GEOTECHNICAL INVESTIGATION REPORT
COMSTOCK DATA CENTER
1111 COMSTOCK STREET
SANTA CLARA, CALIFORNIA

Project No. 20193908.001A

April 25th, 2019

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April 25, 2019
Project No. 20193908.001A

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SUBJECT: Design-Level Geotechnical Investigation

PROJECT: Comstock Data Center
1111 Comstock Street
Santa Clara, California

Dear Mr. Laag:

The attached report presents the results of Kleinfelder's geotechnical investigation for the proposed Comstock Data Center to be located at 1111 Comstock Street in Santa Clara, California. The enclosed report provides a description of the investigation performed and geotechnical recommendations for site grading and foundation design.

Conclusions and recommendations presented in the enclosed report are based on a site reconnaissance, subsurface exploration and laboratory testing programs, review of published geologic and seismic studies, geotechnical analyses, and our experience in the site area. Consequently, variations in subsurface soil conditions may be found in localized areas during construction. If significant variation in the subsurface conditions is encountered during construction, Kleinfelder should observe the encountered conditions, review the recommendations presented herein and provide supplemental recommendations, if deemed necessary.

Additionally, pre-final project plans and specifications should be reviewed by our office prior to their issuance to verify conformance with the general intent of the recommendations presented in the enclosed report.

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We appreciate the opportunity to be of service to you on this project. If you have any questions regarding this report or if we can be of further service, please contact the undersigned.

Respectfully submitted,

KLEINFELDER, INC.

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1 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Comstock Data Center to be located at 1111 Comstock Street in Santa Clara, California. A site vicinity map showing the project location is presented on Figure 1.

Kleinfelder's understanding of the project is based on information provided by Mr. Nicolas Laag of Prime Data Centers, Mr. Kurt Lindorfer, S.E. of Raradigm Structural Engineers, Inc. and Mr. Ted Mahl, A.I.A of CAC Architects, as well as various correspondence over emails until the submission date of this report.

1.1 PROJECT DESCRIPTION

It is our understanding that the project will consist of a 3-story, at-grade structure with an equipment platform above the roof, approximately 70-feet high. The proposed building have a footprint of approximately 23,000 square feet at the approximately 1.5-acres site. The site is currently occupied by a one-story at-grade commercial building surrounded by landscaping and at-grade paved parking. Based on a schematic facility layout plans provided to us, we understand that demolition of the existing building and construction of a new structure at approximate the same footprint is currently planned for the site. A Site Map showing the approximate location of the proposed building is presented on Figure 2.

Grading plans were not provided to us upon submittal of this report; however, we assume site grading will be minor and consist of cuts and fills less than 2 feet. As per the project structural engineer, building column grid spacing are expected to be 30 feet by 42 feet, with maximum combined dead plus live column loads of 1,150 kips (400 kips dead-load and 750 kips live-load). Based on the loads, subsurface conditions, and our communications with the structural engineer, we expect that foundation type will consist of a deep foundation system.

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1.2 PURPOSE AND SCOPE OF SERVICES

Our scope of services was presented in our proposal dated March 13, 2019. The purpose of this geotechnical investigation was to explore and evaluate the site's subsurface conditions at selected locations to obtain data and develop conclusions and geotechnical recommendations to be utilized during the design and construction of the proposed development. The scope of services included the following:

- A site visit to mark exploration locations for the drilling permit and Underground Service Alert, and coordinate site access
- Review of published geologic and seismic reports and maps covering the site area
- Performance of three Cone Penetration Tests (CPTs)
- Drilling and sampling of two borings (60 feet and 40 feet deep)
- Geotechnical laboratory testing to assess the physical characteristics and engineering properties of the site soils
- Analyses of the field and laboratory test data and development of conclusions and recommendations for design and construction
- Preparation of this report

Our scope of services did not include the assessment of site environmental characteristics, particularly those involving hazardous substances.

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2 FIELD EXPLORATION

2.1 PREPARATION FOR EXPLORATIONS

Prior to subsurface exploration, the site was marked and the Underground Service Alert (USA) was contacted to provide clearance for utilities within the public right-of-way. As an added precaution, in order to reduce the risk of encountering buried utilities during our subsurface exploration, a private utility locator was retained to clear our marked-out locations. A drilling permit was obtained from the Santa Clara Valley Water District (SCVWD). A site-specific health and safety plan was prepared for the field exploration activities. This plan was discussed with the field crew prior to the start of field exploration work.

2.2 CONE PENETRATION TESTING

Three Cone Penetration Tests (CPTs) were performed by ConeTec, Inc. of San Leandro, California on March 26, 2019. The CPTs were advanced to depths of approximately 50 to 120 feet below the existing ground surface. The information gathered from the CPTs was used for subsurface characterization identifying potential liquefiable and soft soils, and for evaluating foundation design parameters.

The CPT testing consisted of pushing an instrumented cone-tipped probe (piezocone) into the ground while simultaneously recording the resistance to penetration at the cone tip and along the friction sleeve, as well as pore water pressure. Seismic shear wave velocity measurements were collected from CPT-2. The CPTs were backfilled with cement grout in accordance with local requirements. The approximate locations of our exploratory CPTs are shown on the Exploration Location Plan, Figure 2. The CPT results for CPT-1 through CPT-3 are presented in Appendix A.

The CPT soundings were performed in general accordance with ASTM D5778 using an electronic cone penetrometer. A set of hydraulic rams were used to continuously push the cone and rods into the soil at a rate of approximately 2 centimeters per second while the cone tip resistance (Qt) and sleeve friction resistance (Fs) were recorded in 1.5-centimeter increments. The testing was performed using a 30-ton push capacity, truck-mounted, CTP rig.

The stratigraphic interpretation of the CPT data was performed based on relationships between cone tip resistance and sleeve friction resistance versus penetration depth. The friction ratio,

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which is sleeve friction resistance divided by cone tip resistance, is a calculated parameter which is used to infer soil behavior type. Cohesive soils (clays) generally have high friction ratios, low cone tip resistance values, and generate large excess pore water pressures. Cohesionless soils (sands) generally have lower friction ratios, high cone tip resistance values, and generate small excess pore water pressures. The interpretation of soil behavior type from the cone data was carried out based on Robertson and Campanella (1989). It should be noted that it is not always possible to clearly identify a soil type based on cone tip resistance and sleeve friction resistance. In these situations, experience, judgment, comparison with drilled borings, and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type. The CPT data (cone tip resistance, sleeve friction resistance, pore pressure, shear velocity, and equivalent Standard Penetration Test blow counts) versus penetration depth below the existing ground surface are presented in Appendix A.

2.3 EXPLORATORY BORINGS

Under subcontract to Kleinfelder, Exploration Geoservices Inc. of San Jose, California drilled the borings on April 2, 2019 using a truck-mounted, Mobile B53 drill rig equipped with hollow-stem augers and mud rotary drilling capabilities.

To explore subsurface conditions and to obtain samples for laboratory testing, two exploratory borings, labeled B-1 and B-2 were advanced to depths of about 60 to 40 feet below the ground surface, respectively. The borings (Borings B-1 and B-2) were advanced adjacent to CPT-2 and CPT-3, respectively, for direct evaluation of physical samples to correlated soil behavior. Boring B-1 was drilled using mud rotary drilling methods while Boring B-2 was drilled using hollow stem auger methods.

Borings were located in the field by pacing methods from existing landmarks. The approximate locations of our exploratory borings are shown on the Exploration Location Plan, Figure 2.

A Kleinfelder professional maintained logs of Borings, visually classified the soils encountered according to the Unified Soil Classification System (American Society for Testing and Materials [ASTM] D2488 visual-manual procedure) and obtained samples of the subsurface materials. The Unified Soil Classification System descriptive criteria are presented on the Graphics Key, on Figures 1 and 2 in Appendix B, along with a key to the symbols used on the boring logs. Following laboratory testing, the field visual classifications were revised, as appropriate, based on ASTM

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D2487. A Soil Description Key is provided on Figure B-2 in Appendix B. Logs of Borings B-1 and B-2 are presented on Figures B-3 through B-4.

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 diameter standard penetration test (SPT) sampler into undisturbed soil with a 140-pound automatic hammer free-falling a distance of 30 inches. The California sampler is in general conformance with ASTM D3550 and was used with brass liners. The SPT sampler is in general conformance with ASTM D1586 and was used without liners to obtain SPT blow counts for use in engineering analyses.

Blow counts were recorded at 6-inch intervals for each sample attempt and are reported on the boring logs. Blow counts shown on the boring logs have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency. However, sampler size correction factors were applied to estimate the sample apparent density noted on the boring logs. The consistency terminology used in soil descriptions is based on field observations (shown on Figure B-2). Soil samples obtained from the borings were sealed and packaged in the field to reduce moisture loss and disturbance and returned to Kleinfelder's laboratory for further examination and testing.

After the drilling of the borings was completed, they were backfilled with neat cement grout and patched with pre-mixed concrete. Cuttings and fluids from the borings were containerized in 55-gallon drums and disposed of by our drilling contractor.

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3 LABORATORY TESTING

Laboratory tests were performed on selected samples recovered from the borings to evaluate physical characteristics and engineering properties. Laboratory testing included the following tests:

- Unit Weight (ASTM D2937)
- Moisture Content (ASTM D2216)
- Percent Finer Than No. 200 Sieve (ASTM D1140)
- Atterberg Limits (ASTM D5318)
- Unconsolidated Undrained Triaxial Shear (ASTM D2850)

In addition, one near-surface soil sample was submitted to CERCO Analytical for a brief corrosion analysis. The results of the corrosion testing are presented in Section 6.12. The laboratory test results are summarized on the boring logs and are presented on Figures C-1 through C-3 in Appendix C.

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4 GEOLOGY AND SEISMICITY

This section of the report discusses regional geology, area and site geology, and geologic hazards that could impact the site. The hazards considered include seismic ground shaking fault-related ground surface rupture, seismically-induced ground failures (liquefaction, lateral spreading and dynamic compaction), and expansive soils.

4.1 AREA AND SITE GEOLOGY

The site is centrally located in the Santa Clara Valley, 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 in the region. The basement rock in the north-central portion of this province consists of the Great Valley Complex, a Jurassic (approximately 145 to 175 million years old) volcanic ophiolite sequence with associated Lower Cretaceous to Upper Jurassic (approximately 100 to 160 million years old) sedimentary rocks, and the Franciscan Complex, a subduction complex of diverse groups of igneous, sedimentary, and metamorphic rocks of Cretaceous to Upper Jurassic age (65 to 160 million years old). The Great Valley Complex was tectonically juxtaposed with the Franciscan Complex (most likely during subduction accretion of the Franciscan Complex), and these ancient fault boundaries are truncated by a modern right-lateral fault system that includes the San Andreas, Hayward-Rodgers Creek, and Calaveras faults. Located approximately 11.4 miles southwest of the site, the San Andreas fault defines the westernmost boundary of the local bedrock. In the site vicinity, the Great Valley Sequence and Franciscan Complex are unconformably overlain by Tertiary age (approximately 2.6 to 65 million years old) continental and marine sedimentary and volcanic rocks. These Tertiary age rocks are locally overlain by younger Quaternary (approximately 2.6 million years old to present day) alluvial deposits.

4.2 AREA AND SITE GEOLOGY

The site has been mapped by Wentworth et al. (1999), the California Geological Survey (2002), Witter et al. (2006), and by Dibblee and Minch (2007), among others. Wentworth et al. (1999) have mapped the site as underlain by Holocene age (approximately 11,700 years old to present day) basin deposits consisting of clay and silty clay, rich in organic material. The California

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Geological Survey (2006) and Witter et al. (2006) are in general agreement, and indicate the site is transected by a north-south trending geologic contact. The area east of the contact is shown to be underlain by Holocene age fine facies alluvial fan deposits and flood plain overbank deposits, consisting of silt and clay. The area west of the contact is shown to be underlain by Holocene age alluvial fan deposits, comprised of sand, gravel, silt and clay. Dibblee and Minch (2007) indicate the site is underlain by Quaternary age fossiliferous silty clay and organic clay.

4.3 FAULTING AND SEISMICITY

The site is not located within an Earthquake Fault Zone as defined by the California Geological Survey (CGS, 2002) in accordance with the Alquist-Priolo Earthquake Fault Zone Act of 1972, nor a Santa Clara County Fault Hazard Zone. The nearest zoned active fault is the Hayward-Rodgers Creek fault, which, according to the CGS (2002), is located approximately 6.2 miles northeast of the site boundary.. Moderate to major earthquakes generated on the Hayward-Rodgers Creek and other faults in the region can be expected to cause strong ground shaking at the site.

The U.S. Geological Survey (USGS, 2010) (Quaternary Fault and Fold Database http://gldims.cr.usgs.gov/qfault) and Petersen et al. (2008) indicate the Monte Vista-Shannon fault is located approximately 7.4 miles southwest of the site. This fault has not been zoned as active by the CGS but is considered a source of seismic shaking by the USGS.

The U.S. Geological Survey (USGS, 2010, Quaternary Fault and Fold Database http://gldims.cr.usgs.gov/qfault) also identifies the Silver Creek fault zone approximately 1.9 miles to the northeast of the site, the San Jose fault approximately 2.2 miles to the southwest of the site, and the Stanford fault approximately 3.4 miles to the southwest of the site. According to the USGS, the faults have exhibited activity in the Quaternary. The faults have not been zoned as active by the CGS and are not considered a source of seismic shaking by the USGS.

The proximities and seismic parameters of significant faults in the vicinity of the site are listed in Table 4.1. For faults with multiple segmentation scenarios we have only listed parameters for the scenario rupturing the most segments (i.e., the most severe scenario). The locations of the faults and associated parameters presented on Table 4.1 are based on Petersen et al. (2008). The maximum earthquake magnitudes presented in this table are based on the moment magnitude

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scale developed by Kanamori (1977). Felzer (2008) details calculations of California seismicity rates including correction for magnitude rounding and error, Gutenberg-Richter b value and seismicity rates. The following table identifies the significant faults in the area and their corresponding parameters.

TABLE 4.1
SIGNIFICANT FAULTS

Fault Name	Closest Distance to Site* (miles)	Magnitude of Characteristic Earthquake**	Slip Rate (millimeters/year)
Monte Vista-Shannon	7.4	6.5	0.4
Hayward-Roger's Creek	6.2	7.3	9
Calaveras	9.1	7.0	6
San Andreas	10.9	8.1	17-24
Zayante-Vergales	23.9	7.0	0.1
Greenville Connected	23.9	7.0	2
San Gregorio	24.9	7.5	5.5
Mount Diablo Thrust	25.8	6.7	2
Monterey Bay-Tularcitos	32.4	7.3	0.5

* *Closest distance to the potential rupture.*

** *Moment magnitude: An estimate of an earthquake's magnitude based on the seismic moment (measure of an earthquake's size utilizing rock rigidity, amount of slip, and area of rupture).*

According to Petersen et al. (2008), characterizations of the Hayward-Rodgers Creek, Calaveras, and the San Andreas faults are based on the following fault rupture segments and fault rupture scenarios:

- The Hayward-Rodgers Creek fault has been characterized by three segments and six rupture scenarios plus a floating earthquake. The three segments are the Rodgers Creek fault (RC), the Hayward North (HN), and the Hayward South (HS).
- The Calaveras fault includes three segments and six rupture scenarios, plus a floating earthquake. The three segments are southern (CS), central (CC), and northern (CN).

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- The San Andreas fault has been characterized by four segments and nine rupture scenarios, plus a floating earthquake. The four segments are Santa Cruz Mountains (SAS), Peninsula (SAP), North Coast (SAN), and Offshore (SAO).

Future seismic events in this region can be expected to produce strong seismic ground shaking at this site. The intensity of future shaking will depend on the distance from the site to the earthquake focus, magnitude of the earthquake, and the response of the underlying soil and bedrock.

4.4 SEISMICALLY-INDUCED GROUND FAILURE

4.4.1 Liquefaction

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. Liquefaction is most prevalent in loose to medium dense, sandy and gravely soils below the groundwater table, but can also occur in non-plastic to low-plasticity finer grained soils. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations or “cyclic mobility”, increased lateral earth pressures on retaining walls, liquefaction settlement, and lateral spreading or “flow failures” in slopes.

The site is located within a State of California Seismic Hazard Zones map (CGS, West San Jose Quadrangle, 2002) for liquefaction where areas of historical occurrence of liquefaction, or local geological, geotechnical and ground-water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required (CGS, 2002).

Boulanger and Idriss (2006) was used as the compositional screening basis for the Idriss and Boulanger (2006 and 2008) and Youd et al. (2001) analysis methods. For CPT analyses, we used the recommendations of Youd et al. (2001) to consider layers with soil behavior type index, I_c<2.6 as potentially liquefiable.

For layers that met the compositional criteria, liquefaction triggering (factor of safety) analyses were performed using methodologies proposed by Youd et al. (2001), Seed et al (2003), Idriss &

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Boulanger (2006, 2008), and Moss et al. (2006). The analyses utilized both SPT data from our rotary wash borings and tip resistance data from our CPT soundings. In order to perform liquefaction analysis, estimates of earthquake magnitude and peak ground acceleration (PGA_m) are needed. Based on a probabilistic seismic hazard deaggregation using the USGS Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/, and PGA from the USGS Seismic Design Maps Application (https://earthquake.usgs.gov/designmaps/us/application.php) liquefaction analyses were performed using a moment magnitude of 7.9 and a MCE_R peak ground acceleration (PGA) of 0.5g.

Liquefaction induced volumetric settlements were estimated using the methods of Tokimatsu and Seed (1987), Idriss and Boulanger (2008), and Cetin et al. (2009). In general, the Idriss and Boulanger (2008) and Cetin et al. (2009) methods provide reasonably consistent results, while the Tokimatsu and Seed (1987) method provides lower estimates of settlement.

Our subsurface exploration and subsequent calculations indicate that several sand layers were encountered below the design ground water depth of 8 feet are considered potentially liquefiable. Our analysis indicates that post-liquefaction settlements ranging from about 1¼ to about 3 inches could occur within the upper 50 feet.

4.4.2 Lateral Spreading

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material. These phenomena typically occur adjacent to free faces such as slopes and creek channels. There are no open faces within 200 feet of the site where lateral spreading could occur and no potentially-liquefiable soils or weak planes; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.4.3 Dynamic Compaction

Another type of seismically induced ground failure that can occur as a result of seismic shaking is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. As the soils encountered above the design ground water depth of 8 feet were predominantly cohesive soils, in our opinion, the potential for shaking-related dynamic compaction is low.

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4.4.4 Expansive Soils

Soil expansion potential was characterized via laboratory testing of the near-surface soils during our geotechnical investigation for the site. Highly expansive clayey soils were encountered near the ground surface throughout the site.

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5 SITE AND SUBSURFACE CONDITIONS

5.1 SITE DESCRIPTION

The approximately 1.5-acre project site currently contains an existing unoccupied 1-story at-grade commercial building and a paved parking lot at the north and west sides of the existing building. The project site is relatively flat with a ground surface elevation of approximately 38 feet.

The site is bounded by Digital Realty to the north, Comstock Street to South, and commercial developments to the east and west.

Surface pavements generally consisted of 3 inches of asphalt concrete. Based on visual observations, the existing pavements are in fair to poor condition.

5.2 SUBSURFACE CONDITIONS

The subsurface conditions described below are based on information obtained from the borings and CPTs advanced for this investigation. Logs of the CPTs and borings are presented in Appendixes A and B.

5.2.1 Stratigraphy

Below the surface materials, our exploratory Borings (B-1 and B-2) encountered very stiff high plasticity clay (CH) to a depth of about 4 feet below the existing surface grades.

Below the high plasticity clay in Boring B-1, we encountered a stiff clay to a depth of approximately 7½ feet below ground surface, underlain by a stiff sandy clay material to a depth of approximately 18½ feet below ground surface. This was followed by a medium dense to dense poorly graded sand to a depth of approximately 26 feet, underlain by a dense poorly graded sand to a depth of approximately 29 below ground surface. This was followed by a medium dense layer of silty sand to a depth of approximately 34½ feet, underlain by a stiff sandy clay layer to a depth of approximately 41 feet, underlain by a medium dense poorly graded sand with silt and gravel layer to a depth of approximately 44½ feet, followed by a stiff sandy clay layer to the maximum depth of 60 feet below the existing ground surface.

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Below the high plasticity clay in B-2, we encountered very stiff clays to a depth of approximately 8½ feet, underlain by a soft to medium stiff sandy clay layer to a depth of approximately 10½ feet, followed by a stiff lean clay to a depth of approximately 19½ feet. This was followed by a dense to medium dense poorly graded sand and gravel layer to a depth of approximately 28½ feet, underlain by a very stiff lean clay to a depth of approximately 33½ feet, followed by a medium dense clayey sand to the maximum depth of 40 feet below the existing ground surface.

Our CPTs generally indicated soil behavior types including clay, silt, clayey silt to silty clay, including sands at various depths, and sandy silt to clayey silt to a depth of 120 feet below the ground surface, which is the maximum depth explored.

5.2.2 Groundwater

Groundwater was encountered at a depth of approximately 8 feet below the ground surface in Boring B-2. Groundwater depth was inferred by CPTs (CPT-1 and CPT-2) pore pressure measurements ranging from 7 to 8½ feet below ground surface. All measurements were taken at the time of exploration and may not represent the stabilized levels that can differ from the initial levels encountered. Ground water was not measured in Boring B-1 due to the mud rotary drilling method used.

Based on the historical groundwater data obtained from the Seismic Hazard Zone Reports (CGS, 2002), historic high groundwater is less than 10 feet below the ground surface. Based on the ground water levels encountered within our explorations and the above data, a design groundwater depth of 8 feet below the ground surface was used for our analysis.

It is possible that groundwater conditions at the site could change due to variations in rainfall, groundwater withdrawal or recharge, construction activities, well pumping, or other factors not apparent at the time the explorations were performed.

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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 and CPTs advanced for this project, in addition to our review of available geotechnical data, published geologic maps and reports. The conclusions and recommendations that follow are based on these interpretations. If soil or groundwater conditions exposed during construction vary from those presented in this report, Kleinfelder should be notified to evaluate whether our conclusions or recommendations should be modified.

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6 CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

From a geotechnical standpoint, the proposed construction is considered feasible provided the recommendations presented in this report are incorporated into the project design and construction. The following sections discuss conclusions and recommendations with respect to geologic and seismic hazards, California Building Code (CBC) seismic design considerations, site preparation, foundation design, pavement design, and other construction considerations.

6.2 SEISMIC DESIGN CONSIDERATIONS

6.2.1 Site Class

In developing seismic design criteria, the characteristics of the soils underlying the site are an important input to evaluate the site response. Based on the information obtained from the borings and CPTs, published geologic literature and maps, and on our interpretation of CBC criteria, it is our opinion that the site can be classified as Site Class D according to Table 20.3-1 of the ASCE7-10. Site Class D is defined as a soil profile consisting of stiff soil profile with a shear wave velocity between 600 feet per second and 1,200 feet second, 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 pounds per square foot and 2,000 pound per square foot in the top 100 feet. The shear wave velocity measurements performed for this investigation at CPT-2 resulted in a shear wave velocity of 810 feet per second (215 meters per second). Therefore, we have classified the site as Soil Classification D, according to Chapter 20 of ASCE 7-10.

6.2.2 Seismic Design Parameters

The site is located approximately at the following coordinates:

- Latitude: 37.375046°
- Longitude: -121.953088°

For a 2016 California Building Code (CBC) based design, the estimated Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1), associated soil amplification factors (F_a and F_v), and mapped peak ground acceleration (PGA)

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are presented in Table 5-1. Corresponding site modified (S_{MS} and S_{M1}) and design (S_{DS} and S_{D1}) spectral accelerations, PGA modification coefficient (F_{PGA}), PGA_M, risk coefficients (C_{RS} and C_{R1}), and long-period transition period (T_L) are also presented in Table 5-1. Presented values were estimated using Section 1613.3 of the 2016 California Building Code (CBC), chapters 11 and 22 of ASCE 7-10, and the United States Geological Survey (USGS) U.S. seismic design maps.

Table 5-1

Ground Motion Parameters Based on 2016 CBC

Parameter	Value	Reference
S_S	1.5g	2016 CBC Section 1613.3.1
S_1	0.6g	2016 CBC Section 1613.3.1
Site Class	D	2016 CBC Section 1613.3.2
F_a	1	2016 CBC Table 1613.3.3(1)
F_v	1.5	2016 CBC Table 1613.3.3(2)
PGA	0.5g	ASCE 7-10 Figure 22-7
S_{MS}	1.5g	2016 CBC Section 1613.3.3
S_{M1}	0.9g	2016 CBC Section 1613.3.3
S_{DS}	1 g	2016 CBC Section 1613.4.4
S_{D1}	0.6g	2016 CBC Section 1613.4.4
F_{PGA}	1	ASCE 7-10 Table 11.8-1
PGA_M	0.5g	ASCE 7-10 Section 11.8.3
C_{RS}	1.126	ASCE 7-10 Figure 22-17
C_{R1}	1.071	ASCE 7-10 Figure 22-18
T_L	12 seconds	ASCE 7-10 Figure 22-12

According to Section 1803.5.12 of the 2016 CBC, in the absence of a site-specific ground motion hazard analysis, the MCE geometric mean peak ground acceleration adjusted for Site Class effects can be determined based on Section 11.8.3 of ASCE 7-10.

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6.3 SITE PREPARATION

6.3.1 General

It is anticipated that site grading can be performed with conventional grading equipment and techniques. Site grading is anticipated to be minimal and to consist of minor grading for drainage and site access. Recommendations for site preparation and earthwork construction are presented below. All references to compaction, maximum density and optimum moisture content are based on ASTM D1557, unless otherwise noted.

6.3.2 Stripping and Demolition

As part of the demolition process, existing foundations and other improvements should be removed. Excavations from removal of foundations, underground utilities or other below ground obstructions where located outside of the planned excavation for the Data Center should be cleaned of loose soil and deleterious material and backfilled with compacted engineered fill.

Areas to receive fill and structures should be stripped of existing surface vegetation, organic topsoil, debris, and any other deleterious materials prior to over-excavation or placement of engineered fill. 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 building footprints, concrete flatwork, and any other facilities supported on grade.

6.3.3 Existing Utilities

Active or inactive utilities within the construction area should be protected, relocated, or abandoned as appropriate. All utilities in conflict with future deep foundations will need to be relocated. Pipes between 2 and 6 inches in diameter may be left in place within the limits of structure if they are filled with grout or sand/cement slurry (sand slurry is not acceptable) and capped at both ends. Pipes larger than 6 inches in diameter within planned improvements should be removed. Active utilities to be reused should be carefully located and protected during demolition and construction.

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6.3.4 *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 undocumented fill should be over-excavated to expose firm native soil, as approved by Kleinfelder. Undocumented fill was not encountered during or investigation, however, any fills encountered during site grading should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater, and be replaced as engineered fill.

6.3.5 *Soil Expansion Potential*

Based on the results of laboratory testing performed on a subgrade soil sample collected from Boring B-1 at a depth of about 2½ feet (Plasticity Index of 32 and Liquid Limit of 53), the on-site near-surface soils are considered to have high expansion potential. These surficial soils tend to shrink and 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. These measures may include over-excavating, moisture conditioning and replacing the near-surface soils with non-expansive engineered fill, or by treating them with high-calcium quicklime to reduce their expansion potential.

The limits of over-excavation for the interior floor slabs should extend horizontally a distance of 5 feet beyond the perimeter of the structure. Exterior flatwork sections that abut building doorway thresholds should be constructed similar to the interior building floor slab to reduce the potential for joint offsets. Doweling of the flatwork sections to the perimeter building foundation is also recommended. Subgrade preparation for exterior concrete flatwork or mat slabs should extend a minimum of 2 feet laterally beyond the perimeter.

To reduce the potential for shrinkage and swelling cause by expansive clay soils, we recommend the soils beneath interior floor slabs and exterior concrete flatwork or equipment mat slab foundations be over-excavated and replaced with at least 24 and 12 inches of imported non-expansive engineered fill (NEF), respectively. Beneath the interior slabs, it has been common practice to place a capillary break consisting of at least 4 inches of free draining crushed gravel on the finished subgrade soil that, in turn, is overlain by a flexible sheet membrane, such as Stego Wrap™, Moistop Plus™, or an equivalent meeting the requirements of ASTM E1745-09, that

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serves as a water and/or moisture vapor retarder. The crushed gravel should be graded so that 100 percent passes the 1-inch sieve and less than 5 percent passes the No. 4 sieve. Care should be taken to properly place, lap, and seal the membrane in accordance with the manufacturer's recommendations to provide a vapor tight barrier. Tears and punctures in the membrane should be completely repaired prior to placement of concrete.

As an alternate to non-expansive fill placement (NEF), the on-site soil may be treated in place with high-calcium quicklime to reduce the expansion potential. The depth of treatment should be as recommended for non-expansive engineered fill. If used, the quicklime application rate should be at least 4.5 pounds per cubic foot of soil treated. Lime treatment should be performed by a specialty contractor experienced in this work and should be performed in accordance with Caltrans Standard Specifications. Lime treatment will result in a soil pH over 10 and should not be present within proposed landscape areas. In pavement areas, subgrades treated with high calcium quicklime are generally more stable, less prone to shrink and swell cycles, can be constructed in wet soil conditions, and can provide for a reduction in the aggregate base thickness. If lime treatment is being considered for non-expansive engineered fill areas, it may be cost effective to treat the pavement subgrades as well. The depth of treatment in pavement areas should be 12 inches.

6.3.6 Scarification and Compaction

After stripping and performing any necessary over-excavation, the bottoms of excavations should be observed and approved by a qualified geotechnical engineer or his representative prior to the placement of engineered fill or concrete improvements. The bottom of excavations to receive engineered fill should be scarified to a depth of at least 12 inches, uniformly moisture conditioned to at least 3 percent above the optimum moisture content, and compacted to between 88 and 92 percent relative compaction. In pavement areas that are not being lime treated, the scarified subgrade should be moisture conditioned to at least 4 percent above the optimum moisture content, and compacted to at least 92 percent relative compaction. The subgrade should not be allowed to dry out prior to the placement of engineered fill or aggregate base materials.

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6.4 ENGINEERED FILL

6.4.1 Materials

Material for use in engineered fill should generally be free of visible organic materials, debris, and other deleterious materials, be essentially non-expansive, and have a maximum particle size less than 3 inches in maximum dimension, as described in Table 6-1 below. Onsite, near-surface soils have a high expansion potential and, therefore, should not be used where "non-expansive" fill is required, but may be used outside of non-expansive fill areas. Any import material should be processed to meet the requirements presented in Table 6-1 below.

Table 6-1

Non-Expansive Engineered Fill Recommendations

Fill Requirement		**Test Procedures**	
		ASTM[1]	**Caltrans[2]**
Gradation			
Sieve Size	**Percent Passing**		
3 inch	100	D 422	202
¾ inch	70-100	D 422	202
No. 200	20-70	D 422	202
Plasticity			
Liquid Limit	**Plasticity Index**		
<30	<12	D 4318	204
Organic Content			
Less than 3%		D 2974	---
Expansion Potential			
20 or less		D 4829	---
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 Standard (latest edition)

[2]State of California, Department of Transportation, Standard Test Methods

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Imported 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 aggregate base are not recommended for use as non-expansive 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 the material.

6.4.2 Fill Placement and Compaction Criteria

Fills composed of on-site clay soils should be uniformly moisture conditioned to at least 3 percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to between 88 and 92 percent relative compaction, based on the ASTM D1557 test method. Where required, imported non-expansive, engineered fill materials should be compacted to at least 90 percent relative compaction at a moisture content slightly above the optimum moisture content. In pavement areas, the top 12 inches of subgrade soil and all aggregate base materials should be compacted to at least 95 percent relative compaction at a moisture content slightly above optimum.

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 explorations.

6.5 WET WEATHER CONSTRUCTION

Should site grading be performed during or subsequent to wet weather, the soils may be significantly above optimum moisture content. These conditions could hamper equipment maneuverability and efforts to compact site soils to the required compaction criteria. Disking to aerate, replacement with drier material, stabilization with a geotextile fabric or grid, or stabilization

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with quicklime may be required to reduce excessive soil moisture and facilitate earthwork operations. During wet weather, earthen berms or other methods should be used to prevent runoff water from entering excavations. Runoff water should be collected and disposed of outside the construction limits.

6.6 TEMPORARY EXCAVATIONS

6.6.1 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.

6.6.2 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 utility trenches.

6.7 TRENCH BACKFILL

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

6.8 SURFACE DRAINAGE

It is important that drainage away from the building 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.

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- 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.

6.9 DEEP FOUNDATION SYSTEMS

Based on the structural loads, subsurface conditions, and our communications with the structural engineer, the planned Data Center may be founded on Auger Pressure Grouted Displacement piles (APGD) or Auger-Cast-in-Place (ACIP) pile types with a structural floor slab. supported by the deep foundation elements. APGD and ACIP piles are preferred due to the shallow groundwater conditions and the potential for caving during drilling. It is unknown at this time what the diameter of the deep foundation of choice will be, as such, 16 and 18-inch dimeter options are provided with their associated capacities and analysis.

6.9.1 Pile Axial Capacity

Axial geotechnical resistance of APGD and ACIP piles were developed based on Federal Highway Administration methods using the commercial computer software SHAFT produced by Ensoft, Inc. Figures 3 and 4 are illustrating ultimate static and post-seismic axial compressive capacities, respectively, of 16-inch and 18-inch diameter piles installed from El. +34 feet (roughly bottom of pile cap elevation) to El. -46 feet. The weight of the foundation is not included in our estimates. We have assumed that the top of pile/bottom of pile cap occurs approximately 4 feet below the existing grade.

For evaluation of allowable axial capacity under static conditions, we recommend factor of safety of 2.0 and 1.5 be applied to the ultimate static and post-seismic capacities, respectively.

Ultimate uplift(tension) capacity may be obtained by multiplying the compressive capacity by a factor of 0.8 and adding the buoyant weight of the foundation. A one-third increase in the allowable capacity may be used for consideration of transient loads such as wind or seismic. For allowable

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geotechnical resistance in tension under transient flood, wind or seismic conditions a safety factor of 1.5 may be used. For allowable sustained tension a safety factor of 3 should be used.

For preliminary design purposes, the recommended maximum ultimate axial compressive capacity for the 16-inch pile is 322 kips under post-seismic condition. As for the 18-inch pile, a recommended ultimate geotechnical resistance of 362 kips should be used under post-seismic condition We assumed pile length of 80 feet for the estimates.

To evaluate structural capacity of piles (i.e., rebar design and concrete strength) with the drag load due to liquefaction induced settlement, we recommend an unfactored temporary drag load of 175 kips, acting along the pile in the upper 35 to 40 feet, be added to the dead load and the permanent live load. It should be noted that the estimated drag load is a function of the pile size, pile length, applied dead load, and subsurface soil conditions. If any of these conditions are changed in the future, we need to be notified for re-assessment of the drag load.

6.9.2 Group Effects

The recommended axial geotechnical resistance applies to single, isolated piles with center-to-center spacings of at least three effective pile diameters. For closer pile spacings the geotechnical resistance of individual piles will be reduced. At present pile group configurations have not been determined. Axial geotechnical resistance should be re-evaluated by Kleinfelder when specific pile group configurations are known.

6.9.3 Pile Settlement

Based on the methods outlined by Reese and O'Neill (1999), total static settlement of each ACIP pile should be on the order of 1 percent of the pile diameter for an ACIP pile designed and constructed in accordance with the recommendations presented in this report. This value includes elastic compression of the pile under design loads. The majority of the settlement should occur during and shortly after application of the structure loads.

6.9.4 Lateral Response of Deep Foundations

The response of single piles subjected to lateral loads was analyzed using the commercial program LPILE produced by Ensoft, Inc. The software uses input soil properties to generate soil resistance curves (p-y curves) that are functions of pile deflection. Based on assumed pile-head

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deflections of ¼-inch and ½-inch, as specified by the structural engineer, the analyses produced diagrams of pile deflection, bending moment and shear force versus pile length. These lateral response curves were developed for 16 -inch and 18-inch diameter piles.

Pile caps were assumed to be 3 feet thick, with the bottom of pile cap expected at El +34 feet. Other assumptions:

- Single pile (pile group configurations were not analyzed)
- Free-head and fixed-head conditions
- Modulus of Elasticity = 2,170,000 psi
- Assumed pile length is 80 feet

Lateral response analysis results are presented on Figures 5 through 8.

At present, pile group configurations have not been determined. After pile group configurations are developed, Kleinfelder can conduct lateral response analyses for pile groups.

6.9.5 Construction Considerations – Drilled Piles (APGD & ACIP)

6.9.5.1 General

Installation of all ACIP piles should be observed by the Geotechnical Engineer during construction to verify conformance with our recommendations and project specifications. In the event unforeseen conditions are encountered during construction, the Geotechnical Engineer should evaluate the observed conditions and provide supplemental recommendations as necessary. Drilled piles are normally designed and installed by a specialty contractor under a design-build contract format. Under a design-build arrangement, the contractor will provide the guidelines for development of material specifications (e.g., grout mixture, reinforcing steel requirements) based on the contractor's proprietary means and methods, the subsurface conditions reported in this report, and on the structural load and performance demands. The contractor should use the results of the pile load testing programs, as described in Section 6.9.5.2, in developing the final pile design and planning for installation of the production piles. Kleinfelder should be present in the field to monitor construction during installation of production piles.

6.9.5.2 (APGD & ACIP) Piles Load Testing Program

If performed, full-scale pile load testing and installation of all test piles should be conducted under the direct observation of the Geotechnical Engineer. The test pile locations should be selected by

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the Geotechnical Engineer in conjunction with the Contractor and Structural Engineer. The Geotechnical Engineer should evaluate the data gathered during test pile installation and load testing and provide recommendations for final design of production piles.

The test piles should be of the same type and diameter to be used for production piling and be installed by the same method. We recommend the test piles and reaction piles be installed outside of any proposed pile cap area. Test piles should not be used for structural support of the proposed building. The test pile grout should attain the specified minimum compressive strength prior to testing.

We recommend at least one pile be tested for axial compressive capacity and at least one pile be tested in tension for evaluation of uplift capacity. Static pile compression and tension tests should be conducted in accordance with ASTM methods D1143 and D3689, respectively. The compression test pile should be tested to at least twice the design load. At the load increment of twice the design load, we recommend the test load be held constant for a period of at least 24 hours.

We recommend the compression and tension test piles be installed to the recommended minimum tip elevation. The spacing between individual compression and tension test piles should be at least 6 pile diameters.

6.9.5.3 (APGD and ACIP) Production Piles

The production piles should be installed with the same equipment used to install the load test piles and under the full-time observation of the Geotechnical Engineer. The piles should be installed plumb and to the specified tip elevation. Piles should not deviate from plumb more than 2 percent or reduced capacity may result.

The acceptance criteria for installation of production piles should be based on the results of field observations and testing performed by Kleinfelder during pile installation. Should any pile exhibit defects or otherwise come into question, we recommend high strain dynamic testing be performed to evaluate pile integrity. If needed, high-strain dynamic testing should be performed in accordance with ASTM method D4945.

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6.10 EXTERIOR CONCRETE FLATWORK

Exterior concrete slabs-on-grade should be constructed on soil subgrades prepared as recommended in this report. The slab-on-grade recommendations that follow apply to slabs and flatwork that are not exposed to vehicular traffic.

Once the slab subgrade soil has been moisture conditioned and compacted, the soil should not be allowed to dry prior to concrete placement. If the subgrade soil is too dry, the moisture content of the soil should be restored to the recommended value prior to placement of concrete. Kleinfelder should check the moisture content of the subgrade soil prior to construction of the slabs.

Exterior concrete slabs-on-grade should be cast free from adjacent footings or other non-heaving edge restraints. This may be accomplished by using a strip of ½ -inch asphalt-impregnated felt divider material between the slab edges and the adjacent structure. Frequent construction or control joints should be provided in all concrete slabs where cracking is objectionable. Dowels at the construction and control joints will also aid in reducing uneven slab movements.

A modulus of subgrade reaction (k) of 100 pounds per square inch/per inch of settlement may be used for preliminary design of exterior flatwork supported on subgrades of engineered fill. The structural engineer should design the slab thickness, reinforcing, and control joint spacing. Special care should be taken to place the reinforcement at mid-height within the slab.

6.11 PAVEMENTS

6.11.1 General

Pavement sections are expected to comprise of flexible (asphalt concrete, AC) pavement sections. Traffic indices (TIs) have not been provided to us for design. Therefore, we have assumed TIs between 5 and 7 for this project. The appropriate TI should be selected by the civil engineer. However, we generally recommend using a TI of at least 5 in parking stalls, 6.0 in automobile drive lanes, and 7 in entrance driveways and areas traversed by heavy trucks such as garbage and delivery trucks on an intermittent basis.

Pavement design recommendations provided below are based on the assumption that subgrades will be similar to the clayey soils encountered in our borings.

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6.11.2 *Asphalt Concrete Pavement Sections*

Based on Caltrans design methods and an estimated R-value of 5 for the subgrade clayey soils, the recommended asphalt concrete pavement sections for Traffic Indices between 5 and 7 are provided below. Pavement section parameters include asphalt concrete (AC) and Caltrans Class 2 aggregate base material (AB).

Table 6-3
Recommended Asphalt Concrete Pavement Sections

Traffic Index	AC (inches)	AB (inches)
5.0	3.0	10.0
6.0	3.5	12.0
7.0	4.0	15.5

If lime treatment of the subgrades is performed to a depth of at least 12 inches, as recommended in Section 6.3, the AB section may be reduced to 4 inches for all of the TIs listed above.

Pavement sections provided above are contingent on the following recommendations being implemented during construction.

- Prior to pavement construction, the subgrades are prepared as recommended in this report. Subgrade preparation should extend at least 2 feet laterally beyond the face of the curb or edge of pavement.
- Subgrade soils are in a stable, "non-pumping" condition at the time the aggregate base materials are placed and compacted.
- Aggregate base materials are compacted to at least 95 percent relative compaction.
- Adequate drainage (both surface and subsurface) is provided such that the subgrade soils and aggregate base materials are not allowed to become saturated.
- Aggregate base materials meet current Caltrans specifications for Class 2 aggregate base.
- Asphalt paving materials and placement methods meet current Caltrans specifications for asphalt concrete.

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- Concrete curbs adjacent to pavement sections should extend to the subgrade soil in order to prevent landscape and infiltrating water from migrating into the aggregate base section.

6.12 CORROSIVITY

Kleinfelder has completed laboratory testing to provide data regarding corrosivity of onsite soils. Our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required. Kleinfelder may be able to provide those services, if needed.

Laboratory chloride concentration, sulfate concentration, pH, oxidation reduction potential, and electrical resistivity tests were performed on near-surface soil samples. The results of the tests are presented in Appendix C and summarized in Table 6-4 below. If fill materials will be imported to the project site, similar corrosion potential laboratory testing should be completed on the imported material.

Table 6-4
Corrosion Potential Test Results

Boring	Depth, feet	Material	Resistivity, ohm-cm		pH	Oxidation Reduction Potential, mV	Water-Soluble Ion Concentration, ppm		
			Saturated	In-Situ Moisture			Chloride	Sulfide	Sulfate
B-2	0 to 2	Clay	750	520	8.17	+260	N.D.	N.D.	67

*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.

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Based on the “10-point” method developed by the American Water Works Association (AWWA) in standard AWWA C105/A21.5, the soils at the site have a high corrosion potential for buried ferrous metal piping, cast iron pipes, or other objects made of these materials. 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. The sample has a slight sulfate concentration (67 ppm = 0.0067%). The results of sulfate test 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 not at risk of corrosion when exposed to water-soluble chloride in the soil or groundwater, give that Chloride was not detected in any of the tests performed. The project structural engineer should still review this data to determine if remedial measures are necessary for the concrete reinforcing steel.

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7 ADDITIONAL SERVICES

The review of pre-final plans and specifications and the field observations and testing during construction by Kleinfelder are an integral part of the conclusions and recommendations made in this report. As the geotechnical engineering firm that performed the geotechnical evaluation for this project, 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 if variations in the soil conditions are encountered. As a minimum Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications;
- Observe and evaluate earthwork operations to confirm subgrade soils are suitable for construction;
- Confirm fill materials are placed and compacted per the project specifications; and
- Observe foundation installation operations and foundation bearing soils to confirm conditions are as anticipated.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. If Kleinfelder is not retained for these services, the client will assume Kleinfelder's responsibility for any potential claims that may arise during or after construction.

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8 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 Prime Data Centers, their consultants and partners for this project, the registered design professional in responsible charge 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 review of the referenced documents, our field observations and subsurface explorations, limited 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.

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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FIGURES

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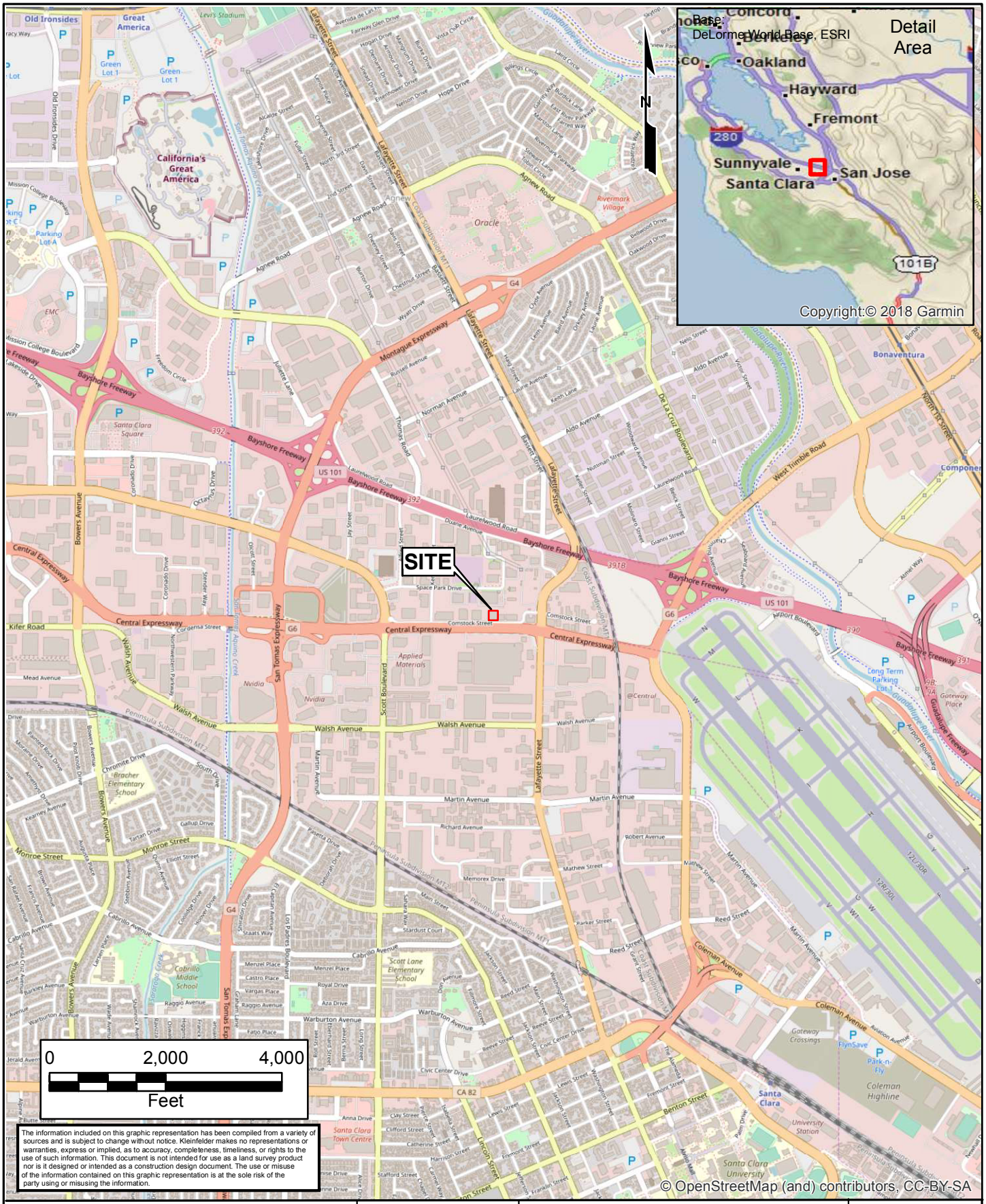

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PROJECT NO.	20193908	SITE VICINITY MAP	FIGURE
DRAWN BY:	JDS		
CHECKED BY:	AA	COMSTOCK DATA CENTER 1111 COMSTOCK STREET SANTA CLARA, CALIFORNIA	1
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REVISED:			

0 60 120

SCALE: 1" = 60' SCALE IN FEET

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BASE MAP IMAGE CREATED FROM DATA
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LEGEND

SITE BOUNDARY

SOIL BORING (Kleinfelder, 2019)

CPT (Kleinfelder, 2019)

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EXPLORATION LOCATION PLAN

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1111 COMSTOCK STREET
SANTA CLARA, CALIFORNIA

FIGURE

2

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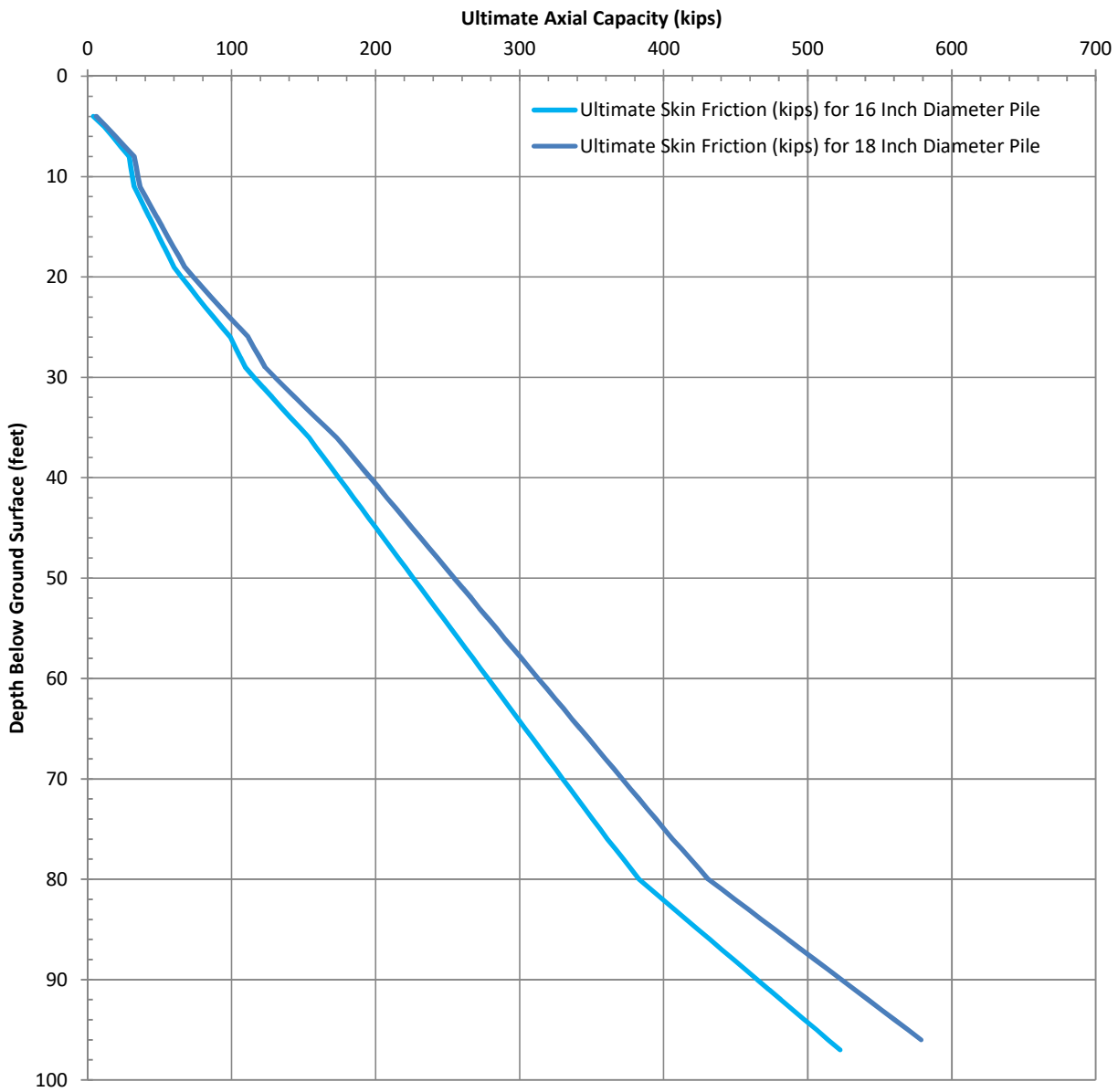
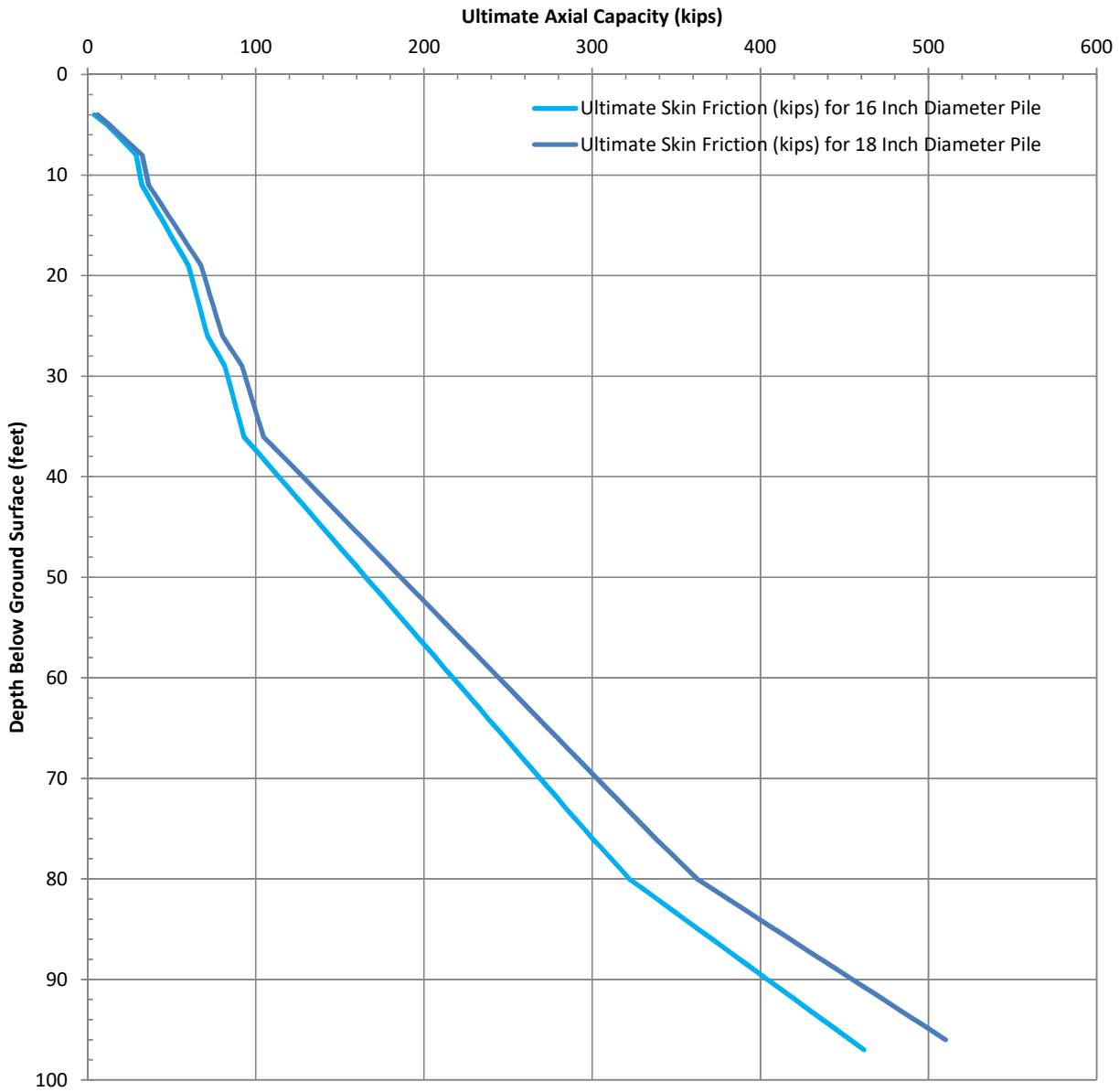


Notes:

1. The curve represents ultimate axial compressive capacity of a single straight-sided drilled pile. See text section 6.9.1 for factor of safety and group effects.
2. We have assumed that the top of pile/bottom of pile cap is 4 feet below the existing grade
3. Ultimate tension capacity may be obtained by multiplying the compressive capacity by a factor of 0.8

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20193908	**ULTIMATE PILE CAPACITY DRILLED PILE STATIC CASE**	FIGURE
	DRAWN BY: HF		3
	CHECKED BY: AA	Comstock Data Center	
	DATE: 4/25/2019	1111 Comstock Street	
	REVISED:	Santa Clara, CA	

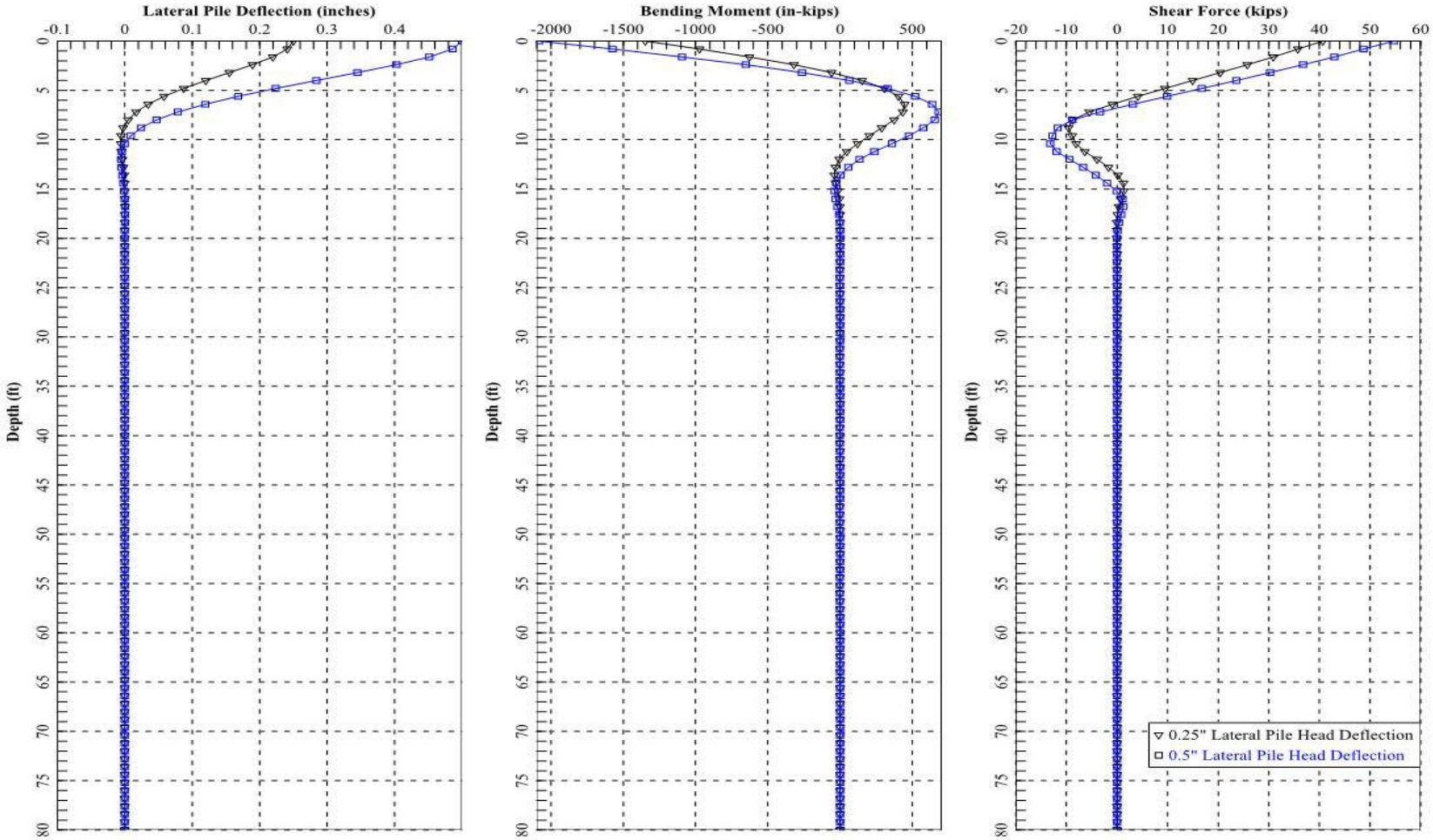


Notes:

1. The curve represents ultimate axial compressive capacity of a single straight-sided drilled pile. See text section 6.9.1 for factor of safety and group effects.
2. We have assumed that the top of pile/bottom of pile cap is 4 feet below the existing grade.
3. Ultimate tension capacity may be obtained by multiplying the compressive capacity by a factor of 0.8.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20193908 DRAWN BY: HF CHECKED BY: AA DATE: 4/25/2019 REVISED:	**ULTIMATE PILE CAPACITY DRILLED PILE POST-LIQUEFACTION CASE** Comstock Data Center 1111 Comstock Street Santa Clara, CA	FIGURE 4

"Depth" is length of pile below bottom of pile cap



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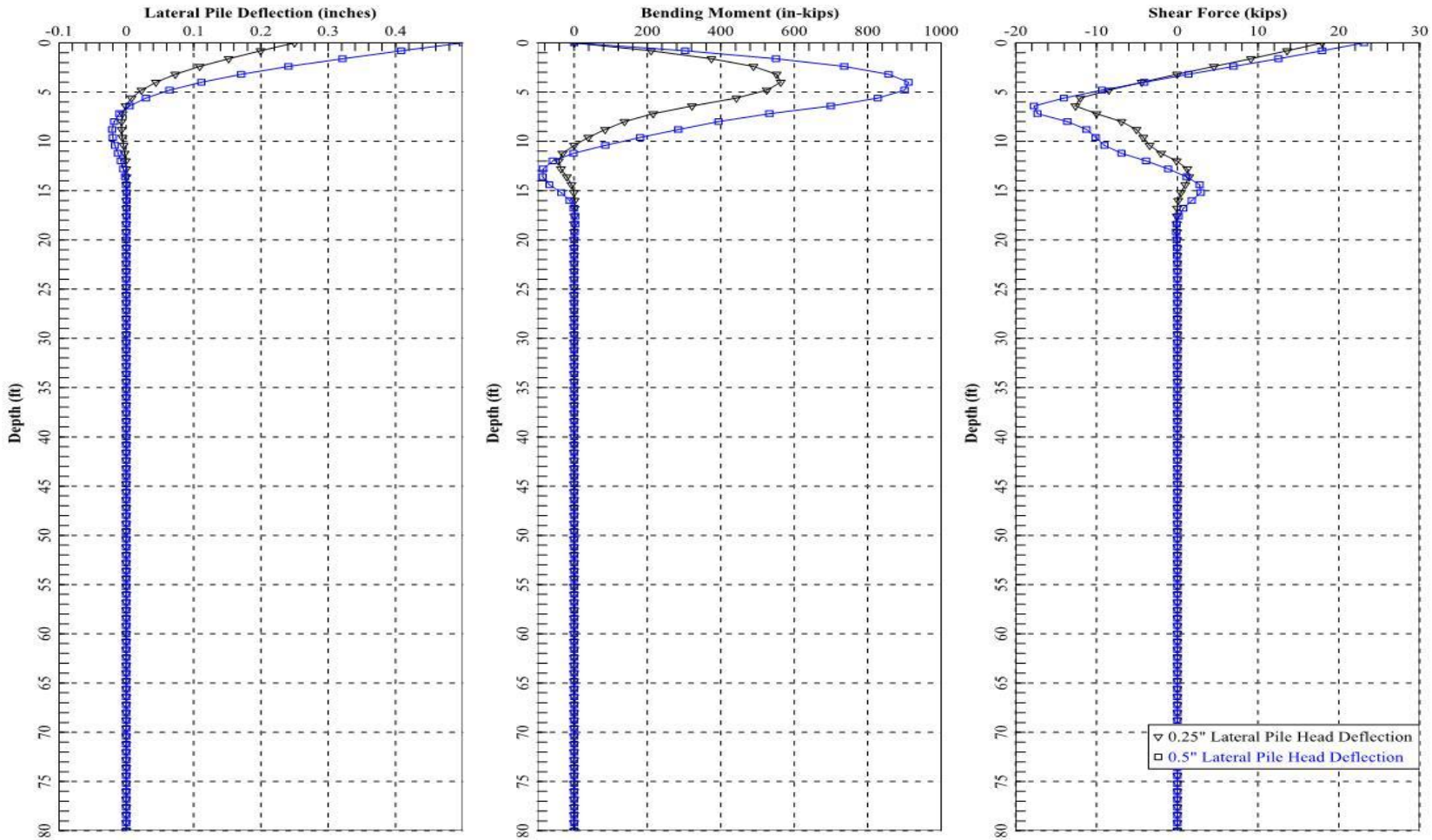
LATERAL PILE RESPONSE
16-INCH DIAMETER PILE
FIXED-HEAD CONDITION

COMSTOCK DATA CENTER
1111 COMSTOCK STREET
SANTA CLARA, CALIFORNIA

FIGURE:

5

"Depth" is length of pile below bottom of pile cap



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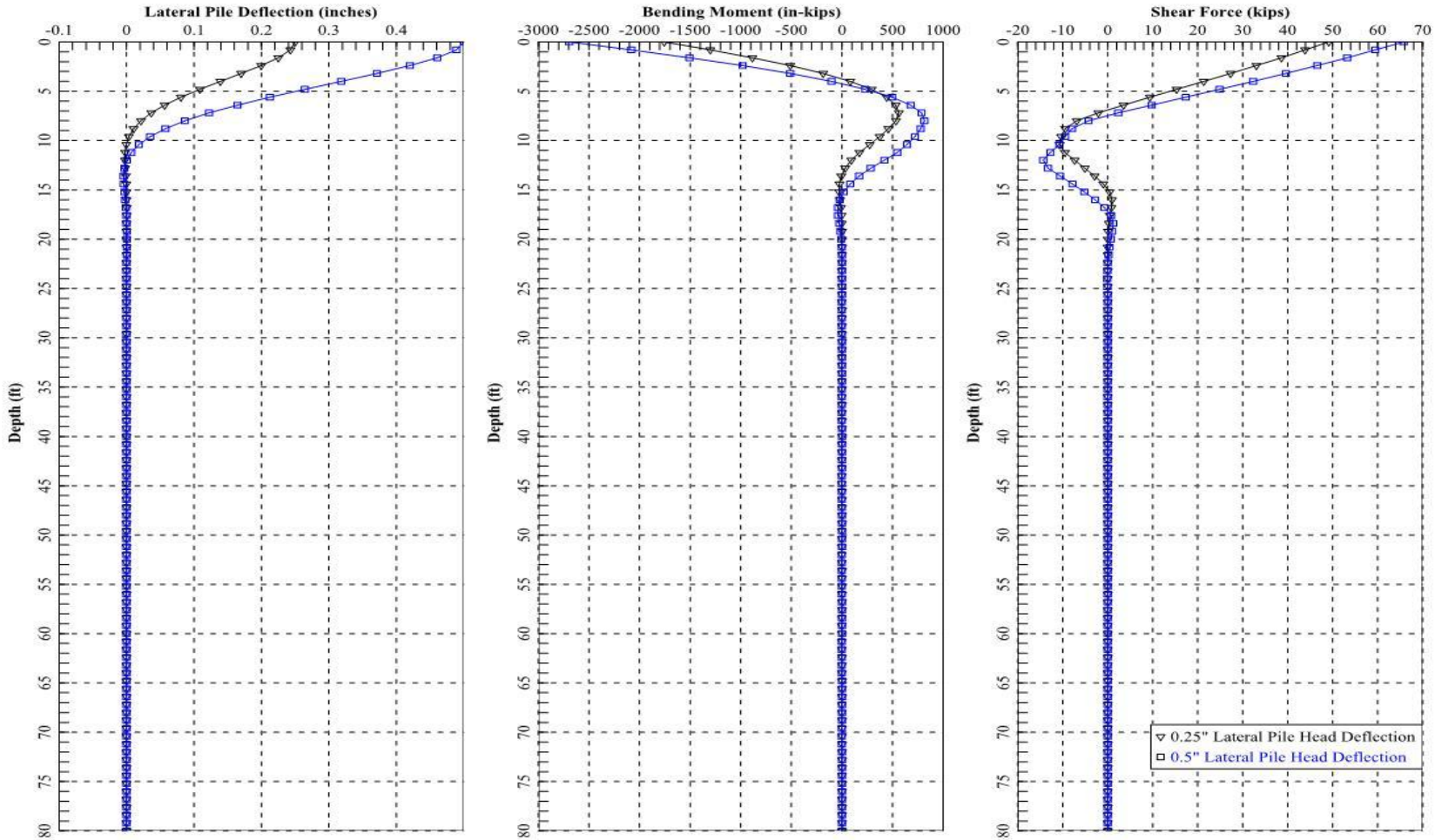
LATERAL PILE RESPONSE
16-INCH DIAMETER PILE
FREE-HEAD CONDITION

COMSTOCK DATA CENTER
1111 COMSTOCK STREET
SANTA CLARA, CALIFORNIA

FIGURE:

6

"Depth" is length of pile below bottom of pile cap



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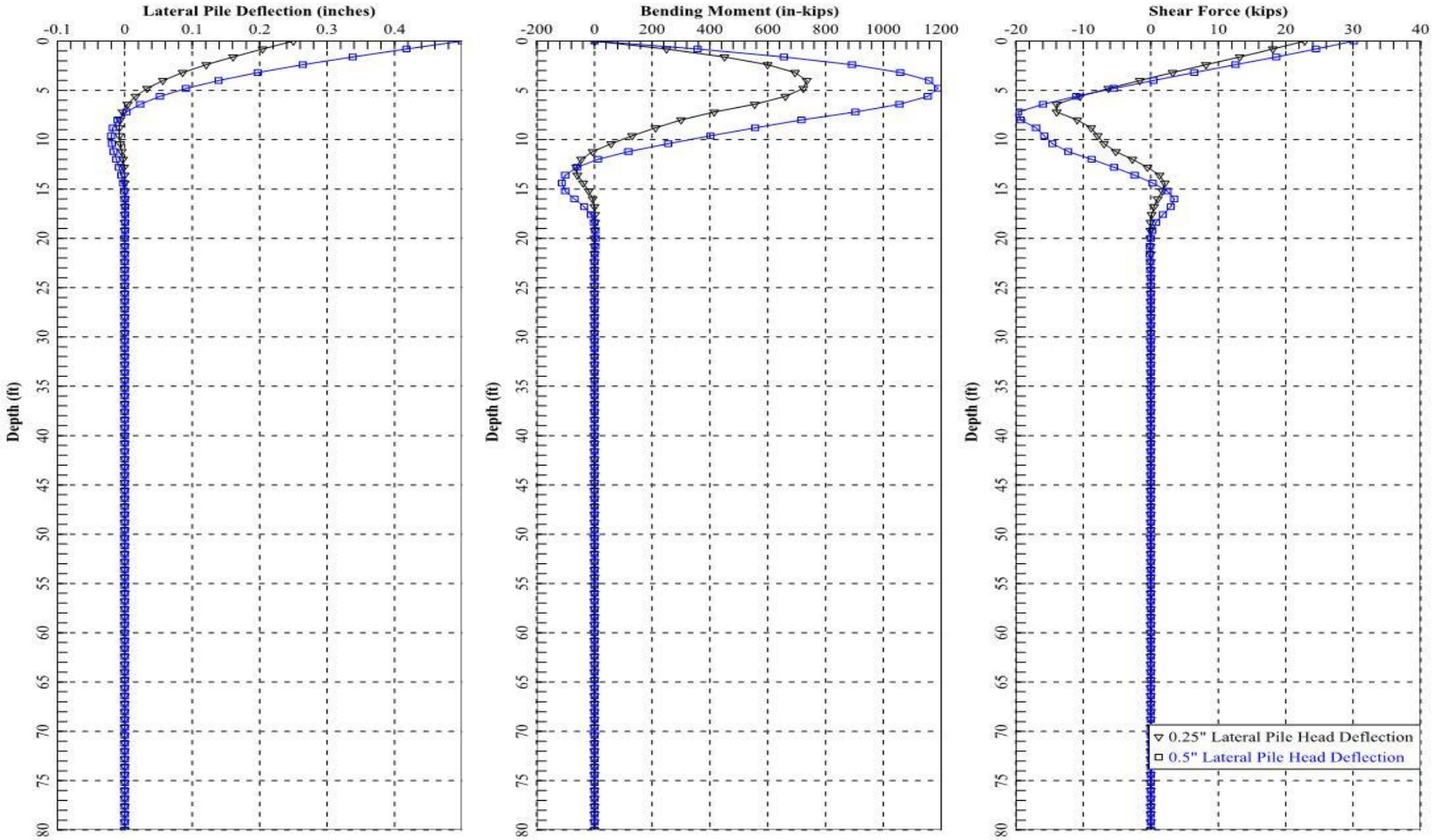
LATERAL PILE RESPONSE
18-INCH DIAMETER PILE
FIXED-HEAD CONDITION

COMSTOCK DATA CENTER
1111 COMSTOCK STREET
SANTA CLARA, CALIFORNIA

FIGURE:

7

"Depth" is length of pile below bottom of pile cap



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LATERAL PILE RESPONSE
18-INCH DIAMETER PILE
FREE-HEAD CONDITION

COMSTOCK DATA CENTER
1111 COMSTOCK STREET
SANTA CLARA, CALIFORNIA

FIGURE:

8

APPENDIX A
CONE PENETRATION REPORT

PRESENTATION OF SITE INVESTIGATION RESULTS

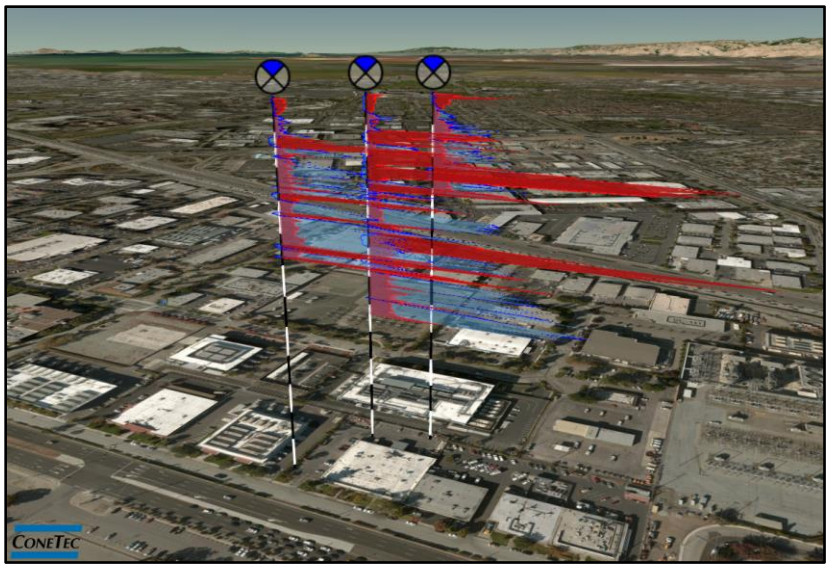
PDC: Comstock Data Center – Santa Clara

Prepared for:

Kleinfelder

ConeTec Inc. Job No: 19-56041

Project Start Date: 26-Mar-2019
Project End Date: 26-Mar-2019
Report Date: 27-Mar-2019



Prepared by:

ConeTec Inc.
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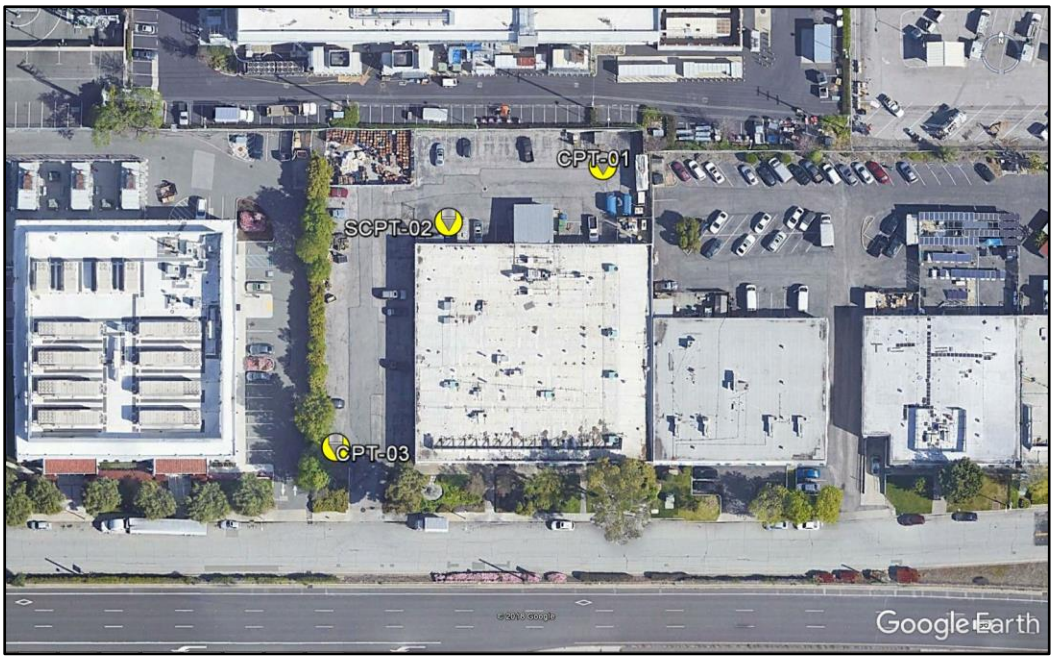
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for Kleinfelder at the Digital Realty Data Center in Santa Clara, CA. The program consisted of two cone penetration tests (CPT) and one seismic cone penetration test (SCPT).

Project Information

Project	
Client	Kleinfelder
Project	PDC: Comstock Data Center - Santa Clara
ConeTec project number	19-56041

An image from Google Earth including the CPT and SCPT test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C17)	30 ton rig cylinder	CPT, SCPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT, SCPT	Consumer grade GPS	32610

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Standard plots with expanded scales, Advanced Plots, Soil Behavior Type (SBT) Scatter plots, and Seismic Shear Wave (Vs) plots were included in the data release package.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
483:T1500F15U500	483	15	225	1500	15	500
Cone 483 was used for all CPT soundings.						

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behavior Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Normalized Soil Behavior Type Chart (SBT Q_{tn}) (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures – clayey silt to silty clay (zone 4).

Limitations

This report has been prepared for the exclusive use of Kleinfelder (Client) for the project titled "PDC: Comstock Data Center - Santa Clara". The report's contents may not be relied upon by any other party without the express written permission of ConeTec Inc. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meet or exceed those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

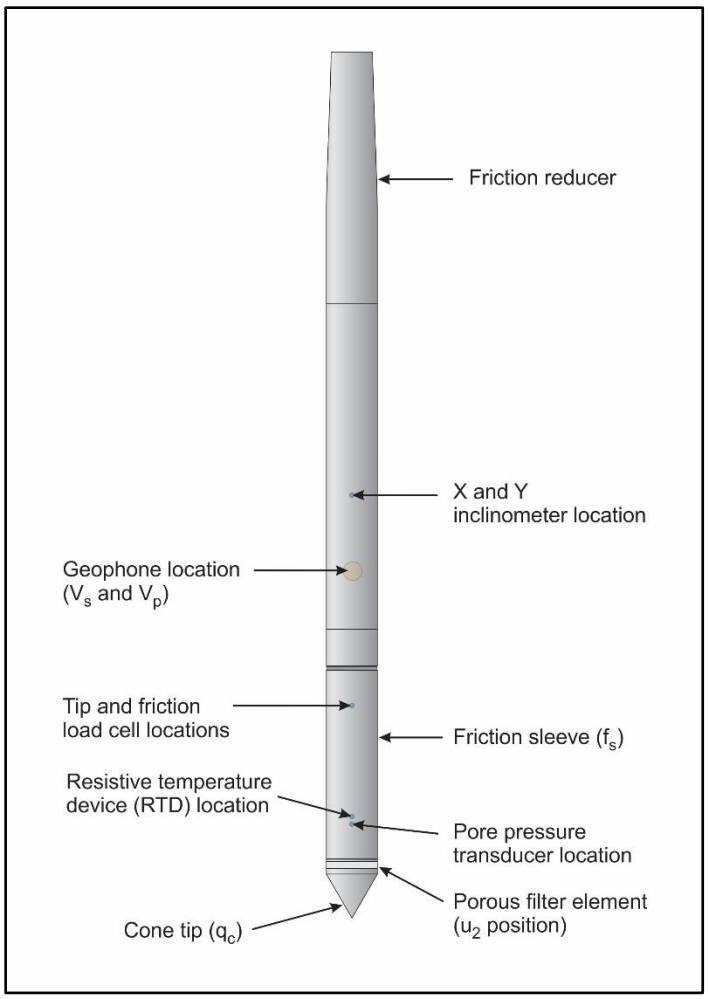


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods.

The typical recording interval is 2.5 cm; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerin or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behavior based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1-a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

Shear wave velocity (Vs) testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave velocity (Vp) testing is also performed.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

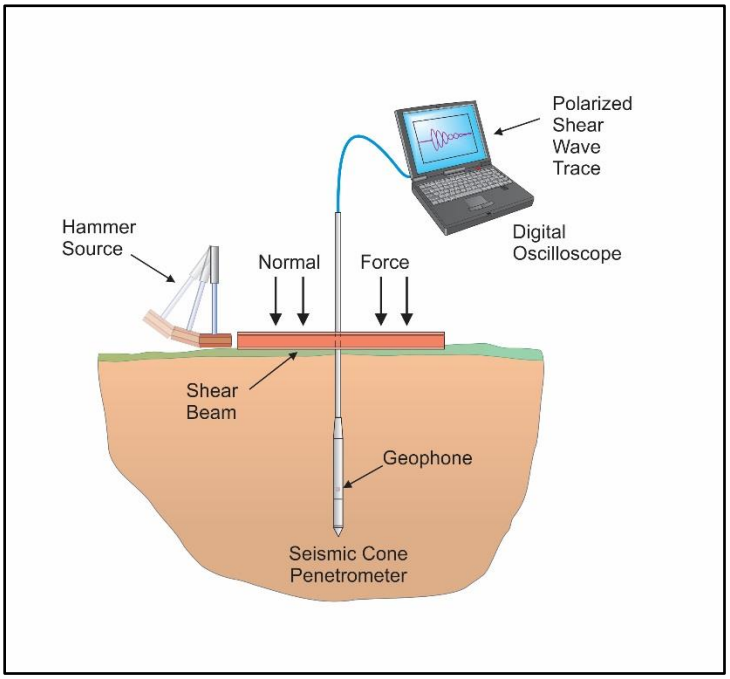


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM 5778 and ASTM D7400 standards.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Typically, five wave traces for each orientation are recorded for quality control purposes and uncertainty analysis. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et. al. (1986).

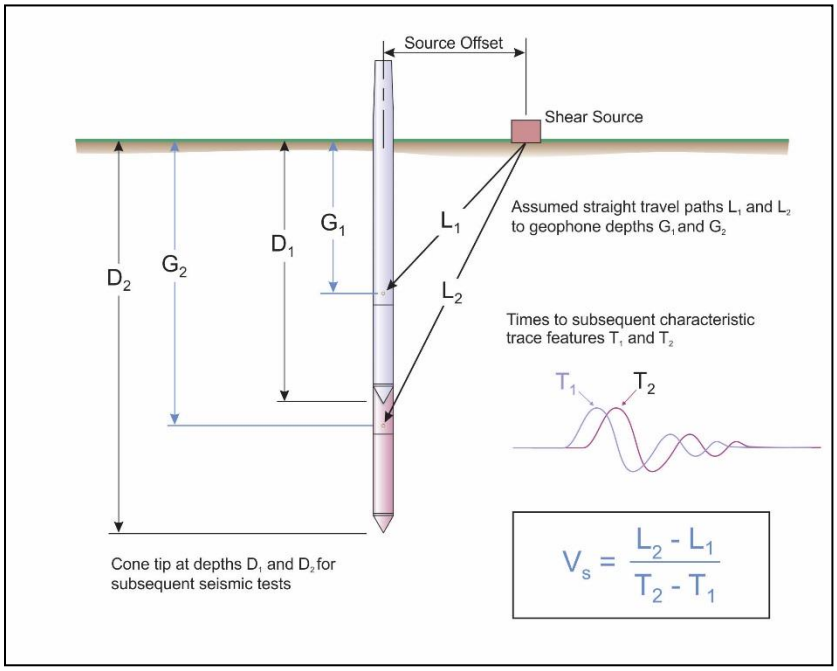


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 100 feet (30 meters) ($\bar{v}_s$) has been calculated and provided for all applicable soundings using the following equation presented in ASCE (2010).

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)
d_i = the thickness of any layer between 0 and 100 ft (30 m)
v_{si} = the shear wave velocity in ft/s (m/s)
$\sum_{i=1}^{n} d_i$ = 100 ft (30 m)

Average shear wave velocity, $\bar{v}_s$ is also referenced to V_{s100} or V_{s30}.

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

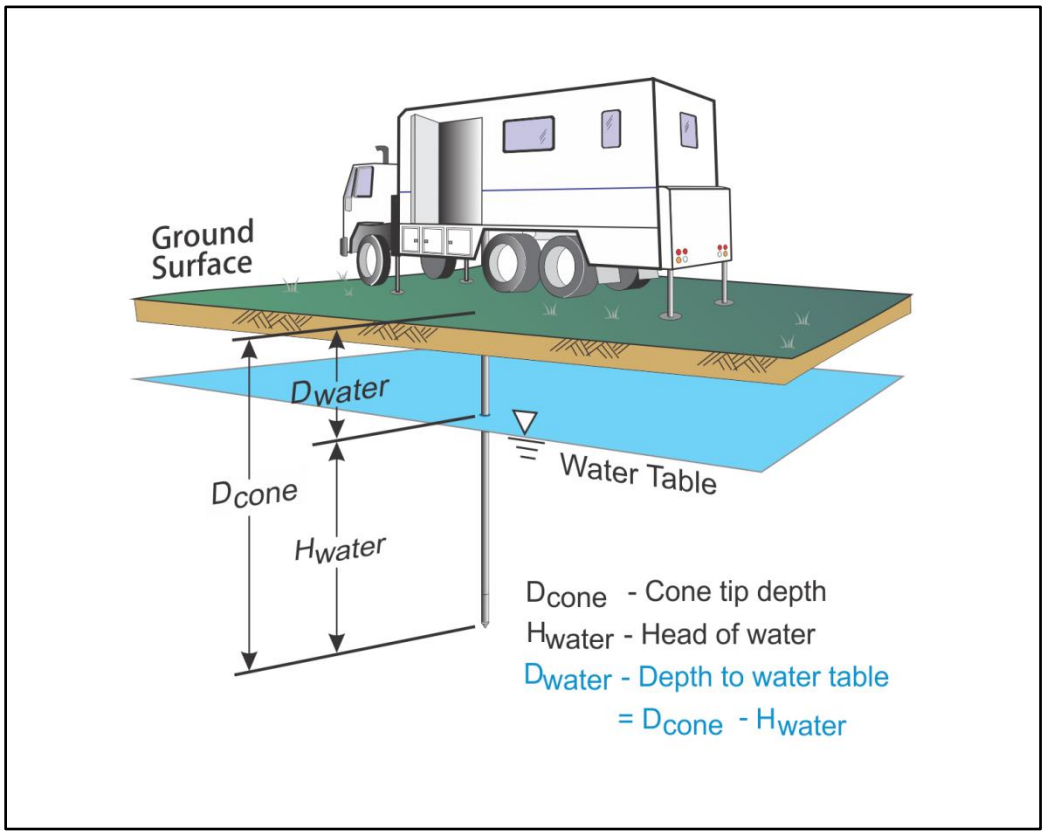


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

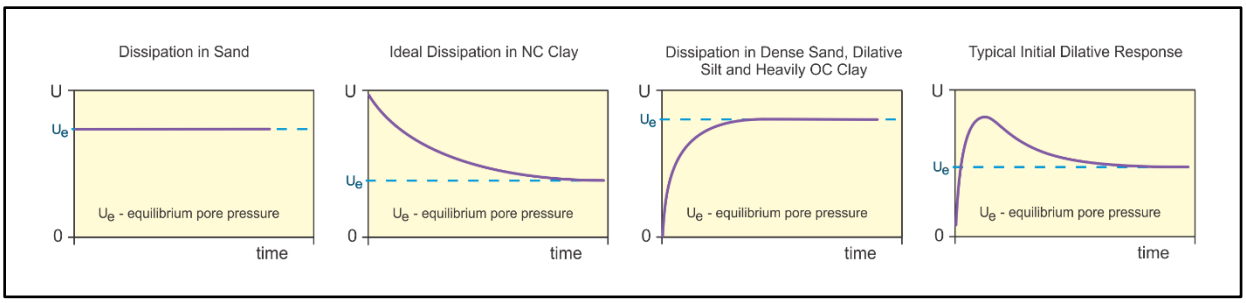


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T* is the dimensionless time factor (Table Time Factor)
a is the radius of the cone
I_r is the rigidity index
t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby (1991))

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby (1991)), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

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Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158.

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Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, “Seismic CPT to Measure In-Situ Shear Wave Velocity”, Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Standard Cone Penetration Test Plots with Expanded Scales
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi and N1(60)Ic
- Soil Behavior Type (SBT) Scatter Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Shear Wave (Vs) Traces
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and
Standard Cone Penetration Test Plots

CONETEC

Job No:	19-56041
Client:	Kleinfelder
Project:	PDC: Comstock Data Center - Santa Clara
Start Date:	26-Mar-2019
End Date:	26-Mar-2019

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting (m)	Refer to Notation Number
CPT-01	19-56041_CP01	26-Mar-2019	483:T1500F15U500	8.5	51.76	4137025	592701	
SCPT-02	19-56041_SP02	26-Mar-2019	483:T1500F15U500	7.1	120.57	4137013	592669	
CPT-03	19-56041_CP03	26-Mar-2019	483:T1500F15U500	7.1	77.92	4136966	592646	3

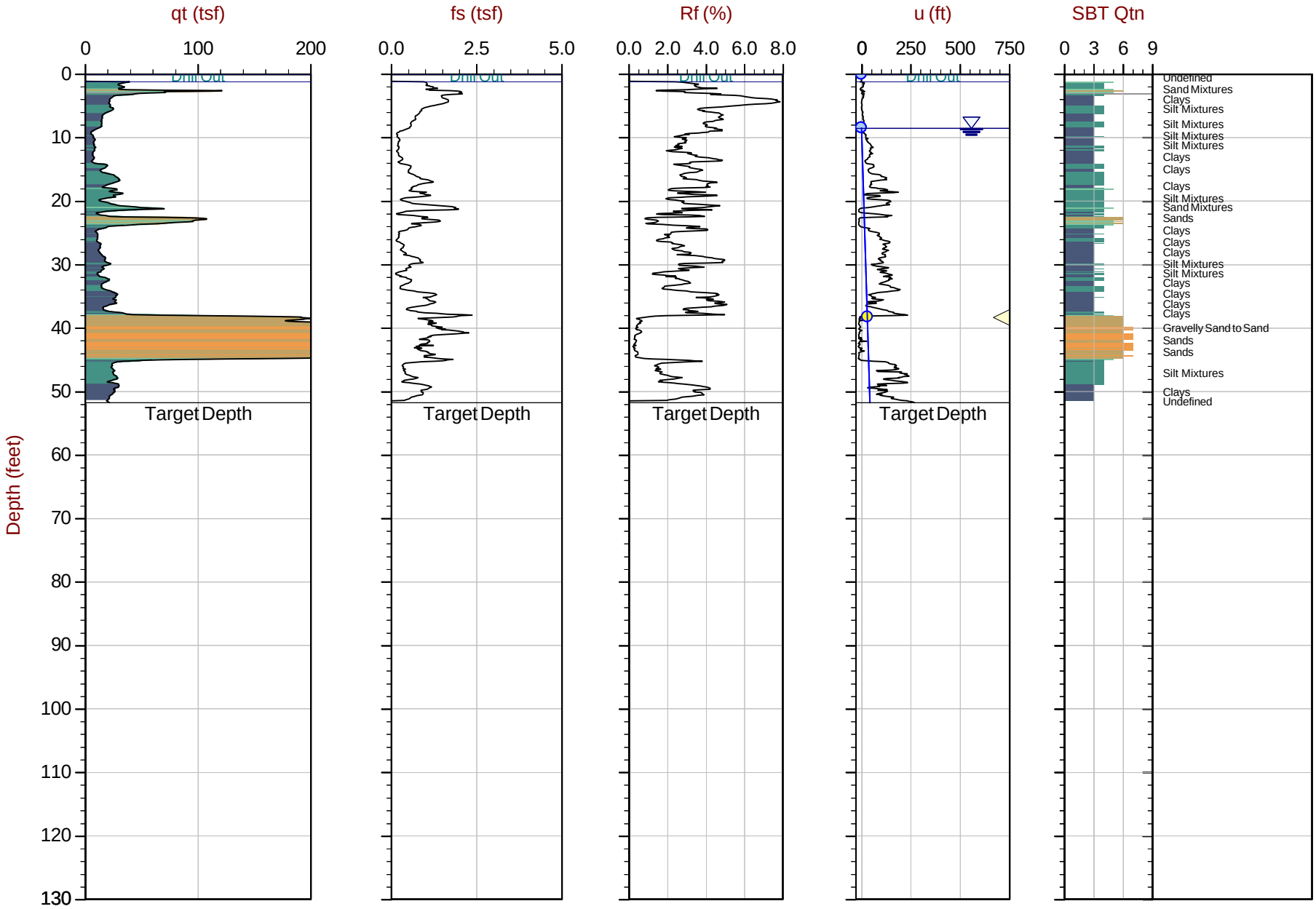
1. The assumed phreatic surface was based on pore pressure dissipation tests, unless otherwised noted. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were acquired with consumer grade GPS equipment in datum: WGS84 / UTM Zone 10 North.
3. The assumed phreatic surface is based on an adjacent CPT.

CONETEC

Kleinfelder

Job No: 19-56041
Date: 2019-03-26 09:45
Site: PDC: Comstock Data Center - Santa Clara

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 15.775 m / 51.75 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56041_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137025m E: 592701m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

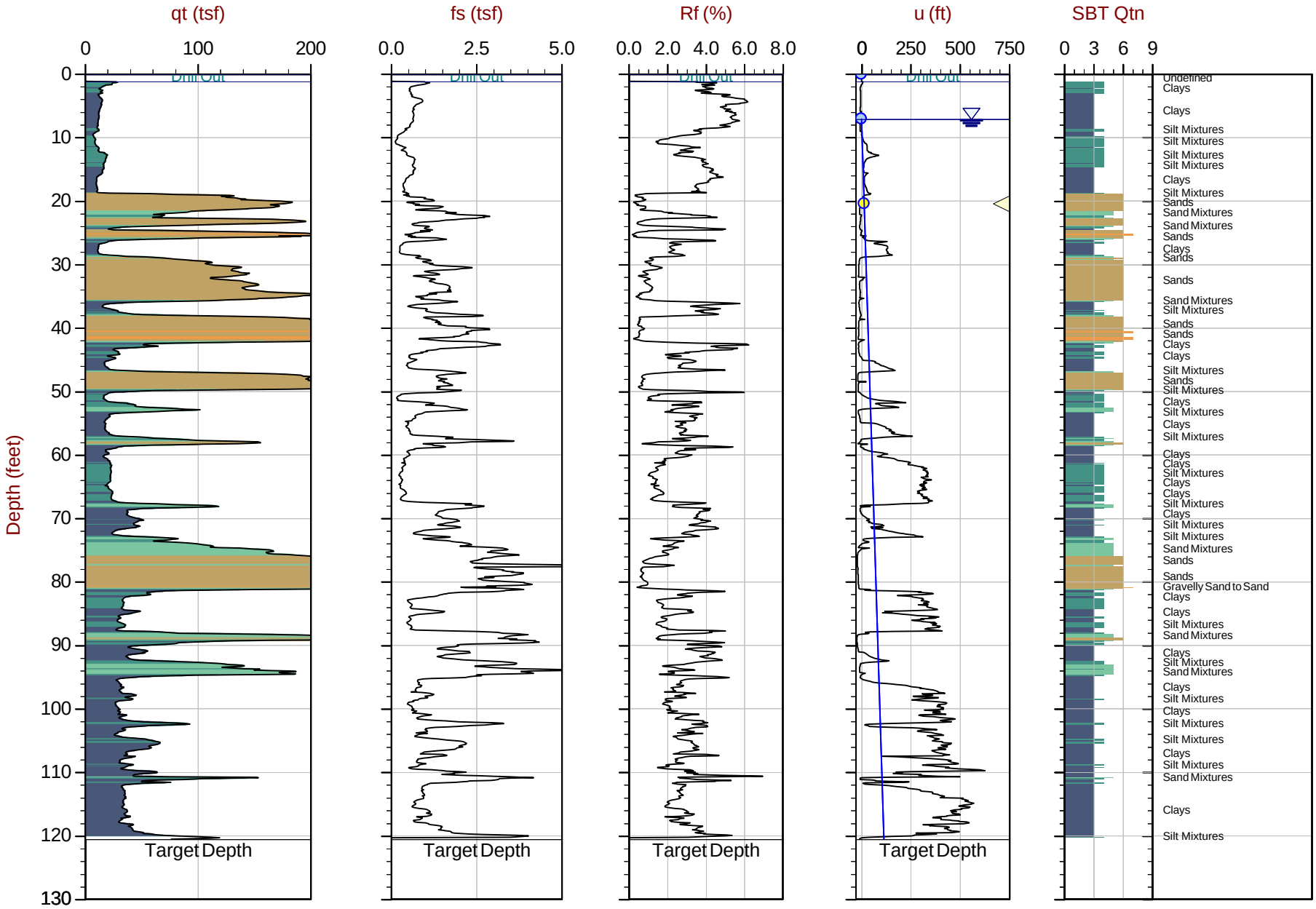
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 19-56041
Date: 2019-03-26 07:27
Site: PDC: Comstock Data Center - Santa Clara

Sounding: SCPT-02
Cone: 483:T1500F15U500



Max Depth: 36.750 m / 120.57 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56041_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137013m E: 592669m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

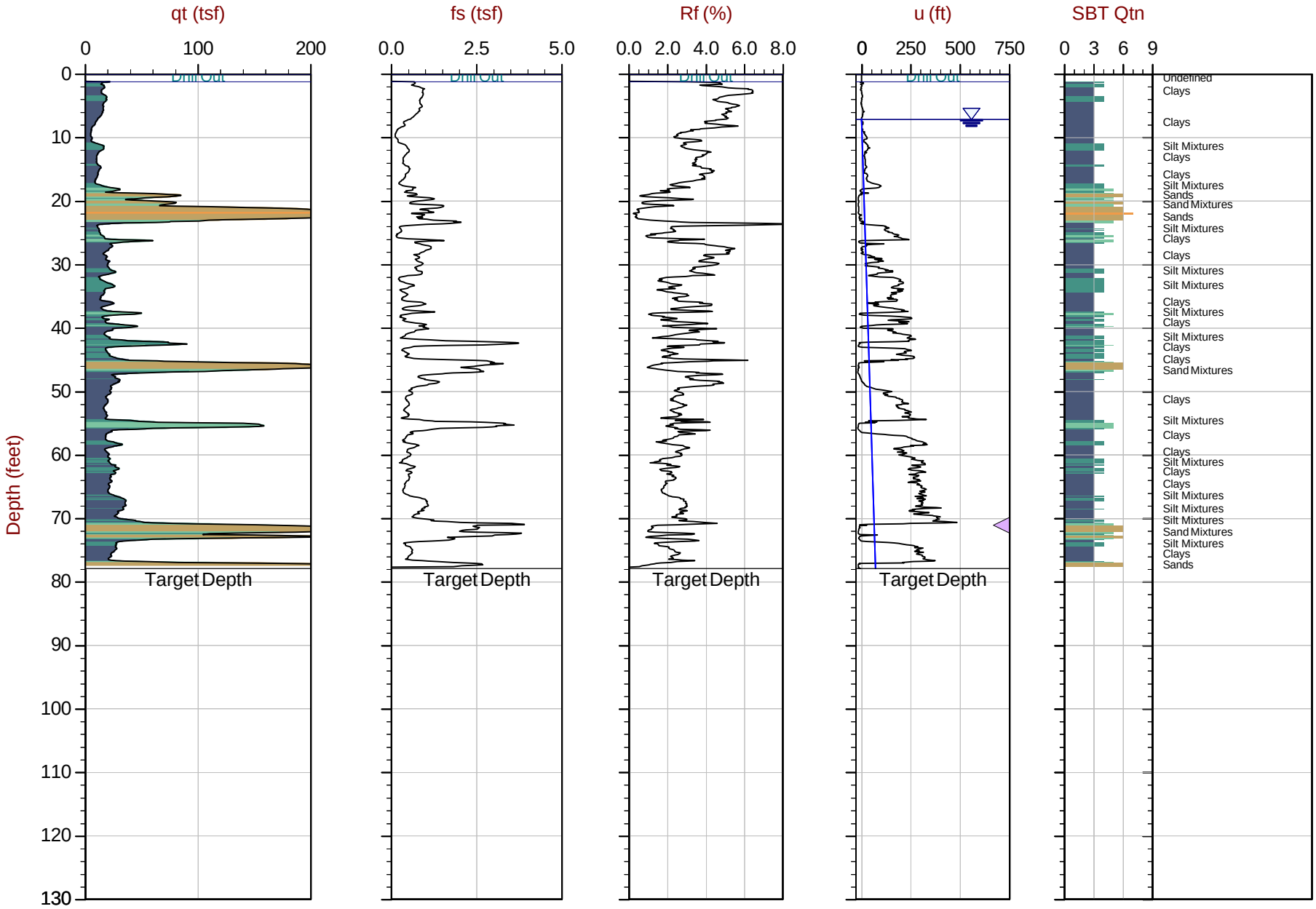
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 19-56041
Date: 2019-03-26 10:42
Site: PDC: Comstock Data Center - Santa Clara

Sounding: CPT-03
Cone: 483:T1500F15U500



Max Depth: 23.750 m / 77.92 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56041_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4136966m E: 592646m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Standard Cone Penetration Test Plots with Expanded Scales

CONETEC

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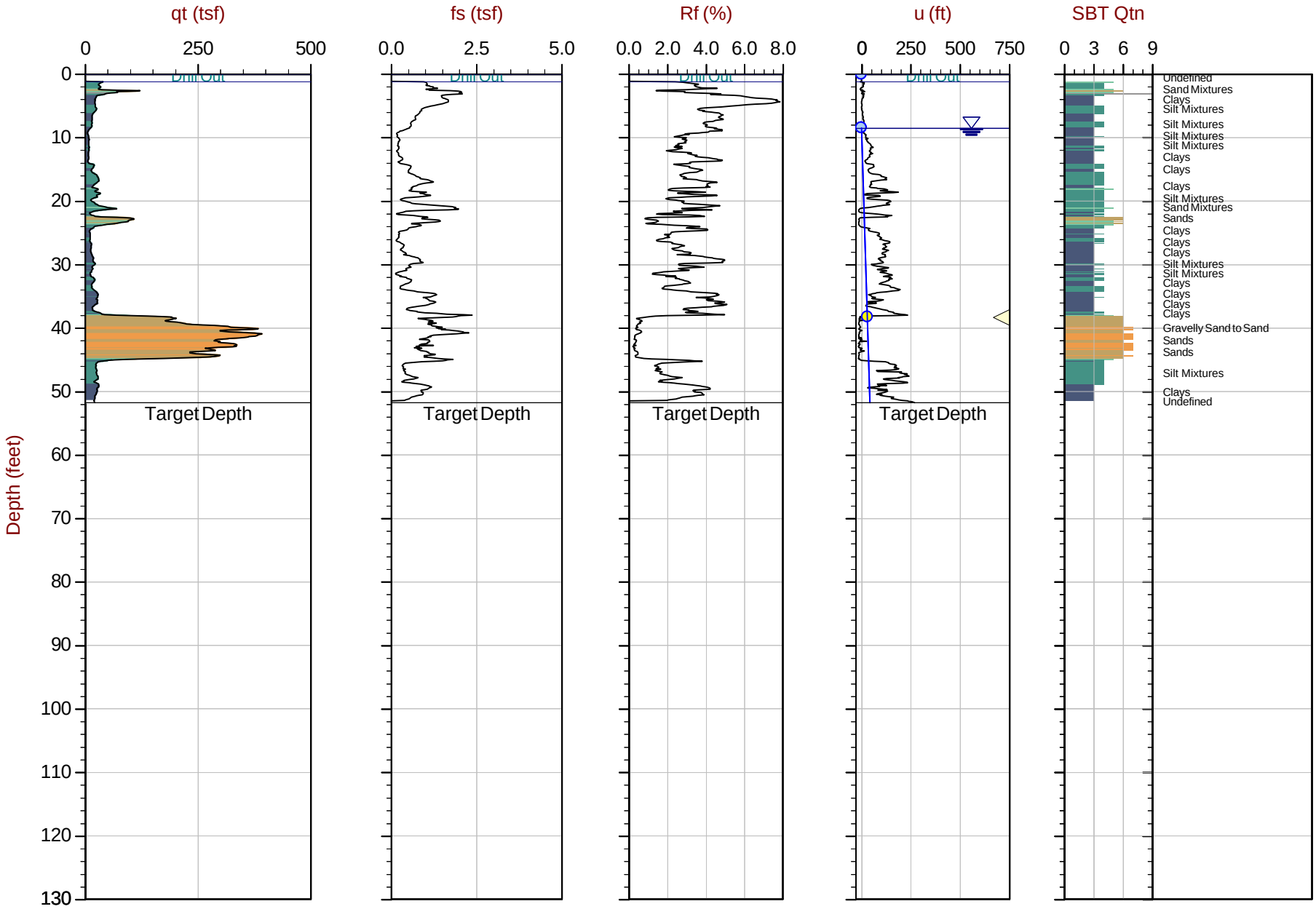
Kleinfelder

Job No: 19-56041
Date: 2019-03-26 09:45
Site: PDC: Comstock Data Center - Santa Clara

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 15.775 m / 51.75 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56041_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137025m E: 592701m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

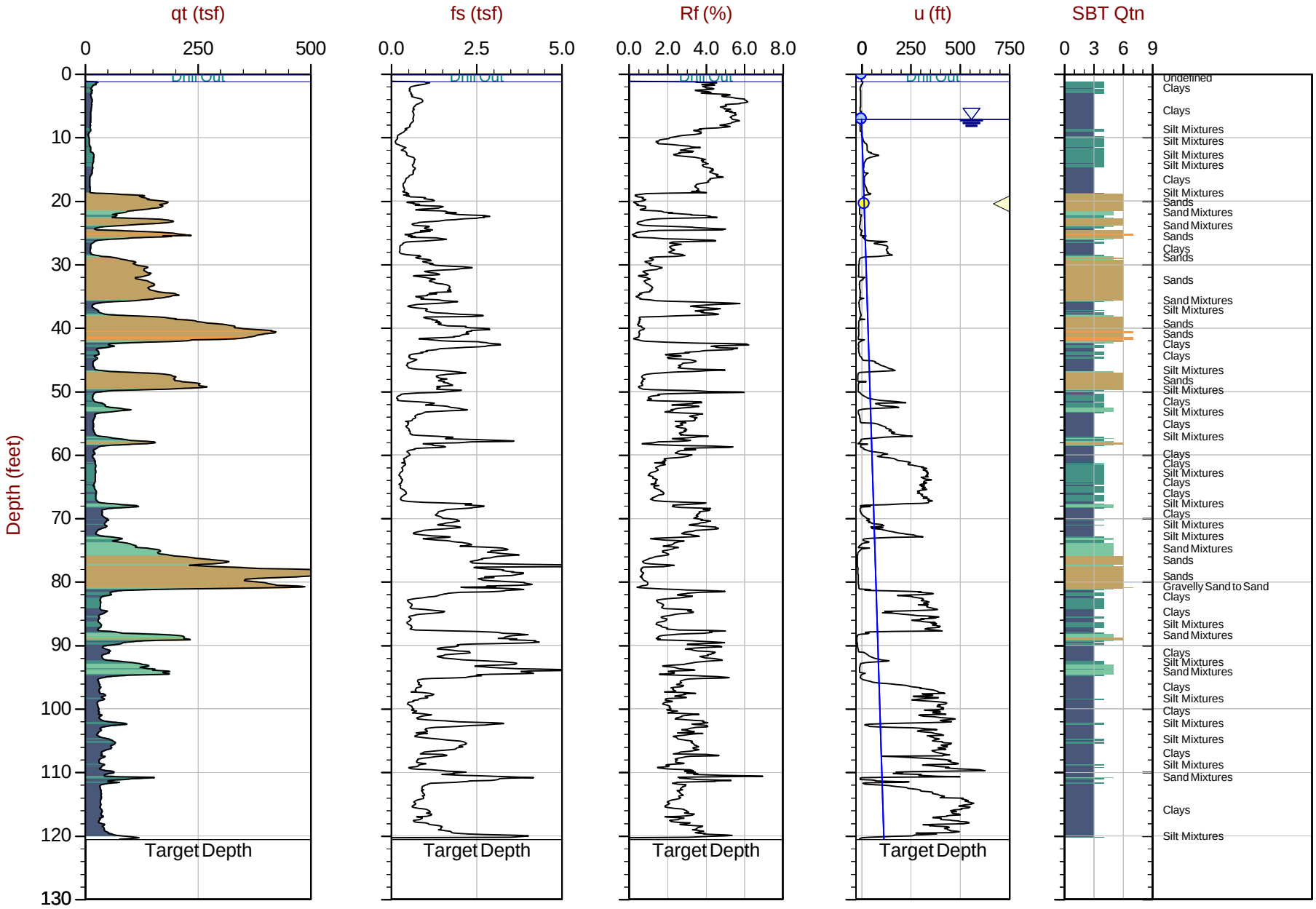
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 19-56041
Date: 2019-03-26 07:27
Site: PDC: Comstock Data Center - Santa Clara

Sounding: SCPT-02
Cone: 483:T1500F15U500



Max Depth: 36.750 m / 120.57 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56041_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137013m E: 592669m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

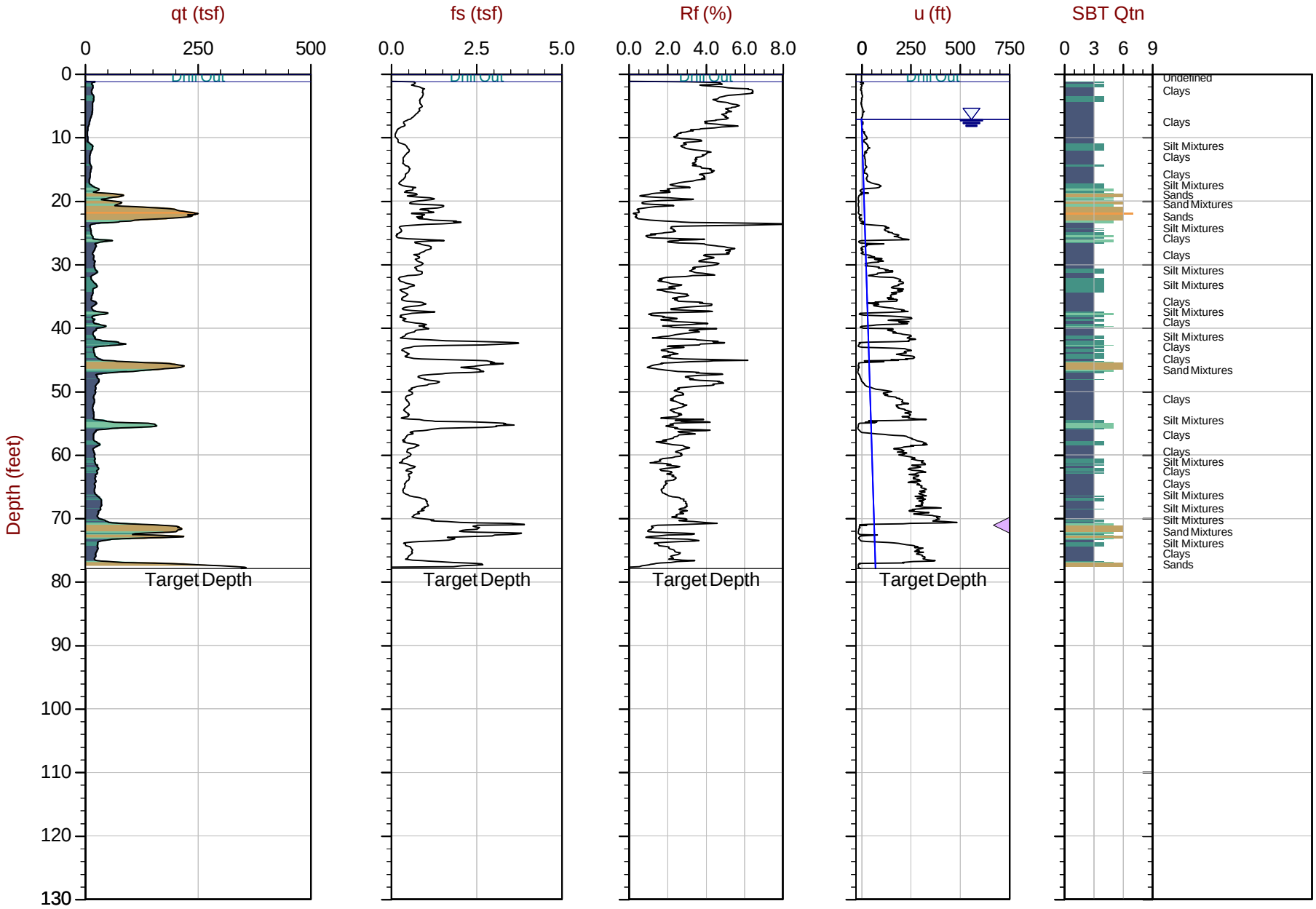
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 19-56041
Date: 2019-03-26 10:42
Site: PDC: Comstock Data Center - Santa Clara

Sounding: CPT-03
Cone: 483:T1500F15U500



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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56041_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4136966m E: 592646m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi and N1(60)Ic

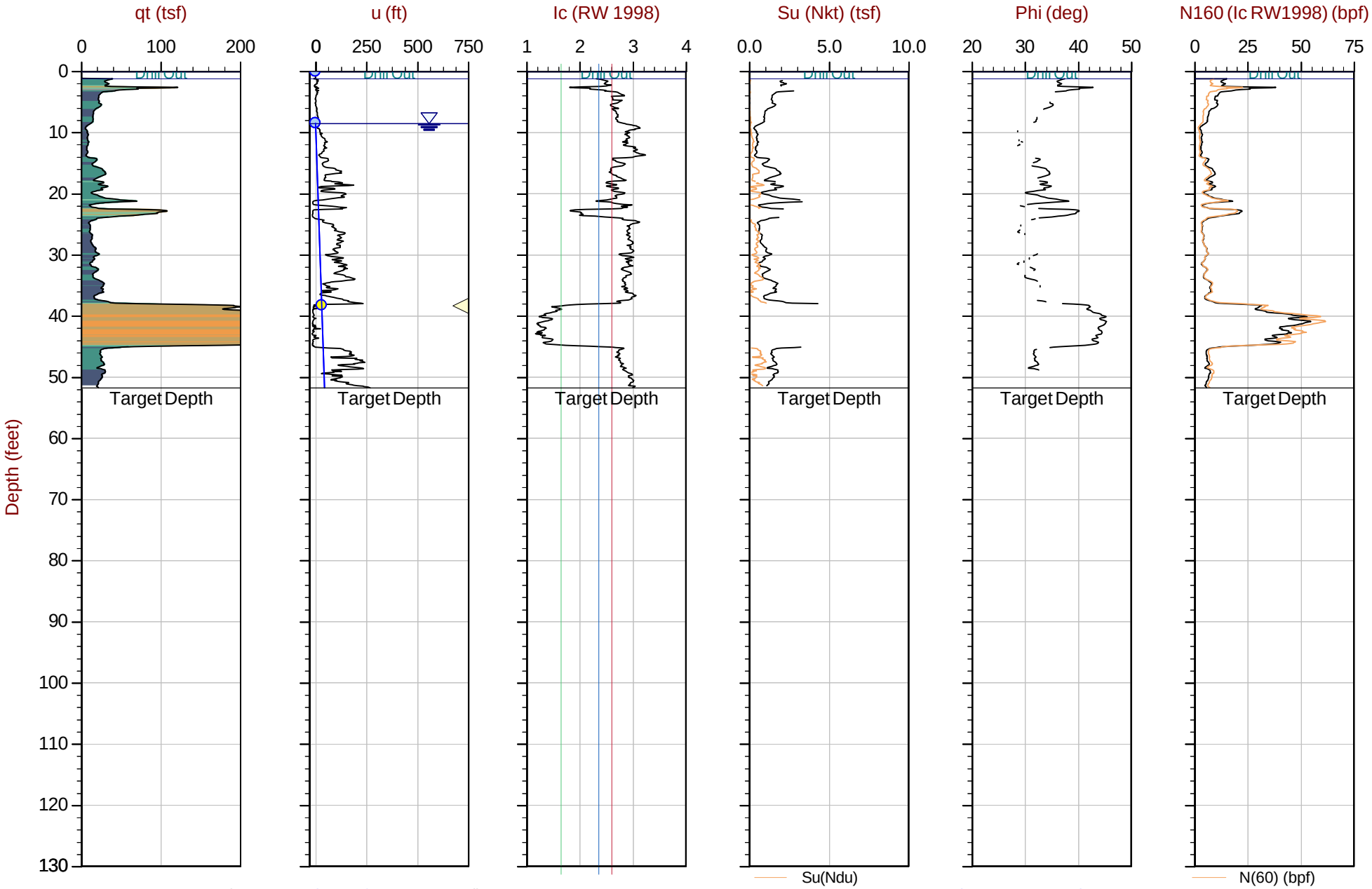
CONETEC

CONETEC

Kleinfelder

Job No: 19-56041
Date: 2019-03-26 09:45
Site: PDC: Comstock Data Center - Santa Clara

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 15.775 m / 51.75 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56041_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137025m E: 592701m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

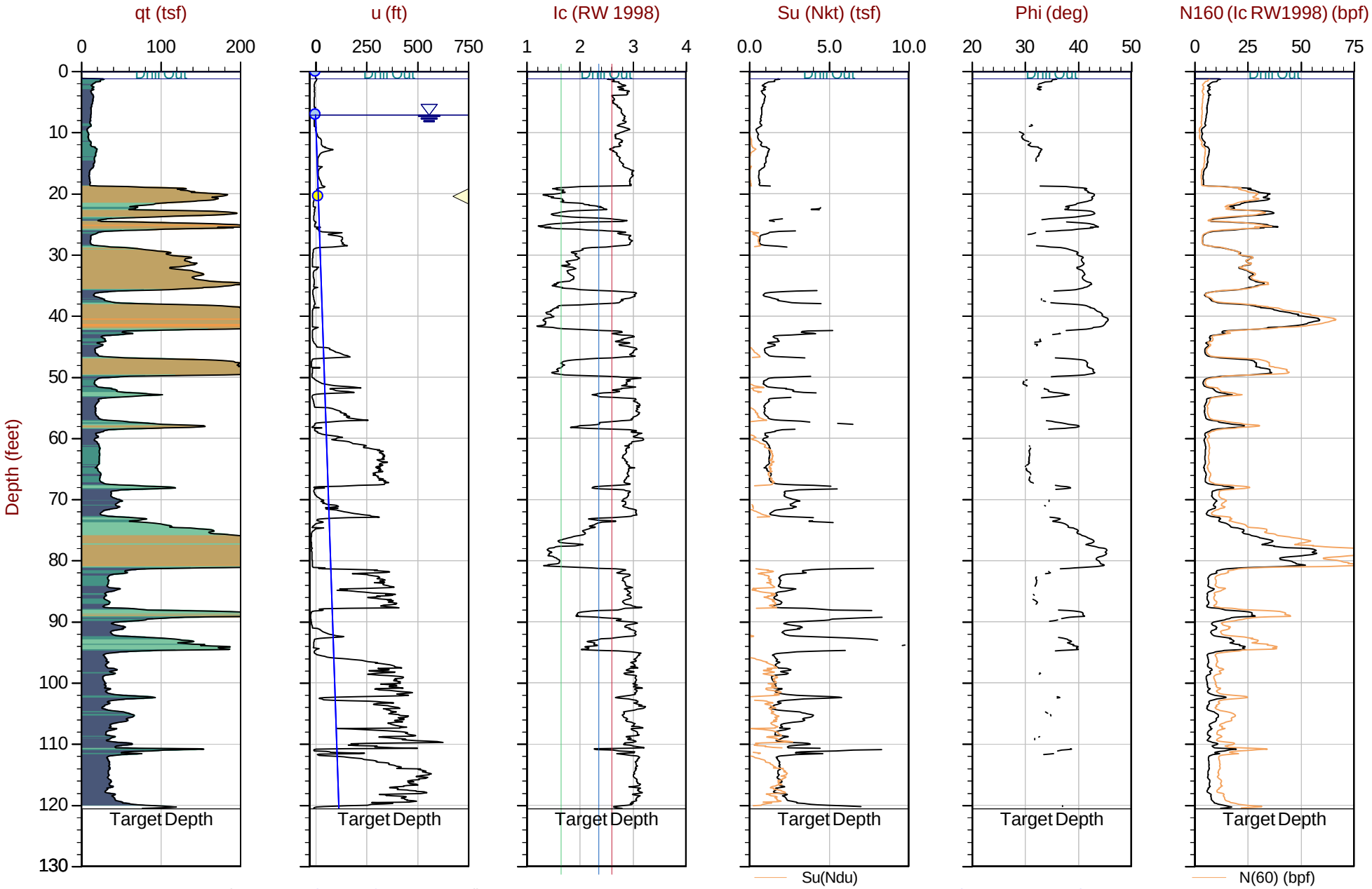
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 19-56041
Date: 2019-03-26 07:27
Site: PDC: Comstock Data Center - Santa Clara

Sounding: SCPT-02
Cone: 483:T1500F15U500



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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56041_SP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137013m E: 592669m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

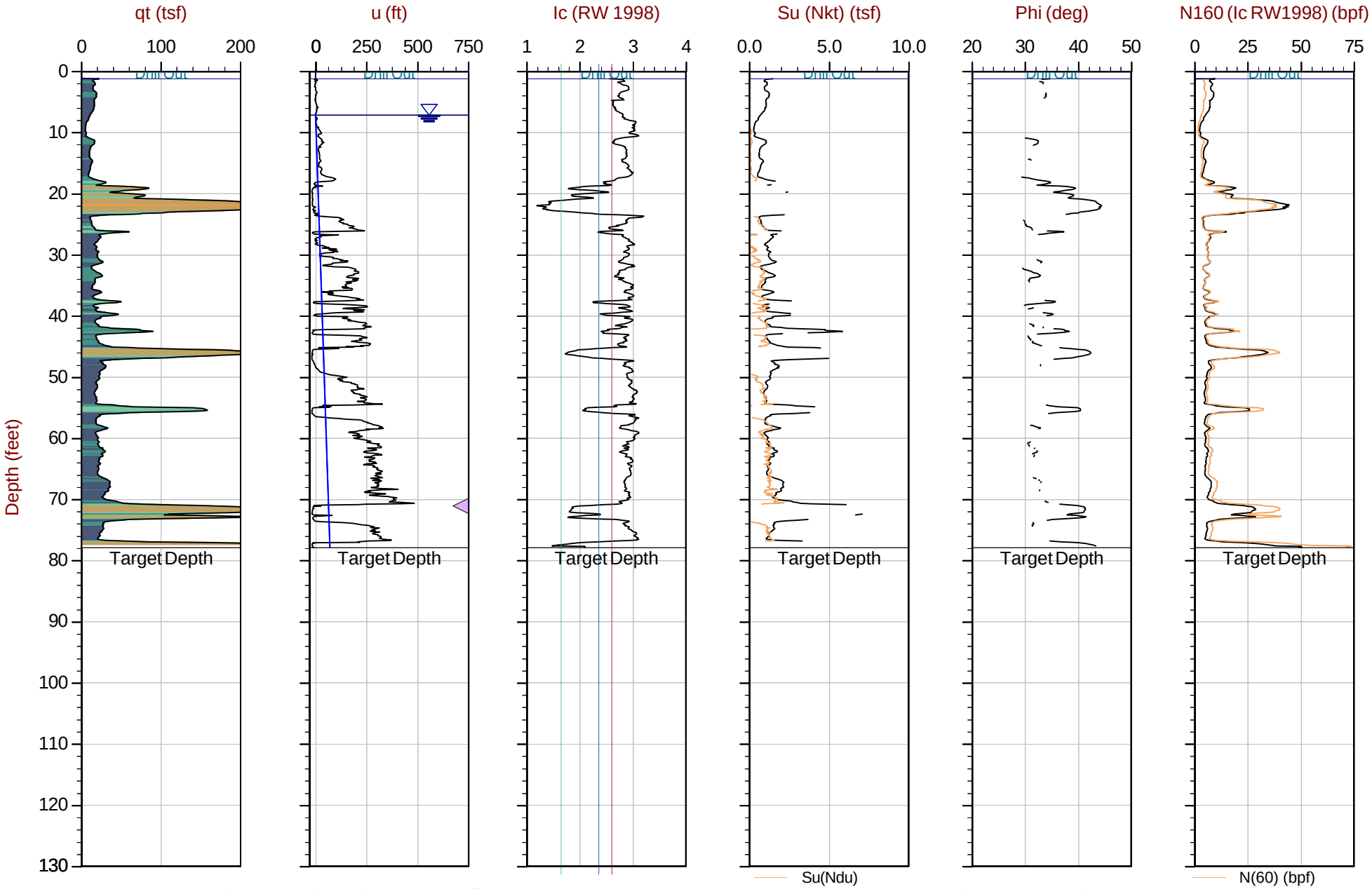
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 19-56041
Date: 2019-03-26 10:42
Site: PDC: Comstock Data Center - Santa Clara

Sounding: CPT-03
Cone: 483:T1500F15U500



Max Depth: 23.750 m / 77.92 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56041_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4136966m E: 592646m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Soil Behavior Type (SBT) Scatter Plots

CONETEC

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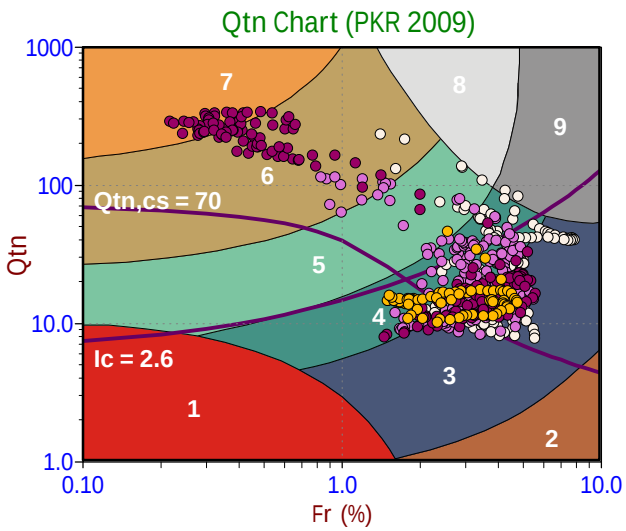
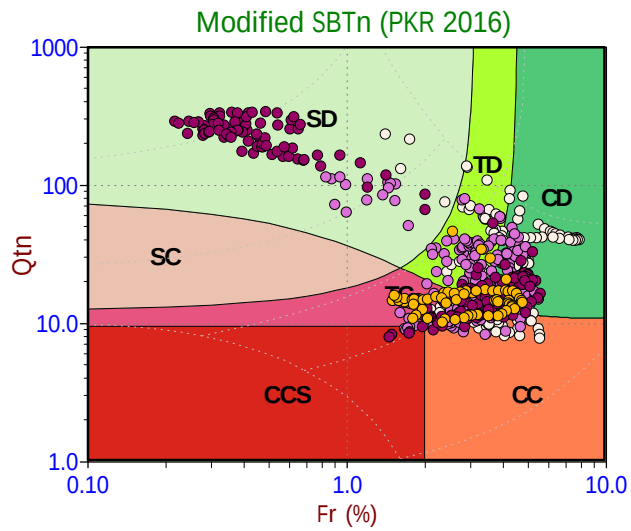
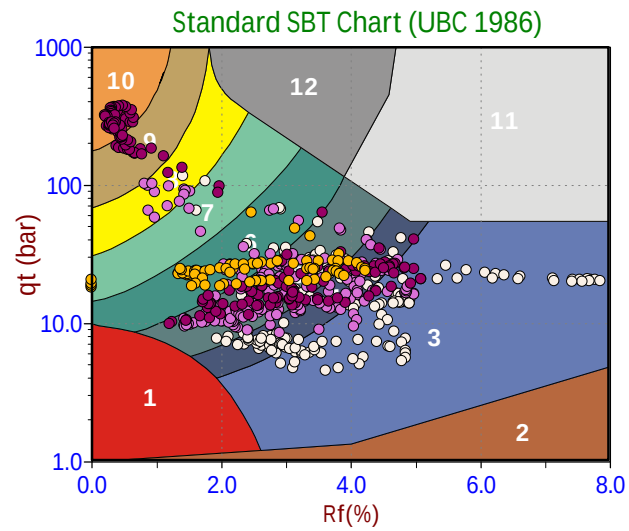
Kleinfelder

Job No: 19-56041
Date: 2019-03-26 09:45
Site: PDC: Comstock Data Center - Santa Clara

Sounding: CPT-01
Cone: 483:T1500F15U500







Depth Ranges
- >0.0 to 15.0 ft
- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
- >75.0 to 90.0 ft
- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

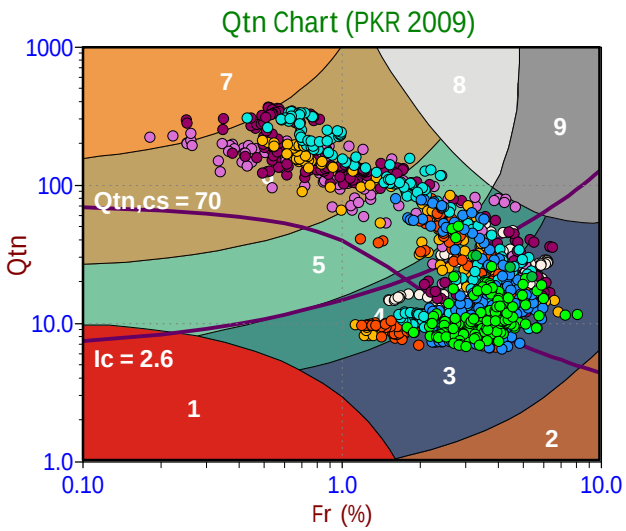
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

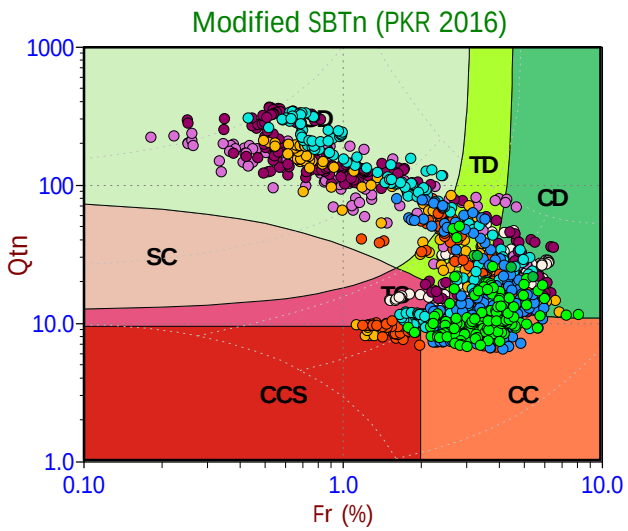
CONETEC

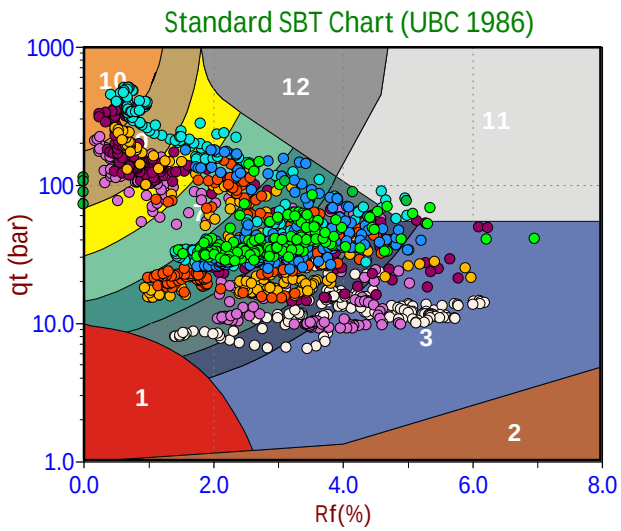
Kleinfelder

Job No: 19-56041
Date: 2019-03-26 07:27
Site: PDC: Comstock Data Center - Santa Clara

Sounding: SCPT-02
Cone: 483:T1500F15U500







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- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
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- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

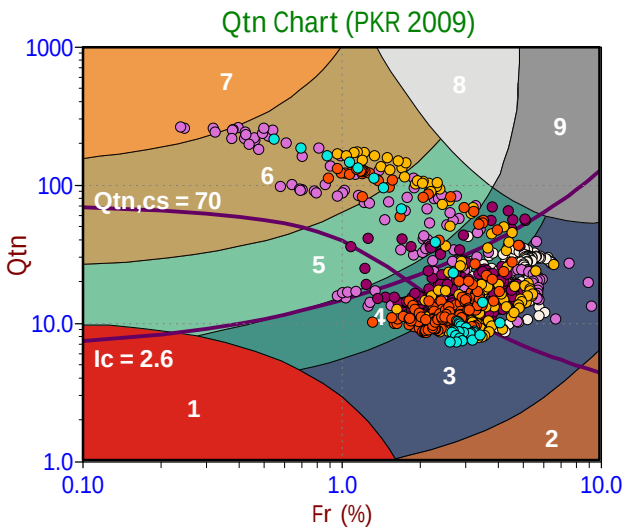
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

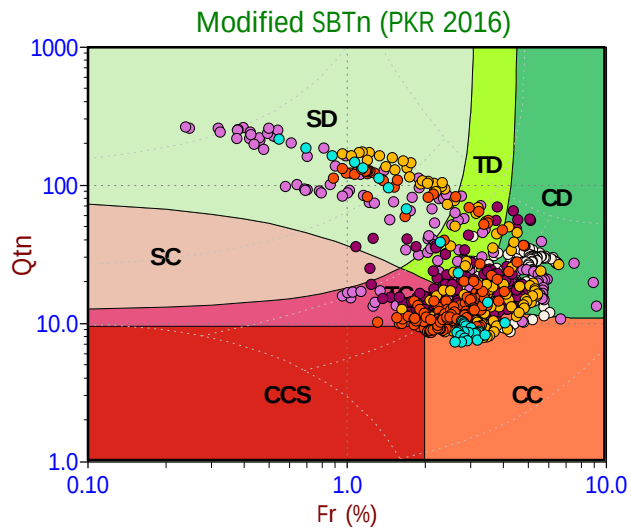
CONETEC

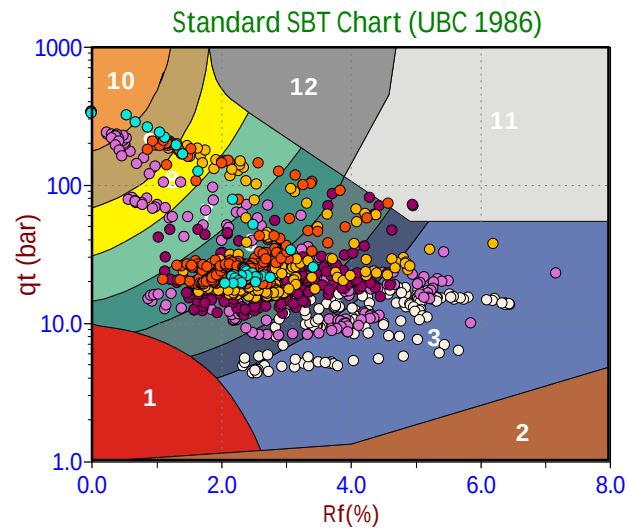
Kleinfelder

Job No: 19-56041
Date: 2019-03-26 10:42
Site: PDC: Comstock Data Center - Santa Clara

Sounding: CPT-03
Cone: 483:T1500F15U500







Depth Ranges
- >0.0 to 15.0 ft
- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
- >75.0 to 90.0 ft
- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Seismic Cone Penetration Test Plots

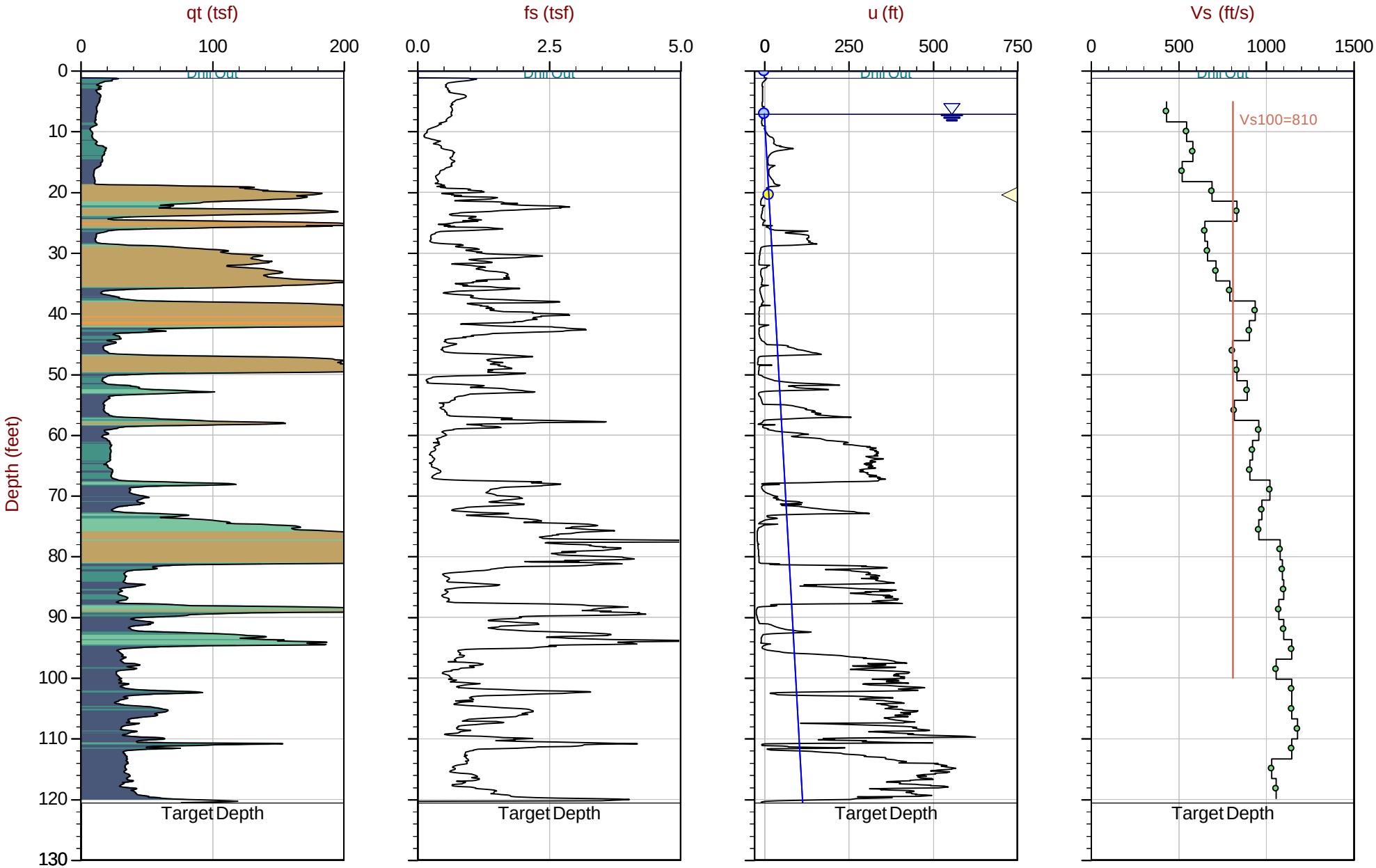
CONETEC

CONETEC

Kleinfelder

Job No: 19-56041
Date: 2019-03-26 07:27
Site: PDC: Comstock Data Center - Santa Clara

Sounding: SCPT-02
Cone: 483:T1500F15U500



Max Depth: 36.750 m / 120.57 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56041_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137013m E: 592669m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Tabular Results

CONETEC

Job No:	19-56041
Client:	Kleinfelder
Project:	PDC: Comstock Data Center - Santa Clara
Sounding ID:	SCPT-02
Date:	26-Mar-2019

Seismic Source:	Beam
Source Offset (ft):	2.10
Source Depth (ft):	0.00
Geophone Offset (ft):	0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
5.74	5.09	5.50			
9.02	8.37	8.63	3.12	7.27	430
12.30	11.65	11.83	3.21	5.88	546
15.58	14.93	15.07	3.24	5.57	582
18.86	18.21	18.33	3.25	6.25	521
22.15	21.49	21.59	3.26	4.73	690
25.43	24.77	24.86	3.27	3.93	832
28.71	28.05	28.13	3.27	5.03	650
31.99	31.33	31.40	3.27	4.91	666
35.27	34.61	34.68	3.27	4.59	714
38.55	37.89	37.95	3.28	4.13	793
41.83	41.17	41.23	3.28	3.49	938
45.11	44.46	44.50	3.28	3.62	905
48.39	47.74	47.78	3.28	4.07	806
51.67	51.02	51.06	3.28	3.94	832
54.95	54.30	54.34	3.28	3.68	890
58.23	57.58	57.62	3.28	4.01	818
61.52	60.86	60.90	3.28	3.42	958
64.80	64.14	64.17	3.28	3.56	921
68.08	67.42	67.45	3.28	3.62	907
71.36	70.70	70.73	3.28	3.21	1022
74.64	73.98	74.01	3.28	3.36	976
77.92	77.26	77.29	3.28	3.43	957
81.20	80.54	80.57	3.28	3.04	1079
84.48	83.83	83.85	3.28	3.01	1091
87.76	87.11	87.13	3.28	2.98	1099
91.04	90.39	90.41	3.28	3.06	1071
94.32	93.67	93.69	3.28	2.98	1099
97.60	96.95	96.97	3.28	2.86	1145

Job No:	19-56041
Client:	Kleinfelder
Project:	PDC: Comstock Data Center - Santa Clara
Sounding ID:	SCPT-02
Date:	26-Mar-2019

Seismic Source:	Beam
Source Offset (ft):	2.10
Source Depth (ft):	0.00
Geophone Offset (ft):	0.66

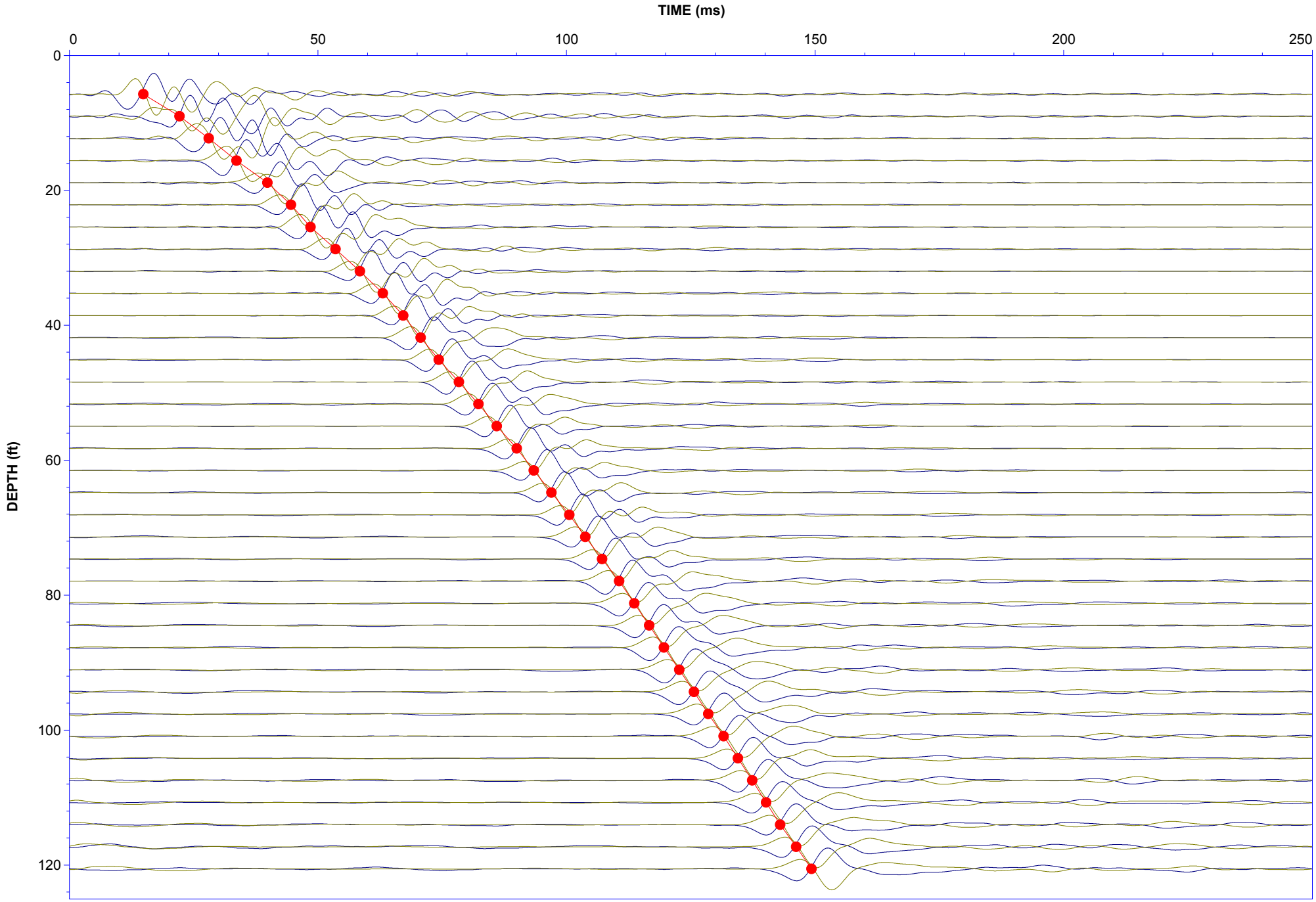
SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
100.89	100.23	100.25	3.28	3.10	1057
104.17	103.51	103.53	3.28	2.86	1145
107.45	106.79	106.81	3.28	2.86	1145
110.73	110.07	110.09	3.28	2.78	1178
114.01	113.35	113.37	3.28	2.86	1145
117.29	116.63	116.65	3.28	3.18	1031
120.57	119.91	119.93	3.28	3.10	1057

Seismic Cone Penetration Test Shear Wave (Vs) Traces

CONETEC

Job No: 19-56041 Client: Kleinfelder Project Title: PDC: Comstock Data Center - Santa Clara Filter: 10-200 Hz Hole: SCPT-02
Date: 03:26:19 07:27



Pore Pressure Dissipation Summary and
Pore Pressure Dissipation Plots

CONETEC

Job No:	19-56041
Client:	Kleinfelder
Project:	PDC: Comstock Data Center - Santa Clara
Start Date:	26-Mar-2019
End Date:	26-Mar-2019

CPTu PORE PRESSURE DISSIPATION SUMMARY

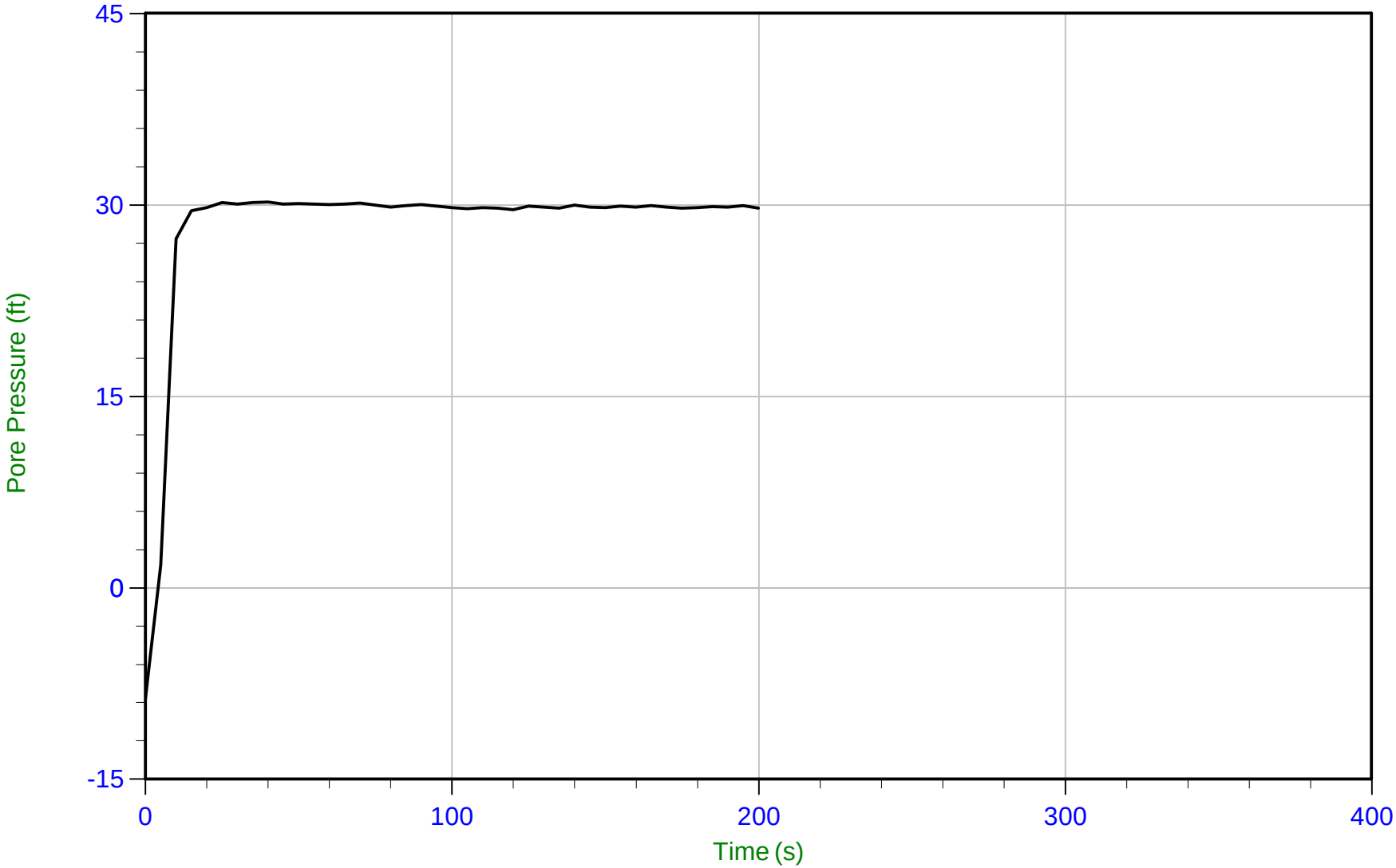
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
CPT-01	19-56041_CP01	15	200	38.30	29.8	8.5
SCPT-02	19-56041_SP02	15	315	20.42	13.3	7.1
CPT-03	19-56041_CP03	15	400	71.03	Not Achieved	

CONETEC

Kleinfelder

Job No: 19-56041
Date: 03/26/2019 09:45
Site: PDC: Comstock Data Center - Santa Clara

Sounding: CPT-01
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56041_CP01.PPF
Depth: 11.675 m / 38.303 ft
Duration: 200.0 s

u Min: -8.7 ft
u Max: 30.2 ft
u Final: 29.7 ft

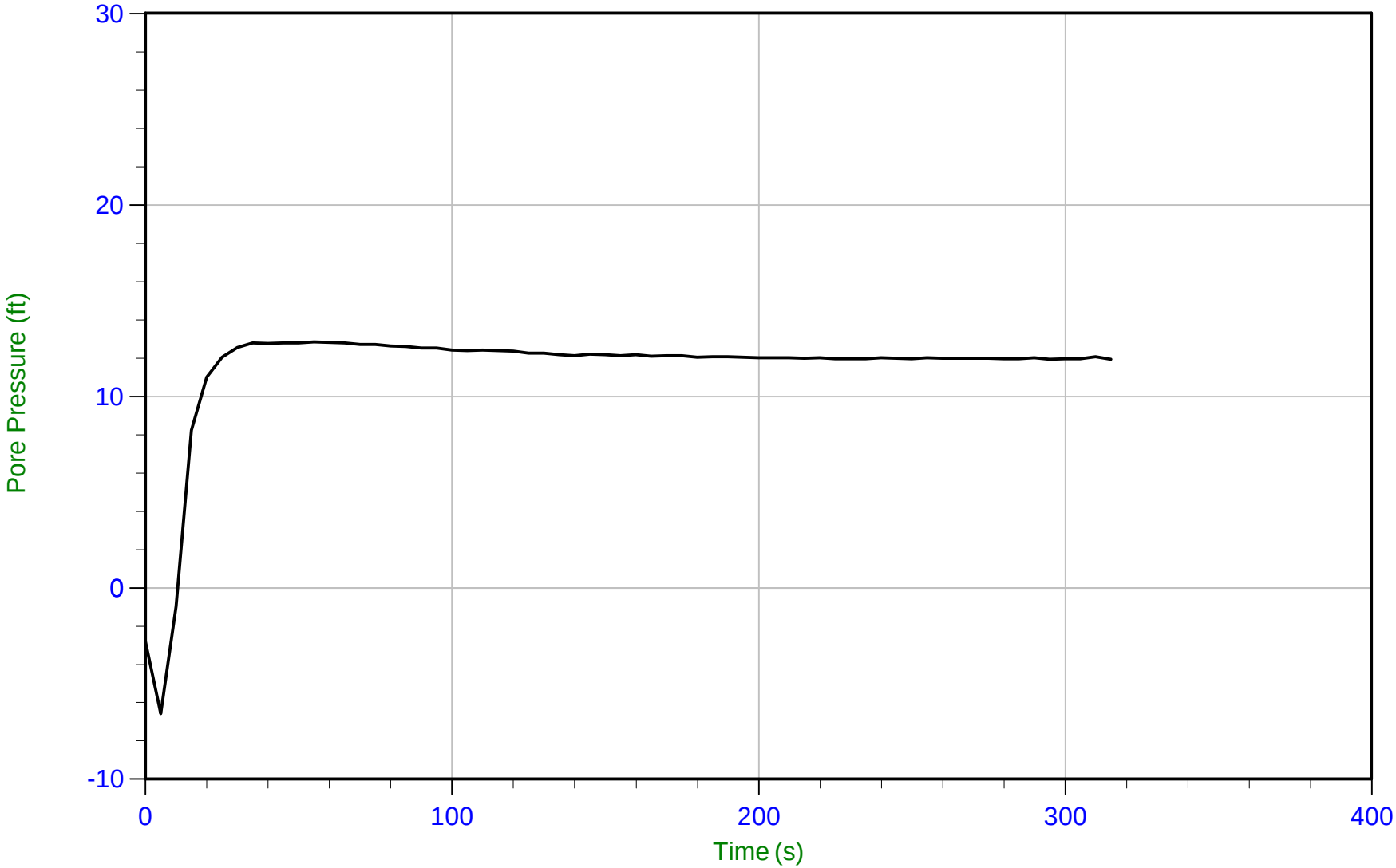
WT: 2.577 m / 8.455 ft
Ueq: 29.8 ft

CONETEC

Kleinfelder

Job No: 19-56041
Date: 03/26/2019 07:27
Site: PDC: Comstock Data Center - Santa Clara

Sounding: SCPT-02
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56041_SP02.PPF
Depth: 6.225 m / 20.423 ft
Duration: 315.0 s

u Min: -6.6 ft
u Max: 12.8 ft
u Final: 11.9 ft

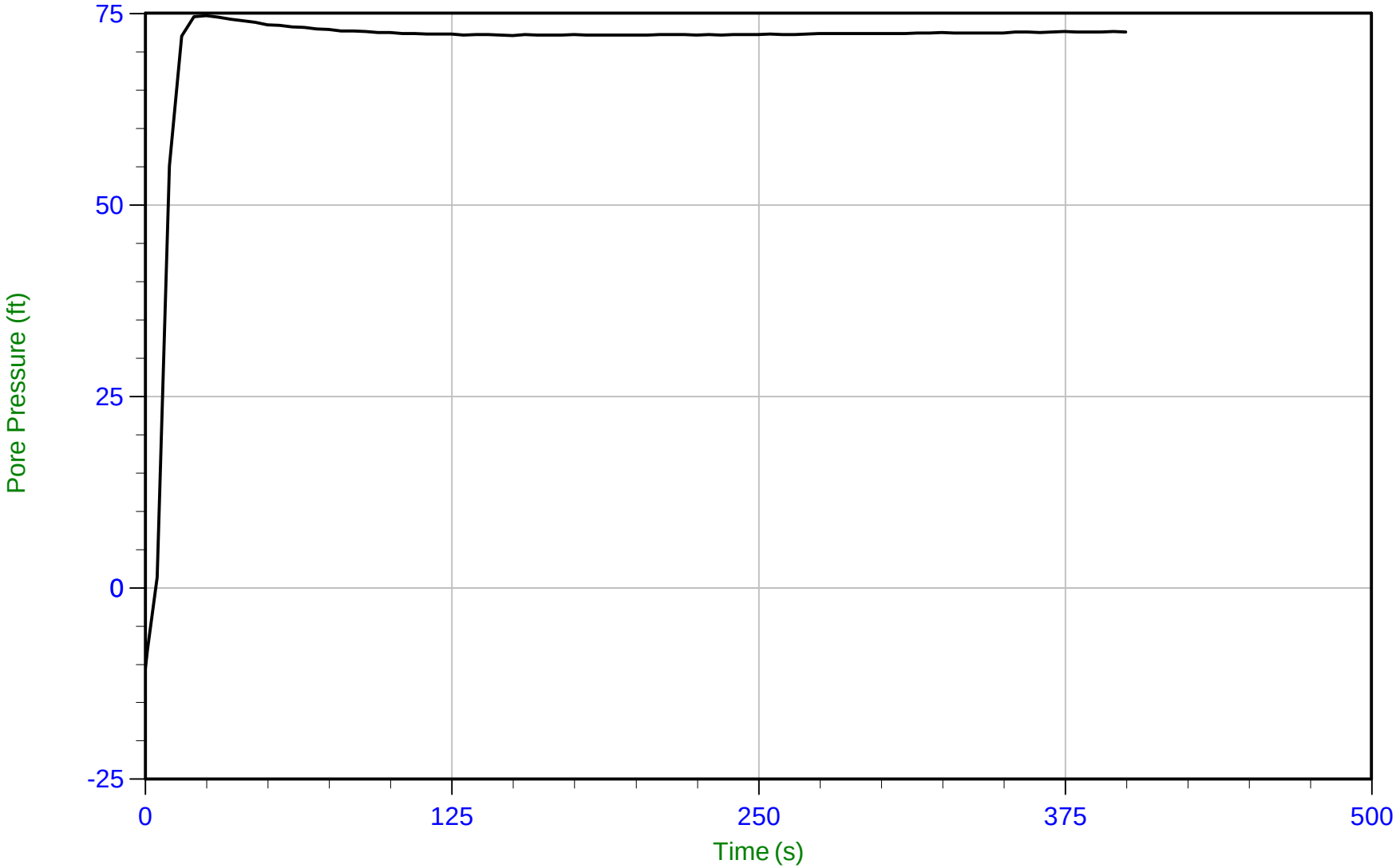
WT: 2.161 m / 7.090 ft
Ueq: 13.3 ft

CONETEC

Kleinfelder

Job No: 19-56041
Date: 03/26/2019 10:42
Site: PDC: Comstock Data Center - Santa Clara

Sounding: CPT-03
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56041_CP03.PPF
Depth: 21.650 m / 71.029 ft
Duration: 400.0 s

u Min: -10.5 ft
u Max: 74.7 ft
u Final: 72.6 ft

APPENDIX B
LOGS OF BORINGS

SAMPLER AND DRILLING METHOD GRAPHICS

- BULK / GRAB / BAG SAMPLE
- MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
- 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)
- SHELBY TUBE SAMPLER
- HOLLOW STEM AUGER
- SOLID STEM AUGER
- WASH BORING

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.

ABBREVIATIONS

WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

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 (Half or more 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
FINE GRAINED SOILS (Half or more 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 50 or greater)			**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.:	20193908
DRAWN BY:	JDS
CHECKED BY:	AA
DATE:	4/16/2019
REVISED:	-

GRAPHICS KEY

COMSTOCK DATA CENTER
1111 COMSTOCK STREET
SANTA CLARA, CALIFORNIA

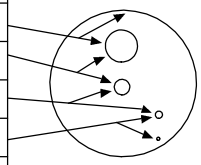
FIGURE

B-1

PLOTTED: 04/18/2019 08:10 AM BY: JSala
OFFICE FILTER: PLEASANTON
PROJECT NUMBER: 20193908.001A
gINT FILE: Klf_gint_master_2019 [LEGEND 1 (GRAPHICS KEY) USCS_STANDARD]
gINT TEMPLATE: E:\KLF_STANDARD_GINT_LIBRARY_2019.GLB

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



SECONDARY CONSTITUENT

	AMOUNT	
Term of Use	Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

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

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

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25 ≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5 ≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1 ≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2 ≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4 ≤ PP	>8000	Thumbnail will not indent soil.

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

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

FROM TERZAGHI AND PECK, 1948

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.

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.

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.

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20193908
DRAWN BY: JDS
CHECKED BY: AA
DATE: 4/16/2019
REVISED: -

SOIL DESCRIPTION KEY

COMSTOCK DATA CENTER
1111 COMSTOCK STREET
SANTA CLARA, CALIFORNIA

FIGURE
B-2

PLOTTED: 04/18/2019 08:11 AM BY: JSala
OFFICE FILTER: PLEASANTON
PROJECT NUMBER: 20193908.001A [LEGEND 2 (SOIL DESC KEY)]
gINT FILE: Klf_gint_master_2019
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2019.GLB

Date Begin - End:	4/02/2019	**Drilling Co.-Lic.#:**	Exploration Geo - #4821288		**BORING LOG B-1**
Logged By:	A. Sadat	**Drill Crew:**	L. Deniston		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B53 Mobile	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	STA/Mud Rotary		
Weather:	Raining	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION

Latitude: 37.37523°
Longitude: -121.95336°
Surface Condition: Asphalt

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
		approximately 3-inches asphalt												
		Sandy Fat CLAY (CH): high plasticity, greenish black, moist, trace gravel, very stiff	Bulk #1											
			2		BC=5 8 10 PP=3.5	18"		19.8	98.9			53	32	
		Lean CLAY (CL): low plasticity, dark gray, moist, trace gravel, stiff												
5		with sand	3		BC=9 5 8 PP=2.0	12"		21.6	103.2					switched to mud rotary
		Sandy Lean CLAY (CL): low plasticity, greenish gray, moist, stiff												
10			4		BC=3 4 4 PP=1.5	18"		22.2	108.2		51	28	8	
15			5		BC=4 5 5 PP=3.5	17"		28.5	96.6			49	28	TXUU: c = 1.44 ksf
		Poorly Graded gravelly SAND (SP): fine to coarse-grained sand, non-plastic, gray, wet, medium dense	6		BC=10 11 14	15"								
20			7		BC=2 7 8	18"								
25			8		BC=9 13 9	18"								
		Poorly Graded SAND (SP): non-plastic, gray, wet, dense	9		BC=10 22 23	18"								
30		**Silty SAND (SM)**: fine-grained sand, non-plastic, brownish gray, wet, medium dense, trace gravel	10		BC=6 9 11	18"								

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PROJECT NO.:	20193908
DRAWN BY:	JDS
CHECKED BY:	AA
DATE:	4/16/2019
REVISED:	-

BORING LOG B-1

COMSTOCK DATA CENTER
1111 COMSTOCK STREET
SANTA CLARA, CALIFORNIA

FIGURE

B-3

PLOTTED: 04/18/2019 02:27 PM BY: JSala
OFFICE FILTER: PLEASANTON
PROJECT NUMBER: 20193908.001A [__KLF_BORING/TEST PIT SOIL LOG]
gINT FILE: Klf_gint_master_2019
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2019.GLB

Date Begin - End:	4/02/2019	**Drilling Co.-Lic.#:**	Exploration Geo - #4821288		**BORING LOG B-1**
Logged By:	A. Sadat	**Drill Crew:**	L. Deniston		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B53 Mobile	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	STA/Mud Rotary		
Weather:	Raining	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION — Latitude: 37.37523° Longitude: -121.95336° Surface Condition: Asphalt

LABORATORY RESULTS

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, brownish gray, wet, medium dense, trace gravel												
35		**Sandy Lean CLAY (CL)**: low plasticity, greenish gray, moist, stiff	11		BC=4 6 7						78			
40		**Poorly Graded SAND with Silt and Gravel (SP-SM)**: low plasticity, olive gray, wet, medium dense	12		BC=12 16 22	16"					7.7			
45		**Sandy Lean CLAY (CL)**: low plasticity, olive, wet, stiff	13		BC=2 4 5 PP=1.5	18"								
50		medium plasticity, brown gray, moist	14		BC=5 6 8 PP=1.5	14"		18.5	113.1					TXUU: c = 1.40 ksf
55		less sand	15		BC=6 9 11 PP=1.5			19.3	112.9					
60														

The boring was terminated at approximately 60 ft. below ground surface. The boring was backfilled with grout on April 02, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion due to switching to mud rotary.
GENERAL NOTES:

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PROJECT NO.:	20193908
DRAWN BY:	JDS
CHECKED BY:	AA
DATE:	4/16/2019
REVISED:	-

BORING LOG B-1

COMSTOCK DATA CENTER
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FIGURE
B-3

PLOTTED: 04/18/2019 02:27 PM BY: JSala
OFFICE FILTER: PLEASANTON
PROJECT NUMBER: 20193908.001A [_KLF_BORING/TEST PIT SOIL LOG]
gINT FILE: Klf_gint_master_2019
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2019.GLB

Date Begin - End:	4/02/2019	**Drilling Co.-Lic.#:**	Exploration Geo - #4821288		**BORING LOG B-2**
Logged By:	A. Sadat	**Drill Crew:**	L. Deniston		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B53 Mobile	**Hammer Type - Drop:**	140 lb. Safety Drive - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Raining	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION

Latitude: 37.37482°
Longitude: -121.95361°
Surface Condition: Asphalt

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
		approximately 3-inches asphalt	Bulk #1											
		Fat CLAY (CH): high plasticity, moist, trace sand, organic, very stiff												
			1		BC=7 8 13 PP=2.5	5"								
		less sand												
5		**Lean CLAY with Sand (CL)**: medium plasticity, dark gray, moist, very stiff	2		BC=5 9 13 PP=3.5	6"								
			3		BC=2 2 3	6"								
10		**Sandy Lean CLAY (CL)**: low plasticity, light brownish gray, wet, soft to medium stiff						26.5	102.0					TXUU: c = 0.25 ksf
		Lean CLAY (CL): medium plasticity, gray, moist, trace sand, stiff												
			4		BC=4 5 6 PP=1.5	13"								
15														
		more sand with depth, less plastic	5		BC=3 5 6	18"					52			
20		**Poorly Graded SAND and Gravel (SP)**: non-plastic, bluish gray, wet, dense	6		BC=9 13 20	12"								
		trace gravel up to 1" in diameter	7		BC=16 17 13	18"		15.7	112.5					
25														based on auger advancement, material appeared uniform.
		Lean CLAY (CL): trace fine to medium-grained sand, olive, moist, very stiff	8		BC=16 17 19 PP=2.0	18"		26.0	100.8					
30														

LABORATORY RESULTS

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DRAWN BY:	JDS
CHECKED BY:	AA
DATE:	4/16/2019
REVISED:	-

BORING LOG B-2

COMSTOCK DATA CENTER
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SANTA CLARA, CALIFORNIA

FIGURE

B-4

PLOTTED: 04/18/2019 02:28 PM BY: JSala
OFFICE FILTER: PLEASANTON
PROJECT NUMBER: 20193908.001A [_KLF_BORING/TEST PIT SOIL LOG]
gINT FILE: Klf_gint_master_2019
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2019.GLB

Date Begin - End:	4/02/2019	**Drilling Co.-Lic.#:**	Exploration Geo - #4821288		**BORING LOG B-2**
Logged By:	A. Sadat	**Drill Crew:**	L. Deniston		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B53 Mobile	**Hammer Type - Drop:**	140 lb. Safety Drive - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Raining	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION

Latitude: 37.37482°
Longitude: -121.95361°
Surface Condition: Asphalt

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 to medium-grained sand, olive, moist, very stiff												
		Sandy Lean CLAY (CL): low plasticity, greenish gray, moist, stiff	9		BC=7 8 11 PP=1.5	18"		26.9	101.4		51			
35														
		Clayey SAND (SC): medium plasticity, greenish gray, wet, medium dense	10		BC=5 7 12 PP=0.5	18"		23.3	106.3					TXUU: c = 0.46 ksf
40														
45														
50														
55														
60														

LABORATORY RESULTS

The boring was terminated at approximately 40 ft. below ground surface. The boring was backfilled with grout on April 02, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion due to switching to mud rotary.
Groundwater was observed at approximately 10 ft. below ground surface at the end of drilling.
GENERAL NOTES:

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PROJECT NO.:	20193908
DRAWN BY:	JDS
CHECKED BY:	AA
DATE:	4/16/2019
REVISED:	-

BORING LOG B-2

COMSTOCK DATA CENTER
1111 COMSTOCK STREET
SANTA CLARA, CALIFORNIA

FIGURE
B-4

PLOTTED: 04/18/2019 02:28 PM BY: JSala
OFFICE FILTER: PLEASANTON
PROJECT NUMBER: 20193908.001A
gINT FILE: Klf_gint_master_2019
gINT TEMPLATE: E:\KLF_STANDARD_GINT_LIBRARY_2019.GLB [_KLF_BORING/TEST PIT SOIL LOG]

APPENDIX C
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	
B-1	2.5		GREENISH BLACK SANDY FAT CLAY WITH GRAVEL (CH)	19.8	98.9				53	21	32	
B-1	5.0			21.6	103.2							
B-1	9.5		GREENISH GRAY SANDY LEAN CLAY (CL)						28	20	8	
B-1	10.0		GREENISH GRAY SANDY LEAN CLAY (CL)	22.2	108.2			51				
B-1	14.0		GREENISH GRAY SANDY LEAN CLAY (CL)						49	21	28	
B-1	14.5		GREENISH GRAY SANDY LEAN CLAY (CL)	28.5	96.6							TXUU: c = 1.44 ksf
B-1	34.5		GREENISH GRAY SANDY LEAN CLAY (CL)					78				
B-1	42.5		OLIVE GRAY POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)					7.7				
B-1	54.5		LIGHT BROWNISH GRAY SANDY LEAN CLAY (CL)	18.5	113.1							TXUU: c = 1.40 ksf
B-1	59.0			19.3	112.9							
B-2	10.0		LIGHT BROWNISH GRAY SANDY LEAN CLAY (CL)	26.5	102.0							TXUU: c = 0.25 ksf
B-2	18.0		GRAY SANDY LEAN CLAY (CL)					52				
B-2	24.5			15.7	112.5							
B-2	29.5			26.0	100.8							
B-2	33.5		GREENISH GRAY SANDY LEAN CLAY (CL)					51				
B-2	34.0			26.9	101.4							
B-2	39.0			23.3	106.3							
B-2	39.5		GREENISH GRAY CLAYEY SAND (SC)									TXUU: c = 0.46 ksf

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

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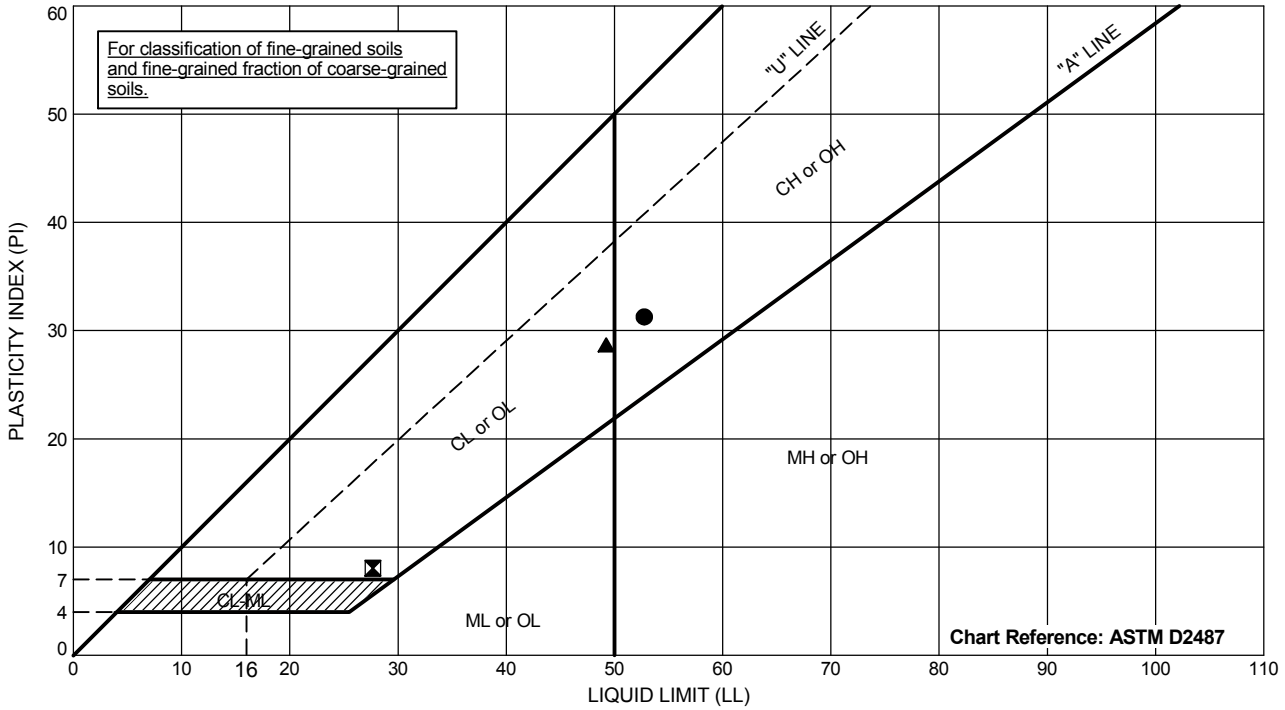
FIGURE
C-1



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	Passing #200	LL	PL	PI
●	B-1	2.5	NA	GREENISH BLACK SANDY FAT CLAY WITH GRAVEL (CH)	NM	53	21	32
⊠	B-1	9.5	NA	GREENISH GRAY SANDY LEAN CLAY (CL)	NM	28	20	8
▲	B-1	14	NA	GREENISH GRAY SANDY LEAN CLAY (CL)	NM	49	21	28

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

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PROJECT NO.:	20193908
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CHECKED BY:	AA
DATE:	4/16/2019
REVISED:	-

ATTERBERG LIMITS

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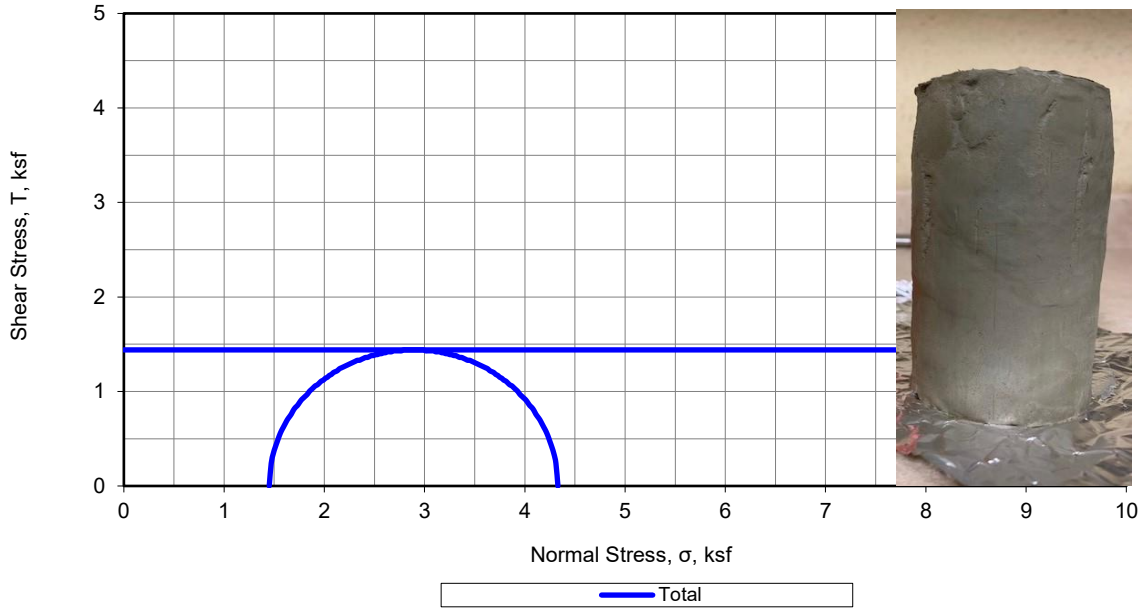
SANTA CLARA, CALIFORNIA

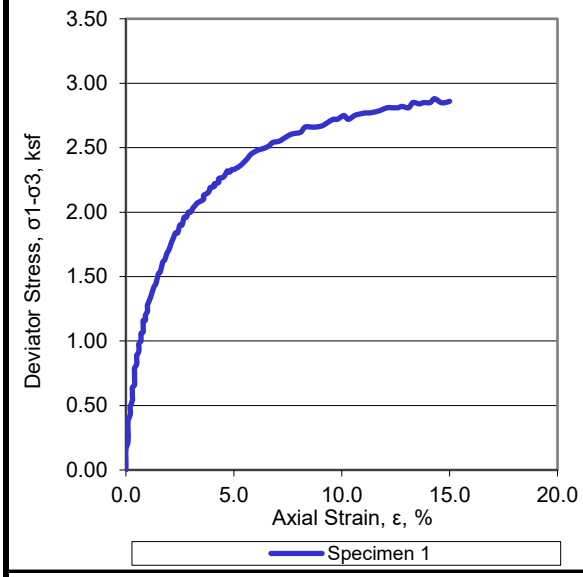
FIGURE
C-2

PLOTTED: 04/18/2019 08:15 AM BY: JSala
OFFICE FILTER: PLEASANTON
PROJECT NUMBER: 20193908.001A
gINT FILE: Klf_gint_master_2019 [KLF_ATTERBERG (ASTM)]
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2019.GLB

Total	
c = 1.44	ksf



Specimen Shear Picture



Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	5.01
	Water Content, %	ω_O	28.5
	Dry Density, lbs/ft^3	$^{\gamma}d_o$	96.7
	Saturation, %	S_O	106
	Void Ratio	e_O	0.710
Minor Principal Stress, ksf		σ_3	1.45
Maximum Deviator Stress, ksf		$(\sigma 1-\sigma 3)_{max}$	2.88
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.33
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	2.86
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.33

Description of Specimen: Greenish Gray Sandy Lean Clay (CL)

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

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

Membrane correction applied

Boring:	B-1
Sample:	C5
Depth, ft:	14.5
Test Date:	4/9/19

Remarks: nm= not measured, na = not applicable

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Project No.:	20193908
Date:	4/12/19
Entry By:	MS
Checked By:	MS
File Name:	HL12014

TRIAXIAL COMPRESSION TEST (UU)

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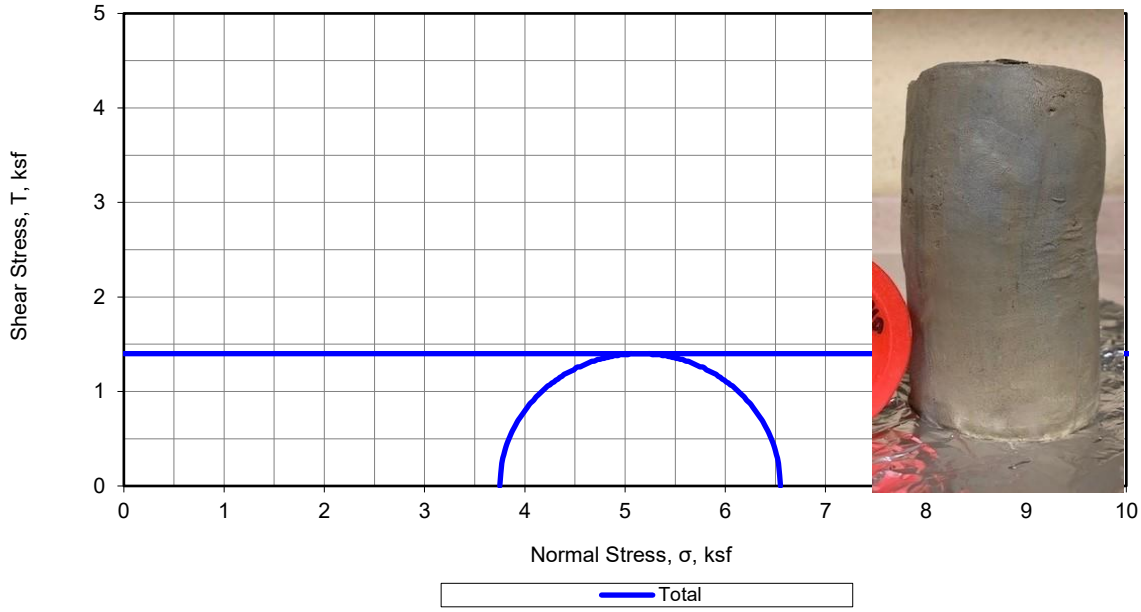
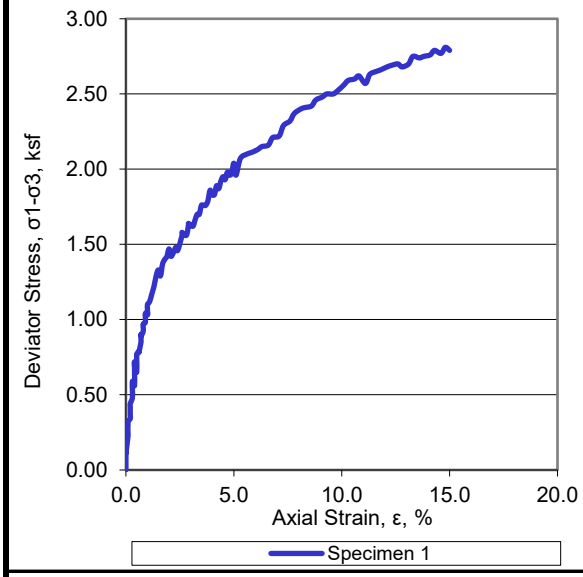
FIGURE
C-3

Total	
c = 1.40	ksf



Specimen Shear Picture



Specimen No.			1
Initial	Diameter, in	D_O	2.42
	Height, in	H_O	5.65
	Water Content, %	ω_O	18.5
	Dry Density, lbs/ft^3	$^{\gamma}d_o$	113.1
	Saturation, %	S_O	106
	Void Ratio	e_O	0.463
Minor Principal Stress, ksf		σ_3	3.75
Maximum Deviator Stress, ksf		$(\sigma 1-\sigma 3)_{max}$	2.81
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.85
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	2.79
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.85

Description of Specimen: Light Brownish Gray Sandy Lean Clay (CL)

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

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

Membrane correction applied

Boring:	B-1
Sample:	C14
Depth, ft:	54.5
Test Date:	4/9/19

Remarks: nm= not measured, na = not applicable

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Project No.:	20193908
Date:	4/12/19
Entry By:	MS
Checked By:	MS
File Name:	HL12014

TRIAXIAL COMPRESSION TEST (UU)

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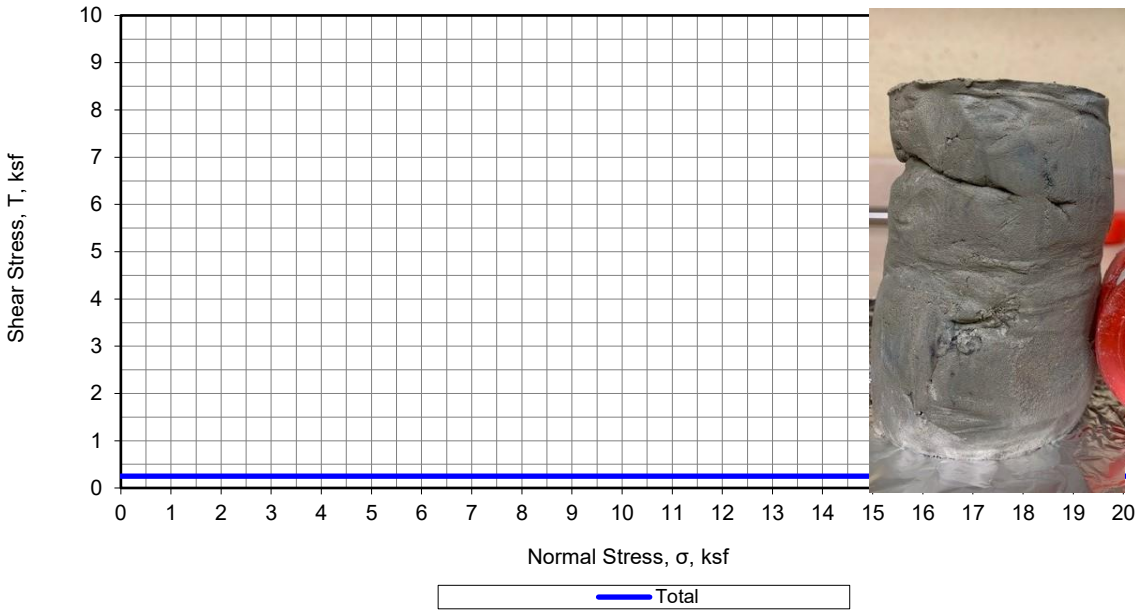
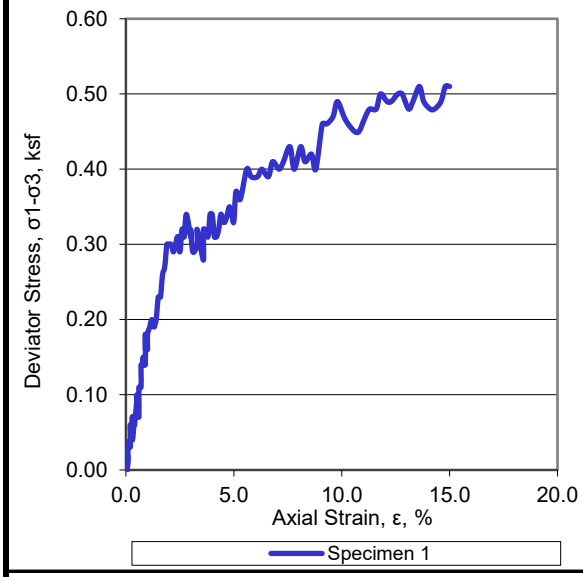
FIGURE
C-4

Total		
c =	0.25	ksf



Specimen Shear Picture



Specimen No.			1
Initial	Diameter, in	D_O	2.33
	Height, in	H_O	5.17
	Water Content, %	ω_O	26.5
	Dry Density, lbs/ft^3	$^{\gamma}d_o$	102.0
	Saturation, %	S_O	113
	Void Ratio	e_O	0.620
Minor Principal Stress, ksf		σ_3	1.20
Maximum Deviator Stress, ksf		$(\sigma 1-\sigma 3)_{max}$	0.51
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	15.00
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	0.51
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	15.00

Description of Specimen: Light Brownish Gray Sandy Lean Clay (CL)

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

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

Membrane correction applied

Boring:	B-2
Sample:	C3
Depth, ft:	10.0
Test Date:	4/10/19

Remarks: nm= not measured, na = not applicable
shear stress vs normal stress graph is in psi
(normally in ksf) to get a better scale.

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Project No.:	20193908
Date:	4/12/19
Entry By:	MS
Checked By:	MS
File Name:	HL12014

TRIAXIAL COMPRESSION TEST (UU)

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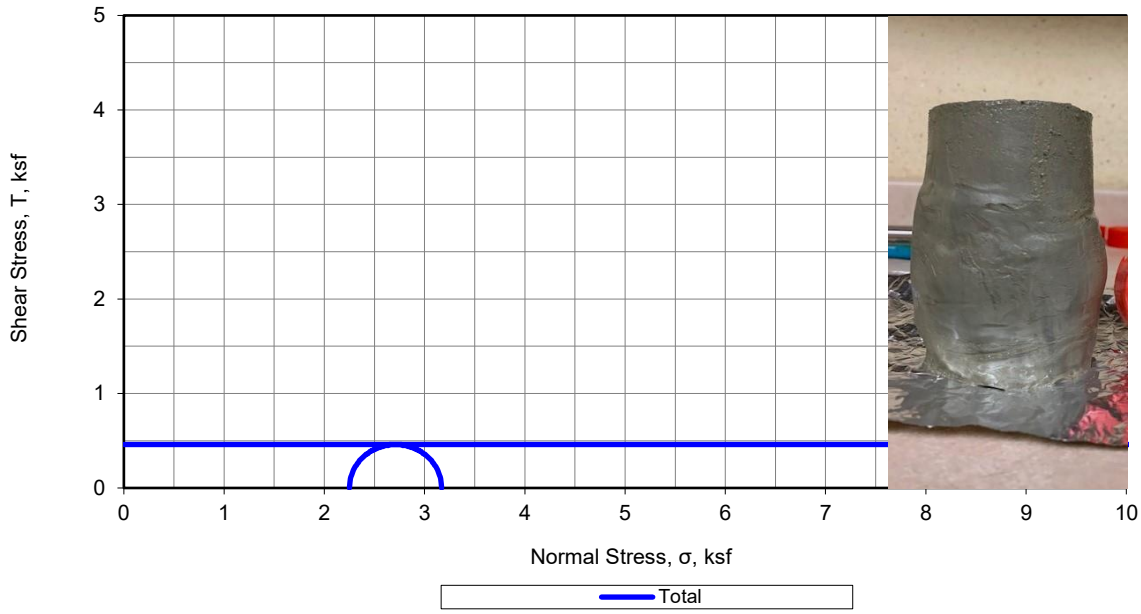
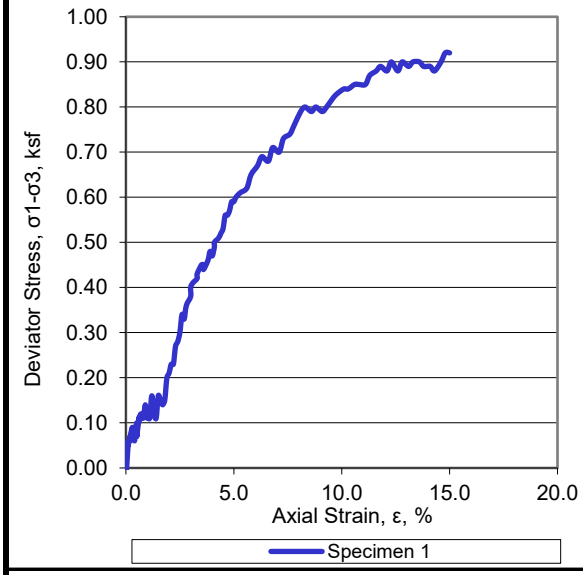
SANTA CLARA, CALIFORNIA

FIGURE
C-5

Total	
c = 0.46	ksf



Specimen Shear Picture



Specimen No.			1
Initial	Diameter, in	D_O	2.37
	Height, in	H_O	5.34
	Water Content, %	ω_O	21.3
	Dry Density, lbs/ft^3	γd_o	110.1
	Saturation, %	S_O	113
	Void Ratio	e_O	0.502
Minor Principal Stress, ksf		σ_3	2.25
Maximum Deviator Stress, ksf		$(\sigma 1-\sigma 3)_{max}$	0.92
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.83
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	0.92
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	14.83

Description of Specimen: Greenish Gray Clayey Sand (SC)

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

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

Membrane correction applied

Boring:	B-2
Sample:	C10
Depth, ft:	39.5
Test Date:	4/10/19

Remarks: nm= not measured, na = not applicable

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Project No.:	20193908
Date:	4/12/19
Entry By:	MS
Checked By:	MS
File Name:	HL12014

TRIAXIAL COMPRESSION TEST (UU)

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FIGURE
C-6

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.: 20193908.001A
Client's Project Name: Comstock Data Center - Santa Clara
Date Sampled: 2-Apr-2019
Date Received: 8-Apr-2019
Matrix: Soil
Authorization: Chain of Custody

Date of Report: 10-Apr-2019

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)*
1904063-001	B-2, Bulk 1 @ 0-2' Bulk	+260	8.17	520	750	N.D.	N.D.	67

Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	9-Apr-2019	9-Apr-2019	9-Apr-2019	9-Apr-2019	8-Apr-2019	9-Apr-2019	9-Apr-2019

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

Cheryl McMillen
Laboratory Director

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

APPENDIX D
GBA INFORMATION SHEET

Important Information about This

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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering 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 given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled*. No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- 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;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the 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. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed*. The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation*.

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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 personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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