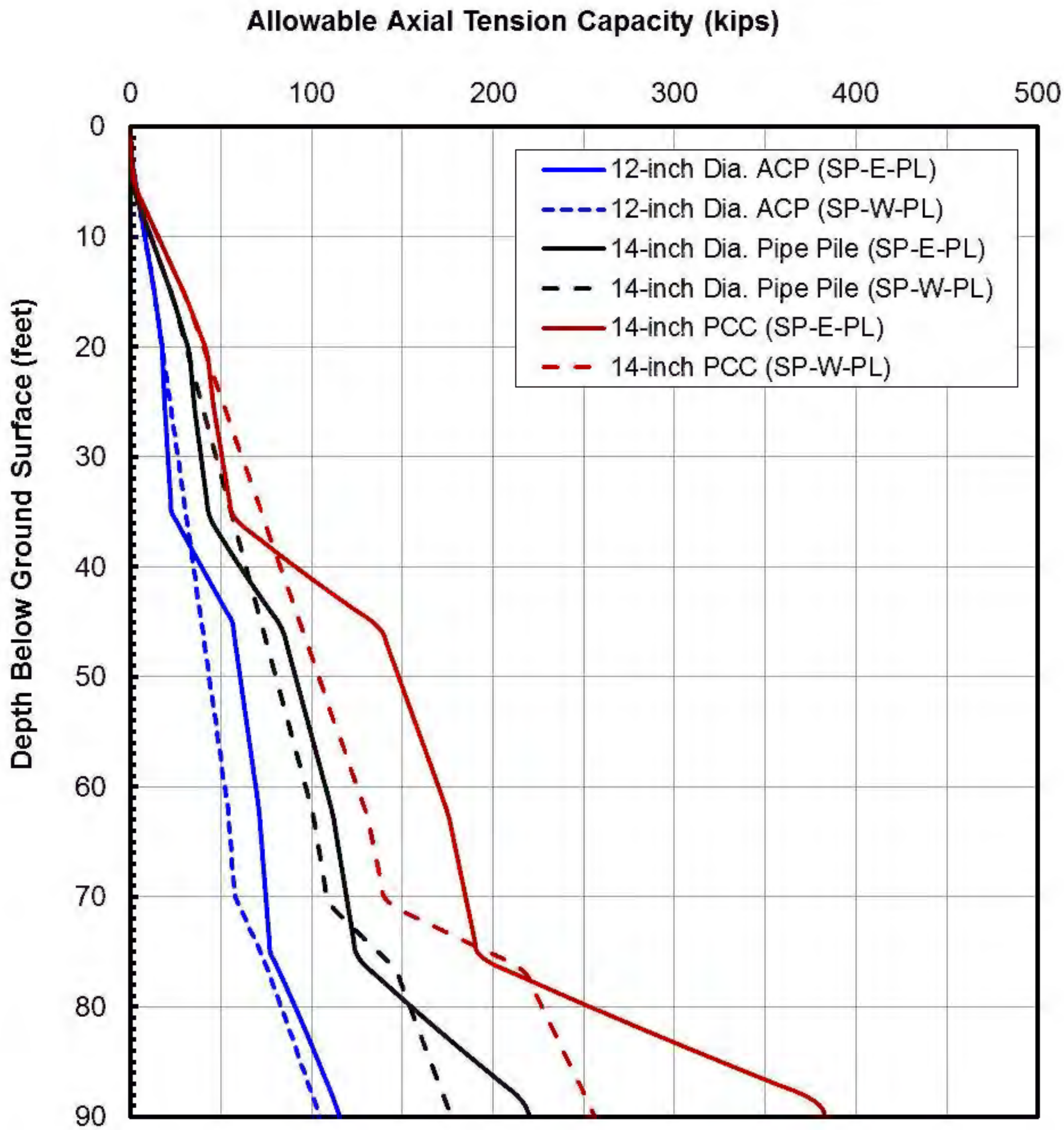
PROJECT NO. 20164522



NOTES:

1. The curves represent allowable compression values for single piles for post-liquefaction condition.
2. A safety factor of 1.5 is applied to side resistance and end bearing resistance.
3. The end bearing is excluded from the axial capacity for the 12-inch diameter auger cast piles and the end bearing is included in the axial capacity for the 14-inch diameter steel pipe piles and the 14-inch square precast concrete piles.

KLEINFELDER Bright People. Right Solutions.	**ALLOWABLE PILE COMPRESSION CAPACITY (POST-LIQUFACTION)**	DRAWN BY: J. Liao
		REVISED BY:
	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	CHECKED BY: MD
		FIGURE 4
DRAWN: 5/17/2016 / APPROVED BY: MD	PROJECT NO. 20164522 / FILE NAME:	



NOTES:

1. The curves represent allowable tension values for single piles for post-liquefaction condition.
2. A safety factor of 1.5 is applied to side resistance.
3. Tension capacity is taken as 75% of compression capacity.

KLEINFELDER Bright People. Right Solutions.		ALLOWABLE PILE TENSION CAPACITY (POST-LIQUFACTION)		DRAWN BY: J. Liao REVISED BY: CHECKED BY: MD
		PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA		FIGURE 5
DRAWN: 5/17/2016	APPROVED BY:	PROJECT NO. 20164522	FILE NAME:	

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Appendix A

APPENDIX A
FIELD EXPLORATIONS

SAMPLE/SAMPLER TYPE GRAPHICS

- BULK SAMPLE
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)

GROUND WATER GRAPHICS

- ∇ WATER LEVEL (level where first observed)
- ▼ WATER LEVEL (level after exploration completion)
- ▼ WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

				Symbol	Description
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu≥4 and 1≤Cc≤3	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu<4 and/ or 1>Cc>3	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu≥4 and 1≤Cc≤3	GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu<4 and/ or 1>Cc>3	GP-GM	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu≥6 and 1≤Cc≤3	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu<6 and/ or 1>Cc>3	SP	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu≥6 and 1≤Cc≤3	SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu<6 and/ or 1>Cc>3	SP-SM	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

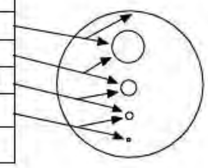
		Symbol	Description
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20164522	GRAPHICS KEY	FIGURE
DRAWN BY: JDS		
CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	A-1
DATE: 4/25/2016		
REVISED: -		

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



Munsell Color

NAME	ABBR
Red	R
Yellow Red	YR
Yellow	Y
Green Yellow	GY
Green	G
Blue Green	BG
Blue	B
Purple Blue	PB
Purple	P
Red Purple	RP
Black	N

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

Particles Present

Amount	Percentage
trace	<5
few	5-10
little	15-25
some	30-45
and	50
mostly	50-100

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

NOTE: AFTER TERZAGHI AND PECK, 1948

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (q_u)(psf)	CRITERIA
Very Soft	< 1000	Thumb will penetrate soil more than 1 in. (25 mm.)
Soft	1000 - 2000	Thumb will penetrate soil about 1 in. (25 mm.)
Firm	2000 - 4000	Thumb will indent soil about 1/4-in. (6 mm.)
Hard	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail
Very Hard	> 8000	Thumbnail will not indent soil

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20164522	SOIL DESCRIPTION KEY	FIGURE
DRAWN BY: JDS		
CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	A-2
DATE: 4/25/2016		
REVISED: -		

Date Begin - End:	3/21/2016	**Drilling Co.-Lic.#:**	Exploration Geo - #484288	**BORING LOG HSA-1**
Logged By:	J. Anderson	**Drill Crew:**	Lauren	
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:** 140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	
Weather:	Cloudy	**Exploration Diameter:**	8 in. O.D.	

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.42930° N
Longitude: -121.93500° W
Surface Condition: Muddy

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	**Sandy Lean CLAY (CL)**: fine to coarse-grained sand, medium plasticity, dark brown, moist, firm												
		1		BC=5 9 14	12"		22.2						
5	**Lean CLAY (CL)**: trace fine to coarse-grained sand, medium plasticity, dark brown, moist, firm	2		BC=8 11 18 PP=2.25	12"								
	Fat CLAY (CH): medium plasticity, dark brown, hard	3		BC=5 10 19 PP=2.5	12"		22.4						
10		4		BC=9 13 16 PP=2.75	12"								
▽	**Sandy Lean CLAY (CL)**: medium plasticity, light brown, moist, firm	5		BC=5 7 10	12"		29.5						
15		6		BC=8 10 11 PP=0.75	12"								
	increased sand content	7		BC=4 5 10	12"		28.0	97.8					
20	hard	8		BC=10 12 14 PP=2.5	6"								

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with cement grout on March 21, 2016.

GROUNDWATER LEVEL INFORMATION:
▽ Groundwater was observed at approximately 12 ft. below ground surface during drilling.

25

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20164522	BORING LOG HSA-1	FIGURE
	DRAWN BY: JDS		
	CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	A-3
	DATE: 4/25/2016		
	REVISED: -		PAGE: 1 of 1

Date Begin - End:	3/25/2016	**Drilling Co.-Lic.#:**	Exploration Geo - #484288		**BORING LOG HSA-2**
Logged By:	J. Anderson	**Drill Crew:**	Lauren		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Not Available	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION — LABORATORY RESULTS

Latitude: 37.42990° N
Longitude: -121.93300° W
Surface Condition: Muddy

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Clayey SAND (SC): medium plasticity, brown, loose												
		Sandy Lean CLAY (CL): low plasticity, dark brown, dry, hard	1		BC=4 4 6 PP=2.5	12"	CL	18.7		100	65			
5		**Fat CLAY (CH)**: high plasticity, dark brown, moist, hard	2		BC=5 8 11 PP=2.5	12"								
		Lean CLAY (CL): medium plasticity, brown, moist, hard	3		BC=5 9 14 PP=2.25	10"								
10		**Fat CLAY (CH)**: high plasticity, brown, moist, hard	4		BC=9 15 21 PP=4									
		Lean CLAY (CL): medium plasticity, brown, moist, hard	5		BC=4 10 24									
15		**Sandy Lean CLAY (CL)**: medium plasticity, light brown, moist, hard	6		BC=4 6 7									
			7		BC=4 6 10									
∇		wet												
20		**Clayey SAND (SC)**: light brown, wet, medium dense	8		BC=6 7 10			30.3						

25

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with cement grout on March 25, 2016.

GROUNDWATER LEVEL INFORMATION:
∇ Groundwater was observed at approximately 19 ft. below ground surface during drilling.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20164522	BORING LOG HSA-2	FIGURE
	DRAWN BY: JDS		
	CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	A-4
	DATE: 4/25/2016		
	REVISED: -		PAGE: 1 of 1

Date Begin - End: 3/25/2016	**Drilling Co.-Lic.#:** Exploration Geo - #484288		**BORING LOG HSA-3**
Logged By: J. Anderson	**Drill Crew:** Lauren		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** B-53	**Hammer Type - Drop:** 140 lb. Auto - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger		
Weather: Not Available	**Exploration Diameter:** 8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.42930° N
Longitude: -121.93200° W
Surface Condition: Muddy

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Clayey SAND (SC): brown, dry, medium dense												
		Lean CLAY with Sand (CL): brown, dry, firm	1		BC=4 6 7		CL				71			
		Lean CLAY (CL): medium plasticity, brown, moist, firm												
5		**Fat CLAY (CH)**: high plasticity, dark brown, moist, hard	2		BC=7 8 13 PP=3.75			22.2 21.9	98.9 101.2					Thermal Resisitivity Test - 77.15°C-cm/W 23.58°C
			3		BC=5 8 11 PP=2.25									
10			4		BC=8 15 22			23.0 22.5	105.5 104.3					Thermal Resisitivity Test - 83.00°C-cm/W 24.23°C
			5		BC=8 15 22 PP=3									
15			6		BC=9 15 14									
∇			7		BC=6 8 13 PP=2.5	6"		23.3						
20			8		BC=7 12 18	6"								

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with cement grout on March 25, 2016.

GROUNDWATER LEVEL INFORMATION:
∇ Groundwater was observed at approximately 17.5 ft. below ground surface during drilling.

25

KLEINFELDER
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PROJECT NO.: 20164522	BORING LOG HSA-3	FIGURE
DRAWN BY: JDS		
CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	A-5
DATE: 4/25/2016		
REVISED: -		PAGE: 1 of 1

Date Begin - End:	3/25/2016	**Drilling Co.-Lic.#:**	Exploration Geo - #484288	**BORING LOG HSA-4**
Logged By:	J. Anderson	**Drill Crew:**	Lauren	
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:** 140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	
Weather:	Not Available	**Exploration Diameter:**	8 in. O.D.	

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.42950° N
Longitude: -121.93100° W
Surface Condition: Muddy

Depth (feet)	Lithologic Description	Sample Number	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	Clayey SAND (SC): low plasticity, light brown, moist, medium dense											
	Silty CLAY (CL-ML): brown, moist, firm	1	BC=4 5 6		CL-ML	13.7	88.3		88	28	7	
5		2	BC=5 6 7									
	Lean CLAY (CL): medium plasticity, brown, moist, hard											
	Fat CLAY (CH): high plasticity, dark brown, moist, hard	3	BC=5 10 11 PP=2.25									
10		4	BC=5 12 21 PP=4									
	Lean CLAY (CL): medium plasticity, light brown, moist, firm	5	BC=7 8 14 PP=1									
15		6	BC=6 10 11 PP=1.5									
∇	**Clayey SAND (SC):** fine to coarse-grained sand, brown, moist, medium dense											sand in shoe
		7	BC=6 10 11			26.8						
20	coarse-grained	8	BC=4 17 20									
	some gravel											
25												

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with cement grout on March 25, 2016.

GROUNDWATER LEVEL INFORMATION:
∇ Groundwater was observed at approximately 16.5 ft. below ground surface during drilling.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20164522	BORING LOG HSA-4	FIGURE
	DRAWN BY: JDS		A-6
	CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	
	DATE: 4/25/2016		
	REVISED: -		PAGE: 1 of 1

Date Begin - End:	3/25/2016	**Drilling Co.-Lic.#:**	Exploration Geo - #484288		**BORING LOG HSA-5**
Logged By:	J. Anderson	**Drill Crew:**	Lauren		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Not Available	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.42720° N
Longitude: -121.93100° W
Surface Condition: Muddy

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Sandy Lean CLAY (CL): medium plasticity, brown, moist, firm												
			1		BC=4 5 9									
		Fat CLAY (CH): high plasticity, dark brown, moist, very hard												
5			2		BC=11 17 20 PP=>4.5			18.7 19.0	105.2 103.7					Thermal Resisitivity Test - 95.31°C-cm / W 30.26°C
			3		BC=8 13 13 PP=>4.5									
10		brown, hard	4		BC=8 13 15 PP=3.5			23.7 23.2	103.2 103.0					Thermal Resisitivity Test - 71.93°C-cm / W 30.26°C
			5		BC=6 10 16 PP=3									
15		**Sandy Lean CLAY (CL):** medium plasticity, brown, moist, hard	6		BC=8 13 15 PP=3.5									
		increasing sand content	7		BC=4 5 6	2"								
∇ 20		**Silty Clayey SAND (SC-SM):** olive brown, wet, loose	8		BC=4 5 7	18"	SC-SM				43	27	7	

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with cement grout on March 25, 2016.

GROUNDWATER LEVEL INFORMATION:
∇ Groundwater was observed at approximately 20 ft. below ground surface during drilling.

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KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20164522	BORING LOG HSA-5	FIGURE
	DRAWN BY: JDS		A-7
	CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	
	DATE: 4/25/2016		
	REVISED: -		PAGE: 1 of 1

Date Begin - End:	3/25/2016	**Drilling Co.-Lic.#:**	Exploration Geo - #484288		**BORING LOG HSA-6**
Logged By:	J. Anderson	**Drill Crew:**	Lauren		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Not Available	**Exploration Diameter:**	8 in. O.D.		

Latitude: 37.42750° N
Longitude: -121.93000° W
Surface Condition: Muddy

Depth (feet)	Graphical Log	Lithologic Description (FIELD EXPLORATION)	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol (LABORATORY RESULTS)	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Sandy Lean CLAY (CL): low plasticity, light brown, dry, hard												
		Silty CLAY (CL-ML): medium plasticity, olive brown, dry, hard	1		BC=4 5 7 PP=2.75	12"	CL-ML				94	28	7	
5		**Lean CLAY (CL)**: medium plasticity, dark brown, moist, hard	2		BC=7 15 27 PP=>4.5	11"								
		Fat CLAY (CH): high plasticity, dark brown, moist, hard	3		BC=5 16 20 PP=2.5	12"								
10			4		BC=5 16 20 PP=3	12"		22.5	104.5					
			5		BC=8 14 19 PP=3.5	5"								
15		**Lean CLAY (CL)**: medium plasticity, light brown, moist, firm	6		BC=4 8 10 PP=1.5	12"								
		low plasticity	7		BC=4 5 7 PP=1.0	12"								
20			8		BC=4 5 9	3"								

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with cement grout on March 25, 2016.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

25

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.:	20164522	BORING LOG HSA-6	FIGURE
DRAWN BY:	JDS		A-8
CHECKED BY:	EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	
DATE:	4/25/2016		
REVISED:	-		PAGE: 1 of 1

Date Begin - End:	3/25/2016	**Drilling Co.-Lic.#:**	Exploration Geo - #484288		**BORING LOG HSA-7**
Logged By:	J. Anderson	**Drill Crew:**	Lauren		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Not Available	**Exploration Diameter:**	8 in. O.D.		

Latitude: 37.42590° N
Longitude: -121.93000° W
Surface Condition: Muddy

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	Sandy Lean CLAY (CL): medium plasticity, brown, moist, firm												
	Lean CLAY with Sand (CL): medium plasticity, olive brown, moist, firm	1		BC=3 4 5 PP=2.5		CL				83	33	16	
5		2		BC=6 7 13 PP=4.5			16.9 16.4	98.1 105.3					Thermal Resisitivity Test - 93.75°C-cm / W 24.86°C
	Lean CLAY (CL): medium plasticity, dark brown, moist, very hard	3		BC=5 7 10 PP=2.75									
10	**Fat CLAY (CH)**: high plasticity, dark brown, moist, hard	4		BC=5 16 15 PP=3.5			23.9 23.3	103.7 104.7					Thermal Resisitivity Test - 80.21°C-cm / W 23.17°C
		5		BC=8 15 26 PP=3.0									
15	**Clayey SAND (SC)**: olive brown, moist, loose	6		BC=5 7 9		SC				82	31	15	
	Sandy Lean CLAY (CL): low plasticity, brown, moist, soft	7		BC=3 4 5									
20		8		BC=3 5 8									

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with cement grout on March 25, 2016.

GROUNDWATER LEVEL INFORMATION:
∇ Groundwater was observed at approximately 21.5 ft. below ground surface during drilling.

25

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20164522	BORING LOG HSA-7	FIGURE
	DRAWN BY: JDS		A-9
	CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	
	DATE: 4/25/2016		
	REVISED: -		PAGE: 1 of 1

Date Begin - End:	3/25/2016	**Drilling Co.-Lic.#:**	Exploration Geo - #484288		**BORING LOG HSA-8**
Logged By:	J. Anderson	**Drill Crew:**	Lauren		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Not Available	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.42420° N
Longitude: -121.93000° W
Surface Condition: Muddy

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	**Poorly graded GRAVEL with Sand (GP)**: brown, moist, loose												
		1		BC=4 6 8									
5	**Lean CLAY (CL)**: medium plasticity, dark brown, moist, very hard	2		BC=5 7 9 PP=4.5			21.3 20.2	97.9 98.2					Thermal Resisitivity Test - 103.11°C-cm / W 24.86°C
	Fat CLAY (CH): high plasticity, dark brown, moist, hard	3		BC=7 12 13 PP=2.75									
10		4		BC=8 11 16 PP=3.5			25.0 23.8	102.2 101.9					Thermal Resisitivity Test - 90.76°C-cm / W 21.71°C
	very hard	5		BC=6 12 17 PP=4.25									
15	hard	6		BC=6 11 15 PP=2.75									
	Sandy Lean CLAY (CL): medium plasticity, brown, moist, firm	7		BC=5 7 9									
20		8		BC=5 7 11									

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with cement grout on March 25, 2016.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 17 ft. below ground surface during drilling.

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PROJECT NO.: 20164522	BORING LOG HSA-8		FIGURE
DRAWN BY: JDS			
CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA		A-10
DATE: 4/25/2016			
REVISED: -			PAGE: 1 of 1

Date Begin - End:	3/25/2016	**Drilling Company:**	Technicon		**BORING LOG HSA-9**
Logged By:	C. Ewing	**Drill Crew:**	Brandon		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Not Available	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.42350° N
Longitude: -121.92800° W
Surface Condition: Muddy

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	**Clayey GRAVEL with Sand (GC)**: fine to coarse-grained sand, low to medium plasticity, brownish gray, dry to moist, loose, subangular to subrounded gravel to 0.75"												hand auger to 5ft
	SAND with Silt/Silty Sand (SM): fine-grained sand, non-plastic, brown to light brown, dry to moist, loose, subangular gravel to 0.75"												
5	**Silty CLAY with Sand (CL-ML)**: fine-grained sand, low plasticity, brown, dry to moist, firm, decreasing sand content with depth	1		BC=2 3 4	14"	CL-ML	11.4 19.4	92.1 97.2		82	26	7	Thermal Resisitivity Test - 86.22°C-cm / W 24.56°C
	SILT with Sand (ML): fine-grained sand, non-plastic, olive brown, moist, firm, trace subrounded gravel to 0.5"	2		BC=13 7 10 PP=1.75	15"		17.7 13.0	104.5 109.4					Thermal Resisitivity Test - 69.10°C-cm / W 23.38°C
10	**Fat CLAY (CH)**: trace fine-grained sand, dark yellowish gray, moist, firm	3		BC=5 7 11 PP=2.0	15"		24.0 23.6	100.8 101.8					Thermal Resisitivity Test - 72.74°C-cm / W 22.82°C
15		4		BC=3 4 6	9"								
20	dark brownish gray	5		BC=4 6 5	9"								
	Lean CLAY / Fat Clay (CL-CH): occasional fine-grained sand, medium to high plasticity, brown to olive yellow, firm, some silt content												

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with cement grout on March 25, 2016.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 17 ft. below ground surface at the end of drilling.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20164522	BORING LOG HSA-9	FIGURE
	DRAWN BY: JDS		
	CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	A-11
	DATE: 4/25/2016		
	REVISED: -		PAGE: 1 of 1

Date Begin - End:	3/31/2016	**Drilling Co.-Lic.#:**	Technicon - #767888		**BORING LOG MR-1**
Logged By:	J. Anderson	**Drill Crew:**	Brandon		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger / Rotary		
Weather:	Sunny	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.43050° N
Longitude: -121.93100° W
Surface Condition: High Brush

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Poorly graded SAND with Clay (SP-SC): yellowish brown, dry, loose												
			1		BC=4 4 6	12"		9.7	97.4					
5		**Silty SAND (SM):** brown, moist, medium dense	2		BC=3 6 9 PP=2.25	9"	SM	22.5	99.2		44	NP	NP	Thermal Resisitivity Test - 92.55°C-cm / W 20.97°C ambient 59°F
		Lean CLAY (CL): medium plasticity, brown, moist, hard	3		BC=3 5 6	12"								Thermal Resisitivity Test - 71.69°C-cm / W 17.69°C ambient 59°F
10			4		BC=4 9 13 PP=2.5	6"	CL	23.5			96			Begin mud rotary drilling at 10ft.
15			5		BC=6 9 4 PP=3.5	7"		24.7						
20		**Sandy SILT (ML):** fine-grained sand, olive brown, wet, soft	6		BC=2 2 4	6"	ML				57	NP	NP	
25		firm to hard	7		BC=7 7 9	12"								
30		olive gray, moist, hard	8		BC=12 13 17	8"								

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PROJECT NO.:	20164522	BORING LOG MR-1	FIGURE
DRAWN BY:	JDS		
CHECKED BY:	EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	A-12
DATE:	4/25/2016		
REVISED:	-		PAGE: 1 of 2

Date Begin - End:	3/31/2016	**Drilling Co.-Lic.#:**	Technicon - #767888		**BORING LOG MR-1**
Logged By:	J. Anderson	**Drill Crew:**	Brandon		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger / Rotary		
Weather:	Sunny	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.43050° N
Longitude: -121.93100° W
Surface Condition: High Brush

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	**Poorly graded SAND with Clay (SP-SC):** yellowish brown, dry, loose												
		1		BC=4 4 6	12"		9.7	97.4					
5	**Silty SAND (SM):** brown, moist, medium dense	2		BC=3 6 9 PP=2.25	9"	SM	22.5	99.2		44	NP	NP	Thermal Resisitivity Test - 92.55°C-cm/W 20.97°C ambient 59°F
	Lean CLAY (CL): medium plasticity, brown, moist, hard	3		BC=3 5 6	12"								Thermal Resisitivity Test - 71.69°C-cm/W 17.69°C ambient 59°F
10		4		BC=4 9 13 PP=2.5	6"	CL	23.5			96			Begin mud rotary drilling at 10ft.
15		5		BC=6 9 4 PP=3.5	7"		24.7						
20	**Sandy SILT (ML):** fine-grained sand, olive brown, wet, soft	6		BC=2 2 4	6"	ML				57	NP	NP	
25	firm to hard	7		BC=7 7 9	12"								
30	olive gray, moist, hard	8		BC=12 13 17	8"								

Date Begin - End:	3/31/2016	**Drilling Co.-Lic.#:**	Technicon - #767888		**BORING LOG MR-1**
Logged By:	J. Anderson	**Drill Crew:**	Brandon		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger / Rotary		
Weather:	Sunny	**Exploration Diameter:**	8 in. O.D.		

Latitude: 37.43050° N
Longitude: -121.93100° W
Surface Condition: High Brush

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Poorly graded SAND with Silt and Gravel (SP-SM): medium to coarse-grained sand, dark gray, moist, dense	9		BC=14 17 22	8"	SP-SM			71	5.8			
40		**Lean CLAY (CL)**: low plasticity, olive brown, moist, hard	10		BC=3 7 12	8"	CL	29.2				30	15	
45		medium plasticity	11		BC=12 14 19 PP=4	9"								
50		**Sandy Lean CLAY (CL)**: medium plasticity, olive brown, moist, hard	12		BC=15 21 24 PP=2.25		CL	17.5			53			

The boring was terminated at approximately 51.5 ft. below ground surface. The boring was backfilled with cement grout on March 31, 2016.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not measured due to drilling method.

55

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Date Begin - End:	3/30/2016 - 3/31/2016	**Drilling Company:**	Technicon		**BORING LOG MR-2**
Logged By:	J. Anderson	**Drill Crew:**	Brandon		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger / Rotary		
Weather:	Sunny	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.42830° N
Longitude: -121.93100° W
Surface Condition: High Brush

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Lean CLAY (CL): medium plasticity, brown, dry, very hard												
			1		BC=6 5 5 PP=4.5	12"		14.9	93.5					
5		**Fat CLAY (CH)**: high plasticity, brown, dry, very hard	2		BC=10 17 23 PP=4.5	6"								Thermal Resisitivity Test - 92.65°C-cm / W 13.76°C ambient 48°F
		dark brown, moist, hard	3		BC=5 8 9 PP=3.5	12"		24.6	101.1					Thermal Resisitivity Test - 88.96°C-cm / W 16.35°C ambient 48°F
10			4		BC=8 11 15 PP=3.5	8"								
15		light brown	5		BC=7 8 12 PP=3.25	8"								
		Lean CLAY (CL): medium plasticity, brown, moist, firm												
20		light olive brown	6		BC=2 3 4 PP=1.25	12"	CL	36.6 23.1	105.3					TXUU c = 1.18 ksf
		Clayey SAND (SC): fine-grained sand, dark olive brown, wet, medium dense												
25		**Lean CLAY (CL)**: medium plasticity, brown, moist, firm	7		BC=2 4 10	12"								
		Poorly graded SAND (SP): dark grayish brown, moist, loose												
30		increase in gravel content, medium dense	8		BC=12 9 13	6"								Borehole cased 0 to 30ft.

BORING LOG MR-2

Date Begin - End:	3/30/2016 - 3/31/2016	**Drilling Company:**	Technicon
Logged By:	J. Anderson	**Drill Crew:**	Brandon
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger / Rotary
Weather:	Sunny	**Exploration Diameter:**	8 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.42830° N
Longitude: -121.93100° W
Surface Condition: High Brush

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. / Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	Well-graded GRAVEL with Silt and Sand (GW-GM): fine to medium-grained sand, olive, wet, loose, fine to coarse subangular gravel, chert fragments	9		BC=15 8 4	8"	GW-GM			51	6.8			
40	gray, moist, medium dense	10		BC=10 14 16	12"								
	Lean CLAY (CL): medium plasticity, light brown to light yellowish brown, moist, firm												
45	olive brown, with gravel	11		BC=2 3 6	12"	CL	26.7				31	15	
50	**Clayey GRAVEL (GC)**: light gray, moist, loose	12		BC=4 5 5									Casing extended to 50ft.
	Lean CLAY (CL): medium plasticity, light brown to light yellowish brown, moist, firm	13		BC=6 6 8									
55	**Gravelly Lean CLAY (CL)**: olive, moist, firm, fine subrounded gravel	14		BC=5 9 12	6"	CL	22.2		65	61			
60	**Sandy Lean CLAY (CL)**: fine-grained sand, olive brown, moist, firm	15		BC=4 4 4	8"	CL	27.9				44	24	
65	**Clayey GRAVEL with Sand (GC)**: olive brown, moist, medium dense	16		BC=6 11 11	12"	GC				41			
	Silty SAND (SM): fine-grained sand, non-plastic, very dark gray, moist, medium dense												

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PROJECT NO.:	20164522
DRAWN BY:	JDS
CHECKED BY:	EMB
DATE:	4/25/2016
REVISED:	-

BORING LOG MR-2

PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA

FIGURE
A-13

Date Begin - End: 3/30/2016 - 3/31/2016	**Drilling Company:** Technicon		**BORING LOG MR-2**
Logged By: J. Anderson	**Drill Crew:** Brandon		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** CME-55	**Hammer Type - Drop:** 140 lb. Auto - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger / Rotary		
Weather: Sunny	**Exploration Diameter:** 8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.42830° N
Longitude: -121.93100° W
Surface Condition: High Brush

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Silty SAND (SM): fine-grained sand, non-plastic, very dark gray, moist, medium dense	17		BC=12 16 17	6"								
75		fine to medium sand interbedded with thin clay lenses (less than 2")	18		BC=7 13 18	12"								
														Drill rods continue to clog
80		**Well-graded GRAVEL with Sand (GW)**: medium to coarse-grained sand, gray, wet, dense, fine to coarse gravel	19		BC=15 15 22	12"								
85		**Well-graded GRAVEL with Clay and Sand (GW-GC)**: fine to medium-grained sand, dark gray, moist, dense, fine to coarse gravel	20		BC=19 19 23	12"								
		Sandy Lean CLAY (CL): fine-grained sand, low plasticity, light brownish gray, moist, firm												
90			21		BC=4 4 7	12"								
95		trace fine sand and interbedded clayey gravel , fine subrounded gravel	22		BC=15 21 25	11"								
100			23		BC=25 30 50/1" PP=3.0	3"								

The boring was terminated at approximately 101.5 ft. below ground surface. The boring was backfilled with cement grout on March 30, 2016.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not measured due to drilling method.

Date Begin - End:	3/31/2016	Drilling Company:	Technicon		BORING LOG MR-3
Logged By:	O. Khan	Drill Crew:	Brandon		
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	CME-55	Hammer Type - Drop:	140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger / Rotary		
Weather:	Clear/Cool	Exploration Diameter:	8 in. O.D.		

Latitude: 37.42640° N
Longitude: -121.92900° W
Surface Condition: High Brush

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Lean CLAY (CL): trace fine-grained sand, medium plasticity, dark yellowish brown, moist, firm, trace rootlets												
		brown, hard, increase in silt content	1		BC=4 5 8 PP=>4.5	12"	CL	22.3	98.9		97	33	14	Thermal Resisitivity Test - 84.52°C-cm / W 25.28°C ambient 63°
5		dark yellowish brown, decrease in sand content	2		BC=9 14 17 PP=>4.5	12"		21.2	102.8					
		Lean CLAY (CL): trace fine-grained sand, medium plasticity, dark olive brown, moist, firm	3		BC=6 7 10 PP=2.75	15"		24.4	104.8					Thermal Resisitivity Test - 78.37°C-cm / W 19.87°C ambient 63°
10		olive brown, with sand	4		BC=4 9 12 PP=2.75	9"		21.9	105.4					Borehole cased 0 to 10ft. TXUU c = 1.38 ksf
15		olive to olive brown, mottled with iron and manganese oxide stains	5		BC=4 8 10 PP=2.0	9"								
		Sandy Lean CLAY (CL): fine-grained sand, medium plasticity, brown, moist, firm, mottled with manganese oxide staines and iron oxide stains												
20			6		BC=4 6 7 PP=1.75	12"	CL	22.5			70	38	18	
25		yellowish brown to olive brown, increase in sand content, oxidized	7		BC=5 4 4 PP=1.0									
		Clayey SAND (SC): fine-grained sand, dark yellowish brown, moist, loose												
		Lean CLAY with Sand (CL): fine-grained sand, medium plasticity, yellowish brown, moist, firm												
30			8		BC=3 13 3	18"	CL	23.7			78			

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20164522	BORING LOG MR-3
	DRAWN BY: JDS	
	CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA
	DATE: 4/25/2016	
	REVISED: -	

FIGURE A-14

Date Begin - End:	3/31/2016	**Drilling Company:**	Technicon		**BORING LOG MR-3**
Logged By:	O. Khan	**Drill Crew:**	Brandon		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger / Rotary		
Weather:	Clear/Cool	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION — Latitude: 37.42640° N; Longitude: -121.92900° W; Surface Condition: High Brush

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. / Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	Lean CLAY with Sand (CL): fine-grained sand, medium plasticity, yellowish brown, moist, firm	9		BC=6 7 10 PP=3.0	9"	CL	20.9	108.8					TXUU c = 1.00 ksf
	medium plasticity, olive brown, hard, increase in sand content												
	Sandy Lean CLAY (CL): fine-grained sand, dark yellowish brown												
40		10		BC=2 4 5	15"								
	Poorly graded SAND with Clay (SP-SC): fine-grained sand, dark yellowish brown, wet, loose												
	Lean CLAY with Sand (CL): fine-grained sand, medium plasticity, olive brown, moist, soft to firm												
45	firm	11		BC=3 4 6	18"	CL	24.7		100	81	28	12	
50	medium plasticity, olive, moist, firm, interbedded with lens of clayey sand about 6" thick, mottled with carbonate nodules	12		BC=3 4 5	18"								
	The boring was terminated at approximately 51.5 ft. below ground surface. The boring was backfilled with cement grout on March 31, 2016.												
55													
60													
65													

GROUNDWATER LEVEL INFORMATION:
Groundwater was not measured due to drilling method.

KLEINFELDER
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PROJECT NO.: 20164522	BORING LOG MR-3		FIGURE
DRAWN BY: JDS			
CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA		A-14
DATE: 4/25/2016			
REVISED: -			PAGE: 2 of 2

Date Begin - End:	4/01/2016	**Drilling Company:**	Technicon		**BORING LOG MR-4**
Logged By:	J. Anderson	**Drill Crew:**	Brandon		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger / Rotary		
Weather:	Sunny	**Exploration Diameter:**	8 in. O.D.		

Latitude: 37.42500° N
Longitude: -121.93000° W
Surface Condition: High Brush

Depth (feet)	Graphical Log	FIELD EXPLORATION: Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Poorly graded SAND with Clay (SP-SC): fine-grained sand, non-plastic, olive, dry, medium dense												
			1		BC=6 8 12	10"		22.3	98.7					
5		**Lean CLAY with Sand (CL):** olive, dry, firm	2		BC=3 5 5	9"	CL	21.9	100.5	100	80			Thermal Resisitivity Test - 95.72°C-cm / W 13.46°C ambient = 50° F
		Sandy Lean CLAY (CL): fine-grained sand, low plasticity, olive, moist, firm	3		BC=2 3 5	4"								Thermal Resisitivity Test - 71.90°C-cm / W 14.16°C ambient = 50° F
10		**Clayey SAND (SC):** fine-grained sand, dark brown, wet, loose	4		BC=3 3 6	12"	SC	24.5	102.0		33			Switched to Mud Rotary at 10ft.
			5		BC=1 2 2	8"								
15		**Fat CLAY (CH):** high plasticity, pale olive, wet, hard	6		BC=4 4 6	8"		24.5						
20		**Silty SAND (SM):** fine-grained sand, olive brown, wet, loose	7		BC=2 3 7	8"	SM	23.7	107.9		48	NP	NP	
25		non-plastic, medium dense	8		BC=4 5 7	5"								
		Poorly graded SAND (SP): medium-grained sand, non-plastic, olive brown, moist, medium dense	9		BC=8 18 11	8"								
30		increased gravel content	10		BC=13 13 15	8"								

KLEINFELDER
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PROJECT NO.: 20164522	BORING LOG MR-4		FIGURE
DRAWN BY: JDS			
CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA		A-15
DATE: 4/25/2016			
REVISED: -			PAGE: 1 of 2

PLOTTED: 05/20/2016 07:56 AM BY: JSala

Date Begin - End:	4/01/2016	**Drilling Company:**	Technicon		**BORING LOG MR-4**
Logged By:	J. Anderson	**Drill Crew:**	Brandon		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger / Rotary		
Weather:	Sunny	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION

Latitude: 37.42500° N
Longitude: -121.93000° W
Surface Condition: High Brush

Lithologic Description

Well-graded SAND with Silt and Gravel (SW-SM): medium to coarse-grained, non-plastic, olive gray, moist, medium dense, fine gravel

Lean CLAY with Sand (CL): medium plasticity, yellowish brown, moist, hard

Poorly graded SAND with Clay (SP-SC): low plasticity, olive, moist, medium dense, interbedded. At 48.5, firm, medium plasticity clay lens

Fat CLAY (CH): high plasticity, olive to olive gray, moist, hard

The boring was terminated at approximately 51.5 ft. below ground surface. The boring was backfilled with cement grout on April 01, 2016.

Depth (feet): 40, 45, 50, 55, 60, 65

Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. / Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
11		BC=15 15 14	8"	SW-SM			61	8.4			
12		BC=17 14 14	8"								
13		BC=5 4 6	8"	CL	25.8				31	14	
14		BC=5 7 8 PP=1.25	12"								
15		BC=15 16 15 PP=2.0	12"								

LABORATORY RESULTS

GROUNDWATER LEVEL INFORMATION:
Groundwater was not measured due to drilling method.

KLEINFELDER *Bright People. Right Solutions.*	PROJECT NO.: 20164522	BORING LOG MR-4	FIGURE
	DRAWN BY: JDS		
	CHECKED BY: EMB	PACLAND PROJECT 1926 SAN JOSE, CALIFORNIA	A-15
	DATE: 4/25/2016		
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Appendix B

APPENDIX B
LABORATORY TEST RESULTS

Exploration ID	Depth (ft.)	Sample No.	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
						Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
HSA-1	3.0			22.2								
HSA-1	8.0			22.4								
HSA-1	13.0			29.5								
HSA-1	18.0			28.0	97.8							
HSA-2	3.0		DARK BROWN SANDY LEAN CLAY (CL)	18.7			100	65				
HSA-2	21.0			30.3								
HSA-3	3.0		BROWN LEAN CLAY WITH SAND (CL)					71				
HSA-3	5.5			22.2	98.9							
HSA-3	6.0			21.9	101.2							
HSA-3	10.5			23.0	105.5							
HSA-3	11.0			22.5	104.3							
HSA-3	18.0			23.3								
HSA-4	3.0		BROWN SILTY CLAY (CL-ML)	13.7	88.3			88	28	21	7	
HSA-4	18.0			26.8								
HSA-5	5.5			18.7	105.2							
HSA-5	6.0			19.0	103.7							
HSA-5	10.5			23.7	103.2							
HSA-5	11.0			23.2	103.0							
HSA-5	20.0	8	OLIVE BROWN SILTY CLAYEY SAND (SC-SM)					43	27	20	7	
HSA-6	3.0		OLIVE BROWN SILTY CLAY (CL-ML)					94	28	21	7	
HSA-6	11.0			22.5	104.5							
HSA-7	3.0		OLIVE BROWN LEAN CLAY WITH SAND (CL)					83	33	17	16	
HSA-7	5.5			16.9	98.1							
HSA-7	6.0			16.4	105.3							
HSA-7	10.5			23.9	103.7							
HSA-7	11.0			23.3	104.7							
HSA-7	16.0		OLIVE BROWN CLAYEY SAND (SC)					82	31	16	15	
HSA-8	5.5			21.3	97.9							

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic
NA = Not Available

KLEINFELDER
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DATE: 4/25/2016
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LABORATORY TEST RESULT SUMMARY

PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA

FIGURE
B-1

Exploration ID	Depth (ft.)	Sample No.	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
						Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
HSA-8	6.0			20.2	98.2							
HSA-8	10.5			25.0	102.2							
HSA-8	11.0			23.8	101.9							
HSA-9	5.5			11.4	92.1							
HSA-9	6.0		BROWN SILTY CLAY WITH SAND (CL-ML)	19.4	97.2			82	26	19	7	
HSA-9	8.0			17.7	104.5							
HSA-9	8.5			13.0	109.4							
HSA-9	10.5			24.0	100.8							
HSA-9	11.0			23.6	101.8							
MR-1	3.5			9.7	97.4							
MR-1	6.0		BROWN SILTY SAND (SM)	22.5	99.2			44	NP	NP	NP	
MR-1	11.0		BROWN LEAN CLAY (CL)	23.5				96				
MR-1	16.0			24.7								
MR-1	21.0		OLIVE BROWN SANDY SILT (ML)					57	NP	NP	NP	
MR-1	36.0		DARK GRAY POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)			94	71	5.8				
MR-1	41.0		OLIVE BROWN LEAN CLAY (CL)	29.2					30	15	15	
MR-1	51.0		OLIVE BROWN SANDY LEAN CLAY (CL)	17.5				53				
MR-2	3.5			14.9	93.5							
MR-2	8.5			24.6	101.1							
MR-2	20.5			36.6								
MR-2	21.0		LIGHT OLIVE BROWN LEAN CLAY (CL)	23.1	105.3							TXUU c = 1.18 ksf
MR-2	36.0		OLIVE WELL GRADED GRAVEL WITH SILTY AND SAND (GW-GM)			87	51	6.8				
MR-2	46.0		OLIVE BROWN LEAN CLAY WITH GRAVEL (CL)	26.7					31	16	15	
MR-2	56.0		OLIVE GRAVELLY LEAN CLAY (CL)	22.2		100	65	61				
MR-2	60.5		OLIVE BROWN SANDY LEAN CLAY (CL)	27.9					44	20	24	
MR-2	66.0		OLIVE BROWN CLAYEY GRAVEL WITH SAND (GC)					41				

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic
NA = Not Available

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LABORATORY TEST RESULT SUMMARY

PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA

FIGURE
B-2

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PLOTTED: 04/27/2016 10:50 AM BY: JSala

Exploration ID	Depth (ft.)	Sample No.	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
						Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
MR-3	3.0		BROWN LEAN CLAY (CL)	22.3	98.9			97	33	19	14	
MR-3	5.5			21.2	102.8							
MR-3	8.5			24.4	104.8							
MR-3	11.0		OLIVE BROWN LEAN CLAY WITH SAND (CL)	21.9	105.4							TXUU c = 1.38 ksf
MR-3	21.0		BROWN SANDY LEAN CLAY (CL)	22.5				70	38	20	18	
MR-3	30.0	8	YELLOWISH BROWN LEAN CLAY WITH SAND (CL)	23.7				78				
MR-3	36.0		OLIVE BROWN LEAN CLAY WITH SAND (CL)	20.9	108.8							TXUU c = 1.00 ksf
MR-3	45.0	11	OLIVE BROWN LEAN CLAY WITH SAND (CL)	24.7			100	81	28	16	12	
MR-4	3.5			22.3	98.7							
MR-4	6.0		BROWN LEAN CLAY WITH SAND (CL)	21.9	100.5		100	80				
MR-4	11.0		DARK BROWN CLAYEY SAND (SC)	24.5	102.0			33				
MR-4	16.0			24.5								
MR-4	21.0		OLIVE BROWN SILTY SAND (SM)	23.7	107.9			48	NP	NP	NP	
MR-4	36.0		OLIVE GRAY WELL GRADED SAND WITH SILT AND GRAVEL			98	61	8.4				
			(SW-SM)									
MR-4	45.0	13	YELLOWISH BROWN LEAN CLAY WITH SAND (CL)	25.8					31	17	14	

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic
NA = Not Available

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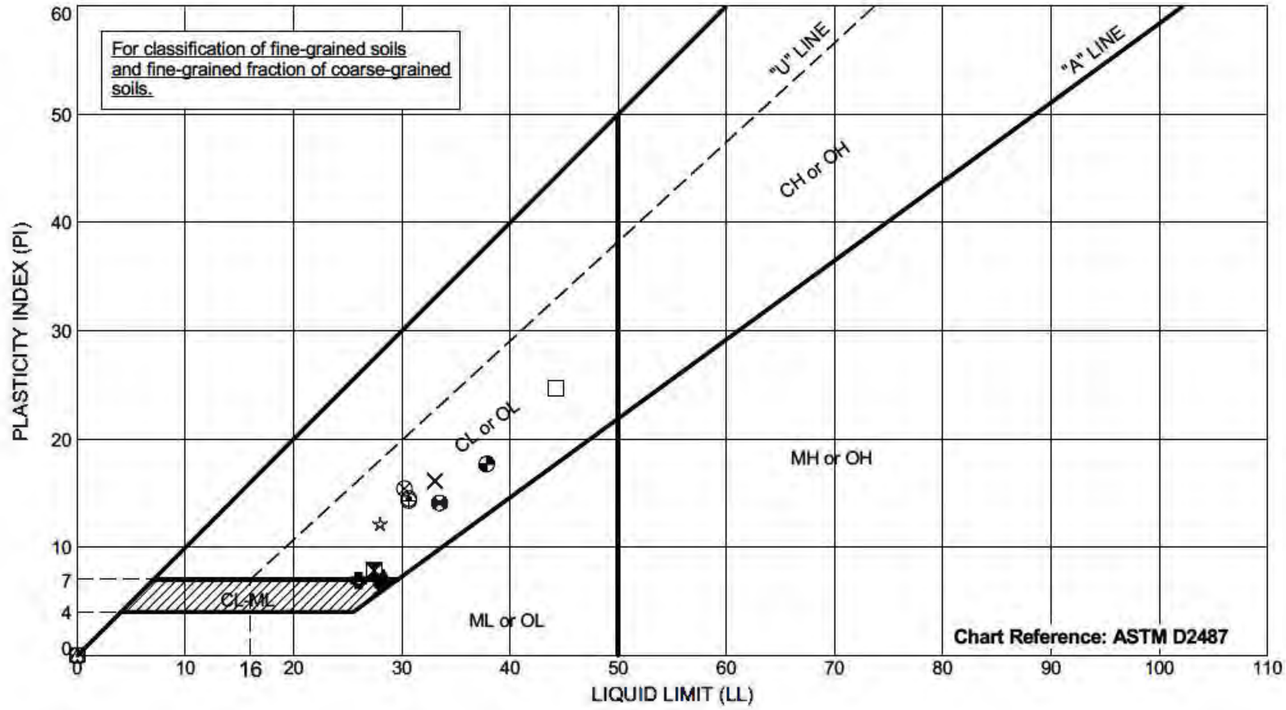
LABORATORY TEST RESULT SUMMARY

PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA

FIGURE

B-3



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	Passing #200	LL	PL	PI
●	HSA-4	3	NA	BROWN SILTY CLAY (CL-ML)	88	28	21	7
⧗	HSA-5	20	8	OLIVE BROWN SILTY CLAYEY SAND (SC-SM)	43	27	20	7
▲	HSA-6	3	NA	OLIVE BROWN SILTY CLAY (CL-ML)	94	28	21	7
×	HSA-7	3	NA	OLIVE BROWN LEAN CLAY WITH SAND (CL)	83	33	17	16
⊙	HSA-7	16	NA	OLIVE BROWN CLAYEY SAND (SC)	82	31	16	15
✿	HSA-9	6	NA	BROWN SILTY CLAY WITH SAND (CL-ML)	82	26	19	7
○	MR-1	6	NA	BROWN SILTY SAND (SM)	44	NP	NP	NP
△	MR-1	21	NA	OLIVE BROWN SANDY SILT (ML)	57	NP	NP	NP
⊗	MR-1	41	NA	OLIVE BROWN LEAN CLAY (CL)	NM	30	15	15
⊕	MR-2	46	NA	OLIVE BROWN LEAN CLAY WITH GRAVEL (CL)	NM	31	16	15
□	MR-2	60.5	NA	OLIVE BROWN SANDY LEAN CLAY (CL)	NM	44	20	24
◒	MR-3	3	NA	BROWN LEAN CLAY (CL)	97	33	19	14
◕	MR-3	21	NA	BROWN SANDY LEAN CLAY (CL)	70	38	20	18
☆	MR-3	45	11	OLIVE BROWN LEAN CLAY WITH SAND (CL)	81	28	16	12
⌘	MR-4	21	NA	OLIVE BROWN SILTY SAND (SM)	48	NP	NP	NP

Testing perfomed in general accordance with ASTM D4318.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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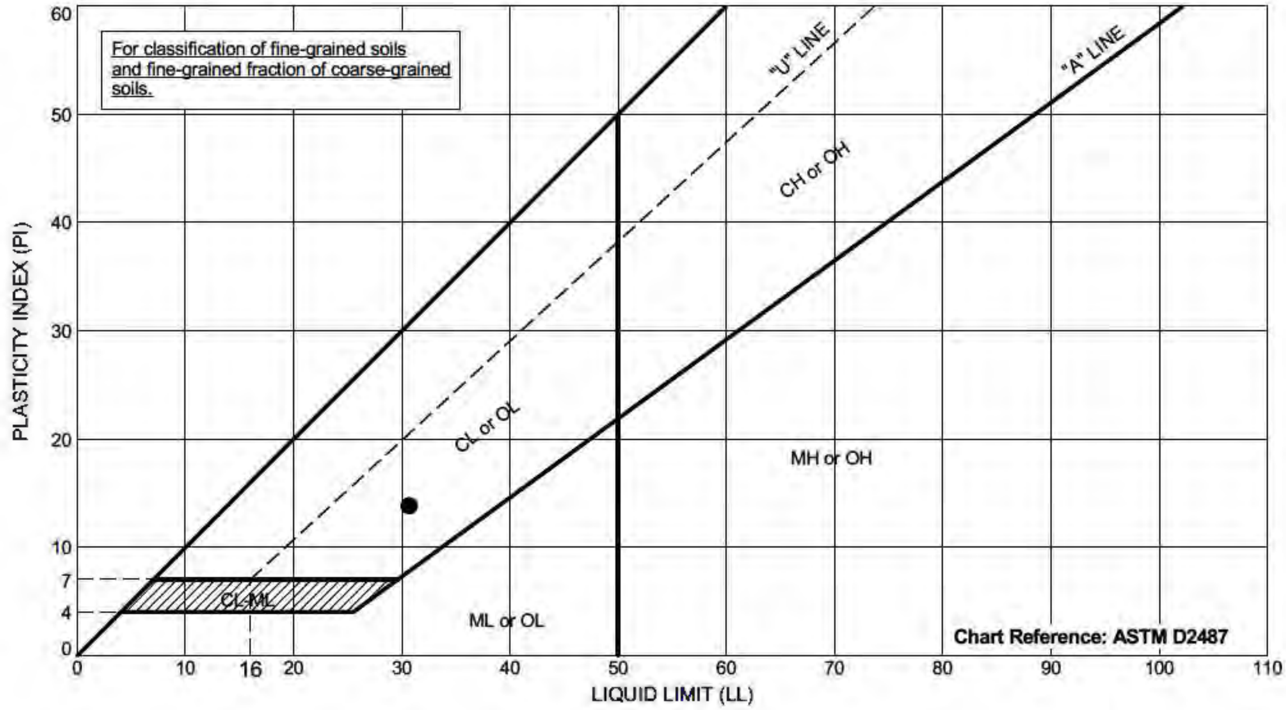
ATTERBERG LIMITS

PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA

B-4



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	Passing #200	LL	PL	PI
●	MR-4	45	13	YELLOWISH BROWN LEAN CLAY WITH SAND (CL)	NM	31	17	14

Testing perfomed in general accordance with ASTM D4318.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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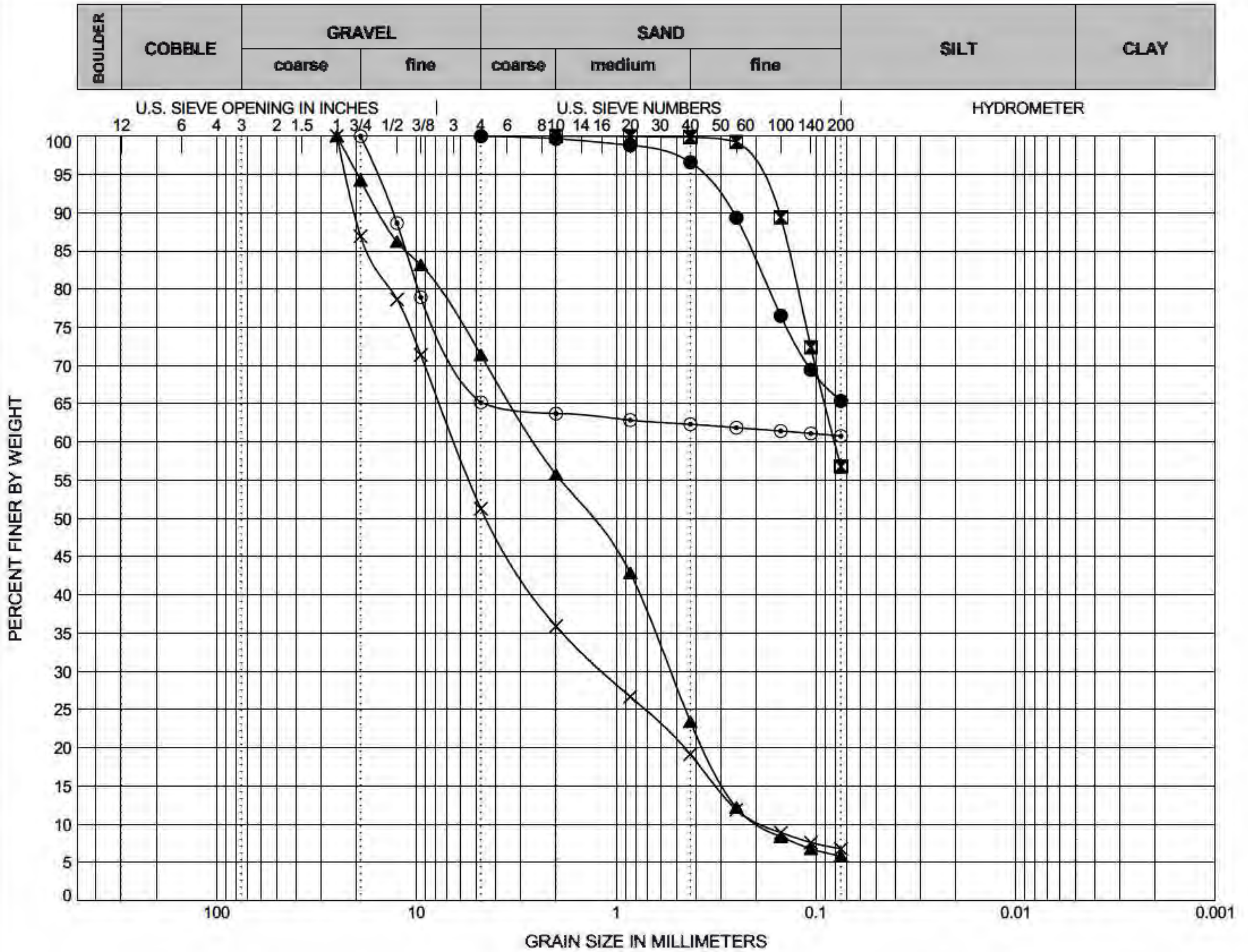
ATTERBERG LIMITS

PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA

B-5



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	LL	PL	PI
●	HSA-2	3	NA	DARK BROWN SANDY LEAN CLAY (CL)	NM	NM	NM
⧗	MR-1	21	NA	OLIVE BROWN SANDY SILT (ML)	NP	NP	NP
▲	MR-1	36	NA	DARK GRAY POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)	NM	NM	NM
×	MR-2	36	NA	OLIVE WELL GRADED GRAVEL WITH SILTY AND SAND (GW-GM)	NM	NM	NM
⊙	MR-2	56	NA	OLIVE GRAVELLY LEAN CLAY (CL)	NM	NM	NM

	Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt	%Clay
●	HSA-2	3	4.75	NM	NM	NM	NM	NM		100	65	NM	NM
⧗	MR-1	21	2	0.081	NM	NM	NM	NM			57	NM	NM
▲	MR-1	36	25	2.53	0.537	0.187	0.61	13.55	94	71	5.8	NM	NM
×	MR-2	36	25	6.413	1.155	0.18	1.15	35.58	87	51	6.8	NM	NM
⊙	MR-2	56	19	NM	NM	NM	NM	NM	100	65	61	NM	NM

Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D422.
NP = Nonplastic
NA = Not Available
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60} D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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SIEVE ANALYSIS

PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA

B-6

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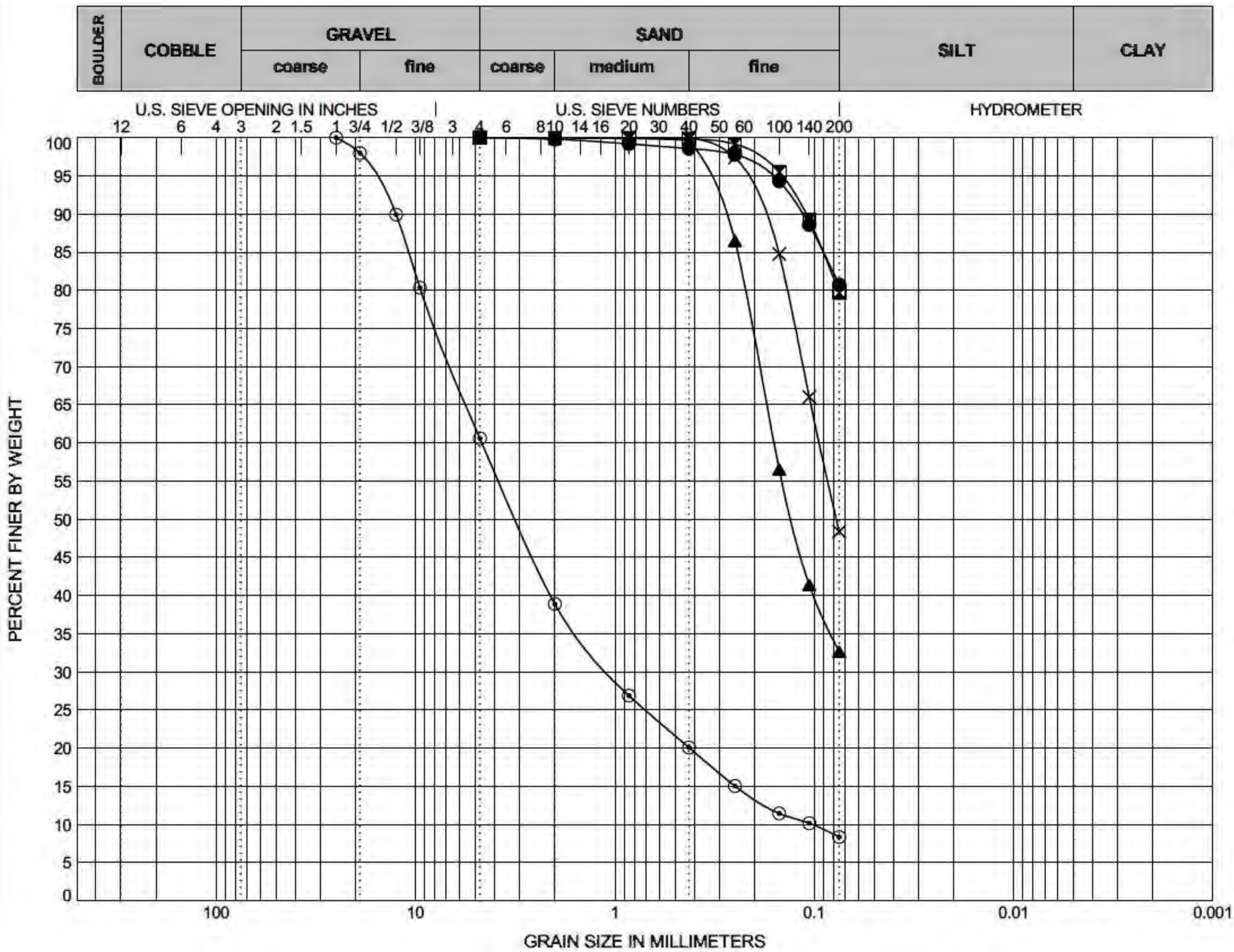

Exploration ID		Depth (ft.)	Sample Number	Sample Description	LL	PL	PI
●	MR-3	45	11	OLIVE BROWN LEAN CLAY WITH SAND (CL)	28	16	12
⧗	MR-4	6	NA	BROWN LEAN CLAY WITH SAND (CL)	NM	NM	NM
▲	MR-4	11	NA	DARK BROWN CLAYEY SAND (SC)	NM	NM	NM
×	MR-4	21	NA	OLIVE BROWN SILTY SAND (SM)	NP	NP	NP
⊙	MR-4	36	NA	OLIVE GRAY WELL GRADED SAND WITH SILT AND GRAVEL (SW-SM)	NM	NM	NM

Exploration ID		Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt	%Clay
●	MR-3	45	4.75	NM	NM	NM	NM	NM		100	81	NM	NM
⧗	MR-4	6	4.75	NM	NM	NM	NM	NM		100	80	NM	NM
▲	MR-4	11	0.85	0.159	NM	NM	NM	NM			33	NM	NM
×	MR-4	21	0.85	0.094	NM	NM	NM	NM			48	NM	NM
⊙	MR-4	36	25	4.637	1.061	0.103	2.37	45.21	98	61	8.4	NM	NM

Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D422.
NP = Nonplastic
NA = Not Available
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60} D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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SIEVE ANALYSIS

PACLAND PROJECT 1926
SAN JOSE, CALIFORNIA

B-7

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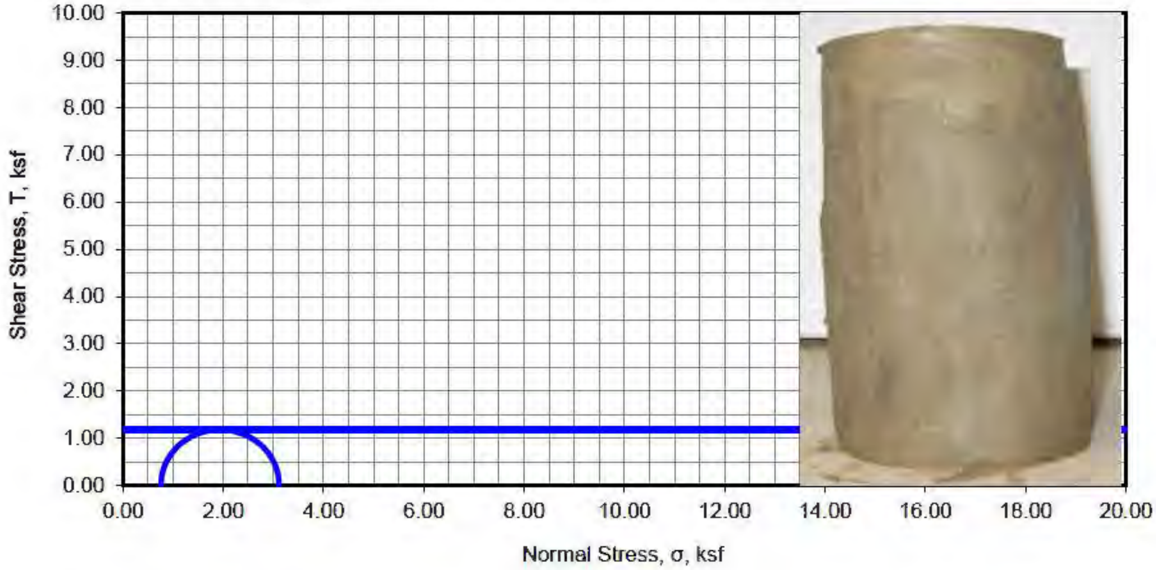
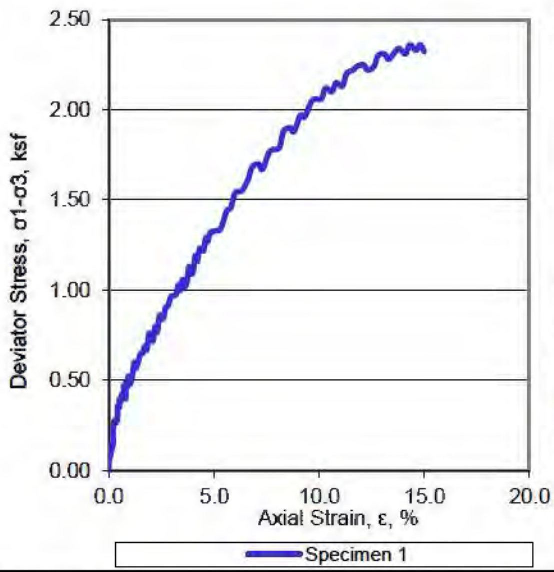
Total	
c = 1.18	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_o	2.37
	Height, in	H_o	5.08
	Water Content, %	ω_o	23.1
	Dry Density, lbs/ft^3	γ_{d_o}	105.3
	Saturation, %	S_o	107
	Void Ratio	e_o	0.571
Minor Principal Stress, ksf		σ_3	0.76
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	2.36
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.83
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	2.32
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$\dot{\varepsilon}$	1.00
Axial Strain at Failure, %		ε_f	14.83

Description of Specimen: Light Olive Brown Lean Clay (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm | PL: nm | PI: nm | G_s: 2.65 Assumed | Specimen Type: Undisturbed | Test Method: ASTM D2850

Membrane correction applied

Boring:	MR-2
Sample:	6C
Depth, ft:	21.0
Test Date:	4/14/16

Remarks: nm= not measured, na = not applicable

* **Sample saturated prior to testing.**

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2601 Barrington Ct, Hayward, CA 94545

Project No.:	20164522
Date:	4/19/16
Entry By:	CP
Checked By:	CP
File Name:	HL8704

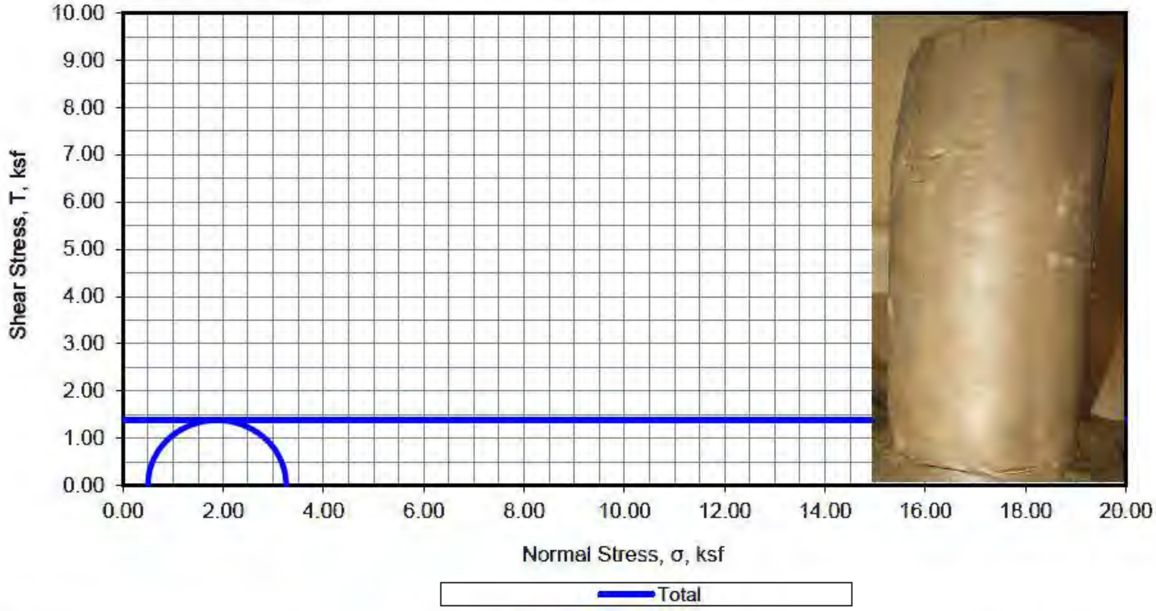
TRIAXIAL COMPRESSION TEST (UU)

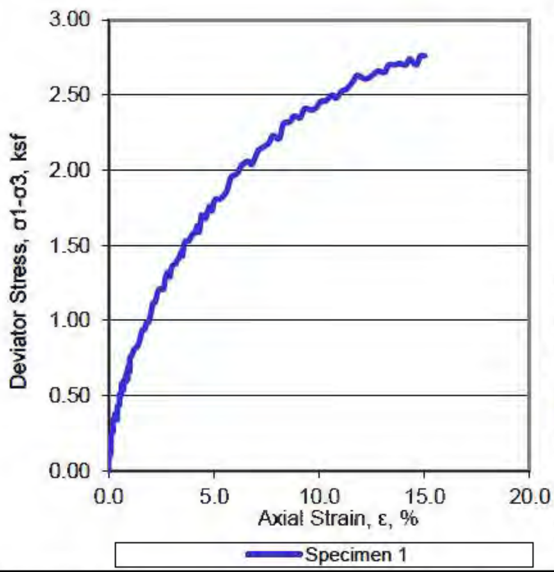
PACLAND PROJECT 1926
SAN JOSE, CALFORNIA

FIGURE
B-8

Total	
c = 1.38	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_o	2.40
	Height, in	H_o	5.62
	Water Content, %	ω_o	21.9
	Dry Density, lbs/ft^3	γ_{d_o}	105.4
	Saturation, %	S_o	102
	Void Ratio	e_o	0.569
Minor Principal Stress, ksf		σ_3	0.50
Maximum Deviator Stress, ksf		$(\sigma 1-\sigma 3)_{max}$	2.76
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	15.02
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	2.76
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$\dot{\varepsilon}$	1.00
Axial Strain at Failure, %		ε_f	15.02

Description of Specimen: Olive Brown Lean Clay with Sand (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm | PL: nm | PI: nm | G_s: 2.65 Assumed | Specimen Type: Undisturbed | Test Method: ASTM D2850

Membrane correction applied

Boring:	MR-3
Sample:	4C
Depth, ft:	11.0
Test Date:	4/15/16

Remarks: nm= not measured, na = not applicable

* **Sample saturated prior to testing.**

KLEINFELDER
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2601 Barrington Ct, Hayward, CA 94545

Project No.:	20164522
Date:	4/19/16
Entry By:	CP
Checked By:	CP
File Name:	HL8704

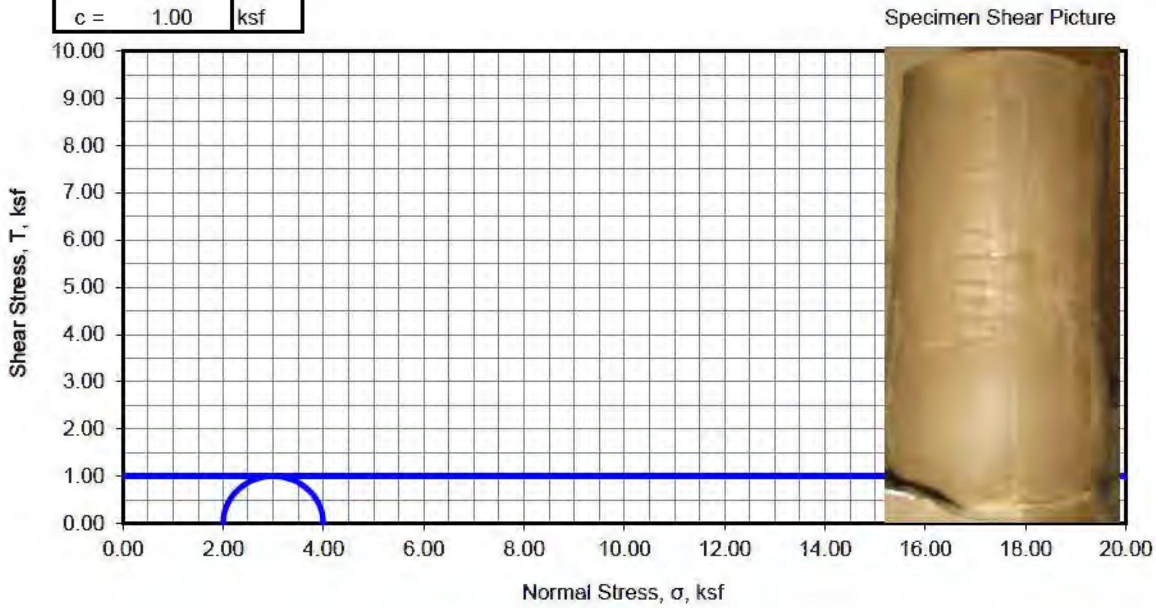
TRIAXIAL COMPRESSION TEST (UU)

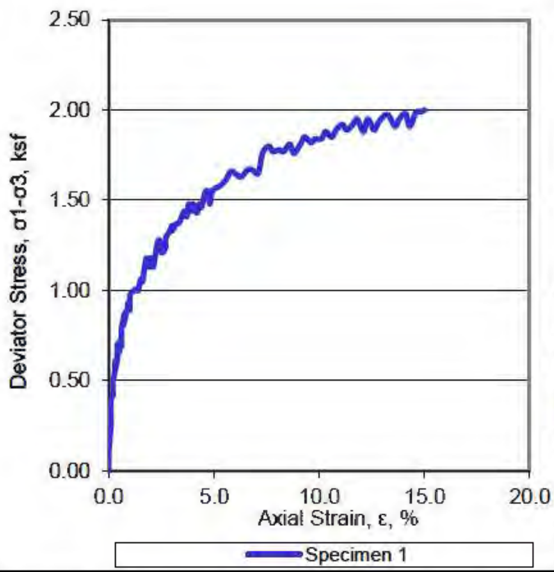
PACLAND PROJECT 1926
SAN JOSE, CALFORNIA

FIGURE

B-9

Total	
c = 1.00	ksf





Specimen No.			1
Initial	Diameter, in	D_o	2.39
	Height, in	H_o	5.67
	Water Content, %	ω_o	20.9
	Dry Density, lbs/ft^3	γ_{d_o}	108.8
	Saturation, %	S_o	106
	Void Ratio	e_o	0.520
Minor Principal Stress, ksf		σ_3	2.00
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	2.00
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	15.02
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	2.00
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$\dot{\varepsilon}$	1.00
Axial Strain at Failure, %		ε_f	15.02

Description of Specimen: Olive Brown Lean Clay with Sand (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm | PL: nm | PI: nm | G_S: 2.65 Assumed | Specimen Type: Undisturbed | Test Method: ASTM D2850

Membrane correction applied

Boring:	MR-3
Sample:	9C
Depth, ft:	36.0
Test Date:	4/15/16

Remarks: nm= not measured, na = not applicable

*** Sample saturated prior to testing.**

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2601 Barrington Ct, Hayward, CA 94545

Project No.:	20164522
Date:	4/19/16
Entry By:	CP
Checked By:	CP
File Name:	HL8704

TRIAXIAL COMPRESSION TEST (UU)

PACLAND PROJECT 1926
SAN JOSE, CALFORNIA

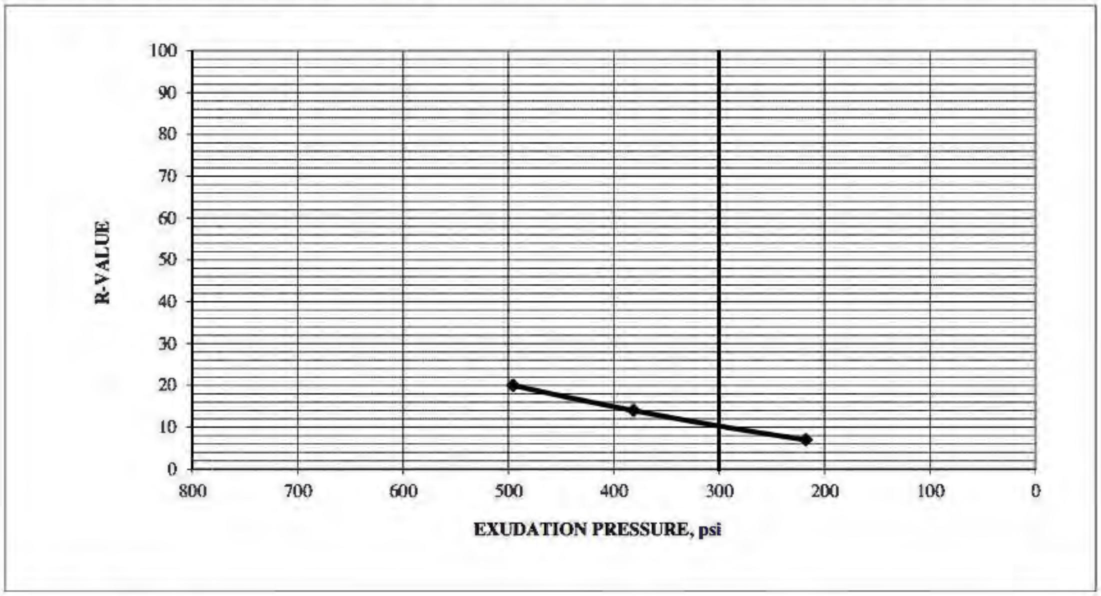
FIGURE

B-10

Laboratory Test Report

Project Name: PACLAND PROJECT 1926
Project No.: 20164522.001A
Lab No.: HL8703
Sample Date: March 31, 2016
Sample No.: HSA-1 @ 0-5'
Sample Location: HSA-1 @ 0-5'
Material Description: Lean Clay with Sand
Report Date: April 15, 2016

Resistance R-Value and Expansion Pressure of Compacted Soils (ASTM D2844, CTM 301)



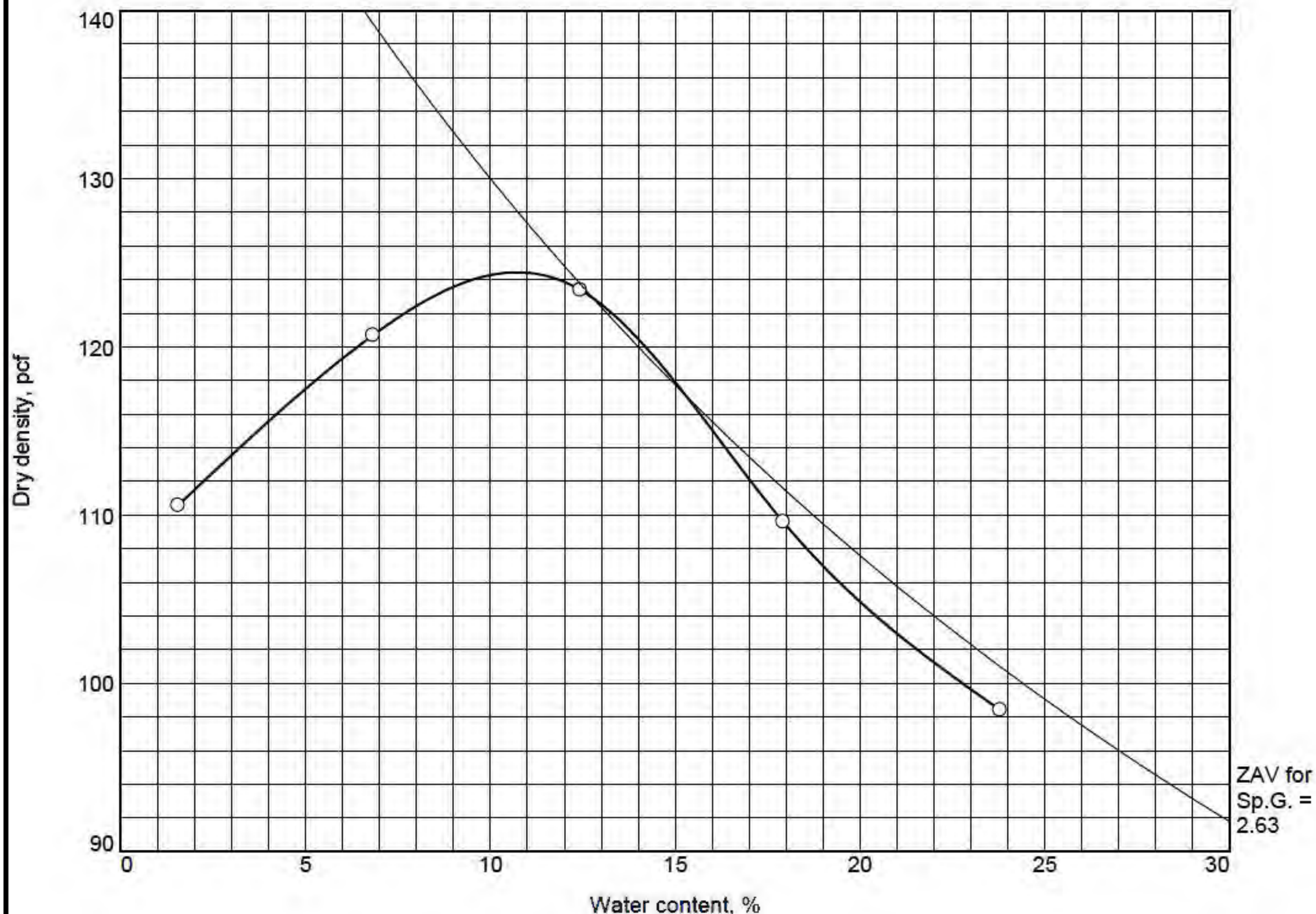
Briquette No.	A	B	C
Moisture at Test, %	16.5	17.8	19.2
Dry Unit Weight at Test, pcf	114.7	111.4	108.3
Expansion Pressure, psf	394	247	139
Exudation Pressure, psi	495	381	218
Resistance Value	20	14	7
R - Value at 300 psi Exudation Pressure:			**10**

Reviewed By on 4/15/2016:

for **Aaron Kidd**
Laboratory Manager

Limitations: Pursuant to applicable building codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided.

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0'-5'			21.1					

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 124.4 pcf Optimum moisture = 10.7 %	

Project No. 20164522 **Client:** PacLand

Project: Pacland 1926

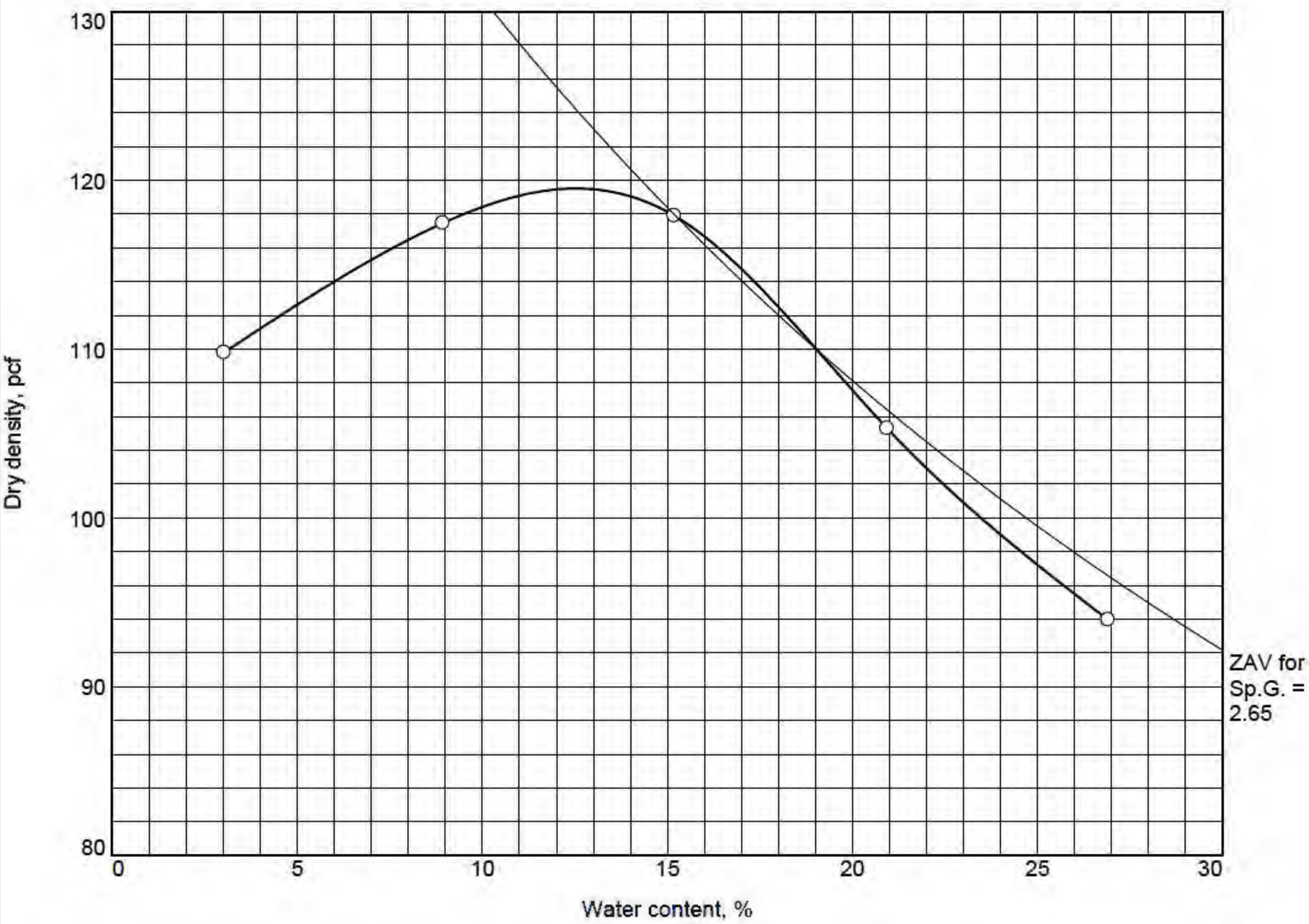
Date: 4/21/2016

○ **Source of Sample:** RHO - 5 **Sample Number:** 12259

Remarks:

Tested By: Nick Averill **Checked By:** Nick Averill

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0'-5'			22.4					

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 119.5 pcf Optimum moisture = 12.5 %	

Project No. 20164522 **Client:** PacLand
Project: Pacland 1926
Date: 4/21/2016
○ **Source of Sample:** HSA - 1 **Sample Number:** 12260

KLEINFELDER

Remarks:

Tested By: Nick Averill **Checked By:** Nick Averill

RHO Laboratory Testing Summary

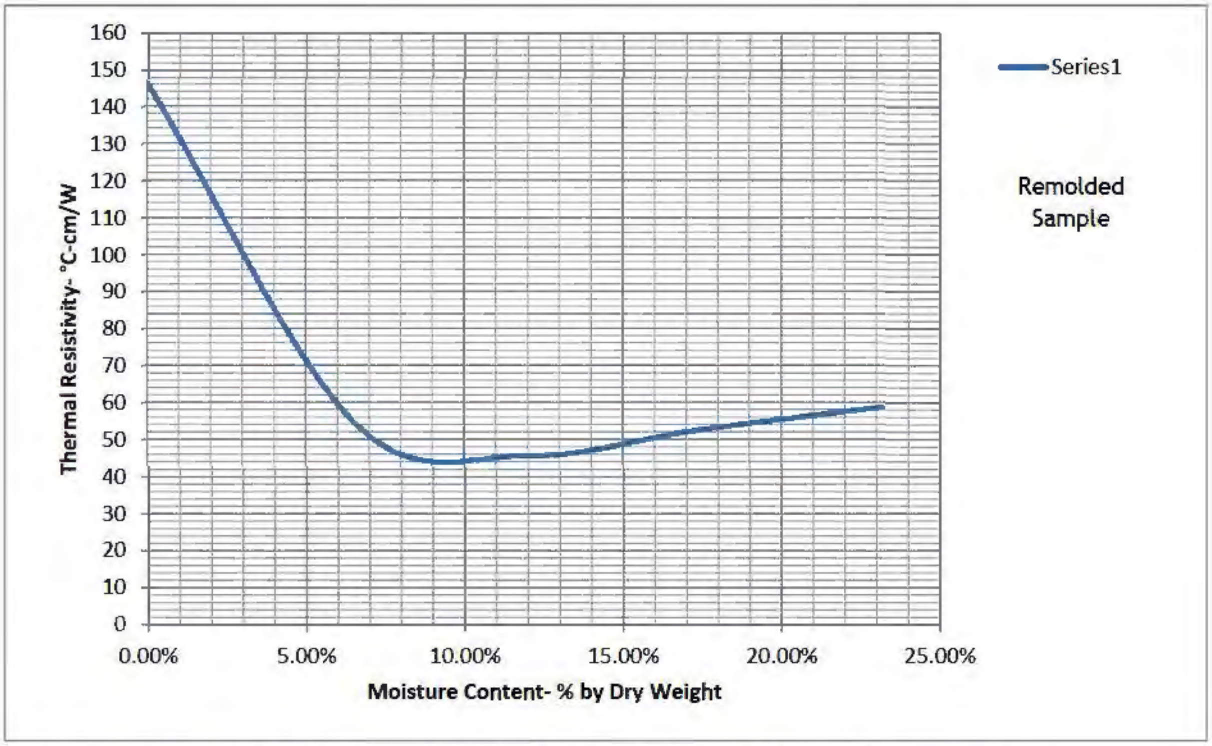
Number	Boring	Depth	Rho (degrees C-cm/W)	Internal Temperature (degrees C)	Ambient Temperature (degrees F)	Moisture (%)	Density (pcf)	USCS Soil Type
1	HSA-3	6	77.2	23.6	70	21.9	101.2	CH
2	HSA-3	11	83.0	24.2	70	22.5	104.3	CH
3	RHO-8	6	122.2	14.1	67	15.1	89.1	CL
4	HSA-9	6	86.2	24.4	50	19.4	97.2	CL/ML
6	HSA-9	8	69.1	23.4	50	17.7	104.5	ML
7	HSA-9	11	72.7	22.8	50	23.6	101.8	CH
8	HSA-5	6	95.3	30.3	65	19	103.7	CH
9	HSA-5	11	71.9	25.8	65	23.2	103	CH
10	HSA-7	6	93.7	24.9	61	16.4	105.3	CL
11	HSA-7	11	80.2	23.2	61	23.3	104.7	CH
12	RHO-11	11	75.6	18.2	59	25.3	100.1	CH
13	RHO-9	11	141.2	17.4	58	18.7	96.1	CH
14	HSA-8	6	103.4	24.9	54	20.2	98.2	CL
15	HSA-8	11	90.8	27.7	54	23.8	101.9	CH
16	RHO-13	11	156.2	17.9	50	10.5	64.3	SC
17	RHO-2	6	105.4	17.5	60	21.8	95.5	CH
18	RHO-3	11	97.5	17.6	60	23.3	104.4	CH
19	RHO-4	6	80.8	15.7	59	27	91.4	CL
20	RHO-7	11	94.7	17.5	59	25.7	98.3	CH
21	RHO-5	11	85.5	16.2	59	25.6	100.3	CL
22	RHO-6	6	109.2	19.9	61	18.9	98.5	CH
23	RHO-10	6	136.7	15.3	64	9.6	88.3	CL
24	RHO-12	6	176.7	15.6	63	10.1	83	CL
25	RHO-14	6	124.8	14.6	62	12.7	92.6	SC
26	RHO-1	11	70.4	19.6	58	23	104.5	CH
27	RHO-15	11	80.1	16.6	55	26.4	100.8	CL
28	MR-2	3.5	92.7	13.8	48	14.9	93.5	CL
29	MR-2	8.5	89.0	16.4	48	24.6	101.1	CH
30	MR-1	3.5	92.6	21.0	59	9.7	97.4	SP-SC
31	MR-1	6	71.7	17.7	59	22.5	99.2	SM
32	MR-3	3	84.5	25.3	63	22.3	98.9	CL
33	MR-3	5.5	78.4	19.9	63	21.2	102.8	CL
34	MR-4	3.5	95.7	13.5	50	22.3	98.7	SP-SC
35	MR-4	6	71.9	14.2	50	21.9	100.5	CL

THERMAL RESISTIVITY REPORT

Method Used: IEEE Std. 442-1981

Project Name:	PacLand 1926	Date:	4/29/2016
Project Number:	20164522.001A		
Client:	PacLand 1926		
		Method of Obtaining Max Dry Density:	D-1557 (A)
Sample I.D.:	RHO-5	Maximum Dry Density, pcf:	124.4
Depth, ft.:	0'-5'	Optimum Moisture Content, %:	10.7
Sample Type:	Bulk	Sample Size:	4" x 4.6"
Visual Description:		Laboratory Sample Number:	12259

Thermal Dry-Out Curve



* As per corresponding Moisture Density curve, this material cannot achieve 95% max density at reported moisture contents.

Point #	Specimen Mass in Grams	Needle Insertion Method	Temperature in Celcius	Moisture Content	Resistivity (°C-cm/W)	Dry Density	% Compaction *
1	1873.8	Pushed	20.75	23.16%	58.81	100.73	81.0
2	2001.6	Pushed	21.48	17.29%	52.49	112.98	90.8
3	1997.1	Pushed	21.84	12.02%	45.65	118.03	94.9
4	1877.5	Pushed	22.68	6.55%	54.33	116.65	93.8
5	1759.5	Pushed	22.77	0.00%	146.30	116.49	93.6

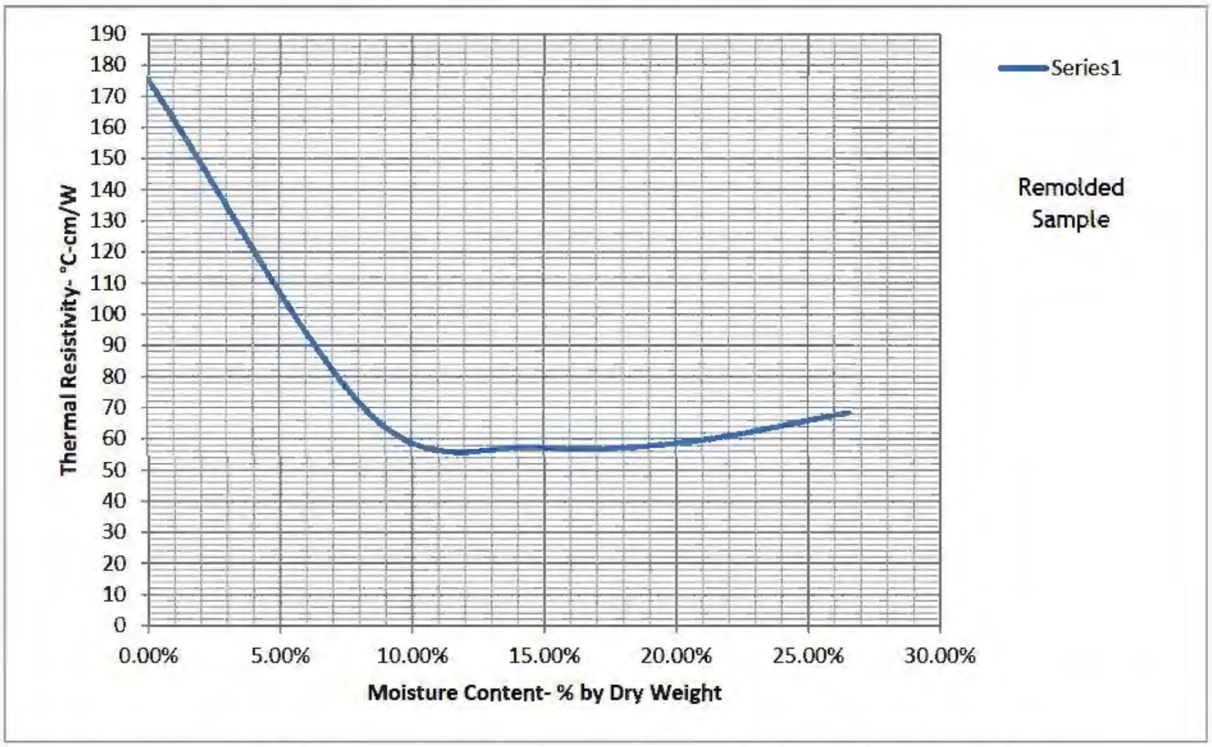
Prepared by: Nick Averill	Tested by: Nick Averill	Reviewed by: Nick Averill

THERMAL RESISTIVITY REPORT

Method Used: IEEE Std. 442-1981

Project Name:	PacLand 1926	Date:	4/29/2016
Project Number:	20164522.001A		
Client:	PacLand 1926		
		Method of Obtaining Max Dry Density:	D-1557 (A)
Sample I.D.:	HSA-1	Maximum Dry Density, pcf:	119.5
Depth, ft.:	0'-5'	Optimum Moisture Content, %:	12.5
Sample Type:	Bulk	Sample Size:	4" x 4.6"
Visual Description:		Laboratory Sample Number:	12260

Thermal Dry-Out Curve



* As per corresponding Moisture Density curve, this material cannot achieve 95% max density at reported moisture contents.

Point #	Specimen Mass in Grams	Needle Insertion Method	Temperature in Celcius	Moisture Content	Resistivity (°C-cm/W)	Dry Density	% Compaction *
1	1850.7	Pushed	21.09	26.53%	68.57	96.83	81.0
2	1960.4	Pushed	21.60	20.51%	59.28	107.69	90.1
3	1974.0	Pushed	22.42	14.80%	57.43	113.84	95.3
4	1831.9	Pushed	22.46	8.51%	67.39	111.77	93.5
5	1686.9	Pushed	22.70	0.00%	175.70	111.68	93.5

Prepared by: Nick Averill	Tested by: Nick Averill	Reviewed by: Nick Averill

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

18 April, 2016

Job No. 1604082
Cust. No. 10527

Mr. Don Adams
Kleinfelder
6700 Koll Center Parkway, Ste. 120
Pleasanton, CA 94566

Subject: Project No.: 20164522
Project Name: Paceland 1926
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Adams:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on April 11, 2016. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the 100% saturation resistivity measurement, the sample is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration is 39 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 52 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The sulfide ion concentrations reflect none detected with a detection limit of 50 mg/kg

The pH of the soil is 8.06 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 350-mV which is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Kleinfelder
Client's Project No.: 20164522
Client's Project Name: Paceland 1926
Date Sampled: 31-Mar-2016
Date Received: 11-Apr-2016
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 18-Apr-2016

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1604082-001	MR-3 1 @ 3'	+350	8.06	2,900	2,100	N.D.	39	52

Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	14-Apr-2016	14-Apr-2016	15-Apr-2016	15-Apr-2016	12-Apr-2016	14-Apr-2016	14-Apr-2016

Cheryl McMillen
Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

KLEINFELDER 1604082 10527

LAB NUMBER: HL8704

LABORATORY TESTING PROGRAM - GEOTECHNICAL

Project: Paland 1926

Date Sampled: 3/31/2016 Project No.: 20164522

Date Submitted: Task No.: 01

Submitted By: dbadams@kleinfelder.com Report To: Don Adams in Pleasant

Test Method
☐ ASTM ☐ DOT
☐ AASHTO ☐ Other (see remarks)

Boring / Test Pit	Sample	Depth, ft.	Sample Type	CLASSIFICATN, VISUAL	WATER CONTENT	WATER/DENSITY, SOIL	SIEVE ANALYSIS, -200	SIEVE ANALYSIS, F		SIEVE ANALYSIS, C&F	HYDROMETER ANALYSIS	ATTERBERG, MULTI-PT.	ATTERBERG, ONE-PT.	SOIL ORGANIC CONTENT	SOIL SPEC. GRAV.				STANDARD PROCTOR, 4"	MODIFIED PROCTOR, 4"	R-VALUE	EXPANSION INDEX		SOIL PH	SOIL RESISTIVITY	CHLORIDE CONTENT	SULFATE CONTENT		SAMPLE MIX&PROCESS	SAMPLE PREPARATION	Remarks
MR-2	14C	56	Liner		/					/																					
MR-2	16C	66	Liner		/		/																								
MR-1	12C	51	Liner		/		/																								
MR-4	2C	6	Liner							/																					
MR-4	6	16	bag		/																										
MR-4	11	36	bag							/																					
HSA-1	1	3	liner		/																										
HSA-2	1	3	liner		/					/																					
HSA-3	7C	18	liner		/																										
HSA-3	1B	3	liner				/																								Standard ASTM +S & Res.
MR-3	1	3	liner																					/	/	/	/				As received. w/ interpretation
Quantity of Tests																															
Unit				L00228	L00294	L00295	L00267	L00270		L00269	L00248	L00223	L00224	L00271	L00275				[illegible]	[illegible]	L00264	L00246		L00272	L00273	L00227	L00281		L00905	L00907	

4/1/16 1435

Boring / Test Pit	Sample	Depth, ft.	Sample Type	UNCONFINED COMP STR.	DIRECT SHEAR, 1 PT.	DIRECT SHEAR, 3 PTS.	DIRECT SHEAR, RESID.	TRIAX, CU, 1 POINT	TRIAX, CU, 3 POINTS	TRIAX, CD, 1 POINT	TRIAX, CD, 3 POINTS	TRIAX, UU, 1 POINT	TRIAX, UU, 3 POINTS	TRIAX, UU SAT, 1 PT.	TRIAX, UU SAT, 3 PT.	PERMEABILITY, FLEX	PERMEABILITY, RIGID	PINHOLE DISPERSION		CONSOL. W/O TR	CONSOL. W/2 TR	CONSOL. ALL TR	CONSOL. ADD. REBOUND		ONEDIM FREE SWELL	ONEDIM SWELL % HEAVE	ONEDIMSWELL W/CONSOL		SAMPLE REMOLDING		Remarks
Quantity of Tests																															
Unit				L00293	L00242	L00243	L00244	L00286	L00287	L00285	Lo0336	L00289	L00290	L00288	L00325	L00258	L00259	L00261		L00233	L00232	L00231	L00230		L00255	L00256	L00257		L00908		

◺ Test requested ⧅ Test in progress ☒ Test complete

Relinquished by:
Received by:
DATE/TIME 4/1/16 1305

Rev. 02/17/2014

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

18 April, 2016

Job No. 1604081
Cust. No. 10527

Mr. Don Adams
Kleinfelder
6700 Koll Center Parkway, Ste. 120
Pleasanton, CA 94566

Subject: Project No.: 20164522
Project Name: Paceland 1926
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Adams:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on April 11, 2016. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the 100% saturation resistivity measurement, the sample is classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration is 68 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 120 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The sulfide ion concentration is none detected with a detection limit of 50 mg/kg

The pH of the soil is 7.80 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 360-mV which is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl Medil for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Kleinfelder
Client's Project No.: 20164522
Client's Project Name: Paceland 1926
Date Sampled: 21-Mar-2016
Date Received: 11-Apr-2016
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 18-Apr-2016

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1604081-001	HSA-1 1C @ 3'	+360	7.80	1,600	860	N.D.	68	120

Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	14-Apr-2016	14-Apr-2016	15-Apr-2016	15-Apr-2016	12-Apr-2016	14-Apr-2016	14-Apr-2016

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

KLEINFELDER 1604081 10527

LAB NUMBER: HL8704

LABORATORY TESTING PROGRAM - GEOTECHNICAL

Project: Pacland 1926

Date Sampled: 3/21/2016 Project No.: 20164522

Date Submitted: 4/11/16 Task No.: 01

Submitted By: dbadams@kleinfelder.com Report To: Don Adams in Pleasant(

Test Method
☐ ASTM ☐ DOT ______
☐ AASHTO ☐ Other (see remarks)

Boring / Test Pit	Sample	Depth, ft.	Sample Type	CLASSIFICATN, VISUAL	WATER CONTENT	WATER/DENSITY, SOIL	SIEVE ANALYSIS, -200	SIEVE ANALYSIS, F		SIEVE ANALYSIS, C&F	HYDROMETER ANALYSIS	ATTERBERG, MULTI-PT.	ATTERBERG, ONE-PT.	SOIL ORGANIC CONTENT	SOIL SPEC. GRAV.				STANDARD PROCTOR, 4"	MODIFIED PROCTOR, 4"	R-VALUE	EXPANSION INDEX		SOIL PH	SOIL RESISTIVITY	CHLORIDE CONTENT	SULFATE CONTENT		SAMPLE MIX&PROCESS	SAMPLE PREPARATION	Remarks
HSA-1	3	8	liner		/																										
HSA-1	5	13	liner		/																										
HSA-1	7	18	liner			/																									
HSA-1	1C	3	liner																					/	/	/	/				Standard ASTM +S & Res. As received. w/ interpretation
HSA-6	1	3	liner				/					/																			
HSA-6	4	11	liner			/																									
HSA-4	1	3	liner			/	/					/																			
HSA-4	7	18	bag		/																										
HSA-9	1	6	liner				/					/																			
HAS-5	8	20	bag				/					/																			
HSA-2	8	21	bag		/																										
Quantity of Tests																															
Unit				L00228	L00294	L00295	L00267	L00270		L00269	L00248	L00223	L00224	L00271	L00275				L00279	L00251	L00264	L00246		L00272	L00273	L00227	L00281		L00905	L00907	

Boring / Test Pit	Sample	Depth, ft.	Sample Type	UNCONFINED COMP STR.	DIRECT SHEAR, 1 PT.	DIRECT SHEAR, 3 PTS.	DIRECT SHEAR, RESID.	TRIAX, CU, 1 POINT	TRIAX, CU, 3 POINTS	TRIAX, CD, 1 POINT	TRIAX, CD, 3 POINTS	TRIAX, UU, 1 POINT	TRIAX, UU, 3 POINTS	TRIAX, UU SAT, 1 PT.	TRIAX, UU SAT, 3 PT.	PERMEABILITY, FLEX	PERMEABILITY, RIGID	PINHOLE DISPERSION		CONSOL. W/O TR	CONSOL. W/2 TR	CONSOL. ALL TR	CONSOL. ADD. REBOUND		ONEDIM FREE SWELL	ONEDIM SWELL % HEAVE	ONEDIM SWELL W/CONSOL		SAMPLE REMOLDING		Remarks
Quantity of Tests																															
Unit				L00293	L00242	L00243	L00244	L00286	L00287	L00285	Lo0336	L00289	L00290	L00288	L00325	L00258	L00259	L00261		L00233	L00232	L00231	L00230		L00255	L00256	L00257		L00908		

1435

[illegible]

◸ Test requested ⊠ Test in progress ☒ Test complete

Relinquished by: [illegible]

Received by: [illegible]

DATE/TIME: 4/11/16 1305

Rev. 02/17/2014

KLEINFELDER

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Appendix C

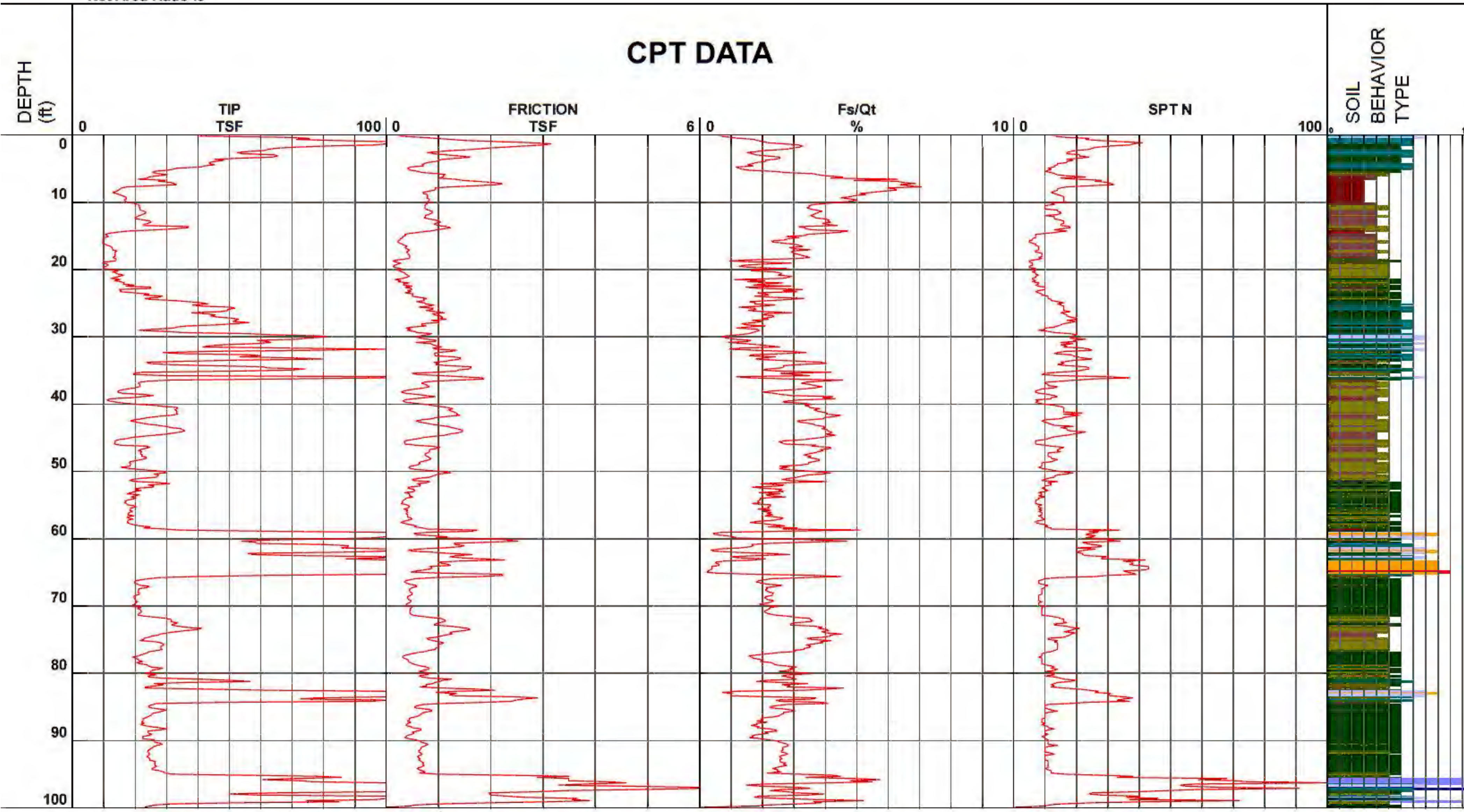
APPENDIX C
CORE PENETRATION TEST RESULTS

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(865).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-01	Date and Time	5/5/2016 5:32:44 PM	Maximum Depth	100.06 ft
EST GW Depth During Test	7.00 ft				

Net Area Ratio .8

CPT DATA



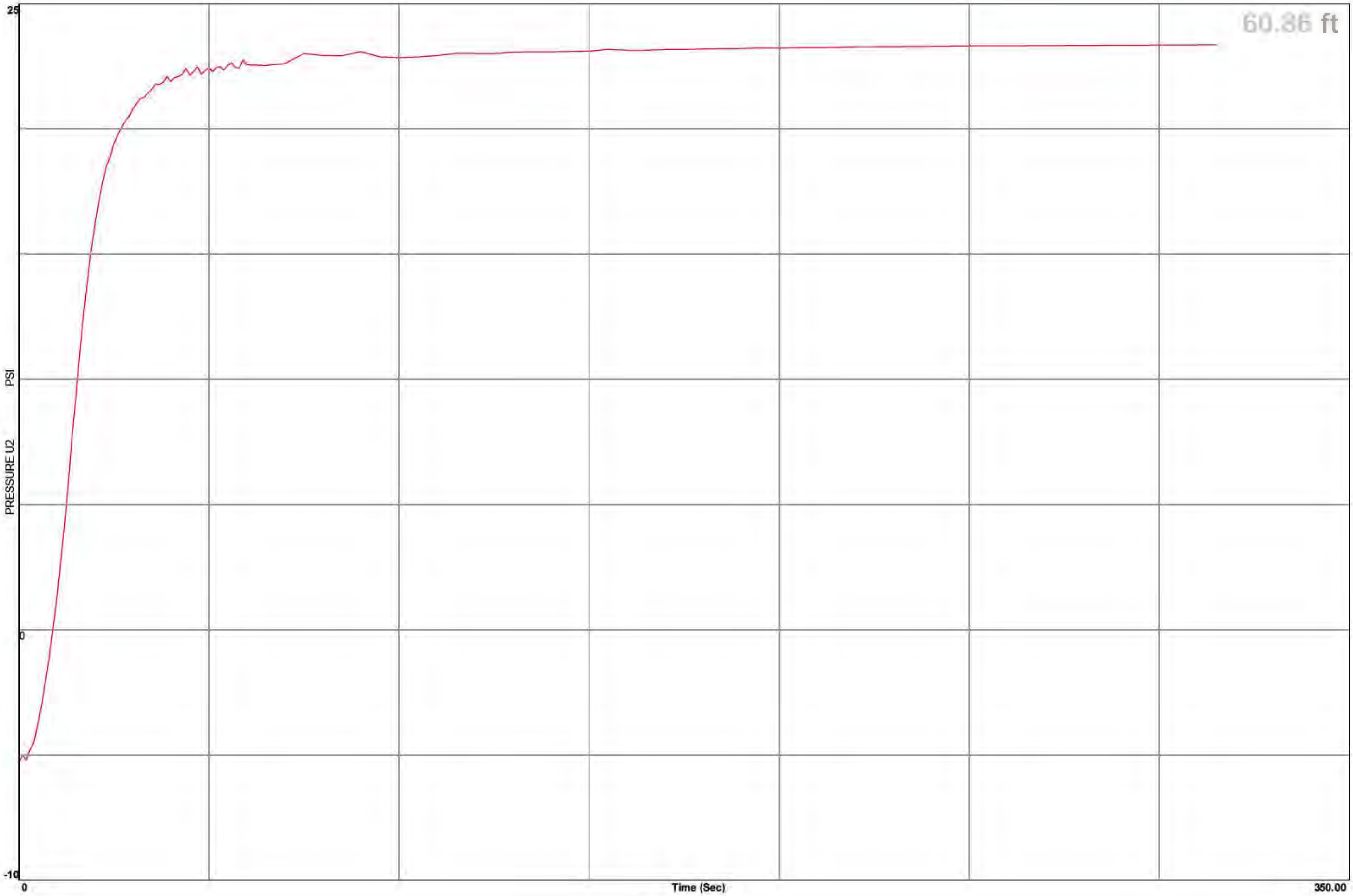
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

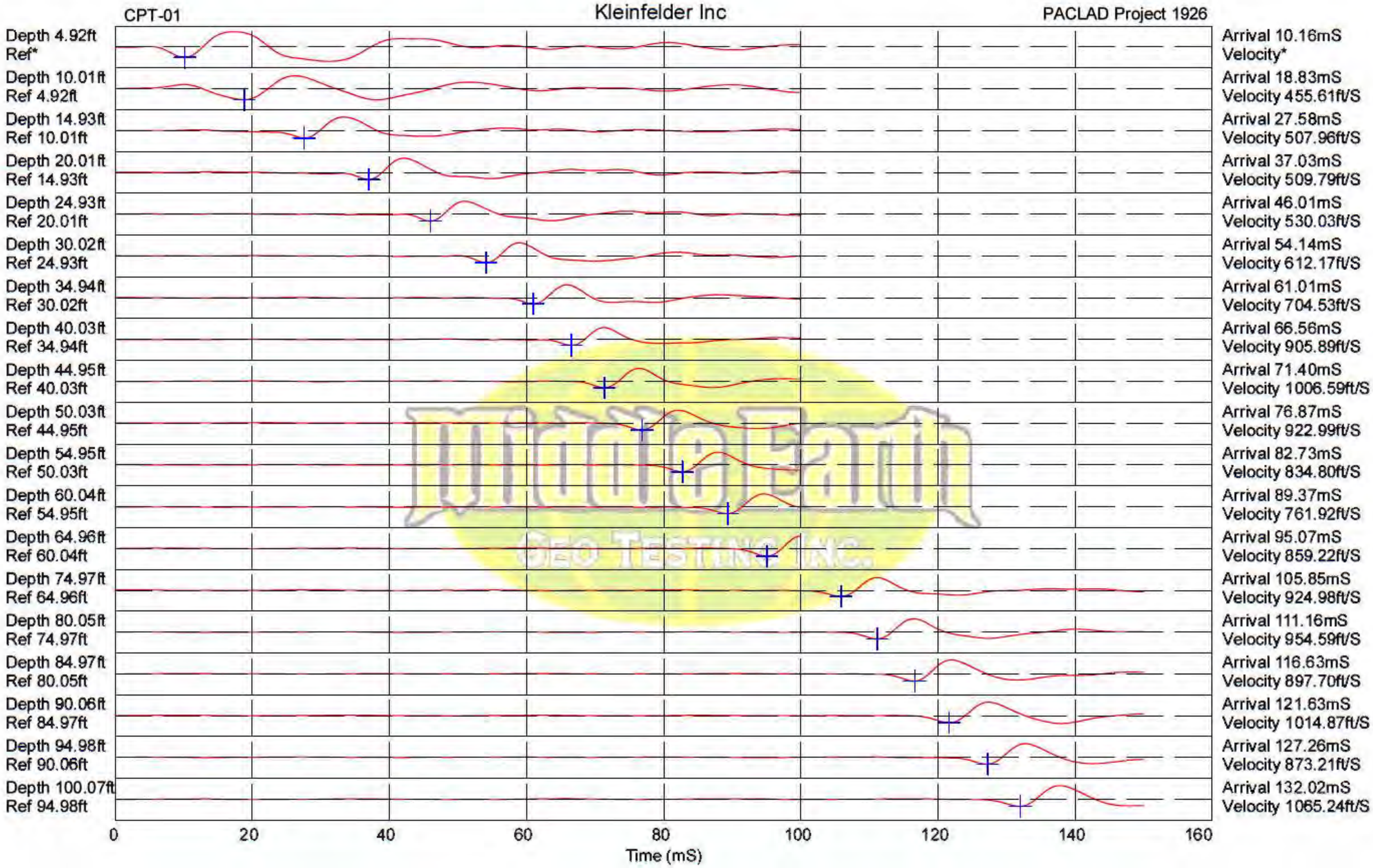
Cone Size 10cm squared

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

Kleinfelder Inc

Location	PACLAD Project 1926	Operator	BH-JH		
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-01	Date and Time	5/5/2016 5:32:44 PM		
Equilized Pressure	23.2	EST GW Depth During Test	7.0		





Hammer to Rod String Distance (ft): 5.83
* = Not Determined

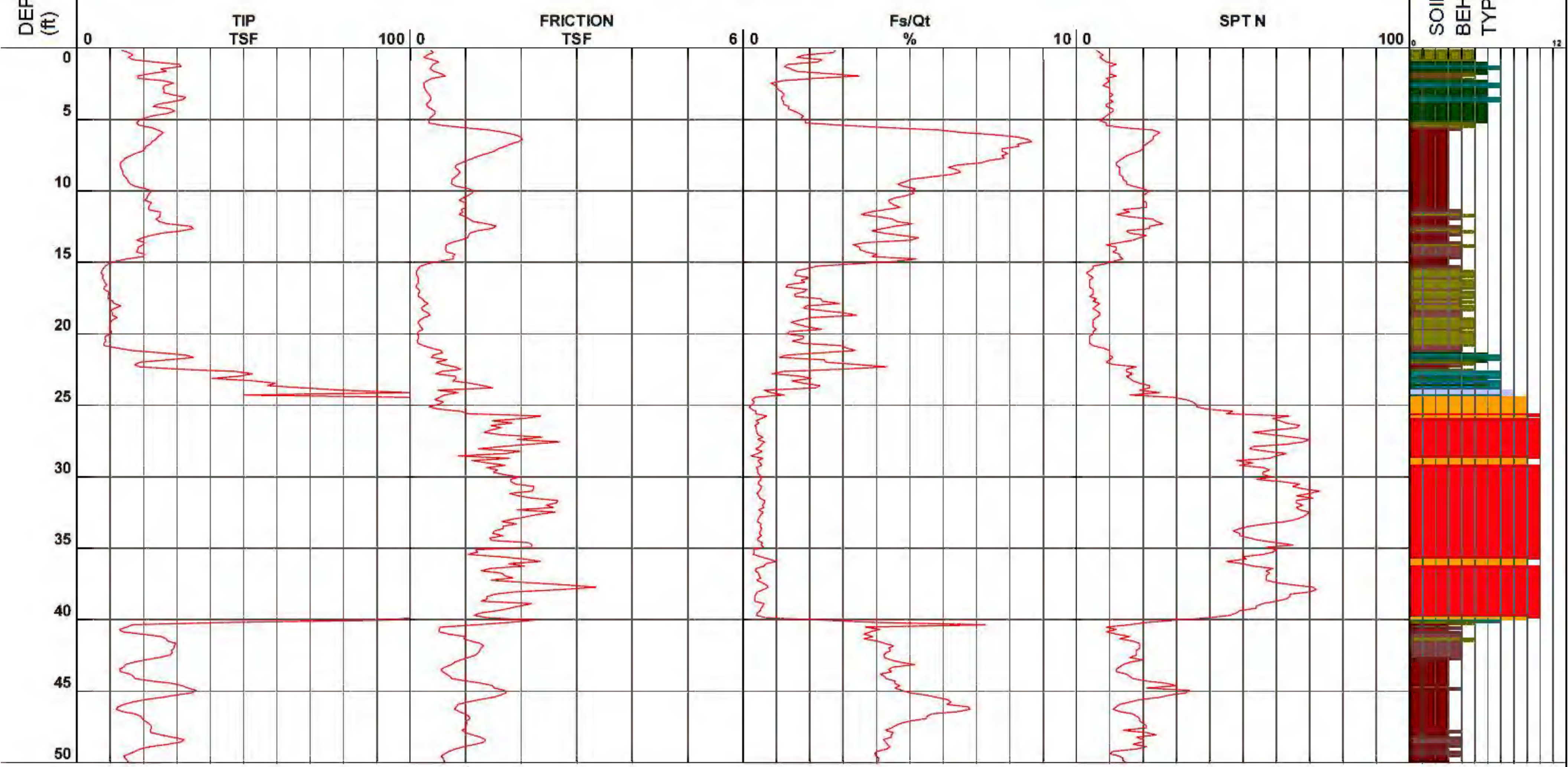
GPS DATA: ,,

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(806).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-02	Date and Time	4/27/2016 2:02:24 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		8.30 ft			

Net Area Ratio .8

CPT DATA



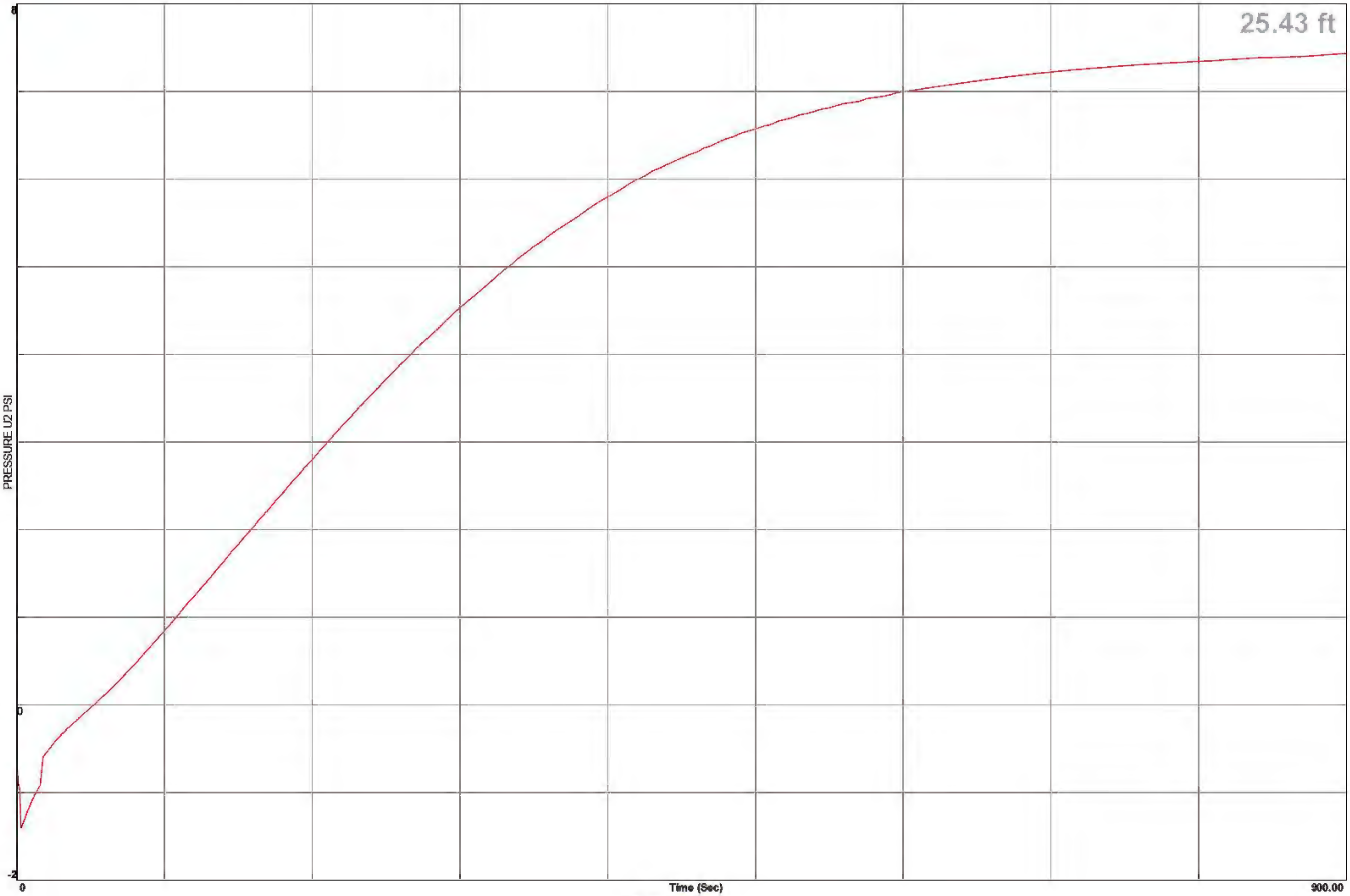
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

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

Kleinfelder Inc

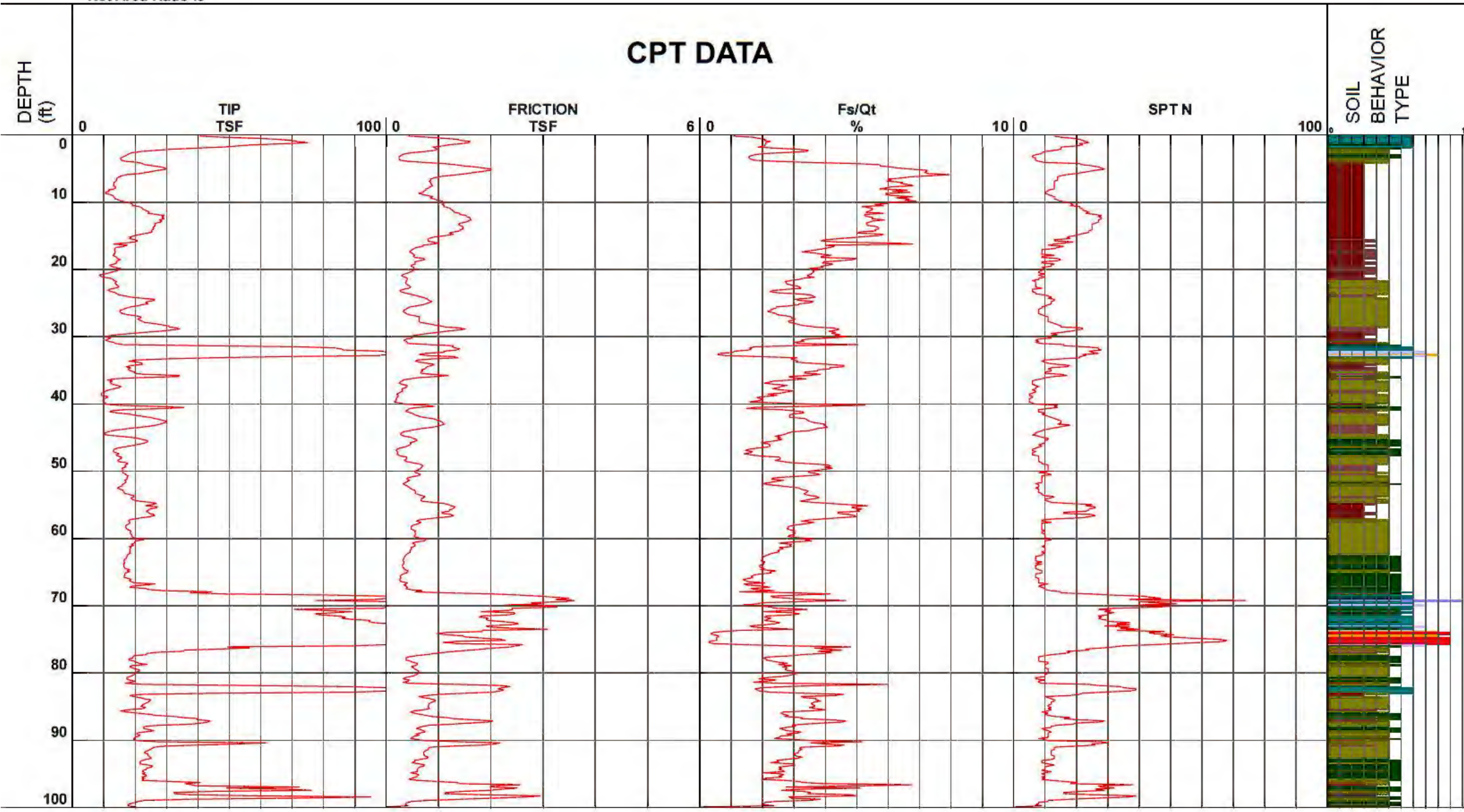
Location	PACLAD Project 1926	Operator	BH-JH		
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-02	Date and Time	4/27/2016 2:02:24 PM		
Equilized Pressure	7.4	EST GW Depth During Test	8.3		



Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(848).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-03	Date and Time	5/4/2016 7:17:12 PM	Maximum Depth	100.06 ft
EST GW Depth During Test		8.50 ft			

Net Area Ratio .8



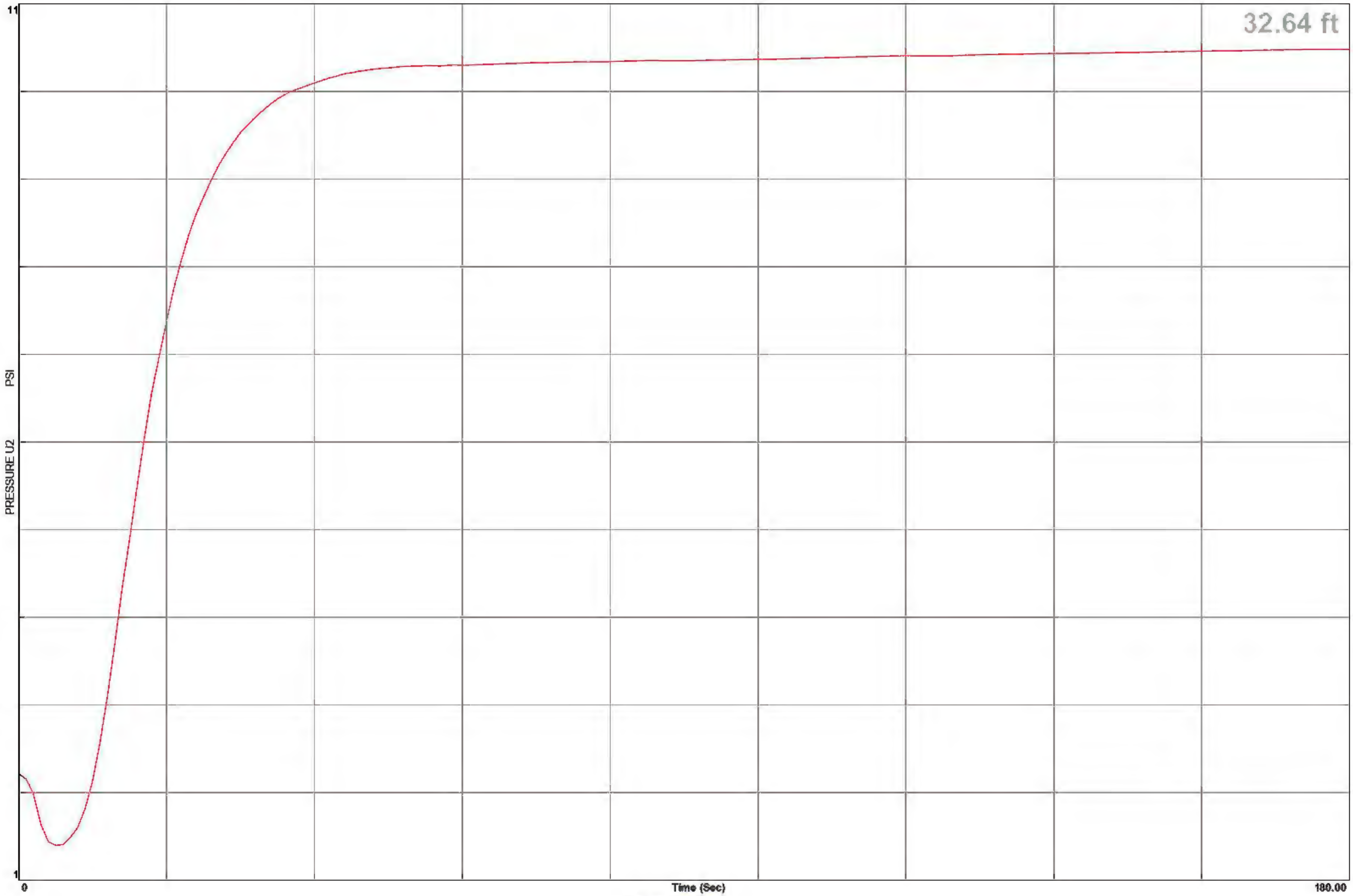
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

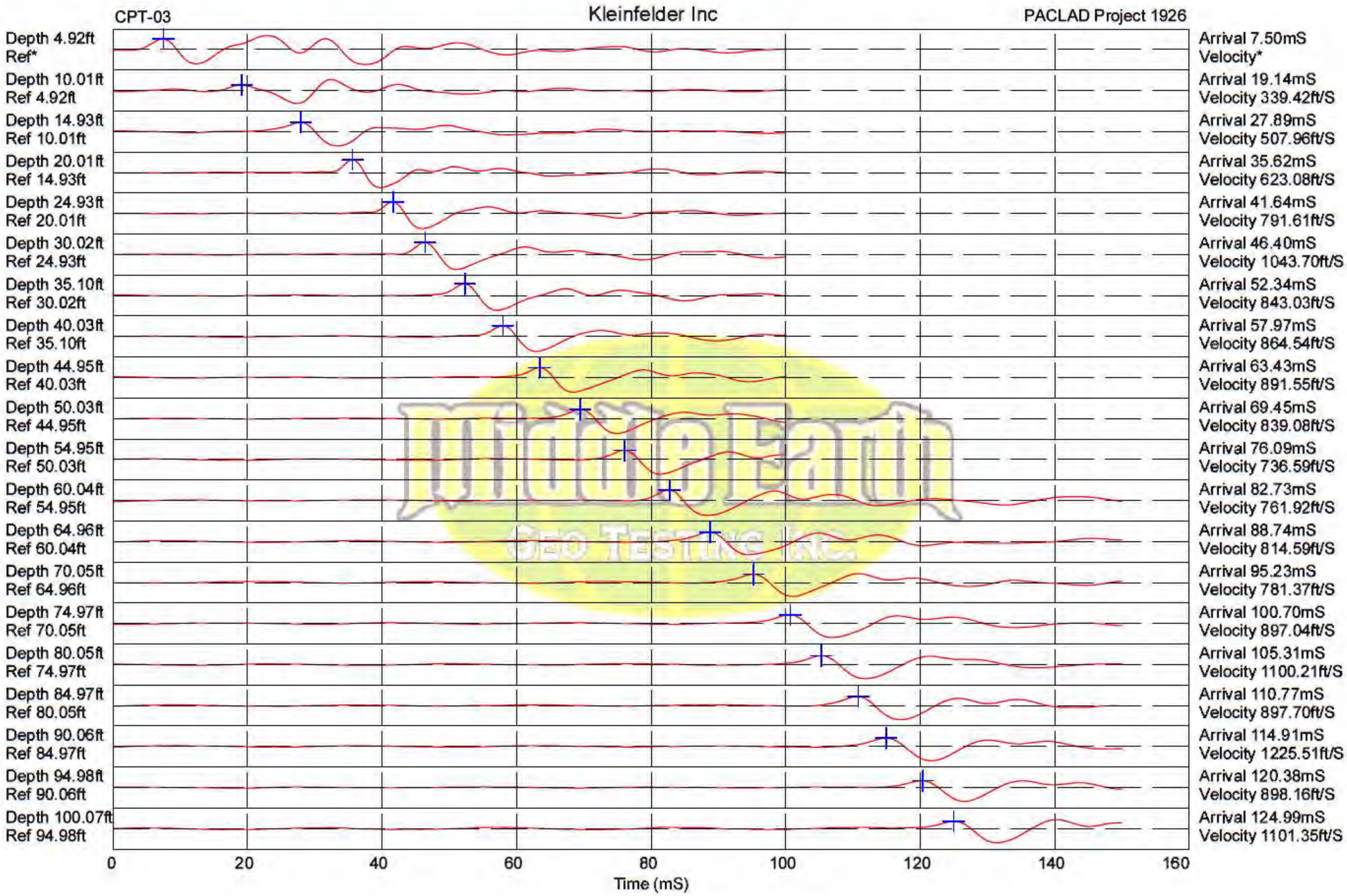
Cone Size 10cm squared

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

Kleinfelder Inc

Location	PACLAD Project 1926	Operator	BH-JH		
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-03	Date and Time	5/4/2016 7:17:12 PM		
Equilized Pressure	10.4	EST GW Depth During Test	8.5		





Hammer to Rod String Distance (ft): 5.83
* = Not Determined

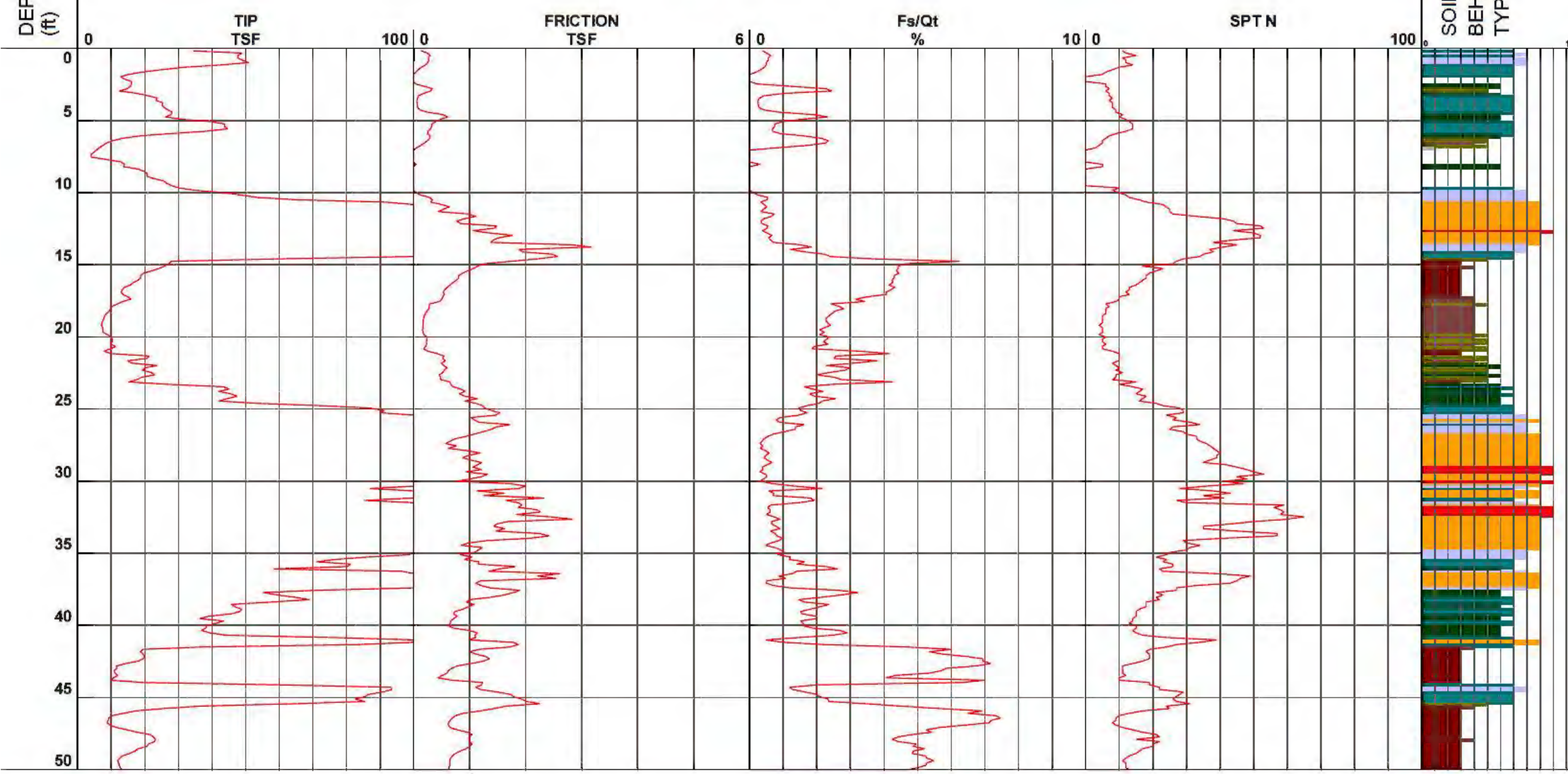
GPS DATA: ,,

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH-SF	Filename	SDF(819).cpt
Job Number	20164522	Cone Number	DDG1281	GPS	
Hole Number	CPT-04	Date and Time	4/28/2016 6:55:58 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		8.00 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

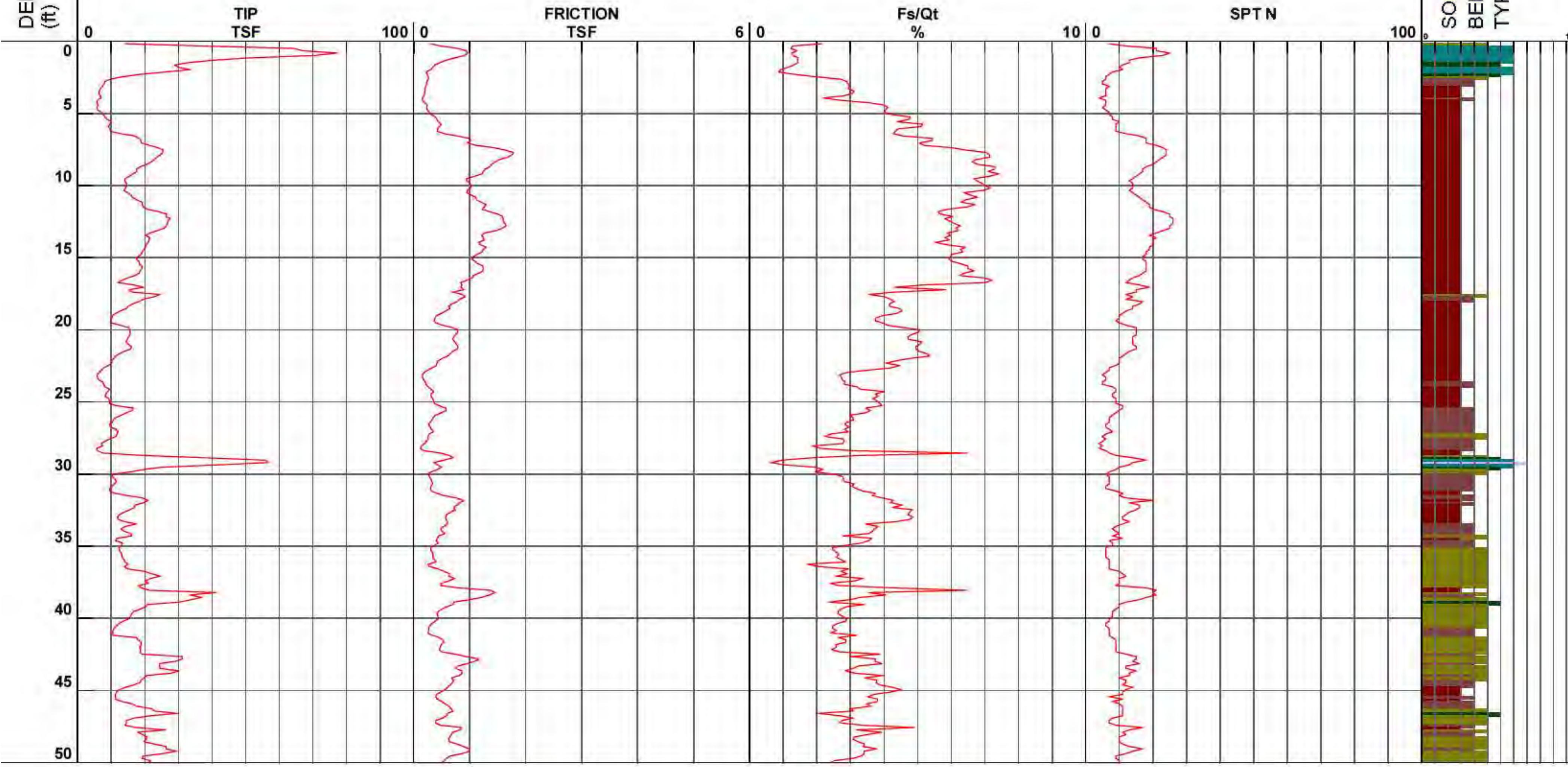
S*Soil behavior type and SPT based on data from UBC-1983

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(801).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-05	Date and Time	4/27/2016 11:24:01 AM	Maximum Depth	50.52 ft
EST GW Depth During Test		8.00 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

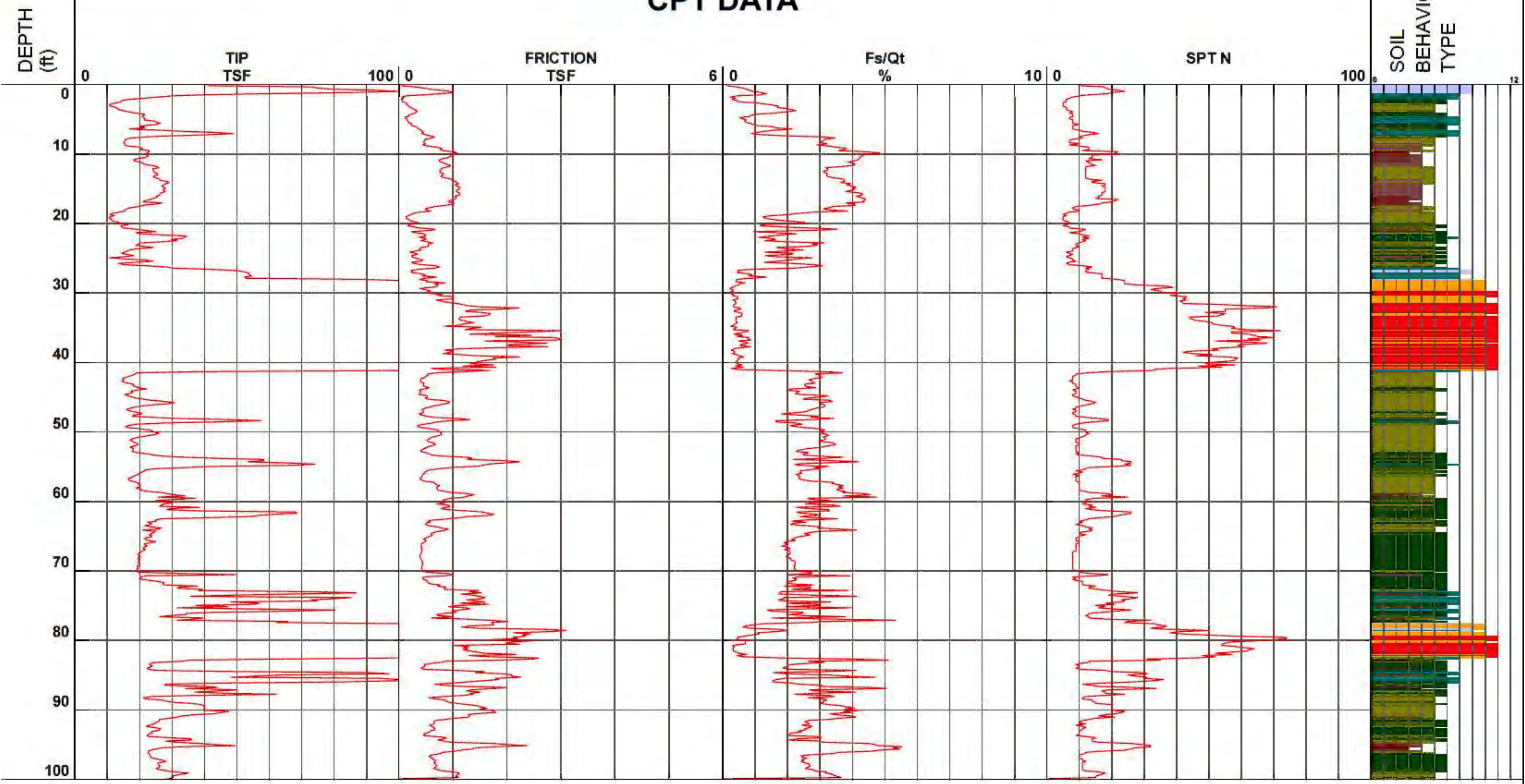
S*Soil behavior type and SPT based on data from UBC-1983

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(847).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-06	Date and Time	5/4/2016 5:46:06 PM	Maximum Depth	100.06 ft
EST GW Depth During Test		12.20 ft			

Net Area Ratio .8

CPT DATA



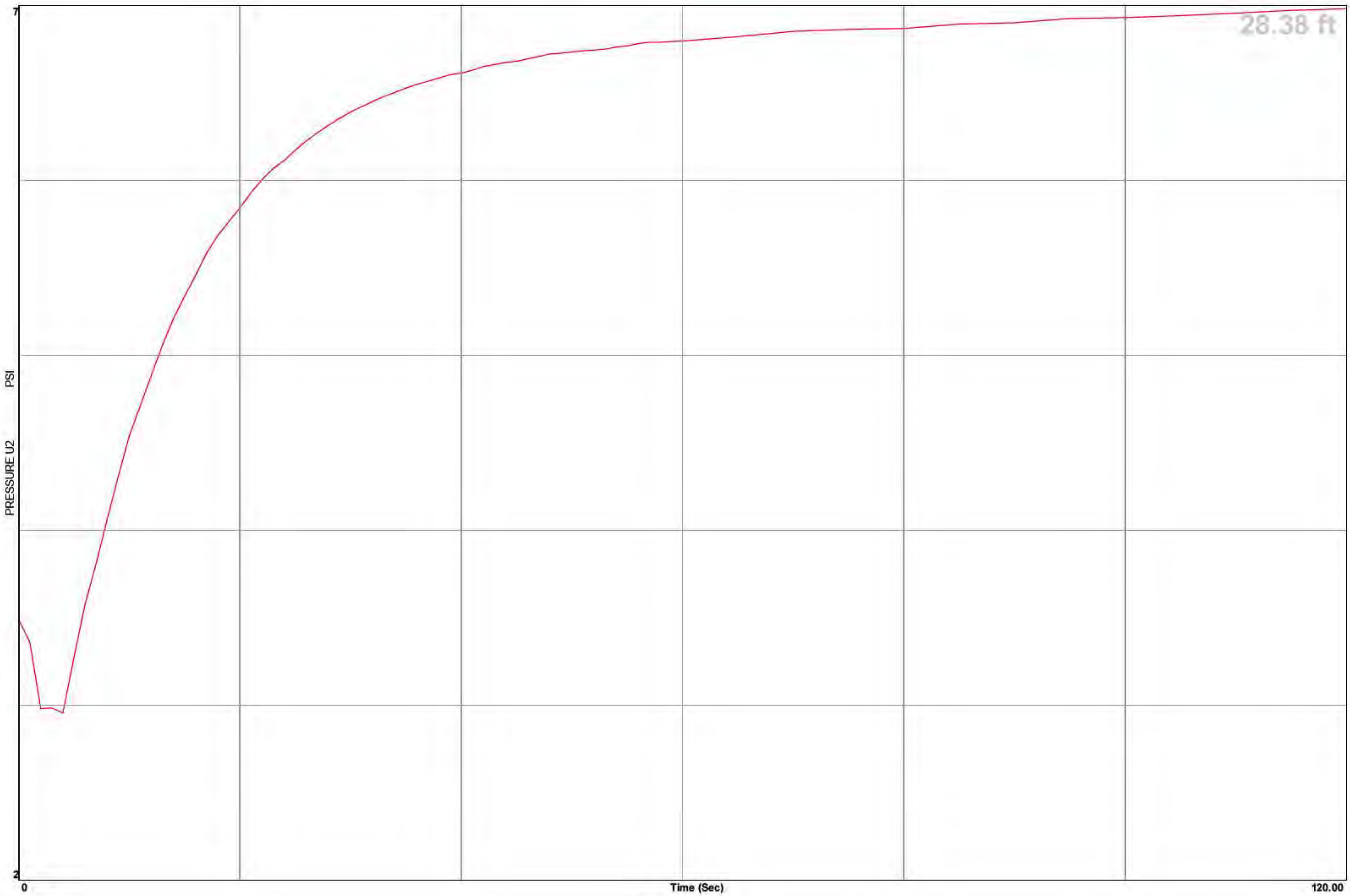
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

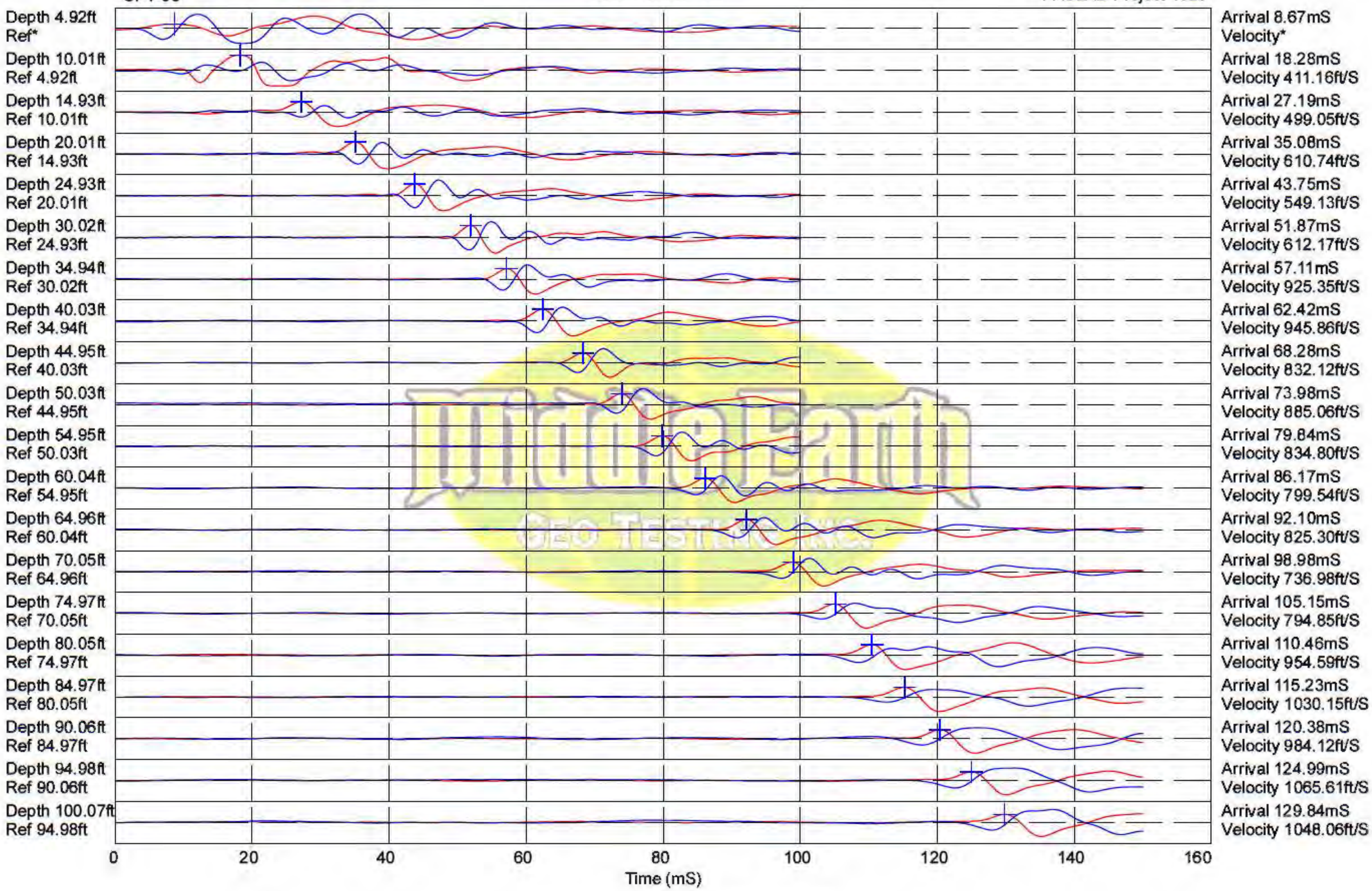
S*Soil behavior type and SPT based on data from UBC-1983

Kleinfelder Inc

Location	PACLAD Project 1926	Operator	BH-JH		
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-06	Date and Time	5/4/2016 5:46:06 PM		
Equilized Pressure	6.9	EST GW Depth During Test	12.2		



CPT-06 Kleinfelder Inc PACLAD Project 1926



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

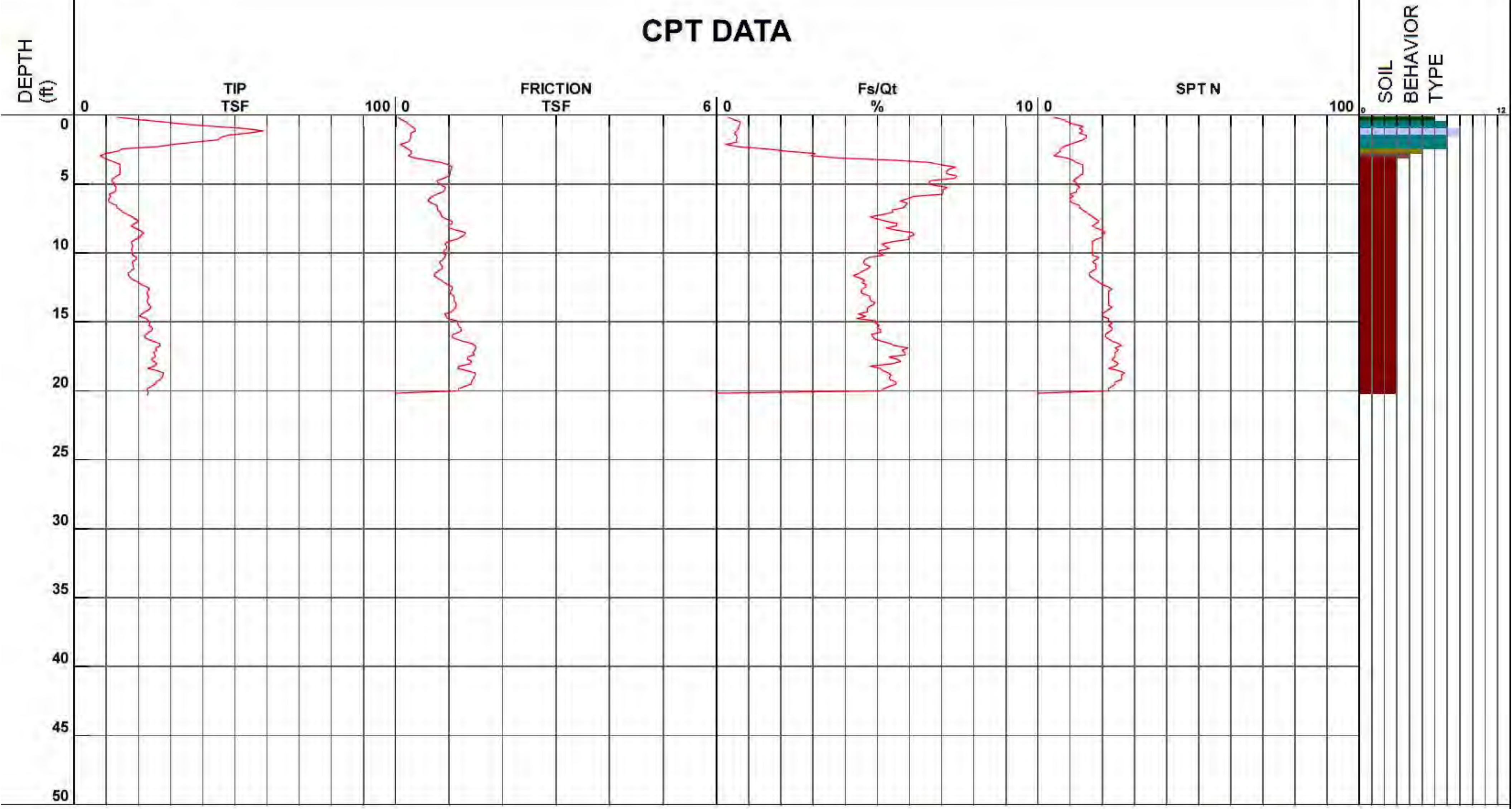
GPS DATA: ,,

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(809).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-07	Date and Time	4/27/2016 4:05:57 PM	Maximum Depth	20.34 ft
EST GW Depth During Test		8.00 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 (*)

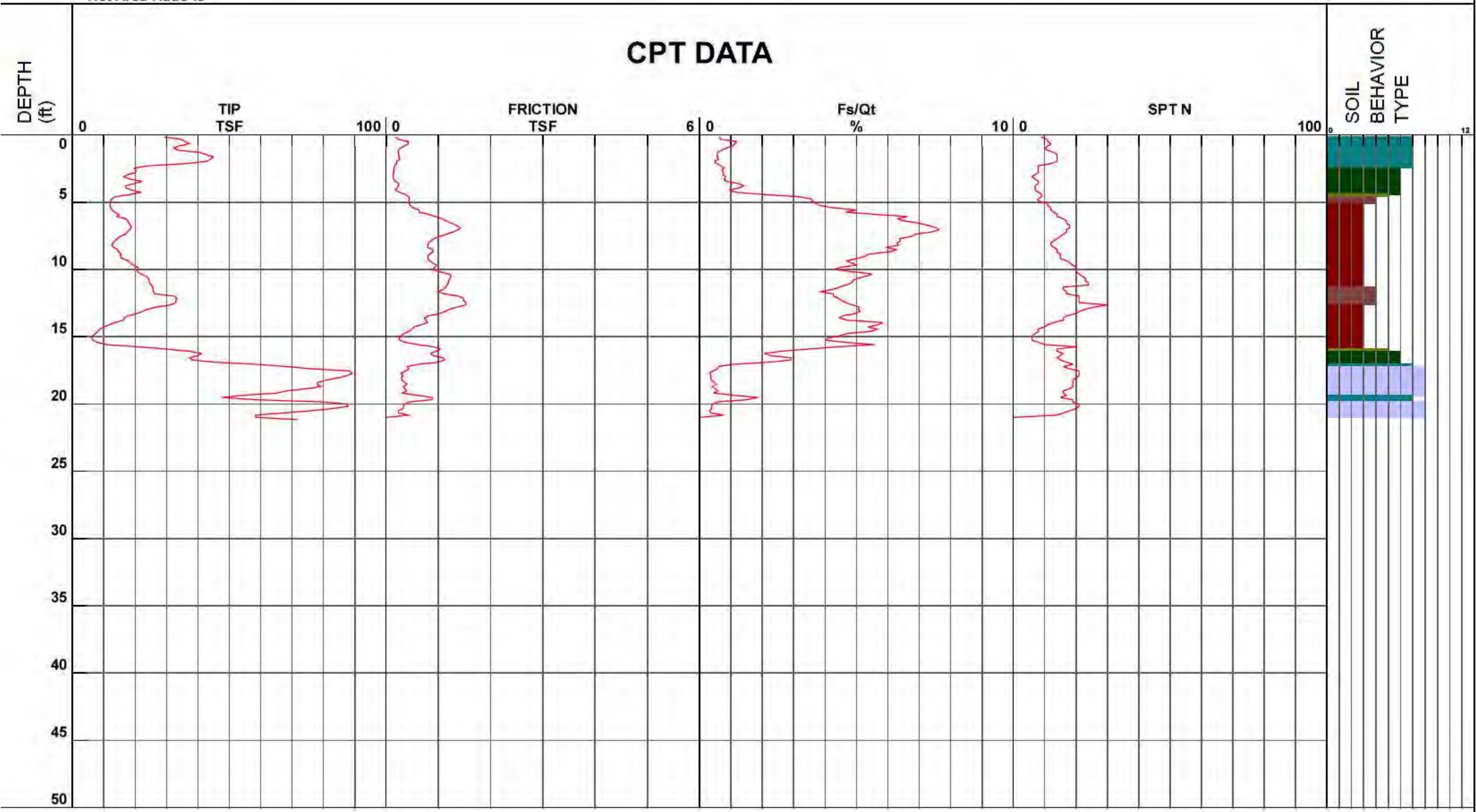
Cone Size 10cm squared

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

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(808).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-08	Date and Time	4/27/2016 3:37:13 PM	Maximum Depth	21.16 ft
EST GW Depth During Test		8.00 ft			

Net Area Ratio .8



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

Cone Size 10cm squared

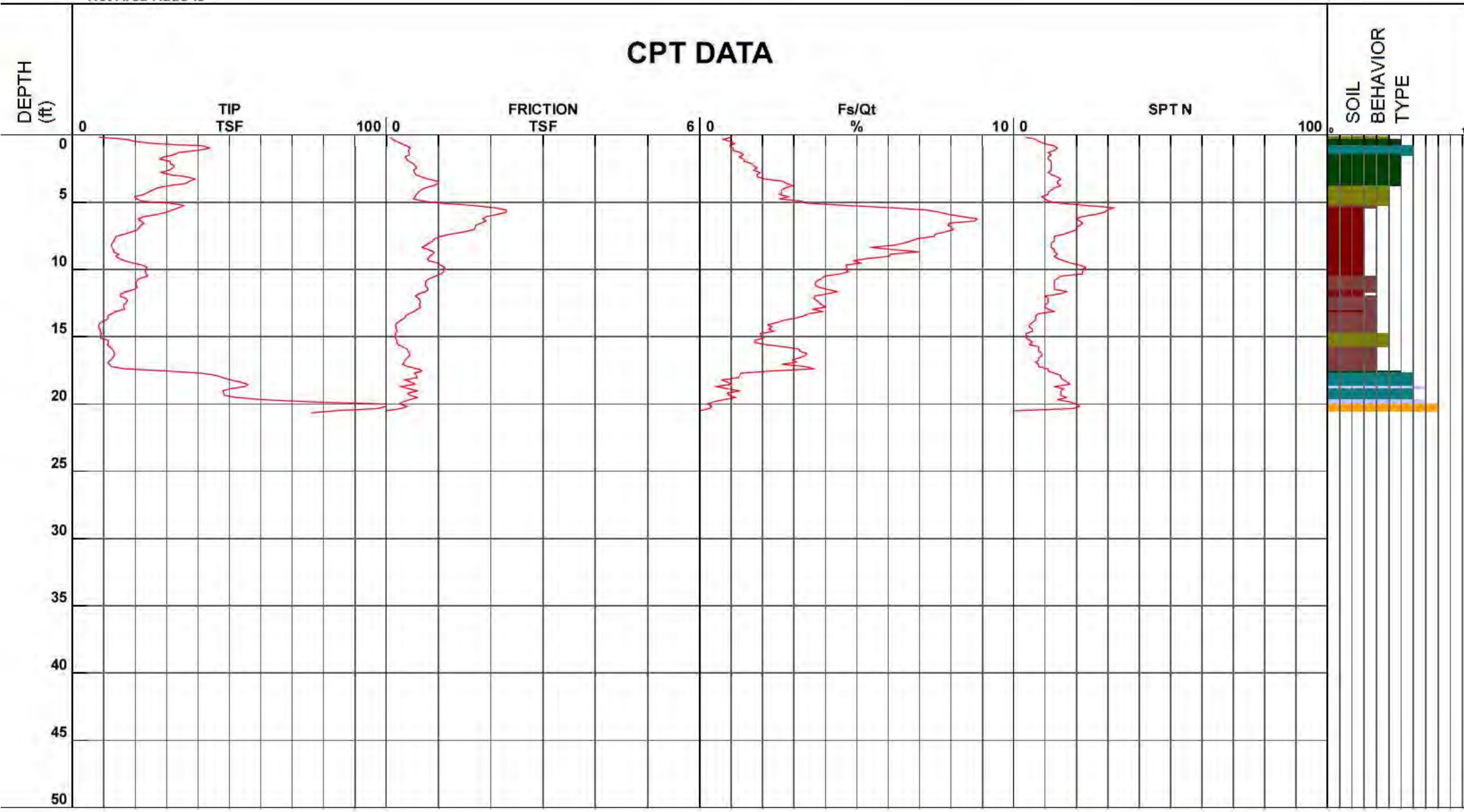
S*Soil behavior type and SPT based on data from UBC-1983

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(807).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-09	Date and Time	4/27/2016 3:11:08 PM	Maximum Depth	20.67 ft
EST GW Depth During Test		8.00 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 (*)

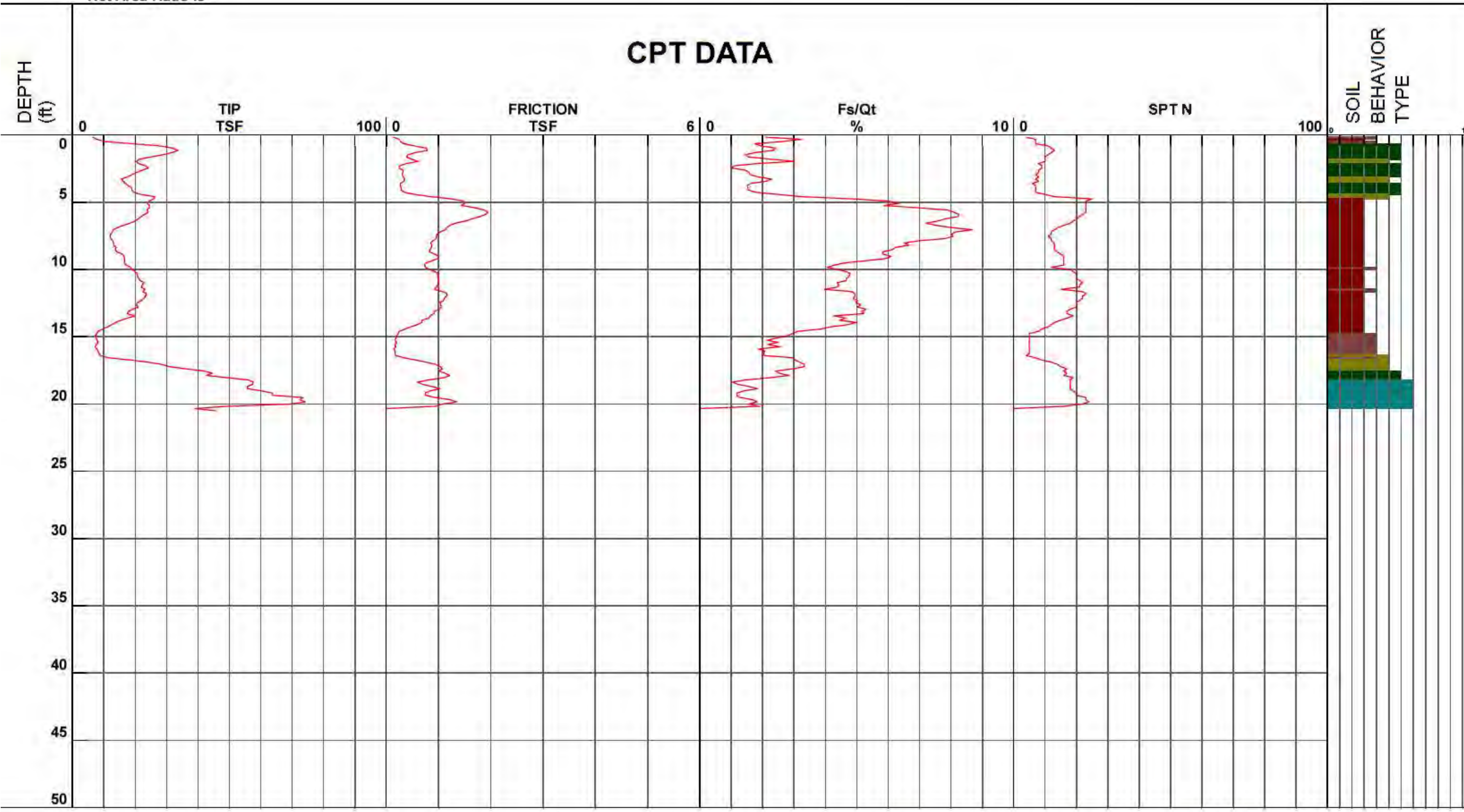
Cone Size 10cm squared

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

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(805).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-10	Date and Time	4/27/2016 1:23:04 PM	Maximum Depth	20.51 ft
EST GW Depth During Test	8.00 ft				

Net Area Ratio .8



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

Cone Size 10cm squared

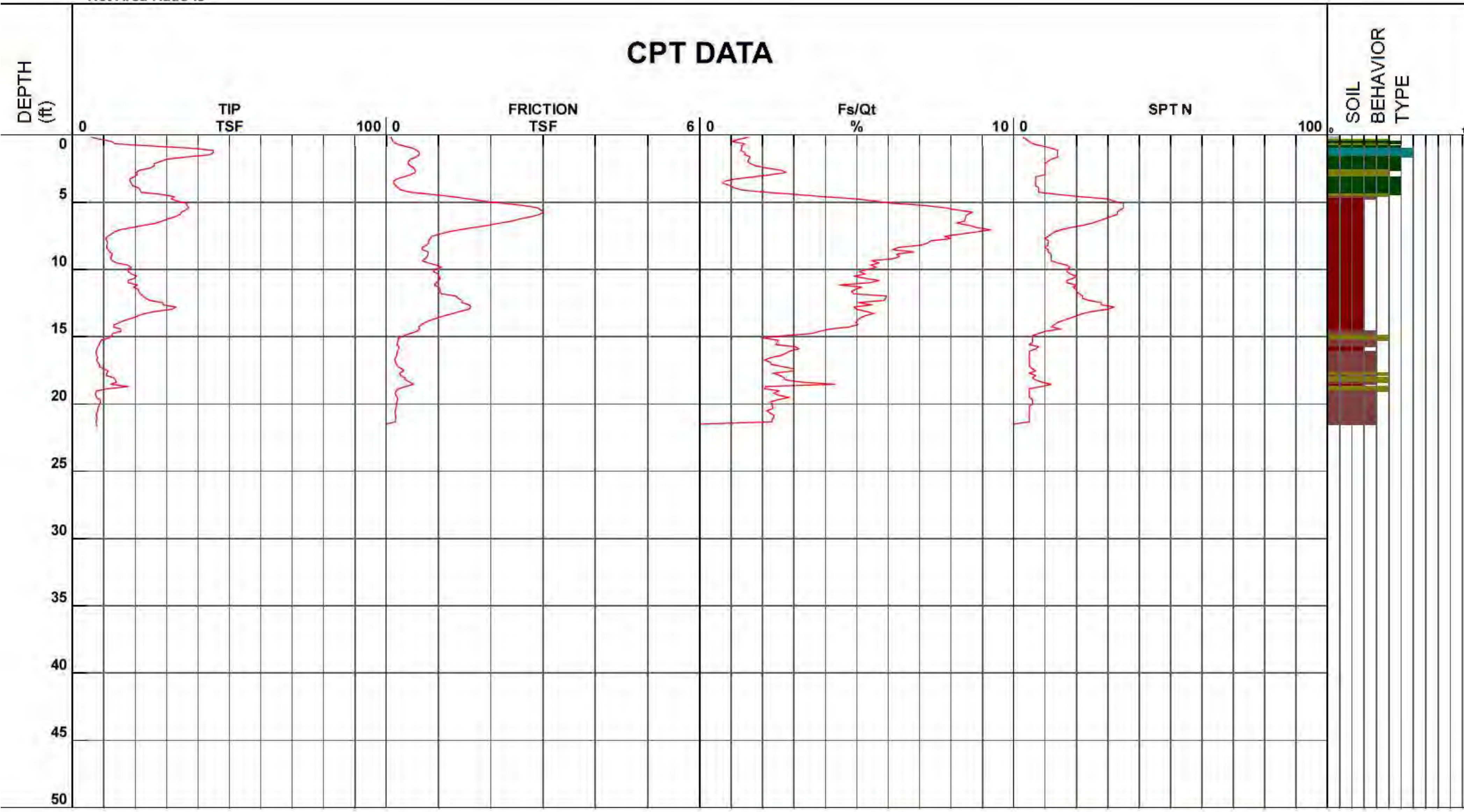
S*Soil behavior type and SPT based on data from UBC-1983

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(804).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-11	Date and Time	4/27/2016 1:00:29 PM	Maximum Depth	21.65 ft
EST GW Depth During Test		8.00 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 (*)

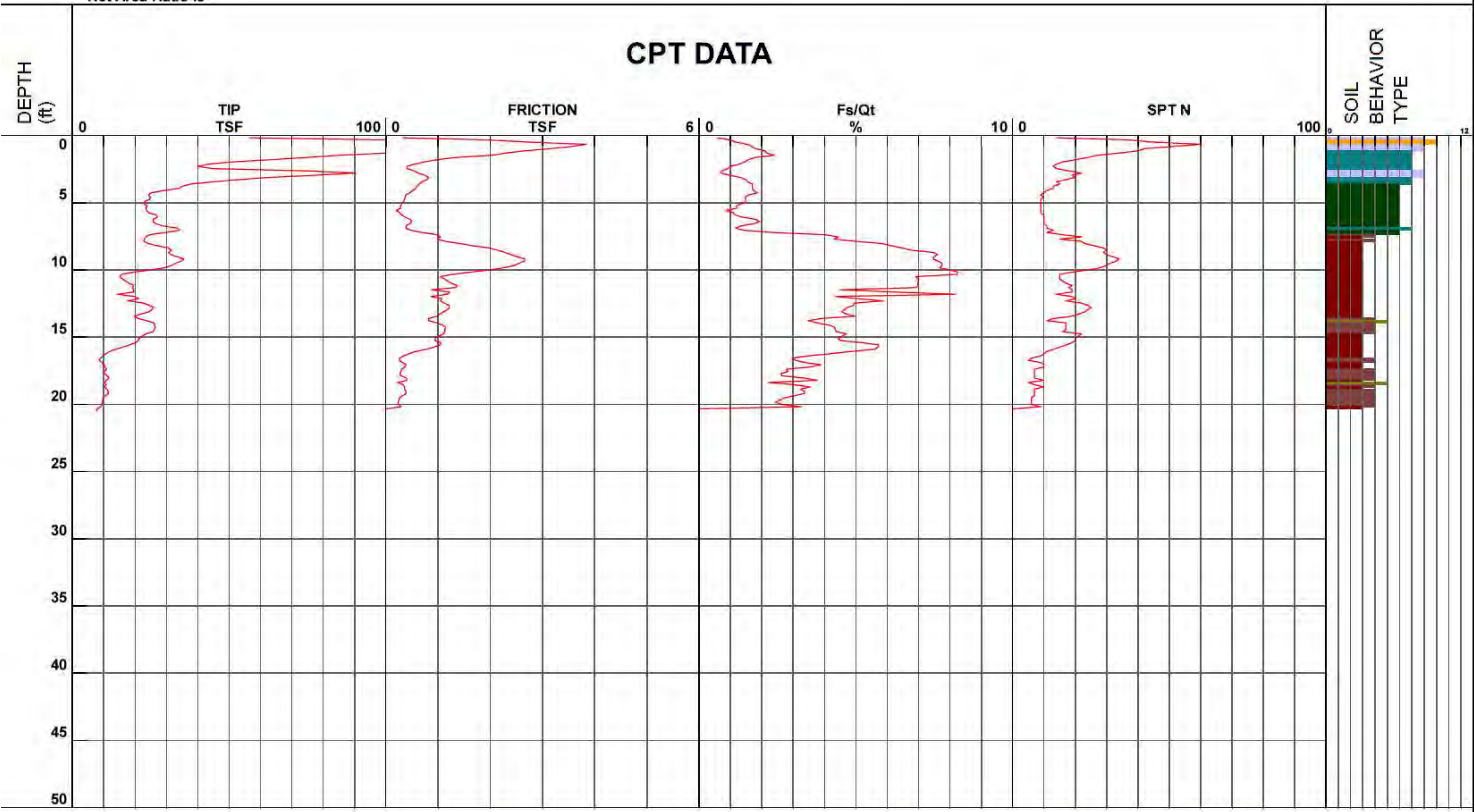
Cone Size 10cm squared

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

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	CB	Filename	SDF(618).cpt
Job Number	20164522	Cone Number	DDG1281	GPS	
Hole Number	CPT-12	Date and Time	3/15/2016 8:07:16 AM	Maximum Depth	20.51 ft
EST GW Depth During Test	8.00 ft				

Net Area Ratio .8



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

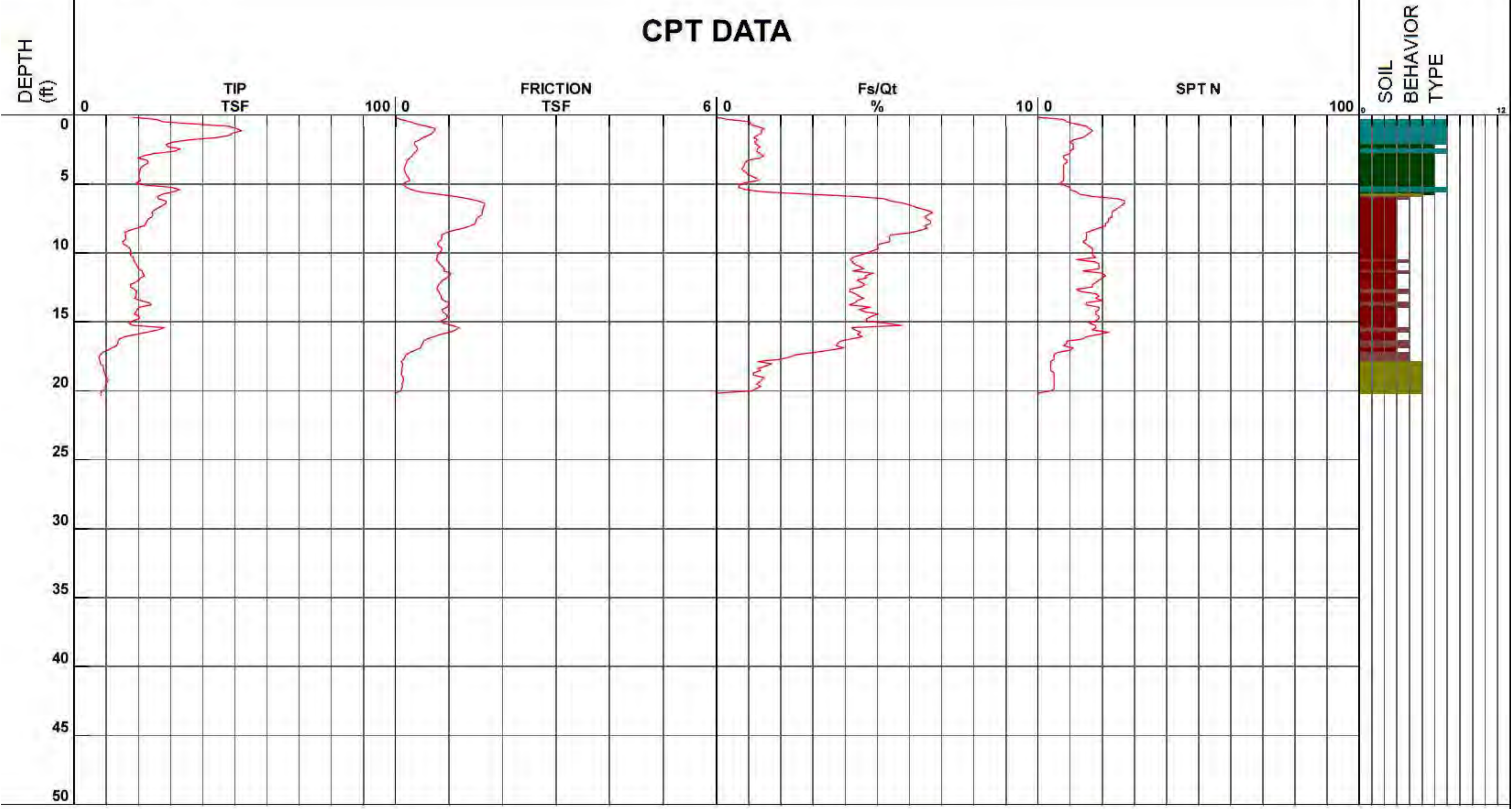
Cone Size 10cm squared

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

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH-SF	Filename	SDF(818).cpt
Job Number	20164522	Cone Number	DDG1281	GPS	
Hole Number	CPT-13	Date and Time	4/28/2016 6:35:35 PM	Maximum Depth	20.34 ft
EST GW Depth During Test		8.00 ft			

Net Area Ratio .8



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

Cone Size 10cm squared

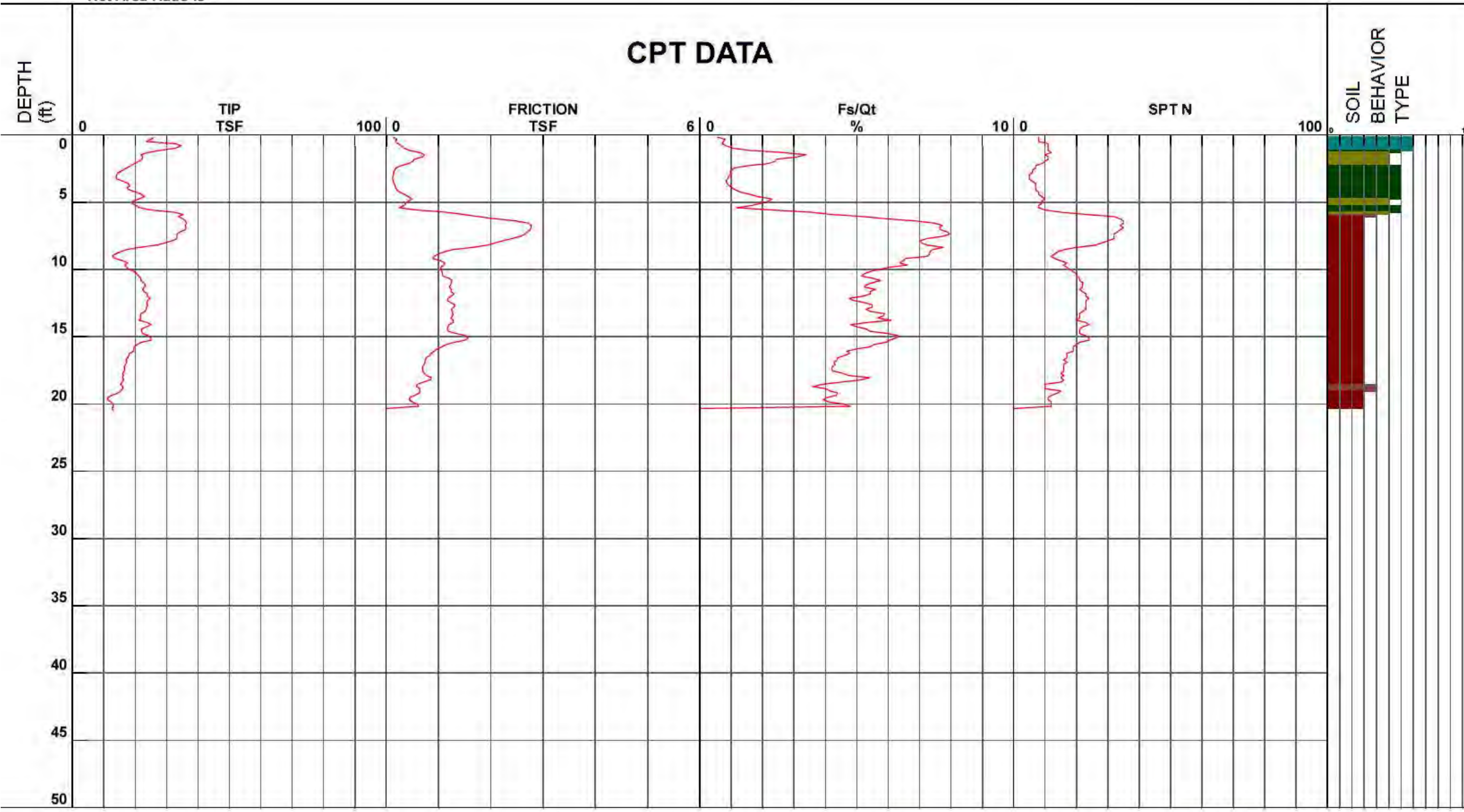
S*Soil behavior type and SPT based on data from UBC-1983

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH	Filename	SDF(802).cpt
Job Number	20164522	Cone Number	DDG1350	GPS	
Hole Number	CPT-14	Date and Time	4/27/2016 11:57:43 AM	Maximum Depth	20.51 ft
EST GW Depth During Test		8.00 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 (*)

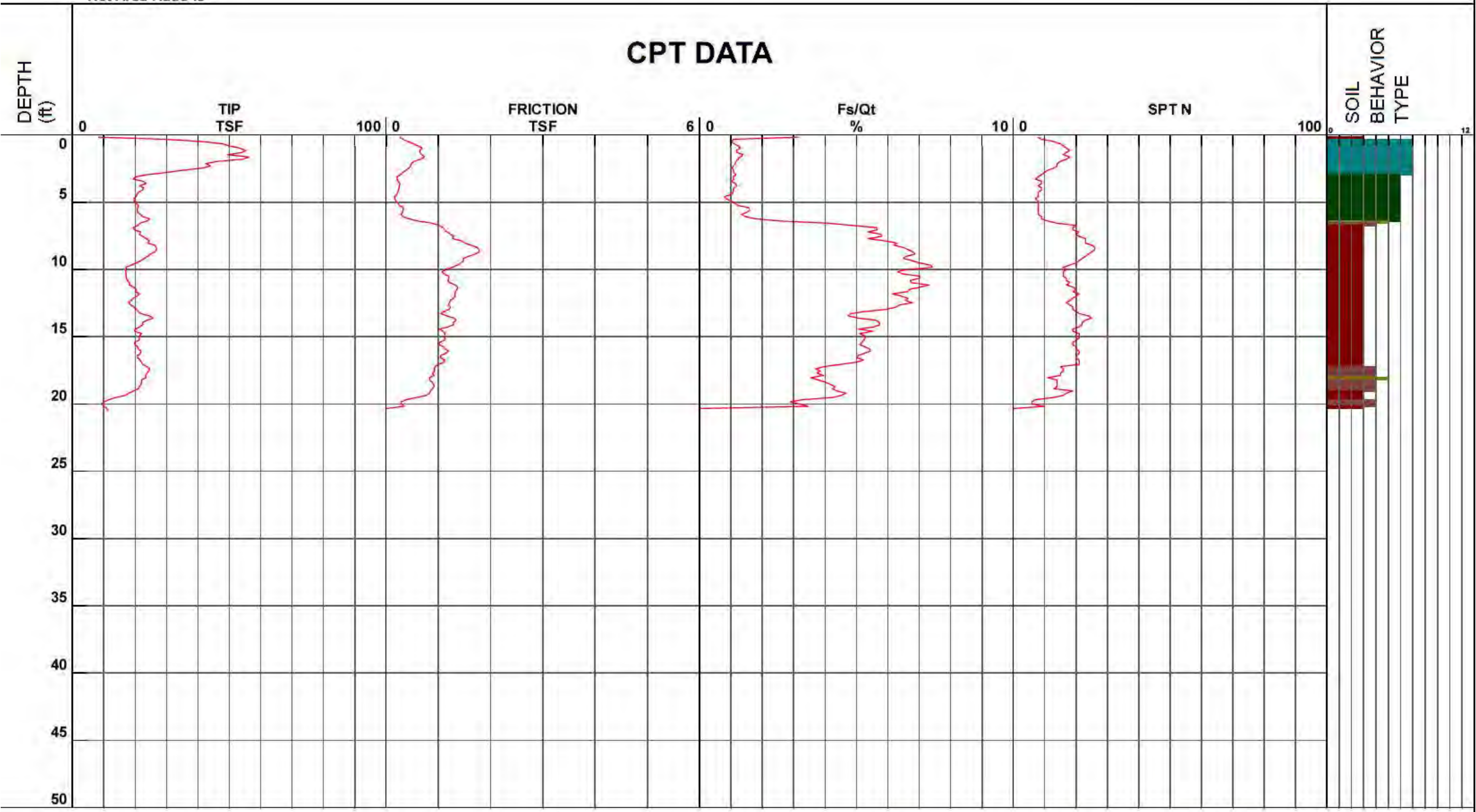
Cone Size 10cm squared

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

Kleinfelder Inc

Project	PACLAD Project 1926	Operator	BH-JH-SF	Filename	SDF(820).cpt
Job Number	20164522	Cone Number	DDG1281	GPS	
Hole Number	CPT-15	Date and Time	4/28/2016 7:37:29 PM	Maximum Depth	20.51 ft
EST GW Depth During Test	8.00 ft				

Net Area Ratio .8



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

Cone Size 10cm squared

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

KLEINFELDER
Bright People. Right Solutions.

Appendix D

APPENDIX D

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

Important Information about Your

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical-Engineering Report Is Based on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical-engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical-engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold-prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical-engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold-prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your GBA-Member Geotechncial Engineer for Additional Assistance

Membership in the GEOPROFESSIONAL BUSINESS ASSOCIATION exposes geotechnical engineers to a wide array of risk confrontaton techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your GBA-member geotechnical engineer for more information.

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