GEOTECHNICAL INVESTIGATION REPORT
DUKE SILVER CREEK VALLEY ROAD WAREHOUSE
SAN JOSE, CALIFORNIA

Project No. 20221404.001A

MARCH 4, 2022

March 4, 2022
Project No. 20221404.001A

Mr. Jason Bernstein
Duke Realty
1904 Franklin St., 8th Floor
Oakland, CA 94612

Jason.Bernstein@dukerealty.com
Cc: tom.jodry@dukerealty.com

SUBJECT: GEOTECHNICAL INVESTIGATION REPORT
DUKE SILVER CREEK VALLEY ROAD WAREHOUSE
SAN JOSE, CALIFORNIA

Dear Mr. Bernstein:

The attached report presents the results of Kleinfelder's geotechnical investigation for the proposed industrial warehouse building to be located on Silver Creek Valley Road in San Jose, 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, project plans and specifications should be reviewed by our office prior to finalization and their submittal and issuance to verify conformance with the general intent of the recommendations presented in the enclosed report.

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.

Renie Yuen, PE 81159
Senior Project Engineer

Brian O'Neill, PE, GE 2516
Sr. Principal Geotechnical Engineer

William V. McCormick, CEG 1673
Sr. Principal Engineering Geologist, VP

TABLE OF CONTENTS

FIGURES

APPENDICES

1 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed industrial warehouse building to be located at the northwest corner of the intersection of Silver Creek Valley Road and Fontanoso Way in San Jose, 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 Duke Realty (Duke) staff, as well as various correspondence over emails with members of Duke's engineering design team until the submission date of this report.

1.1 PROJECT DESCRIPTION

According to email correspondence with Duke and our review of the overall site plan provided by HPA Architecture and dated October 1, 2021, it is our understanding the project consists of construction of a new warehouse building and surrounding pavement and parking. The currently proposed single-story warehouse is approximately 282,430 square feet in plan view. We understand that the building preferably would be constructed using a shallow foundation system, with slab-on-grade floors and concrete exterior wall panels. At the time of preparation of this report, we understand that typical 100- to 125-kip column loads are assumed, and that the structure most likely will be constructed utilizing standard concrete tilt-up wall construction.

Grading plans have not been provided at the time of this report. However, it is anticipated that grading will consists of minor cuts and fills of about 1 to 3 feet to achieve building pad and truck bay loading dock area grades and provide adequate gradients for site drainage.

1.2 PURPOSE AND SCOPE OF SERVICES

Our scope of services was presented in our proposals dated June 24, 2021 and September 3, 2021. 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
- Obtain a drilling permit from the Santa Clara Valley Water District for cone penetration tests (CPTs) advanced deeper than 45 feet below the ground surface
- Coordination with a private utility locating subconsultant to clear exploration locations
- Review of existing geotechnical data and published geologic and seismic reports and maps covering the site area
- Field investigation program comprised of:
    - Eight (8) exploratory soil borings
    - Six (6) CPTs
    - Three (3) percolation tests
- Geotechnical laboratory testing to assess the physical characteristics and engineering properties of the site soils
- In-situ soil resistivity measurements and soil corrosivity laboratory testing to evaluate corrosion potential
- Environmental laboratory testing of samples from the drilled borehole cuttings to determine hazardous or non-hazardous status of the cuttings for disposal purposes
- Developing site-specific seismic design parameters based on a Site Response Analysis (SRA)
- Analyses of the field and laboratory test data and development of geotechnical conclusions and recommendations for design and construction
- Preparation of this report

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

2 FIELD EXPLORATION

2.1 PREPARATION FOR EXPLORATIONS

Prior to our subsurface exploration program, the site was delineated with white paint, the boring and CPT locations were staked, and the Underground Service Alert (USA) network was contacted to provide clearance for utilities within the public right-of-way. In addition, prior to performing the field investigation, a private utility location survey was performed by our subcontractor Ground Penetrating Radar Systems, LLC (GPRS) of San Francisco, California, to locate underground utilities and infrastructure at each exploration location. A drilling permit was obtained from Valley Water for explorations advanced deeper than 45 feet below the ground surface. A site-specific health and safety plan was prepared for the field exploration activities. This plan was discussed with the field crew daily prior to the start of field exploration activities.

2.2 EXPLORATORY BORINGS

To evaluate the subsurface conditions at the site for the proposed building, eight borings (KB-1 through KB-8) were drilled by our subcontractor, Exploration Geoservices, Inc. of San Jose, California, on July 29 and July 30, 2021, using a Mobile B-53 truck-mounted drill rig equipped with hollow flight augers and mud rotary drilling system. The depth of the borings ranged from approximately 11½ to 44½ feet below the existing ground surface. The borings were approximately located in the field by visual sighting and/or measuring with a mobile GPS application. Therefore, the locations of the borings shown on Figure 2 should be considered approximate. The physical attributes of the borings are summarized in the following table.

Table 2-1. Summary of Borings

Boring No.	Approx. Coordinates		Drill Date	Approx. Depth of Boring (feet)
	Latitude (°)	Longitude (°)		
KB-1	37.25913	-121.78974	7/29/2021	11½
KB-2	37.25817	-121.78940	7/30/2021	26½
KB-3	37.25897	-121.78847	7/29/2021	15
KB-4	37.25844	-121.78822	7/30/2021	44½
KB-5	37.25803	-121.78809	7/29/2021	15
KB-6	37.25765	-121.78791	7/29/2021	11½
KB-7	37.25960	-121.78793	7/29/2021	11½
KB-8	37.25884	-121.78703	7/29/2021	25

A Kleinfelder professional maintained logs of the borings, visually classified the soils encountered according to the Unified Soil Classification System (presented on Figure A-1 in Appendix A) and obtained samples of the subsurface materials. Soil classifications made in the field from samples were done in accordance with American Society for Testing and Materials (ASTM) Method D2488. These classifications were re-evaluated in the laboratory after further examination and testing in accordance with ASTM D2487. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the boring logs.

Keys to the graphics and soil descriptions used on the boring logs are presented on Figures A-1 and A-2 in Appendix A. Logs of the borings are presented on Figures A-3 through A-10. The approximate locations of the exploration borings are shown on the Site Plan, Figure 2.

Samples were obtained from the borings at selected depths by driving either a 2½-inch inside diameter (I.D.) California sampler or a 1⅜-inch I.D. standard penetration test (SPT) sampler into undisturbed soil with either a 140-pound automatic or wireline downhole 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 A-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 cement grout. Cuttings and fluids (Investigation Derived Waste [IDW]) from the borings were containerized in 55-gallon drums, removed from the site, and later disposed of by our drilling contractor following review of laboratory analytical testing results to confirm that the IDW was deemed non-hazardous.

2.3 CONE PENETRATION TESTS

Four CPTs (SCPT-01, CPT-02, CPT-03, and SCPT-4) were performed by ConeTec, Inc. of San Leandro, California, on July 29, 2021. The CPTs were advanced to a depth of approximately 44½ feet below the existing ground surface. An additional two CPTs (SCPT-05 and SCPT-06) were advanced to cone refusal depths of approximately 88 feet and 62 feet, respectively, on January 20, 2022. The information gathered from the CPTs was used for subsurface characterization identifying potential liquefiable and soft soils, measuring shear wave velocity values (in SCPTs), and for evaluating foundation design parameters.

The CPTs 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. A pore pressure dissipation test was performed in each of the CPTs at various depths. Seismic shear wave velocity measurements were collected from the SCPTs (SCPT-01, SCPT-04, SCPT-05, and SCPT-06) at approximate 1-meter increments to the bottom of the soundings. The CPTs were all backfilled with cement grout in accordance with local permit requirements. The soundings were located by the CPT operator using consumer grade GPS equipment. The physical attributes of the CPTs are summarized in the following table.

Table 2-2. Summary of CPTs

CPT No.	Coordinates		Date Advanced	Approx. Depth of CPT (feet)
	Latitude (°)	Longitude (°)		
SCPT-01	37.258619	-121.789520	7/29/21	44½
CPT-02	37.257715	-121.789208	7/29/21	44½
CPT-03	37.259312	-121.787592	7/29/21	44½
SCPT-04	37.258401	-121.786727	7/29/21	44½
SCPT-05	37.258180	-121.788840	1/20/2022	88
SCPT-06	37.258890	-121.787757	1/20/2022	61¾

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 while the cone tip resistance (Qt) and sleeve friction resistance (Fs) were recorded in 2.5-centimeter increments. The testing was performed using a 30-ton push capacity, truck-mounted CPT 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, 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 et al. (1986) and Robertson (1990 and 2009). 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 B.

2.4 PERCOLATION TESTING

We evaluated the feasibility for in-situ soil infiltration in the vicinity of the proposed drainage basins at the north and south ends and the southeast corner of the property by performing a total of three borehole percolation tests (KP-1 through KP-3). The three test holes were drilled 10 feet deep, measured 8 inches in diameter, and were pre-soaked for between 22 to 24 hours before initiating the tests. Infiltration may be controlled primarily by factors such as the type and porosity of the surface filtering media, maintenance of these media, surface slope, surface vegetation, and intensity, duration, and type of precipitation. Surface drainage and maintenance will typically determine the site's infiltration rate and the amount of water that will infiltrate for any given storm.

Based on visual soil classification and laboratory testing of the soil samples collected during our field explorations, the upper approximately 10 feet of the subsurface soils consist predominantly of low plasticity clays. The following table summarizes the short-term in-situ infiltration rates for each test location.

Table 2-3. Short-Term Infiltration Rates

Percolation Test Location	Depth (feet)	Soil Description	Tested Depth from Ground Surface (feet)	Short-Term Infiltration Rate (inch/hour)
KP-1	0 to 10	Lean Clay (CL) [5 to 10 ft: 95.3% passing No. 200]	4⅓ to 10	0.05
KP-2	0 to 10	Lean Clay (CL)	4 to 10	0.02
KP-3	0 to 10	Lean Clay (CL) [5 to 10 ft: 92.8% passing No. 200]	3⅓ to 10	0.02

Note: Short-term infiltration rates include a reduction factor using the "Porchet Method" to adjust for non-vertical percolation through the sides of the borehole.

The short-term infiltration rates provided in the above table have been reduced to account for non-vertical infiltration through the sides of the borehole, but do not include any safety factors for long-term performance. While Santa Clara County does not provide specific guidance on a factor of safety, we recommend using a factor of safety of at least 3 due to the variability in test results and to account for long-term performance. The civil engineer should determine the applicability of the factor of safety and may apply a higher factor of safety depending on the performance objectives.

In general, since the infiltration rates are low, the near-surface soils are not expected to be conducive for use as material for surface water runoff infiltration basins. However, if infiltration basins are constructed in the near-surface soils, then the recommended design infiltration rate assumes the basins are protected from unintended, indirect compaction during construction of the basins. If it occurs that soils become densified during construction of the basin, then the surface soils within the basin should be scarified or shallow ripped. Periodic maintenance of infiltration ponds is necessary to maintain adequate infiltration rates. Maintenance should include repair of any areas of developing erosion and remove of vegetation, accumulation of organics, and sediment build-up. Vegetation, organics, and sediments should not be disced or mixed into the basin bottom; instead, they should be removed from the basin to maintain performance.

3 LABORATORY TESTING

3.1 GEOTECHNICAL 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)
- Atterberg Limits (ASTM D5318)
- Grain Size – Percent Passing No. 200 Sieve (ASTM D1140)
- Unconsolidated Undrained Triaxial Shear (ASTM D2850)
- R-Value (ASTM D2844)

The laboratory test results are summarized on the boring logs and are presented on Figures C-1 through C-7 in Appendix C of this report.

3.2 SOIL CORROSION POTENTIAL SCREENING TESTING

One near-surface soil sample was submitted to CERCO Analytical, Inc. (CERCO) of Concord, California, for a brief corrosion analysis. Laboratory testing included the following:

- pH (ASTM D4972)
- Electrical resistivity (ASTM G57)
- Soluble chloride (ASTM D4327)
- Soluble sulfate (ASTM D4327)

The results of the preliminary corrosion screening testing are presented in Section 6.13. The soil corrosivity test results are presented in Appendix D.

3.3 ANALYTICAL TESTING (ENVIRONMENTAL CONTAMINANT SCREENING)

Two samples of the containerized IDW drill cuttings and fluid were submitted to McCampbell Analytical, Inc. of Pittsburg, California, for laboratory testing and included the following:

- PCBs (EPA Method 8082)
- CAM 17 Metals and STLC (EPA Method 6020)
- Volatile Hydrocarbons as Gasoline with BTEX and MTBE (EPA Method 8015B/8015Bm)
- pH (EPA Method 9045C)
- Total Petroleum Hydrocarbons purgeable as gasoline and extractable as diesel and motor oil (EPA Method 8015B)

The IDW waste was determined to be non-hazardous for landfill disposal. The analytical test results were transmitted to Duke on August 5, 2021, and are included in Appendix E.

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 and seismically induced secondary ground failures (liquefaction, lateral spreading, and dynamic compaction).

4.1 REGIONAL GEOLOGY

The project site lies within the central portion of the Coast Ranges geomorphic province of California. This province is comprised of a discontinuous series of northwest-southeast trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The dominant geologic structure within the Coast Ranges province is generally controlled by the San Andreas fault system, which is a major tectonic transform plate boundary. This right-lateral strike-slip fault system extends from the Gulf of California in Mexico to Cape Mendocino in Northern California and forms a portion of the boundary between two tectonic plates. In this portion of the Coast Ranges province, the Pacific Plate (located west of the transform boundary) moves north relative to the North American Plate (located east of the transform boundary). Deformation along this plate boundary occurs across a wide zone that is referred to as the San Andreas Fault (SAF) system.

Basement rocks west of the SAF are generally granitic, while those to the east consist of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (about 199 to 65 million years old). Overlying the basement rocks are Cretaceous (about 145 to 65 million years old) marine, as well as Tertiary (about 65 to 2.6 million years old [USGS, 2010]) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have been extensively folded and faulted as a result of late Tertiary and Quaternary regional compressional forces.

The inland valleys, as well as the structural depression within which the Santa Clara Valley and San Francisco Bay are located, are filled with unconsolidated to semi-consolidated continental deposits of Quaternary age (about the last 2.6 million years). Continental surficial deposits (alluvium, colluvium, and landslide deposits) consist of unconsolidated to semi-consolidated sand, silt, clay, and gravel while the Bay deposits typically consist of very soft organic-rich silt and clay (Young Bay Mud) and sand.

4.2 SITE GEOLOGY

The site has been mapped by Wentworth et al. (1999), the California Geological Survey (2000), and Witter et al. (2006), among others. Wentworth et al. (1999) indicate the site is underlain by Holocene age (approximately 11,700 years old to present day) alluvial fan deposits, consisting of moderately dense to dense gravelly sand, sandy and clayey gravel, grading upward to sandy and silty clay. The California Geological Survey (2000) and Witter et al. (2006) are in agreement and indicate the northeast half of the site is underlain by Holocene age alluvial fan deposits, while the southwest half the site is underlain by Holocene age alluvial fan levee deposits. According to Witter et al. (2006), the fan deposits consist of moderately to poorly sorted, moderately to poorly bedded sand, gravel, silt and clay, and the levee deposits are comprised of loose, moderately to well sorted sand silt and clay.

Witter et al. (2006) indicate the alluvial fan deposits and alluvial fan levee deposits are moderately susceptible to liquefaction. The County of Santa Clara (2021) and the California Geological Survey (2021) have located the site within liquefaction zones, where liquefaction related ground surface effects have historically occurred, or where subsurface soil and groundwater conditions indicate the potential for liquefaction to occur during a seismic event.

4.3 FAULTING AND SEISMICITY

The site is not located within an Earthquake Fault Zone as defined by the California Geological Survey (CGS, 2021) in accordance with the Alquist-Priolo Earthquake Fault Zone Act of 1972. According to the CGS (2021), the nearest zoned active fault to the site is the Hayward fault, located approximately 3.0 miles northeast of the site. The Working Group on California Earthquake Probabilities (2015), however, indicate the Silver Creek fault is the most proximal, located approximately 1.9 miles northeast of the site. Moderate to major earthquakes generated on the Hayward, the Silver Creek, and other faults in the region can be expected to cause strong ground shaking at the site. The proximities of significant faults in the vicinity of the site are listed in the SRA presented in Appendix F.

The United States Geological Survey (2021) identifies the Coyote Creek fault zone located approximately 0.2 miles northeast of the site. According to the U.S. Geological Survey (2021), the trace is designated undifferentiated Quaternary (exhibits deformation in the last 1.6 million years). This trace is not zoned as active by the CGS (2021) and is not considered a source of seismic shaking by the Working Group on California Earthquake Probabilities (2015). That said, the County of Santa Clara (2021) has incorporated the Coyote Creek fault zone in their County Geologic Hazard Zonation program. The site is located approximately 600 feet southwest of the County Fault Rupture Hazard Zone boundary (i.e., outside the zone of County ordinance required investigation). None of the reference documents indicate the presence of active faulting on this site and therefore the potential for ground rupture is low to non-existent.

Future seismic events in this region can be expected to produce strong seismic ground shaking at this site during the lifetime of the proposed improvements. 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 SECONDARY 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 gravelly soils below the groundwater level, 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.

Liquefaction triggering analyses of borings were performed using the method proposed by Idriss and Boulanger (2008) utilizing the information obtained from the rotary wash boring advanced for the geotechnical investigation. Liquefaction triggering analyses of CPTs were performed using the liquefaction assessment software program CLiq (v.3.0.3.4 by GeoLogismiki) and the method proposed by Boulanger and Idriss (2014). The factor of safety against liquefaction triggering (e.g., cyclic resistance ratio to cyclic stress ratio) for design purposes considered a factor of safety of 1.0.

An initial site-specific GMHA, followed by a more precise SRA were completed for the current work consistent with the requirements of the 2019 California Building Code (CBC) which references ASCE 7-16. The SRA is presented in Appendix F of this report. Results of the site-specific analysis indicated that, in the absence of liquefaction, the free-field ground motion design peak ground acceleration may be as high as 0.74g with an associated moment magnitude of 6.89.

Generally only the upper 50 feet of soil layers are considered in calculating total liquefaction-induced settlements for building projects of this type, and the deeper soils are comprised of dense coarse-grained and stiff fine-grained soils that are less susceptible to liquefaction. Liquefaction-induced settlement estimates were calculated to essentially be nil in the depth range interval from the ground surface up to a depth of about 38 feet. Although our calculations show that soil layers encountered in some of the explorations at depths below 38 feet may liquefy with settlement estimates ranging from about ½ to 1 inch, we do not anticipate that the liquefaction of these deeper materials would significantly manifest settlement near the ground surface or adversely affect the building's shallow foundations or slabs. Similarly, we do not expect ground surface disruption such as liquefaction-related sand boils or ground oscillation to happen at this site.

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 significant open faces within 200 feet of the warehouse building site where lateral spreading could occur. Although Coyote Creek flows approximately 230 feet from the northwest corner of the parcel, the potential for lateral spreading to affect the building site, in our opinion, 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 soils encountered above the design groundwater depth of 20 feet were predominantly cohesive soils, the potential for shaking-related dynamic compaction, in our opinion, is low.

5 SITE AND SUBSURFACE CONDITIONS

5.1 SITE CONDITIONS

The site is located on two undeveloped parcels located at the west corner of the Silver Creek Valley Road and Fontanoso Way intersection. The ground surface is generally flat, dry, and covered by brush, with a few trees and shrubs near the center and the northern boundary of the site. Also near the center of the proposed warehouse are several wood logs from at least one former on-site mature tree.

5.2 SUBSURFACE CONDITIONS

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

5.2.1 Stratigraphy

The near-surface soils consist of medium stiff to hard lean clay to sandy lean clay. In exploratory borings and CPTs along the west perimeter and the northeast corner of the site of the site, sand and silty sand layers were encountered between depths of approximately 7 feet to 13 ½ feet. In explorations completed to a depth of approximately 44½ feet, thin silt layers were encountered between depths of about 37 and 42 feet.

5.2.2 Groundwater

Groundwater levels were encountered in one boring at approximately 23 feet below the ground surface and interpreted in the CPTs between 23 and 36 feet based on pore pressure dissipation tests. It is possible that groundwater conditions at the site could change due to variations in rainfall and runoff, construction activities, well pumping and irrigation, or other factors not apparent at the time the field investigation was performed. Based on the above groundwater levels, a design groundwater depth of 20 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.

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 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.

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, CBC seismic design considerations, site preparation for earthwork, foundation design, pavement design, and other construction considerations. These conclusions and recommendations are based on the data collected during our investigations and are subject to the limitations stated in this report.

6.2 SEISMIC DESIGN CONSIDERATIONS

6.2.1 Site Class and Seismic Design Parameters

Based on the site-specific SRA included in Appendix F of this report, the site may be classified as Site Class D. The general CBC ground motion parameters based on the site conditions, site coordinates, and the risk category are presented in Appendix F.

6.2.2 Seismic Settlement Due to Liquefaction

Ground surface and structural (foundation and floor) displacements due to ground movement from seismic liquefaction settlement hazards, as described in Section 4.4.1, have been considered for design and foundation type selection recommendations.

6.3 SITE PREPARATION

6.3.1 General

It is anticipated that site grading can be performed with conventional grading equipment and techniques. Following clearing and grubbing, site grading is anticipated to be minimal and to consist of minor fills to raise the building pad above the surrounding areas by up to approximately 1 to 2 feet, and for drainage and minor cuts of up to about 3 feet for the depressed loading dock areas along the edge of the building. 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 clearing and demolition process, existing slabs, foundations, and other improvements (if any) should be removed. Excavations from removal of slabs, foundations, underground utilities, or other below ground obstructions 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, tree roots, organic topsoil, debris, and any other deleterious materials to a minimum depth of approximately 4 to 5 inches prior to compaction (including proof-rolling) followed by placement of engineered fill to raise the grade. Any stripped organic materials or debris should not be reused as engineered fill.

Soft or loose areas may be encountered during construction that will require over-excavation and replacement as engineered fill (moisture conditioned and compacted). Unit prices for deeper over-excavation and replacement with engineered fill should be obtained during bidding.

Stripping and removals as well as over-excavation of loose soil should extend laterally a minimum of 5 feet beyond the building footprints, concrete flatwork, and any other facility improvements 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 foundations will need to be removed and relocated. Pipes between 2 inches and 6 inches in diameter may be left in place beyond the limits of the structure footprint 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 all planned improvement areas should be removed and backfilled. Active utilities to be reused should be carefully located and protected during demolition and construction.

6.3.4 Scarification and Compaction

In areas requiring placement of fill, it is recommended that the fill be placed and compacted as engineered fill. Following site stripping and performing any required grubbing and/or over-excavation, the area to receive engineered fill should be scarified to a depth of at least 8 inches, uniformly moisture conditioned to at least 2 percent above the optimum moisture content and compacted to at least 90 percent relative compaction. Fill areas 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.

For the upper 12 inches below finished subgrade in pavement areas and at the building pad, the scarified subgrade should be moisture conditioned to about 2 percent above the optimum moisture content and compacted to at least 95 percent relative compaction. The subgrade should not be allowed to dry out prior to the placement of engineered fill or aggregate base materials.

6.4 EARTHWORK – ENGINEERED FILL

Grading and earthwork recommendations presented in the following sections below include use of non-expansive fill, earthwork for slab-on-grade preparation as well as exterior slabs and flatwork, and a design value for modulus of subgrade reaction for use in slab-on-grade design.

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 the following table. The on-site near-surface soils are expected to be acceptable for use where "non-expansive" fill is required but may also be used outside of non-expansive fill areas or as a general fill. Any import material should be processed to meet the requirements presented in the following table.

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*

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 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.

In addition, we recommend that a laboratory corrosion test series (pH, resistivity, redox, sulfides, chlorides, and sulfates) be performed on all proposed import materials.

6.4.2 Fill Placement and Compaction Criteria

Fills composed of any 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 90 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 and for finished subgrade at the building pad, 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 SHALLOW FOUNDATION DESIGN INPUT

Spread footings and grade beams for the building should extend a minimum depth of 24 inches below the bottom of the floor slab for interior foundations or below adjacent finished grade for exterior footings. For interior and exterior continuous footings, a minimum width of 24 inches is recommended. Isolated interior and exterior footings should measure a minimum of 24 inches by 24 inches. The recommended allowable soil bearing pressure for engineering design purposes is 3,000 psf. Allowable soil bearing pressures may be increased by one-third for transient loads such as wind and seismic loads.

Total estimated static case settlement due to DL+LL of spread footings will vary depending on the plan dimensions of the foundation and the actual load supported. Based on anticipated foundation dimensions and loads, the estimated total static load case settlement of footings is expected to typically range from ½ inch to ¾ inch. For footings founded on similar subgrade materials, the estimated magnitude of differential settlements between adjacent isolated footings (spaced at approximately 60 feet apart) are expected to be up to one-half of the magnitudes provided for total settlement.

Where footings are located adjacent to below-grade structures or near major underground utilities, the footings should extend below a 1.5H:1V (horizontal to vertical) plane projected upward from the structure footing or bottom of the underground utility to avoid surcharging the below-grade structure and underground utility with building loads.

Resistance to lateral loads can also be provided by passive soil pressure against the foundations in the direction of loading, and by soil frictional resistance against the sides and bottoms of footings. For design purposes, the passive pressure should be calculated using equivalent fluid pressure value of 300 pounds per cubic foot (pcf). Friction along the sides and bottoms of shallow foundations may be used in combination with the passive resistance. The frictional resistance can be estimated by using a coefficient of friction of 0.35. The effective at-rest pressures normal to the sides of the structural elements should be used in estimating frictional resistance along the sides. We recommend using equivalent fluid weight of 60 pcf for the effective at-rest earth pressure in soils above the groundwater level.

The resistance from the upper 12 inches of footings should be neglected in lateral resistance calculations unless the adjacent soil surface is covered by a permanent pavement or floor slab. However, the pressure distribution for any case should be calculated from the soil surface. The friction coefficient and passive resistance may be used concurrently, and the passive resistance can be increased by one-third for wind and/or seismic loading.

6.6 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 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.

6.7 TEMPORARY EXCAVATIONS

6.7.1 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.

Based on the measured depths to groundwater levels during our investigation, we do not anticipate that excavations during the construction phase for structural foundations will encounter groundwater.

6.7.2 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 1.5H:1V plane projected downward from the bottoms of new footings to avoid undermining the footings during the excavation of utility trenches.

6.8 TRENCH BACKFILL

Trench backfill should be placed and compacted in accordance with recommendations provided in this report for engineered fill. Trench bedding and backfill should conform to specific utility agency and local city standards.

Utility trenches beneath the warehouse should be plugged with a low permeability cutoff collar to reduce moisture infiltration along the pipe/utility bedding beneath the building. Cutoff collars should be constructed of lean concrete or controlled density fill (low strength cementitious slurry) that is at least 12 inches thick. The collars should extend into the trench bottom and wall soils at least 18 inches. They should also extend at least 18 inches above the pipe, through any initial backfill above the pipe, and engage the overlying less permeable trench backfill soils.

6.9 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.
- Closed pipe drainage systems to an approved discharge point away from foundation soils.
- Defined drainage gradients of at least 2 percent 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.

6.10 CONCRETE SLAB ON GRADE AND EXTERIOR FLATWORK

Interior and exterior concrete slabs-on-grade should be constructed on soil subgrades prepared as recommended in this letter. Final grading to meet finished subgrade elevations beneath slab areas is expected to require some minor fills and cuts generally on the order of up to 1 to 3 feet thick. We recommend that soil subgrade material in the upper 12 inches below finished subgrade elevation meet the non-expansive engineered fill requirements listed above. A representative geotechnical staff member from Kleinfelder should be involved during rough and final grading to observe and test subgrade soils for material type suitability, as well as meeting compaction requirements.

Once the slab subgrade soil has been moisture conditioned and compacted to at least 95 percent relative compaction for the upper 8 inches below finished subgrade elevation, 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.

A modulus of subgrade reaction (k) value of 125 pounds per square inch per inch (psi) of settlement may be used for design of interior slabs supported on subgrades of engineered fill. We recommend that interior slabs be reinforced, and the structural engineer should design the slab thickness, reinforcing, and control joint spacing. From a geotechnical standpoint, we recommend that floor slabs be at least 6 inches thick; however, we understand that Duke's typical guidelines for design and construction of warehouse floors will likely require slabs thicker than 6 inches. Special care should be taken to place the bar reinforcement at mid-height within the slab.

Beneath the interior slabs, it has been common practice to place a capillary break consisting of at least 6 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 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. The edges of the vapor retarder membrane should be draped over the interior side of the footing excavations.

We are not moisture proofing experts and, as such, we recommend that a flooring expert be retained to design any required moisture proofing of slabs or walls. For some projects, a thin layer of clean sand (about 2 inches thick) is placed on the membrane to facilitate concrete curing and to decrease the likelihood of slab curling. The final decision for the need and thickness of sand above the vapor retarder is the purview of the slab designer/structural engineer. The vapor retarder is intended only to reduce water vapor transmission from the soil beneath the concrete and will not provide a waterproof or vapor proof barrier or reduce vapor transmission from sources above the retarder.

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

Various factors such as surface grades, adjacent planters, the quality of slab concrete (water/cement ratio) and the permeability of the on-site soils affect slab moisture and can influence performance. In many cases, floor moisture problems are the result of water/cement ratio, improper curing of floor slabs, improper application of flooring adhesives, or a combination of these factors. Studies have shown that concrete water/cement ratios lower than 0.5 and proper slab curing can significantly reduce the potential for vapor transmission through floor slabs. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding your proposed flooring applications. Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water/cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking or curl.

The exterior slab-on-grade recommendations that follow apply to slabs and flatwork that are not exposed to vehicular truck traffic. 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) value of 125 psi 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 be comprised of flexible Asphalt Concrete (AC) pavement sections for parking and driveway areas, and Portland Cement Concrete (PCC) for truck aprons at the loading dock ramp areas. Traffic Indices (TIs) of 5 through 8 were provided to us by the project Civil Engineer. The appropriate TI should be selected by the Civil Engineer. However, we generally use a TI of at least 5 in automobile parking stalls, 6 in automobile drive lanes, and 7 to 8 in entrance driveways and areas traversed by heavy trucks such as delivery and garbage trucks on an intermittent basis. Actual design TIs should be determined by the project Civil Engineer.

AC pavement design recommendations provided below assume that subgrades will be similar to the near-surface soils encountered in our borings.

6.11.2 Asphalt Concrete Pavement Sections

Based on Caltrans design methods and our laboratory R-value of 20 for the on-site surface soils, the recommended AC pavement sections for Traffic Indices between 5 and 8 are provided in the following table. Pavement section parameters include AC and Caltrans Class 2 aggregate base material (AB).

Table 6-2. Recommended Asphalt Concrete Pavement Sections (R-Value = 20)

Traffic Index (TI)	AC (inches)	AB (inches)
5	3.0	7.0
6	3.0	10.5
7	4.0	12.0
8	5.0	13.0

If a thinner pavement section is desired, lime treatment of the lean clay subgrade may be considered. Based on an assumed R-value of 50 for lime-treated subgrade soils (upper 12 inches), the recommended AC pavement sections, including AB, for Traffic Indices between 5 and 8 are provided in the following table.

Table 6-3. Preliminary* Asphalt Concrete Pavement Sections on Lime-Treated Subgrade Soil (Assumed R-Value = 50)

Traffic Index (TI)	AC (inches)	AB (inches)
5	2.5	3.0
6	3.0	4.0
7	4.0	5.0
8	5.0	5.0

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. The R-value for any new (imported) fill placed to reach finished subgrade elevation should meet or exceed a value of 5 for untreated subgrade soils, or 50 for lime treated subgrade soils.
- Subgrade soils are in a stable, "non-pumping" condition at the time the AB materials are placed and compacted.
- AB materials are compacted to at least 95 percent relative compaction.
- Adequate drainage (both surface and subsurface) is provided such that the subgrade soils and AB materials are not allowed to become saturated.
- AB materials meet current Caltrans specifications for Class 2 aggregate base.
- Asphalt paving materials and placement methods meet current Caltrans specifications for asphalt concrete.
- Concrete curbs adjacent to pavement sections should extend to the subgrade soil to prevent landscape and infiltrating water from migrating into the aggregate base section.

6.11.3 Portland Cement Concrete (PCC) Pavement

Recommendations for PCC pavement at truck loading bay apron areas and for heavy truck traffic areas assume that the upper 12 inches of subgrade will consist of compacted native lean clay soil or non-expansive fill with a minimum R-value of 20. For these types of heavy-duty pavements with an assumed Traffic Index of 8, we recommend the PCC pavement section provided in the following table be used.

Table 6-4. Recommended Portland Cement Concrete Pavement Section (R-Value = 20)

Traffic Index (TI)	PCC (inches)	AB (inches)
8	8	10

If a thinner pavement section is desired for heavy-duty pavements, lime treatment of the subgrade may be considered. Based on an assumed R-value of 50 for lime-treated subgrade soils (upper

12 inches), the recommended PCC pavement section, including AB, for an assumed Traffic Index of 8 is provided in the following table.

Table 6-5. Preliminary* Portland Cement Concrete Pavement Section on Lime-Treated Subgrade Soil (Assumed R-Value = 50)

Traffic Index (TI)	PCC (inches)	AB (inches)
8	8	6.0

* The preliminary AC and PCC pavement sections provided above are contingent on future laboratory testing of the subgrade soils blended with lime for assessment of the actual R-value of lime-treated native soil. These preliminary pavement section recommendations for AC and/or PCC sections may need to be adjusted following the results of actual laboratory testing.

6.11.4 Aggregate Base

Aggregate base materials underlying all pavement should meet current Caltrans specifications for Class 2 aggregate base and be compacted to at least 95 percent relative compaction (ASTM D1557).

6.12 LOW RETAINING WALL STRUCTURES AT LOADING DOCKS

Design earth pressures for low retaining wall structures (up to 6 feet in height) at the loading docks depend primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharges, and drainage. Earth pressures provided assume that granular (sandy) soils will used as backfill. Due to the potential for soil with some expansion potential, the on-site clay soil should not be used as retaining wall backfill. Granular backfill, which meets the requirements for imported fill as defined in Section 6.4.1, should extend behind walls a horizontal distance of at least one-half the height of the wall. If a drainage system is not installed, the wall should be designed to resist hydrostatic pressure in addition to the earth pressure.

Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the walls. Walls that are free to rotate at least 0.002 radians (deflection at the top of the wall of at least 0.002H, where H is the unbalanced wall height) may be designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The recommended active and at-rest earth pressures and passive resistance values are provided in the following table.

Table 6-6. Lateral Earth Pressures for Low Earth Retaining Structures (Using Imported Granular Backfill)

Wall Movement	Backfill Condition	Equivalent Fluid Pressure (pcf)	Seismic Increment (pcf)
Free to Deflect (active condition)	Level	40	22H*
Restrained (at-rest condition)		62	N/A**

Notes: * Walls supporting more than 6 feet of backfill should be designed to support an incremental seismic lateral pressure, which is applied as a triangular pressure distribution with a maximum pressure at the bottom of the wall, not inverted, and H is the height of the wall.
** For restrained walls, use the static active earth pressure and seismic increment to check the seismic condition and use at-rest earth pressure only to check the static condition; the larger loading of both cases should be used for the design of restrained walls.

Walls supporting more than 6 feet of backfill should be designed to support an incremental seismic lateral pressure noted in the above table applied using regular (not inverted) triangular distribution, where H is the wall height H (in feet). The seismic lateral earth pressure was evaluated based on a PGA value corresponding to one-half of the Design Earthquake PGA, which is two-thirds of PGA_M. When designing for seismic loads of walls below grade or restrained walls retaining more than 6 feet of backfill, the seismic lateral earth pressure should be combined with the active earth pressure (not the at-rest pressure).

The above lateral earth pressures do not include the effects of surcharges (e.g., traffic, footings), compaction, or truck-induced wall pressures. Any surcharge (live load, including traffic, or dead load) located within a 1H:1V plane drawn upward from the base of the excavation should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind walls may be calculated by multiplying the surcharge by 0.33 for cantilevered walls under active conditions, and 0.50 for restrained walls under at-rest conditions. Walls adjacent to areas subject to vehicular traffic should be designed for a 2-foot equivalent soil surcharge (250 psf). Lateral load contributions from other surcharges located behind walls may be provided once the load configurations and layouts are known.

Walls should be properly drained or designed to resist hydrostatic pressures. Adequate drainage is essential to provide a free-drained backfill condition so that there is no hydrostatic buildup behind the wall. Walls should also be appropriately waterproofed to reduce the potential for staining. Drainage behind loading dock walls can consist of weepholes placed along the base of the wall. Weepholes should be spaced 10 to 15 feet apart and connected with a gravel drain consisting of approximately 3 cubic feet of clean gravel per foot of wall length wrapped with filter fabric. Other types of retaining walls should have a continuous back drain as described below.

Except for the upper 2 feet, the backfill immediately behind retaining walls (minimum horizontal distance of 2 feet measured perpendicular to the wall) should consist of free-draining, ¾-inch crushed rock wrapped with filter fabric. The upper 2 feet of cover backfill should consist of relatively impervious material. A 4-inch-diameter perforated PVC pipe, with perforations placed facing down at the bottom of the rock layer leading to a suitable gravity outlet, should be installed at the base of the walls.

As an alternative to the gravel drain noted above, a manufactured drain panel may be utilized behind retaining walls in addition to normal waterproofing. This system generally consists of a prefabricated drain panel lined with filter fabric. At the wall base, we recommend that a gravel drain be installed to collect and discharge drainage to a suitable outlet. The drain should consist of a 4-inch-diameter perforated PVC pipe, placed perforations down at the bottom of approximately 3 cubic feet of clean gravel per foot of wall length. The gravel drain should be wrapped in filter fabric (Mirafi 140N or equivalent). The pipe should be sloped to drain to a suitable outlet and cleanouts should be provided at appropriate intervals. If drainage behind the wall is omitted, the wall should be designed for full hydrostatic pressure. The design of any drain panel system should be submitted to Kleinfelder for review to check that our recommendations have been properly incorporated into the design. Installation of the drainage system should be reviewed and documented by a Kleinfelder representative.

6.13 CORROSIVITY

Metal and concrete elements in contact with soil, whether part of a foundation or part of the supported structure, might be subject to degradation due to corrosion or chemical attack if the soils are deemed to be corrosive. Therefore, buried ferrous metal and concrete elements should be designed to resist corrosion and degradation based on accepted practices. As part of Kleinfelder's geotechnical field exploration (boring) program, we submitted one soil sample for laboratory screening testing to provide initial data regarding corrosivity potential of on-site soils. Laboratory chloride concentration, sulfate concentration, pH, and electrical resistivity tests were performed on the near-surface soil sample. A brief evaluation and the results of the tests conducted by CERCO are presented in Appendix D and summarized in the following table. CERCO also prepared a brief technical letter describing the general corrosivity potential of the soil material based on the lab test results.

Table 6-7. Initial Corrosion Potential Screening Test Results

Boring	Sample Depth (feet)	Soil Description	Resistivity (ohm-cm)	pH	Water-Soluble Ion Concentration (ppm)	
			Saturated		Chloride	Sulfate
B-4	0 to 5	Lean Clay with Gravel (CL)	2,600	8.10	N.D.	65

N.D. - None Detected

If fill materials will be imported to the project site, similar corrosion potential laboratory testing should be completed on the imported material.

At the request of Duke, JDH Corrosion Consultants, Inc. (JDH) of Concord, California, was retained to perform a site-specific evaluation of soil corrosion potential for the planned underground project infrastructure features. The purpose of JDH's study also included development of site-specific recommendations for long-term corrosion control for protection of buried features. JDH's work included on-site measurement of in-situ soil resistivities at selected locations throughout the project area using the Wenner 4-pin technique. In-situ soil resistivities were measured to a depth of about 15 feet, and analyses were done to determine the variation of soil resistivity versus soil depth. JDH's summary report including findings, analysis of data, conclusions and recommendations for corrosion control is attached in Appendix E. All fieldwork and recommendations by JDH are in general accordance with the National Association of Corrosion Engineers (NACE).

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.
- Observe foundation installation operations, and foundation bearing soils to confirm conditions are as anticipated.
- Provide Special Inspection observation and testing during construction

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 and Kleinfelder will cease to be the Geotechnical-Engineer-of-Record.

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 Duke Realty, 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, and inspection services, 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, the 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.

9 REFERENCES

American Society of Civil Engineers (2017), Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-16), American Society of Civil Engineers.

Boulanger, R. W., and Idriss, I. M. (2014). "CPT and SPT Based Liquefaction Triggering Procedures." Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.

California Building Code (2019). California Code of Regulations, Title 24, Part 2, Volume 2, International Code Council.

California Geological Survey, 2000, Seismic Hazard Zone for the San Jose East 7.5-Minute Quadrangle, Santa Clara County, California, CGS Seismic Hazard Zone Report 044.

California Geological Survey, Regulatory Maps Portal, Maps indicating Earthquake Zones of Required Investigation, url: http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps, accessed August 20, 2021.

GeoLogismiki (2020). CLiq v.3.0.3.4 – CPT Liquefaction Assessment Software.

County of Santa Clara, Geologic Hazard Zone Maps Portal, accessed August 20, 2021, url: https://www.sccgov.org/sites/dpd/OrdinancesCodes/GeoHazards/Pages/GeoMaps.aspx.

Graymer, R.W., Moring, B.C., Saucedo, G.J., Wentworth, C.M., Brabb, E.E., Knudsen, K.L. (2006). Geologic map of the San Francisco Bay region, U.S. Geological Survey, Scientific Investigations Map SIM-2918, scale 1:275,000.

Idriss, I.M. and Boulanger, R.W. (2008), Soil Liquefaction During Earthquakes, Engineering Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, CA, 261 pp.

Knudsen, K.L., Sowers, J.M., Witter, R.C., Wentworth, C.M., Helley, E.J., Nicholson, R.S., Wright, H.M., and Brown, K.H. (2000). Preliminary maps of Quaternary deposits and liquefaction susceptibility, nine-county San Francisco Bay region, California: a digital database, U.S. Geological Survey, Open-File Report OF-2000-444, scale 1:275,000.

U.S. Geological Survey, Quaternary Fault and Fold Database for the United States, url: https://www.usgs.gov/natural-hazards/earthquake-hazards/faults, accessed August 20, 2021.

U.S. Geological Survey (2014), USGS Earthquake Scenario Map (BSSC 2014), url: https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=14d2f75c7c4f4619936dac0d14e1e468, accessed February 8, 2021.

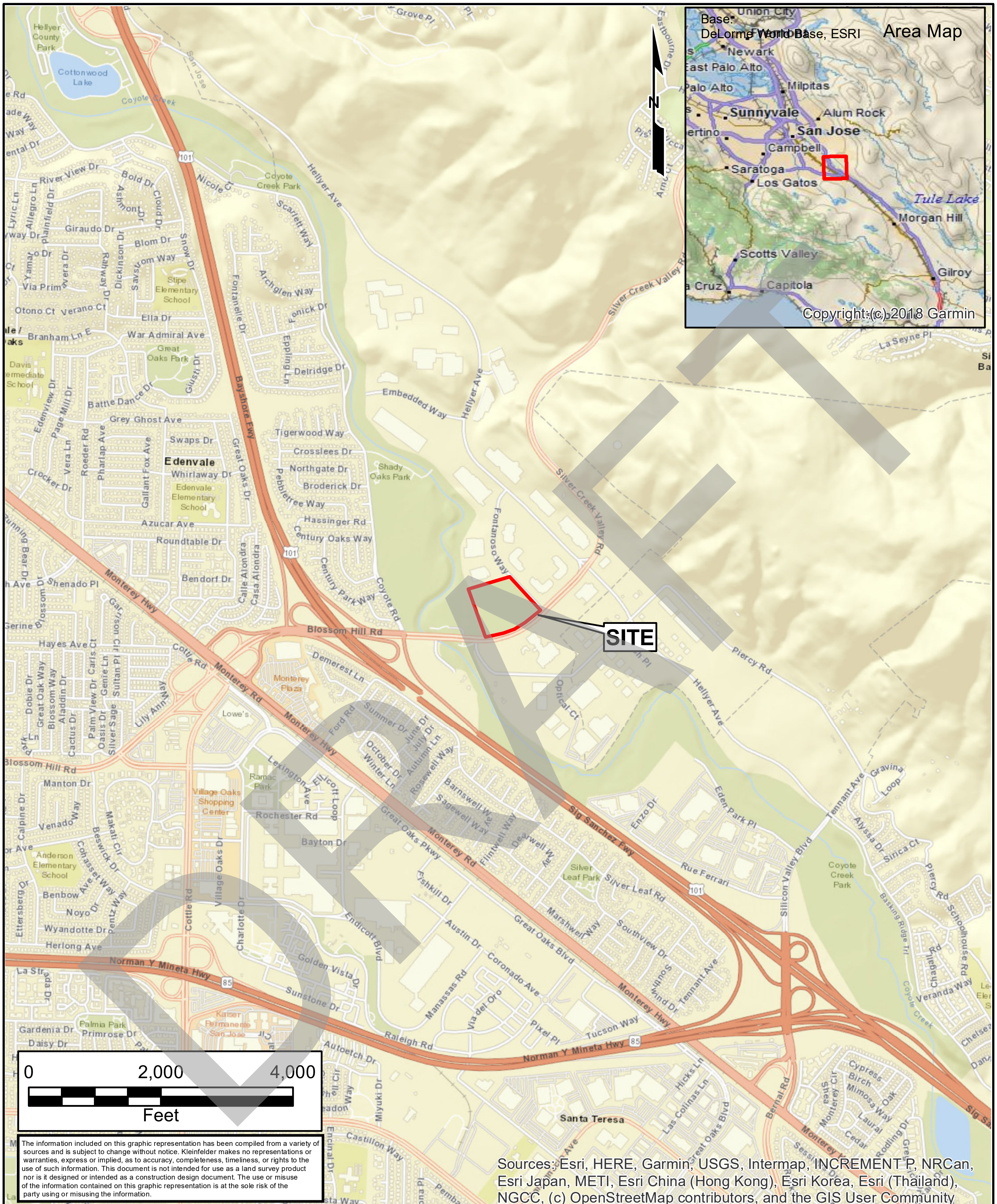
University of California Pavement Research Center (UCPRC) (2020). CalFP-Web, CalME: Caltrans Mechanistic-Empirical Tool, Version: 3.DD001.3, url: http://www.ucprc.ucdavis.edu/calme/, accessed October 19, 2021.

Wentworth, C.M., Blake, M.C., McLaughlin, R.J., and Graymer, R.W., 1999, Preliminary geologic map of the San Jose 30 X 60-minute quadrangle, California: a digital database, U.S. Geological Survey, Open-File Report OF-98-795, 1:100,000.

Witter, R.C., Knudsen, K.L., Sowers, J.M., Wentworth, C.M., Koehler, R.D., Randolph, C.E., Brooks, S, K., and Gans, K.D., 2006, Maps of Quaternary deposits and liquefaction susceptibility in the central San Francisco Bay region, California, U.S. Geological Survey, Open-File Report OF-2006-1037, 1:200,000.

Bright People. Right Solutions.

Figures



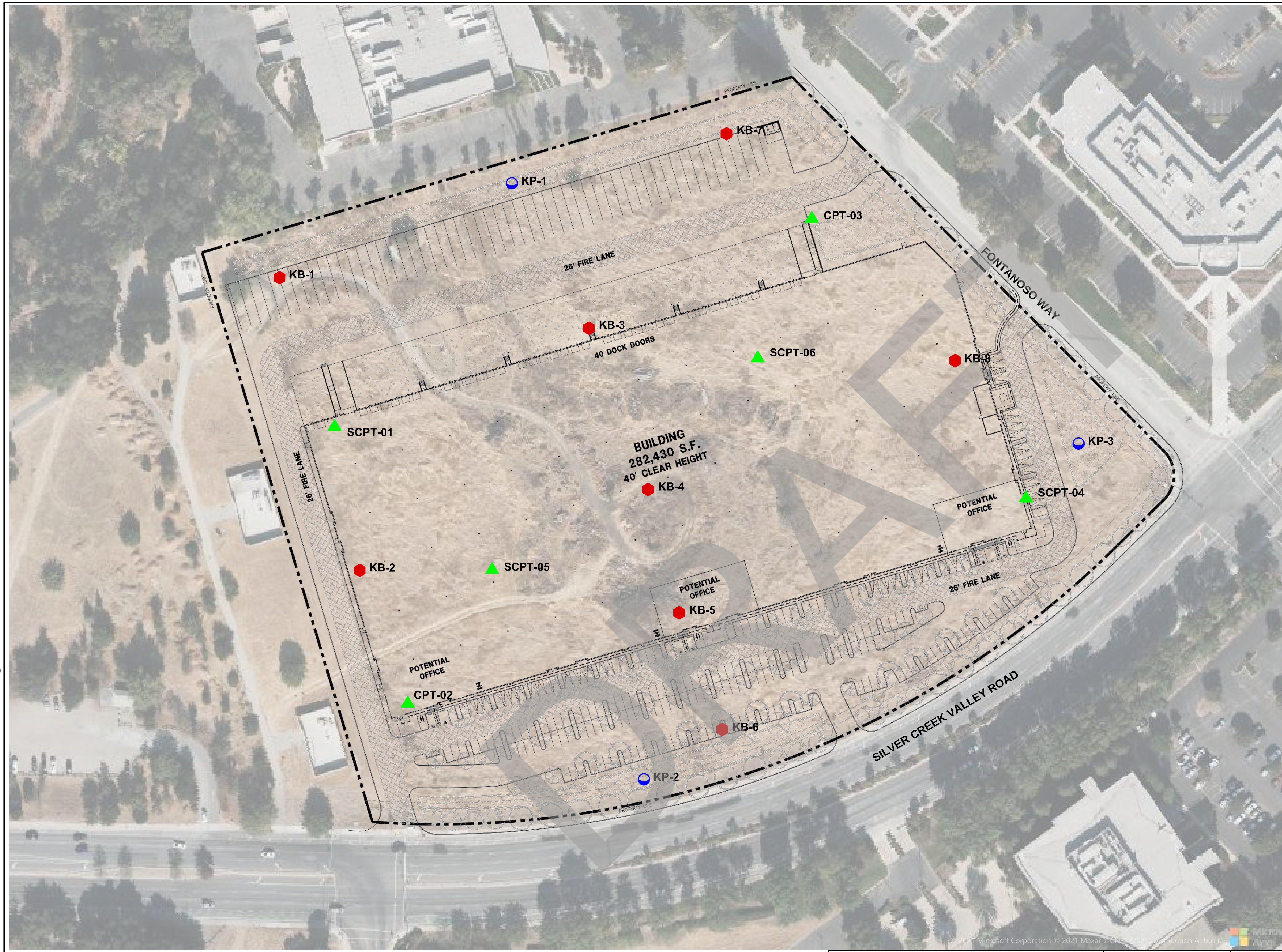
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.	20221404
DRAWN BY:	JDS
CHECKED BY:	RY
DATE:	12-09-2021
REVISED:	

SITE VICINITY MAP

DUKE WAREHOUSE SILVER CREEK
5977 & 6001 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA

FIGURE
1



LEGEND

SOIL BORING

CONE PENETRATION TEST

PERCOLATION TEST

NOTE:
BORING AND PERCOLATION TEST LOCATIONS WERE NOT SURVEYED AND SHOULD BE CONSIDERED APPROXIMATE.

N

0 120 240

SCALE: 1" = 120' SCALE IN FEET

REFERENCE:
SHEET DAB-A1.1, OVERALL SITE PLAN, 5977-6001 SILVER CREEK VALLEY ROAD, SAN JOSE, CA, PROJECT NUMBER: 21367, DATED OCTOBER 1, 2021, BY HPA ARCHITECTURE.

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.	20221404
DRAWN BY	JDS
CHECKED BY	RY
DATE:	02/02/2022
REVISED:	

SITE MAP

DUKE WAREHOUSE SILVER CREEK
5977 & 6001 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA

FIGURE
2

KLEINFELDER
Bright People. Right Solutions.

Appendix A

SAMPLE/SAMPLER TYPE GRAPHICS

- BULK SAMPLE
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)

GROUND WATER GRAPHICS

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

NOTES

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

ABBREVIATIONS

WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

Major Division	Subdivision	Fines	Criteria	Symbol	Description
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	$Cu \geq 4$ and $1 \leq Cc \leq 3$	**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	$Cu \geq 4$ and $1 \leq Cc \leq 3$	**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	$Cu \geq 6$ and $1 \leq Cc \leq 3$	**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	$Cu \geq 6$ and $1 \leq Cc \leq 3$	**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

Major Division	Subdivision	Symbol	Description
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

NOTE: USE MATERIAL DESCRIPTION ON THE LOG TO DEFINE A GRAPHIC THAT MAY NOT BE PROVIDED ON THIS LEGEND.

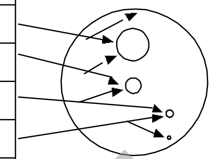
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20221404.001A	GRAPHICS KEY		FIGURE
DRAWN BY: DJS	DUKE WAREHOUSE SILVER CREEK 5977 & 6001 SILVER CREEK ROAD SAN JOSE, CALIFORNIA		A-1
CHECKED BY: RY			
DATE: 12/8/2021			

PLOTTED: 12/08/2021 11:52 AM BY: JSala
OFFICE FILTER: SANTA ROSA
PROJECT NUMBER: 20221404.001A
gINT FILE: Klf_gint_master_2022
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2022.GLB [_KLF_GEO-LEG1 (GRAPHICS KEY) WITH USCS]

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

Term of Use	AMOUNT: Secondary Constituent is Fine Grained	AMOUNT: 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.

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	PI
Non-Plastic	NP	NP
Low	< 30	< 15
Medium	30 - 50	15 - 25
High	> 50	> 25

Either the LL or the PI (or both) may be used to describe the soil plasticity. The ranges of numbers shown here do not imply that the LL ranges correlate with the PI ranges for all soils.

LL is from Casagrande, 1948. PI is from Holtz , 1959.

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.: 20221404.001A	SOIL DESCRIPTION KEY
DRAWN BY: DJS	DUKE WAREHOUSE SILVER CREEK 5977 & 6001 SILVER CREEK ROAD SAN JOSE, CALIFORNIA
CHECKED BY: RY	
DATE: 12/8/2021	

FIGURE A-2

PLOTTED: 12/08/2021 11:53 AM BY: JSala
OFFICE FILTER: SANTA ROSA
PROJECT NUMBER: 20221404.001A
gINT FILE: Klf_gint_master_2022
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2022.GLB [KLF_GEO-LEG 2 (SOIL DESC KEY)]

Date Begin - End:	7/29/2021	**Drilling Co.-Lic.#:**	Exploration Geoservices Inc. - #484288		**BORING LOG KB-1**
Logged By:	D. Sullivan	**Drill Crew:**	Kyle, Abraham		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	B-53R	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	79.1%
Weather:	Sunny, Warm	**Exploration Diameter:**	7 in. O.D.	**Hammer Cal. Date:**	8/08/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.25913°
Longitude: -121.78974°
Approximate Ground Surface Elevation (ft.): 204.00
Surface Condition: Grass

Approximate Elevation (feet)	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
	0		**Lean CLAY with Sand (CL)**: dark brown, moist, medium stiff to very stiff, fine sand												
				1B 1C		BC=6 BC=4 BC=3 PP=3.0	12"		17.2	103.7					
200															
	5		**Sandy Lean CLAY (CL)**: brown, moist, medium stiff to hard, fine to medium sand												
				2B 2C		BC=3 BC=3 BC=5 PP=4.5	12"								
			Poorly Graded SAND (SP): brown, moist, loose, fine to medium sand												
				3B 3C		BC=3 BC=4 BC=6	12"								
195															
	10		mostly medium sand	4B 4C		BC=4 BC=5 BC=7	9"								
190															
	15														
185															

The boring was terminated at approximately 11.5 ft. below ground surface. The boring was backfilled with cement grout on July 29, 2021.

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

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

DRAFT

BORING LOG KB-2

Date Begin - End:	7/30/2021	**Drilling Co.-Lic.#:**	Exploration Geoservices Inc. - #484288		
Logged By:	D. Sullivan	**Drill Crew:**	Loren, Lyle		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	B-53B	**Hammer Type - Drop:**	140 lb. Wireline - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	61.0%
Weather:	Sunny, Warm	**Exploration Diameter:**	7 in. O.D.	**Hammer Cal. Date:**	8/08/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.25817°
Longitude: -121.78940°
Approximate Ground Surface Elevation (ft.): 204.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
204	0	**Lean CLAY (CL)**: dark brown, moist, stiff to hard											
200		brown, hard	1B 1C	BC=7 BC=8 BC=8 PP=4.5	12"		12.1	92.7					
	5		2B 2C	BC=7 BC=6 BC=9	11"								
		Silty SAND (SM): brown, moist, loose to medium dense, fine to medium sand	3B 3C	BC=6 BC=7 BC=8	13"					37			
195	10		4B 4C	BC=5 BC=6 BC=7	15"								
190		**Sandy Lean CLAY (CL)**: dark brown, moist, very stiff, fine to medium sand											
	15		5B 5C	BC=6 BC=9 BC=12 PP=3.5	18"		17.3	106.9					
185		**Lean CLAY (CL)**: mottled brown and brownish gray, moist, very stiff to hard											
	20		6B 6C	BC=9 BC=16 BC=20 PP=4.0	15"								
180		**Lean CLAY with Sand (CL)**: mottled brown and brownish gray, moist, stiff, fine to medium sand											
	25		7A 7B 7C	BC=5 BC=7 BC=8 PP=1.5	18"		25.9	99.5		70			
175		The boring was terminated at approximately 26.5 ft. below ground surface. The boring was backfilled with cement grout on July 30, 2021.											

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

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

BORING LOG KB-3

Date Begin - End:	7/29/2021	**Drilling Co.-Lic.#:**	Exploration Geoservices Inc. - #484288
Logged By:	D. Sullivan	**Drill Crew:**	Kyle, Abraham
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	B-53R
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Sunny, Warm	**Exploration Diameter:**	7 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.
Hammer Efficiency: 79.1%
Hammer Cal. Date: 8/08/2020

FIELD EXPLORATION — LABORATORY RESULTS

Latitude: 37.25897°
Longitude: -121.78847°
Approximate Ground Surface Elevation (ft.): 203.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	0	**Lean CLAY (CL)**: dark brown, moist, very stiff to hard												
200			1B 1C		BC=5 BC=7 BC=12 PP=4.5+	8"		12.1	99.2			34	14	
	5		2B 2C		BC=3 BC=7 BC=11 PP=4.5+	10"								
		Sandy Lean CLAY (CL): brown, moist, very stiff to hard, fine sand												
195			3B 3C		BC=5 BC=7 BC=10 PP=4.5+	9"								
	10	**Lean CLAY with Sand (CL)**: light brown to brown, moist, stiff, fine sand	4B 4C		BC=4 BC=5 BC=6	11"					83			
190		very stiff	5B 5C		BC=5 BC=10 BC=13 PP=3.0	10"		18.7	110.1					
	15													
185														

The boring was terminated at approximately 15 ft. below ground surface. The boring was backfilled with cement grout on July 29, 2021.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20221404.001A		BORING LOG KB-3	FIGURE
DRAWN BY:	DJS	DUKE WAREHOUSE SILVER CREEK 5977 & 6001 SILVER CREEK ROAD SAN JOSE, CALIFORNIA	A-5
CHECKED BY:	RY		
DATE:	12/8/2021		PAGE: 1 of 1

Date Begin - End:	7/30/2021	**Drilling Co.-Lic.#:**	Exploration Geoservices Inc. - #484288		**BORING LOG KB-4**
Logged By:	D. Sullivan	**Drill Crew:**	Loren, Lyle		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	B-53B	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Mud Rotary	**Hammer Efficiency:**	61.0%
Weather:	Sunny, Warm	**Exploration Diameter:**	4 in. O.D.	**Hammer Cal. Date:**	8/08/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.25844°
Longitude: -121.78822°
Approximate Ground Surface Elevation (ft.): 203.00
Surface Condition: Grass

Approximate Elevation (feet)	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
	0		**Lean CLAY with Gravel (CL)**: dark brown, brownish black, moist, stiff to very stiff, fine to coarse grained sand, subangular to subrounded gravel to 1"	1											
200				2B 2C		BC=3 BC=4 BC=4 PP=2.0	9"								
	5			3B 3C		BC=7 BC=10 BC=8 PP=4.5	11"								
195			**Lean CLAY with Sand (CL)**: dark brown, moist, very stiff, fine to medium sand	4B 4C		BC=4 BC=4 BC=5 PP=3.0	9"		18.0	111.2					
	10		medium stiff	5B 5C		BC=2 BC=3 BC=3 PP=1.0	10"		22.0	102.5			29	11	TXUU: c = 0.65 ksf
190			brown to dark brown, moist, stiff to very stiff, fine sand												
	15			6B 6C		BC=4 BC=5 BC=6 PP=2.5	12"								
185			mottled brown and brownish gray, moist, stiff, fine to coarse sand												
	20			7B 7C		BC=4 BC=6 BC=9 PP=2.0	14"	CL	21.0	106.2		81	32	13	
180 ∇			**Sandy Lean CLAY (CL)**: mottled brown and grayish brown, black, wet, soft, fine to medium sand												
	25			8B 8C		BC=2 BC=2 BC=3 PP=0.5	14"		28.5	96.8			31	11	TXUU: c = 0.40 ksf
175															

Date Begin - End:	7/30/2021	**Drilling Co.-Lic.#:**	Exploration Geoservices Inc. - #484288		**BORING LOG KB-4**
Logged By:	D. Sullivan	**Drill Crew:**	Loren, Lyle		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	B-53B	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Mud Rotary	**Hammer Efficiency:**	61.0%
Weather:	Sunny, Warm	**Exploration Diameter:**	4 in. O.D.	**Hammer Cal. Date:**	8/08/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.25844°
Longitude: -121.78822°
Approximate Ground Surface Elevation (ft.): 203.00
Surface Condition: Grass

Approximate Elevation (feet)	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): wet, medium stiff, trace rootlets	9B 9C		BC=Push BC=5 BC=5 PP=1.0	14"								
170															
	35		moist	10B 10C		BC=Push BC=4 BC=5 PP=1.0	13"		29.8	95.3		90			
165			**Sandy SILT (ML)**: brown, wet, stiff, fine to medium sand												
	40			11B 11C		BC=4 BC=5 BC=7	14"	ML	25.6	102.0		68	NP	NP	
160			**Lean CLAY (CL)**: mottled brown and gray, wet, stiff	12B 12C		BC=4 BC=6 BC=7 PP=2.0	10"								
	45														
155															
	50														
150															
	55														
145															

The boring was terminated at approximately 44.5 ft. below ground surface. The boring was backfilled with cement grout on July 30, 2021.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 23 ft. below ground surface during drilling.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Date Begin - End:	7/29/2021	**Drilling Co.-Lic.#:**	Exploration Geoservices Inc. - #484288
Logged By:	D. Sullivan	**Drill Crew:**	Kyle, Abraham
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	B-53R
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Sunny, Warm	**Exploration Diameter:**	7 in. O.D.

BORING LOG KB-5

Hammer Type - Drop:	140 lb. Auto - 30 in.
Hammer Efficiency:	79.1%
Hammer Cal. Date:	8/08/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.25803°
Longitude: -121.78809°
Approximate Ground Surface Elevation (ft.): 203.00
Surface Condition: Grass

Approximate Elevation (feet)	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): brown to dark brown, moist, very stiff to hard, occasional subrounded gravel to 3/4"	1											
200				2B 2C		BC=5 BC=8 BC=10 PP=4.5+	12"								
	5		occasional fine sand	3B 3C		BC=7 BC=9 BC=12 PP=4.5+	9"								
195			**Lean CLAY with Sand (CL)**: brown, moist, very stiff to hard, fine sand	4B 4C		BC=5 BC=8 BC=13 PP=4.5+	7"								
	10		**Sandy Lean CLAY (CL)**: brown to dark brown, moist, very stiff to hard	5B 5C		BC=4 BC=7 BC=10 PP=4.5+	9"		13.8	111.1			26	10	TXUU: c = 4.13 ksf
190			occasional fine to medium sand	6B 6C		BC=5 BC=11 BC=15 PP=4.0	12"								
	15														
185															

The boring was terminated at approximately 15 ft. below ground surface. The boring was backfilled with cement grout on July 29, 2021.

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

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20221404.001A	BORING LOG KB-5	FIGURE
DRAWN BY: DJS	DUKE WAREHOUSE SILVER CREEK 5977 & 6001 SILVER CREEK ROAD SAN JOSE, CALIFORNIA	A-7
CHECKED BY: RY		
DATE: 12/8/2021		PAGE: 1 of 1

BORING LOG KB-6

Date Begin - End:	7/29/2021	**Drilling Co.-Lic.#:**	Exploration Geoservices Inc. - #484288		
Logged By:	D. Sullivan	**Drill Crew:**	Kyle, Abraham		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	B-53R	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	79.1%
Weather:	Sunny, Warm	**Exploration Diameter:**	7 in. O.D.	**Hammer Cal. Date:**	8/08/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.25765°
Longitude: -121.78791°
Approximate Ground Surface Elevation (ft.): 204.00
Surface Condition: Grass

Approximate Elevation (feet)	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): brown and dark brown, dry to moist, very stiff to hard												
				1B 1C		BC=5 BC=7 BC=11 PP=4.5+	9"								
200	5														
			dark brown, moist	2B 2C		BC=7 BC=6 BC=10 PP=4.5+	10"								
			Lean CLAY with Sand (CL): dark brown, moist, very stiff to hard, fine sand												
				3B 3C		BC=5 BC=11 BC=13 PP=4.5+	11"								
195	10														
				4B 4C		BC=5 BC=12 BC=17 PP=4.5	12"								
190	15														
185															

The boring was terminated at approximately 11.5 ft. below ground surface. The boring was backfilled with cement grout on July 29, 2021.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Date Begin - End:	7/29/2021	**Drilling Co.-Lic.#:**	Exploration Geoservices Inc. - #484288		**BORING LOG KB-7**
Logged By:	D. Sullivan	**Drill Crew:**	Kyle, Abraham		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	B-53R	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	79.1%
Weather:	Sunny, Warm	**Exploration Diameter:**	7 in. O.D.	**Hammer Cal. Date:**	8/08/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.25960°
Longitude: -121.78793°
Approximate Ground Surface Elevation (ft.): 203.00
Surface Condition: Grass

Approximate Elevation (feet)	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): brown and dark brown, moist, stiff to hard, trace rootlets												
200				1B 1C		BC=7 BC=13 BC=17 PP=4.5+	8"		12.4	107.5					
	5		occasional fine sand	2B 2C		BC=5 BC=7 BC=8	7"								
195				3B 3C		BC=5 BC=6 BC=7	15"								
			Silty SAND (SM): brown, moist, loose, fine to medium sand												
	10		**Poorly Graded SAND with Silt (SP-SM)**: brown, moist, loose, fine to medium sand	4B 4C		BC=4 BC=5 BC=5	12"								
190															
	15														
185															

The boring was terminated at approximately 11.5 ft. below ground surface. The boring was backfilled with cement grout on July 29, 2021.

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

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20221404.001A	BORING LOG KB-7	FIGURE
	DRAWN BY: DJS	DUKE WAREHOUSE SILVER CREEK 5977 & 6001 SILVER CREEK ROAD SAN JOSE, CALIFORNIA	A-9
	CHECKED BY: RY		
	DATE: 12/8/2021		PAGE: 1 of 1

Date Begin - End:	7/29/2021	Drilling Co.-Lic.#:	Exploration Geoservices Inc. - #484288	BORING LOG KB-8	
Logged By:	D. Sullivan	Drill Crew:	Kyle. Abraham		
Hor.-Vert. Datum:	WGS84	Drilling Equipment:	B-53R	Hammer Type - Drop:	140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger	Hammer Efficiency:	79.1%
Weather:	Sunny, Warm	Exploration Diameter:	7 in. O.D.	Hammer Cal. Date:	8/08/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.25884°
Longitude: -121.78703°
Approximate Ground Surface Elevation (ft.): 202.00
Surface Condition: Grass

Approximate Elevation (feet)	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
	0		**Lean CLAY (CL)**: brown and dark brown, moist, very stiff to hard												
200				1B 1C		BC=5 BC=7 BC=10 PP=4.5+	9"								
195	5		dark brown	2B 2C		BC=5 BC=7 BC=10 PP=4.5+	6"	CL	16.8	106.5		95	34	15	
				3B 3C		BC=4 BC=6 BC=11 PP=4.5+	7"								
190	10		trace fine sand	4B 4C		BC=5 BC=7 BC=12 PP=4.5+	6"								
			brown and dark brown, moist, very stiff to hard, trace fine to coarse sand												
185	15			5B 5C		BC=5 BC=10 BC=16 PP=4.5+	10"								
180	20		becomes mottled brown and brownish gray	6B 6C		BC=5 BC=11 BC=17	12"		19.3	110.9					
	25		stiff, trace subrounded gravel to 1/4"	7A 7B 7C		BC=4 BC=5 BC=7 PP=1.5	18"		23.0	105.2					
175			The boring was terminated at approximately 25 ft. below ground surface. The boring was backfilled with cement grout on July 29, 2021.												

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

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

KLEINFELDER

Bright People. Right Solutions.

Appendix B

PRESENTATION OF SITE INVESTIGATION RESULTS

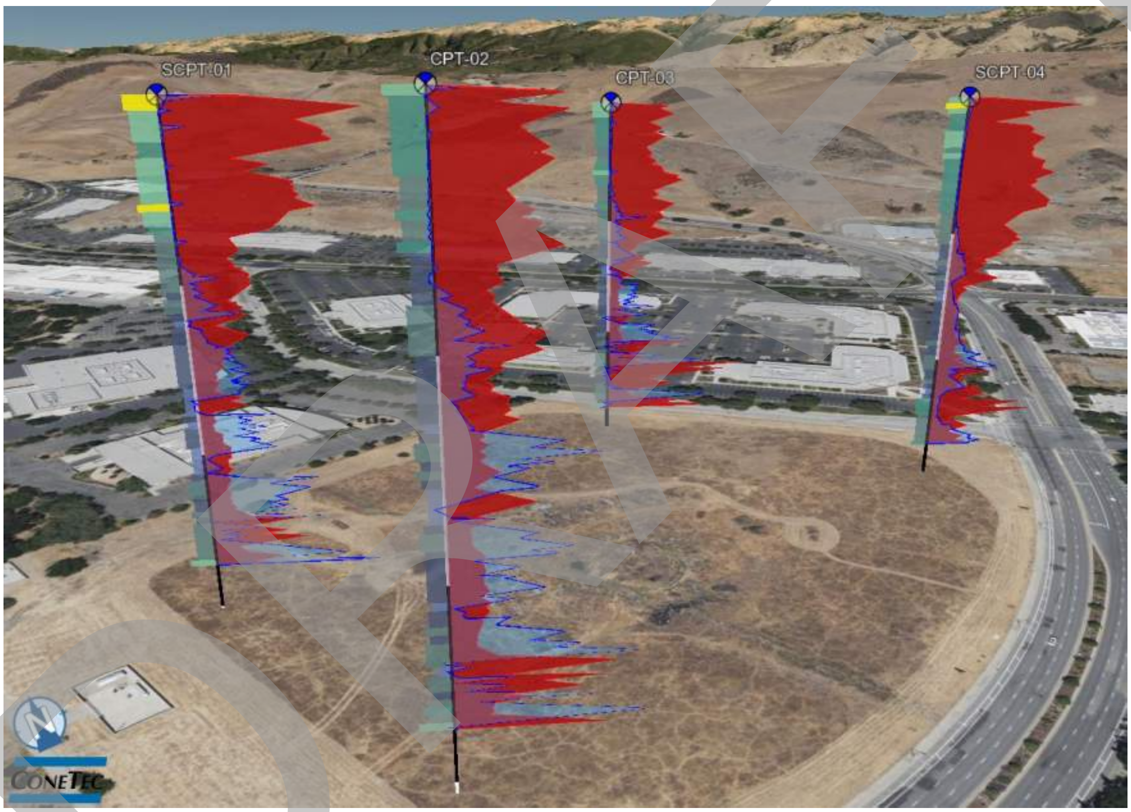
Duke Warehouse Silver Creek

Prepared for:

Kleinfelder

ConeTec Job No: 21-56-22781

Project Start Date: 29-Jul-2021
Project End Date: 29-Jul-2021
Report Date: 02-Aug-2021



Prepared by:

ConeTec Inc.
820 Aladdin Avenue
San Leandro, CA 94577

Tel: (510) 357-3677

ConeTecCA@conetec.com
www.conetec.com
www.conetecdataservices.com

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. for Kleinfelder of Oakland, California. The program consisted of cone penetration testing (CPTu) at four (4) locations. Shear wave velocities were recorded in two (2) soundings. The assumed phreatic surface used for the calculated parameters is based on the shallowest pore pressure dissipation test to reach equilibrium within each sounding.

Project Information

Project	
Client	Kleinfelder
Project	Duke Warehouse Silver Creek
ConeTec Project #	21-56-22781

An aerial overview from Google Earth including the CPT test locations is presented below.

Rig Description	Deployment System	Test Type
CPT truck rig (C17)	30-ton truck mounted cylinder	CPTu/SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
CPTu/SCPTu	Consumer grade GPS	32610

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)
446:T1500F15U500	446	15	225	1500	15	500
Cone 446 was used on all soundings.						

Cone Penetration Test	
Depth reference	Depths are referenced to the existing ground surface at the time of test.
Tip and sleeve data offset	0.1 Meter This has been accounted for in the CPT data files.
Additional Plots	Advanced, Seismic, and Soil Behavior Type (SBT) scatter plots
Additional Comments	None

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behaviour 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 CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu 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 Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Limitations

This report has been prepared for the exclusive use of Kleinfelder (Client) for the project titled "Duke Warehouse Silver Creek". 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 5 cm^2, 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 forty-four millimeter diameter over a length of thirty-two millimeter with tapered leading and trailing edges) located at a distance of 585 millimeters 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 sixty-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 six millimeters 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 meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

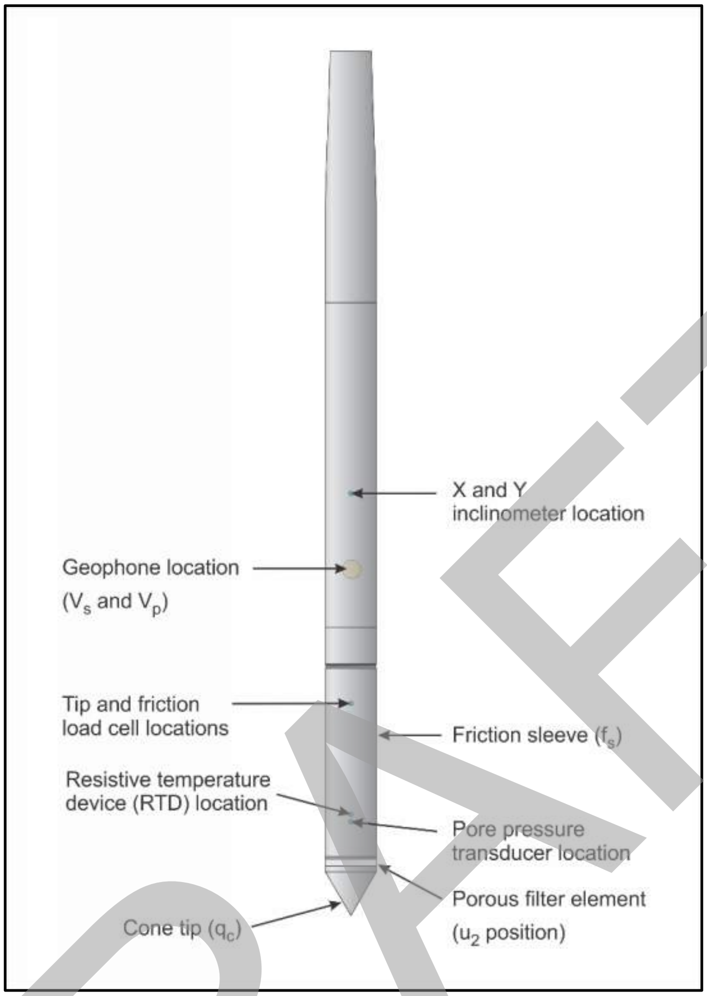


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a sixteen 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 centimeters; 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 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 two centimeters per second, within acceptable tolerances. Typically, one-meter length rods with an outer diameter of 1.5 inches (38.1 millimeters) 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 type 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 may be 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 uphole 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.

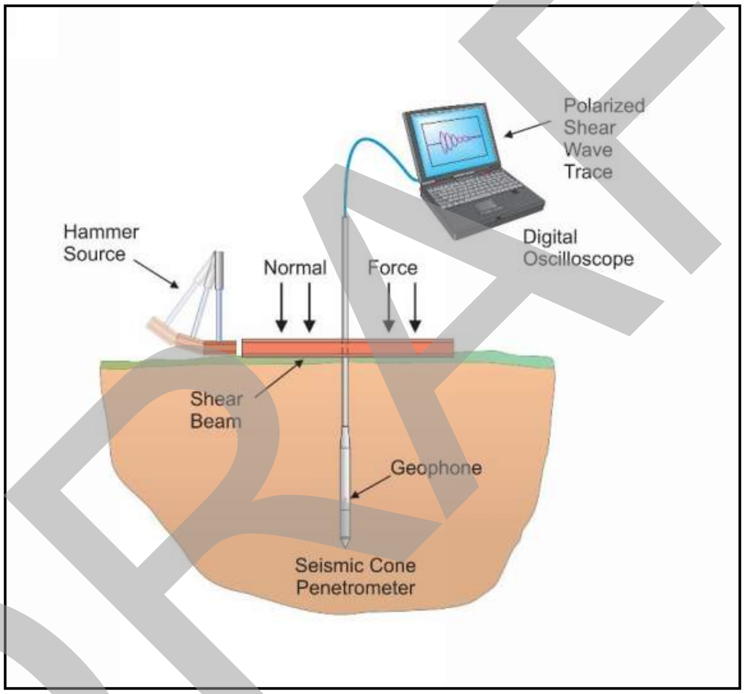


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM D5778 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 and uncertainty analysis purposes. 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).

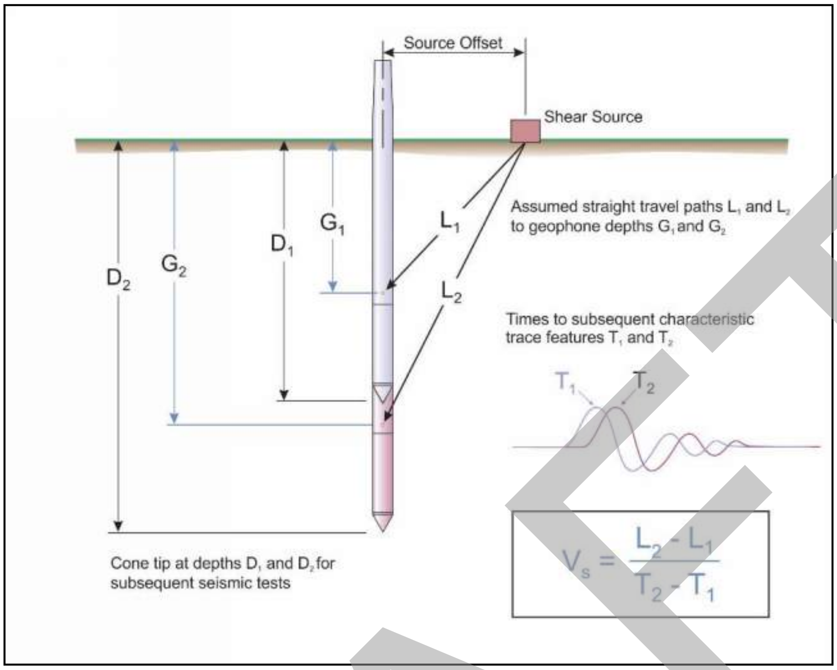


Figure SCPTu-2. Illustration of a seismic cone penetration test

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.

For all SCPTu soundings that have achieved a depth of at least 100 feet (30 meters), the average shear wave velocity to a depth of 100 feet ($\bar{v}_s$) 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)
$\sum_{i=1}^{n} d_i$ = the total thickness of all layers between 0 and 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).

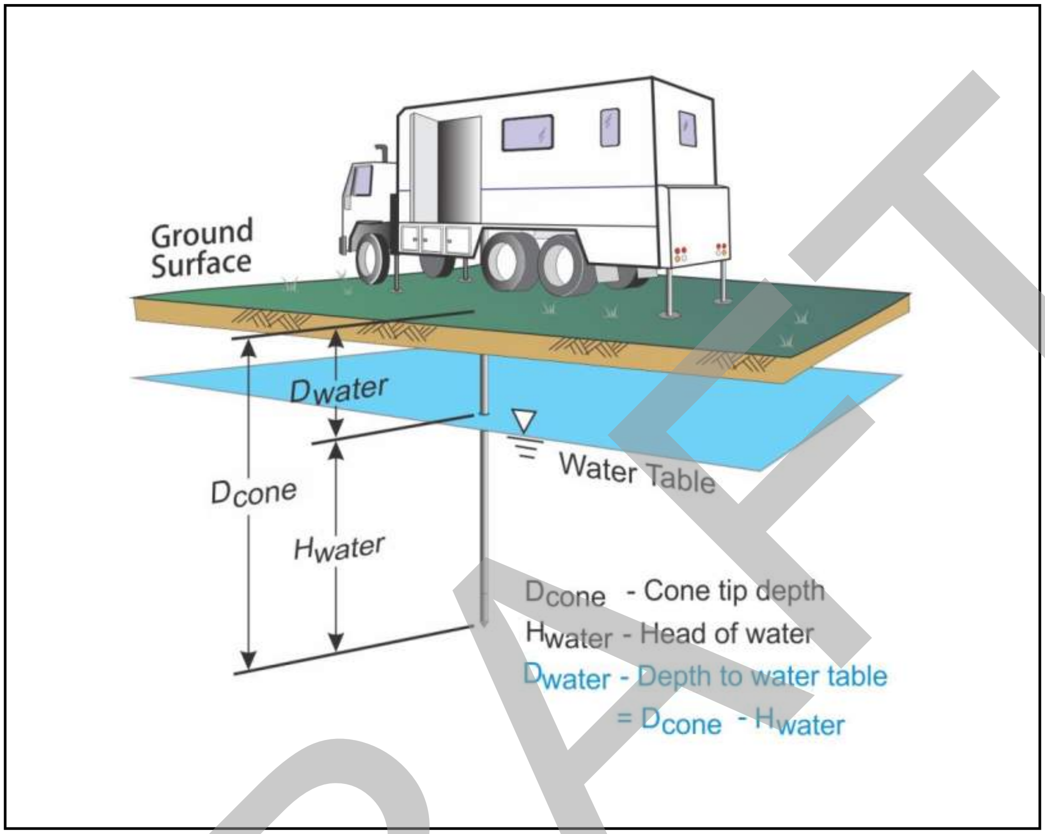


Figure PPD-1. Pore pressure dissipation test setup

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.

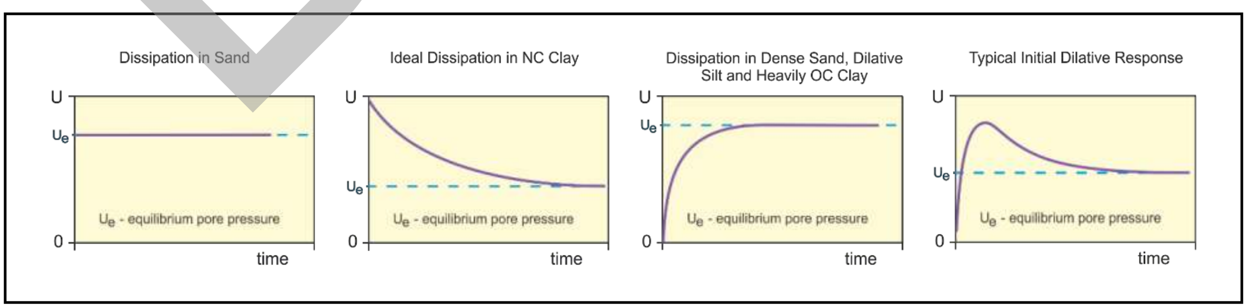


Figure PPD-2. Pore pressure dissipation curve examples

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.

American Society of Civil Engineers (ASCE), 2010, "Minimum Design Loads for Buildings and Other Structures", Standard ASCE/SEI 7-10, American Society of Civil Engineers, ISBN 978-0-7844-1085-1, Reston, Virginia. DOI: 10.1061/9780784412916.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-12.

ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM International, West Conshohocken, PA. DOI: 10.1520/D7400_D7400M-19.

Burns, S.E. and Mayne, P.W., 1998, "Monotonic and dilatory pore pressure decay during piezocone tests", Canadian Geotechnical Journal 26 (4): 1063-1073. DOI: 1063-1073/T98-062.

Burns, S.E. and Mayne, P.W., 2002, "Analytical cavity expansion-critical state model cone dissipation in fine-grained soils", Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, "The piezometer probe: a useful investigation tool", Proceedings, 10^{th} International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: 10.1061/9780784412770.027.

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization *4*, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

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. DOI: 10.1061/(ASCE)0733-9410(1986)112:8(791).

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158. DOI: 10.1139/T90-014.

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): 539-550. DOI: 10.1139/T92-061.

Robertson, P.K., 2009, “Interpretation of cone penetration tests – a unified approach”, Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

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. DOI: 10.1139/T98-105.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34. DOI: 10.1680/geot.1991.41.1.17.

The following appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots
- SBT Zone Scatter Plots
- Seismic Cone Penetration Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Wave Traces
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No: 21-56-22781
Client: Kleinfelder
Project: Duke Warehouse Silver Creek
Start Date: 29-Jul-2021
End Date: 29-Jul-2021

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting[2] (m)	Elevation[3] (ft)	Refer to Notation Number
SCPT-01	21-56-22781_SP01	29-Jul-2021	446:T1500F15U500	25.4	44.54	4124249	607340	204	
CPT-02	21-56-22781_CP02	29-Jul-2021	446:T1500F15U500	25.8	44.54	4124149	607369	205	
CPT-03	21-56-22781_CP03	29-Jul-2021	446:T1500F15U500	23.3	44.54	4124328	607510	202	
SCPT-04	21-56-22781_SP04	29-Jul-2021	446:T1500F15U500	23.8	44.54	4124228	607588	203	

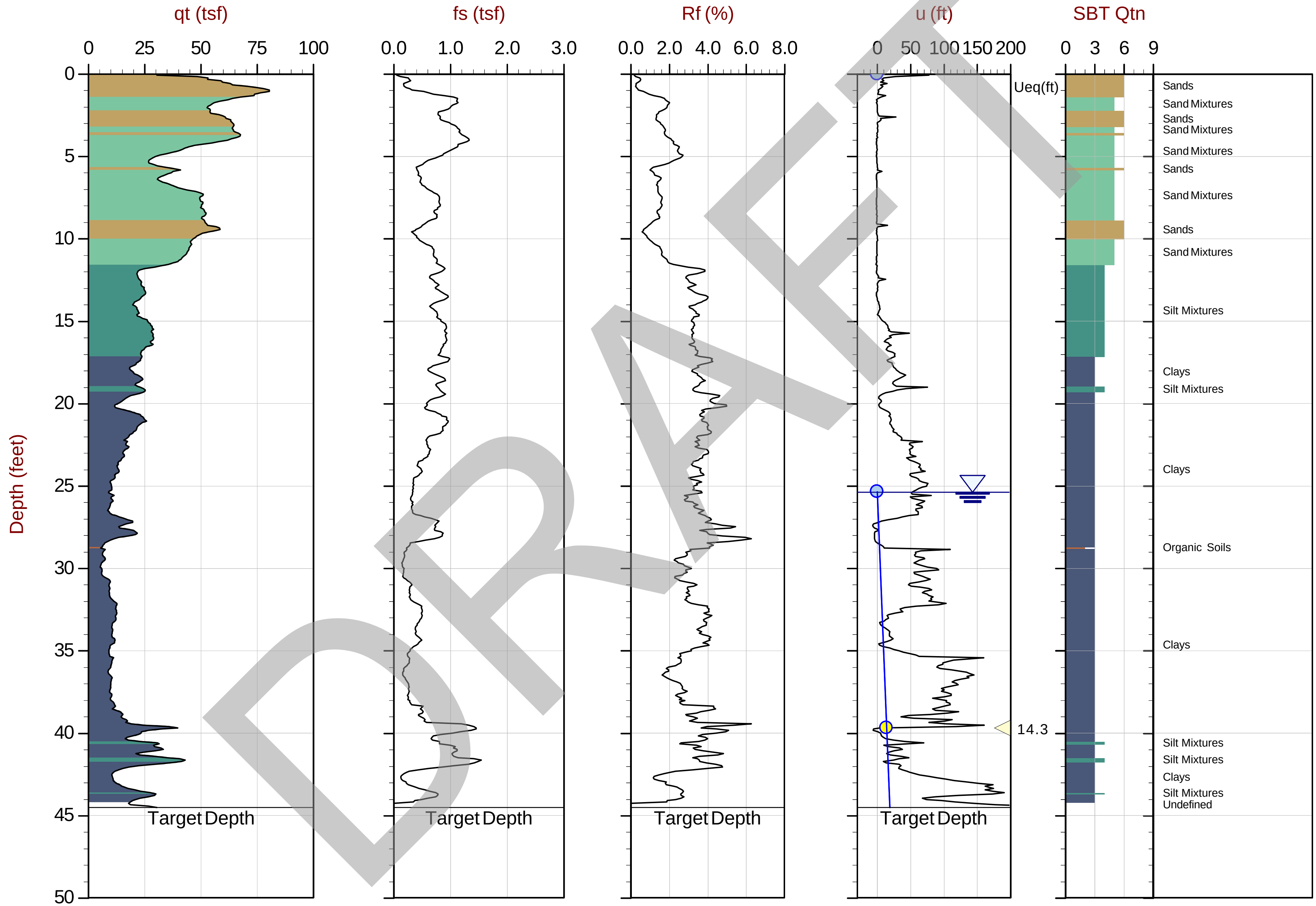
1. The assumed phreatic surface is based on the shallowest pore pressure dissipation test performed within the sounding. Hydrostatic conditions are assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 10S.
3. Elevations are referenced to the ground surface and are derived from the Google Earth Elevation for the recorded coordinates.

ConeTec

Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 08:50
Site: Duke Warehouse Silver Creek

Sounding: SCPT-01
Cone: 446:T1500F15U500



Max Depth: 13.575 m / 44.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124249m E: 607340m

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

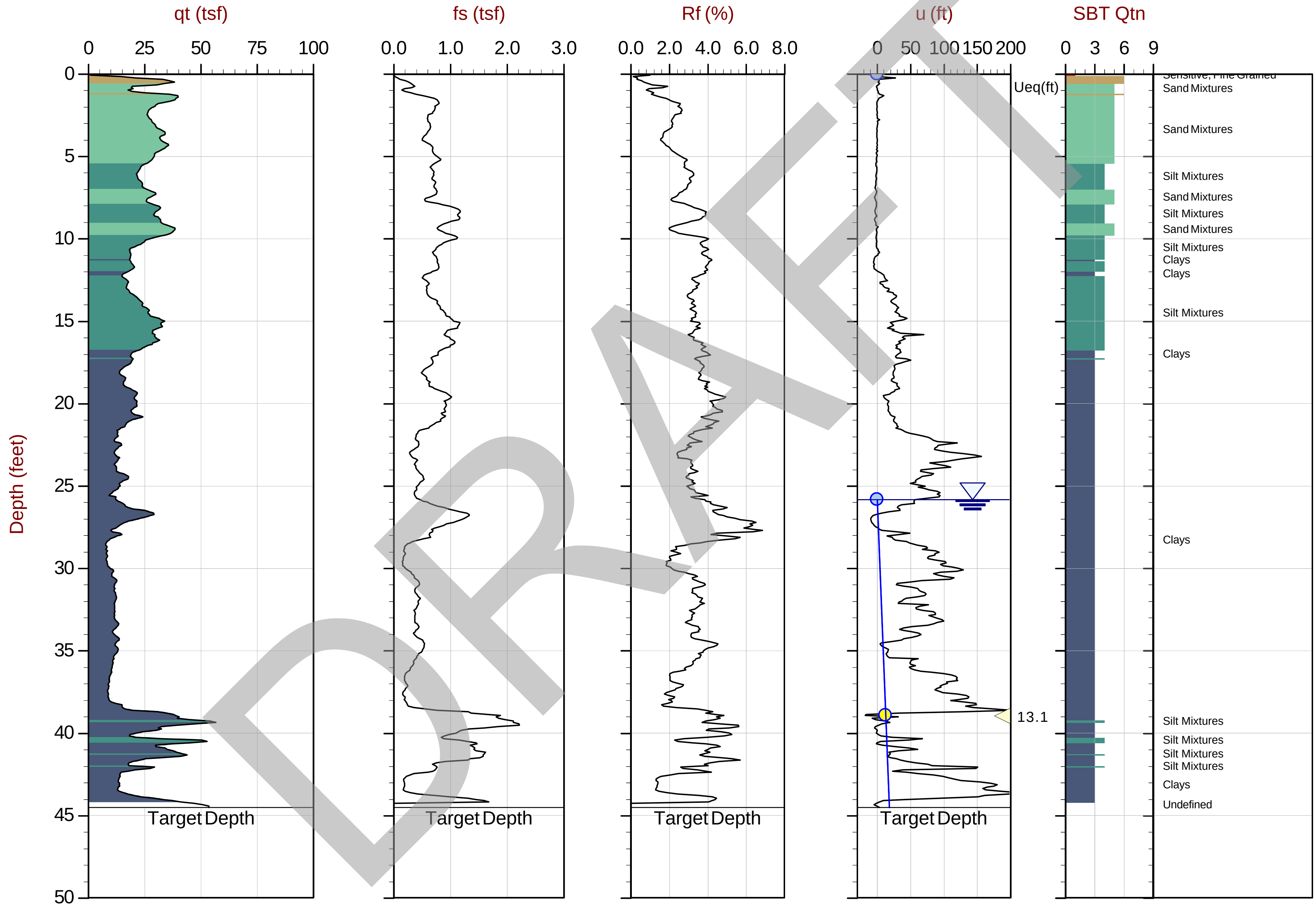
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 09:37
Site: Duke Warehouse Silver Creek

Sounding: CPT-02
Cone: 446:T1500F15U500



Max Depth: 13.575 m / 44.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124149m E: 607369m

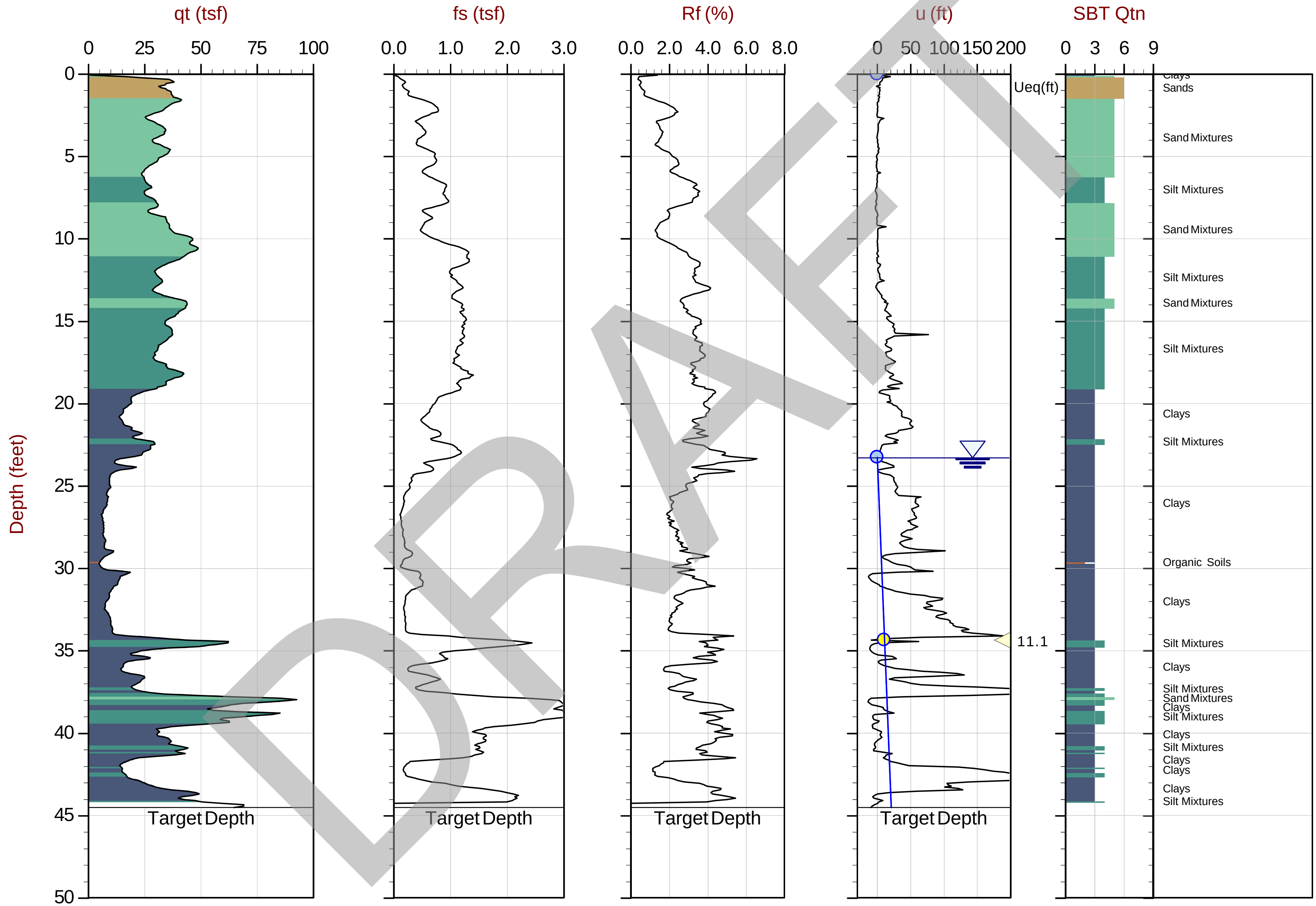
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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: 21-56-22781
Date: 2021-07-29 10:19
Site: Duke Warehouse Silver Creek
Sounding: CPT-03
Cone: 446:T1500F15U500



Max Depth: 13.575 m / 44.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124328m E: 607510m

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

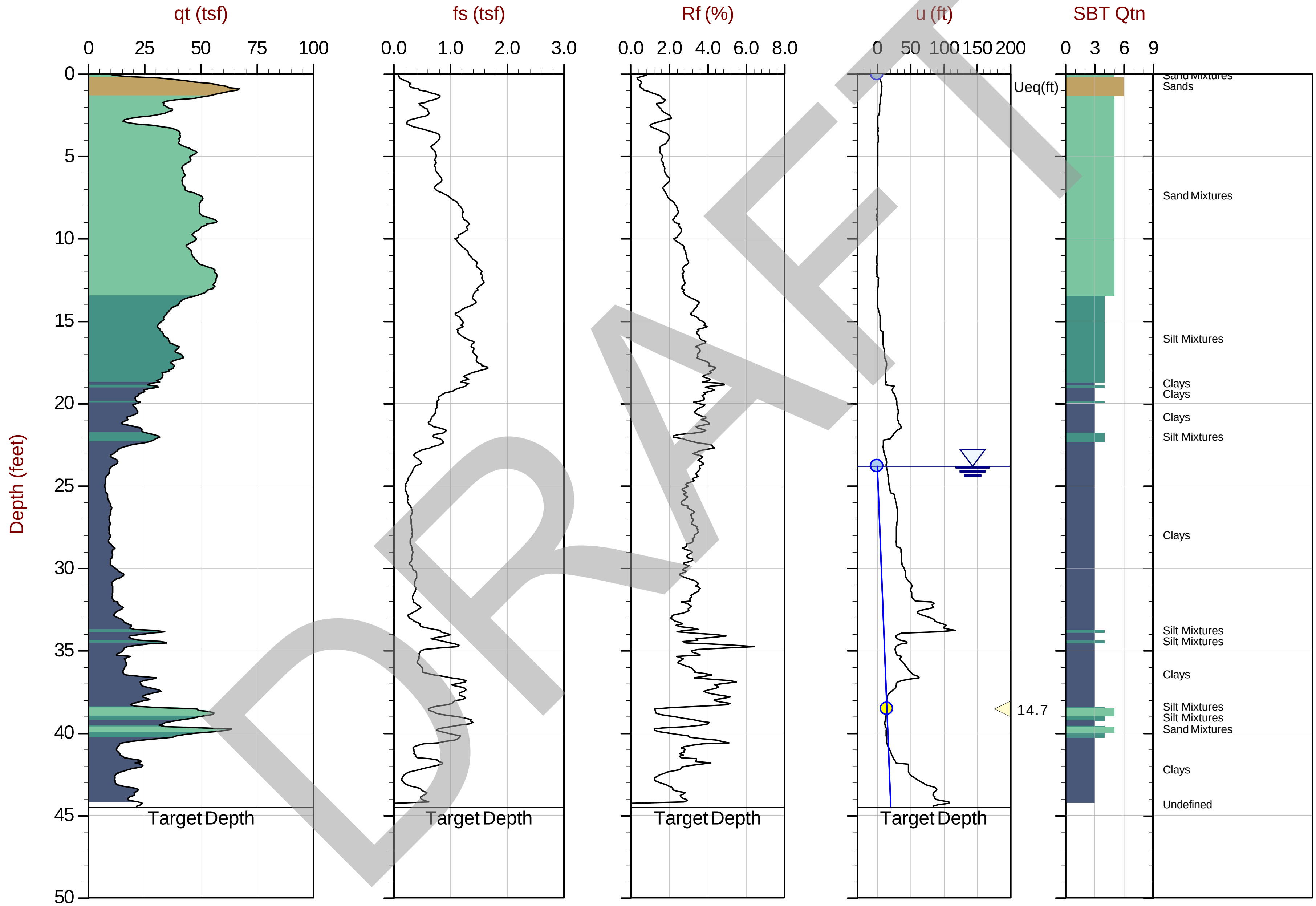
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 08:03
Site: Duke Warehouse Silver Creek

Sounding: SCPT-04
Cone: 446:T1500F15U500



Max Depth: 13.575 m / 44.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124228m E: 607588m

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.

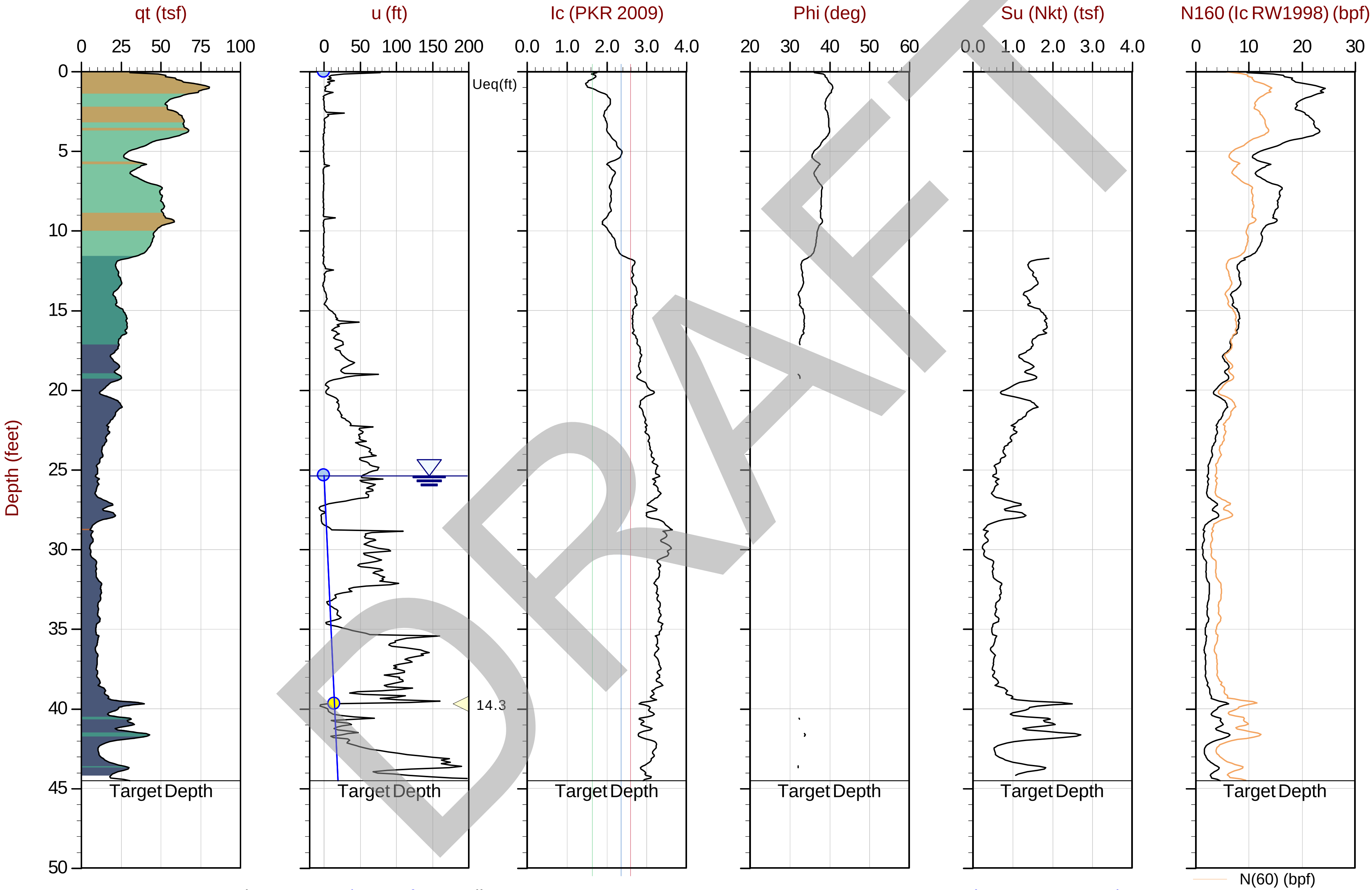
Advanced Cone Penetration Test Plots

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 08:50
Site: Duke Warehouse Silver Creek

Sounding: SCPT-01
Cone: 446:T1500F15U500



Max Depth: 13.575 m / 44.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124249m E: 607340m

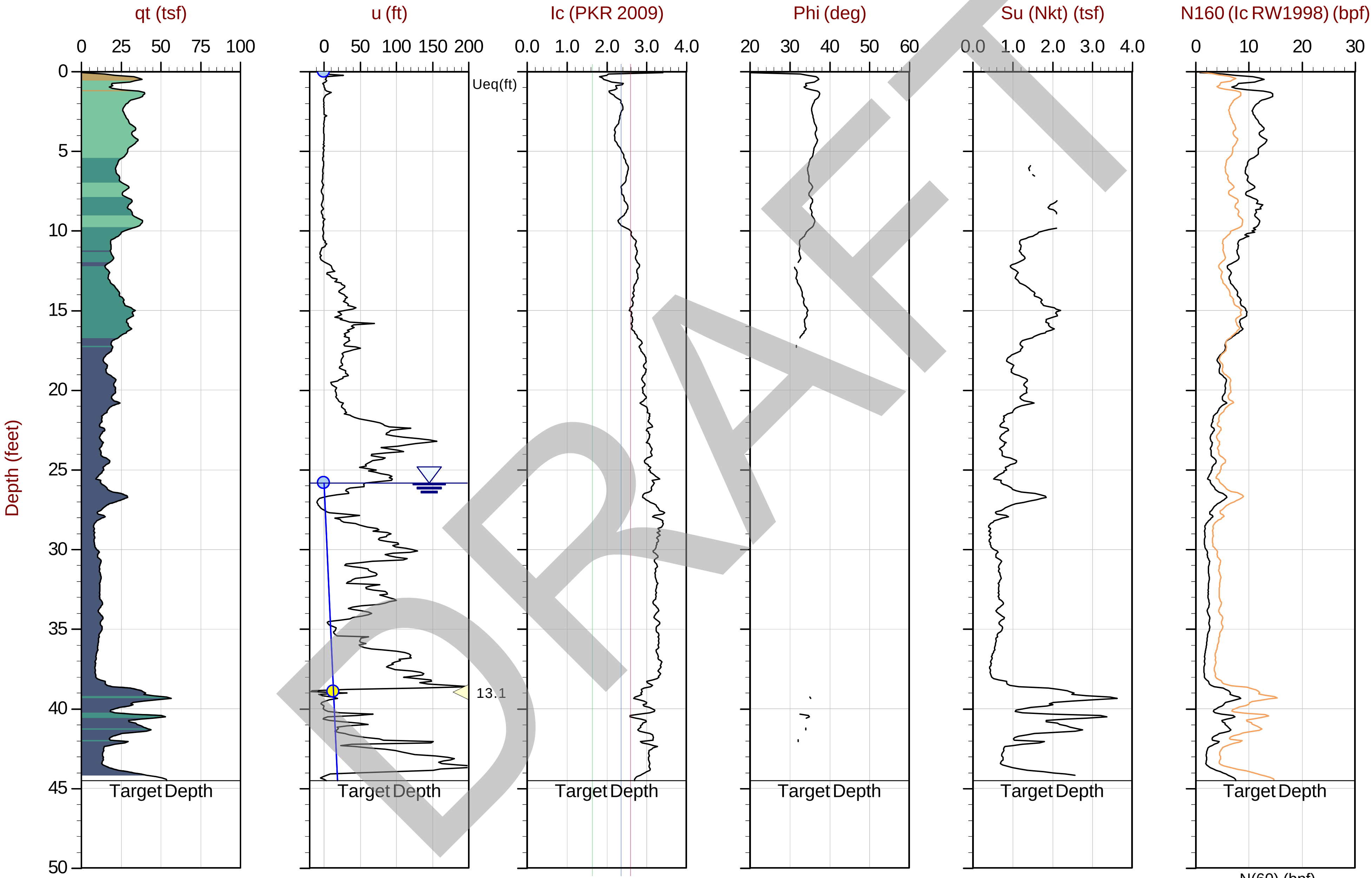
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 09:37
Site: Duke Warehouse Silver Creek

Sounding: CPT-02
Cone: 446:T1500F15U500



Max Depth: 13.575 m / 44.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124149m E: 607369m

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

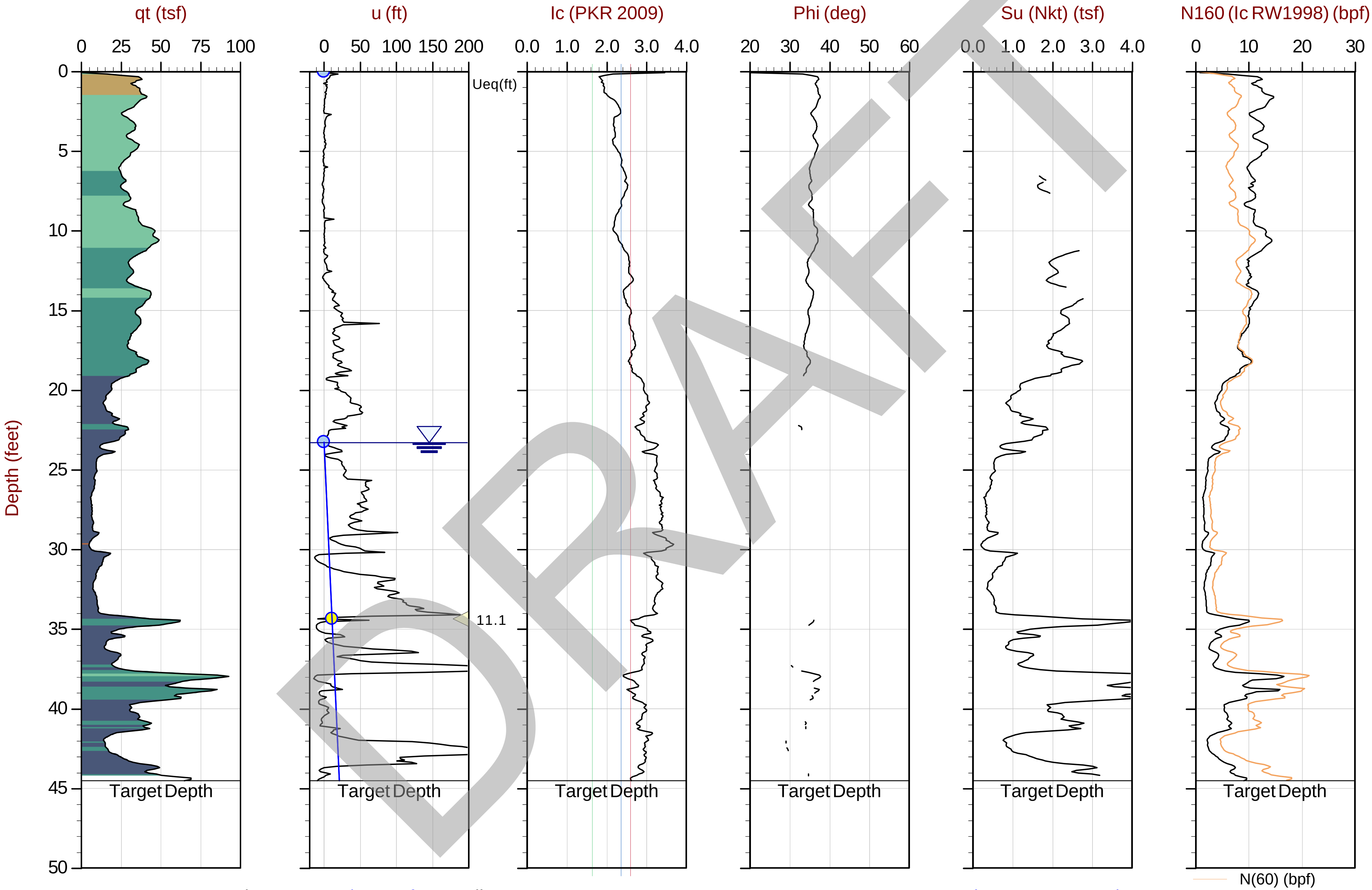
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 10:19
Site: Duke Warehouse Silver Creek

Sounding: CPT-03
Cone: 446:T1500F15U500



Max Depth: 13.575 m / 44.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124328m E: 607510m

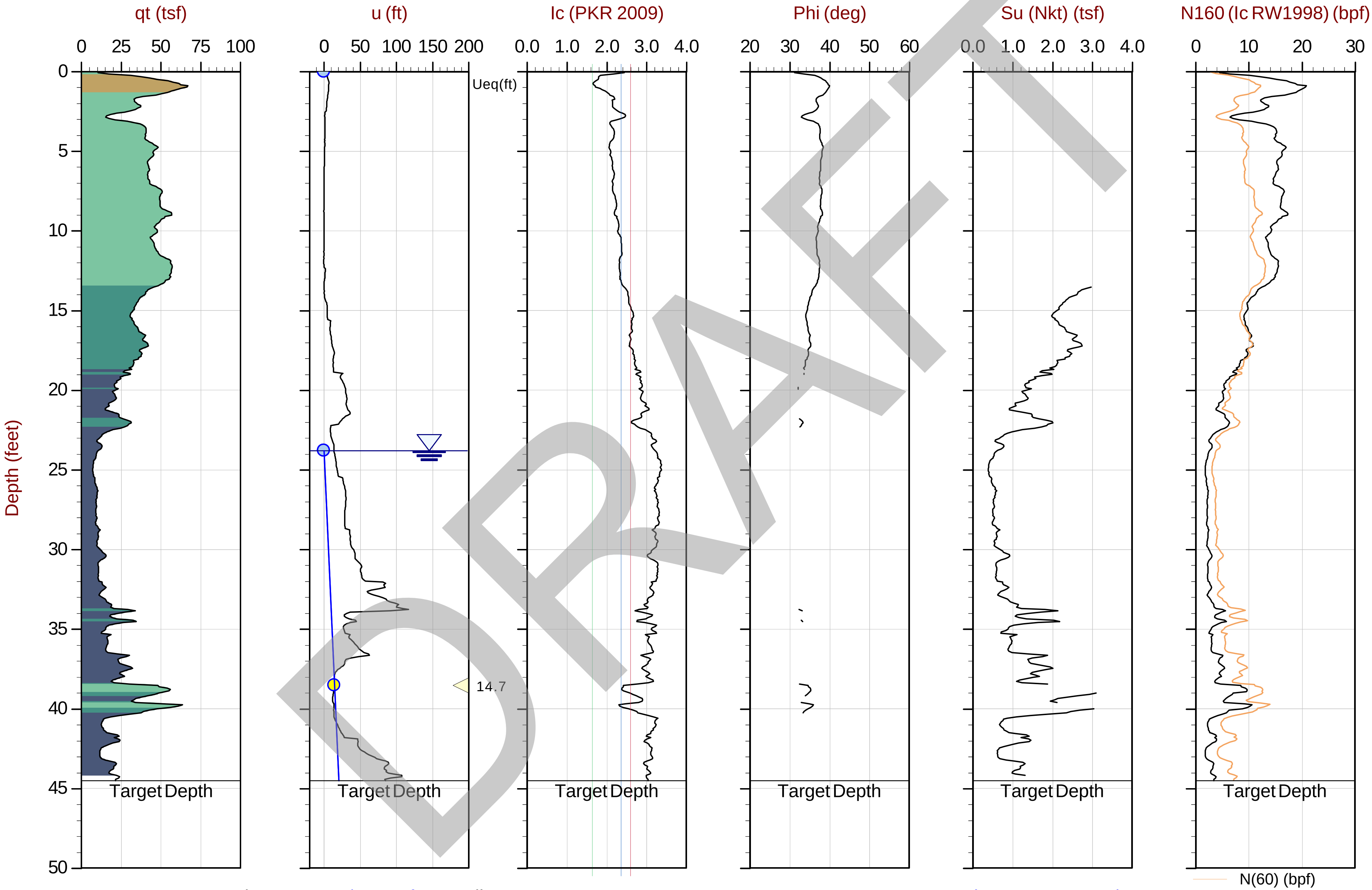
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 08:03
Site: Duke Warehouse Silver Creek

Sounding: SCPT-04
Cone: 446:T1500F15U500



Max Depth: 13.575 m / 44.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124228m E: 607588m

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

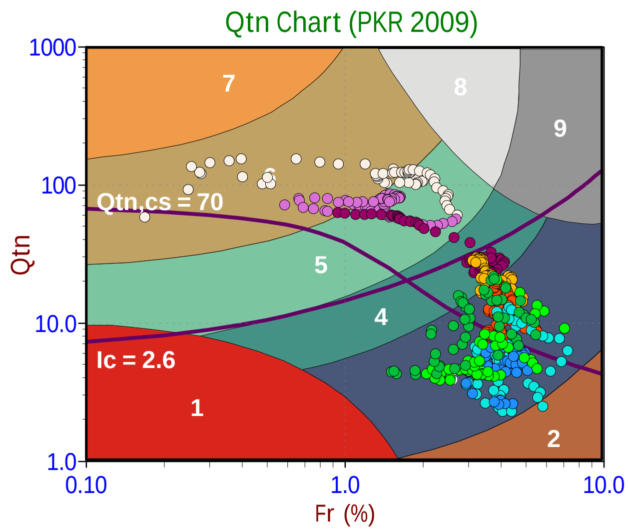
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

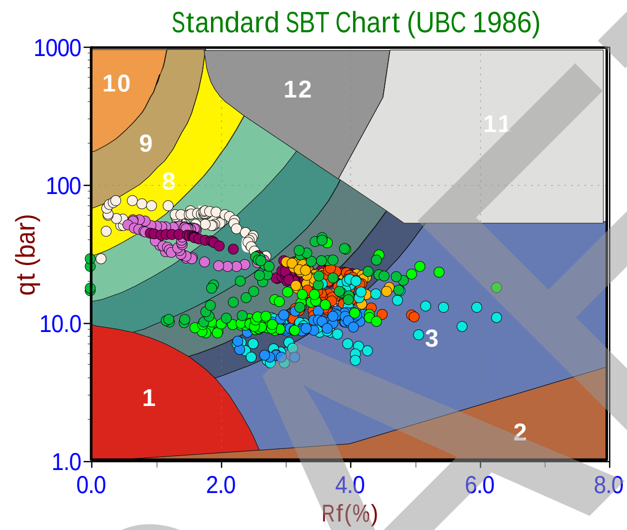
Soil Behavior Type (SBT) Scatter Plots

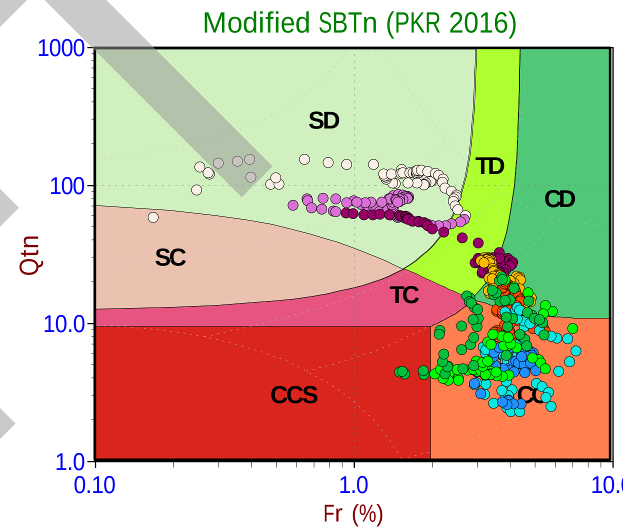
CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 08:50
Site: Duke Warehouse Silver Creek
Sounding: SCPT-01
Cone: 446:T1500F15U500







Depth Ranges

- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 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

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

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)

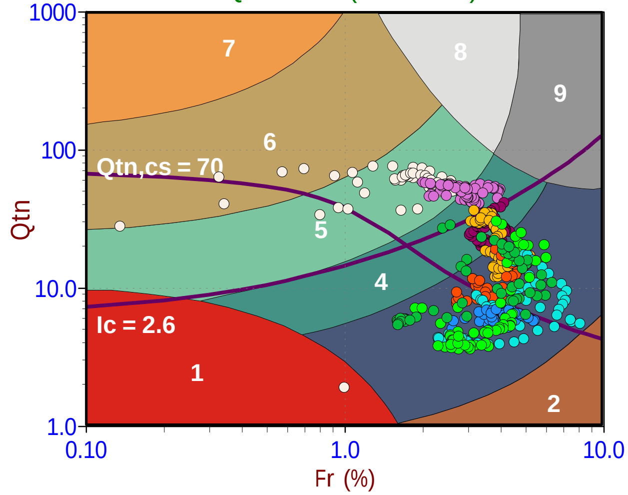
CONETEC

Kleinfelder

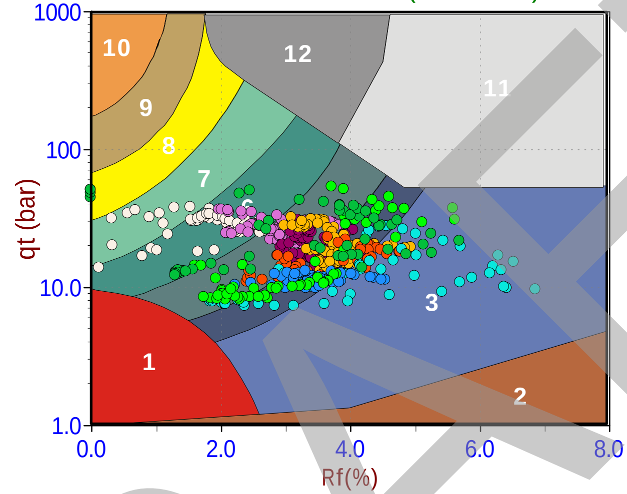
Job No: 21-56-22781
Date: 2021-07-29 09:37
Site: Duke Warehouse Silver Creek

Sounding: CPT-02
Cone: 446:T1500F15U500

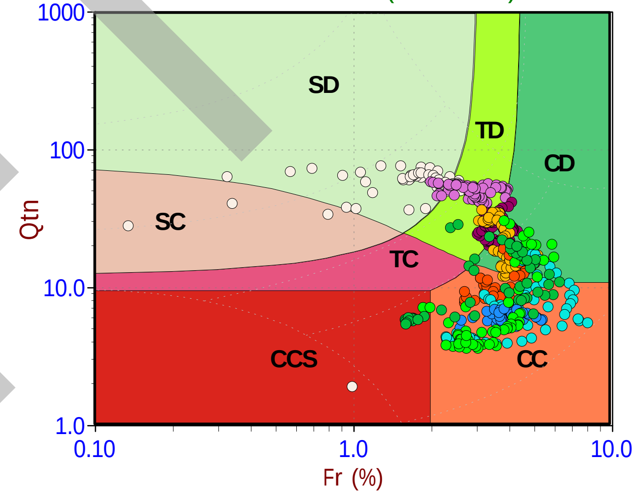
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 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

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

Legend

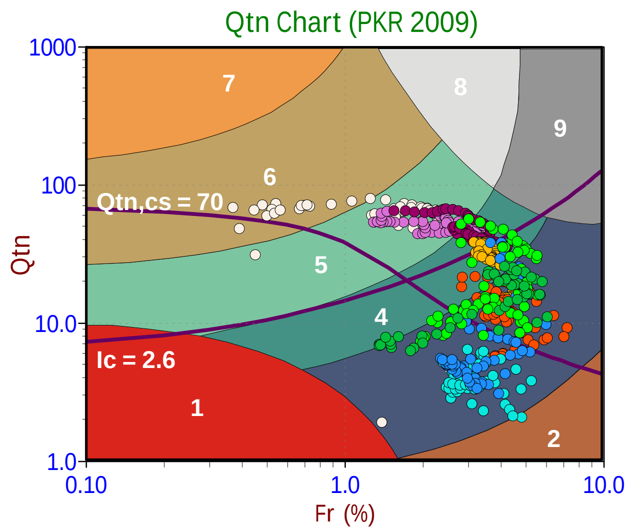
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

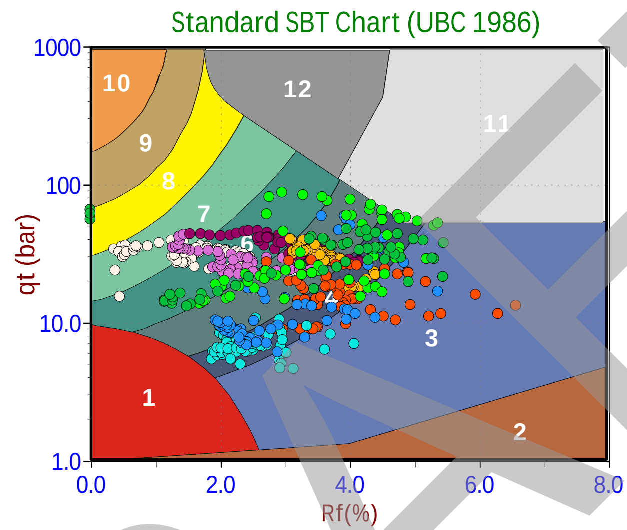
CONETEC

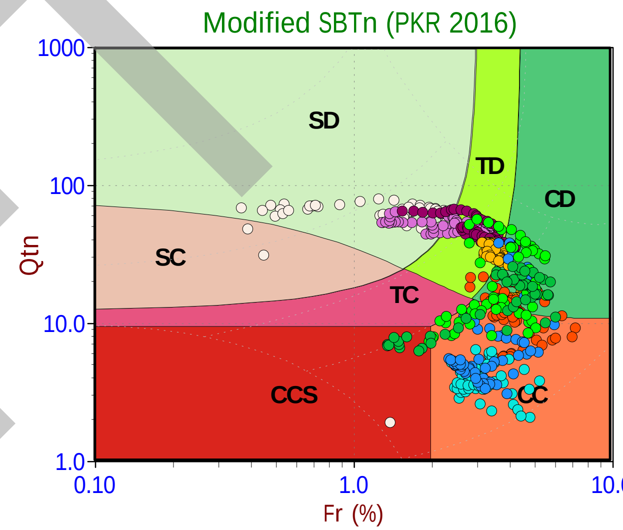
Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 10:19
Site: Duke Warehouse Silver Creek

Sounding: CPT-03
Cone: 446:T1500F15U500







Depth Ranges
- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 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
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

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)

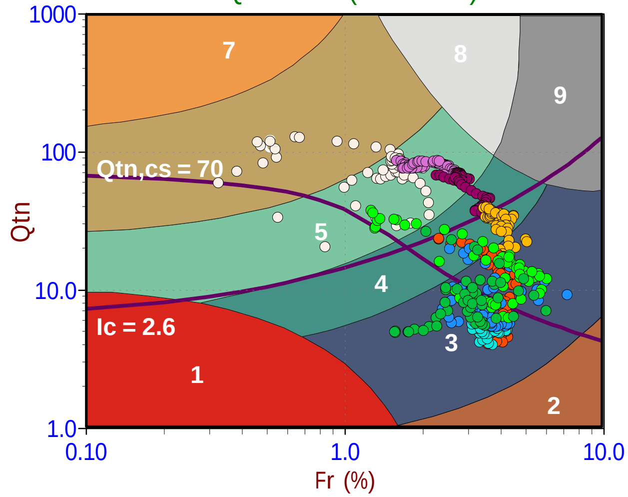
CONETEC

Kleinfelder

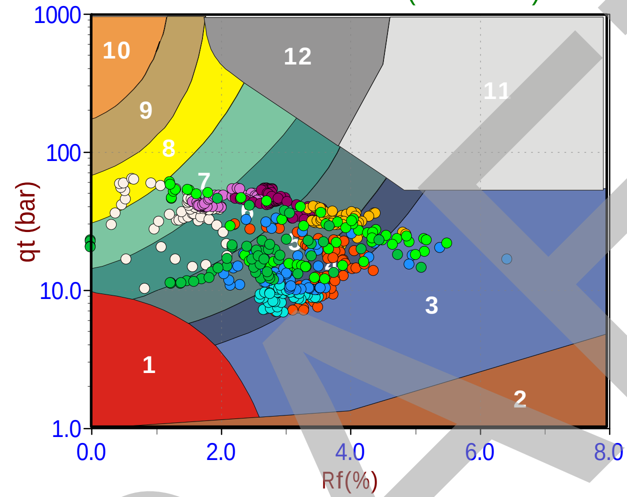
Job No: 21-56-22781
Date: 2021-07-29 08:03
Site: Duke Warehouse Silver Creek

Sounding: SCPT-04
Cone: 446:T1500F15U500

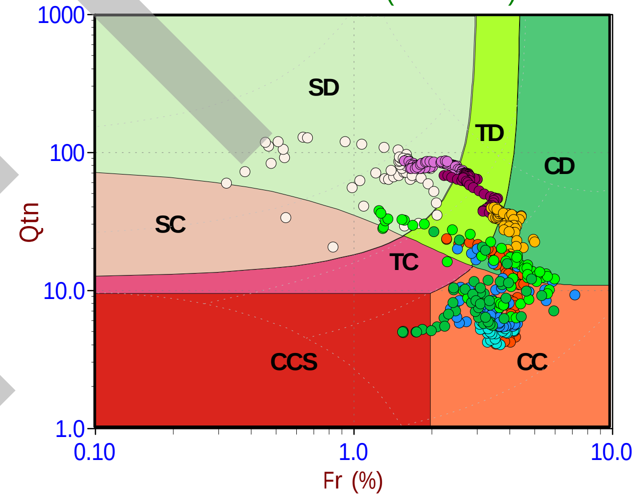
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges
- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 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
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

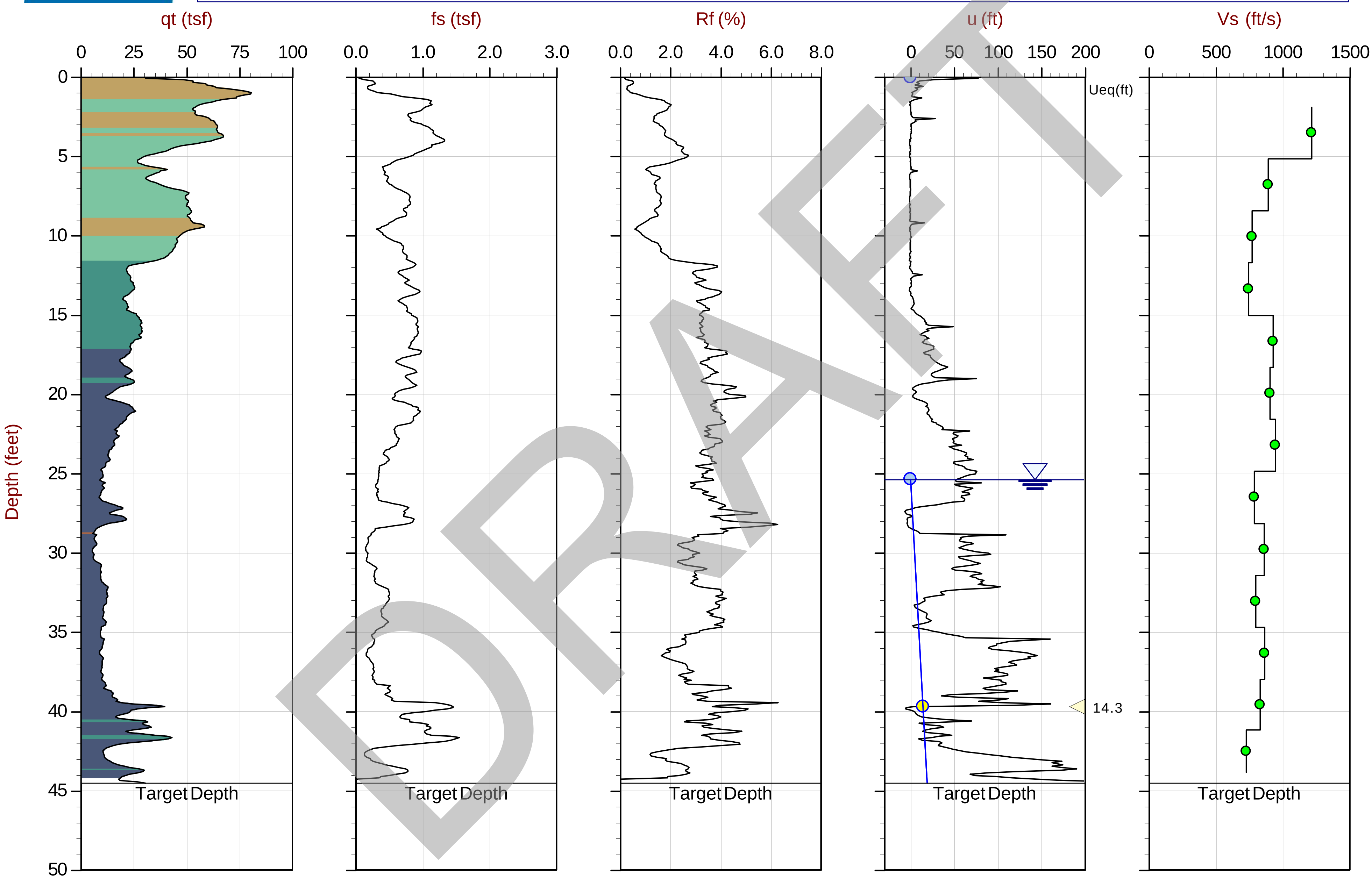
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Seismic Cone Penetration Test Plots

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 08:50
Site: Duke Warehouse Silver Creek
Sounding: SCPT-01
Cone: 446:T1500F15U500



Max Depth: 13.575 m / 44.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124249m E: 607340m

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

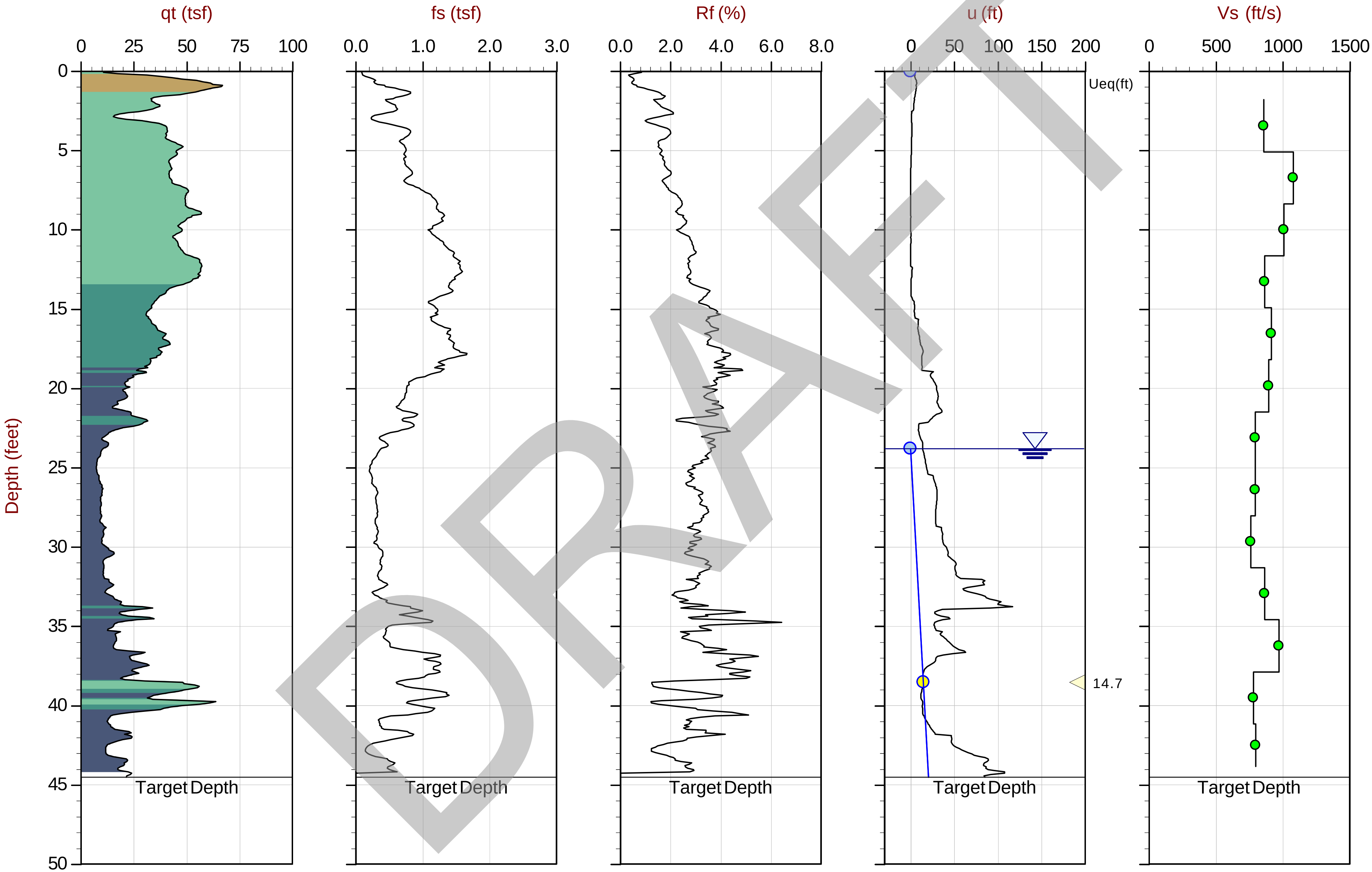
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 2021-07-29 08:03
Site: Duke Warehouse Silver Creek

Sounding: SCPT-04
Cone: 446:T1500F15U500



Max Depth: 13.575 m / 44.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124228m E: 607588m

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

Job No:	21-56-22781
Client:	Kleinfelder
Project:	Duke Warehouse Silver Creek
Sounding ID:	SCPT-01
Date:	07:29:21 08:50
Seismic Source:	Beam
Seismic 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)
2.56	1.90	2.83			
5.81	5.15	5.56	2.73	2.24	1217
9.09	8.43	8.69	3.13	3.50	893
12.37	11.71	11.90	3.21	4.16	771
15.68	15.03	15.17	3.27	4.39	745
18.96	18.31	18.43	3.26	3.50	930
22.24	21.59	21.69	3.26	3.60	906
25.53	24.87	24.96	3.27	3.46	945
28.81	28.15	28.23	3.27	4.15	788
32.09	31.43	31.50	3.27	3.80	861
35.37	34.71	34.77	3.27	4.10	798
38.65	37.99	38.05	3.28	3.79	865
41.83	41.18	41.23	3.18	3.83	831
44.52	43.86	43.91	2.69	3.69	728

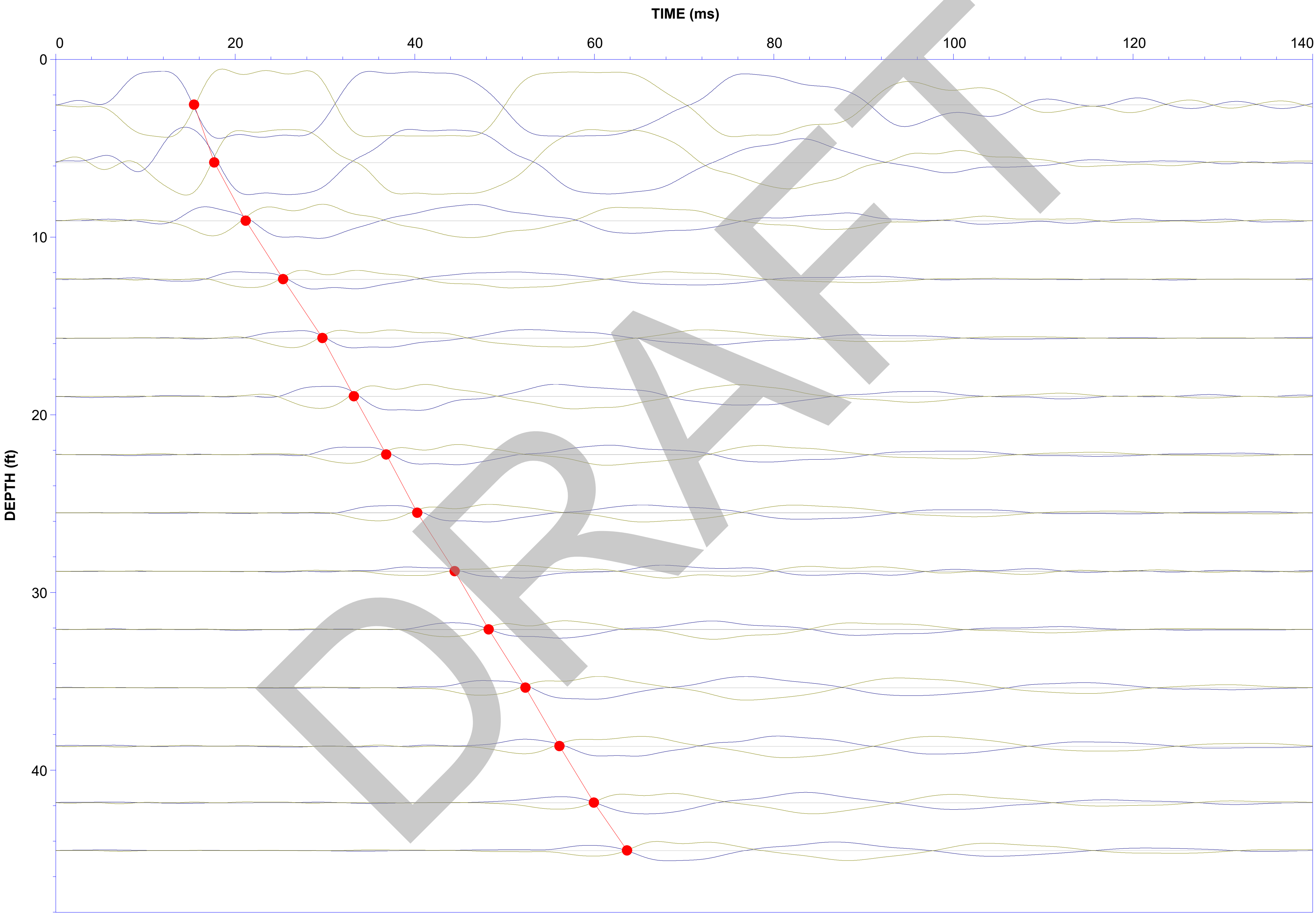
	Job No:	21-56-22781
	Client:	Kleinfelder
	Project:	Duke Warehouse Silver Creek
	Sounding ID:	SCPT-04
	Date:	07:29:21 08:03
	Seismic Source:	Beam
	Seismic Offset (ft):	2.10
	Source Depth (ft):	0.00
	Geophone Offset (ft):	0.66

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)
2.46	1.80	2.77			
5.74	5.09	5.50	2.73	3.18	860
9.02	8.37	8.63	3.12	2.90	1078
12.30	11.65	11.84	3.21	3.18	1010
15.58	14.93	15.08	3.24	3.74	867
18.87	18.21	18.33	3.26	3.55	916
22.15	21.49	21.59	3.26	3.64	895
25.43	24.77	24.86	3.27	4.11	795
28.71	28.05	28.13	3.27	4.11	795
31.99	31.33	31.40	3.27	4.30	761
35.27	34.61	34.68	3.28	3.79	864
38.55	37.89	37.95	3.28	3.36	974
41.83	41.18	41.23	3.28	4.20	780
44.52	43.86	43.91	2.69	3.36	799

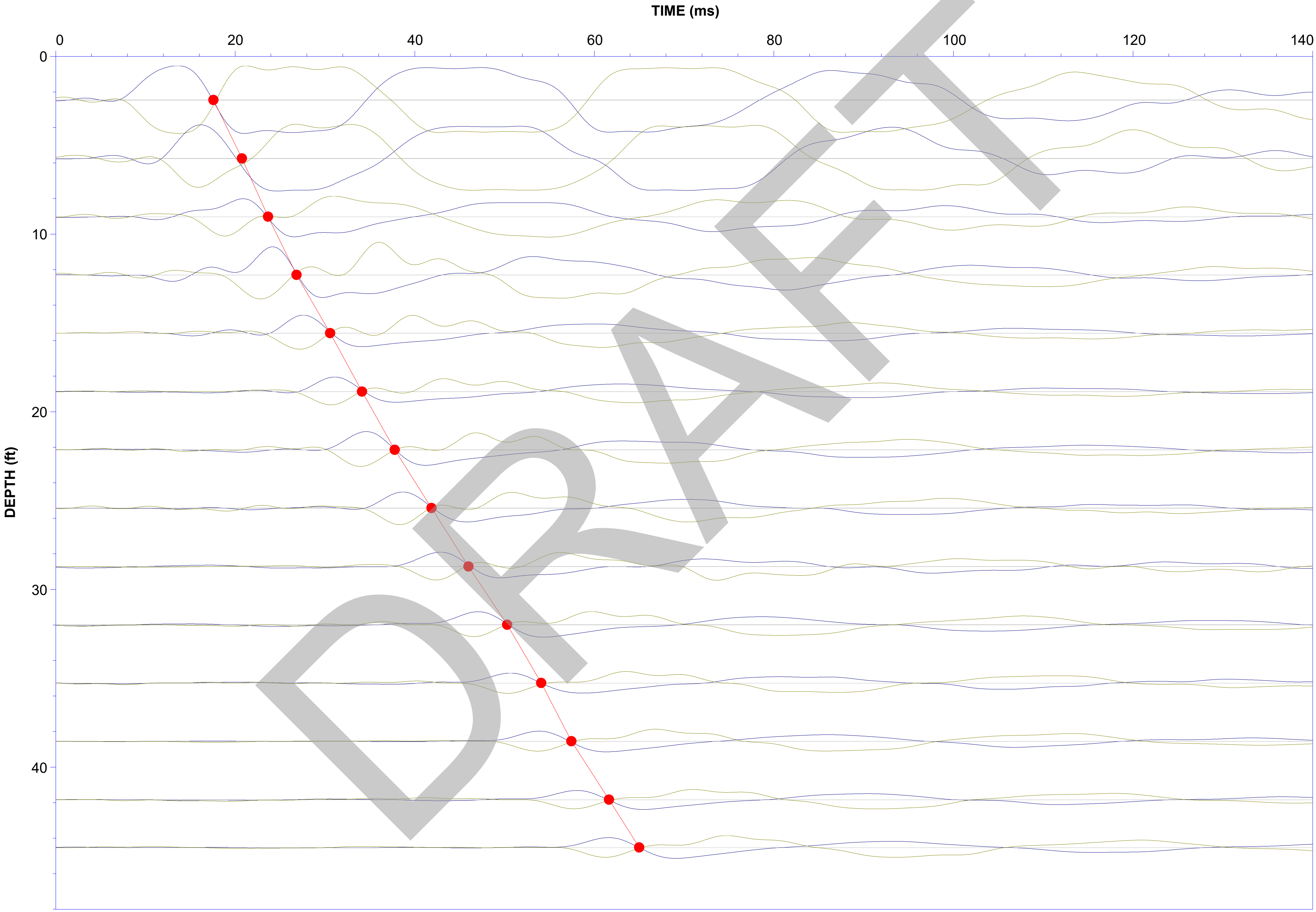
Seismic Cone Penetration Test Shear Wave (Vs) Traces

Job No: 21-56-22781	Client: Kleinfelder	Project: Duke Warehouse Silver Creek	Filter: BP 0-200 Hz	Sounding: SCPT-01
Date: 07:29:21 08:50	Cone: 446:T1500F15U500			



Job No: 21-56-22781 | Client: Kleinfelder | Project: Duke Warehouse Silver Creek | Filter: BP 0-200 Hz | Sounding: SCPT-04

Date: 07:29:21 08:03 | Cone: 446:T1500F15U500



Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No: 21-56-22781
Client: Kleinfelder
Project: Duke Warehouse Silver Creek
Start Date: 29-Jul-2021
End Date: 29-Jul-2021

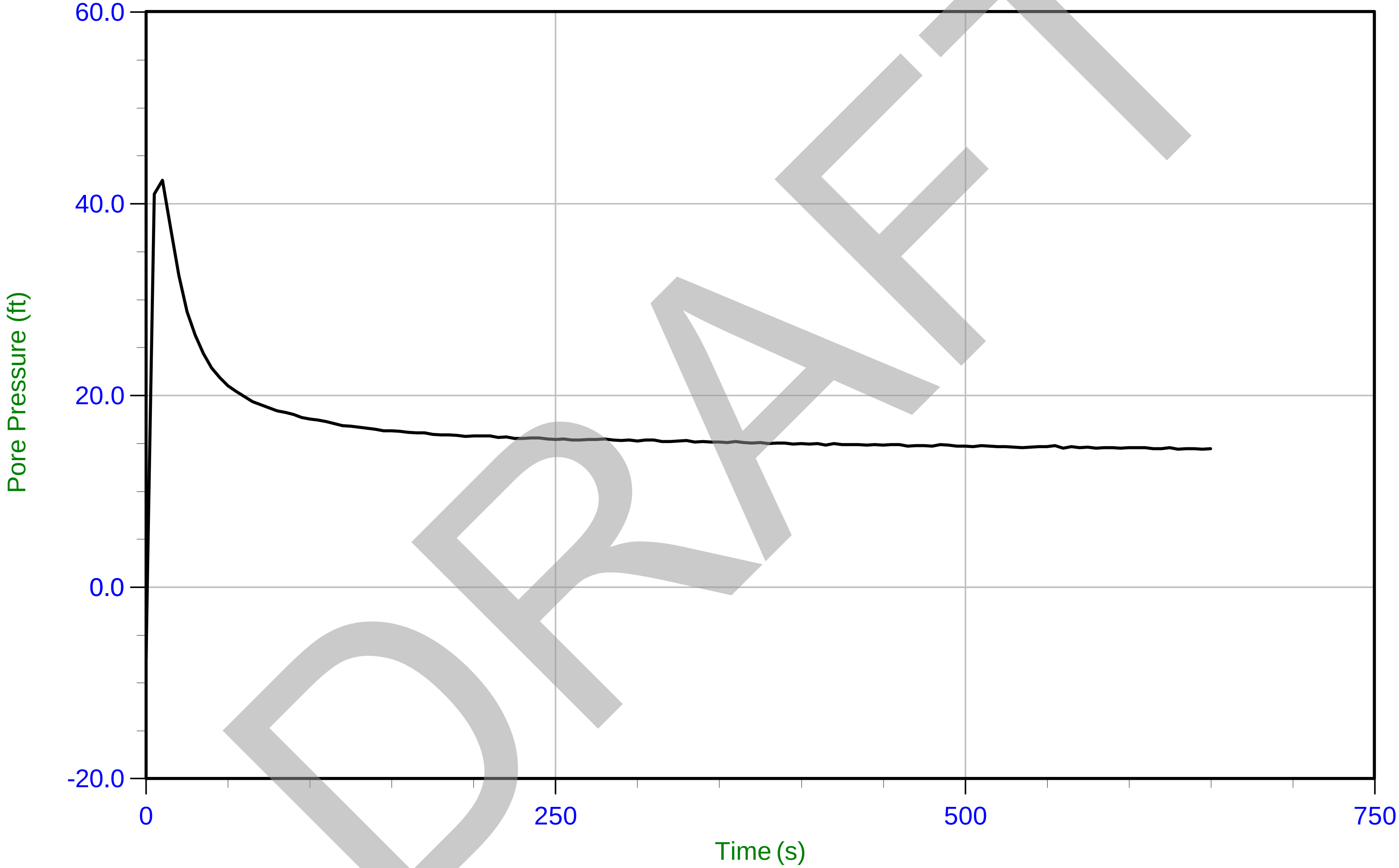
CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
SCPT-01	21-56-22781_SP01	15	650	39.70	14.3	25.4
CPT-02	21-56-22781_CP02	15	300	38.96	13.1	25.8
CPT-03	21-56-22781_CP03	15	320	34.37	11.1	23.3
SCPT-04	21-56-22781_SP04	15	300	38.55	14.7	23.8

Kleinfelder

Job No: 21-56-22781
Date: 07/29/2021 08:50
Site: Duke Warehouse Silver Creek

Sounding: SCPT-01
Cone: 446:T1500F15U500 Area=15 cm²

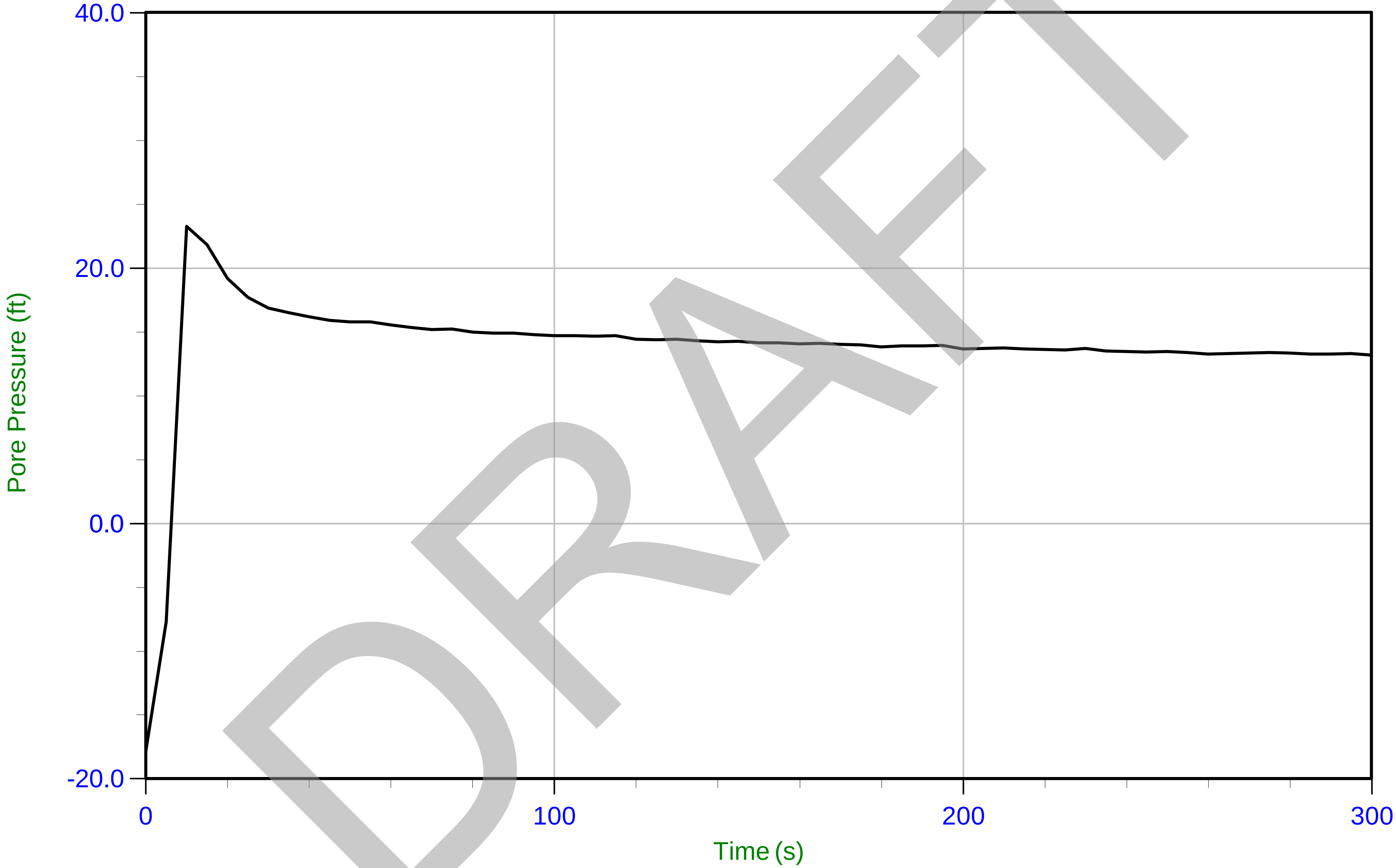


Trace Summary:

Filename: 21-56-22781_SP01.PPF	u Min: -7.5 ft	WT: 7.732 m / 25.367 ft
Depth: 12.100 m / 39.698 ft	u Max: 42.4 ft	Ueq: 14.3 ft
Duration: 650.0 s	u Final: 14.4 ft	

CONETEC

Kleinfelder	Job No: 21-56-22781 Date: 07/29/2021 09:37 Site: Duke Warehouse Silver Creek	Sounding: CPT-02 Cone: 446:T1500F15U500 Area=15 cm²



Trace Summary:

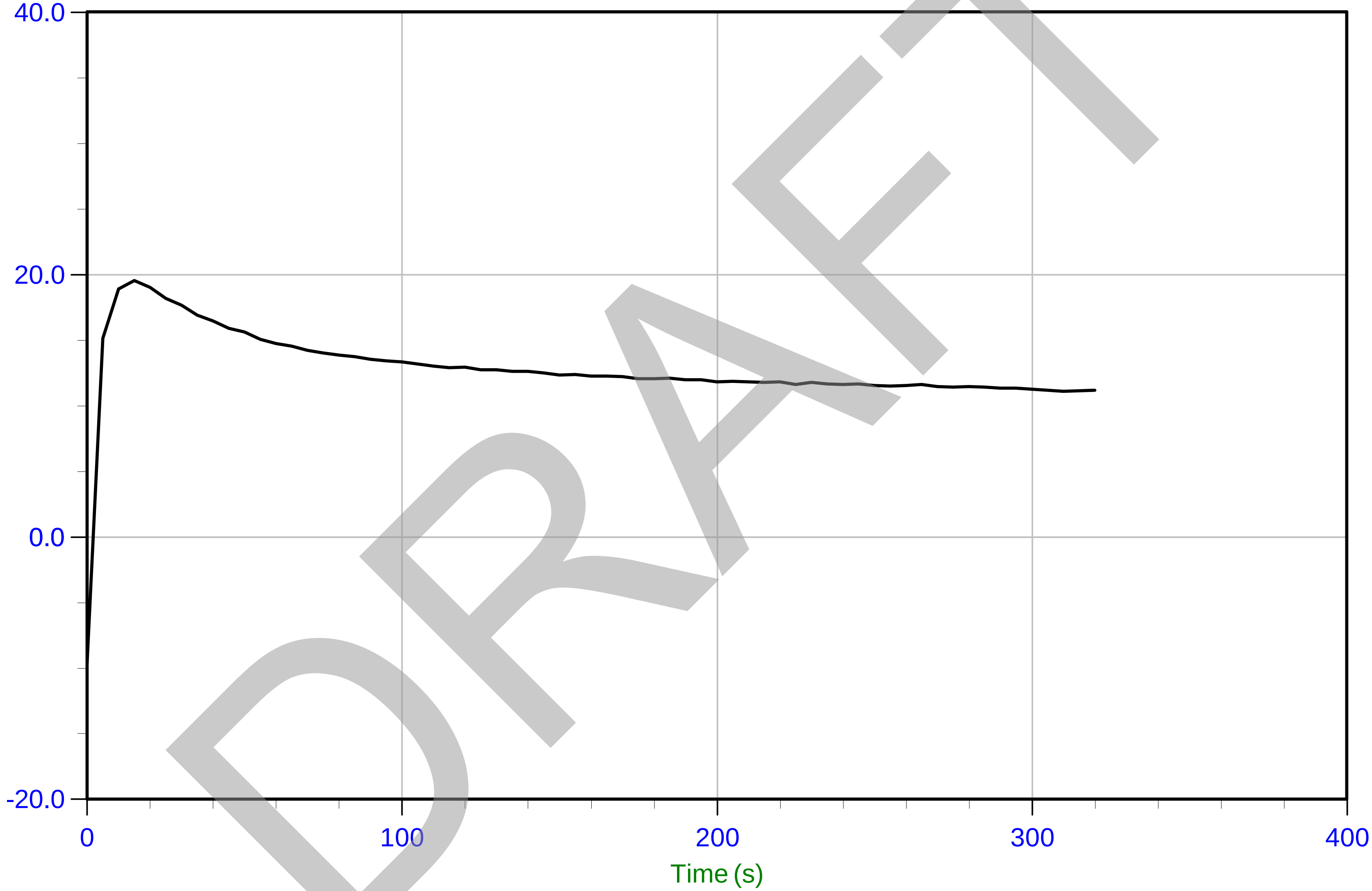
Filename: 21-56-22781_CP02.PPF	u Min: -17.8 ft	WT: 7.876 m / 25.839 ft
Depth: 11.875 m / 38.959 ft	u Max: 23.3 ft	Ueq: 13.1 ft
Duration: 300.0 s	u Final: 13.2 ft	

ConeTec

Kleinfelder

Job No: 21-56-22781
Date: 07/29/2021 10:19
Site: Duke Warehouse Silver Creek

Sounding: CPT-03
Cone: 446:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 21-56-22781_CP03.PPF
Depth: 10.475 m / 34.366 ft
Duration: 320.0 s

u Min: -9.6 ft
u Max: 19.5 ft
u Final: 11.2 ft

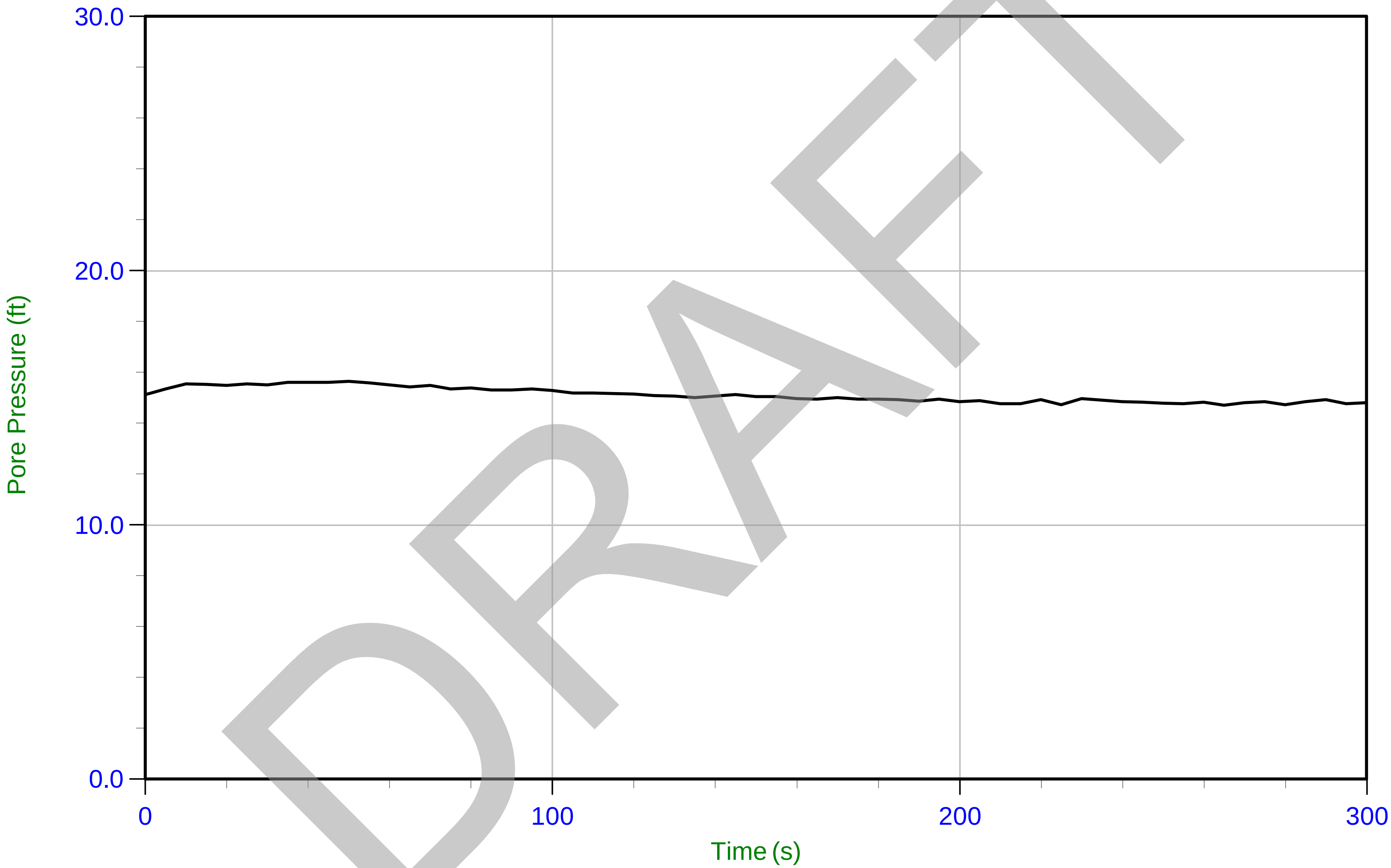
WT: 7.097 m / 23.284 ft
Ueq: 11.1 ft

ConeTec

Kleinfelder

Job No: 21-56-22781
Date: 07/29/2021 08:03
Site: Duke Warehouse Silver Creek

Sounding: SCPT-04
Cone: 446:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 21-56-22781_SP04.PPF
Depth: 11.750 m / 38.549 ft
Duration: 300.0 s

u Min: 14.7 ft
u Max: 15.6 ft
u Final: 14.8 ft

WT: 7.259 m / 23.815 ft
Ueq: 14.7 ft

PRESENTATION OF SITE INVESTIGATION RESULTS

Duke Warehouse Silver Creek

Prepared for:

Kleinfelder

ConeTec Job No: 21-56-22781

Project Start Date: 2022-Jan-20
Project End Date: 2022-Jan-20
Report Date: 2022-Jan-25

Prepared by:

ConeTec Inc.

820 Aladdin Avenue, San Leandro, CA 95477

Tel: (510) 357-3677

ConeTecCA@conetec.com

www.conetec.com
www.conetecdataservices.com

ABOUT THIS REPORT

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. The program consisted of Seismic Piezocone Penetration Testing and Pore Pressure Dissipation Testing. Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

Project Information	
Client	Kleinfelder
Project	Duke Warehouse Silver Creek
ConeTec Project Number	21-56-22781
Rig Description	30-ton Truck CPT Rig (C-15)

Coordinates	
Collection Method	Consumer Grade GPS
EPSG Number	32610 (WGS 84 / UTM 10S)

Cone Penetration Test (CPTu)	
Depth Reference	Existing ground surface at the time of the investigation
Sleeve data offset	0.1 Meters

Calculated Geotechnical Parameters Tables	
Additional Information	The Normalized Soil Behaviour 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 CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu 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 Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Please refer to the list of attached documents following the text of this report. A test summary, location map, and plots are included. Thank you for the opportunity to work on this project.

LIMITATIONS

3rd Party Disclaimer

- The “Report” refers to this report titled Duke Warehouse Silver Creek
- The Report was prepared by ConeTec for Kleinfelder

The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

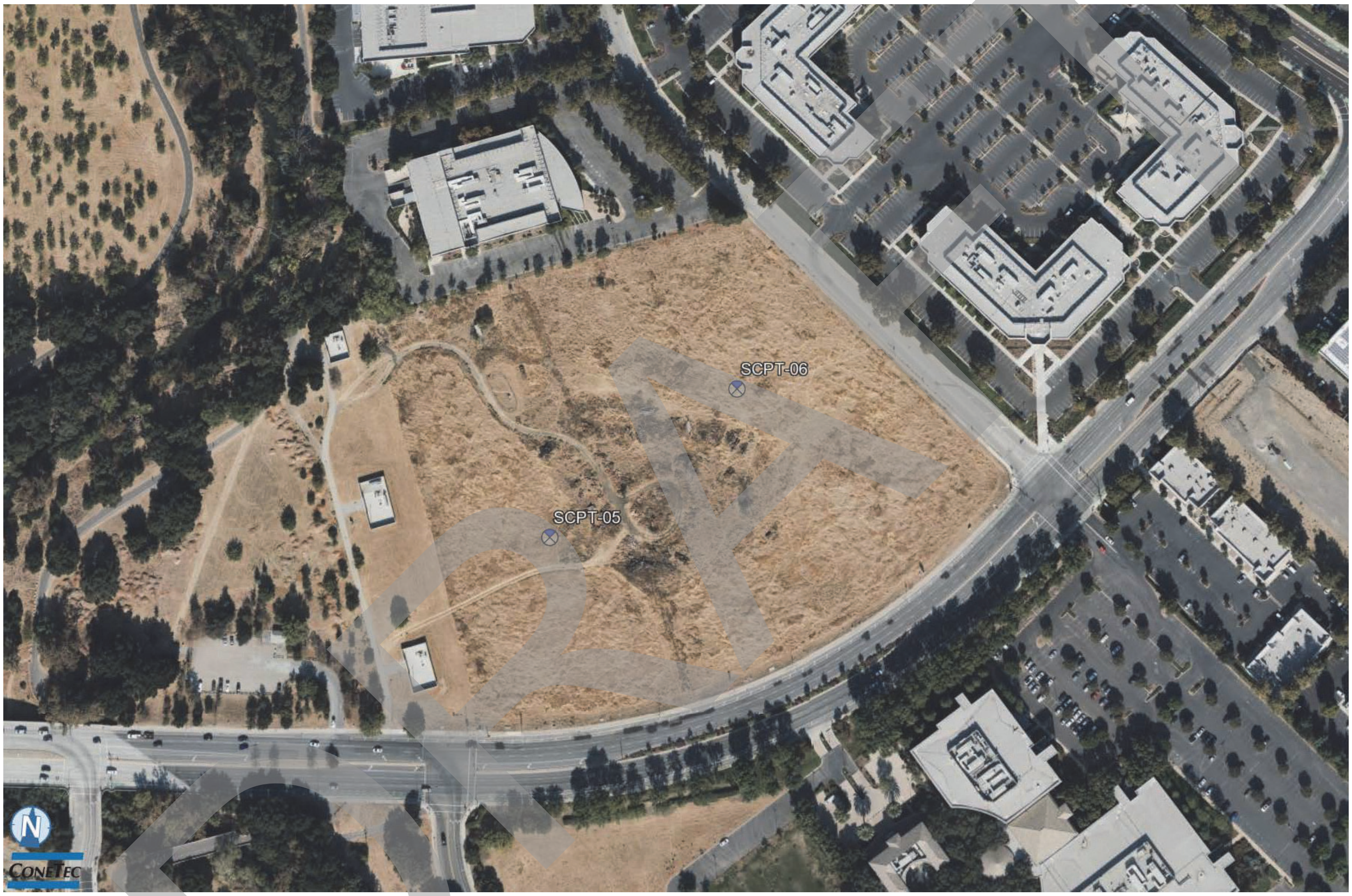
- ConeTec was retained by Kleinfelder
- The “Report” refers to this report titled Duke Warehouse Silver Creek
- ConeTec was retained to collect and provide the raw data (“Data”) which is included in the Report.

ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

CONTENTS

The following listed below are included in the report:

SITE MAP



ConeTec Job Number: 21-56-22781

Client: Kleinfelder

Project: Duke Warehouse Silver Creek

Report Date: 2022-Jan-25

Sounding Location

All sounding locations are approximate

CONETEC

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

CONETEC

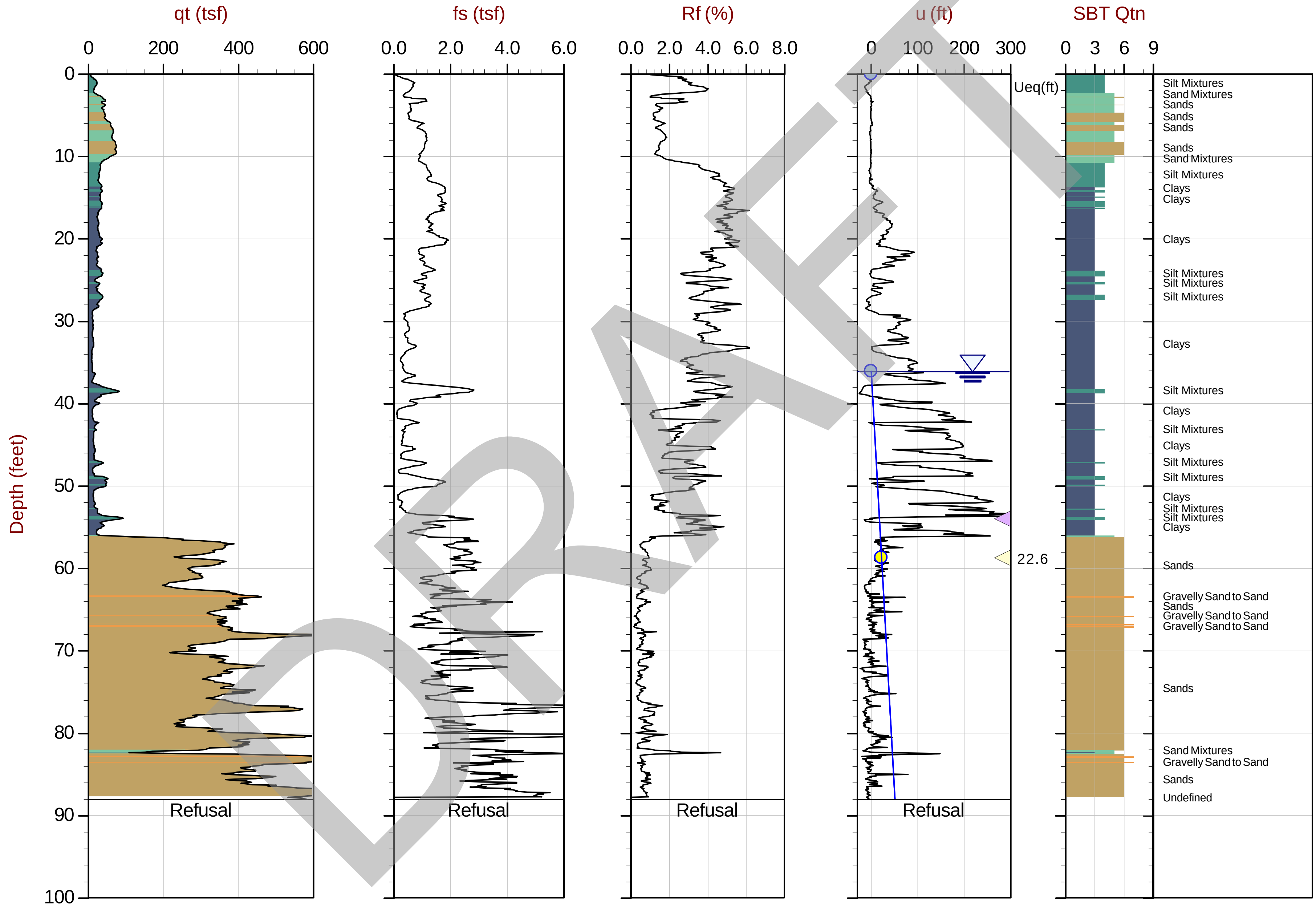
Job No: 21-56-22781
Client: Kleinfelder
Project: Duke Warehouse Silver Creek
Start Date: 20-Jan-2022
End Date: 20-Jan-2022

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2]	Easting[2]	Elevation[3] (ft)	Refer to Notation Number
SCPT-05	21-56-22781_SP05	20-Jan-2022	EC741:T1500F15U35	15	36.1	88.09	4124201	607401	204	
SCPT-06	21-56-22781_SP06	20-Jan-2022	EC741:T1500F15U35	15	29.6	61.84	4124281	607496	202	

1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS equipment. EPSG number: 32610 (WGS84 / UTM Zone 10S).
3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.

Job No: 21-56-22781
Date: 2022-01-20 07:58
Site: Duke Warehouse Silver Creek
Sounding: SCPT-05
Cone: 741:T1500F15U35



Max Depth: 26.850 m / 88.09 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124201m E: 607401m

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

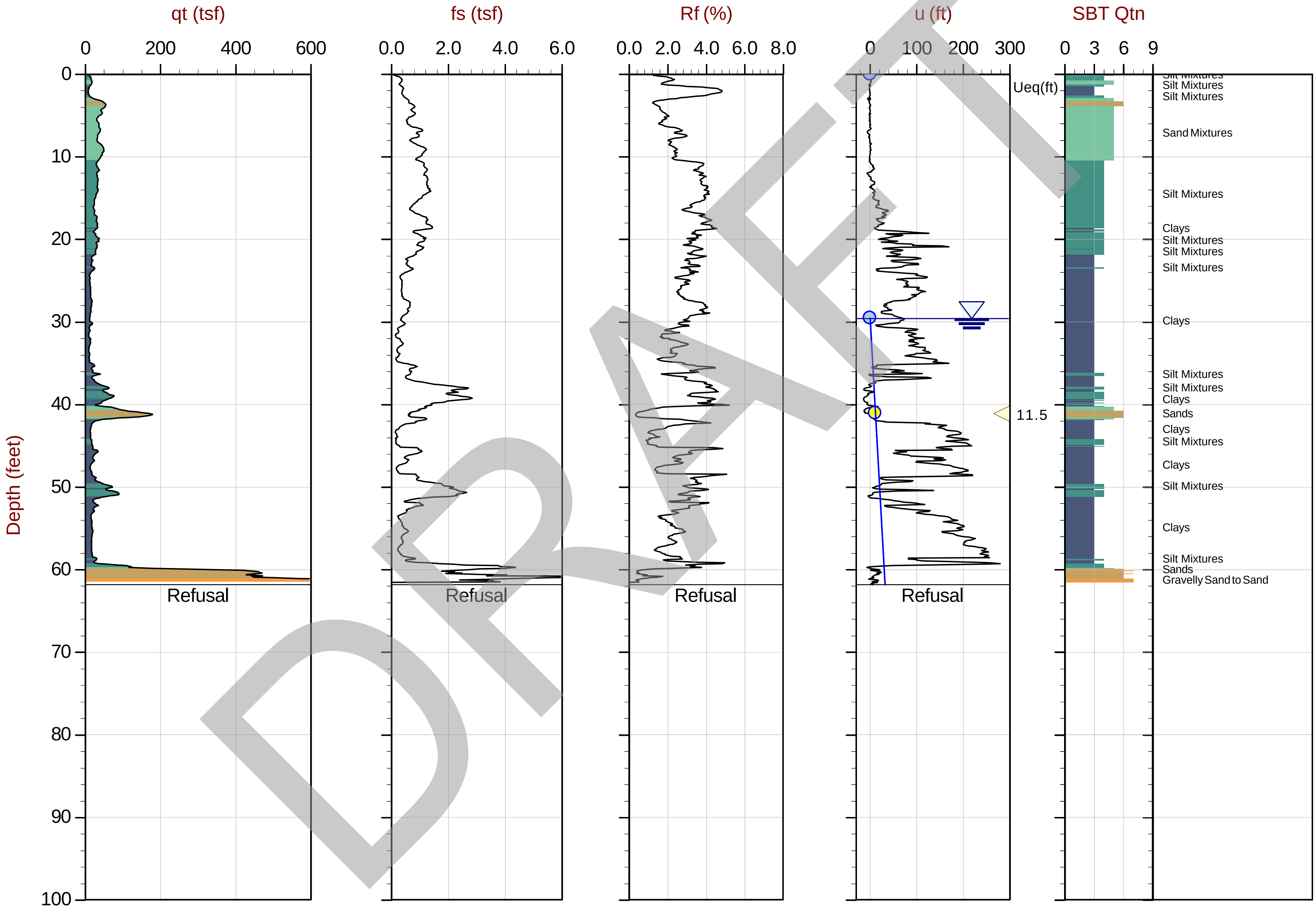
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 2022-01-20 09:45
Site: Duke Warehouse Silver Creek

Sounding: SCPT-06
Cone: 741:T1500F15U35



Max Depth: 18.850 m / 61.84 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124281m E: 607496m

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.

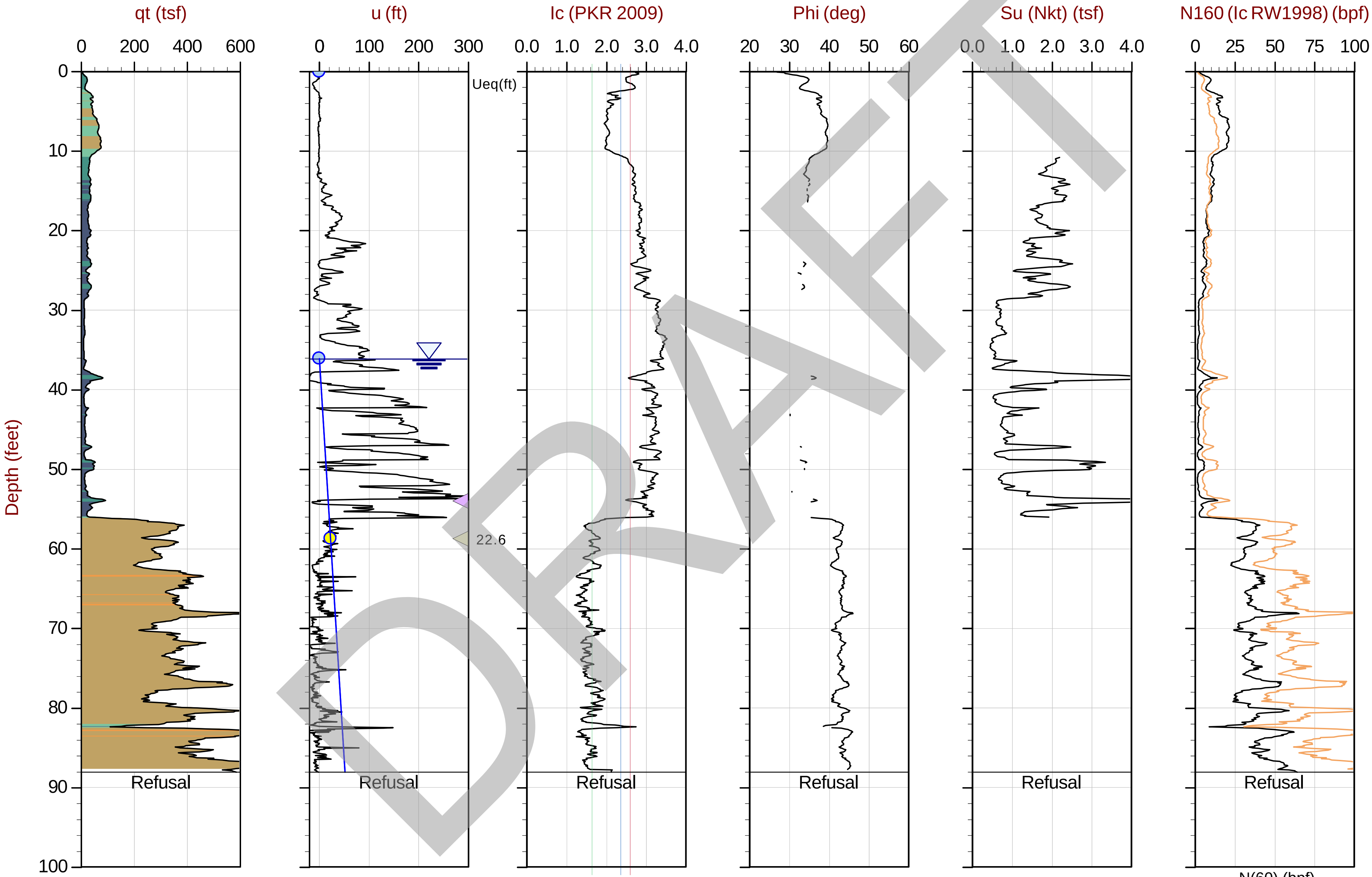
Advanced Cone Penetration Test Plots

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 2022-01-20 07:58
Site: Duke Warehouse Silver Creek

Sounding: SCPT-05
Cone: 741:T1500F15U35



Max Depth: 26.850 m / 88.09 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

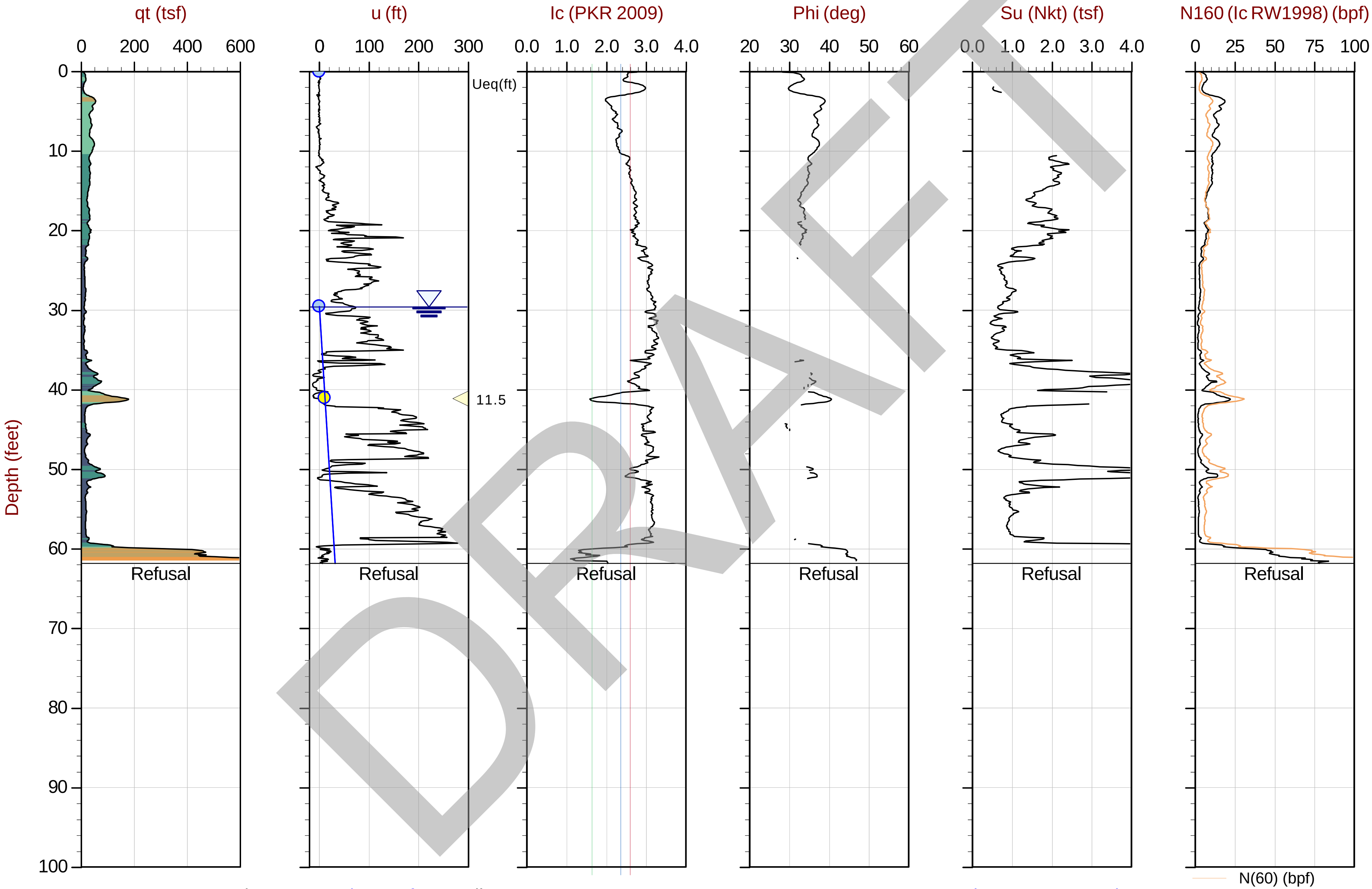
SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124201m E: 607401m

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.

Kleinfelder

Job No: 21-56-22781 Sounding: SCPT-06
Date: 2022-01-20 09:45 Cone: 741:T1500F15U35
Site: Duke Warehouse Silver Creek



Max Depth: 18.850 m / 61.84 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124281m E: 607496m

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

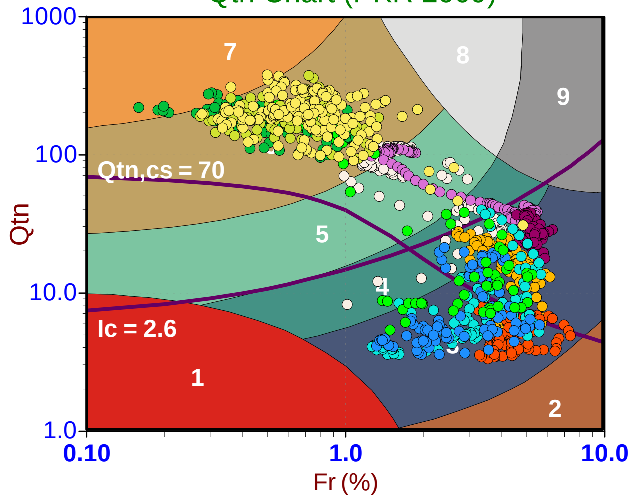
ConeTec

Kleinfelder

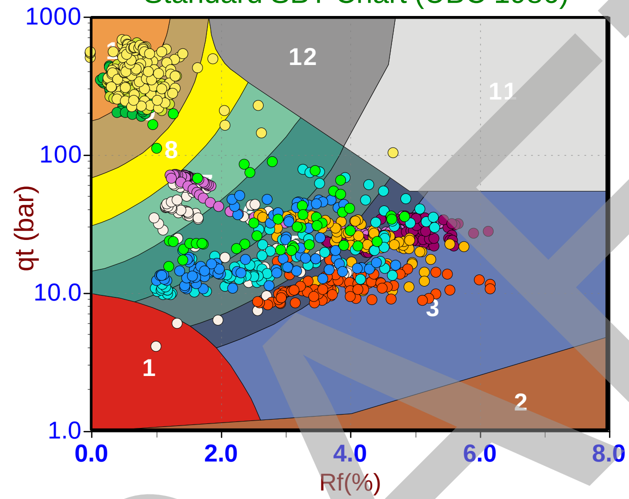
Job No: 21-56-22781
Date: 2022-01-20 07:58
Site: Duke Warehouse Silver Creek

Sounding: SCPT-05
Cone: 741:T1500F15U35

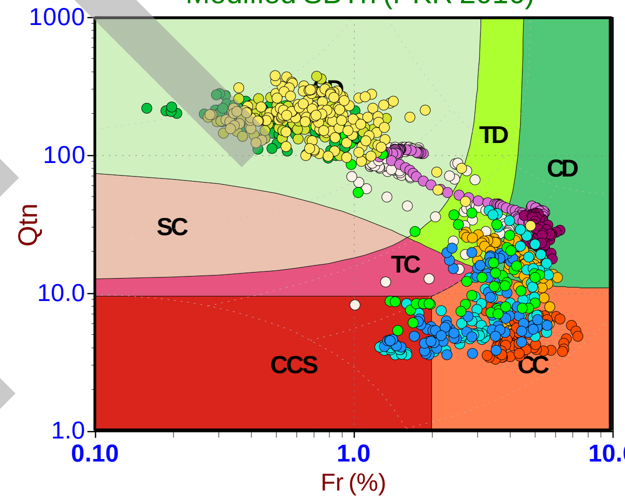
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 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

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

Legend

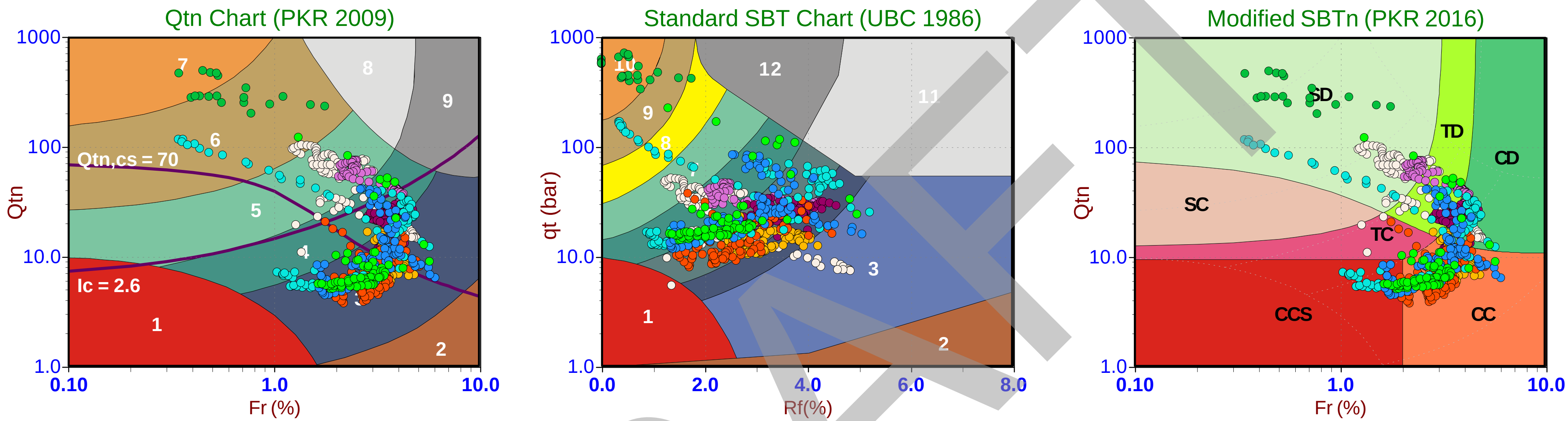
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 2022-01-20 09:45
Site: Duke Warehouse Silver Creek

Sounding: SCPT-06
Cone: 741:T1500F15U35



Depth Ranges
- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 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
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

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)

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

CONETEC	**Job No:**	21-56-22781
	Client:	Kleinfelder
	Project:	Duke Warehouse Silver Creek
	Start Date:	20-Jan-2022
	End Date:	20-Jan-2022

CPTu PORE PRESSURE DISSIPATION SUMMARY

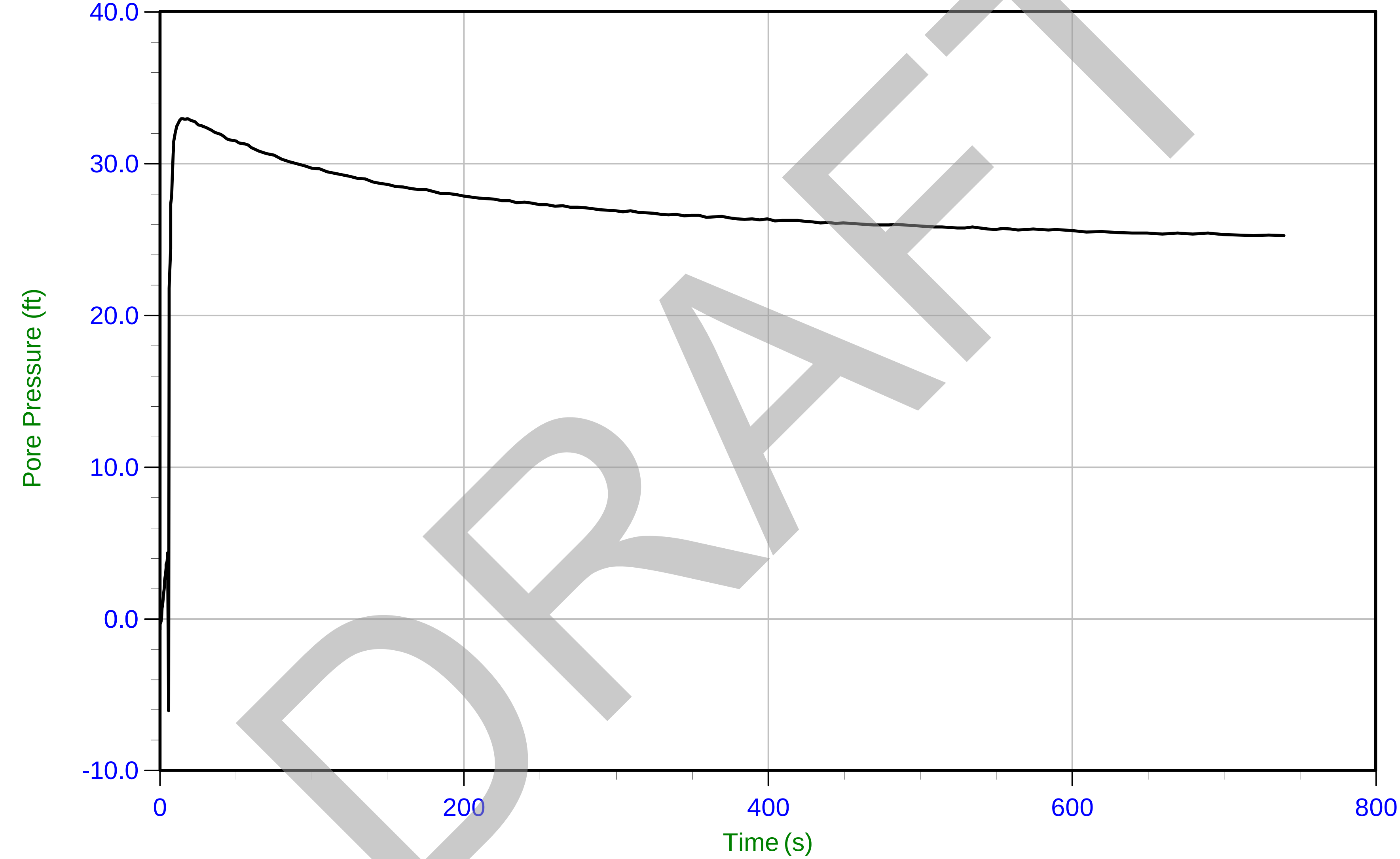
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft.)	Calculated Phreatic Surface (ft.)
SCPT-05	21-56-22781_SP05	15	740	53.97	Not Achieved	
SCPT-05	21-56-22781_SP05	15	300	58.73	22.6	36.1
SCPT-06	21-56-22781_SP06	15	385	41.09	11.5	29.6

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 01/20/2022 07:58
Site: Duke Warehouse Silver Creek

Sounding: SCPT-05
Cone: 741:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-56-22781_SP05.ppd2
Depth: 16.450 m / 53.969 ft
Duration: 739.9 s

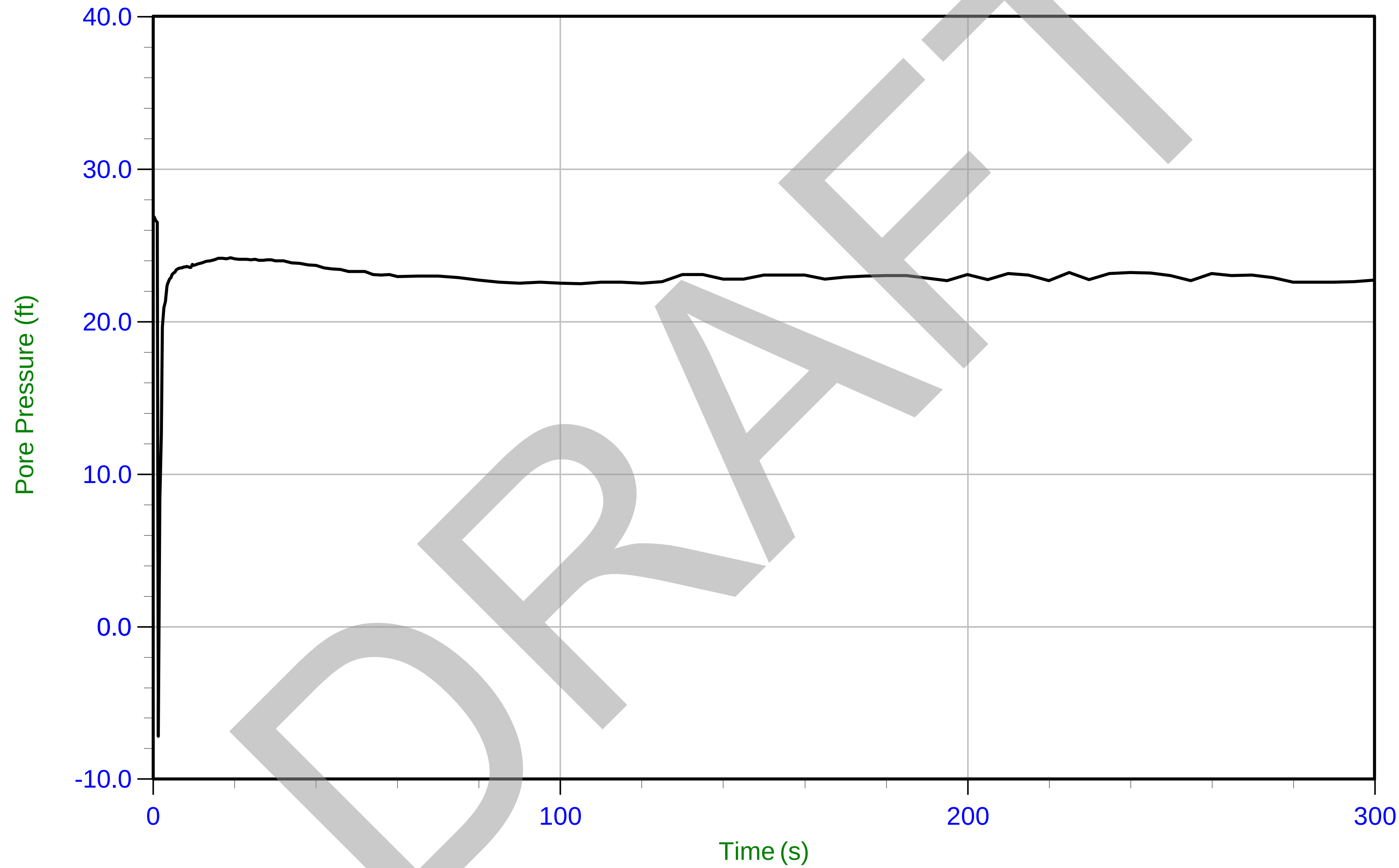
u Min: -6.1 ft
u Max: 32.9 ft
u Final: 25.2 ft

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 01/20/2022 07:58
Site: Duke Warehouse Silver Creek

Sounding: SCPT-05
Cone: 741:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-56-22781_SP05.ppd2
Depth: 17.900 m / 58.726 ft
Duration: 299.9 s

u Min: -7.2 ft
u Max: 26.9 ft
u Final: 22.7 ft

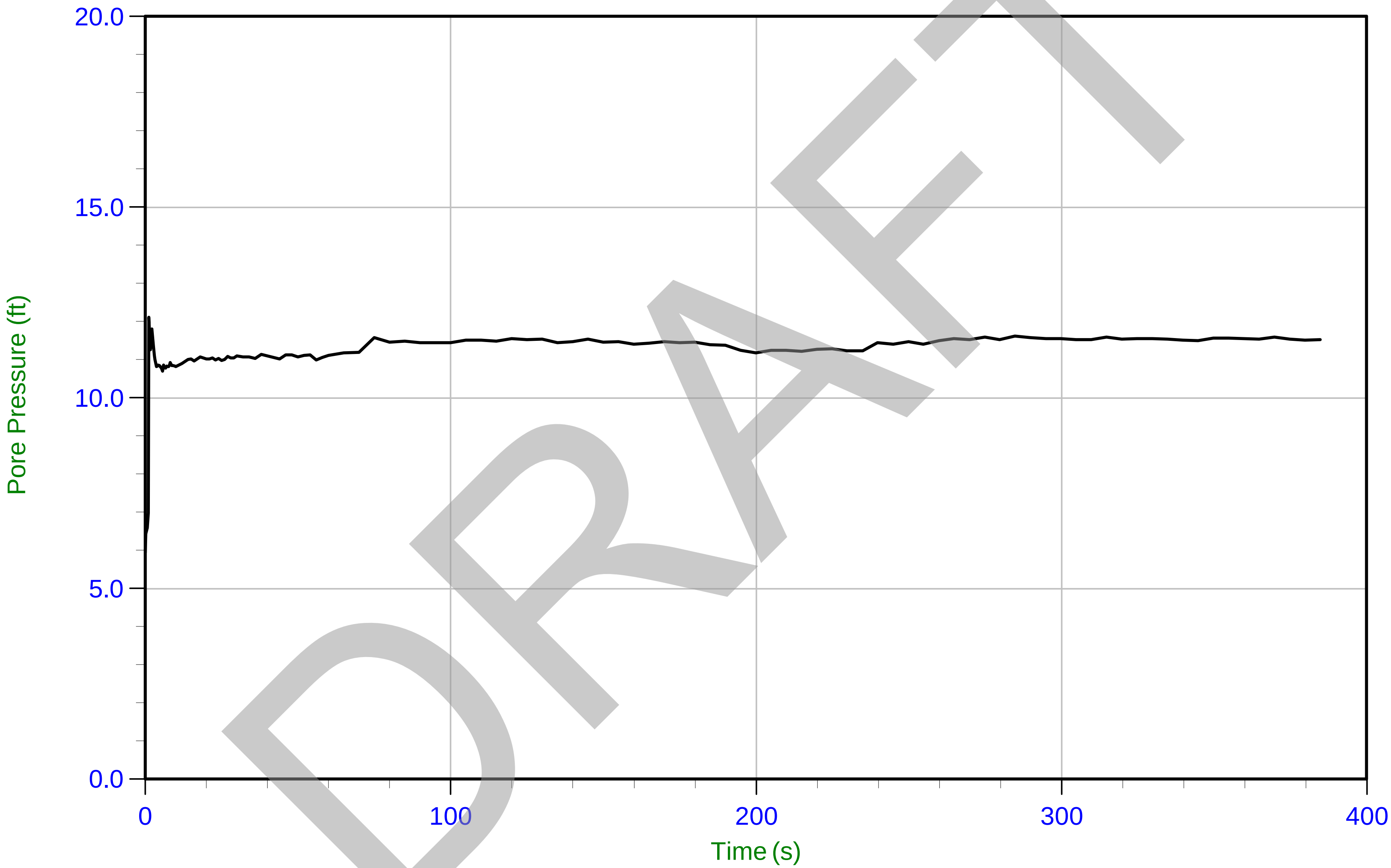
WT: 11.005 m / 36.106 ft
Ueq: 22.6 ft

ConeTec

Kleinfelder

Job No: 21-56-22781
Date: 01/20/2022 09:45
Site: Duke Warehouse Silver Creek

Sounding: SCPT-06
Cone: 741:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-56-22781_SP06.ppd2
Depth: 12.525 m / 41.092 ft
Duration: 384.9 s

u Min: 5.9 ft
u Max: 12.1 ft
u Final: 11.5 ft

WT: 9.017 m / 29.584 ft
Ueq: 11.5 ft

Seismic Cone Penetration Test Tabular Results

Job No: 21-56-22781
Client: Kleinfelder
Project: Duke Warehouse Silver Creek
Sounding ID: SCPT-05
Date: 01:20:22 07:58

Seismic Source: Beam
Seismic Offset (ft): 1.87
Source Depth (ft): 0.00
Geophone Offset (ft): 0.81

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)
2.95	2.14	2.84			
6.30	5.49	5.80	2.96	5.91	500
9.51	8.70	8.90	3.10	3.90	796
12.73	11.92	12.06	3.16	4.10	772
16.08	15.26	15.38	3.31	4.15	800
19.29	18.48	18.57	3.20	3.80	840
22.57	21.76	21.84	3.27	3.71	882
26.02	25.21	25.27	3.43	3.53	972
29.20	28.39	28.45	3.18	3.93	809
32.42	31.60	31.66	3.21	4.39	731
39.04	38.23	38.28	6.62	8.27	800
42.26	41.45	41.49	3.21	4.32	744
45.54	44.73	44.77	3.28	4.25	772
48.82	48.01	48.04	3.28	4.34	755
52.10	51.29	51.32	3.28	4.07	805
58.73	57.92	57.95	6.62	6.18	1071
61.94	61.13	61.16	3.21	2.46	1309
65.22	64.41	64.44	3.28	2.80	1173
68.57	67.76	67.78	3.35	2.63	1271
71.85	71.04	71.06	3.28	2.59	1264
75.13	74.32	74.34	3.28	2.31	1419
78.41	77.60	77.62	3.28	2.48	1323
81.69	80.88	80.90	3.28	2.21	1485
84.97	84.16	84.18	3.28	1.69	1937
88.09	87.28	87.30	3.12	2.36	1323

	Job No:	21-56-22781
	Client:	Kleinfelder
	Project:	Duke Warehouse Silver Creek
	Sounding ID:	SCPT-06
	Date:	01:20:22 09:45
	Seismic Source:	Beam
	Seismic Offset (ft):	1.87
	Source Depth (ft):	0.00
	Geophone Offset (ft):	0.81

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)
2.89	2.08	2.79			
6.07	5.26	5.58	2.79	5.62	496
9.45	8.64	8.84	3.26	3.90	836
12.63	11.82	11.97	3.13	4.31	725
16.01	15.20	15.31	3.35	4.04	828
19.29	18.48	18.57	3.26	3.72	876
22.47	21.66	21.74	3.17	3.48	911
25.75	24.94	25.01	3.27	3.70	884
29.04	28.22	28.29	3.27	3.62	903
32.32	31.50	31.56	3.27	3.62	905
35.70	34.88	34.93	3.38	3.62	933
38.88	38.07	38.11	3.18	3.79	838
42.16	41.35	41.39	3.28	3.35	978
45.54	44.73	44.77	3.38	4.50	751
48.72	47.91	47.94	3.18	4.35	731
52.17	51.35	51.39	3.44	4.06	849
55.38	54.57	54.60	3.21	4.41	728
58.56	57.75	57.78	3.18	3.72	855
61.84	61.03	61.06	3.28	3.18	1031

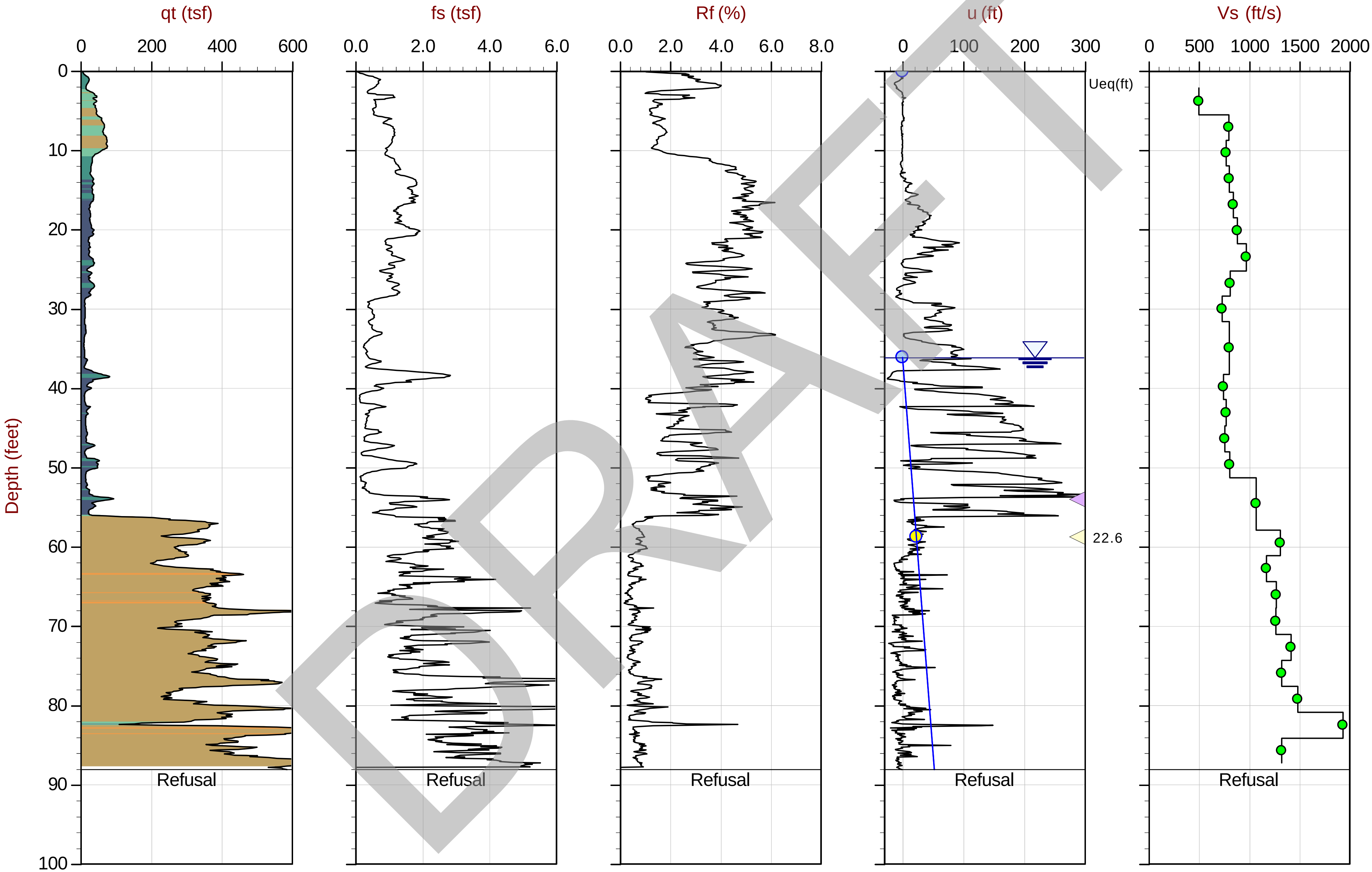
Seismic Cone Penetration Test Plots

CONETEC

Kleinfelder

Job No: 21-56-22781
Date: 2022-01-20 07:58
Site: Duke Warehouse Silver Creek

Sounding: SCPT-05
Cone: 741:T1500F15U35



Max Depth: 26.850 m / 88.09 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124201m E: 607401m

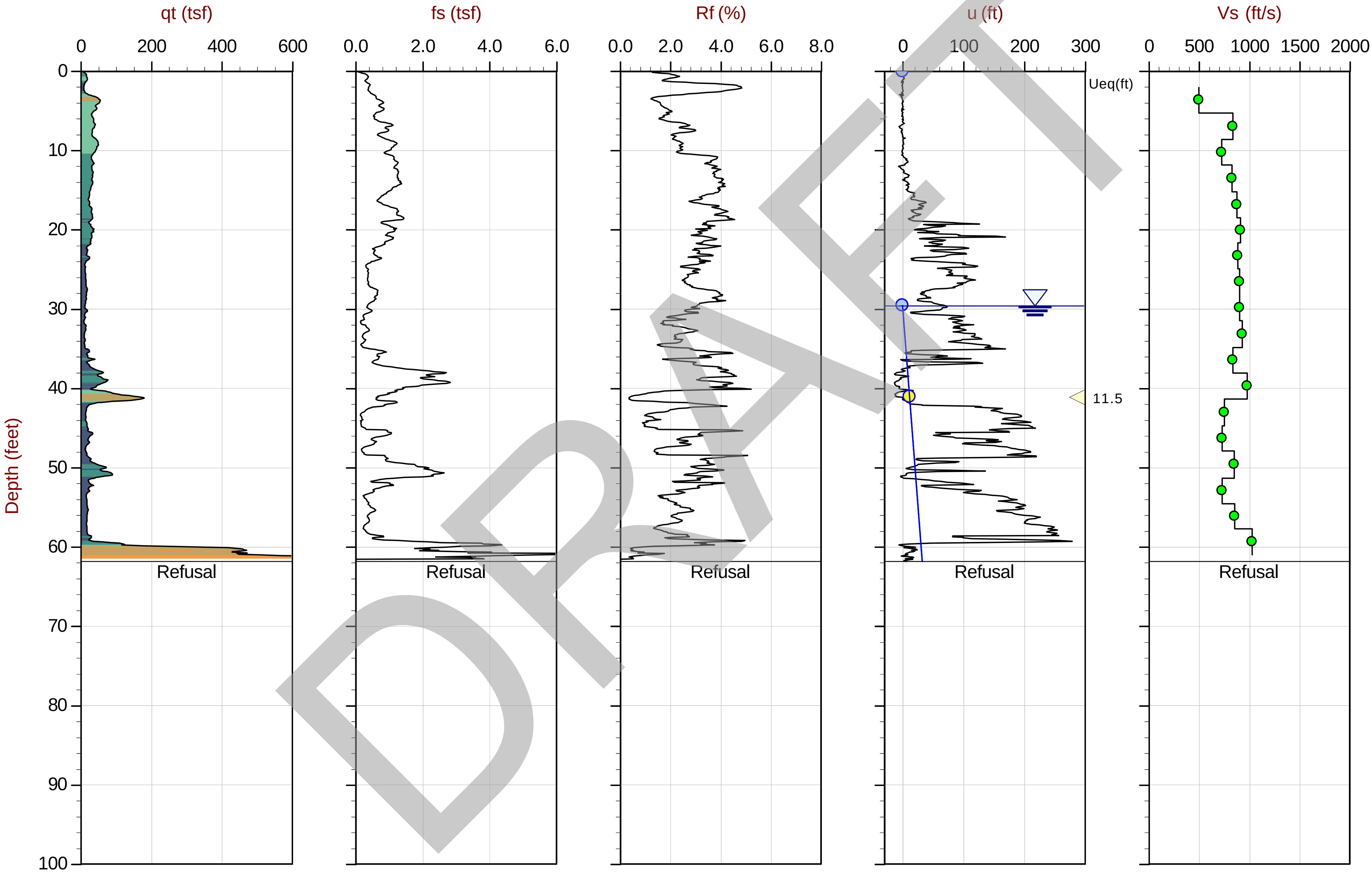
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

Kleinfelder

Job No: 21-56-22781
Date: 2022-01-20 09:45
Site: Duke Warehouse Silver Creek

Sounding: SCPT-06
Cone: 741:T1500F15U35



Max Depth: 18.850 m / 61.84 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22781_SP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4124281m E: 607496m

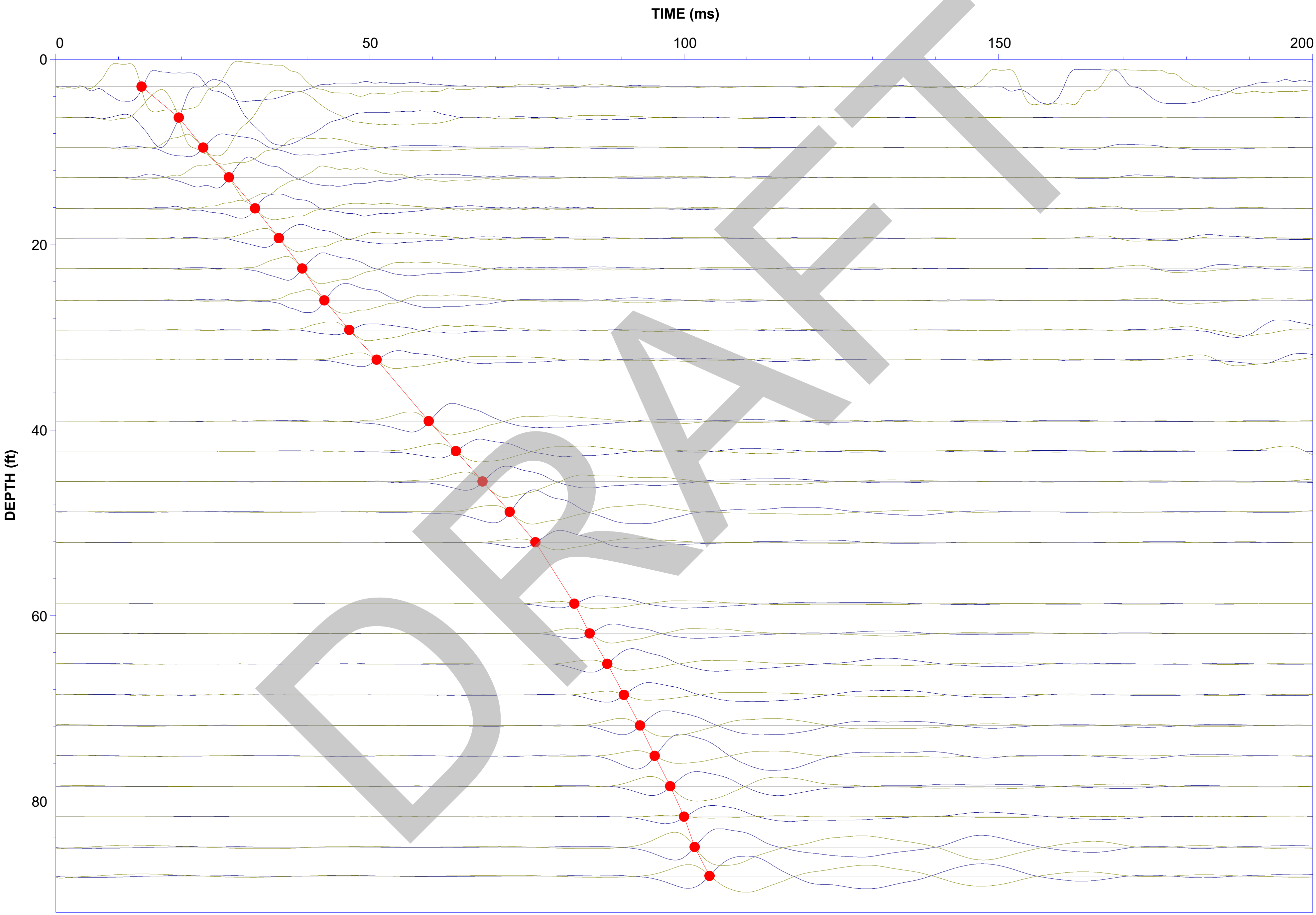
Equilibrium Pore Pressure (Ueq) · Assumed Ueq · Dissipation, Ueq achieved · Dissipation, Ueq not achieved · Hydrostatic Line

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

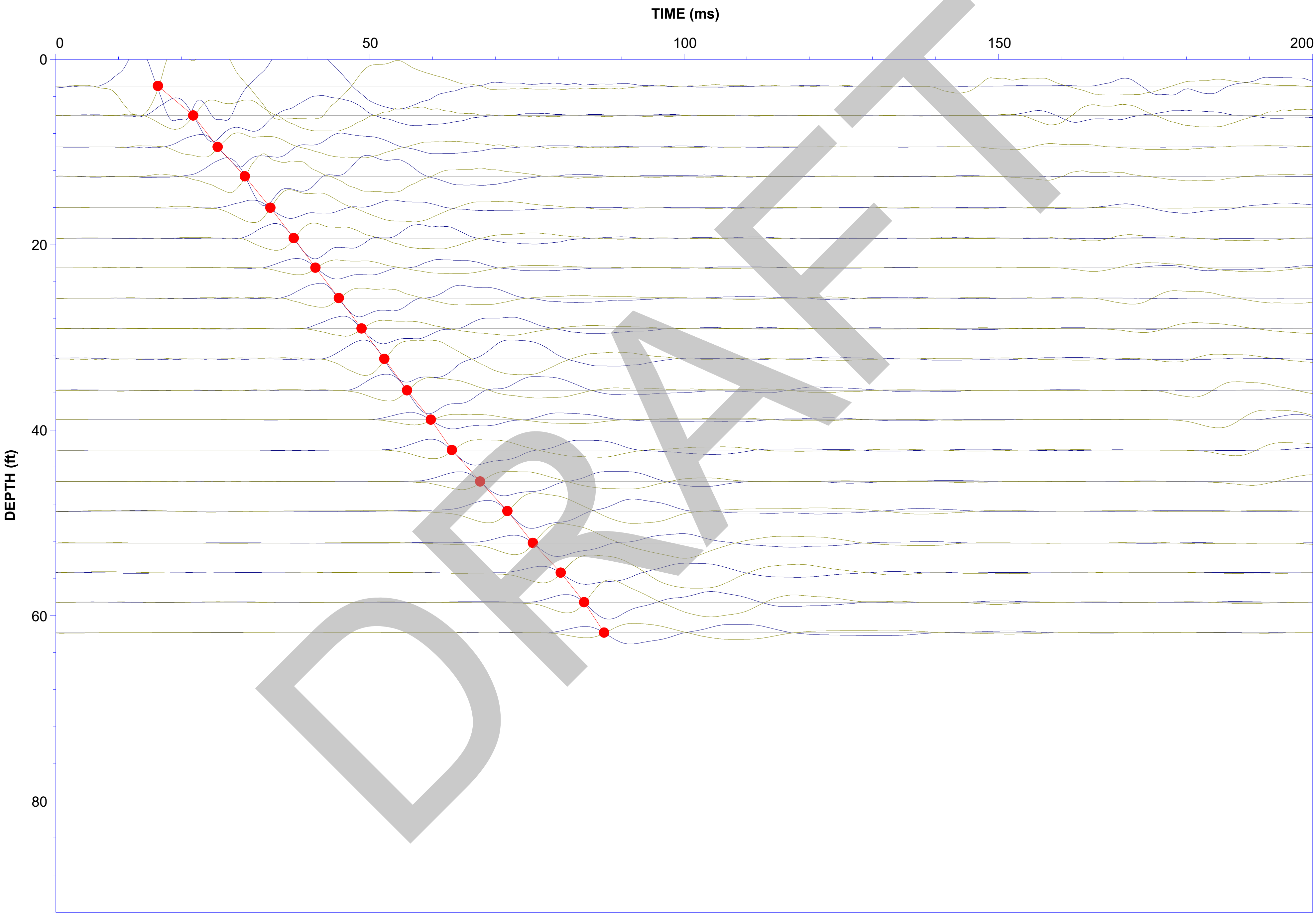
Seismic Cone Penetration Test Shear Wave (Vs) Traces

ConeTec

Job No: 21-56-22781 Cone: 741:T1500F15U35	Client: Kleinfelder	Project Title: Duke Warehouse Silver Creek	Filter: BP 0-500 Hz	Hole: SCPT-05	Date: 01:20:22 07:58



ConeTec	Job No: 21-56-22781 Cone: 741:T1500F15U35	Client: Kleinfelder	Project Title: Duke Warehouse Silver Creek	Filter: BP 0-500 Hz	Hole: SCPT-06	Date: 01:20:22 09:45



Methodology Statements and Data File Formats

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 5 cm^2, 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 forty-four millimeter diameter over a length of thirty-two millimeter with tapered leading and trailing edges) located at a distance of 585 millimeters 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 sixty-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 six millimeters 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 meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

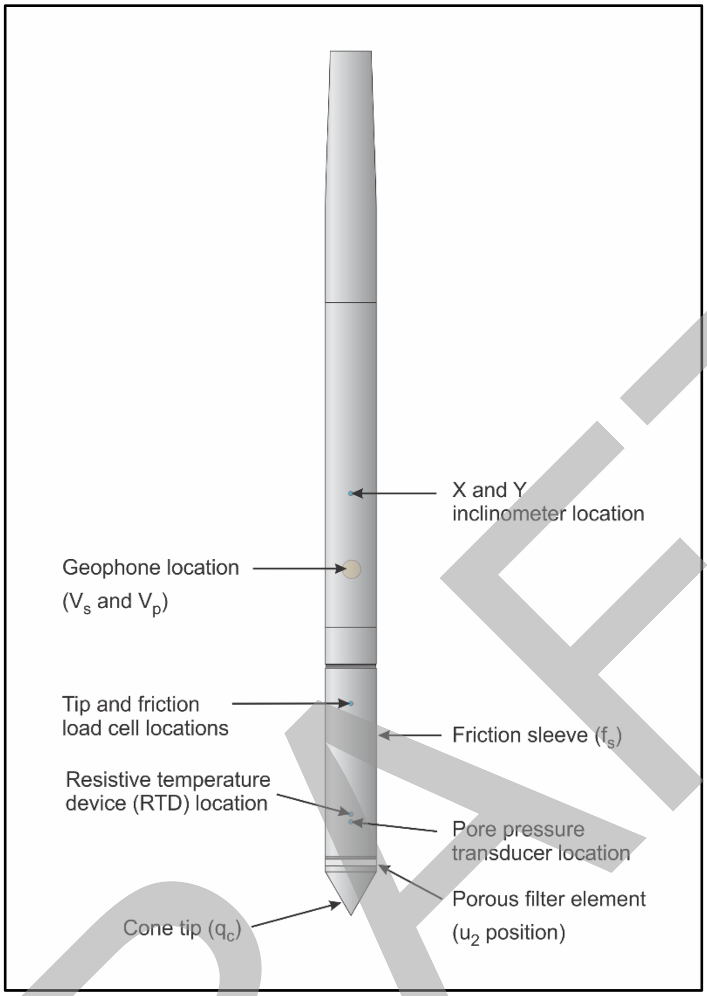


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a sixteen 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 centimeters; 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 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 two centimeters per second, within acceptable tolerances. Typically, one-meter length rods with an outer diameter of 1.5 inches (38.1 millimeters) 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 type 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 may be 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 uphole 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.

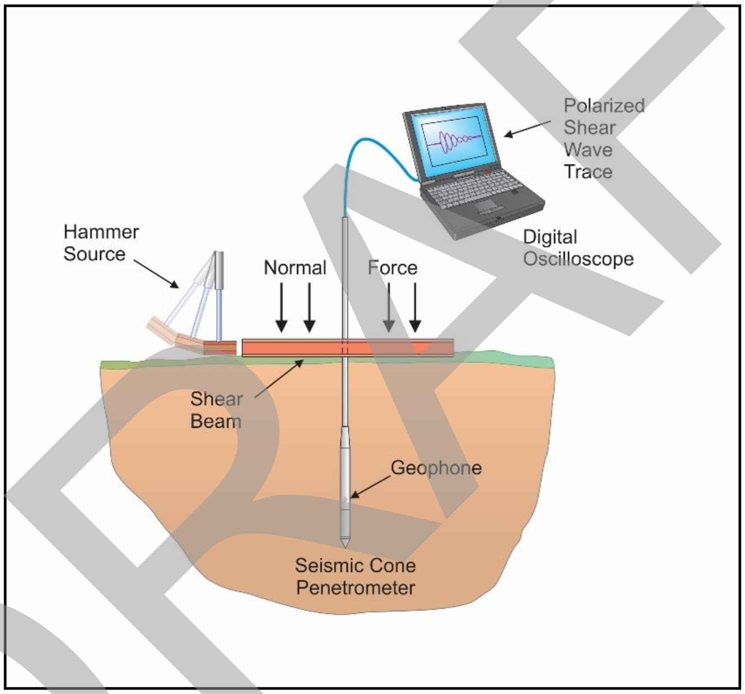


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM D5778 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 and uncertainty analysis purposes. 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).

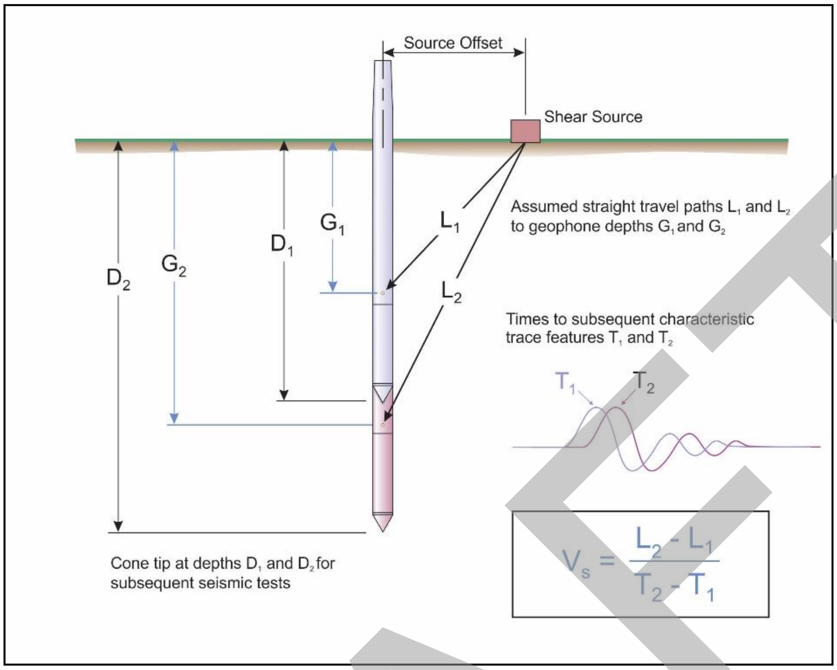


Figure SCPTu-2. Illustration of a seismic cone penetration test

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.

For all SCPTu soundings that have achieved a depth of at least 100 feet (30 meters), the average shear wave velocity to a depth of 100 feet ($\overline{v}_s$) has been calculated and provided for all applicable soundings using the following equation presented in ASCE (2010).

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

where: $\overline{v}_s$ = 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)

$\sum_{i=1}^{n} d_i$ = the total thickness of all layers between 0 and 100 ft (30 m)

Average shear wave velocity, $\overline{v}_s$ 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).

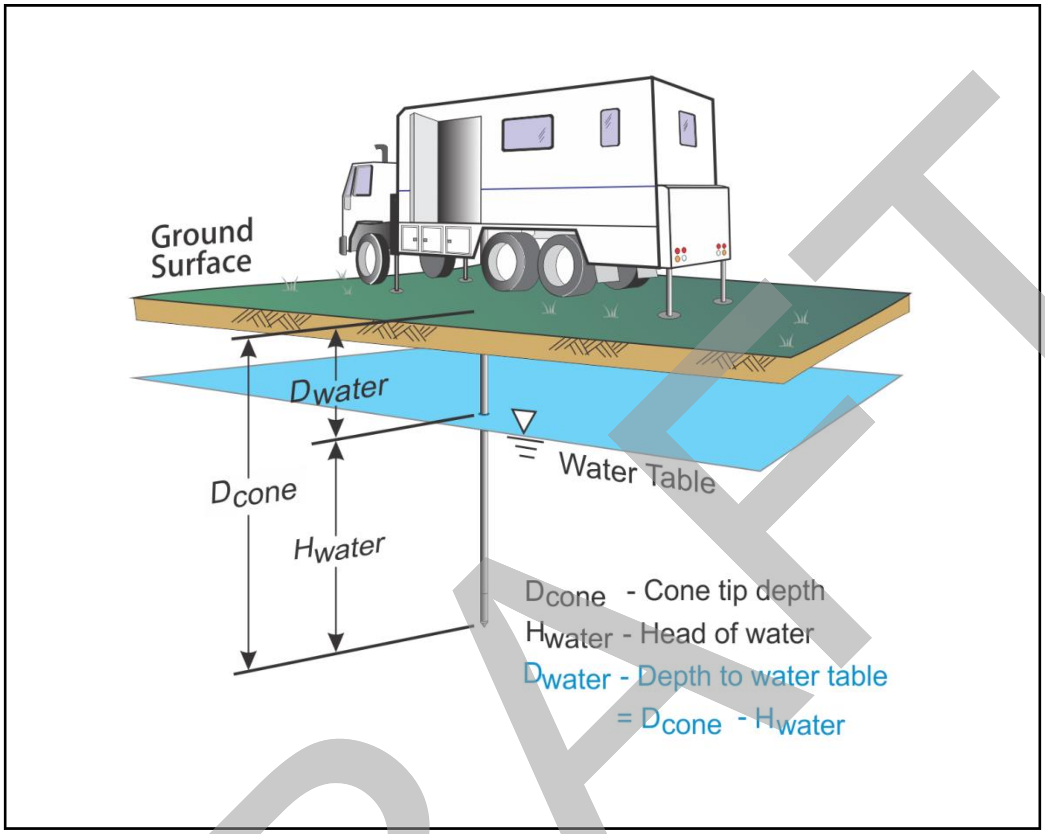


Figure PPD-1. Pore pressure dissipation test setup

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.

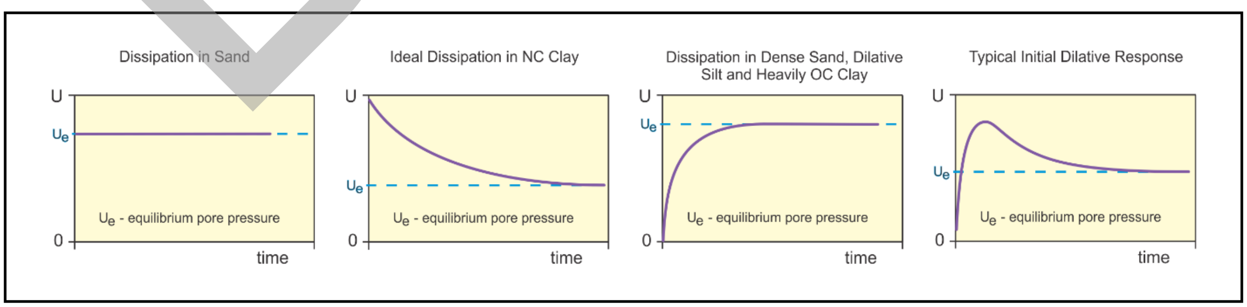


Figure PPD-2. Pore pressure dissipation curve examples

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.

American Society of Civil Engineers (ASCE), 2010, “Minimum Design Loads for Buildings and Other Structures”, Standard ASCE/SEI 7-10, American Society of Civil Engineers, ISBN 978-0-7844-1085-1, Reston, Virginia. DOI: 10.1061/9780784412916.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-12.

ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM International, West Conshohocken, PA. DOI: 10.1520/D7400_D7400M-19.

Burns, S.E. and Mayne, P.W., 1998, “Monotonic and dilatory pore pressure decay during piezocone tests”, Canadian Geotechnical Journal 26 (4): 1063-1073. DOI: 1063-1073/T98-062.

Burns, S.E. and Mayne, P.W., 2002, “Analytical cavity expansion-critical state model cone dissipation in fine-grained soils”, Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “Cone Penetration Testing in Geotechnical Practice”, Blackie Academic and Professional.

Mayne, P.W., 2013, “Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests”, Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: 10.1061/9780784412770.027.

Mayne, P.W. and Peuchen, J., 2012, “Unit weight trends with cone resistance in soft to firm clays”, Geotechnical and Geophysical Site Characterization *4*, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, “Interpretation of geotechnical parameters from seismic piezocone tests”, CPT’14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

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. DOI: 10.1061/(ASCE)0733-9410(1986)112:8(791).

Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158. DOI: 10.1139/T90-014.

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): 539-550. DOI: 10.1139/T92-061.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

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. DOI: 10.1139/T98-105.

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", Geotechnique, 41(1): 17-34. DOI: 10.1680/geot.1991.41.1.17.

CONE PENETRATION DIGITAL FILE FORMATS - eSeries

CPT Data Files (COR Extension)

ConeTec CPT data files are stored in ASCII text files that are readable by almost any text editor. ConeTec file names start with the job number (which includes the two digit year number) an underscore as a separating character, followed by two letters based on the type of test and the sounding ID. The last character position is reserved for an identifier letter (such as b, c, d etc) used to uniquely distinguish multiple soundings at the same location. The CPT sounding file has the extension COR. As an example, for job number 21-02-00001 the first CPT sounding will have file name 21-02-00001_CP01.COR

The sounding (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. End of data marker
4. Units information

Header Lines

Line 1: Columns 1-6 may be blank or may indicate the version number of the recording software
Columns 7-21 contain the sounding Date and Time (Date is MM:DD:YY)
Columns 23-38 contain the sounding Operator
Columns 51-100 contain extended Job Location information

Line 2: Columns 1-16 contain the Job Location
Columns 17-32 contain the Cone ID
Columns 33-47 contain the sounding number
Columns 51-100 may contain extended sounding ID information

Data Records

The data records contain 4 or more columns of data in floating point format. A comma and spaces separate each data item:

Column 1: Sounding Depth (meters)
Column 2: Tip (q_c), recorded in units selected by the operator
Column 3: Sleeve (f_s), recorded in units selected by the operator
Column 4: Dynamic pore pressure (u), recorded in units selected by the operator
Column 5: Empty or may contain other requested data such as Gamma, Resistivity or UVIF data

End of Data Marker

After the last line of data there is a line containing an ASCII 26 (CTL-Z) character (small rectangular shaped character) followed by a newline (carriage return / line feed). This is used to mark the end of data.

Units Information

The last section of the file contains information about the units that were selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth, q_c, f_s and u. The third line contains the conversion values required for ConeTec's software to convert the recorded data to an internal set of base units (bar for q_c, bar for f_s and meters for u). Additional lines intended for internal ConeTec use may appear following the conversion values.

CPT Data Files (XLS Extension)

Excel format files of ConeTec CPT data are also generated from corresponding COR files. The XLS files have the same base file name as the COR file with a -BSC suffix. The information in the file is presented in table format and contains additional information about the sounding such as coordinate information, and tip net area ratio.

The BSCI suffix is given to XLS files which are enhanced versions of the BSC files and include the same data records in addition to inclination data collected for each sounding.

CPT Dissipation Files (XLS Extension)

Pore pressure dissipation files are provided in Excel format and contain each dissipation trace that exceeds a minimum duration (selected during post-processing) formatted column wise within the spreadsheet. The first column (Column A) contains the time in seconds and the second column (Column B) contains the time in minutes. Subsequent columns contain the dissipation trace data. The columns extend to the longest trace of the data set.

Detailed header information is provided at the top of the worksheet. The test depth in meters and feet, the number of points in the trace and the particular units are all presented at the top of each trace column.

CPT Dissipation files have the same naming convention as the CPT sounding files with a "–PPD" suffix.

Data Records

Each file will contain dissipation traces that exceed a minimum duration (selected during post-processing) in a particular column. The dissipation pore pressure values are typically recorded at varying time intervals throughout the trace; rapidly to start and increasing as the duration of the test lengthens. The test depth in meters and feet, the number of points in the trace and the trace number are identified at the top of each trace column.

Cone Type Designations

Cone ID	Cone Description	Tip Cross Sect. Area (cm^2)	Tip Capacity (bar)	Sleeve Area (cm^2)**	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
EC###	A15T1500F15U35	15	1500	225	15	35
EC###	A15T375F10U35	15	375	225	10	35
EC###	A10T1000F10U35	10	1000	150	10	35

refers to the Cone ID number
**Outer Cylindrical Area

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software

Revision SZW-Rev 14

Revised November 26, 2019
Prepared by Jim Greig, M.A.Sc, P.Eng (BC)

Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

ConeTec's Calculated CPT Geotechnical Parameters as of November 26, 2019

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g. 0.20 m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. The corrected tip resistance (corrected using u_2 pore pressure values) is used for all of the calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not required.

The tip correction is: $q_t = q_c + (1\text{-}a) \bullet u_2$ (consistent units are implied)

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 cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

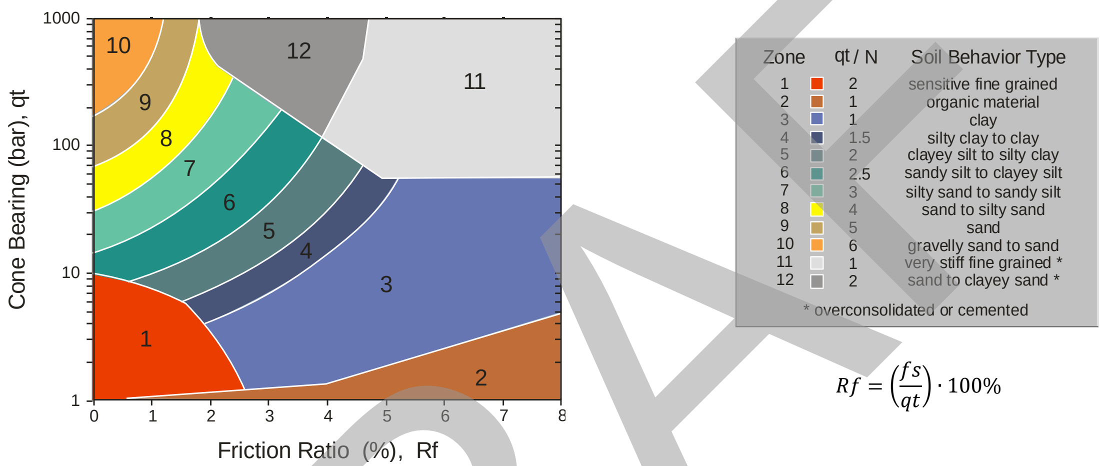
Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline have been taken into account as has the appropriate unit weight of water. How this is done depends on where the instruments were zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 5. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBT chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The

Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Please note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that used by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson.



$$Rf = \left(\frac{fs}{qt}\right) \cdot 100\%$$

Figure 1. Non-Normalized Soil Behavior Type Classification Chart (SBT)

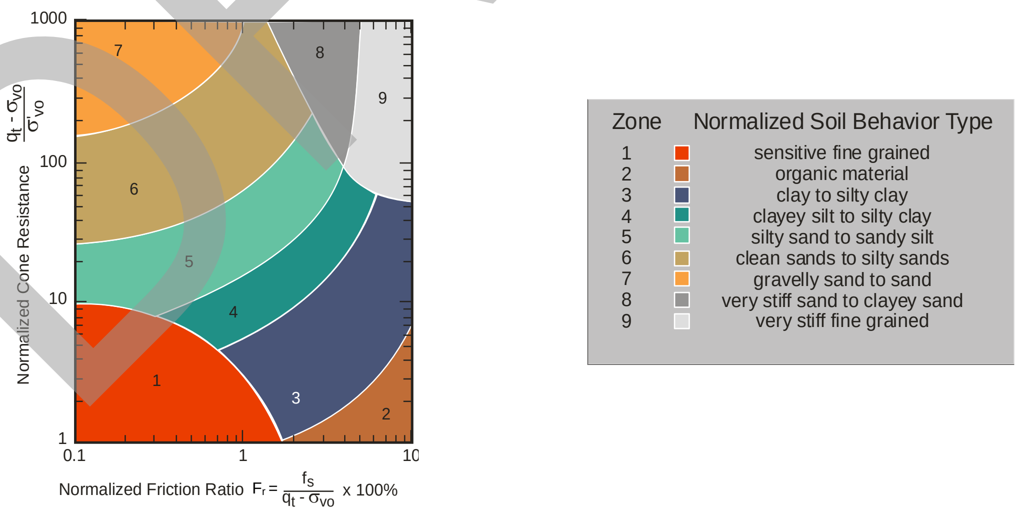


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

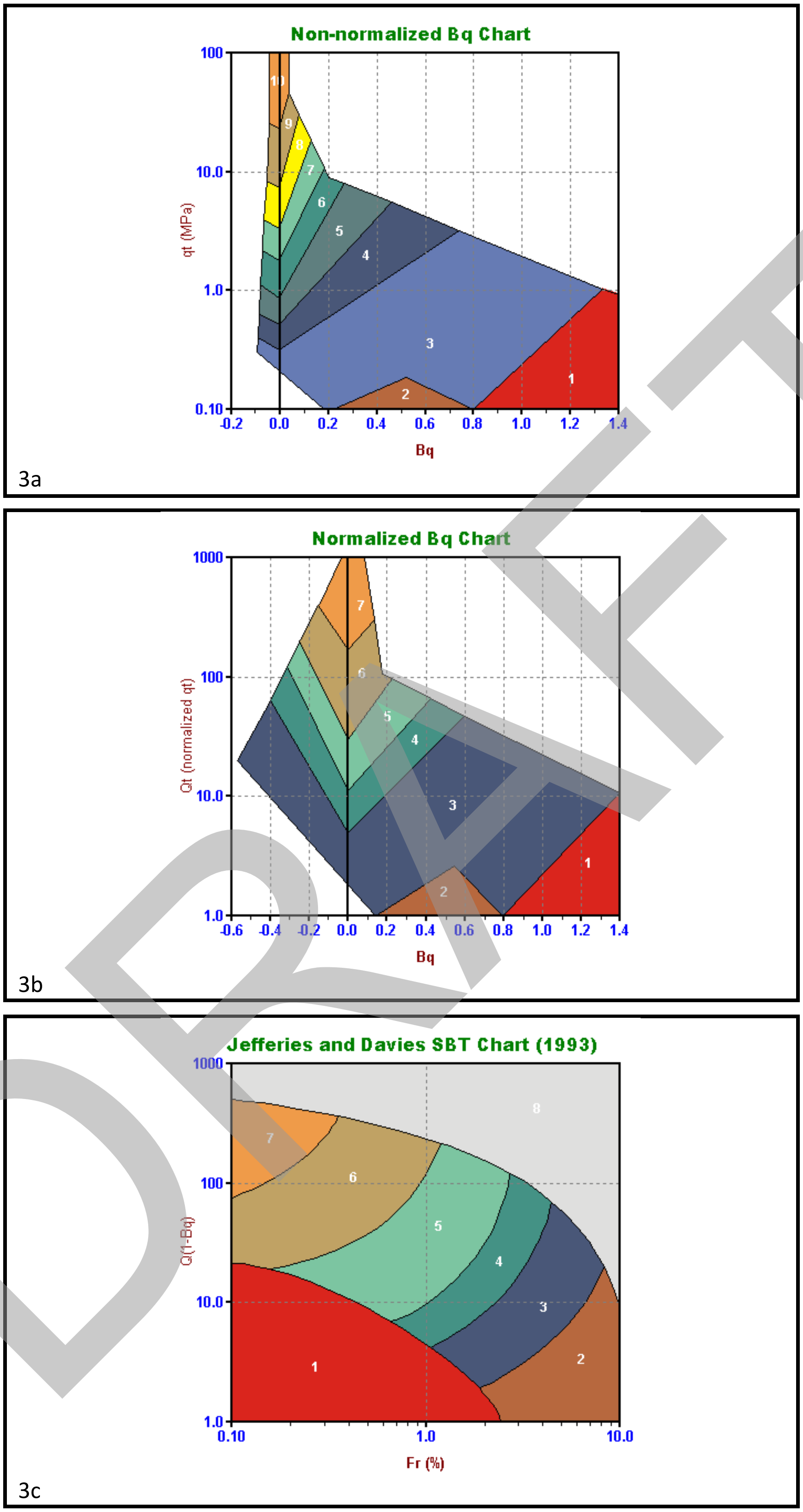


Figure 3. Alternate Soil Behavior Type Charts

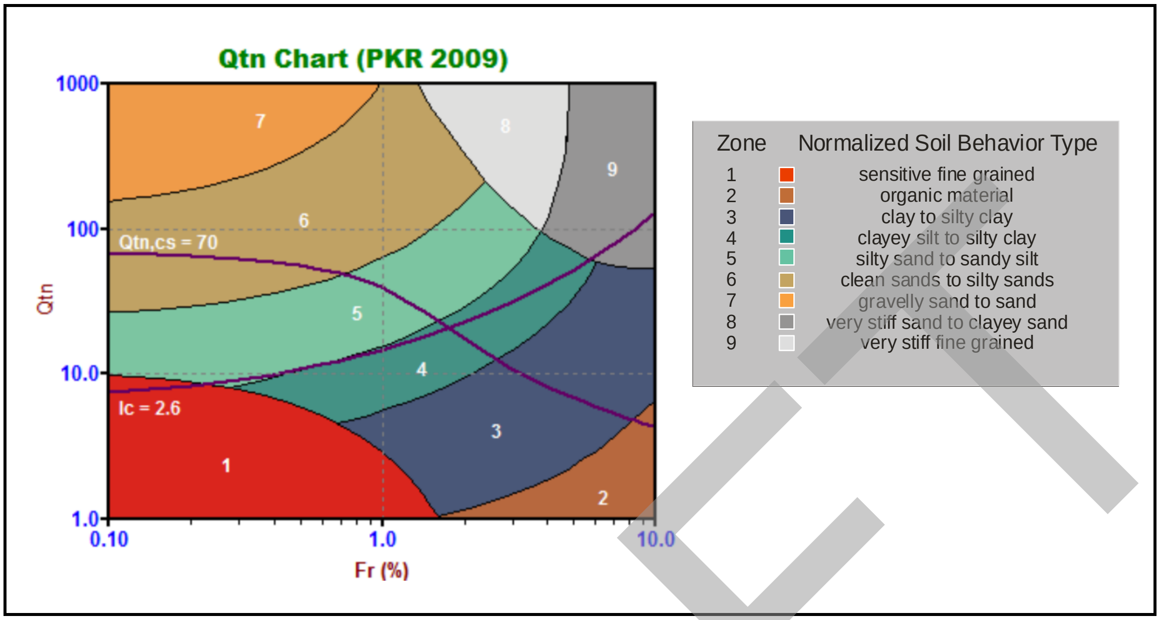


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

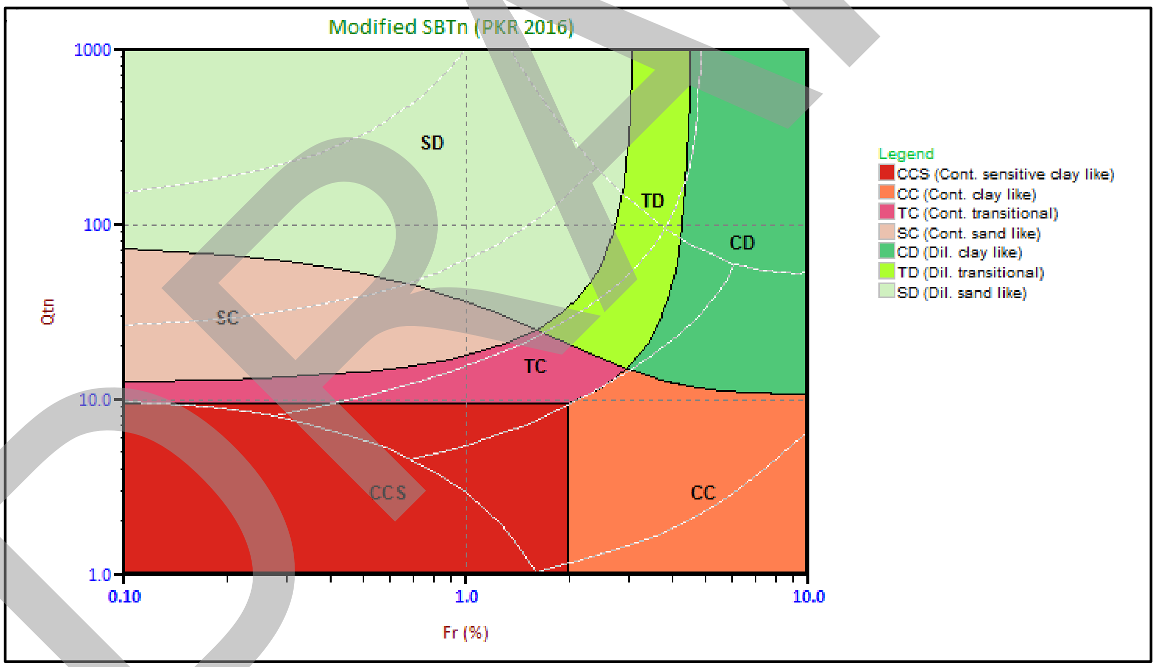


Figure 5. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary the user should refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed *'invalid'* the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g. drilled out section or data gap).
2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving as an undrained material (and vice versa).
3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.
4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Table 1 may be included in the output files delivered with this report.

The output files are typically provided in Microsoft Excel XLS or XLSX format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or requested by the client. Each output file is named using the original COR file base name followed by a three or four letter indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer based on sounding collar elevation supplied by client or through site survey	Elevation = Collar Elevation - Depth	CK*
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a) \bullet u_2$	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s)	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $Rf = 100\% \bullet \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *n=1 when calculations are done at each point*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*

Calculated Parameter	Description	Equation	Ref
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available since it requires specialized calibrations)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization)	See Figure 2	2, 5
SBT-Bq	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	1, 2, 5
SBT-Bqn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on I_c	See Figure 4	15
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior.	See Figure 5	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) Mayne fs (sleeve friction) method 7) Robertson 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options	See references	3, 5, 15, 21, 24, 29

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* *For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point.* *Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point.* *For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.*	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressure determined from one of the following user selectable options: 1) hydrostatic below water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point (“assumed value”) will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These “assumed” values will be indicated on our plots and in tabular summaries.	For hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0	$K_O = (1 - \sin\Phi') \, OCR^{\sin\Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters	$C_n = (P_a/\sigma_v')^{0.5}$ where $0.0 < C_n < 2.0$ *(user adjustable, typically 1.7)* P_a *is atmospheric pressure (100 kPa)*	12
C_q	Overburden stress normalizing factor	$C_q = 1.8 / (0.8 + (\sigma_v'/P_a))$ where $0.0 < C_q < 2.0$ *(user adjustable)* P_a *is atmospheric pressure (100 kPa)*	3, 12

Calculated Parameter	Description	Equation	Ref
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
(N1)60	SPT N_{60} value corrected for overburden pressure	$(N_1)_{60} = C_n \bullet N_{60}$	4
N60Ic	SPT N_{60} values based on the I_c parameter [as defined by Roberston and Wride 1998 (5), or by Robertson 2009 (15)].	$(q_t/P_a)/\ N_{60} = 8.5\ (1 - I_c/4.6)$ $(q_t/P_a)/\ N_{60} = 10^{\ (1.1268 - 0.2817Ic)}$ Pa being atmospheric pressure	5 15, 31
(N1)60Ic	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \bullet (N_{60}\, I_c)$ 2) $q_{c1n}/\ (N_1)_{60}I_c = 8.5\ (1 - I_c/4.6)$ 3) $(Q_{tn})/\ (N_1)_{60}I_c = 10^{\ (1.1268 - 0.2817Ic)}$	4 5 15, 31
Su or Su (Nkt)	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable	$Su = \dfrac{qt - \sigma_v}{N_{kt}}$	1, 5
Su or Su (Ndu)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable	$Su = \dfrac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
Dr	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Hokksund Sand c) Schmertmann (1978) d) Jamiolkowski (1985) - All Sands e) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods a through d) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods a through d are for sands and method e is for silts and clays): a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne e) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/qt	Differential pore pressure ratio (older parameter used before B_q was established)	$= \dfrac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	CK*
Bq	Pore pressure parameter	$Bq = \dfrac{\Delta u}{qt - \sigma_v}$ where : $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	1, 2, 5
Net qt or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	CK*
qe	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$qt - u_2$	CK*

Calculated Parameter	Description	Equation	Ref
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v'}$	CK*
Q_t or Norm: Qt	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}.	$Qt = \frac{qt - \sigma_v}{\sigma_v'}$	2, 5
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
Q(1-Bq)	Q(1-Bq) grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter	$Q \cdot (1 - Bq)$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_t, defined above*	6, 7
qc1	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	21
qc1 (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}\ (0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	5
qc1 (Cn)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
qc1 (Cq)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}\ (Cq) = C_q * q_t$ (some papers use q_c)	5, 12
qc1n	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3, 5
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Fear (1995) and Robertson and Wride (1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *depending on the iteration in determining I_c* *And Fr is in percent* *P_a = atmospheric pressure* *n varies between 0.5, 0.70 and 1.0 and is selected in an iterative manner based on the resulting I_c*	3, 5, 21
Ic (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine Ic (PKR 2009) and its corresponding n (PKR 2009).	$I_c\ (PKR\ 2009) = [(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15

Calculated Parameter	Description	Equation	Ref
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding Ic (PKR 2009).	*n (PKR 2009)* $= 0.381\ (I_c) + 0.05\ (\sigma_v'/P_a) - 0.15$	15
Qtn (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Qtn (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ *where* P_a *= atmospheric pressure (100 kPa)* *n = stress ratio exponent described above*	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $I_c > 3.5$ $FC=0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) - vertical effective stress is used rather than a mean normal stress	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods a) General method b) 1st order approximation using q_tNet (clays) c) 1st order approximation using Δu_2 (clays) d) 1st order approximation using q_e (clays)	All stresses in kPa a) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)^{m'}\ (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}}$ b) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)$ c) $\sigma_p' = 0.54\cdot(\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ d) $\sigma_p' = 0.60\cdot(q_t - u_2)$	19 20 20 20
OCR OCR(JS1978) OCR(Mayne2014) OCR (qtNet) OCR (deltaU) OCR (qe) OCR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on a) Schmertmann (1978) method involving a plot plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR b) based on Yield stresses described above c) approximate version based on qtNet d) approximate version based on Δu e) approximate version based on effective tip, q_e f) approximate version based on shear wave velocity, V_s g) based on Qt	a) requires a user defined value for NC S_u/P_c' ratio b through f) *based on yield stresses* g) OCR = $0.25\cdot(Qt)^{1.25}$	9 19 20 20 20 18 32

Calculated Parameter	Description	Equation	Ref
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Es Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. Es is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}\left(\sigma'_v + \sigma'_h + \sigma'_h\right)^3$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \bullet \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	CK*
Delta U/Estress, P Value, Excess Pore Pressure Ratio	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ *where:* $\Delta u = u - u_{eq}$	25, 25a, CK*
Su/EStress	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su\ (N_{kt}) / \sigma_v'$	CK*
Gmax	G_{max} determined from SCPT shear wave velocities (not estimated values)	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/Gmax	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30

*CK – common knowledge

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT}	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \bullet (FC - 5))$	10
K_{CPT} or K_C (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$	3, 10
Kc (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}Ic$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs}) / (N_1)_{60cs}I_c = 8.5\,(1 - I_c/4.6)$ FC ≤ 5%: $\alpha = 0$, $\beta=1.0$ FC ≥ 35% $\alpha = 5.0$, $\beta=1.2$ 5% < FC < 35% $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \bullet K_{cpt}$	3
Qtn,cs (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{Su(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
Su (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress		16
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\,[(N_1)_{60}]^{4.79}$ *qc1* is calculated from specified qt(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\,[q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\,[q_{c1ncs}/1000]^3 + 0.08$	10
Kg	Small strain Stiffness Ratio Factor, Kg	$[Gmax/qt]/[qc1n^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Qtn chart from plotted point to state parameter Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP Distance calculation		25
URS NP Qtn	Normalized tip resistance (Qtn) point on Ψ = -0.05 curve used in SP Distance calculation		25

Table 2. References

No.	Reference
1	Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
2	Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27. This includes the discussions and replies.
3	Robertson, P.K. and Wride (Fear), C.E., 1998, "Evaluating cyclic liquefaction potential using the cone penetration test", Canadian Geotechnical Journal, 35: 442-459.
4	Robertson, P.K. and Wride, C.E., 1997, "Cyclic Liquefaction and its Evaluation Based on SPT and CPT", NCEER Workshop Paper, January 22, 1997.
5	Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, " Cone Penetration Testing in Geotechnical Practice," Blackie Academic and Professional.
6	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, "CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility", 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.
7	Jefferies, M.G. and Davies, M.P., 1993, "Use of CPTu to Estimate equivalent N_{60}", Geotechnical Testing Journal, 16(4): 458-467.
8	Been, K. and Jefferies, M.P., 1985, "A state parameter for sands", Geotechnique, 35(2), 99-112.
9	Schmertmann, 1978, "Guidelines for Cone Penetration Test Performance and Design", Federal Highway Administration Report FHWA-TS-78-209, U.S. Department of Transportation.
10	Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, 1996, chaired by Leslie Youd.
11	Kulhawy, F.H. and Mayne, P.W., 1990, "Manual on Estimating Soil Properties for Foundation Design, Report No. EL-6800", Electric Power Research Institute, Palo Alto, CA, August 1990, 306 p.
12	Olson, S.M. and Stark, T.D., 2002, "Liquefied strength ratio from liquefied flow failure case histories", Canadian Geotechnical Journal, 39: 951-966.
13	Olson, Scott M. and Stark, Timothy D., 2003, "Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, August 2003.
14	Jamiolkowski, M.B., Lo Presti, D.C.F. and Manassero, M., 2003, "Evaluation of Relative Density and Shear Strength of Sands from CPT and DMT", Soil Behaviour and Soft Ground Construction, ASCE, GSP N0. 119, 201-238.
15	Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, 46: 1337-1355.
16	Robertson, P.K., 2010, "Evaluation of Flow Liquefaction and Liquefied Strength Using the Cone Penetration Test", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, June 2010.
17	Mayne, P.W. and Kulhawy, F.H., 1982, "Ko-OCR Relationships in Soil", Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, GT6, pp. 851-872.
18	Mayne, P.W., Robertson P.K. and Lunne T., 1998, "Clay stress history evaluated from seismic piezocone tests", Proceedings of the First International Conference on Site Characterization – ISC '98, Atlanta Georgia, Volume 2, 1113-1118.

No.	Reference
19	Mayne, P.W., 2014, "Generalized CPT Method for Evaluating Yield Stress in Soils", Geocharacterization for Modeling and Sustainability (GSP 235: Proc. GeoCongress 2014, Atlanta, GA), ASCE, Reston, Virginia: 1336-1346.
20	Mayne, P.W., 2015, "Geocharacterization by In-Situ Testing", Continuing Education Course, Vancouver, BC, January 6-8, 2015.
21	Robertson, P.K. and Fear, C.E., 1995, "Liquefaction of sands and its evaluation", Proceedings of the First International Conference on Earthquake Engineering, Keynote Lecture IS Tokyo '95, Tokyo Japan, 1995.
22	Mayne, P.W., Peuchen, J. and Boumeester, D., 2010, "Soil unit weight estimation from CPTs", Proceeding of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Vol 2, Huntington Beach, California; Omnipress: 169-176.
23	Mayne, P.W., 2007, "NCHRP Synthesis 368 on Cone Penetration Test", Transportation Research Board, National Academies Press, Washington, D.C., 118 pages.
24	Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests.", Key note address #2, proceedings, 3rd International Symposium on Cone Penetration Testing (CPT'14, Las Vegas), ISSMGE Technical Committee TC102.
25	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Tailings and Mine Waste, 2014.
25a	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Powerpoint presentation, Tailings and Mine Waste, 2014.
26	Schneider, J.A. and Moss, R.E.S., 2011, "Linking cyclic stress and cyclic strain based methods for assessment of cyclic liquefaction triggering in sands", Geotechnique Letters 1, 31-36.
27	Rice, A., 1984, "The Seismic Cone Penetrometer", M.A.Sc. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
28	Gillespie, D.G., 1990, "Evaluating Shear Wave Velocity and Pore Pressure Data from the Seismic Cone Penetration Test", Ph.D. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
29	Robertson, P.K and Cabal, K.L., 2010, "Estimating soil unit weight from CPT", Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Huntington Beach, California.
30	Robertson, P.K., 2016, "Cone penetration test (CPT)-based soil behaviour type (SBT) classification system – an update", Canadian Geotechnical Journal, July 2016.
31	Robertson, P.K., 2012, "Interpretation of in-situ tests – some insights", Mitchell Lecture, ISC'4, Recife, Brazil.
32	Robertson, P.K., Cabal, K.L. 2015, "Guide to Cone Penetration Testing for Geotechnical Engineering", 6th Edition.

KLEINFELDER
Bright People. Right Solutions.

Appendix C

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	
KB-1	3.5	1C	LEAN CLAY WITH SAND (CL)	17.2	103.7							
KB-2	3.5	1C	LEAN CLAY (CL)	12.1	92.7							
KB-2	8.5	3C	SILTY SAND (SM)					37				
KB-2	16.0	5C	SANDY LEAN CLAY (CL)	17.3	106.9							
KB-2	26.0	7C	LEAN CLAY WITH SAND (CL)	25.9	99.5			70				
KB-3	3.5	1C	LEAN CLAY (CL)	12.1	99.2				34	20	14	
KB-3	11.0	4C	LEAN CLAY WITH SAND (CL)					83				
KB-3	14.5	5C	LEAN CLAY WITH SAND (CL)	18.7	110.1							
KB-4	8.5	4B	LEAN CLAY WITH SAND (CL)	18.0	111.2							
KB-4	11.0	5C	LEAN CLAY WITH SAND (CL)	22.0	102.5				29	18	11	TXUU: c = 0.65 ksf
KB-4	21.0	7C	LEAN CLAY WITH SAND (CL)	21.0	106.2			81	32	19	13	
KB-4	26.0	8C	SANDY LEAN CLAY (CL)	28.5	96.8				31	20	11	TXUU: c = 0.40 ksf
KB-4	36.0	10C	LEAN CLAY (CL)	29.8	95.3			90				
KB-4	41.0	11C	SANDY SILT (ML)	25.6	102.0			68	NP	NP	NP	
KB-5	11.0	5C	SANDY LEAN CLAY (CL)	13.8	111.1				26	16	10	TXUU: c = 4.13 ksf
KB-7	3.5	1C	LEAN CLAY (CL)	12.4	107.5							
KB-8	6.0	2C	LEAN CLAY (CL)	16.8	106.5			95	34	19	15	
KB-8	19.5	6C	LEAN CLAY (CL)	19.3	110.9							
KB-8	24.5	7C	LEAN CLAY (CL)	23.0	105.2							

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

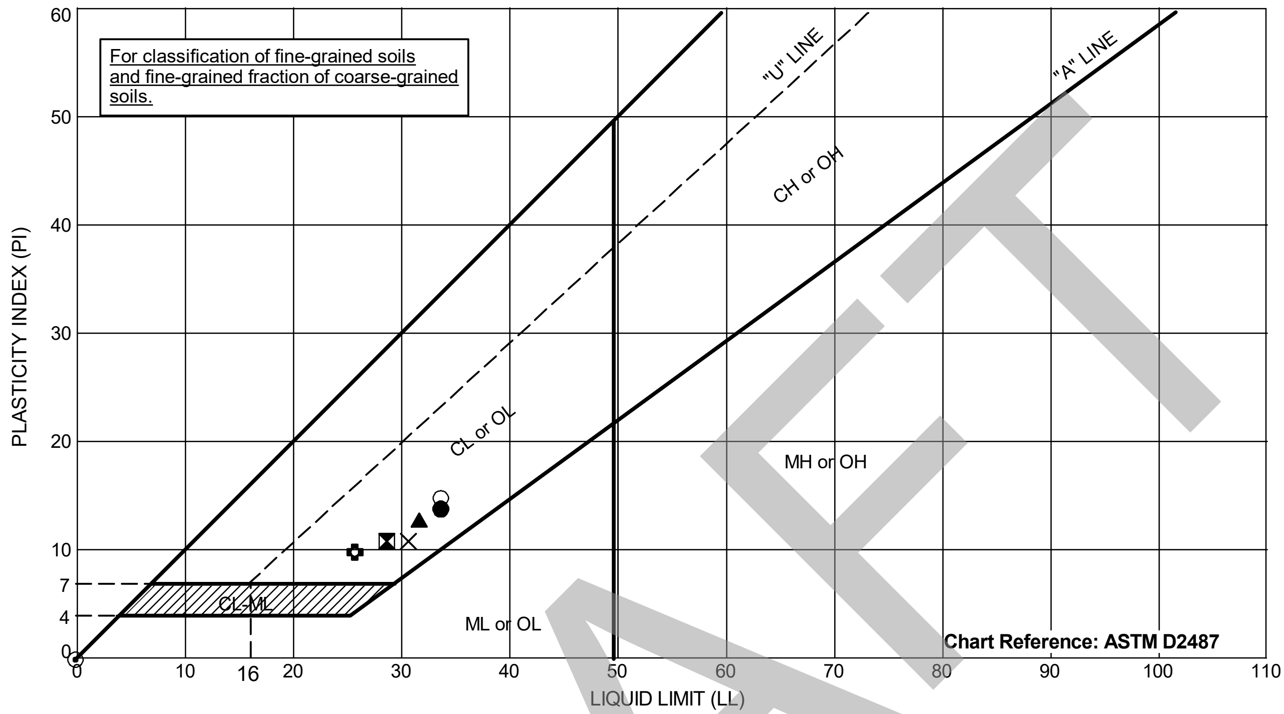
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20221404.001A
DRAWN BY: DJS
CHECKED BY: RY
DATE: 12/8/2021

LABORATORY TEST RESULT SUMMARY

DUKE WAREHOUSE SILVER CREEK
5977 & 6001 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA

FIGURE
C-1



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	Passing #200	LL	PL	PI
●	KB-3	3.5	1C	LEAN CLAY (CL)	NM	34	20	14
⧗	KB-4	11	5C	LEAN CLAY WITH SAND (CL)	NM	29	18	11
▲	KB-4	21	7C	LEAN CLAY WITH SAND (CL)	81	32	19	13
×	KB-4	26	8C	SANDY LEAN CLAY (CL)	NM	31	20	11
⊙	KB-4	41	11C	SANDY SILT (ML)	68	NP	NP	NP
✿	KB-5	11	5C	SANDY LEAN CLAY (CL)	NM	26	16	10
○	KB-8	6	2C	LEAN CLAY (CL)	95	34	19	15

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

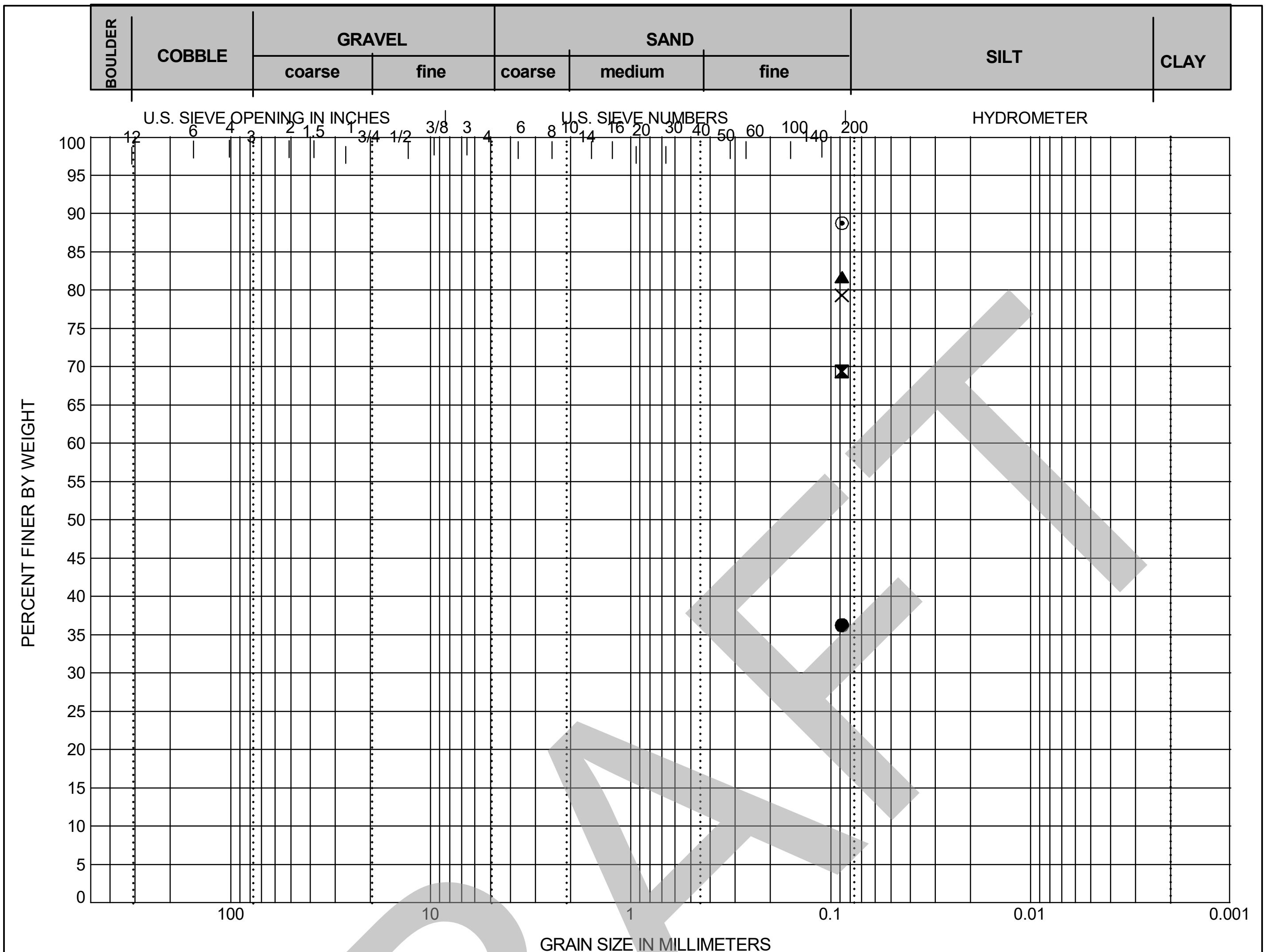
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20221404.001A
DRAWN BY: DJS
CHECKED BY: RY
DATE: 12/8/2021

ATTERBERG LIMITS

DUKE WAREHOUSE SILVER CREEK
5977 & 6001 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA

FIGURE
C-2



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	LL	PL	PI
●	KB-2	8.5	3C	SILTY SAND (SM)	NM	NM	NM
⧗	KB-2	26	7C	LEAN CLAY WITH SAND (CL)	NM	NM	NM
▲	KB-3	11	4C	LEAN CLAY WITH SAND (CL)	NM	NM	NM
×	KB-4	21	7C	LEAN CLAY WITH SAND (CL)	32	19	13
⊙	KB-4	36	10C	LEAN CLAY (CL)	NM	NM	NM

	Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt*	%Clay*
●	KB-2	8.5	0.075	NM	NM	NM	NM	NM			37	NM	NM
⧗	KB-2	26	0.075	NM	NM	NM	NM	NM			70	NM	NM
▲	KB-3	11	0.075	NM	NM	NM	NM	NM			83	NM	NM
×	KB-4	21	0.075	NM	NM	NM	NM	NM			81	NM	NM
⊙	KB-4	36	0.075	NM	NM	NM	NM	NM			90	NM	NM

*These numbers represent silt-sized and clay-sized content but may not indicate the percentage of the material with the engineering properties of silt or clay.
Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D6913(Sieve Analysis) and ASTM D7928 (Hydrometer Analysis).
NP = Nonplastic
NA = Not Available
NM = Not Measured

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

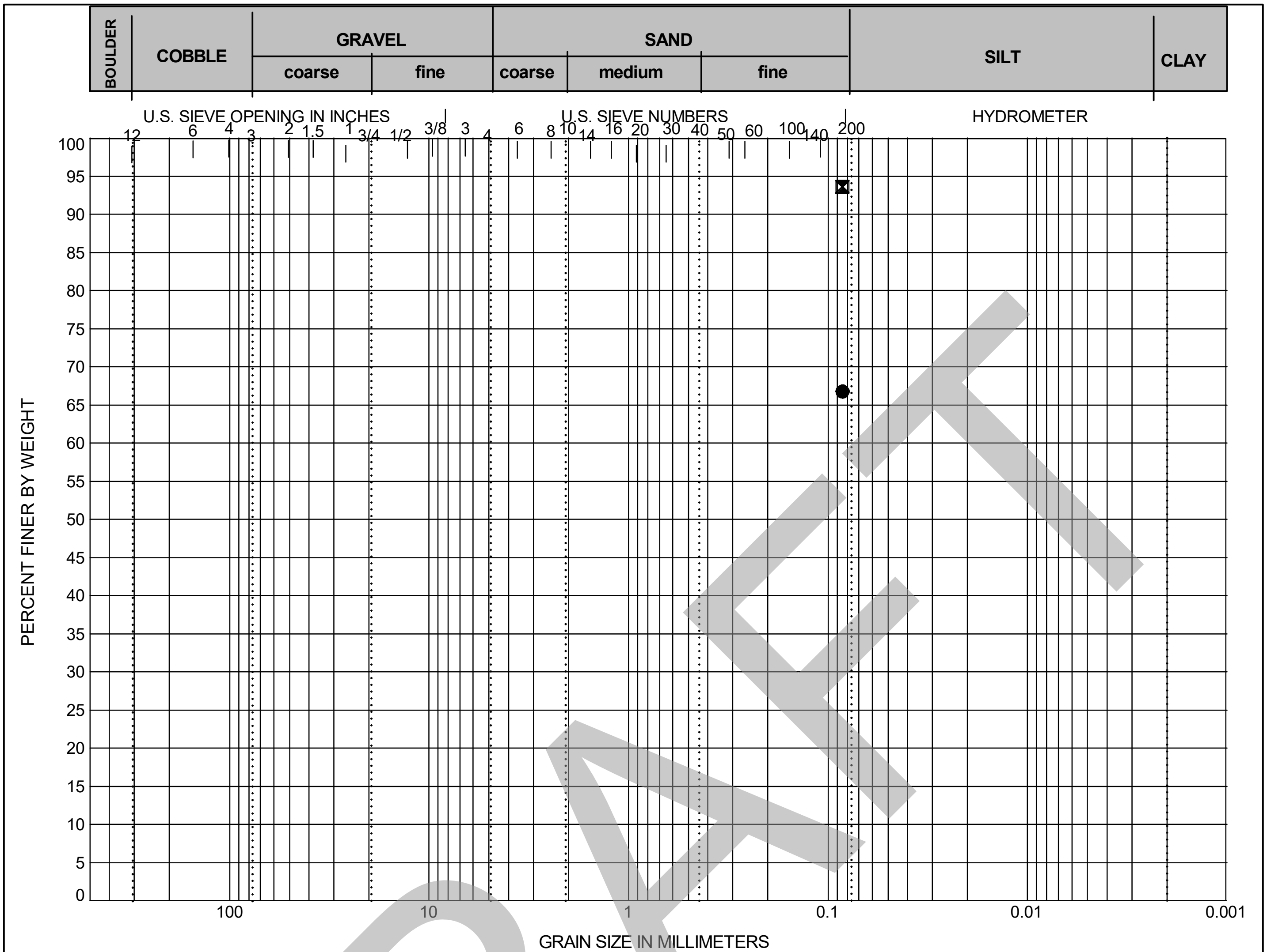
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20221404.001A
DRAWN BY: DJS
CHECKED BY: RY
DATE: 12/8/2021

SIEVE ANALYSIS

DUKE WAREHOUSE SILVER CREEK
5977 & 6001 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA

FIGURE
C-3



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	LL	PL	PI
●	KB-4	41	11C	SANDY SILT (ML)	NP	NP	NP
⧗	KB-8	6	2C	LEAN CLAY (CL)	34	19	15

	Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt*	%Clay*
●	KB-4	41	0.075	NM	NM	NM	NM	NM			68	NM	NM
⧗	KB-8	6	0.075	NM	NM	NM	NM	NM			95	NM	NM

*These numbers represent silt-sized and clay-sized content but may not indicate the percentage of the material with the engineering properties of silt or clay.
Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D6913(Sieve Analysis) and ASTM D7928 (Hydrometer Analysis).
NP = Nonplastic
NA = Not Available
NM = Not Measured

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

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20221404.001A
DRAWN BY: DJS
CHECKED BY: RY
DATE: 12/8/2021

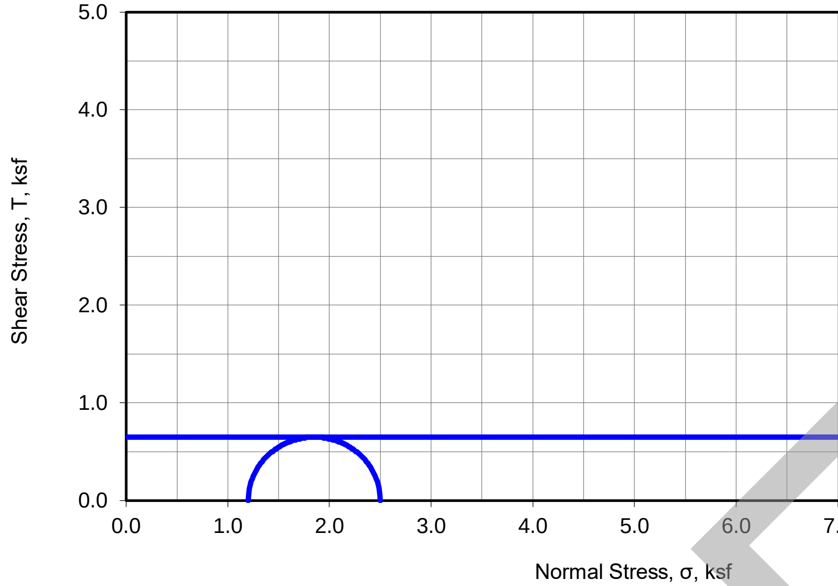
SIEVE ANALYSIS

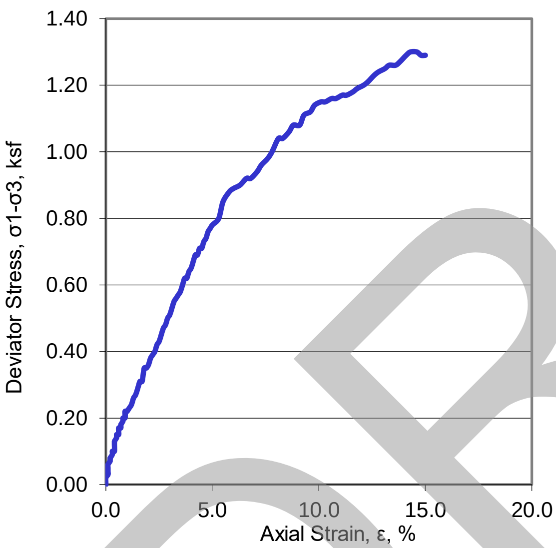
DUKE WAREHOUSE SILVER CREEK
5977 & 6001 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA

FIGURE
C-3
cont'd

Total		
c =	0.65	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.41
	Height, in	H_O	5.36
	Water Content, %	ω_O	22.0
	Dry Density, lbs/ft^3	γd_o	102.5
	Saturation, %	S_O	95
	Void Ratio	e_O	0.613
Minor Principal Stress, ksf		σ_3	1.20
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	1.30
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.33
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	1.29
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	14.33

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

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

LL: 29 | PL: 18 | PI: 11 | G_S: 2.65 Assumed | Specimen Type: Undisturbed | Test Method: ASTM D2850

Membrane correction applied

Boring:	KB-4
Sample:	5C
Depth, ft:	11.0
Test Date:	8/5/21

Remarks: nm= not measured, na = not applicable

KLEINFELDER
Bright People. Right Solutions.

2601 Barrington Ct, Hayward, CA 94545

Project No.:	20221404.001A
Date:	8/13/21
Entry By:	CP
Checked By:	CP
File Name:	HL14010

TRIAXIAL COMPRESSION TEST (UU)

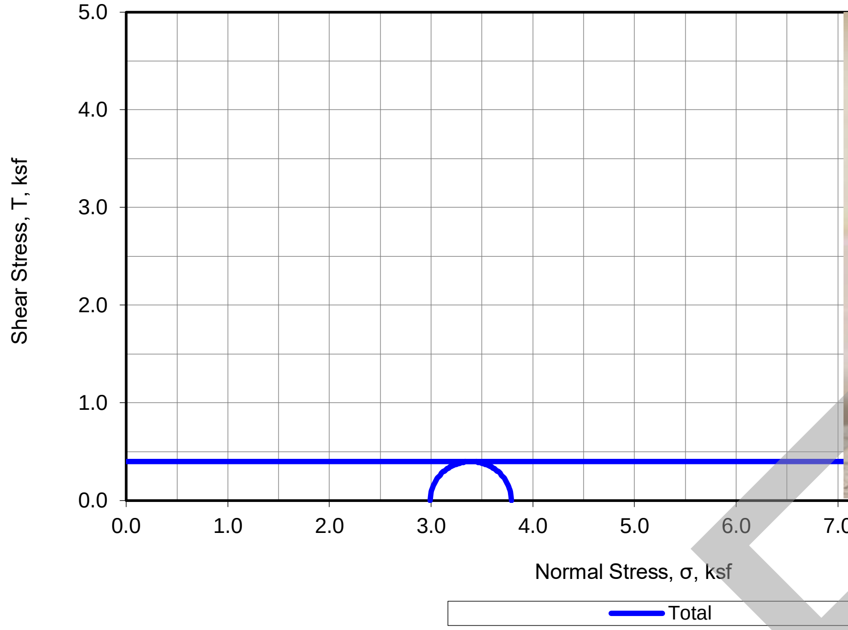
DUKE WAREHOUSE SILVER CREEK
5977 & 6001 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA

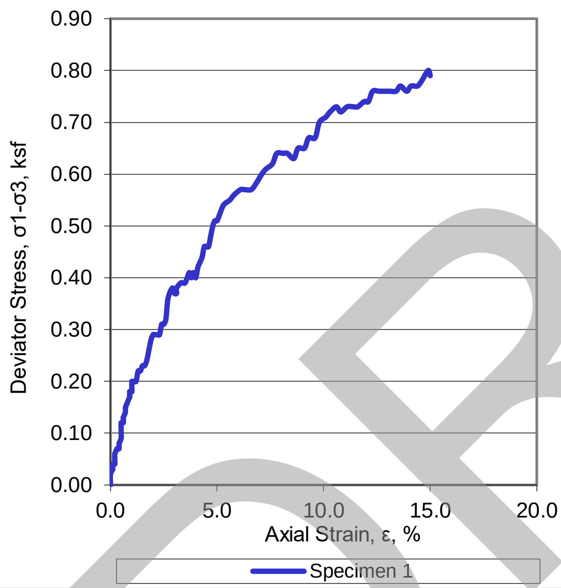
FIGURE

C-4

Total	
c = 0.40	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.39
	Height, in	H_O	5.02
	Water Content, %	ω_O	28.5
	Dry Density, lbs/ft^3	γd_o	96.8
	Saturation, %	S_O	107
	Void Ratio	e_O	0.708
Minor Principal Stress, ksf		σ_3	3.00
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	0.80
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.87
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	0.79
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	14.87

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

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

LL: 31 | PL: 20 | PI: 11 | G_S: 2.65 Assumed | Specimen Type: Undisturbed | Test Method: ASTM D2850

Membrane correction applied

Boring:	KB-4
Sample:	8C
Depth, ft:	26.0
Test Date:	8/5/21

Remarks: nm= not measured, na = not applicable

KLEINFELDER
Bright People. Right Solutions.
2601 Barrington Ct, Hayward, CA 94545

Project No.:	20221404.001A
Date:	8/13/21
Entry By:	CP
Checked By:	CP
File Name:	HL14010

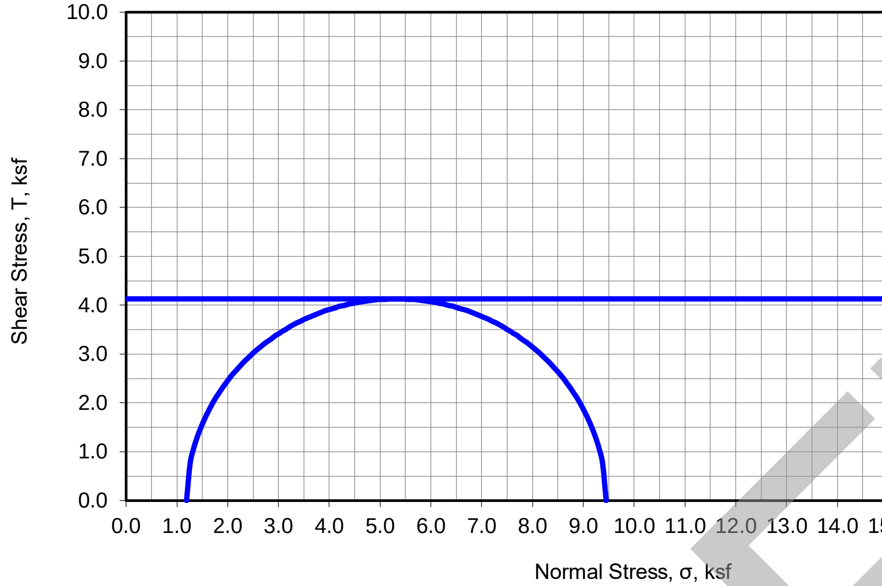
TRIAXIAL COMPRESSION TEST (UU)

DUKE WAREHOUSE SILVER CREEK
5977 & 6001 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA

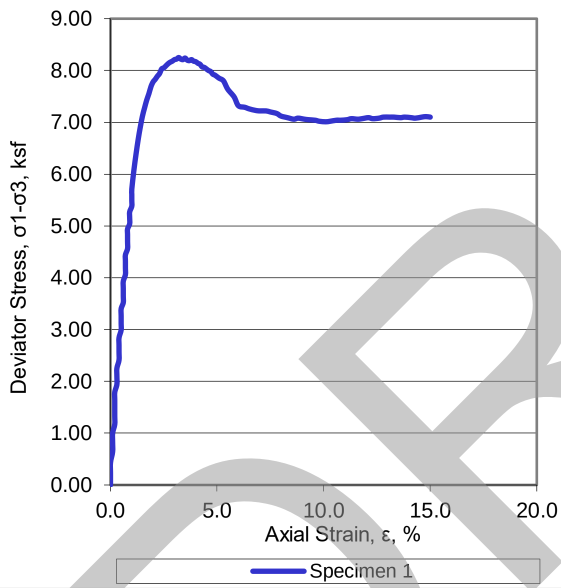
FIGURE

C-5

Total	
c = 4.13	ksf



Specimen Shear Picture



Specimen No.			1
Initial	Diameter, in	D_O	2.41
	Height, in	H_O	5.83
	Water Content, %	ω_O	13.8
	Dry Density, lbs/ft^3	γ_{d_o}	111.1
	Saturation, %	S_O	75
	Void Ratio	e_O	0.488
Minor Principal Stress, ksf		σ_3	1.20
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	8.25
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	3.23
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	7.10
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	3.23

Description of Specimen: Dark Yellowish Brown Sandy Lean Clay (CL)

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

LL: 26	PL: 16	PI: 10	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

Boring:	KB-5
Sample:	5C
Depth, ft:	11.0
Test Date:	8/5/21

Remarks: nm= not measured, na = not applicable

KLEINFELDER
Bright People. Right Solutions.
2601 Barrington Ct, Hayward, CA 94545

Project No.:	20221404.001A
Date:	8/13/21
Entry By:	CP
Checked By:	CP
File Name:	HL14010

TRIAXIAL COMPRESSION TEST (UU)

DUKE WAREHOUSE SILVER CREEK
5977 & 6001 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA

FIGURE

C-6

Laboratory Test Report

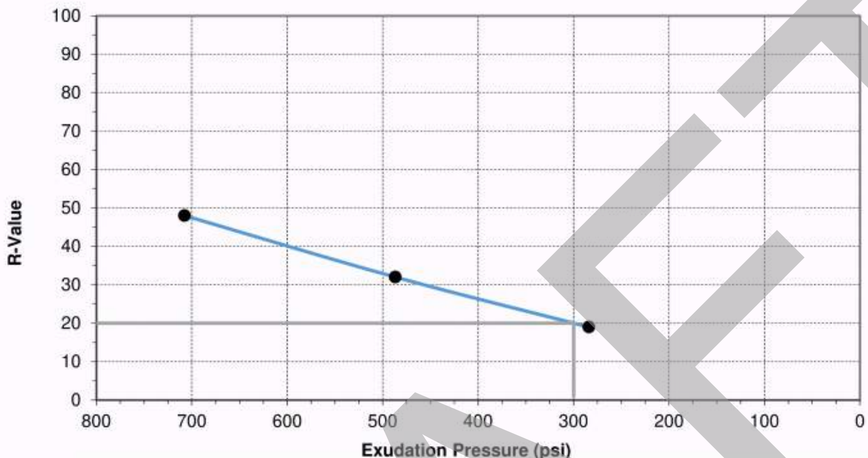
Client:	**Duke Realty Limited Partnership**	Report No.:	**21-HAY-01199 Rev. 0**	Issued:	**8/13/2021**
Project:	**20221404.001A**			Field ID:	**HL14010**
	Duke - Silver Creek	Sampled by:	**D. Sullivan**	Date:	**7/30/2021**
	04-000L - Labs	Submitted by:	**D. Sullivan**	Date:	**7/30/2021**

Tested on **8/12/2021** by **B. O'Neil**

Test Method: **ASTM D2844**

Material Description: **Brown Lean Clay (CL)**

Specific Location: **KB-5 @ 0-5'**



Briquette No.	**A**	**B**	**C**
Dry Unit Weight at Test (pcf)	111.3	110.4	116.1
Expansion Pressure (psf)	277	108	416
Exudation Pressure (psi)	487	284	708
Moisture at Time of Test (%)	15.5	16.6	14.6
Resistance Value	32	19	48
R - VALUE AT 300 PSI EXUDATION PRESSURE:			**20**

Reviewed on 8/13/2021 by Cindy Pimentel,
Senior Technician

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20221404	R-VALUE	FIGURE
	DRAWN BY: JDS		C-7
	CHECKED BY: RY	DUKE WAREHOUSE SILVER CREEK 5977 & 6001 SILVER CREEK ROAD SAN JOSE, CALIFORNIA	
	DATE: 12/09/21		
	REVISED:		

KLEINFELDER
Bright People. Right Solutions.

Appendix D

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

11 August 2021

Job No.2108006
Cust. No. 12312

Mr. Renie Yuen
Kleinfelder
2601 Barrington Court
Hayward, CA 94545-1100

Subject: Project No.: 20221404.001A
Project Name: Duke Warehouse Silver Creek
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Yuen:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on August 05, 2021. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

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

The chloride ion concentration reflects none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentration is 65 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil is 8.10 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

Sheri Moore
for, J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

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

Client: Kleinfelder
Client's Project No.: 20221404.001A
Client's Project Name: Duke Warehouse Silver Creek
Date Sampled: 30-Jul-21
Date Received: 5-Aug-21
Matrix: Soil
Authorization: Chain of Custody

Date of Report: 11-Aug-2021

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)*
2108006-001	KB-4, Bulk @ 0-5'	-	8.10		2,600	-	N.D.	65

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	-	9-Aug-2021		10-Aug-2021	-	9-Aug-2021	9-Aug-2021

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

Sherri Moore
for Cheryl McMillen
Laboratory Director

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

KLEINFELDER
Bright People. Right Solutions.

2108006

CU# 12312

Renie Yuen
RYuen@kleinfelder.com
510-628-9000

LAB NUMBER:
HL14010

LABORATORY TESTING PROGRAM - GEOTECHNICAL

Test Method
- [ ] ASTM
- [ ] AASHTO
- [ ] DOT
- [ ] Other (see remarks) ______

Project: Duke Warehouse Silver Creek

Date Sampled: 7/30

Date Submitted: 8/3

Submitted By: C Pimentel

Project No.: 20221404.004A

Task No.: 04-500L

Report To: R. Yuen, B. O'Neill

Boring / Test Pit	Sample	Depth, ft.	Sample Type	Tests	Remarks
KB-4	bulk	0-5	bag	SOIL PH /, SOIL RESISTIVITY /, CHLORIDE CONTENT /, SULFATE CONTENT /	resistivity @ 100% + brief eval.

Quantity of Tests

Test	Unit
CLASSIFICATN, VISUAL	L00228
WATER CONTENT	L00204
WATER/DENSITY, SOIL	L00205
SIEVE ANALYSIS, -200	L00267
SIEVE ANALYSIS, F	L00270
SIEVE ANALYSIS, C&F	L00269
HYDROMETER ANALYSIS	L00248
ATTERBERG, MULTI-PT.	L00223
ATTERBERG, ONE-PT.	L00224
SOIL ORGANIC CONTENT	L00271
SOIL SPEC. GRAV.	L00275
STANDARD PROCTOR, 4"	L00279
MODIFIED PROCTOR, 4"	L00251
R-VALUE	L00264
EXPANSION INDEX	L00246
SOIL PH	L00272
SOIL RESISTIVITY	L00273
CHLORIDE CONTENT	L00227
SULFATE CONTENT	L00281
SAMPLE MIX&PROCESS	L00905
SAMPLE PREPARATION	L00907

Steve Moore 8/5/21 1000

Boring / Test Pit	Sample	Depth, ft.	Sample Type	Remarks

Quantity of Tests

Test	Unit
UNCONFINED COMP STR.	L00293
DIRECT SHEAR, 1 PT.	L00242
DIRECT SHEAR, 3 PTS.	L00243
DIRECT SHEAR, RESID.	L00244
TRIAX, CU, 1 POINT	L00286
TRIAX, CU, 3 POINTS	L00287
TRIAX CD, 1 POINT	L00285
TRIAX CD, 3 POINTS	L00336
TRIAX, UU, 1 POINT	L00289
TRIAX, UU, 3 POINTS	L00290
TRIAX, UU SAT, 1 PT.	L00288
TRIAX, UU SAT, 3 PT.	L00325
PERMEABILITY, FLEX	L00258
PERMEABILITY, RIGID	L00259
PINHOLE DISPERSION	L00261
CONSOL. W/O TR	L00233
CONSOL. W/2 TR	L00232
CONSOL. ALL TR	L00231
CONSOL. ADD. REBOUND	L00230
ONEDIM FREE SWELL	L00255
ONEDIM SWELL % HEAVE	L00256
ONEDIMSWELL W/CONSOL	L00257
SAMPLE REMOLDING	L00900

Test requested / Test in progress / Test complete

LAB-30 REV 02/14

12/3/2021

jdh corrosion consultants, inc.

Soil Corrosivity Evaluation & Recommendations for Corrosion Control
5977 & 6001 Silver Creek Valley Road
San Jose, CA

SUBMITTED TO

Mr. Bill McCormick, CEG

Senior Principal Engineering Geologist/V.P.

Kleinfelder

2240 Northpoint Parkway

Santa Rosa, CA 95407

JDH JOB NUMBER

2021395

jdh corrosion consultants, inc.

Protecting the infrastructure through innovative Corrosion Engineering Solutions

December 3, 2021

Kleinfelder
2240 Northpoint Parkway
Santa Rosa, CA 95407

Attention: **Mr. Bill McCormick, CEG**
Senior Principal Engineering Geologist/V.P.

Subject: **Soil Corrosivity Evaluation & Recommendations for Corrosion Control**
Underground Water Piping Systems
5977 & 6001 Silver Creek Valley Road
San Jose, CA

Dear Mr. McCormick,

Pursuant to your request, **JDH Corrosion Consultants, Inc**. has conducted a site corrosivity evaluation for the above referenced project site and we have provided herein recommendations for long-term corrosion control for the proposed materials of construction for the underground utilities at this site.

Purpose

The purpose for this evaluation is to determine the corrosion potential, resulting from the soils at the subject site and to provide recommendations for long-term corrosion control for the buried metallic utilities.

Background

The project involves the construction of a warehouse facility at 5977 & 6001 Silver Creek Valley Road in San Jose, California. There will be new buried utilities associated with the domestic and fire water pipelines.

Soil Testing and Analysis

Soil Testing Results

One (1) bulk soil sample from the project site was chemically analyzed for corrosivity by **CERCO Analytical**. The sample was analyzed for chloride and sulfate concentration, pH and resistivity at 100% saturation. The test results were presented in CERCO Analytical report dated August 11, 2021. The results of the chemical analysis were as follows:

Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	None Detected	Non-corrosive*
Sulfates	65 (mg/kg)	Non-corrosive**
pH	8.10	Non-corrosive *
Resistivity at 100% Saturation	2,600 ohm-cm	**Moderately Corrosive***

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Chemical Testing Analysis

The chemical analysis provided by **CERCO Analytical** indicates that based on this soil data, the soils are generally classified as "moderately corrosive to" based on the saturated resistivity measurement. The chloride levels indicate "non-corrosive" conditions to steel and ductile iron and the sulfate levels indicate "non-corrosive" conditions for concrete structures placed into these soils with regard to sulfate attack. The pH of the soils is alkaline which classifies them as "non-corrosive" to buried steel and concrete structures.

In-Situ Soil Resistivity Measurements

The in-situ resistivity of the soil was measured at four (4) locations at the project site by **JDH Corrosion Consultants, Inc.** field personnel. Resistance measurements were conducted with probe spacing of 2.5, 5, 7.5, 10, and 15-feet at each location. For analysis purposes we have calculated the resistivity of soil layers 0-2.5, 2.5-5, 5-7.5, 7.5-10 and 10-15' using the Barnes Method as follows:

ρ_{b-a} = KR (b-a)

Where;

ρ_{b-a} = soil resistivity of layer depth b-a (ohm-cm)
a = soil depth to top layer (ft)
b = soil depth to bottom layer (ft)
R_a = soil resistance read at depth a (ohms)
R_b = soil resistance read at depth b (ohms)

$R_{b\text{-}a}$ = resistance of soil layer from a to b (ft)

K = layer constant = $60.96\pi(b-a)$ (cm)

and $$\frac{1}{R_{b\text{-}a}} = \frac{1}{R_a} - \frac{1}{R_b}$$

The visual diagrams below describe the Wenner 4-pin testing configuration.

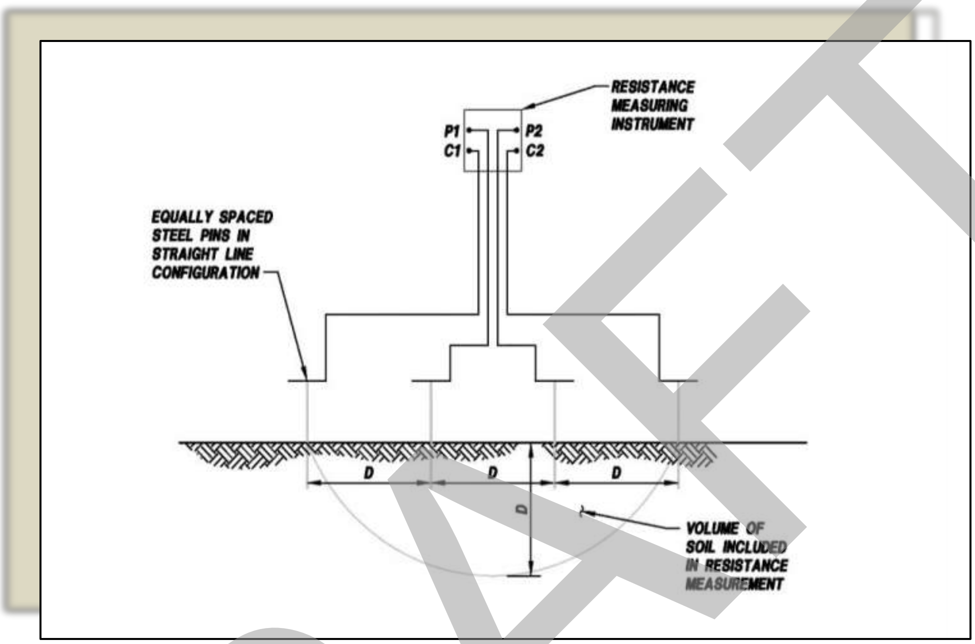


Fig 1: Wenner 4-Pin Resistivity Schematic No.1

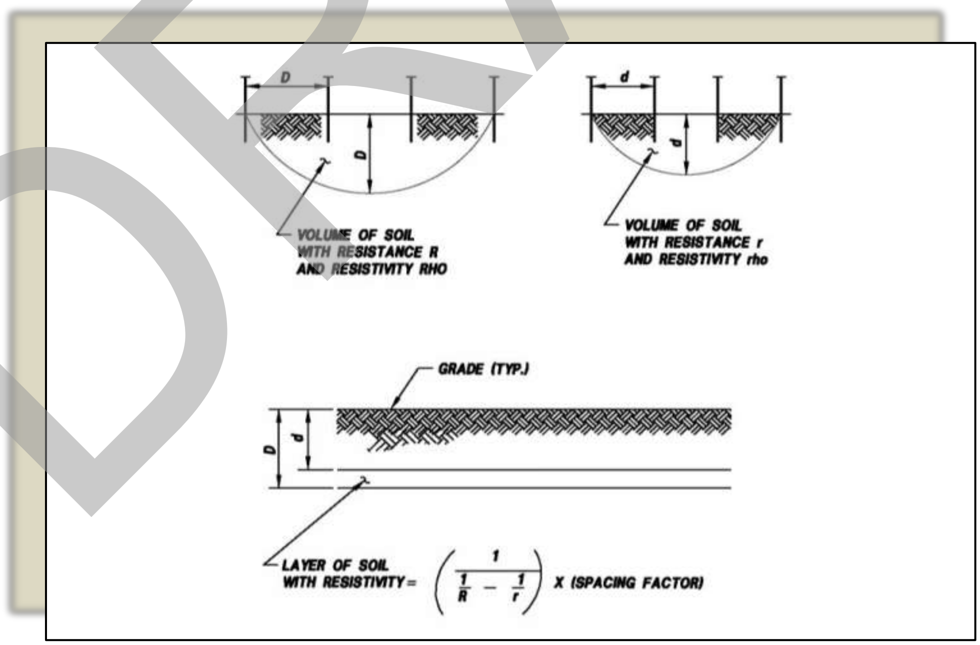


Fig 2: Illustration of Barnes Layer Calculations

In-Situ Soil Resistivity Analysis

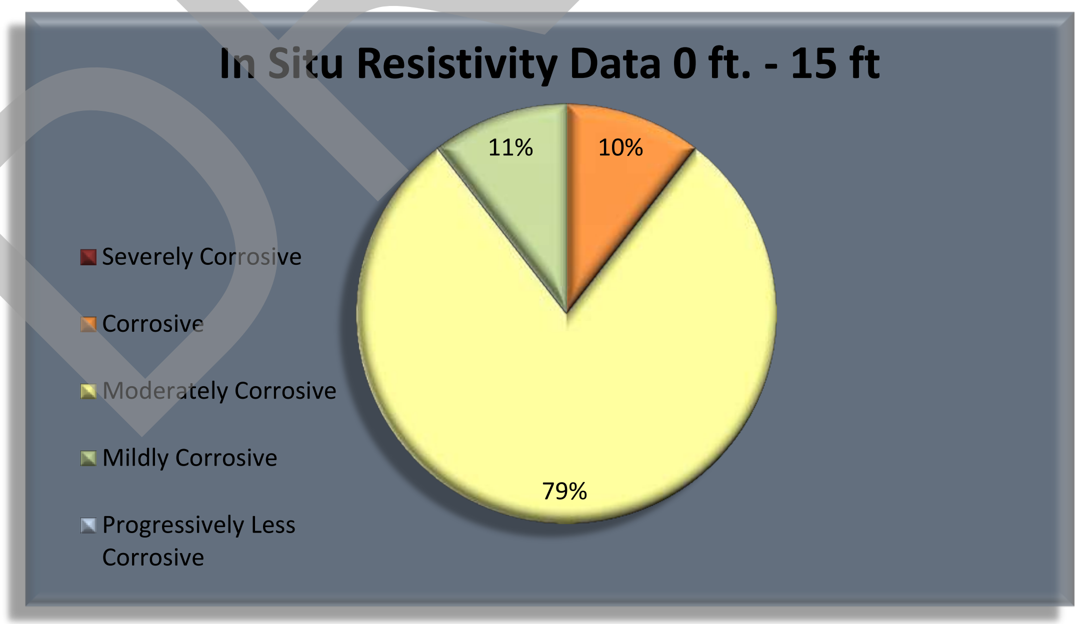
Corrosion of a metal is an electro-chemical process and is accompanied by the flow of electric current. Resistivity is a measure of the ability of a soil to conduct an electric current and is, therefore, an important parameter in consideration of corrosion data. Soil resistivity is primarily dependent upon the chemical content and moisture content of the soil mass.

The greater the amount of chemical constituents present in the soil, the lower the resistivity will be. As moisture content increases, resistivity decreases until maximum solubility of dissolved chemicals is attained. Beyond this point, an increase in moisture content results in dilution of the chemical concentration and resistivity increases. The corrosion rate of steel in soil normally increases as resistivity decreases. Therefore, in any particular group of soils, maximum corrosion will generally occur in the lowest resistivity areas. The following classification of soil corrosivity, developed by William J. Ellis[1], is used for the analysis of the soil data for the project site.

Resistivity (Ohm-cm)	Corrosivity Classification
0 – 500	Very Corrosive
501 – 2,000	Corrosive
2,001 – 8,000	Moderately Corrosive
8,001 – 32,000	Mildly Corrosive
> 32,000	Progressively Less Corrosive

The above classifications are appropriate for the project site and the results are presented in the graph below. In general, the soils are classified as "corrosive to mildly corrosive" with respect to corrosion of buried steel structures throughout the top 0 to 15 feet of the site.

The chart of the in-situ soil resistivity data for the soil layers 0 to 15 feet indicate that 10% of the soils are classified as "corrosive", 79% of the soils are classified as "moderately corrosive" and 11% of the soils are classified as "mildly corrosive".



Discussion

Reinforced Concrete In Contact With Soil

The presence of water-soluble sulfate ions in the soils tested in the upper levels of the soil at the site was at a relatively low level. As such, Type II cement can be utilized for the concrete foundations that do not extend beyond the fill soil zone. It is recommended that the water/cement ratio should not exceed 0.50 with a minimum depth of cover of 3" over the reinforcing bars, especially in the areas where the foundation is more than a few feet deep.

Underground Metallic Pipelines

The soils at the project sites are generally considered to be "corrosive to mildly corrosive" to ductile/cast iron, steel and dielectric coated steel based on the in-situ resistivity measurements. Therefore, special requirements for corrosion control are required for buried metallic utilities at these sites depending upon the critical nature of the piping. Pressure piping systems such as domestic and fire water should be provided with appropriate coating systems and cathodic protection, where warranted. In addition, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

Recommendations

Reinforced Concrete In Contact With Soil

For application in reinforced concrete slab foundations, we recommend using a Type II modified cement mix with a maximum water-to-cement ratio of 0.50 and a minimum depth of cover for the reinforcing steel of 3-inches.

Ductile Iron Pipe (Pressure Piping such as Domestic Water and Fire)

1. Direct buried ductile iron pipe should be encased in 8-mil polyethylene as specified in AWWA specification C-105. Epoxy coatings are also an acceptable alternative type of coating system for the pipe and/or fittings such as valves.

2. All rubber gasket joints, fusion-bonded epoxy coated flanges and flexible couplings on ductile iron pipelines should be bonded with insulated copper cable to insure electrical continuity of the pipeline and fittings.

3. Insulating flanges and/or couplings should be installed to electrically isolate the buried portion of pipeline from other metallic pipelines, reinforced concrete structures and above grade buildings or structures.

4. Test stations shall be installed on all ductile iron pipelines at a spacing of 800 to 1,000 feet. Bonding and test stations shall comply with NACE Standards.

5. A sacrificial type of cathodic protection utilizing ***magnesium*** anodes should be installed to protect the entire length of buried metallic pipeline. Cathodic protection should be designed in accordance with NACE Standard SP0169-13 and applicable local standards and included with the contract documents to permit installation along with the pipeline.

6. As an alternate, non-metallic piping may be used in lieu of ductile iron piping as allowed by State and local codes. Non-metallic piping does not require the implementation of any special type of corrosion prevention measures. However, all metallic valves, fittings and appurtenances on non-metallic piping will require protection as specified below.

Ductile Iron Fittings & Metallic Valves (On Plastic Pressure Piping)

1. All direct buried ductile iron fittings installed on non-metallic piping shall be provided with a bituminous coating from the factory and encased in an 8-mil polyethylene bag in the field in accordance with AWWA Specification C-105. All bolts, restraining rods, etc. shall be coated with bitumastic prior to encasement in the polyethylene bag.

2. All metallic valves shall be coated from the factory (i.e. using powdered epoxy or equivalent type of coating system) and all bolts shall be stainless steel.

3. A sacrificial type of cathodic protection utilizing ***magnesium*** anodes should be installed to protect the valves and fittings. Cathodic protection should be designed in accordance with NACE Standard SP0169-13 and applicable local standards and included with the contract documents to permit installation along with the pipeline.

Steel Pipelines (Natural Gas Pipelines & Risers)

1. A fusion-bonded epoxy coating system or a suitable tape coating should be applied to all buried steel pipelines in accordance with ANSI/AWWA C214-95, "AWWA Standard for Tape Coating Systems for the Exterior of Steel Water Pipelines." Also, a tape coating per AWWA Standard C209-95 is recommended for special sections, connections and fittings.

2. Insulating flanges and/or couplings should be installed to electrically isolate the buried portions of steel pipelines from other metallic pipelines, reinforced concrete structures and above grade structures.

3. All rubber gasket joints, fusion epoxy coated flanges and flexible couplings should be bonded with insulated copper cable to insure electrical continuity of the pipeline and fittings.

4. A sacrificial type of cathodic protection using ***magnesium*** anodes should be installed to protect the buried portions of steel pipelines used for the natural gas piping systems. Cathodic protection should be designed in accordance with NACE Standard SP0169-13 and applicable local standards and included with the contract documents to permit installation along with the subject pipeline.

5. As an alternate, non-metallic piping may be used in lieu of steel piping as allowed by State and local codes. Non-metallic piping does not require the implementation of any special type of corrosion prevention measures.

Cast Iron (Gravity Sewer and Storm Drain Lines)

1. Sewer and storm drain lines should be wrapped in 8-mil polyethylene as specified in AWWA specification C-105, if they are under the building footprint.

Copper Water Pipelines (Service Lines)

1. Direct buried copper water service lines should be encased in 6-mil minimum polyethylene as specified in the AWWA specification C-105.

2. All copper water laterals shall be electrically isolated from metallic water mains via the use of insulating type corporation stops installed at the water main.

Stainless Steel Risers

1. Direct buried stainless steel risers should be primed and wrapped with Polyguard 'RD-6' coating system.

2. Insulating flanges and/or couplings should be installed to electrically isolate the buried portion of the stainless steel riser from other metallic pipelines, reinforced concrete structures and above grade buildings or structures.

3. A sacrificial type of cathodic protection utilizing ***magnesium*** anodes should be installed to protect the buried portions of the stainless steel riser used for the water piping systems. Cathodic protection should be designed in accordance with NACE Standard SP0169-13 and applicable local standards and included with the contract documents to permit installation along with the subject pipeline.

LIMITATIONS

The conclusions and recommendations contained in this report reflect the opinion of the author of this report and are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees or guarantees either expressed or implied are provided.

We thank you for the opportunity to be of assistance on this important project. If you have any questions concerning this report or the recommendations provided herein, please feel free to contact us at (925) 927-6630.

Respectfully submitted,

Brendon Hurley
JDH CORROSION CONSULTANTS, INC.
Field Technician

Mohammed Ali

Mohammed Ali., P.E.
JDH Corrosion Consultants, Inc.
Senior Corrosion Engineer

CC: File 2021395

REFERENCES

1. *Ellis, William J., Corrosion of Concrete Pipelines, Western States Corrosion Seminar, 1978*

2. *AWWA Manual of Water Supply Practices - M27, First Edition, External Corrosion - Introduction to Chemistry and Control (Denver, CO: 1987)*

3. *National Association of Corrosion Engineers, Standard Recommended Practice, SP 01-69-13, Control of External Corrosion on Underground or Submerged Pipeline*

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.: 20221404.001A
Client's Project Name: Duke Warehouse Silver Creek
Date Sampled: 30-Jul-21
Date Received: 5-Aug-21
Matrix: Soil
Authorization: Chain of Custody

Date of Report: 11-Aug-2021

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)*
2108006-001	KB-4, Bulk @ 0-5'	-	8.10		2,600	-	N.D.	65

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	-	9-Aug-2021		10-Aug-2021	-	9-Aug-2021	9-Aug-2021

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

Sherri Moore
for, Cheryl McMillen
Laboratory Director

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

Client: *Kleinfelder*
Project: *5977 & 6001 Silver Creek Valley Road*
Location: *San Jose, CA*
Date: *12/2/2021*
Job No: *2021395*
Subject: *In-Situ Soil Resistivity Data*

Severely Corrosive
Corrosive
Moderately Corrosive
Mildly Corrosive
Progressively Less Corrosive

*Test #	Location Description	Resistance Data From AEMC Meter					Soil Resistivities (ohm-cm)					Barnes Layer Analysis (ohm-cm)				
		2.5	5	7.5	10	15	2.5	5	7.5	10	15	0-2.5'	2.5-5'	5-7.5'	7.5-10"	10-15'
1	Location 1	7.62	3.31	1.35	1.34	0.96	3,648	3,169	1,939	2,566	2,758	3,648	2,802	1,091	86,606	3,241
2	Location 2	9.49	4.74	2.18	1.47	1.12	4,543	4,539	3,131	2,815	3,217	4,543	4,534	1,932	2,161	4,504
3	Location 3	19.20	8.69	4.30	2.32	1.55	9,192	8,321	6,176	4,443	4,452	9,192	7,600	4,075	2,412	4,472
4	Location 4	20.20	8.82	4.55	2.81	1.38	9,671	8,445	6,535	5,381	3,964	9,671	7,495	4,499	3,518	2,596

In-situ Locations

jdh corrosion consultants, inc.

Bright People. Right Solutions.

Appendix E

McCampbell Analytical, Inc.

"When Quality Counts"

Analytical Report

WorkOrder: 2108095

Report Created for: Kleinfelder, Inc.

380 North 1st Street, Suite A
San Jose, CA 95132

Project Contact: Renie Yuen
Project P.O.: 20221404.001A/01-0000
Project: 20221404.001A; Industrial Warehouse Building, Silver Creek Parcel
Project Received: 08/03/2021

Analytical Report reviewed & approved for release on 08/05/2021 by:

Yen Cao

Yen Cao
Project Manager

NELAP ACCREDITED TNI LABORATORY

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Glossary of Terms & Qualifier Definitions

Client: Kleinfelder, Inc.
Project: 20221404.001A; Industrial Warehouse Building, Silver Creek Parcel
WorkOrder: 2108095

Glossary Abbreviation

%D	Serial Dilution Percent Difference
95% Interval	95% Confident Interval
CPT	Consumer Product Testing not NELAP Accredited
DF	Dilution Factor
DI WET	(DISTLC) Waste Extraction Test using DI water
DISS	Dissolved (direct analysis of 0.45 µm filtered and acidified water sample)
DLT	Dilution Test (Serial Dilution)
DUP	Duplicate
EDL	Estimated Detection Limit
ERS	External reference sample. Second source calibration verification.
ITEF	International Toxicity Equivalence Factor
LCS	Laboratory Control Sample
LQL	Lowest Quantitation Level
MB	Method Blank
MB % Rec	% Recovery of Surrogate in Method Blank, if applicable
MDL	Method Detection Limit
ML	Minimum Level of Quantitation
MS	Matrix Spike
MSD	Matrix Spike Duplicate
N/A	Not Applicable
ND	Not detected at or above the indicated MDL or RL
NR	Data Not Reported due to matrix interference or insufficient sample amount.
PDS	Post Digestion Spike
PDSD	Post Digestion Spike Duplicate
PF	Prep Factor
RD	Relative Difference
RL	Reporting Limit (The RL is the lowest calibration standard in a multipoint calibration.)
RPD	Relative Percent Deviation
RRT	Relative Retention Time
SPK Val	Spike Value
SPKRef Val	Spike Reference Value
SPLP	Synthetic Precipitation Leachate Procedure
ST	Sorbent Tube
TCLP	Toxicity Characteristic Leachate Procedure
TEQ	Toxicity Equivalents
TZA	TimeZone Net Adjustment for sample collected outside of MAI's UTC.
WET (STLC)	Waste Extraction Test (Soluble Threshold Limit Concentration)

Glossary of Terms & Qualifier Definitions

Client: Kleinfelder, Inc.
Project: 20221404.001A; Industrial Warehouse Building, Silver Creek Parcel
WorkOrder: 2108095

Analytical Qualifiers

S	Surrogate recovery outside accepted recovery limits.
c2	Surrogate recovery outside of the control limits due to matrix interference.
e2	Diesel range compounds are detected; no recognizable pattern.
e7	Oil range compounds are detected.

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Received:	08/03/2021 13:40	**Extraction Method:**	SW3550B
Date Prepared:	08/03/2021	**Analytical Method:**	SW8082
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Unit:**	mg/kg

Polychlorinated Biphenyls (PCBs) Aroclors

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Boring KB-4	**2108095-001A**	**Sludge**	**07/30/2021 10:00**	**GC23 08042107.d**	**226836**

Analytes	Result	RL	DF	Date Analyzed
Aroclor1016	ND	0.050	1	08/04/2021 13:26
Aroclor1221	ND	0.050	1	08/04/2021 13:26
Aroclor1232	ND	0.050	1	08/04/2021 13:26
Aroclor1242	ND	0.050	1	08/04/2021 13:26
Aroclor1248	ND	0.050	1	08/04/2021 13:26
Aroclor1254	ND	0.050	1	08/04/2021 13:26
Aroclor1260	ND	0.050	1	08/04/2021 13:26
PCBs, total	ND	0.050	1	08/04/2021 13:26

Surrogates	REC (%)	Limits		
Decachlorobiphenyl	82	60-130		08/04/2021 13:26

Analyst(s): CN

1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Received:	08/03/2021 13:40	**Extraction Method:**	SW3050B
Date Prepared:	08/03/2021	**Analytical Method:**	SW6020
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Unit:**	mg/Kg

Metals

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Boring KB-4	**2108095-001A**	**Sludge**	**07/30/2021 10:00**	**ICP-MS5 114SMPL.d**	**226839**

Analytes	Result	RL	DF	Date Analyzed
Antimony	ND	0.50	1	08/04/2021 11:19
Arsenic	**2.5**	0.50	1	08/04/2021 11:19
Barium	**65**	5.0	1	08/04/2021 11:19
Beryllium	ND	0.50	1	08/04/2021 11:19
Cadmium	ND	0.50	1	08/04/2021 11:19
Chromium	**18**	0.50	1	08/04/2021 11:19
Cobalt	**3.9**	0.50	1	08/04/2021 11:19
Copper	**9.3**	0.50	1	08/04/2021 11:19
Lead	**2.6**	0.50	1	08/04/2021 11:19
Molybdenum	ND	0.50	1	08/04/2021 11:19
Nickel	**26**	0.50	1	08/04/2021 11:19
Selenium	ND	0.50	1	08/04/2021 11:19
Silver	ND	0.50	1	08/04/2021 11:19
Thallium	ND	0.50	1	08/04/2021 11:19
Vanadium	**14**	0.50	1	08/04/2021 11:19
Zinc	**18**	5.0	1	08/04/2021 11:19

Surrogates	REC (%)	Limits	Date Analyzed
Terbium	103	70-130	08/04/2021 11:19

Analyst(s): WV

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Received:	08/03/2021 13:40	**Extraction Method:**	SW5035
Date Prepared:	08/03/2021	**Analytical Method:**	SW8021B/8015Bm
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Unit:**	mg/Kg

Gasoline Range (C6-C12) Volatile Hydrocarbons as Gasoline with BTEX and MTBE

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Boring KB-4	2108095-001A	Sludge	07/30/2021 10:00	GC3 08032126.D	226768

Analytes	Result	RL	DF	Date Analyzed
TPH(g) (C6-C12)	ND	1.0	1	08/03/2021 20:56
MTBE	ND	0.050	1	08/03/2021 20:56
Benzene	ND	0.0050	1	08/03/2021 20:56
Toluene	ND	0.0050	1	08/03/2021 20:56
Ethylbenzene	ND	0.0050	1	08/03/2021 20:56
m,p-Xylene	ND	0.010	1	08/03/2021 20:56
o-Xylene	ND	0.0050	1	08/03/2021 20:56
Xylenes	ND	0.0050	1	08/03/2021 20:56

Surrogates	REC (%)	Qualifiers	Limits	
2-Fluorotoluene	51	S	62-126	08/03/2021 20:56

Analyst(s): TD

Analytical Comments: c2

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Received:	08/03/2021 13:40	**Extraction Method:**	SW7471B
Date Prepared:	08/04/2021	**Analytical Method:**	SW7471B
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Unit:**	mg/Kg

Mercury by Cold Vapor Atomic Absorption

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Boring KB-4	2108095-001A	Sludge	07/30/2021 10:00	AA1 _15	226842

Analytes	Result	RL	DF	Date Analyzed
Mercury	ND	0.017	1	08/04/2021 15:34

Analyst(s): MIG

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Received:	08/03/2021 13:40	**Extraction Method:**	SW9045C
Date Prepared:	08/04/2021	**Analytical Method:**	SW9045C
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Unit:**	pH units @ 25°C

pH

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Boring KB-4	**2108095-001A**	**Sludge**	**07/30/2021 10:00**	**WetChem**	**226898**

Analytes	Result	Accuracy	DF	Date Analyzed
pH	**9.42**	±0.1	1	08/04/2021 12:32

Analyst(s): HAD

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Received:	08/03/2021 13:40	**Extraction Method:**	SW3550B
Date Prepared:	08/03/2021	**Analytical Method:**	SW8015B
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Unit:**	mg/Kg

Total Extractable Petroleum Hydrocarbons w/out SG Clean-Up

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Boring KB-4	**2108095-001A**	**Sludge**	**07/30/2021 10:00**	**GC9a 08032130.D**	**226767**

Analytes	Result	RL	DF	Date Analyzed
TPH-Diesel (C10-C23)	**8.4**	1.0	1	08/04/2021 04:17
TPH-Motor Oil (C18-C36)	**22**	5.0	1	08/04/2021 04:17

Surrogates	REC (%)	Limits	Date Analyzed
C9	90	70-130	08/04/2021 04:17

Analyst(s): JIS

Analytical Comments: e2,e7

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Prepared:	08/03/2021	**BatchID:**	226836
Date Analyzed:	08/04/2021	**Extraction Method:**	SW3550B
Instrument:	GC23	**Analytical Method:**	SW8082
Matrix:	Soil	**Unit:**	mg/kg
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Sample ID:**	MB/LCS/LCSD-226836 2108095-001AMS/MSD

QC Summary Report for SW8082

Analyte	MB Result	MDL	RL	SPK Val	MB SS %REC	MB SS Limits
Aroclor1016	ND	0.00510	0.0500	-	-	-
Aroclor1221	ND	0.0110	0.0500	-	-	-
Aroclor1232	ND	0.00630	0.0500	-	-	-
Aroclor1242	ND	0.00670	0.0500	-	-	-
Aroclor1248	ND	0.00400	0.0500	-	-	-
Aroclor1254	ND	0.00680	0.0500	-	-	-
Aroclor1260	ND	0.00610	0.0500	-	-	-
Surrogate Recovery						
Decachlorobiphenyl	0.0476			0.05	95	70-130

Analyte	LCS Result	LCSD Result	SPK Val	LCS %REC	LCSD %REC	LCS/LCSD Limits	RPD	RPD Limit
Aroclor1016	0.169	0.154	0.15	112	103	70-130	8.78	20
Aroclor1260	0.140	0.135	0.15	93	90	70-130	2.94	20
Surrogate Recovery								
Decachlorobiphenyl	0.0435	0.0419	0.050	87	84	70-130	3.71	20

Analyte	MS DF	MS Result	MSD Result	SPK Val	SPKRef Val	MS %REC	MSD %REC	MS/MSD Limits	RPD	RPD Limit
Aroclor1016	1	0.157	0.154	0.15	ND	104	103	60-130	1.35	20
Aroclor1260	1	0.118	0.117	0.15	ND	78	78	60-130	0.714	20
Surrogate Recovery										
Decachlorobiphenyl	1	0.0431	0.0423	0.050		86	85	60-130	1.94	20

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Prepared:	08/03/2021	**BatchID:**	226839
Date Analyzed:	08/04/2021	**Extraction Method:**	SW3050B
Instrument:	ICP-MS5	**Analytical Method:**	SW6020
Matrix:	Soil	**Unit:**	mg/kg
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Sample ID:**	MB/LCS/LCSD-226839 2108095-001AMS/MSD

QC Summary Report for Metals

Analyte	MB Result	MDL	RL	SPK Val	MB SS %REC	MB SS Limits
Antimony	ND	0.160	0.500	-	-	-
Arsenic	ND	0.150	0.500	-	-	-
Barium	ND	0.570	5.00	-	-	-
Beryllium	ND	0.0730	0.500	-	-	-
Cadmium	ND	0.0940	0.500	-	-	-
Chromium	ND	0.130	0.500	-	-	-
Cobalt	ND	0.0520	0.500	-	-	-
Copper	ND	0.180	0.500	-	-	-
Lead	ND	0.140	0.500	-	-	-
Molybdenum	ND	0.160	0.500	-	-	-
Nickel	ND	0.170	0.500	-	-	-
Selenium	ND	0.150	0.500	-	-	-
Silver	ND	0.120	0.500	-	-	-
Thallium	ND	0.0670	0.500	-	-	-
Vanadium	ND	0.130	0.500	-	-	-
Zinc	ND	3.00	5.00	-	-	-
Surrogate Recovery						
Terbium	523			500	105	70-130

(Cont.)

Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Prepared:	08/03/2021	**BatchID:**	226839
Date Analyzed:	08/04/2021	**Extraction Method:**	SW3050B
Instrument:	ICP-MS5	**Analytical Method:**	SW6020
Matrix:	Soil	**Unit:**	mg/kg
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Sample ID:**	MB/LCS/LCSD-226839 2108095-001AMS/MSD

QC Summary Report for Metals

Analyte	LCS Result	LCSD Result	SPK Val	LCS %REC	LCSD %REC	LCS/LCSD Limits	RPD	RPD Limit
Antimony	50.4	48.8	50	101	98	75-125	3.10	20
Arsenic	52.0	50.3	50	104	101	75-125	3.38	20
Barium	519	501	500	104	100	75-125	3.58	20
Beryllium	52.7	52.1	50	105	104	75-125	1.16	20
Cadmium	50.1	49.4	50	100	99	75-125	1.34	20
Chromium	49.0	48.8	50	98	98	75-125	0.530	20
Cobalt	51.4	50.4	50	103	101	75-125	2.05	20
Copper	50.3	48.3	50	101	97	75-125	4.06	20
Lead	50.9	50.0	50	102	100	75-125	1.67	20
Molybdenum	50.6	49.1	50	101	98	75-125	3.02	20
Nickel	50.5	48.5	50	101	97	75-125	3.98	20
Selenium	51.6	49.7	50	103	99	75-125	3.64	20
Silver	50.2	49.2	50	100	98	75-125	1.92	20
Thallium	52.8	50.4	50	106	101	75-125	4.59	20
Vanadium	50.6	49.8	50	101	100	75-125	1.55	20
Zinc	496	482	500	99	96	75-125	2.83	20
Surrogate Recovery								
Terbium	544	525	500	109	105	70-130	3.53	20

Analyte	MS DF	MS Result	MSD Result	SPK Val	SPKRef Val	MS %REC	MSD %REC	MS/MSD Limits	RPD	RPD Limit
Antimony	1	49.0	48.6	50	ND	98	97	75-125	0.928	20
Arsenic	1	52.5	54.2	50	2.493	100	103	75-125	3.14	20
Barium	1	574	605	500	64.59	102	108	75-125	5.21	20
Beryllium	1	50.2	49.2	50	ND	100	98	75-125	2.02	20
Cadmium	1	48.8	48.3	50	ND	98	97	75-125	1.10	20
Chromium	1	66.3	75.0	50	18.31	96	113	75-125	12.4	20
Cobalt	1	52.4	53.2	50	3.913	97	99	75-125	1.47	20
Copper	1	57.4	62.4	50	9.268	96	106	75-125	8.37	20
Lead	1	52.6	52.8	50	2.647	100	100	75-125	0.529	20
Molybdenum	1	49.4	48.4	50	ND	98	96	75-125	2.03	20
Nickel	1	73.8	88.5	50	26.37	95	124	75-125	18.2	20
Selenium	1	50.4	50.7	50	ND	100	100	75-125	0.678	20
Silver	1	49.5	48.2	50	ND	99	96	75-125	2.71	20
Thallium	1	50.6	49.8	50	ND	101	100	75-125	1.55	20
Vanadium	1	65.5	72.3	50	14.30	102	116	75-125	9.79	20

(Cont.)

Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Prepared:	08/03/2021	**BatchID:**	226839
Date Analyzed:	08/04/2021	**Extraction Method:**	SW3050B
Instrument:	ICP-MS5	**Analytical Method:**	SW6020
Matrix:	Soil	**Unit:**	mg/kg
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Sample ID:**	MB/LCS/LCSD-226839 2108095-001AMS/MSD

QC Summary Report for Metals

Analyte	MS DF	MS Result	MSD Result	SPK Val	SPKRef Val	MS %REC	MSD %REC	MS/MSD Limits	RPD	RPD Limit
Zinc	1	508	514	500	18.45	98	99	75-125	1.15	20
Surrogate Recovery										
Terbium	1	523	517	500		105	103	70-130	1.05	20

Analyte	DLT Result	DLTRef Val	%D	%D Limit
Antimony	ND<2.50	ND	-	-
Arsenic	2.60	2.493	4.29	-
Barium	65.2	64.59	0.944	-
Beryllium	ND<2.50	ND	-	-
Cadmium	ND<2.50	ND	-	-
Chromium	19.1	18.31	4.31	20
Cobalt	4.08	3.913	4.27	-
Copper	9.91	9.268	6.93	-
Lead	2.64	2.647	0.264	-
Molybdenum	ND<2.50	ND	-	-
Nickel	26.9	26.37	2.01	20
Selenium	ND<2.50	ND	-	-
Silver	ND<2.50	ND	-	-
Thallium	ND<2.50	ND	-	-
Vanadium	14.7	14.30	2.80	20
Zinc	ND<25.0	18.45	-	-

%D Control Limit applied to analytes with concentrations greater than 25 times the reporting limits.

Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Prepared:	08/02/2021	**BatchID:**	226768
Date Analyzed:	08/03/2021 - 08/04/2021	**Extraction Method:**	SW5035
Instrument:	GC3	**Analytical Method:**	SW8021B/8015Bm
Matrix:	Soil	**Unit:**	mg/Kg
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Sample ID:**	MB/LCS/LCSD-226768

QC Summary Report for SW8021B/8015Bm

Analyte	MB Result	MDL	RL	SPK Val	MB SS %REC	MB SS Limits
TPH(g) (C6-C12)	ND	0.610	1.00	-	-	-
MTBE	ND	0.00340	0.0500	-	-	-
Benzene	ND	0.00190	0.00500	-	-	-
Toluene	ND	0.00240	0.00500	-	-	-
Ethylbenzene	ND	0.00170	0.00500	-	-	-
m,p-Xylene	ND	0.00260	0.0100	-	-	-
o-Xylene	ND	0.000910	0.00500	-	-	-
Surrogate Recovery						
2-Fluorotoluene	0.0852			0.1	85	75-134

Analyte	LCS Result	LCSD Result	SPK Val	LCS %REC	LCSD %REC	LCS/LCSD Limits	RPD	RPD Limit
TPH(btex)	0.600	0.615	0.60	100	103	82-118	2.62	20
MTBE	0.102	0.0975	0.10	102	97	61-119	4.61	20
Benzene	0.0864	0.0914	0.10	86	91	77-128	5.55	20
Toluene	0.0897	0.0926	0.10	90	93	74-132	3.13	20
Ethylbenzene	0.0901	0.0904	0.10	90	90	84-127	0.409	20
m,p-Xylene	0.181	0.180	0.20	91	90	80-120	0.536	20
o-Xylene	0.0908	0.0882	0.10	91	88	80-120	2.91	20
Surrogate Recovery								
2-Fluorotoluene	0.0879	0.0910	0.10	88	91	75-134	3.45	20

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Prepared:	08/04/2021	**BatchID:**	226842
Date Analyzed:	08/04/2021	**Extraction Method:**	SW7471B
Instrument:	AA1	**Analytical Method:**	SW7471B
Matrix:	Soil	**Unit:**	mg/Kg
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Sample ID:**	MB/LCS/LCSD-226842 2108095-001AMS/MSD

QC Summary Report for Mercury

Analyte	MB Result	MDL	RL			
Mercury	ND	0.0150	0.0170	-	-	-

Analyte	LCS Result	LCSD Result	SPK Val	LCS %REC	LCSD %REC	LCS/LCSD Limits	RPD	RPD Limit
Mercury	0.156	0.165	0.17	93	99	80-120	5.60	20

Analyte	MS DF	MS Result	MSD Result	SPK Val	SPKRef Val	MS %REC	MSD %REC	MS/MSD Limits	RPD	RPD Limit
Mercury	1	0.196	0.191	0.17	ND	117	114	80-120	2.68	20

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Quality Control Report

Client: Kleinfelder, Inc.
Date Prepared: 08/04/2021
Date Analyzed: 08/04/2021
Instrument: WetChem
Matrix: Water
Project: 20221404.001A; Industrial Warehouse Building, Silver Creek Parcel

WorkOrder: 2108095
BatchID: 226898
Extraction Method: SW9045C
Analytical Method: SW9045C
Unit: pH units @ 25°C
Sample ID: CCV-226898

QC Summary Report for pH

Analyte	CCV Result	CCV Limits
pH	7.01	6.8-7.2

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	2108095
Date Prepared:	08/02/2021	**BatchID:**	226767
Date Analyzed:	08/03/2021	**Extraction Method:**	SW3550B
Instrument:	GC39B	**Analytical Method:**	SW8015B
Matrix:	Soil	**Unit:**	mg/Kg
Project:	20221404.001A; Industrial Warehouse Building, Silver Creek Parcel	**Sample ID:**	MB/LCS/LCSD-226767

QC Report for SW8015B w/out SG Clean-Up

Analyte	MB Result	MDL	RL	SPK Val	MB SS %REC	MB SS Limits
TPH-Diesel (C10-C23)	ND	0.750	1.00	-	-	-
TPH-Motor Oil (C18-C36)	ND	3.90	5.00	-	-	-
Surrogate Recovery						
C9	20.8			25	83	70-130

Analyte	LCS Result	LCSD Result	SPK Val	LCS %REC	LCSD %REC	LCS/LCSD Limits	RPD	RPD Limit
TPH-Diesel (C10-C23)	42.5	42.6	40	106	106	70-130	0.246	20
Surrogate Recovery								
C9	20.9	20.8	25	84	83	70-130	0.601	20

McCampbell Analytical, Inc.

1534 Willow Pass Rd
Pittsburg, CA 94565-1701
(925) 252-9262

CHAIN-OF-CUSTODY RECORD

WorkOrder: 2108095 **ClientCode: KFSJ**

☐ WaterTrax ☐ WriteOn ☐ EDF ☐ EQuIS ☐ Dry-Weight ☑ Email ☐ HardCopy ☐ ThirdParty ☐ J-flag
☐ Detection Summary ☐ Excel

Report to:
Renie Yuen
Kleinfelder, Inc.
380 North 1st Street, Suite A
San Jose, CA 95132
(408) 586-7611 FAX: (408) 586-7688

Email: ryuen@kleinfelder.com
cc/3rd Party:
PO: 20221404.001A/01-0000
Project: 20221404.001A; Industrial Warehouse Building, Silver Creek Parcel

Bill to:
F. Mwape
Kleinfelder, Inc.
380 North 1st Street, Suite A
San Jose, CA 95132
AccountsPayableUS@kleinfelder.com

Requested TAT: 2 days;

Date Received: 08/03/2021
Date Logged: 08/03/2021

Lab ID	Client ID	Matrix	Collection Date	Hold	1	2	3	4	5	6	7	8	9	10	11	12
2108095-001	Boring KB-4	Sludge	7/30/2021 10:00	☐	A	A	A	A	A	A	A					

Requested Tests (See legend below)

Test Legend:

1	8082_PCB_S	2	CAM17MS_TTLC_S	3	G-MBTEX_S	4	HG_S
5	PH_S	6	PRDisposal Fee	7	TPH(DMO)_S	8	
9		10		11		12	

Project Manager: Angela Rydelius

Prepared by: Cassandra Gallegos

Comments:

NOTE: Soil samples are discarded 60 days after results are reported unless other arrangements are made (Water samples are 30 days).
Hazardous samples will be returned to client or disposed of at client expense.

McCampbell Analytical, Inc.
"When Quality Counts"

1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

WORK ORDER SUMMARY

Client Name: KLEINFELDER, INC.
Client Contact: Renie Yuen
Contact's Email: ryuen@kleinfelder.com

Project: 20221404.001A; Industrial Warehouse Building, Silver Creek Parcel
Comments:

Work Order: 2108095
QC Level: LEVEL 2
Date Logged: 8/3/2021

☐ WaterTrax ☐ WriteOn ☐ EDF ☐ Excel ☐ EQuIS ☑ Email ☐ HardCopy ☐ ThirdParty ☐ J-flag

LabID	ClientSampID	Matrix	Test Name	Containers /Composites	Bottle & Preservative	Head Space	Dry-Weight	Collection Date & Time	TAT	Test Due Date	Sediment Content	Hold	SubOut
001A	Boring KB-4	Sludge	SW8015B (Diesel & Motor Oil)	3 / (3:1)	4OZ aGJ, Unpres	☐	☐	7/30/2021 10:00	2 days	8/5/2021		☐	
			SW9045C (pH)			☐	☐		2 days	8/5/2021		☐	
			SW7471B (Mercury)			☐	☐		2 days	8/5/2021		☐	
			SW8021B/8015Bm (G/MBTEX)			☐	☐		2 days	8/5/2021		☐	
			SW6020 (CAM 17)			☐	☐		2 days	8/5/2021		☐	
			SW8082 (PCBs Only)			☐	☐		2 days	8/5/2021		☐	

NOTES: * STLC and TCLP extractions require 2 days to complete; therefore, all TATs begin after the extraction is completed (i.e., One-day TAT yields results in 3 days from sample submission).

- MAI assumes that all material present in the provided sampling container is considered part of the sample - MAI does not exclude any material from the sample prior to sample preparation unless requested in writing by the client.

General COC

MAI Work Order # 2108095

RUSH!

McCAMPBELL ANALYTICAL, INC.

1534 Willow Pass Rd. Pittsburg, Ca. 94565-1701
Telephone: (877) 252-9262 / Fax: (925) 252-9269
www.mccampbell.com main@mccampbell.com

CHAIN OF CUSTODY RECORD

Turn Around Time: 1 Day Rush		2 Day Rush	●	3 Day Rush		STD	Quote #
J-Flag / MDL		ESL		Cleanup Approved		Dry Weight	Bottle Order #
Delivery Format:	PDF	●	GeoTracker EDF	EDD		Write On (DW)	Detect Summary

Report To: Renie Yuen   Bill To: AccountsPayableUS@kleinfelder.com
Company: Kleinfelder
Address: 5977 & 6001 Silver Creek Valley Road
Email: ryuen@kleinfelder.com   Tele: (510) 715-9237
Project Name: Industrial Warehouse Building, Silver Creek Parcel   Project #: 20221404.001A
Project Location: San Jose, California   PO # 20221404.001A/01-0000
Sampler Signature:

Analysis Requested

SAMPLE ID Location / Field Point	Sampling Date	Sampling Time	#Containers	Matrix	Preservative	Multi Range as Gas, Diesel, and Motor Oil (8021/8015)	BTEX & TPH as Gas (8021/ 8015) MTBE	TPH as Diesel (8015) + Motor Oil Without Silica Gel	TPH as Diesel (8015) + Motor Oil With Silica Gel	Total Oil & Grease (1664 / 9071) Without Silica Gel	Total Petroleum Hydrocarbons - Oil & Grease (1664 / 9071) With Silica Gel	Total Petroleum Hydrocarbons (418.1) With Silica Gel	EPA 505/ 608 / 8081 (Cl Pesticides)	EPA 608 / 8082 PCB's ; Aroclors only	EPA 524.2 / 624 / 8260 (VOCs)	EPA 525.2 / 625 / 8270 (SVOCs)	EPA 8270 SIM / 8310 (PAHs / PNAs)	CAM 17 Metals (200.8 / 6020)*	Metals (200.8 / 6020)*	Baylands Requirements	Lab to filter sample for dissolved metals analysis	Mercury (7471)	pH (9045C)	
Boring KB-4	7/30/21	10:00a	3	S	1		●	●						●				●				●	●	
			[illegible]	[illegible]	[illegible]																			
			[illegible]	[illegible]	[illegible]																			

MAI clients MUST disclose any dangerous chemicals known to be present in their submitted samples in concentrations that may cause immediate harm or serious future health endangerment as a result of brief, gloved, open air, sample handling by MAI staff. Non-disclosure incurs an immediate $250 surcharge and the client is subject to full legal liability for harm suffered. Thank you for your understanding and for allowing us to work safely.

* If metals are requested for water samples and the water type (Matrix) is not specified on the chain of custody, MAI will default to metals by E200.8.

Please provide an adequate volume of sample. If the volume is not sufficient for a MS/MSD a LCS/LCSD will be prepared in its place and noted in the report.

Comments / Instructions
-Composite three samples

Relinquished By / Company Name	Date	Time	Received By / Company Name	Date	Time
A. Kidd	8/2	10:15	L Meson	8/3	1015
	~~8/3~~	~~1015~~			
[illegible]	8/3	1340	Cassandra Gallegos	8/3	1340

Matrix Code: DW=Drinking Water, GW=Ground Water, WW=Waste Water, SW=Seawater, S=Soil, SL=Sludge, A=Air, WP=Wipe, O=Other

Preservative Code: 1=4°C 2=HCl 3=H_2SO_4 4=HNO_3 5=NaOH 6=ZnOAc/NaOH 7=None

Temp 2.9 °C Initials CG

Page 1 of 1

DRAFT

McCampbell Analytical, Inc.
"When Quality Counts"

1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Sample Receipt Checklist

Client Name:	**Kleinfelder, Inc.**	Date and Time Received:	**8/3/2021 13:40**
Project:	**20221404.001A; Industrial Warehouse Building, Silver Creek Parcel**	Date Logged:	**8/3/2021**
		Received by:	Cassandra Gallegos
WorkOrder №:	**2108095** Matrix: Sludge	Logged by:	Cassandra Gallegos
Carrier:	Laurie Moore (MAI Courier)		

Chain of Custody (COC) Information

	Yes	No	NA
Chain of custody present?	✔		
Chain of custody signed when relinquished and received?	✔		
Chain of custody agrees with sample labels?	✔		
Sample IDs noted by Client on COC?	✔		
Date and Time of collection noted by Client on COC?	✔		
Sampler's name noted on COC?		✔	
COC agrees with Quote?			✔

Sample Receipt Information

	Yes	No	NA
Custody seals intact on shipping container/cooler?			✔
Custody seals intact on sample bottles?	✔		
Shipping container/cooler in good condition?	✔		
Samples in proper containers/bottles?	✔		
Sample containers intact?	✔		
Sufficient sample volume for indicated test?	✔		

Sample Preservation and Hold Time (HT) Information

	Yes	No	NA
All samples received within holding time?	✔		
Samples Received on Ice?	✔		
(Ice Type: WET ICE)			
Sample/Temp Blank temperature	Temp: 2.9°C		
ZHS conditional analyses: VOA meets zero headspace requirement (VOCs, TPHg/BTEX, RSK)?			✔
Sample labels checked for correct preservation?	✔		
pH acceptable upon receipt (Metal: <2; Nitrate 353.2/4500NO3: <2; 522: <4; 218.7: >8)?			✔
UCMR Samples:			
pH tested and acceptable upon receipt (200.8: ≤2; 525.3: ≤4; 530: ≤7; 541: <3; 544: <6.5 & 7.5)?			✔
Free Chlorine tested and acceptable upon receipt (<0.1mg/L)?			✔

Comments:

KLEINFELDER
Bright People. Right Solutions.

Appendix F

APPENDIX F
SITE RESPONSE ANALYSIS

SUBJECT: Duke Warehouse
Silver Creek Valley Road, San Jose, California

PRJ. NO.: 20221404.001A

FROM: Zia Zafir, PhD, PE, GE and Troy Covill

INTRODUCTION

This appendix presents the results of Kleinfelder's site response analyses for the proposed project in San Jose, California. Based on the review of the available geotechnical investigation data by Kleinfelder in 2021 and 2022, and the liquefaction analyses, there is a liquefaction potential at the site for some layers deeper than 35 to 40 feet below ground surface. However, this liquefaction and its consequences wouldn't impact the foundation near surface. Therefore, the site is not classified as Site Class F, it does not require site response analysis. However, based on our discussions with the design team, we believed that a site response analysis is appropriate for this site because it would be able to capture the response of softer clayey soils under seismic loading. According to Section 11.4.8 of ASCE 7-16, site response analysis is allowed for any site. Therefore, a site response analysis was performed in general accordance with Chapter 21 of ASCE 7-16, which is referenced in the 2019 California Building Code (CBC). The purpose of this analysis is to develop the site-specific seismic design parameters which will be used for the seismic design of the proposed structure.

Project Location

The project site is located in San Jose, California. The approximate site coordinates are:

Latitude: 37.2586° N

Longitude: 121.7883° W

Analysis Approach

The site response analysis was performed in general accordance with the requirements of the 2019 CBC and Section 21.1 of ASCE 7-16. The scope of the analysis includes the following:

- Review of geotechnical investigation data and development of one idealized soil profile, to be used in the site response analyses;
- Estimate of an appropriate V_{S30} value for the soil/decomposed rock at the bottom of the soil profile;
- Development of a Risk-Targeted Maximum Considered Earthquake (MCE_R) target response spectrum at the base of the soil column per Section 21.2 of ASCE 7-16 using the estimated V_{S30} value at that depth;
- Deaggregation analyses of the hazard to estimate the controlling seismic sources, magnitudes, and distances associated with the period range of interest for the target spectrum;

- Selection and spectral modification of five acceleration time histories based on the target spectral shape, earthquake magnitude, distance, site condition, frequency content, and other factors from historical earthquake records;
- Nonlinear (NL) (total stress and effective stress) site response analyses for the soil profile using the spectrally-matched time histories; and
- Development of the site-specific design acceleration response spectrum and seismic design parameters using the results of site response analyses per Sections 21.3 and 21.4 of ASCE 7-16, respectively.

SUBSURFACE CHARACTERIZATION

Subsurface characterization was developed based on the information available from the site-specific field investigation performed in 2021 and 2022 especially two seismic CPTs performed in January 2022. Primarily, we used the data from the two seismic CPTs (SCPT-05 and SCPT-06) performed in January 2022 to develop soil profile for the site response analysis.

Subsurface Geology and Stratigraphy

The project site is generally underlain by alluvial deposits, consisting mostly of clays with layers of sands. Further details regarding the characteristics and conditions of the site geology are provided in the main report. The generalized best estimate profile of material properties was developed and is presented in Table F1.

TABLE F1: MATERIAL PROPERTIES FOR SITE RESPONSE ANALYSIS

Layer		Layer Design Values												
		Depth (ft)	Elevation (ft)	$(N_1)_{60}$	φ' (deg)	Su (psf)	σ'_p (psf)	$\sigma'_{v\text{-}avg}$ (psf)	OCR	K_o	PI	FC (%)	γ (pcf)	Vs (ft/sec)
Silty SAND to Sandy SILT	Top	0	203	20	37	500	-	640	1.0	0.40	-	60	122.5	815
	Bottom	11	192	12	37	500	-	640	1.0	0.40	-	60	122.5	815
OC CLAY (a)	Top	11	192	10	34	3750	20000	1760	11.4	1.00	11	75	122.5	850
	Bottom	17	186	10	34	3750	20000	1760	11.4	1.00	11	75	122.5	850
OC CLAY (b)	Top	17	186	10	32	3000	15000	2450	6.1	1.00	11	75	122.5	850
	Bottom	21	182	7	32	3000	15000	2450	6.1	1.00	11	75	122.5	850
OC CLAY (c) (Above GW)	Top	21	182	7	30	1750	10000	2875	3.5	0.93	11	75	122.5	850
	Bottom	25	178	4	30	1750	10000	2875	3.5	0.93	11	75	122.5	850
NC CLAY (Below GW)	Top	25	178	4	25	1200	5000	3365	1.5	0.68	11	75	117.5	900
	Bottom	36	167	4	25	1200	5000	3365	1.5	0.68	11	75	117.5	900
Lean CLAY (a)	Top	36	167	7	29	1500	7500	3915	1.9	0.71	11	70	117.5	900
	Bottom	45	158	5	29	1500	7500	3915	1.9	0.71	11	70	117.5	900
Lean CLAY (b)	Top	45	158	5	29	2000	10000	4440	2.3	0.76	11	70	117.5	825
	Bottom	55	148	5	29	2000	10000	4440	2.3	0.76	11	70	117.5	825
Dense SAND (a)	Top	55	148	36	42	-	-	5235	1.0	0.33	-	3	127.5	1250
	Bottom	70	133	36	42	-	-	5235	1.0	0.33	-	3	127.5	1250
Dense SAND (b)	Top	70	133	36	43	-	-	6275	1.0	0.32	-	3	130	1485
	Bottom	87	116	50	43	-	-	6275	1.0	0.32	-	3	130	1485
Half Space	Top	87	116	-	-	-	-	-	-	-	-	-	130	1750

Notes:
1. K_0 is the at-rest earth pressure coefficient.
2. OCR is over-consolidation ratio.
3. PI is plasticity index.

Based on the in-situ groundwater levels, the design groundwater is selected at the depth of 20 feet below ground surface (bgs), for the site response analyses. The shear strength is calculated at the mid-depth of each soil layer using the vertical effective stress and friction angle for sands and estimated as the undrained shear strength using the available field and laboratory data and correlations for clays. The design shear wave velocity profiles are estimated primarily using the available shear wave velocity data measured from the two seismic cone penetration tests (CPTs), SCPT-05 and CPT-06, completed by Kleinfelder in 2022. The layer thickness is selected for the nonlinear analyses such that the maximum frequency that can propagate through each soil layer is greater than 30 Hz.

Site Class

Based on the shear wave velocity data, the site is classified as a Site Class D site per Section 20.3.1 of ASCE 7-16.

For the purpose of site response analysis, the base of the soil column is defined as Site Class C as discussed below.

The shear wave velocities from the seismic CPTs are plotted in Figure F1, along with the design shear wave velocity profiles. It should be noted that the V_{S30} value used to evaluate the site class is different from that, as mentioned below, used to develop the base target spectrum.

For site response analysis, base of the soil column was selected at a depth of about 88 feet based on the data provided in SCPT-05. The CPT sounding encountered refusal at that depth and shear wave velocity was in excess of 1,500 feet/sec, it represented a Site Class C for the purposed of developing target spectrum for site response analysis. At the depth of 88 feet, the V_{S30} value was assumed to be about 1,750 ft/s (533 m/s) at the base of the soil column which is within the limits used to define Site Class C and will be used to develop the input base target spectrum.

DEVELOPMENT OF BASE GROUND MOTIONS

Development of base ground motions includes developing a target response spectrum at the base of the soil columns and then selecting and spectrally matching time histories to the target spectrum to be used in the site response analyses. According to Section 21.1.1 of ASCE 7-16, the base target spectrum should be an MCE_R response spectrum. A site-specific ground motion hazard analysis (GMHA) was performed per Section 21.2 of ASCE 7-16 to develop the base target MCE_R spectrum which will be used for selecting and spectrally matching the input base ground motions. The GMHA generally includes a representative seismic source model (geometry, style of faulting, magnitude, etc.), appropriate recurrence relationships, and appropriate ground motion models (aka. attenuation relationships).

First, a brief discussion of the seismotectonic setting, regional faulting and historic seismicity is provided below. The regional seismotectonic setting, regional faulting and historic seismicity inform the selection of an appropriate seismic source model and provide context for the likely potential for future earthquakes to impact the site.

Seismotectonic Setting

The site is located within the Hollister Valley, at the southern end of the greater Santa Clara Valley, in the Coast Ranges geomorphic province in the Western United States (WUS) along the predominantly right lateral transform margin between the Pacific and North American Plates. The Coast Ranges are northwest-trending mountain ranges and valleys, subparallel to the San Andreas fault zone. The Hollister Valley is bounded by the Gabilan Range to the southwest and the Diablo Range to the northeast. Regionally, stress build up is associated with the northeast relative movement of the Pacific Plate and extensional relaxation of the Basin and Range. These stresses are accommodated primarily by displacements on faults within the San Andreas fault zone, and to a lesser extent by displacements on faults within the Walker Lane Belt (near California/Nevada border, Unruh and Humphrey, 2017; Field et al., 2013).

Regional Faulting and Historic Seismicity

Based on the information provided in Bryant and Hart (2007), the site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone where site-specific studies addressing the potential for surface fault rupture are required and no known active faults are mapped traversing the site. However, the site is located close to several active faults where historic seismic activities have been observed in the past.

The nearest fault to the site is the Hayward fault zone located approximately 5.5 km to the east of the site. Other nearby major fault sources include the N. San Andreas fault zone, the Sargent fault, the Monte Vista-Shannon fault and the Calaveras fault. Any seismic activity on these faults could cause significant ground shaking at the site. These and other potential seismogenic sources are identified on Figure F2.

Patterns of historic seismicity are used to identify potentially active sources, develop on- and off-fault recurrence rates, and understand the historic impacts from seismicity at a site. A catalog of events is typically used, such as those developed and used by the Uniform Earthquake Rupture Forecast version 3 (UCERF3, Field et al., 2013). For this study, we compiled and reviewed data from the USGS ANSS Comprehensive Earthquake Catalog which contains data from multiple sources from 1800 to 2021 within 300 km of the site. We also reviewed the catalog of historic events developed and used by the UCERF3 project. Comparison of these two catalogs indicates generally good agreement. In addition, we obtained information about California earthquakes having M≥5 from CGS Map Sheet 49 (Toppozada et al., 2000).

Significant regional seismicity includes the 1868 (M_w 6.8) Hayward earthquake, the 1906 San Francisco (M_w 7.9) earthquake, the 1838 San Andreas Fault (M_w 7.4) earthquake, and the 1989 Loma Prieta (M_w 6.9) earthquake. Historic seismicity within 100 km of the site is depicted on Figure F-2.

A publication prepared by the U.S. Geological Survey regarding earthquake probabilities in the Bay Area (Working Group on California Earthquake Probabilities, 2014) concludes that there is a 72 percent chance that one of the major faults within the Bay Area will experience a major (M 6.7+) earthquake during the period of 2014 to 2043. This publication also shows that there is a 51 percent chance of M 7+ earthquake and 20 percent chance of M 7.5+ happening before 2043. These probabilities are significant and require mitigation. As has been seen in the past earthquakes such as the 1994 (M 6.7) Northridge earthquake, that this level of shaking could cause significant damage to the built environment.

Seismic Source Model

Based on our review of the seismotectonic setting and nearby active sources we have selected the Petersen et al. (2014) source model, which has been used in USGS 2014 National Seismic Hazard Mapping program, as the base model for our evaluations. The Petersen et al. (2014) source model generally uses the sources developed by the UCERF3 project within California (which utilizes two alternative fault models, FM 3.1 and 3.2) to model on-fault seismicity. Off-fault seismicity (e.g. background seismicity) is modeled using gridded seismic sources from UCERF3. The UCERF3 source model has been used in developing the 2014 USGS National Seismic Hazard Maps and is appropriate for use in modeling seismicity based on the 2019 CBC.

Shallow crustal fault sources from the regional model within 300 km of the site have been included in the model, with subduction earthquake sources included out to 1000 km as recommended by the USGS (Petersen et al., 2014). Based on review of the nearby and significant sources it was felt that the existing UCERF3 model generally adequately captured the seismicity in the region. The source model used for this work is shown on Figure F-3

'Grand Inversion' and Recurrence Rates

The earthquake recurrence rates used within the source model used for this project were derived from work completed for UCERF3 as implemented by Petersen et al. (2014) using the branch averaged solutions of the 'grand inversion'. The 'grand inversion' scheme used by the UCERF3 project team 'solved' the on-fault and off-fault recurrence rates at a system level using a set of defined constraints including the spatial probability density of off-fault seismicity, slip rate balancing, paleoseismic event rate matching, fault smoothness constraint, regional magnitude frequency distribution constraints, and fault section specific magnitude frequency distribution constraints. In simple terms the 'grand inversion' solves for three things: large on-fault (supra-seismogenic) event rates; small, near-fault (subseismogenic) event rates; and truly off-fault (unassociated) event rates. The supra-seismogenic 'on-fault' events are ultimately modeled using linear fault sources; while the latter two categories (subseismogenic and off-fault) are considered 'background seismicity' and are modeled using spatially smoothed 'grid' of evenly spaced cells (aka. gridded seismicity). The combined on-fault and off-fault solution set (fault system solution) used the logic tree solution framework shown in a generalized form on Figure E4; and our model implemented the branch averaged solutions.

In the source model used for this work, the on-fault seismicity considers two potential alternative fault models, equally weighted, identified as fault model 3.1 (FM 3.1) and fault model 3.2 (FM 3.2). These fault models each contain a slightly different collection of fault traces that are broken into 'segments' for modeling purposes, with individual 'segments' strung together to create hundreds of thousands of potential fault-based ruptures or multi-rupture events. In our model, fault segments are modeled using a 'characteristic' magnitude frequency distribution (originally described by Schwartz and Coppersmith, 1984) with the recurrence rates constrained during the 'grand inversion' by the UCERF2 'characteristic' inversion branch. Fault slip rates (deformations) are constrained by a combination of a 'pure' geologic deformation model and three other models that consider geologic and geodetic data including the average fault block model, NeoKinema model, and Zeng-Shen model. The magnitude-area relationships used along with the associated slip-length models as well as other solution constraints applied are shown with weights on Figure F-4 and discussed in detail in Field et al. (2013).

Off-fault seismicity (e.g. background seismicity) recurrence rates are solved for simultaneously by the 'grand inversion'. This off-fault background seismicity is intended to include all smaller earthquakes (M_w<~6.5) on major faults and earthquakes of all sizes that are not associated with known faults. The off-fault background seismicity considers spatial smoothing of a 'grid' of evenly spaced cells using the spatial probability density function (PDF) grids of off-fault seismicity from UCERF2 and UCERF3 (equally weighted), considering the regional constraints on the model including the total regional magnitude-frequency distribution (e.g. Gutenberg-Richter distribution), maximum off-fault magnitude (7.3, 7.6, or 7.9 weighted as 0.1, 0.8 and 0.1 respectively), number of regional events per year greater than a magnitude 5 (considered between 6.5 and 9.6) and other constraints as shown on the logic tree (see Figure E4). The branch averaged gridded seismicity solution for each fault model, as implemented by Petersen et al. (2014), is used in the source model for this work.

Ground Motion Models

Site-specific ground motions can be influenced by the styles of faulting, magnitudes of the earthquakes, and local soil conditions. Other effects such as near source or basin effects can also influence the ground motions. The ground motion models (GMMs) used to estimate ground motion from an earthquake source need to directly or indirectly consider these effects. Many GMMs have been developed to estimate the variation of spectral acceleration with earthquake magnitude and distance from the site to the source of an earthquake.

We have used four of the Next Generation Attenuation (NGA) West 2 relationships including Abrahamson et al. (2014), Boore et al. (2014), Chiou and Youngs (2014), and Campbell and Bozorgnia (2014) with equal weights applied for all crustal faults (e.g. reverse, strike-slip, normal) included in the fault model. Idriss (2014) GMM has not been used as the seismic source distances used in our analyses are outside the limits of Idriss (2014) GMM and the V_{S30} used for the base target spectrum is outside the range of this relationship. The $Z_{1.0}$ and $Z_{2.5}$ values of about 295 m and 0.9 km were used based on the values obtained from nearby recording stations and using the V_{S30} value of 533 m/s per Abrahamson et al. (2014) and Campbell and Bozorgnia (2014) relationships, respectively.

Spectral acceleration values were obtained by averaging the individual hazard results. These GMMs provide 'mean' (RotD50) values of ground motions associated with magnitude, distance, site soil conditions, and mechanism of faulting.

Ground Motion Hazard Analysis

According to ASCE 7-16, the Risk-Targeted Maximum Considered Earthquake (MCE_R) is the most severe earthquake load considered by that standard and is considered at the orientation that results in the maximum response to horizontal ground motions with adjustment for targeted risk as defined by that standard. The site-specific MCE_R is developed in accordance with Section 21.2 of ASCE 7-16 using a site-specific ground motion hazard analysis procedure and is the lesser of: (1) the probabilistic MCE_R ground motion taken as the five percent damped uniform hazard spectrum for a 2 percent probability of exceedance in 50 years (i.e., return period of about 2,475 years) adjusted for risk factors and for the maximum direction; and (2) the deterministic MCE_R ground motion taken as the 84th percentile (median + 1 standard deviation) deterministic values (adjusted for the maximum direction) from the controlling fault(s) factored as required by Section 21.2.2 of ASCE 7-16. The resulting site-specific spectrum may not be less than the 80 percent of the code spectrum developed in accordance with Chapter 11 of ASCE 7-16.

Both probabilistic and deterministic seismic hazard analyses should be used to estimate the spectral accelerations used to develop the site-specific MCE_R unless the deterministic spectrum need not be calculated per Section 21.2.2 of ASCE 7-16 Supplement 1. Details of our evaluation are provided below.

For this work, a probabilistic seismic hazard analysis (PSHA) procedure was used to estimate the ground motion parameters (e.g. peak and spectral ground accelerations). The PSHA approach uses a logic tree approach to appropriately account for epistemic and aleatoric uncertainty in the model. The logic tree includes information about uncertainties in the source models, ground motion models, and other items impacting the results. Important source characteristics include such items as magnitude and recurrence interval of potential seismic events, distance from the site to the causative source, and other parameters. The effects of site soil conditions and other considerations such as basin effects can be accounted for using ground motion models (GMMs).

The theory behind the empirical probabilistic approach to seismic risk analysis has been developed over many years (Cornell, 1968, 1971; Merz and Cornell, 1973; SSHAC, 1997), and is based on the "total probability theorem". Generally, this work uses an assumption that earthquake events are independent of time and space from one another (e.g. time-independent models). According to this approach, the probability of exceedance $P_E(z)$ at a given level of ground motion, z, at the site within a specified time period, t, is related to the annual frequency of exceedance v(z) by:

$$P_E(z) = 1 - \exp(-v(z) * t)$$

Different probabilities of exceedance may be selected, depending on the level of performance required. The return period is essentially equivalent to the reciprocal of v(z).

The PSHA is conducted using three generalized steps: 1) development of an appropriate seismic source model including source characterization, development of recurrence relationships, and appropriately capturing uncertainty, 2) selection of appropriate ground motion models (and site amplification models if appropriate), and 3) conducting the calculation and processing the results. The annual frequency of exceedance of a certain ground motion level can be found by summing the rates for all sources, N, with the rate for each source determined by summing over all magnitudes and source to site distances, and so forth. The annual frequency of occurrence of earthquakes of magnitude, m_i, on seismic source, n, is $\lambda(m_i)$. The probability of an earthquake of magnitude m_i on source n occurring at a certain distance, r_j, from the site is $P(R = r_j \mid m_i)$ while the probability that the ground motion level, z, will be exceeded is given as $P(Z>z \mid m_i, r_j)$. Thus, mathematically the basic formulation for the annual frequency of exceedance, v(z), is given by:

$$v(z) = \sum_N \left[\sum_M \lambda(m_i) * \sum_R P\left(R = r_j \middle| m_i\right) * P\left(Z > z \middle| m_i, r_j\right) \right]$$

Modern computers make the above calculation, while computationally expensive, easily implementable. We have used the computer program HAZ (Powers, 2017) for our probabilistic analysis which implements the above general equation and evaluations of the probability of exceedance. Uncertainties are accounted for within the source model using the logic tree approach and source model discussed previously.

The deterministic seismic hazard analysis (DSHA) approach is also based on the characteristics of the earthquake and the causative fault associated with the earthquake. These characteristics include such items as magnitude of the earthquake and distance from the site to the causative fault. The effects of site soil conditions and mechanism of faulting are also accounted for in the GMMs. Per ASCE 7-16, the 84th percentile deterministic site-specific spectral acceleration values should be used for DSHA with the exception that the deterministic spectrum need not be calculated when the largest spectral acceleration from the probabilistic spectrum is less than $1.2*F_a$. If the largest spectral acceleration from the resulting deterministic 84^{th} percentile maximum horizontal spectrum is less than $1.5*F_a$ then the spectrum is scaled by a single factor such that the maximum spectral value equals $1.5*F_a$. The value of F_a is taken from either Table 11.4.1 (Site Classes A to D) with a value of S_s equal to 1.5, or set equal to 1.0 (Site Class E) for purposes of these comparisons.

Since the site is located close to several significant faults in the region, for the deterministic evaluations, we used the NGA West 2 spreadsheet (PEER, 2015) to calculate deterministic values for some of the nearby faults, and used the USGS Earthquake Scenario Map (BSSC 2014) (from https://earthquake.usgs.gov/scenarios/catalog/) in determining the contributing faults and their parameters for deterministic analysis. The faults include the Hayward fault zone, N. San Andreas fault zone, Sargent fault, Monte Vista-Shannon and Calaveras fault. Based on our results, the deterministic spectrum is controlled by the Hayward fault zone with M7.58 and Rrup of about 5.5 km. The computed 84th percentile deterministic spectra adjusted for maximum direction is shown on Figure F-5.

Site-Specific Target MCE$_R$ Spectrum

To develop the site-specific target response spectrum, we first obtained the general seismic design parameters, as presented in Table F2, based on the site class, site coordinates, and the risk category based on Chapter 11 of ASCE 7-16 using the OSHPD online tool (https:/seismicmaps.org) which accesses the USGS database.

TABLE F2: GENERAL GROUND MOTION PARAMETERS BASED ON ASCE 7-16

Parameter	Value	ASCE 7-16 Reference
S_S	1.602g	Fig 22-1
S_1	0.6g	Fig 22-2
Site Class	C	Table 20.3-1
F_a	1.2	Table 11.4-1
F_v	1.4	Section 11.4.8
S_{MS}	1.922g	Eq. 11 4-1
S_{M1}	0.840g	Section 11.4.8
S_{DS}	1.281g	Eq. 11.4-3
S_{D1}	0.56g	Section 11.4.8
C_{RS}	0.95	Fig 22-18A
C_{R1}	0.93	Fig 22-19A
PGA	0.673g	Fig 22-9
F_{pga}	1.2	Table 11.8-1
PGA_M	0.807g	Eq. 11.8-1
T_L	12 seconds	Fig 22-14

[1]N/A = Not Applicable; Section 11.4.8 of ASCE 7-16 requires a site-specific ground motion hazard analysis be performed for Site Class D sites with S_1 values greater than or equal to 0.2g. However, if exceptions are taken, then an F_v value of 1.7 could be used only to calculate the Ts value.

The MCE$_R$ response spectrum is generally developed by comparing probabilistic, deterministic, and 80% of the general procedure code spectra. The NGA West 2 GMMs present the spectral accelerations in terms of 'mean' (RotD50) values of the rotated two horizontal components of ground motion. To estimate spectral accelerations in the direction of the maximum horizontal response (e.g. RotD100) at each period from geometric mean values, we have used the scaling factors of Shahi and Baker (2014). These values were used as they more accurately represent the appropriate factors to apply using the NGA West 2 relationships. These factors are shown in Table F3. In addition, the probabilistic spectrum was adjusted for targeted risk using risk coefficients C_{RS} and C_{R1} (e.g. Method 1 of Section 21.2.1 of ASCE 7-16). C_{RS} and C_{R1} were estimated from Figures 22-18A and 22-19A of ASCE 7-16 and are listed in Table F3. C_{RS} is applied on periods of 0.2s or less and C_{R1} is applied on periods of 1.0s or greater and linear interpolation in between as shown in Table F3.

TABLE F3: RISK COEFFICIENTS AND ROTATION FACTORS

Period (second)	Risk Coefficient (ASCE 7-16)	Shahi and Baker (2014) Max Rotation Factor
0.010	0.950	1.190
0.200	0.950	1.210
0.300	0.948	1.220
0.500	0.943	1.230
1.000	0.930	1.240
2.000	0.930	1.240
5.000	0.930	1.260

As mentioned earlier, the geometric mean 84th percentile deterministic values were estimated for the nearby faults and the largest values were then adjusted for the maximum direction. Since the maximum deterministic spectral acceleration is greater than 1.5*F_a, as shown in Figure F-5, there is no need to scale it up to estimate the deterministic lower limit and this spectrum is the governing deterministic spectrum.

Probabilistic and deterministic spectral values are compared in Table F4 for some specific periods and the two spectra are compared in Figure F-6. Figure F-6 and Table F4 show that the deterministic spectrum is lower than the probabilistic spectrum for some periods and higher at some periods. Therefore, the site-specific MCE_R spectrum is governed by both the probabilistic and deterministic spectra. The DE spectrum was developed by taking two-thirds of the MCE_R spectrum. Spectral values for the site-specific MCE_R and DE spectra are listed in Table F5 for some periods, which also compares the DE spectrum with the 80% of the code DE spectrum at those periods. Figure F-7 shows the comparison of the DE spectrum with the 80% of the code DE spectrum. Figure F-7 and Table F5 show that the DE spectrum is higher than the 80% of the code DE spectrum for all periods. Therefore, the site-specific DE spectrum is controlled by the site-specific spectrum. The site-specific MCE_R spectrum is taken as 1.5 times the site-specific DE spectrum, as shown on Figure F-8. Spectral acceleration values for the MCE_R spectrum are listed in Table F6. This MCE_R spectrum is the target spectrum used for selection and spectral matching of the input time histories for site response analyses.

TABLE F4: RESULTS FROM SITE-SPECIFIC GMHA

Period (second)	84th-Percentile Deterministic Geometric Mean Sa (g)	Max Dir Deterministic Sa (g)	2% in 50 Year Probabilistic Geometric Mean Sa (g)	Risk-Targeted Max Dir Probabilistic Sa (g)
0.010	0.799	0.951	1.038	1.174
0.200	1.921	2.324	2.632	3.025
0.300	1.866	2.276	2.403	2.778
0.500	1.508	1.855	1.824	2.115
1.000	0.914	1.134	1.029	1.187
2.000	0.451	0.560	0.477	0.550
5.000	0.166	0.209	0.172	0.201

TABLE F5: COMPARISON OF DESIGN EARTHQUAKE AND 80% CODE SPECTRA

Period (second)	Site-Specific MCE_R Sa (g)	Site-Specific DE Sa (g)	80% Code DE Sa (g)
0.010	0.951	0.634	0.410
0.200	2.324	1.549	1.025
0.300	2.276	1.518	1.025
0.500	1.855	1.237	0.896
1.000	1.134	0.756	0.448
2.000	0.550	0.366	0.224
5.000	0.201	0.134	0.090

TABLE F6: SITE-SPECIFIC HORIZONTAL TARGET MCE_R SPECTRAL ACCELERATIONS

Period (second)	MCE_R Sa (g) 5% Damping
0.010	0.951
0.020	0.968
0.030	1.041
0.050	1.264
0.075	1.572
0.100	1.806
0.150	2.154
0.200	2.324
0.250	2.354
0.300	2.276
0.400	2.074
0.500	1.855
0.750	1.423
1.000	1.134
1.500	0.757
2.000	0.550
3.000	0.351
4.000	0.257
5.000	0.201

Controlling Earthquake Magnitude and Distance

The target spectrum is controlled primarily by the Hayward fault with a magnitude of 7.58 at a distance of about 5.5 km. However, there are other major faults such as Monte Vista-Shannon, N. San Andreas, and Sargent faults within 15 km of the site which could impact the seismic hazard at the site. Due to this, we considered an earthquake magnitude range of 6.5 to 7.7 and distance range of 1 to 15 km for selecting time histories for site response analysis. In addition, we considered both strike slip and reverse fault mechanisms when selecting time histories.

Time History Selection and Spectral Matching

Using the target MCE_R response spectrum provided in Figure F8 and Table F6, a suite of five time histories were selected from the PEER Strong Ground Motion Database (PEER, 2014) and spectrally matched to the target spectrum for use in the site response analysis in accordance with ASCE 7-16. The time histories were selected based on several criteria including near-fault pulse motions, scaling factor, site-to-source distance, magnitude, V_{S30}, Arias intensity, duration, style of faulting, shape of response spectrum, etc. These time histories were selected and modified for use in site response analysis only and may not be appropriate for other applications.

Due to the site's close proximity to the Hayward fault zone, both pulse and non-pulse motions were considered during selection of time histories. Based on the methodology presented in Hayden et al. (2014), the distance from the site to the Calaveras fault zone, and an estimated epsilon value of about 1.5 to 2, we estimated that the proportion of pulse motions to be selected for the site response analysis is four pulse motions out of five, with the remainder being non-pulse motion.

Other selection parameters included magnitude and V_{S30}, in which time histories with the magnitude and V_{S30} values relatively close to the dominant magnitudes from the deaggregation analyses and the V_{S30} value of 533 m/s (1,750 ft/sec) used for the target spectrum were selected. Considerations for Arias intensity and duration of the ground motions used the methodologies of Travasarou et al. (2003) and Bommer et al. (2009), respectively. The significant durations of the selected ground motions were found to be within the expected durations and sufficiently long enough to induce potential liquefiable behavior of specific layers of the soil profiles during the site response analyses.

Based on these criteria, a suite of five time histories was selected from the PEER database that had a spectral shape after scaling (scaling factors less than 3) generally around the base target response spectrum. These selected ground motion time histories and their associated characteristics are provided in Table F7.

TABLE F7: SELECTED TIME HISTORIES FROM PEER DATABASE

Record No.	Event Name/Station	Year	M_w	Distance R_{rup} (km)	V_{S30} (m/s)	Faulting Mechanism	$D_{5\text{-}95}$ (sec)	I_A (m/s)	LUF (Hz)	Pulse Period (sec)
RSN 802	Loma Prieta/Saratoga – Aloha Ave	1989	6.93	8.5	381	RO	9.5	1.5	0.125	4.571
RSN 828	Cape Mendocino/ Petrolia	1992	7.01	8.2	422	R	17.7	3.8	0.07	2.996
RSN 1111	Kobe-Japan/Nishi-Akashi	1995	6.9	7.08	609	SS	11.2	3.4	0.125	-
RSN 1511	Chi-Chi - Taiwan/TCU076	1999	7.62	2.74	615	RO	29.5	3.7	0.125	4.732
RSN 4847	Chuetsu-Oki-Japan/Joetsu-Kakizakiku	2007	6.8	11.94	383	R	20.3	2.3	0.0875	1.4

Notes: Definitions: M_w – Moment Magnitude; R – Reverse fault; RO – Reverse Oblique fault; SS – Strike-slip fault; $D_{5\text{-}95}$ – Significant Duration; I_A – Arias Intensity; LUF – Lowest Usable Frequency

The selected ground motions from the PEER database were then modified by performing spectral matching using the RSPMatch program developed by Atik and Abrahamson (2010) as implemented in the computer program EZ-FRISK^{TM} (Risk Engineering, 2018) which generally implements the spectral matching algorithm proposed by Lilhanand and Tseng (1987, 1988) with an updated wavelet adjustment to preserve the non-stationary characteristics of the ground motions. Spectral matching was completed such that the resulting spectrum was generally in good agreement with the target spectrum particularly over the period range of interest. The spectrally matched ground motions were compared with the PEER database original ground motions to ensure that the matching process retained the non-stationary characteristics of the record.

The selected, spectrally matched time histories used as the input "outcrop" ground motions in the site response analyses, along with the original time histories as obtained from the PEER database, are presented in Figures F-9 through F-13.

Site Response Analysis

Site response analysis was completed in general accordance with the 2019 CBC and Section 21.1 of ASCE 7-16. Evaluations were completed using the design soil profiles, the selected and spectrally matched time histories as the “outcrop” ground motions in conjunction with one-dimensional (1D) nonlinear time-domain total stress (without porewater pressure generation), site response analyses using the computer program DEEPSOIL V7.0 (Hashash et al., 2020). Nonlinear effective stress analysis was not needed as the liquefaction potential for this site is quite low. Results of the site response analysis were used to develop the site-specific design acceleration response spectrum and seismic design parameters for the project. Details of the site response analysis methodology and results are presented in the subsequent sections.

Models and Parameters

The shear modulus reduction and damping ratio curves for the soils were estimated using the Darendeli (2001) model. In addition to the material properties listed in Tables F1, the loading frequency and number of loading cycles were specified 1 Hz and 10, respectively, per the recommendations of Darendeli (2001). To account for the design shear strengths at large shear strain levels, the reference backbone curves by Darendeli (2001) were used to fit the general quadratic/hyperbolic (GQ/H) backbone curve, as described by Groholski et al. (2016) and implemented in DEEPSOIL V7.0. In addition, the reference damping ratio curves were used to fit the non-Masing unloading-reloading relationship using the modulus reduction and damping curve fitting (MRDF) Pressure-Dependent Hyperbolic model (Phillips and Hashash, 2009). The fitting procedure was applied to each layer in the soil model. Therefore, the soil model can capture better both the small-strain soil behavior and the design shear strength at larger strain levels.

The viscous small-strain damping used a frequency independent formulation implemented in DEEPSOIL V7.0 as recommended by Hashash et al. (2020). The selected, spectrally matched acceleration time histories were inputted as “outcrop” ground motions at the base of the two soil profiles. The elastic half space is selected to model the “bedrock” at the base of the soil columns, as the “outcrop” ground motions are being used.

Results and Recommendations

Results of site response analysis in terms of shear strains with depth for each time history are presented in Figure F-14. The surface response spectra of all the ground motions and the average response spectrum curves are plotted in Figure F-15, along with the input base response spectrum plotted for comparison. To develop the site-specific design acceleration response spectrum, amplification factors were calculated based on the input motions and surface motions. The resulting average amplification factors are presented in Figure F-16. Figure F-17 shows the two-thirds of average surface spectrum using the amplification factors. This figure also shows the two-thirds average of the surface response spectra at the ground surface for all time histories.

According to Section 21.3 of ASCE 7-16, the ground surface MCE_R response spectrum should not be less than 80 percent of the code MCE_R spectrum for Site Class D. The Site Class D can be justified using the site-specific shear wave velocity measurements, therefore, the exception in Section 21.3 of ASCE 7-16 is taken. The site-specific MCE_R response spectrum as developed per Section 21.1.3 of ASCE 7-16 was compared with 80 percent of the code MCE_R spectrum for Site Class D developed in accordance with Section 11.4.6 of ASCE 7-16, where the site coefficients F_a and F_v are determined per Section 21.3 of ASCE 7-16.

The site-specific design spectrum is then compared with the 80% of the code spectrum as shown in Figure F-17. We enveloped the site-specific and 80% of the code spectrum to obtain the site-specific design response spectrum. Site-specific MCE_R spectrum was obtained by multiplying the design spectrum by 1.5. Resulting DE and MCE_R horizontal response spectra are shown in Figure F-18. The recommended site-specific DE and MCE_R spectral acceleration values are provided in Table F8.

TABLE F8: RECOMMENDED SITE-SPECIFIC HORIZONTAL SPECTRAL ACCELERATIONS

Period (second)	DE Sa (g)	MCE_R Sa (g)
	5% Damping	
0.01	0.369	0.554
0.02	0.454	0.681
0.03	0.516	0.774
0.045	0.740	1.111
0.05	0.561	0.842
0.060	0.589	0.884
0.075	0.752	1.128
0.08	0.810	1.215
0.10	0.838	1.258
0.135	0.804	1.205
0.15	0.752	1.129
0.19	0.854	1.282
0.20	0.854	1.282
0.25	0.854	1.282
0.30	0.854	1.282
0.40	0.854	1.282
0.50	0.854	1.282
0.75	0.854	1.282
0.94	0.854	1.282
1.00	0.800	1.200
1.10	0.747	1.120
1.50	0.560	0.840
1.75	0.579	0.868
2.00	0.530	0.796
2.50	0.454	0.681
3.00	0.340	0.510
4.00	0.235	0.352
5.00	0.178	0.267

Site-specific ground motion parameters were estimated using the site-specific design response spectrum presented above. According to Section 21.4 of ASCE 7-16, the S_{DS} value should be taken as 90 percent of the maximum spectral acceleration at any period between 0.2 and 5 seconds. Since the site's V_{S30} value is less than 1,200 ft/s (366 m/s), the S_{D1} value is taken as the maximum value of $T*S_a$ between periods of 1 and 5 seconds, where T is the period and S_a is the corresponding spectral acceleration. These spectral accelerations shall not be taken as less than 80 percent of the code-based values as determined by Sections 11.4.4 and 11.4.5 of ASCE 7-16.

For this site, S_{DS} value is controlled by the 80 percent of the code-based value, and S_{D1} value is controlled by the spectral acceleration value at 2.5 seconds. The parameters S_{MS} and S_{M1} are taken as 1.5 times S_{DS} and S_{D1}. Site-specific values of S_{DS}, S_{D1}, S_{MS}, and S_{M1} are presented in Table F9. It should be noted that site-specific S_{D1} and S_{M1} are greater than S_{DS} and S_{MS}, respectively.

TABLE F9: SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Parameter	Value (5% Damping)
S_{DS}	0.854g
S_{D1}	1.135g
S_{MS}	1.282g
S_{M1}	1.702g

According to Section 21.5 of ASCE 7-16, the site-specific Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration, PGA_M, should not be less than 80 percent of the value obtained from equation 11.8-1 of ASCE 7-16. Therefore, the site-specific PGA_M is 0.646g and controlled by the 80% of the code-based PGA_M value, as listed in Table F2. The moment magnitude of about 7.58 from the Hayward fault zone may be used in any geotechnical evaluations at this site.

The Seismic Design Category is determined as specified in the 2019 California Building Code Section 1613.2.5. We understand that the structure is classified as a Risk Category II structure. Based on this and the site-specific seismic design parameters developed above the structure is classified as a Seismic Design Category D.

REFERENCES

Abrahamson, N., Silva, N., and Kamai, R. (2014). Summary of the ASK14 Ground Motion Relation for Active Crustal Regions. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1025-1055.

American Society of Civil Engineers (2016). Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16.

Atik, L.A. and Abrahamson, N., 2010, An Improved Method for Nonstationary Spectral Matching, Earthquake Spectra, Vol. 26, No. 3, pp. 601-617, August 2010.

Bommer, J.J., Stafford, P.J., and Alarcón, J.E., 2009, Empirical Equations for the Prediction of the Significant, Bracketed, and Uniform Duration of Earthquake Ground Motion, Bulletin of the Seismological Society of America, Vol. 99, No. 6, pp. 3217-3233, December 2009.

Boore, D., Stewart, J., Seyhan, E., and Atkinson, G. (2014) NGA-West2 Equations for Predicting PGA, PGV, and 5% Damped PSA for Shallow Crustal Earthquakes. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1057-1085.

Bryant, W.A., and Hart, E.W. (2007) Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps: California Geological Survey Special Publication 42 (Interim Revision 2007).

California Building Standards Commission (2019). 2019 California Building Code (CBC), California Code of Regulations, Title 24, Part 2.

Campbell, K., and Bozorgnia, Y. (2014) NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1087-1115.

Carlton, B. (2014), An Improved Description of the Seismic Response of Sites with High Plasticity Soils, Organic Clays, and Deep Soft Soil Deposits, Ph.D. Dissertation, University of California, Berkeley.

Chiou, B., and Youngs, R. (2014) Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1117-1153.

Cornell, C.A. (1968), Engineering Seismic Risk Analysis, Bulletin of the Seismological Society of America, Vol. 58, No. 5.

Cornell, C.A. (1971), Probabilistic Analysis of Damage to Structures Under Seismic Loads, Dynamic Waves in Civil Engineering, London.

Darendeli, M. B. (2001), Development of a New Family of Normalized Modulus Reduction and Material Damping Curves, Ph.D. Dissertation, Department of Civil, Architectural and Environmental Engineering, University of Texas, Austin, Texas.

Field, E., Biasi, G., Bird, P., et al., 2013, The Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3). USGS Open File Report 2013-1165; CGS Special Report 228, Southern California Earthquake Center Publication 1792.

Groholski, D., Hashash, Y., Kim, B., Musgrove, M., Harmon, J., and Stewart, J., 2016, Simplified Model for Small-Strain Nonlinearity and Strength in 1D Seismic Site Response Analysis, Journal of Geotechnical and Geoenvironmental Engineering.

Gutenberg, B. and Richter, C.F. (1956), Earthquake Magnitude, Intensity, Energy and Acceleration, Bulletin of the Seismological Society of America, Vol. 46, No. 2.

Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Ilhan, O., Xing, G., Numanoglu, O., Groholski, D.R., Phillips, C.A., and Park, D. (2020), DEEPSOIL 7.0, User Manual, Urbana, IL, Board of Trustees of University of Illinois at Urbana-Champaign.

Hayden, C.P., Bray, J.D., and Abrahamson, N.A., 2014, Selection of Near-Fault Pulse Motions, Journal of Geotechnical and Geoevironmental Engineering, March 2014.

Idriss, I. M. (2014). An NGA-West2 Empirical Model for Estimating the Horizontal Spectral Values Generated by Shallow Crustal Earthquakes. *Earthquake Spectra, 30*(3), 1155-1177.

Kleinfelder (2021), Geotechnical Investigation Report, Duke Almond Project, Hollister, California, dated October 29, 2021.

Lilhanand, K., and Tseng, W. S. (1987), Generation of synthetic time histories compatible with multiple-damping response spectra, SMiRT-9, Lausanne, K2/10.

Lilhanand, K., and Tseng, W. S. (1988), Development and application of realistic earthquake time histories compatible with multiple damping response spectra, in Ninth World Conference on Earthquake Engineering, Tokyo, Japan, Vol 2, 819-824.

Matasovic, N., and Vucetic, M. (1993), Cyclic Characterization of Liquefiable Sands, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 119, No. 11, 1805-1822.

Matasovic, Neven, and M. Vucetic (1995), Generalized Cyclic Degradation-Pore Pressure Generation Model for Clays, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 121, No. 1, 33-42.

Mei, X., Olson, S.M., and Hashash, Y.M. (2015), Empirical curve-fitting parameters for a porewater pressure generation model for use in 1-D effective stress-based site response analysis, 6th International Conference on Earthquake Geotechnical Engineering, 1-4 November 2015, Christchurch, New Zealand.

Merz, H.L. and Cornell, C.A. (1973), Seismic Risk Analysis Based on a Quadratic Magnitude-Frequency Law, Bulletin of the Seismological Society of America, Vol. 63, No. 6.

Pacific Earthquake Engineering Research Center (PEER), 2014, PEER NGA-West2 Database (PEER Report 2013/03), by: Timothy D. Ancheta, Robert B. Darragh, Jonathan P. Stewart, Emel Seyhan, Walter J. Silva, Brian S.J. Chiou, Katie E. Wooddell, Robert W. Graves, Albert R. Kottke, David M. Boore, Tadahiro Kishida, and Jennifer L. Donahue.

Petersen, M., Moschetti, M., et al. (2014). Documentation for the 2014 Update of the United States National Seismic Hazard Maps. USGS Open File Report 2014-1091.

Petersen, M., Shumway, A., Powers, P., et al., 2019, The 2018 Update of the US National Seismic Hazard Model: Overview of Model and Implications.

Phillips, C. and Hashash, Y. (2009), Damping formulation for non-linear 1D site response analyses, Soil Dynamics and Earthquake Engineering 29(7): Pages 1143-1158.

Powers, PM (2017), National Seismic Hazard Model Project computer code (NSHMP-HAZ), software. Available at https://doi.org/10.5066/F7ZW1K31.

Risk Engineering, Inc. (2018), EZ-FRISK™ Online User's Manual, EZ-FRISK™ Version 8. Risk Engineering, Inc., Boulder, Colorado.

Schwartz, D.P. and Coppersmith, K.J. (1984), "Fault behavior and characteristic earthquakes: examples from Wasatch and San Andreas fault zones," Journal of Geophysical Research, Vol. 89, pp. 5681-5698.

Senior Seismic Hazard Analysis Committee (SSHAC), 1997, Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts. U.S. Nuclear Regulatory Commission, Report No. CR-6372.

Shahi, S. and Baker, J. (2014a). NGA-West2 Models for Ground Motion Directionality. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1285-1300.

Shahi, S.K. and Baker, J.W., 2014b, An Efficient Algorithm to Identify Strong-Velocity Pulses in Multicomponent Ground Motions, Bulletin of the Seismological Society of America, Vol. 104, No. 5, pp. 2456–2466, October 2014.

Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M. (2000), Epicenters of and Areas Damaged by M≥5 California Earthquakes, 1800-1999, Map Sheet 49, California Geological Survey, California Department of Conservation.

Travasarou, T., Bray, J.D., and Abrahamson, N.A., 2003, Empirical Attenuation Relationship for Arias Intensity, Earthquake Engineering and Structural Dynamics, Vol. 32, pp. 1133-1155.

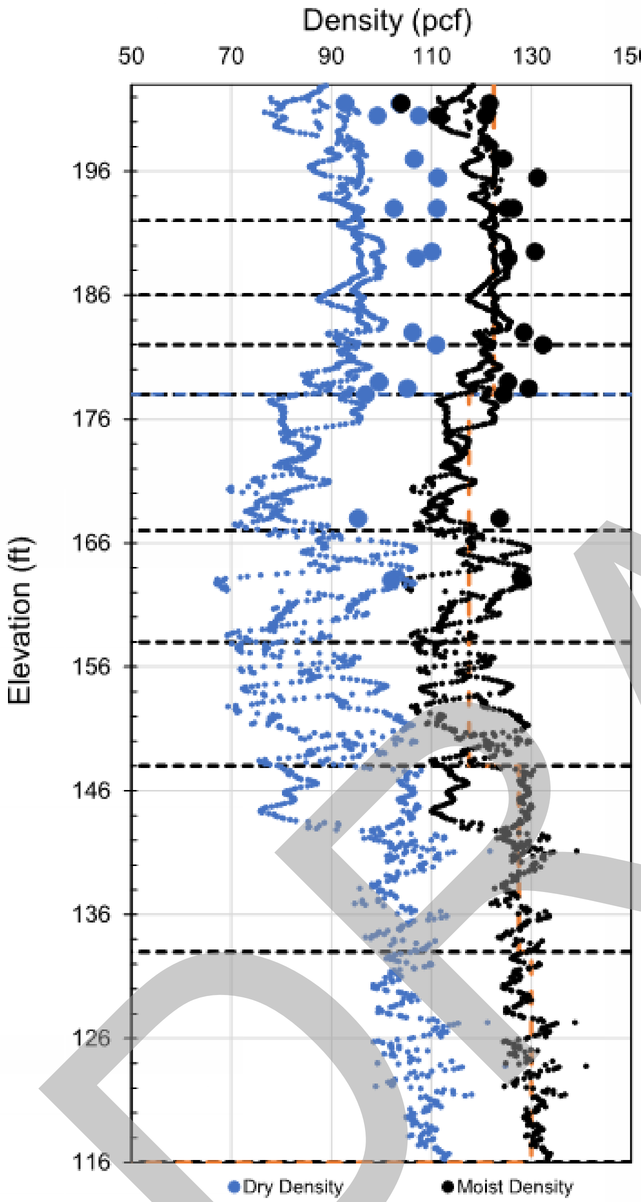
Unruh, J., and Humphrey, J., 2017, Seismogenic Deformation Between the Sierran Microplate and Oregon Coast Block, California, USA. The Geological Society of America v. 45, no. 5, pp. 415-418.

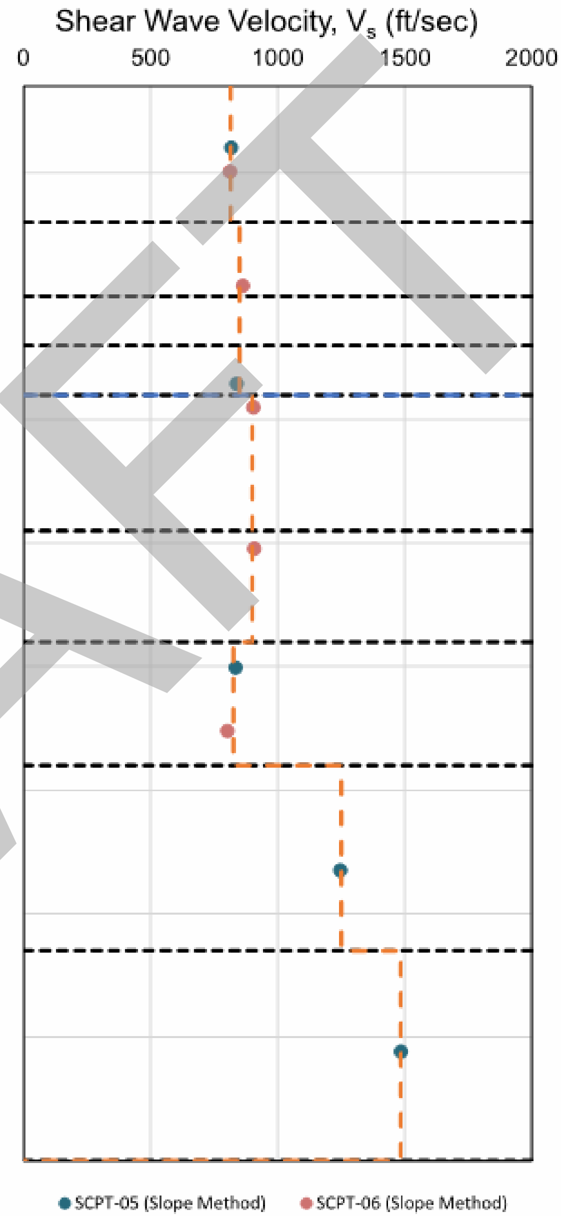
U.S. Geological Survey, ANSS Comprehensive Earthquake Catalog (ComCat).

U.S. Geological Survey, Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/.

U.S. Geological Survey (2010). Quaternary Fault and Fold Database.

U.S. Geological Survey (2015). UCERF3: A New Earthquake Forecast for California's Complex Fault System, March 2015.





KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

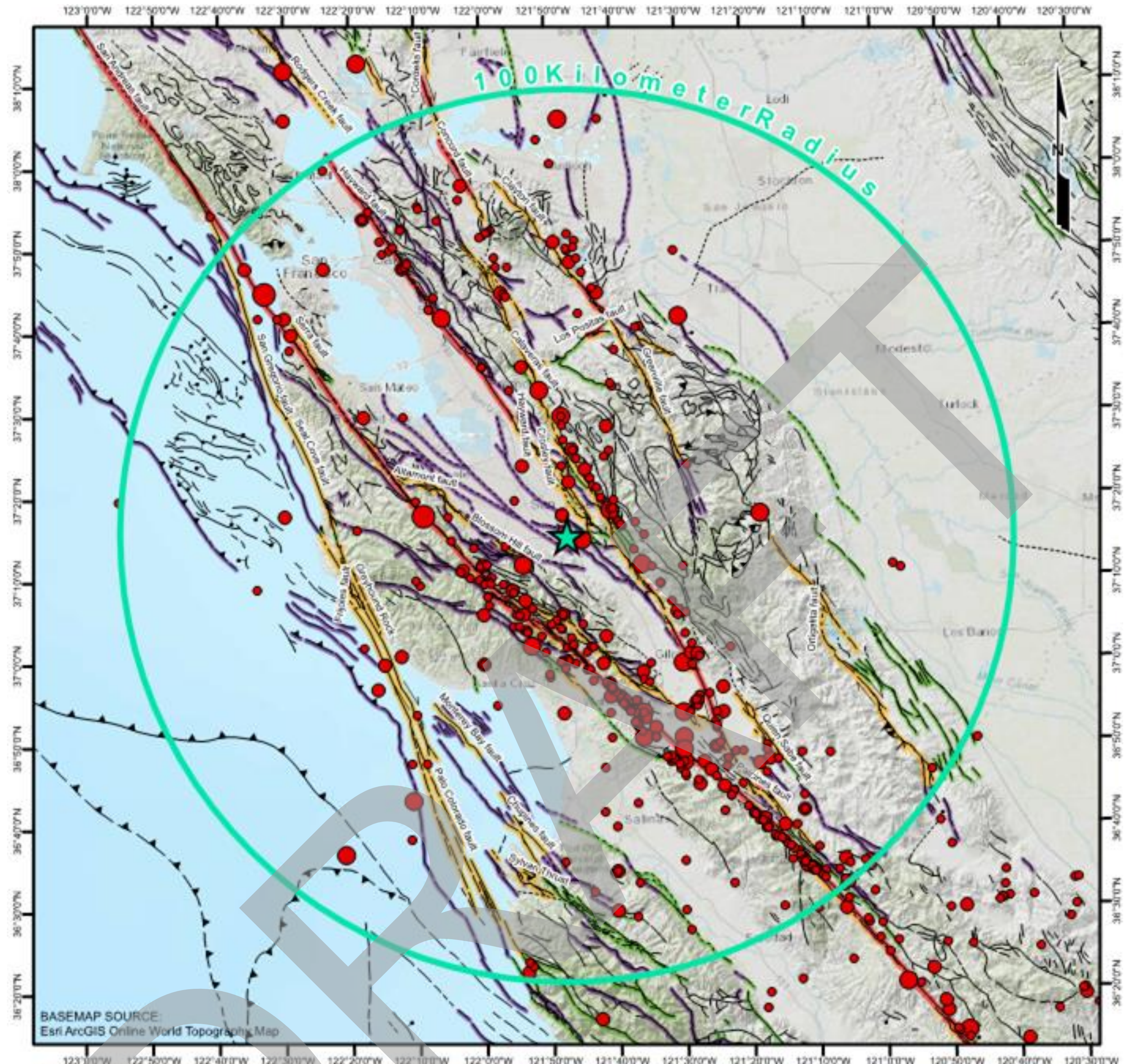
PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

SHEAR WAVE VELOCITY
SCPT-05 AND SCPT-06

PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE

F-1



Quaternary Faults (Jennings and Bryant, 2010)

Historic displacement (< 150 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Holocene displacement (< 15,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Late Quaternary displacement (< 750,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

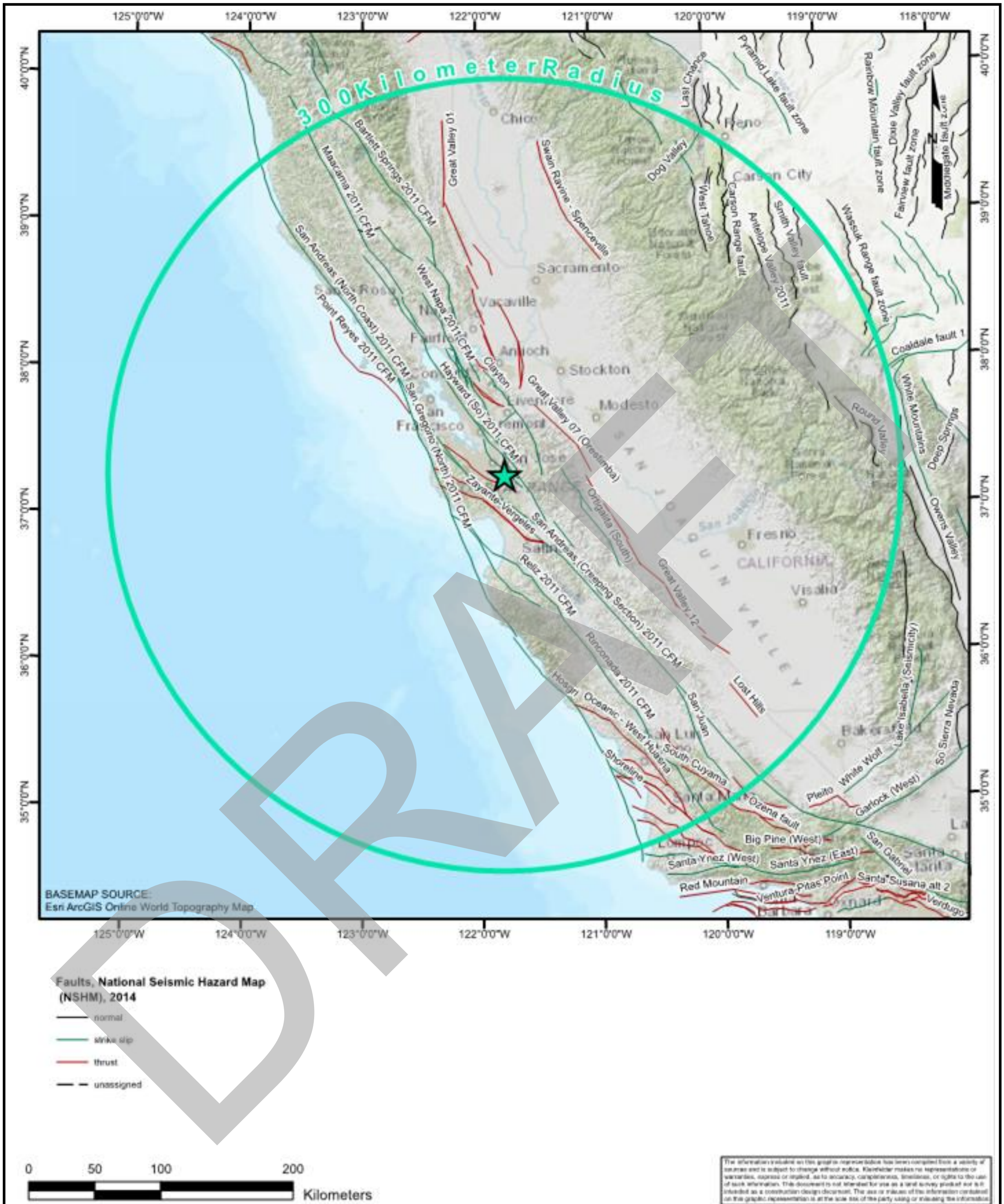
Quaternary & unspecified displacement (< 1,600,000 years)
- undifferentiated Quaternary Well Constrained
- undifferentiated Quaternary Moderately Constrained
- undifferentiated Quaternary Inferred

Pre-Quaternary geologic structures (CGS, 2010)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

ANSS Earthquakes
Magnitude
- 4.0 - 4.9
- 4.9 - 5.9
- 5.9 - 6.9
- 6.9 - 7.9
- 7.9 - 8.9

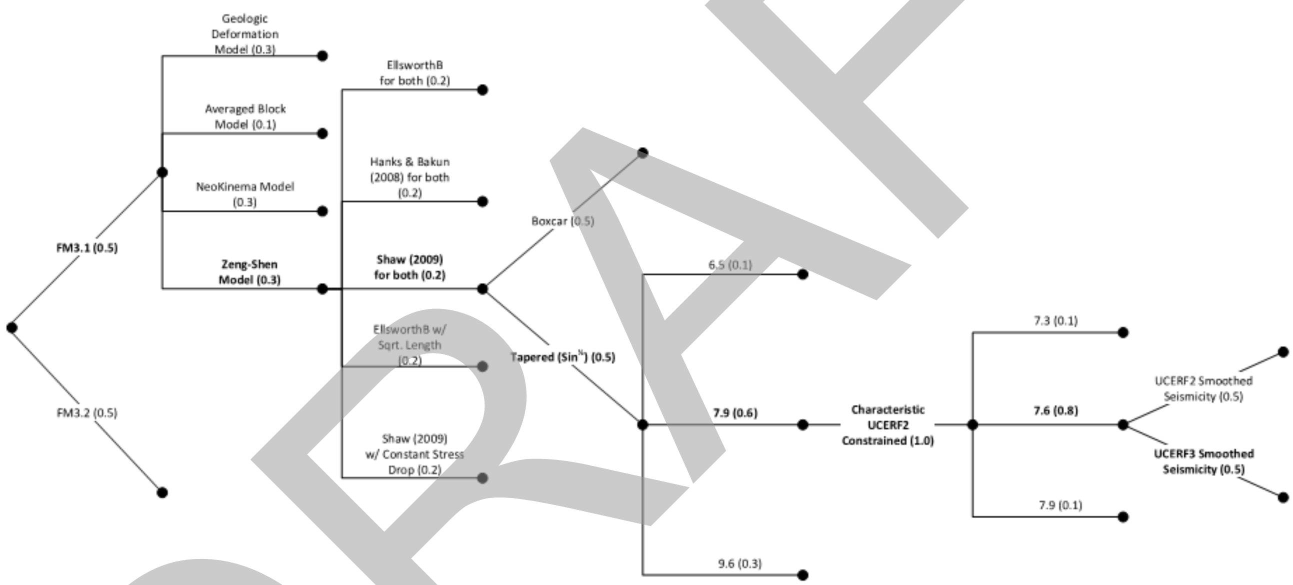
0 17 34 68
Kilometers

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO.	20221404	REGIONAL FAULT AND HISTORICAL SEISMICITY MAP (1800 – DEC 2021)	FIGURE
	DATE:	3/4/2022		
	DRAWN:	Z. Zafir		F-2
	CHECKED BY:	T. Covill	PROPOSED INDUSTRIAL WAREHOUSE 5977 & 6001 SILVER CREEK VALLEY ROAD SAN JOSE, CALIFORNIA	
	File Name:	fname.ppt		



KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 20221404 DATE: 3/4/2022 DRAWN: Z. Zafir CHECKED BY: T. Covill File Name: fname.ppt	**UCERF3 SEISMIC SOURCE MODEL)** PROPOSED INDUSTRIAL WAREHOUSE 5977 & 6001 SILVER CREEK VALLEY ROAD SAN JOSE, CALIFORNIA	FIGURE **F-3**

Fault Models	Deformation Models	Scaling Relationships (mag-area & slip-length relationships)	Slip Along Rupture (Dsr)	Total M ≥5 Event Rate (yr^{-1})	Inversion Model	$M^{off\text{-}fault}_{max}$	Off-Fault Spatial Seismicity PDF (aka SpatialPDF)



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

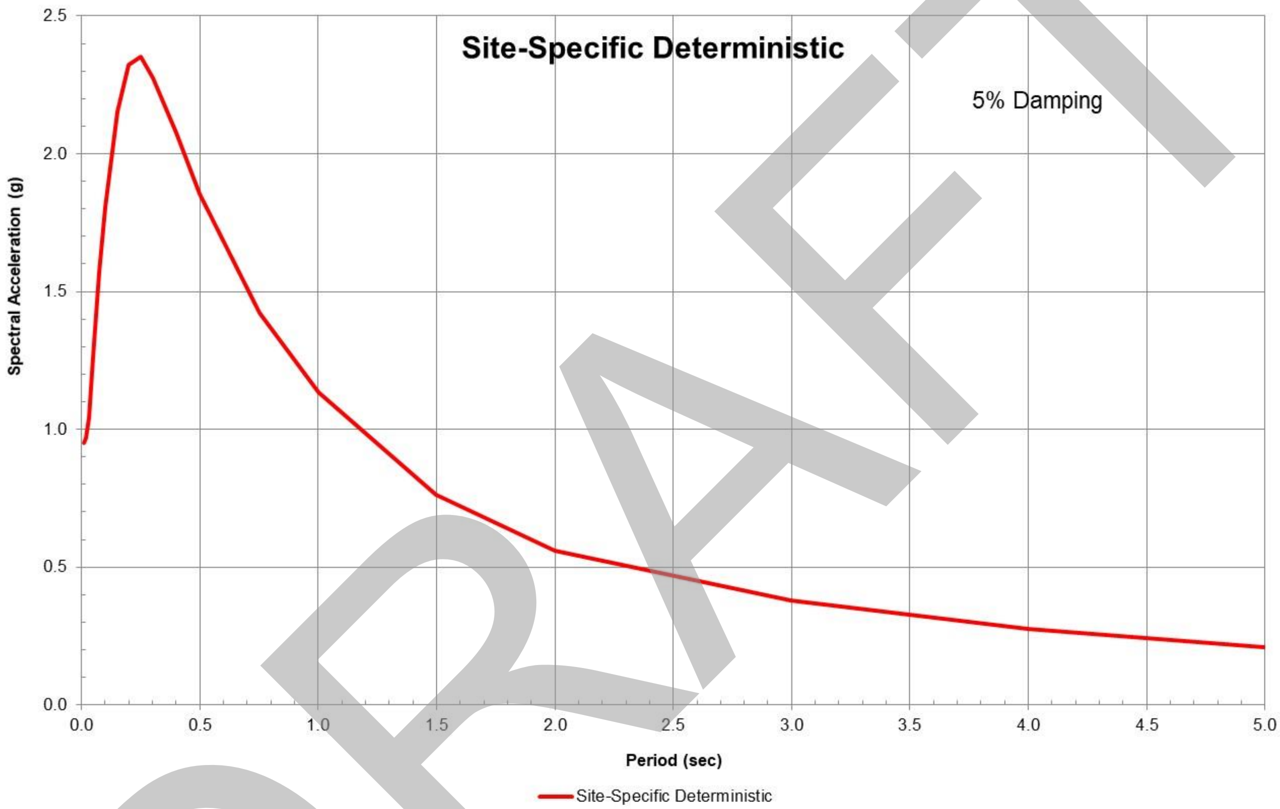
PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

UCERF3 SOURCE MODEL LOGIC TREEE

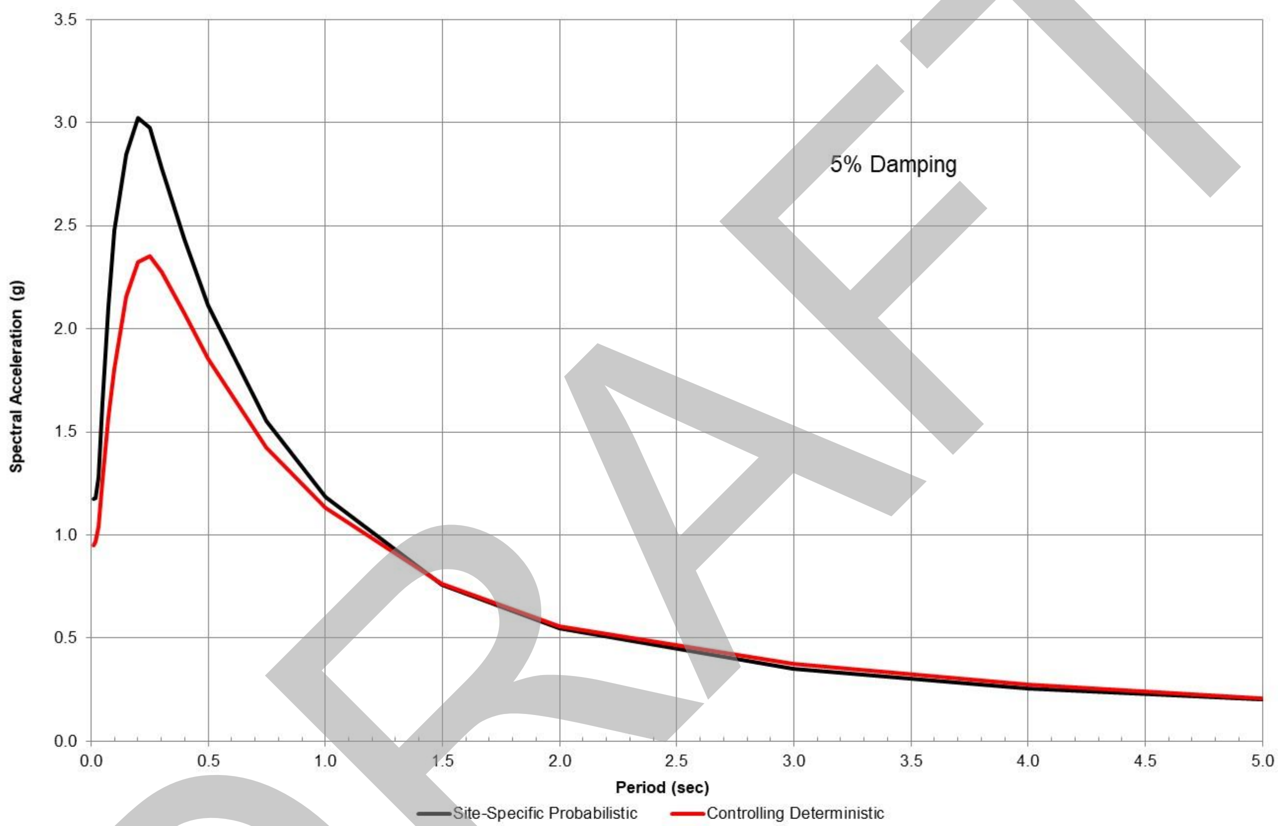
PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE

F-4



KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 20221404	**SITE-SPECIFIC 84TH PERCENTILE DETERMINISTIC SPECTRUM AT BASE OF SOIL COLUMN**	FIGURE
	DATE: 3/4/2022		**F-5**
	DRAWN: Z. Zafir		
	CHECKED BY: T. Covill	PROPOSED INDUSTRIAL WAREHOUSE 5977 & 6001 SILVER CREEK VALLEY ROAD SAN JOSE, CALIFORNIA	
	File Name: fname.ppt		



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

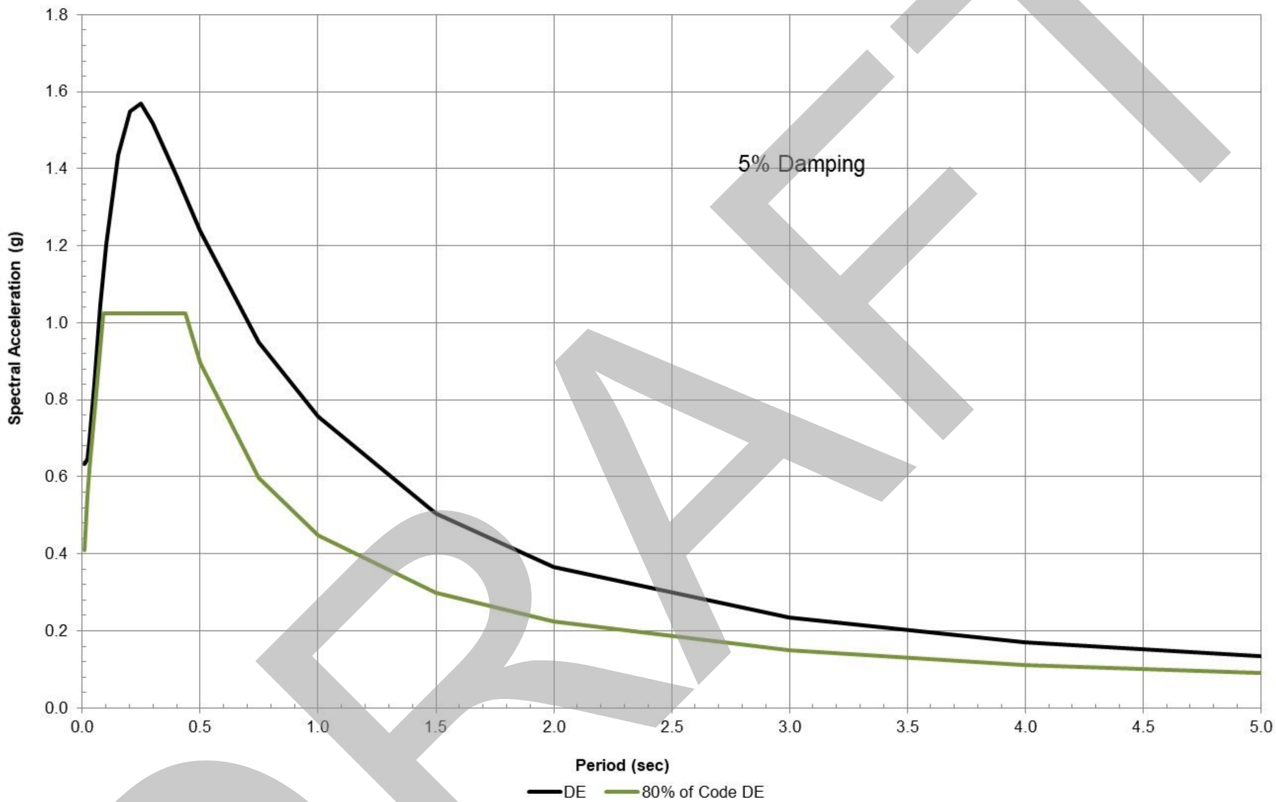
PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

COMPARISON OF SITE-SPECIFIC PROBABILISTIC AND DETERMINSITIC SPECTRA – AT BASE OF SOIL COLUMN

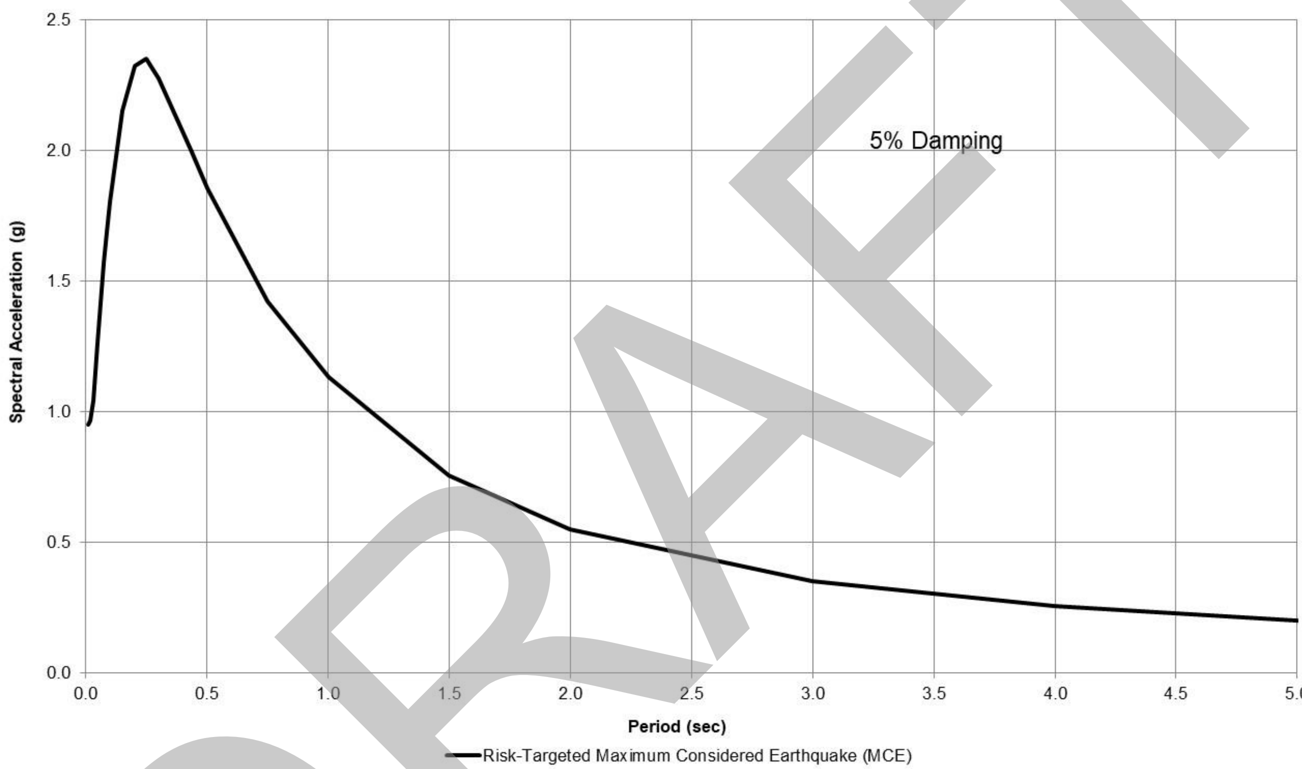
PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE

F-6



KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 20221404 DATE: 3/4/2022 DRAWN: Z. Zafir CHECKED BY: T. Covill File Name: fname.ppt	**COMPARISON OF SITE-SPECIFIC DE AND 80% OF CODE DE SPECTRA AT BASE OF SOIL SOLUMN** PROPOSED INDUSTRIAL WAREHOUSE 5977 & 6001 SILVER CREEK VALLEY ROAD SAN JOSE, CALIFORNIA	FIGURE **F-7**



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

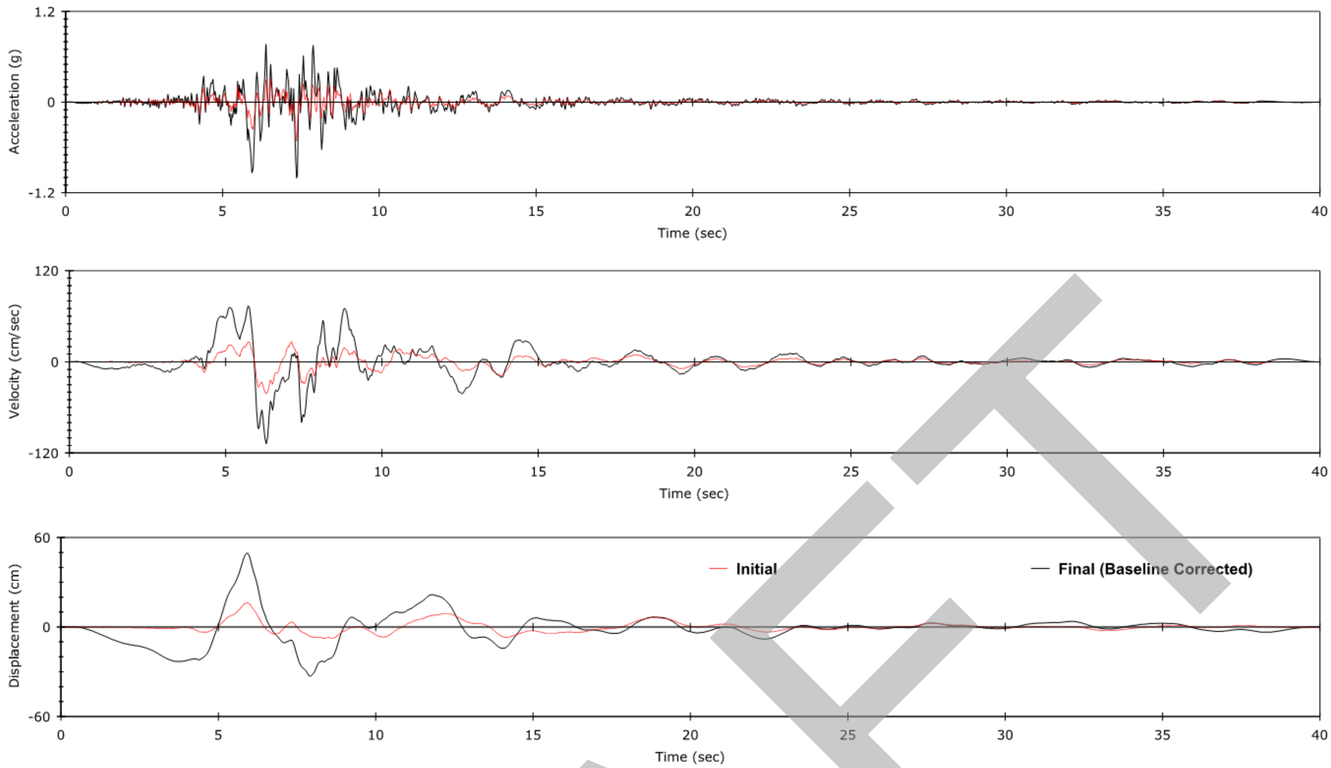
PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

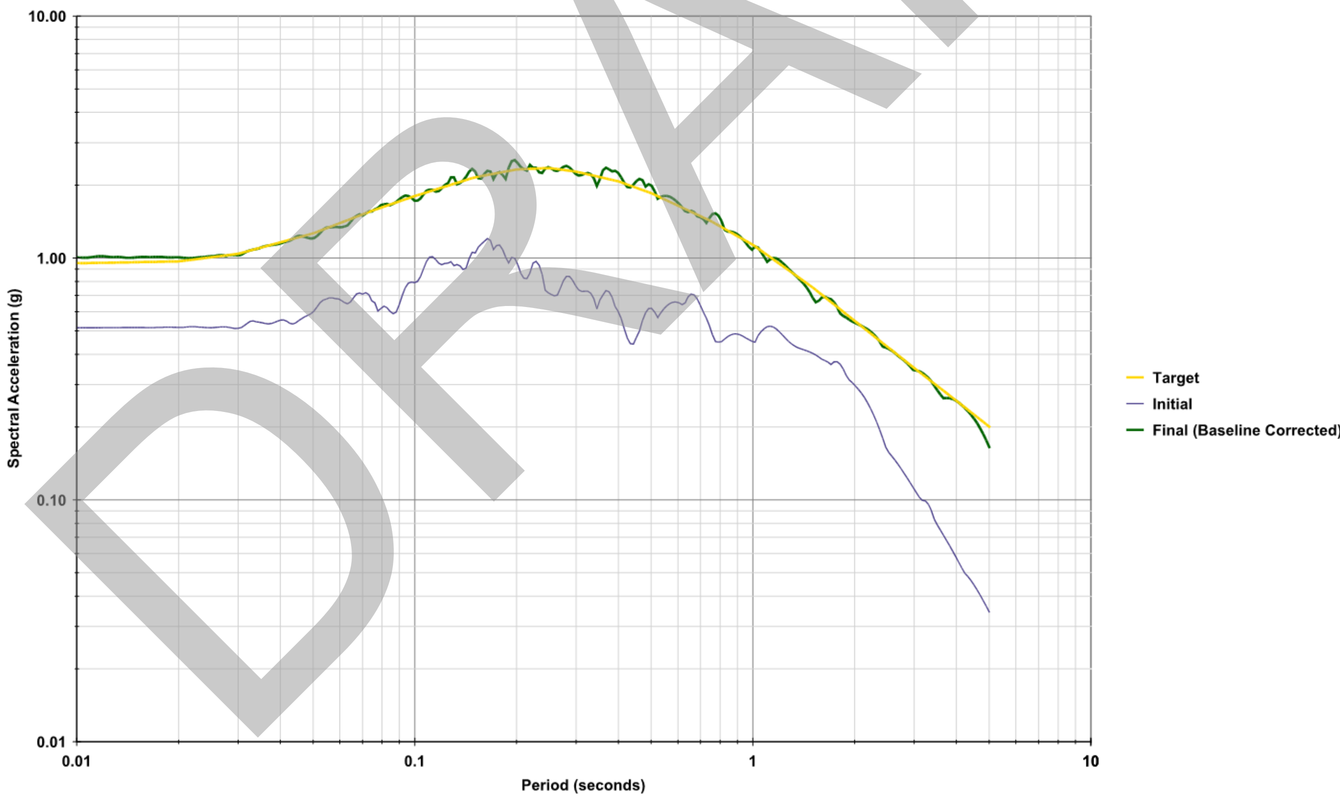
SITE-SPECIFIC TARGET HORIZONTAL MCE_R RESPONSE SPECTRUM AT BASE OF SOIL SOLUMN

PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE

F-8





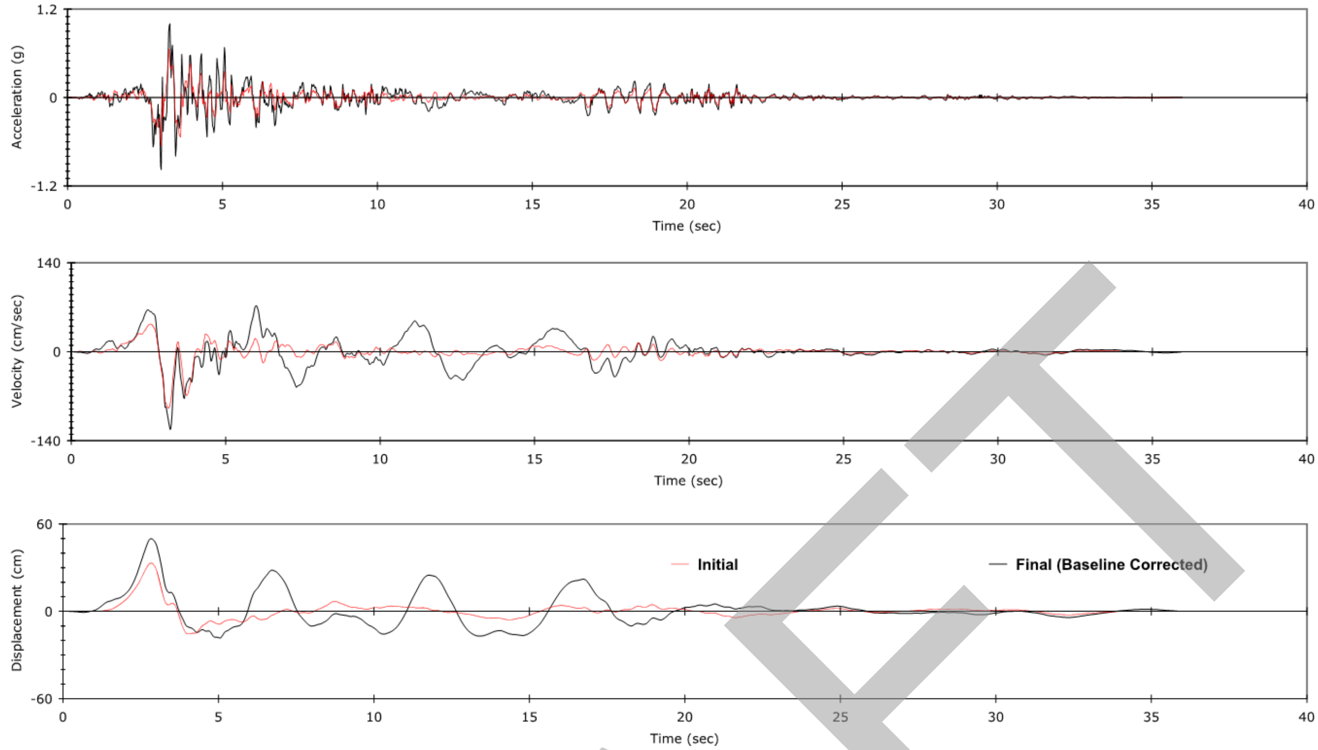
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

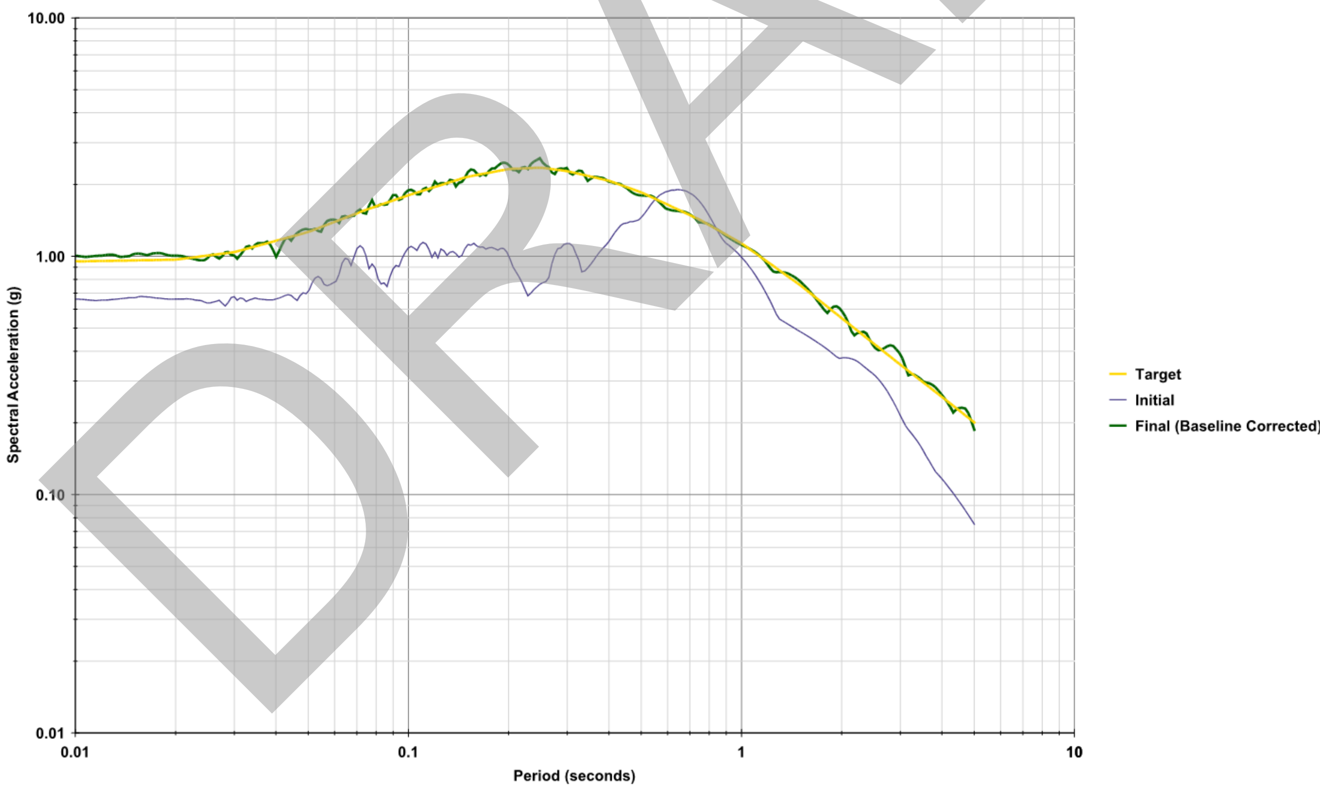
PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

SPECTRAL MATCHING RESULTS
1989 LOMA PRIETA EARTHQUAKE
RSN 802 (0º) SARATOGA – ALOHA AVE

PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE
F-9





KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

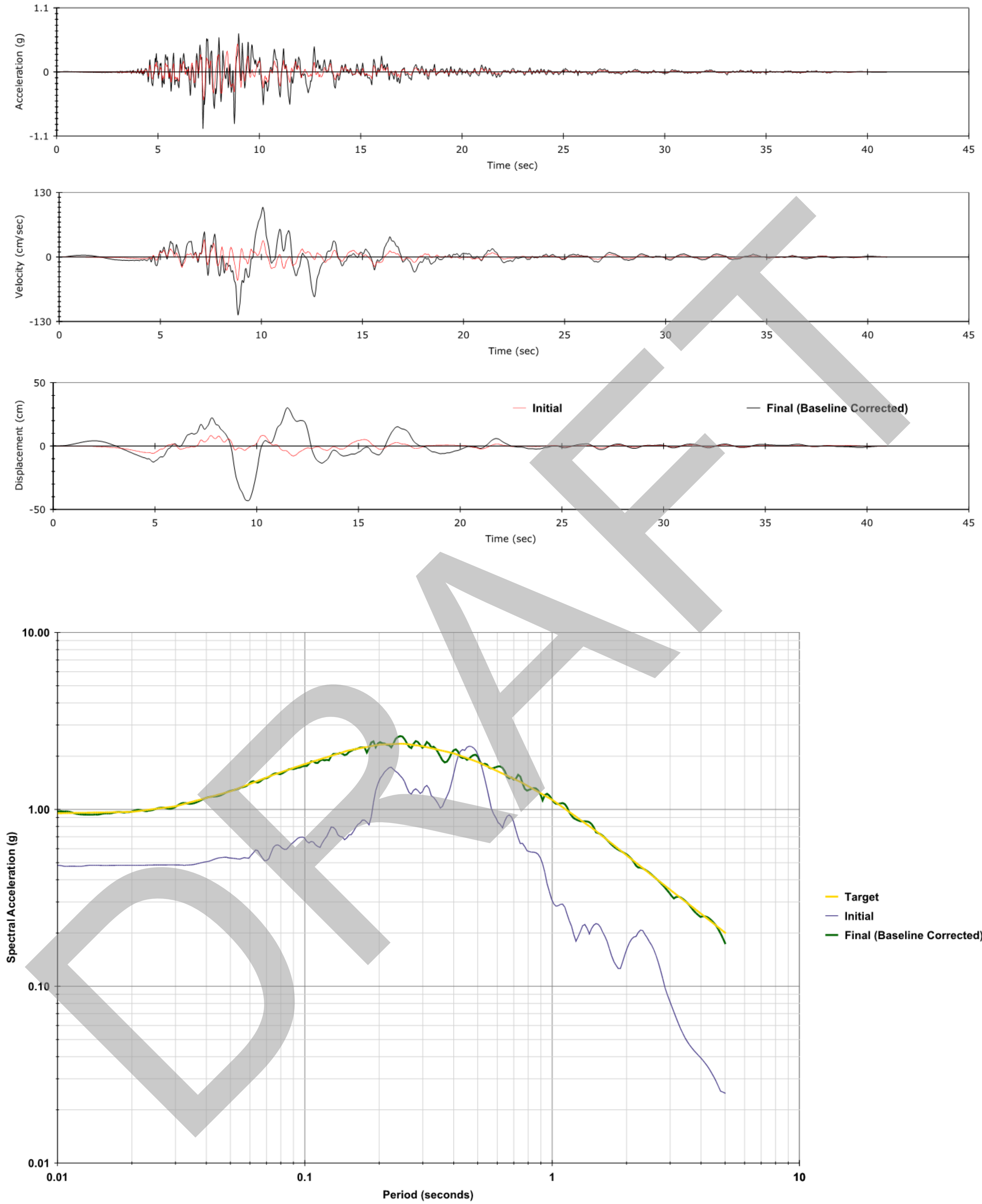
PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

SPECTRAL MATCHING RESULTS
1992 CAPE MENDOCINO EARTHQUAKE
RSN 828 (90°) PETROLIA

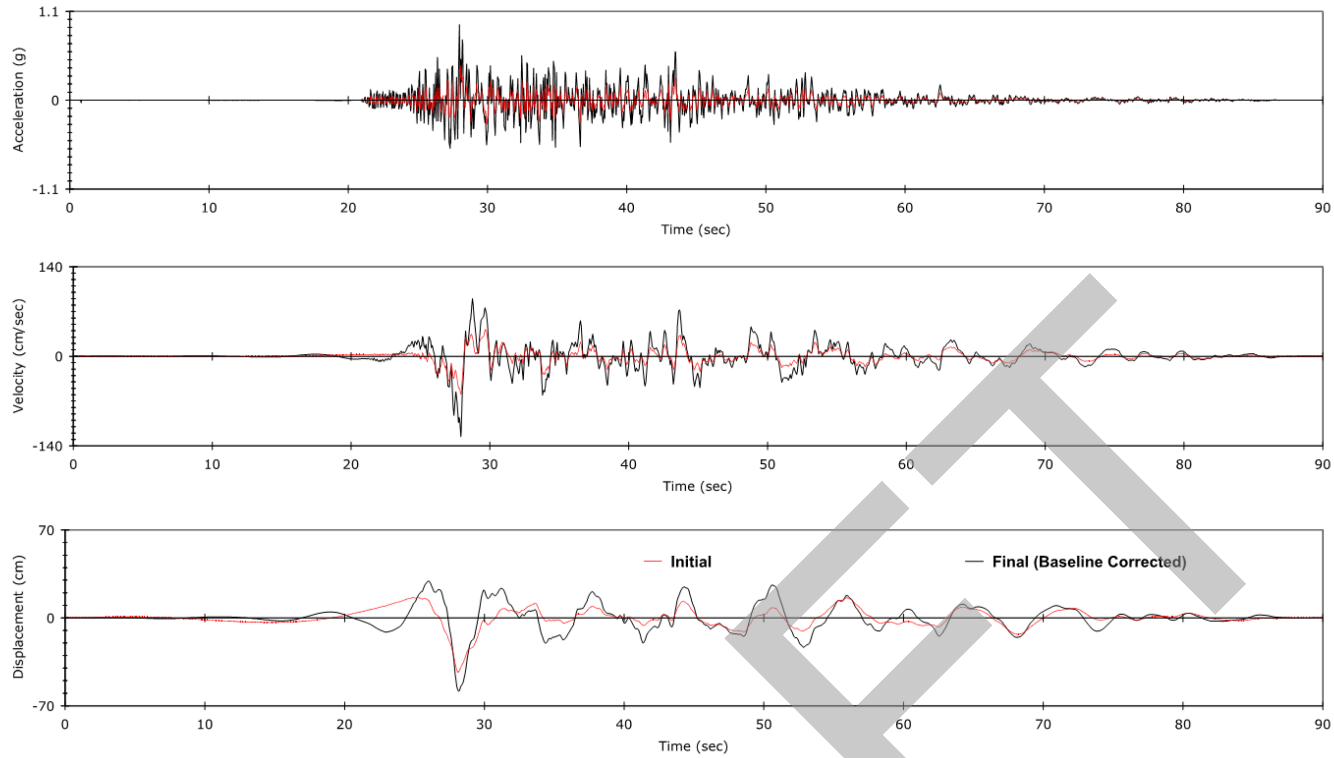
PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

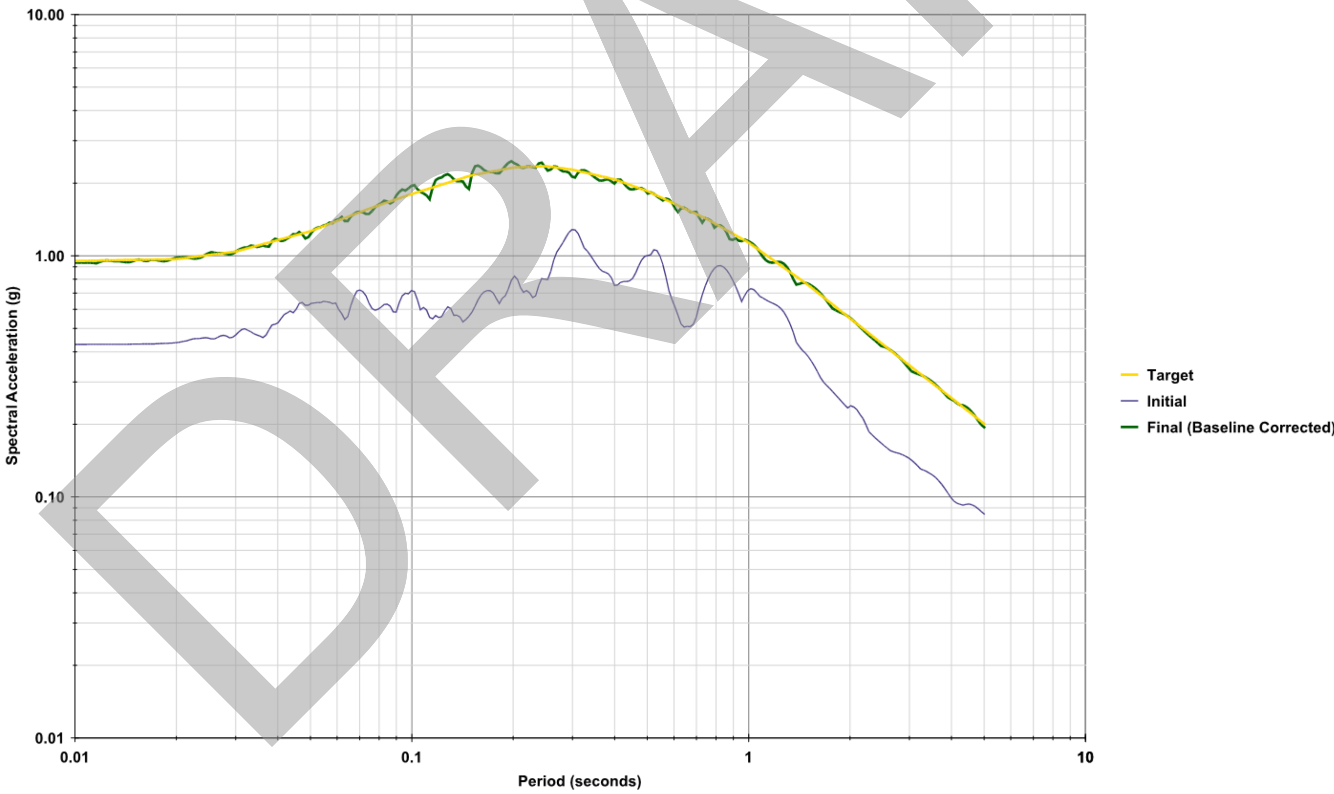
FIGURE

F-10



KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 20221404	**SPECTRAL MATCHING RESULTS 1995 KOBE - JAPAN EARTHQUAKE RSN 1111 (0º) NISHI-AKASHI**	FIGURE
	DATE: 3/4/2022		**F-11**
	DRAWN: Z. Zafir		
	CHECKED BY: T. Covill	PROPOSED INDUSTRIAL WAREHOUSE 5977 & 6001 SILVER CREEK VALLEY ROAD SAN JOSE, CALIFORNIA	
	File Name: fname.ppt		





KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

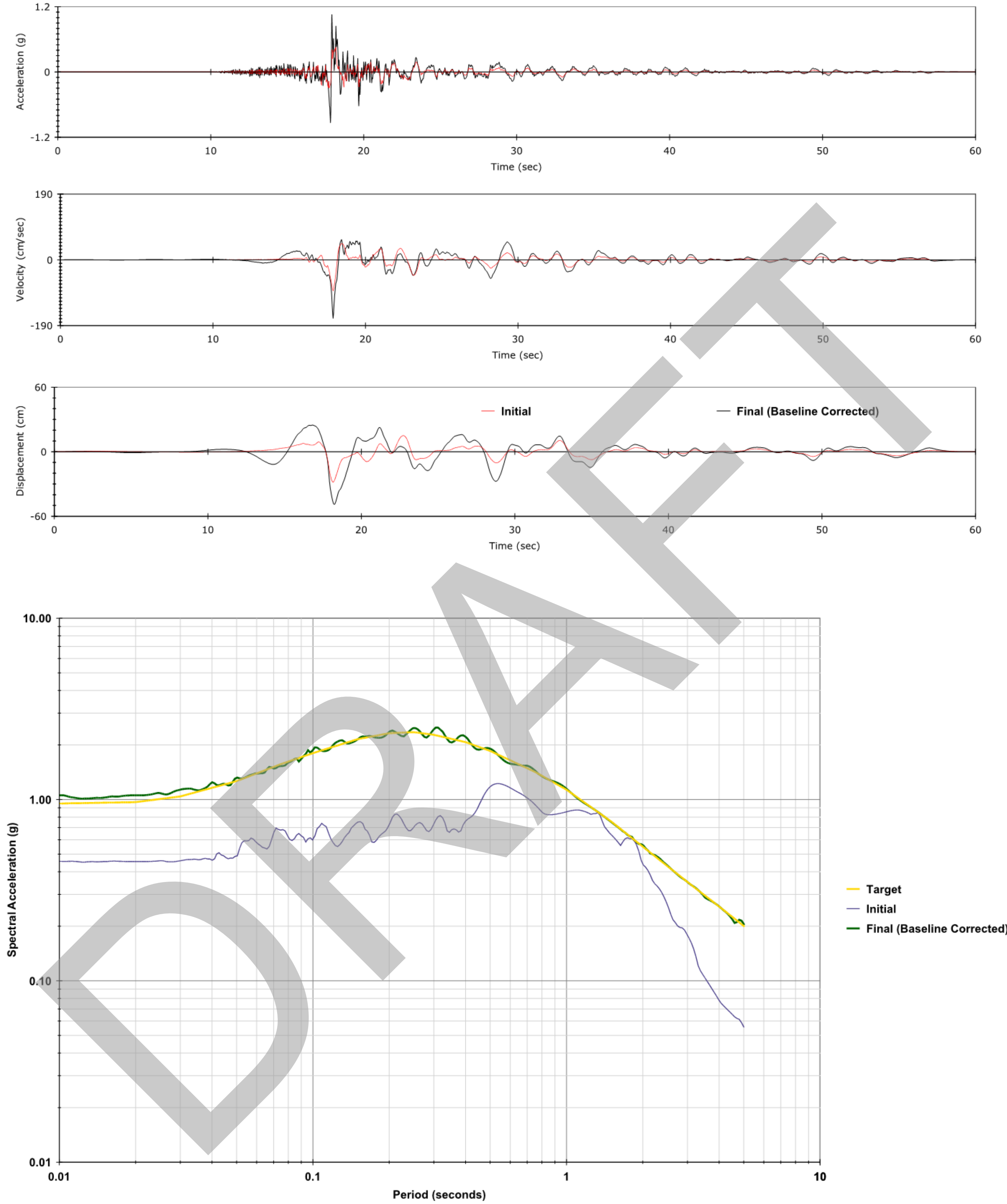
PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

SPECTRAL MATCHING RESULTS
1999 CHI-CHI - TAIWAN EARTHQUAKE
RSN 1511 (N) TCU076

PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE

F-12



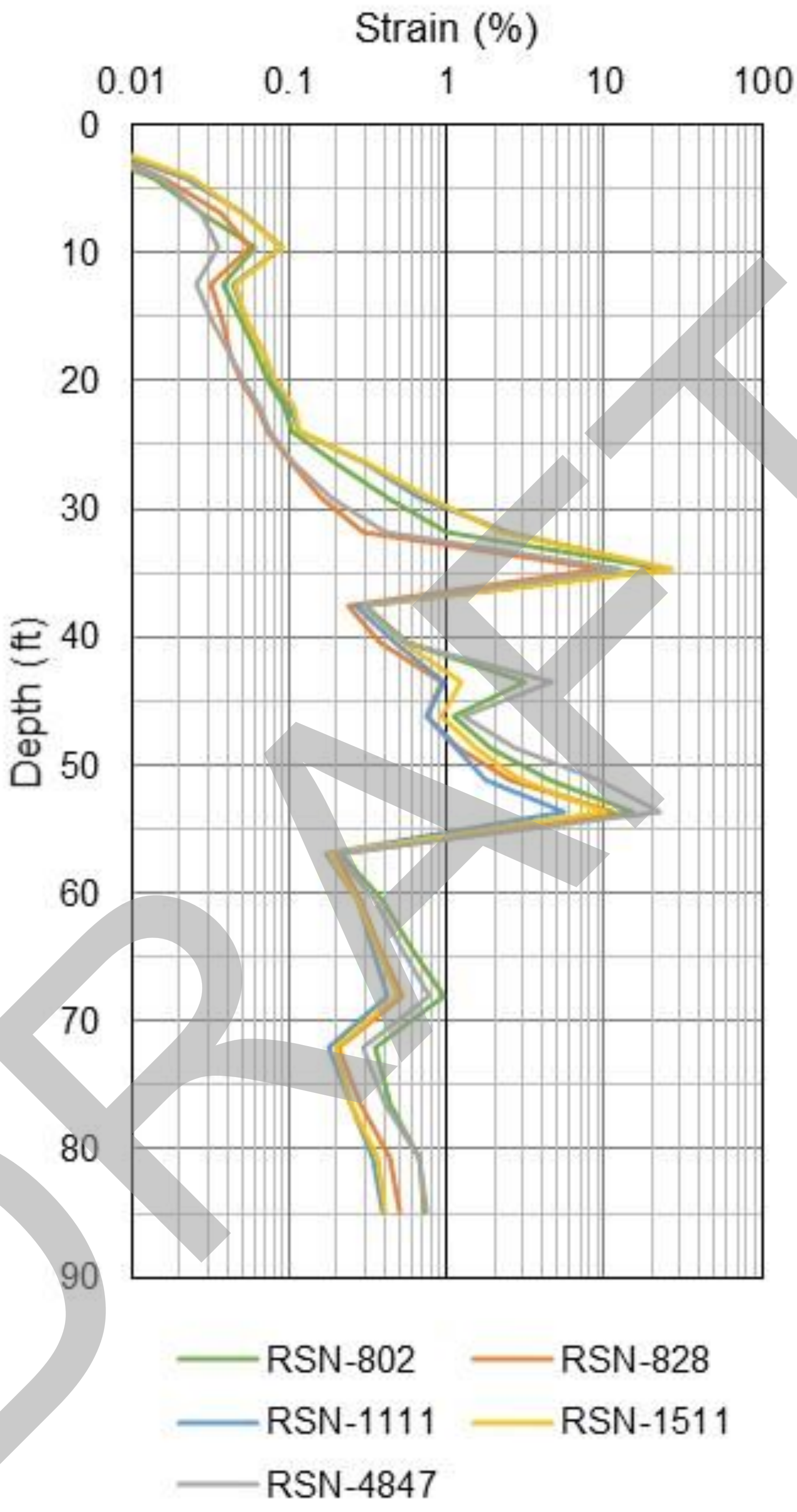
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

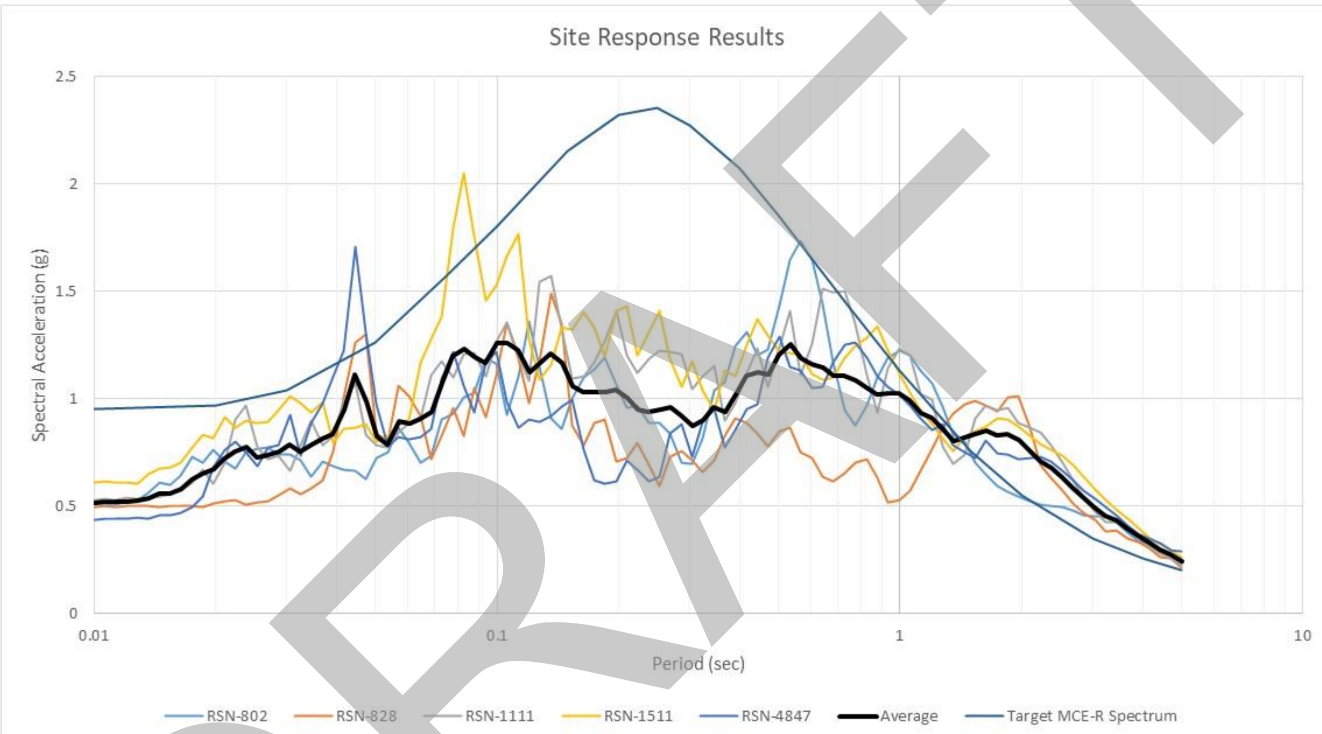
SPECTRAL MATCHING RESULTS
2007 CHUETSU-OKI - JAPAN EARTHQUAKE
RSN 4847 (EW) JOETSU - KAKIZAKIKU

PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE
F-13



KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO.	20221404	**SHEAR STRAIN PROFILE FROM SITE RESPONSE ANALYSIS**	FIGURE
	DATE:	3/4/2022		
	DRAWN:	Z. Zafir		**F-14**
	CHECKED BY:	T. Covill	PROPOSED INDUSTRIAL WAREHOUSE 5977 & 6001 SILVER CREEK VALLEY ROAD SAN JOSE, CALIFORNIA	
	File Name:	fname.ppt		



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

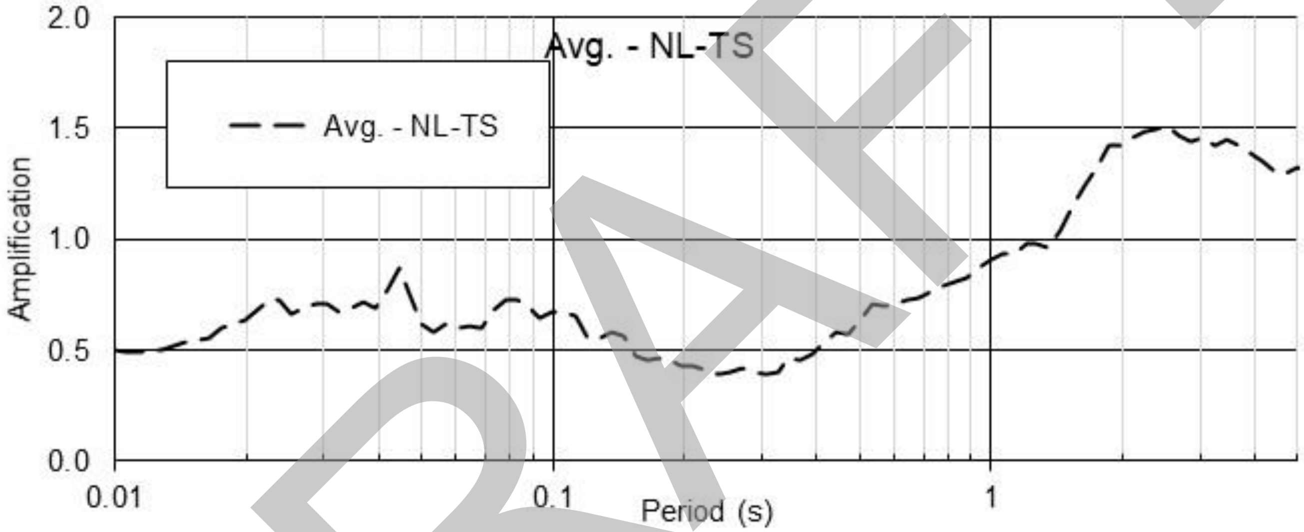
PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

SITE RESPONSE ANALYSIS RESULTS
SURFACE RESPONSE SPECTRA

PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE

F-15



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

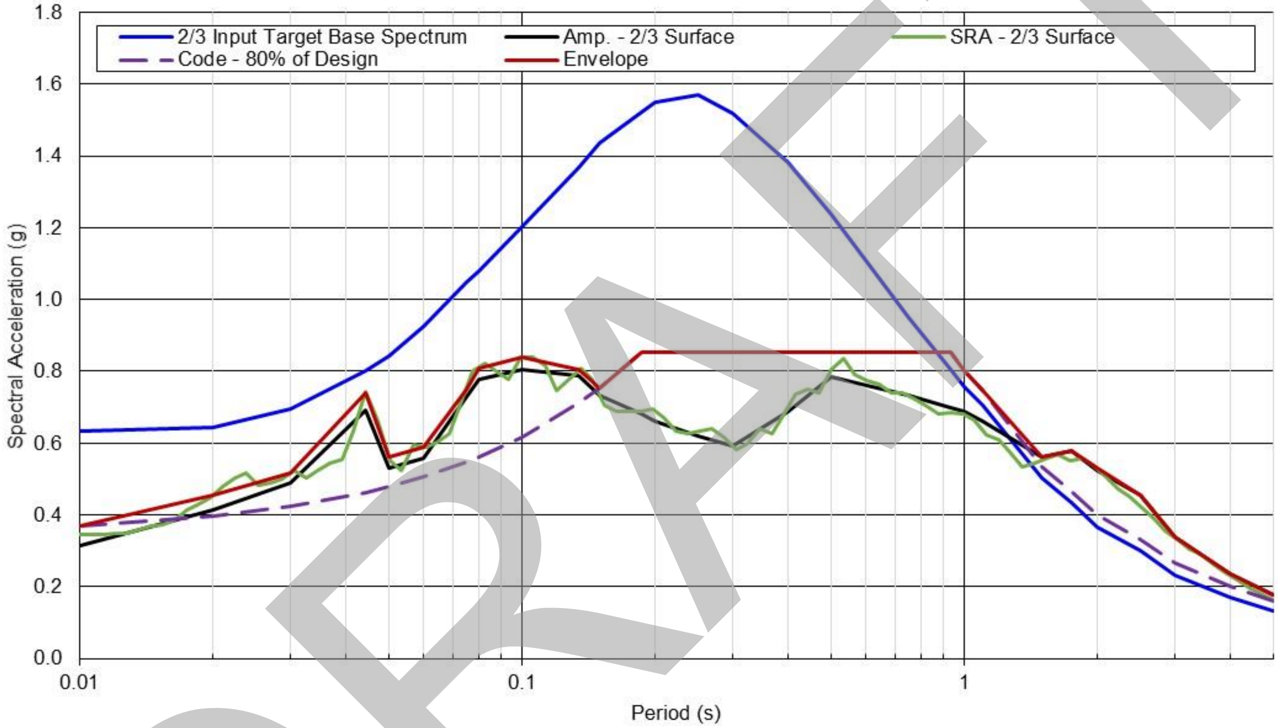
PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

AMPLIFICATION FACTORS FROM SITE RESPONSE RESULTS

PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE

F-16



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

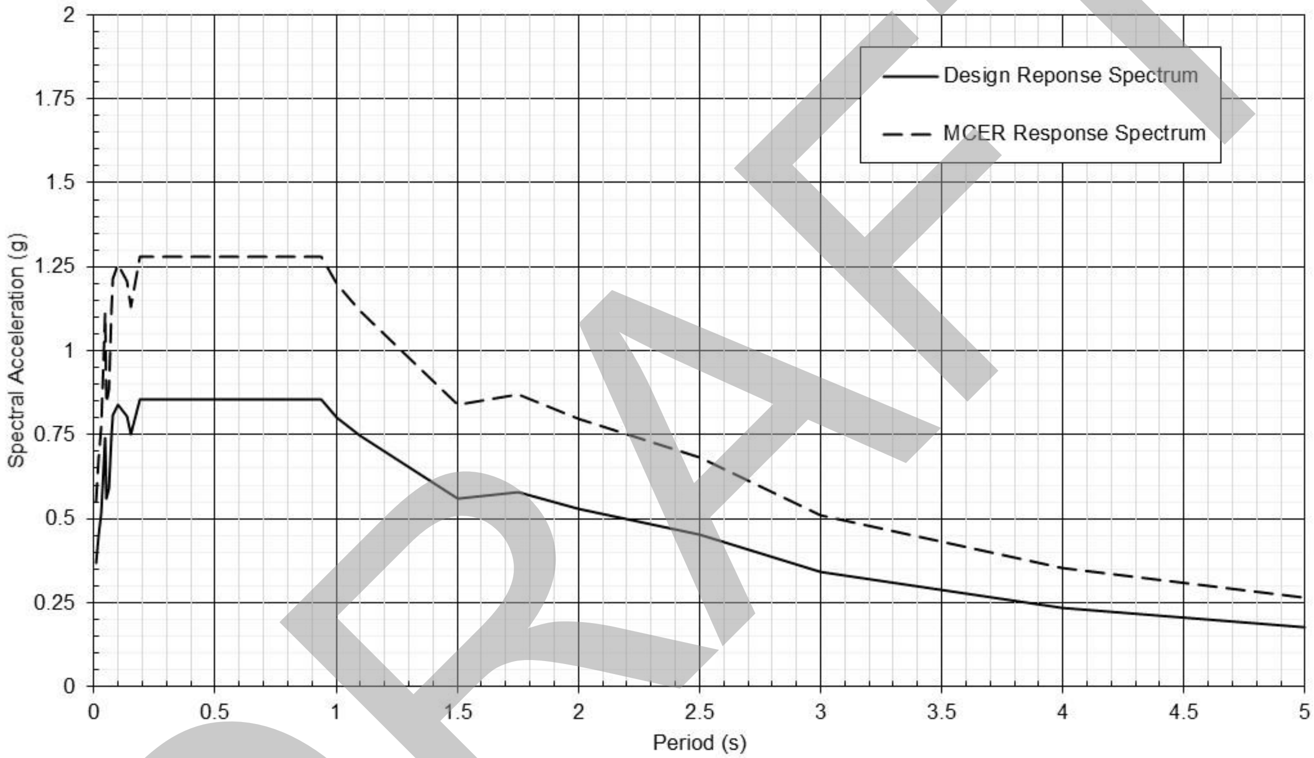
PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

COMPARISON OF 2/3 AVG SURFACE RESULTS WITH 80% OF CODE DESIGN SPECTRUM

PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE

F-17



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO.	20221404
DATE:	3/4/2022
DRAWN:	Z. Zafir
CHECKED BY:	T. Covill
File Name:	fname.ppt

SITE-SPECIFIC HORIZONTAL MCER AND DESIGN RESPONSE SPECTRUM 5% DAMPING

PROPOSED INDUSTRIAL WAREHOUSE
5977 & 6001 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA

FIGURE

F-18

KLEINFELDER
Bright People. Right Solutions.

Appendix G

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, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them 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 herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences 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.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:
- for a different client;
- for a different project or purpose;
- 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, the reliability of a geotechnical-engineering report can be 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 you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site 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.*

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

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement 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 through project completion to obtain 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 judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed 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 continuing 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 available 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-phase observations.

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 information 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. 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. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. 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 provide 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 obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

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, the engineer's services were not designed, conducted, or intended to prevent 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.***

GBA GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org