APPENDIX D:
GEOTECHNICAL REPORT

March 25, 2021
File No. 20213246.001A

APPLE, INC.
Real Estate & Development
One Apple Park Way, MS 952-31CP
Cupertino, CA 95014

Attention: Benjamin Louie
bloui@apple.com

SUBJECT: Geotechnical Investigation Report
Apple VP01
19191 Vallco Parkway
Cupertino, CA 95014

Dear Mr. Louie:

Kleinfelder is pleased to present this report of our geotechnical investigation results for the existing and proposed new office developments located at 19191 Vallco Parkway in Cupertino, California. Our services were authorized by Purchase Order No. 6000486932 dated November 30, 2020 and our work was conducted in general accordance with the geotechnical service scope presented in our proposal dated November 9, 2020.

This report summarizes the project geotechnical conditions, engineering analysis results, and geotechnical engineering recommendations for the design and construction of the new office building, parking garage and associated improvements. The primary geotechnical considerations for the project design and construction include demolition of the existing building and its foundation system, presence of the historical creek alignment within the site, expansion/shrinkage potential of the onsite soils and excavation and retaining for the new parking garage basement. From a geotechnical engineering viewpoint, the proposed project construction is feasible provided the recommendations presented in this report are incorporated into the project design and construction.

We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,

KLEINFELDER, INC.

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GEOTECHNICAL INVESTIGATION REPORT
APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CA 95014
KLEINFELDER PROJECT NO. 20213246.001A

MARCH 25, 2021

A Report Prepared for:

APPLE, INC.
Real Estate & Development
One Apple Parkway, MS 952-31CP
Cupertino, CA 95014

GEOTECHNICAL INVESTIGATION REPORT
APPLE VP01
19191 VALLCO PARKWAY
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March 25, 2021
Kleinfelder Project No. 20213246.001A

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EXECUTIVE SUMMARY

This report presents the results of Kleinfelder's geotechnical study for the proposed Apple VP01 office development project located at 19191 Vallco Parkway in Cupertino, California, as shown on Figure 1. The purpose of our geotechnical study was to evaluate surface and subsurface conditions at the site and provide geotechnical recommendations for project design and construction. Our study was based on the proposed new structures and improvements indicated on the project preliminary architecture plans prepared by Studios Architecture and dated January 30, 2021.

As shown on Figure 2, Kleinfelder understands the project will consist of demolishing an existing office building and constructing a new office building, a new parking garage and associated improvements on an about 8-acre parcel. The existing building is at-grade, two-story and has a footprint of about 77,250 square feet (sf). The new office building will be at-grade, about 73-foot high, four-story and will have a footprint of about 74,000 sf. The new parking garage will be about 45-foot high and have four above-grade levels and two below-grade levels. The bottom of the garage basement will be at about 21 feet below existing ground surface. The new structures will be surrounded by paved parking and drive aisles and landscape features. We anticipate the finished grade of the new developments will generally match the existing grade. The office building is expected to be founded on shallow foundation and the parking garage will be on mat slab foundation. Estimated column loads are about 415 to 670 kips for the office building and about 542 to 1,052 kips for the parking garage.

Subsurface conditions at the site were explored by drilling four exploratory borings and advancing four cone penetration tests (CPTs). The borings were drilled to depths of about 51½ to 76½ feet below the existing ground surface (bgs) by using a truck-mounted drill rig equipped with hollow stem augers. The CPTs were advanced to depths of about 51 to 70 feet bgs by using a 30-ton truck rig with an electronic piezocone penetrometer.

The site, aside from the area of the existing building footprint, is generally covered by asphalt concrete pavement and landscaping features. Various thicknesses of surficial fills were encountered in our borings, which appeared to be compacted and derived from the previous grading of the existing site development. According to available historic topographic maps and aerial photographs, the historic natural alignment of the Calabazas Creek was located within the site prior to relocating to its current improved channel location along the northwestern site

boundary in the 1970s. The old creek channel was probably backfilled with compacted fills that may extend to depths of about 15 to 20 feet bgs. The native soils below the site consist of alluvial stiff to hard clays and medium dense to very dense sands and gravels that extend to the maximum depth explored to about 76½ feet bgs. No groundwater was encountered by any of the borings or CPTs to the completed depths.

Based on the results of our data review, field reconnaissance, subsurface exploration, lab testing, and engineering analyses, it is our professional opinion that the proposed project is geotechnically feasible, provided the recommendations presented in this report are incorporated into the project design and construction. We identified the following key geotechnical considerations.

- The proposed office building can be supported on a conventional shallow footing foundation system. An 18-inch thick layer of imported, predominantly granular, "non-expansive" engineered fills should be provided below interior slab-on-grade. Alternatively, the onsite fills and soils can be lime-treated to create the non-expansive fill layer.
- The proposed parking garage can be supported on a structural mat slab system. Construction of the two-level basement will require temporary excavation or shoring.
- Due to the moderate to high expansion potential of onsite fills and soils, we recommend a layer of 18 inches non-expansive engineered fills or lime-treated subgrade be provided below exterior concrete flatwork. The non-expansive fills or lime-treatment should extend at least 3 feet laterally beyond edges of flatwork.
- The existing building foundation underpinning piers can be left in place provided the pier caps and haunches be completely removed. The top of pier shafts should be cut down to a depth of at least 5 feet below bottom of the new building ground floor slab or at least 2 feet below the bottom of new building footing.
- There is no grading or fill compaction record available in regard to the fills placed during the construction of the existing development as well as extents and details of the backfill for the previous Calabazas Creek channel. Based on the results of borings and CPTs at the site, these fills and backfills appear to be compacted. We recommend test pits or potholing be performed during the site demolition and grading to identify any potential weak fills that may exist within the site.

- The demolition of the existing building and improvements at the site will loosen and disturb the about upper 3 to 5 feet of onsite site fills and soils. The disturbed weak fills and soils located within the areas of proposed new structures and improvements should be completely removed and re-compacted to depths where competent fills and soils are located. Deeper over-excavation may be required in local areas where thicker weak fills and soils are encountered and where the underpinning pier shafts are removed to a depth of 5 feet.
- Following the site clearing and preparation, soil subgrades in areas to receive structure foundations and improvements (such as engineered fills, slabs-on-grade, exterior flatwork, and pavements) should be proof-rolled with a fully-loaded tandem-axle dump truck or water truck. Areas identified as being soft or yielding may require additional compaction or over-excavation as determined in the field by the project Geotechnical Engineer.
- We recommend engineered fills be compacted to at least 90 percent relative compaction, as determined by ASTM D1557. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately eight inches in uncompacted thickness (loose measurement).
- We recommend engineered fills be moisture conditioned to at least 3 percent above optimum water content. In order to achieve satisfactory compaction of fill materials, it may be necessary to adjust the water content at the time of earthwork operations. This may require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet. The moisture content of the fill is considered very important, and therefore, both relative compaction and moisture content should be used to evaluate compaction acceptance. If both criteria are not within the specified tolerances, the fill should not be accepted, and the contractor should rework the material until the fill is placed within the specified tolerances.
- Onsite fills and soils and imported fills when used for trench backfill should be compacted to at least 90 percent relative compaction. Imported sands and aggregate bases when used for trench backfill should be compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper three feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction.

- The site surficial soils are fine-grained, moisture sensitive, and susceptible to disturbance, rutting, and pumping during construction. The contractor should plan to repair subgrade conditions that become unstable/disturbed and should develop a plan to manage subgrade trafficability across the site throughout the construction period. Features of this plan may include temporary surface haul roads, limited traffic routes, etc.
- If planned, we recommend bio-retention swales and basins where they are located within 10 feet from structures and improvements (such as building foundations, exterior flatwork and pavements) be completely lined with a relatively impermeable membrane to reduce water seepage and the potential for damage to the adjacent structures and improvements. The bio-treatment soil mix materials within swales and basins should be considered as having no lateral load resistant. Alternatively, properly designed below-grade concrete boxes/planters with a drainage system at the bottom can be used to build the swales and basins and to retain surrounding ground and structures/improvements.
- The results of corrosion testing of an onsite composite soil sample indicate the resistivities of onsite soils are considered as corrosive to ferrous metals. 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 facilities. The measured redox potential indicates anaerobic soil conditions, which is considered as slightly corrosive. We recommend a corrosion specialist be consulted for specific site recommendation regarding corrosion potential and specific mitigation alternatives if need.

The findings, conclusions, and recommendations presented in this executive summary should not be relied upon without consulting our geotechnical report for more information. The conclusions and recommendations presented in this report are subject to the Limitations presented in Section 7.

1 INTRODUCTION

This report presents the results of Kleinfelder's geotechnical study for the proposed Apple VP01 office development project located at 19191 Vallco Parkway in Cupertino, California. The location of the project site is shown on Figure 1, Site Vicinity Map. The purpose of our geotechnical study was to evaluate soil and groundwater conditions at the site and provide geotechnical recommendations for project design and construction. Our study was based on the information for the proposed new structures and improvements as indicated on the project preliminary architecture plans prepared by Studios Architecture and dated January 30, 2021.

This report summarizes the geotechnical services performed, discusses the geotechnical conditions observed at the site, and presents conclusions and recommendations developed from our engineering analyses of field and laboratory data. Individuals using this report should read the Limitations presented in Section 7.

1.1 PROJECT DESCRIPTION

According to the project preliminary architecture plans and as shown on Figure 2, Site Plan, we understand that the project will consist of demolishing an existing office building and constructing a new office building, a new parking garage and associated improvements on an about 8-acre parcel. The existing building is at-grade, two-story and has a footprint of about 77,250 square feet (sf). The new office building will be at-grade, about 73-foot high, four-story and have a footprint of about 74,000 sf. The new parking garage will be about 45-foot high and have four above-grade levels and two below-grade level. The bottom of the garage basement will be at about 21 feet below the existing ground surface. The new structures will be surrounded by paved parking and drive aisles, and landscape features. We anticipate the finished grade of the new developments will generally match the existing grade. The anticipated structure foundation type and estimated column loads were provided by Structural Engineer, Inc. (SEI) on February 11, 2021 via email and are listed in the table below:

Table 1
Anticipated Structure Foundation Types and Column Loads

Proposed Structure	Foundation Type	Element	Dead Load (kips)	Live Load (kips)	Total Loads (kips)
Office Building	Shallow Footing	Interior Columns	425	245	670
		Exterior Columns	280	135	415
Parking Garage	Mat Slab	Interior Columns	740	312	1,052
		Exterior Columns	386	156	542

1.2 SCOPE OF SERVICES

The scope of our geotechnical study consisted of literature review, field reconnaissance, subsurface exploration, laboratory testing, engineering evaluation and analysis, and preparation of this report. Environmental hazards assessments of the soil and groundwater at the site were beyond our geotechnical scope of services. The following sections summarize our scope of services.

1.2.1 Task 1 – Background Data Review

We reviewed readily available published and unpublished geologic literature in our files and from database maintained by public agencies, including selected publications prepared by the California Geological Survey (CGS) and the U.S. Geological Survey (USGS). Readily available online historic topographic maps and aerial photographs were also reviewed for past site development and use. We also reviewed available seismic and fault information, including maps of up-to-date earthquake fault zones and seismic hazards zones designated by the California State and local agencies.

In addition, record drawings of the existing development (Cabak Randall Jasper Griffiths Associates, 1981) as well as information about the previous building settlement investigation (BAGG Engineers, 2014) and structure underpinning measures (SEI Engineers, 2014 and 2020) were also reviewed.

1.2.2 Task 2 – Field Exploration

Subsurface conditions at the site were explored by drilling four borings and advancing four cone penetration tests (CPTs) on January 26 to January 28, 2021. The borings were performed by

Exploration Geoservices, Inc. of San Jose, California and the CPTs were performed by ConeTec, Inc. of San Leandro, California.

The borings were drilled to depths of about 51½ to 76½ feet below the existing ground surface (bgs) by using a truck-mounted drill rig equipped with hollow stem augers. The CPTs were advanced to depths of about 51 to 70 feet bgs (where penetration refusal was encountered) by using a 30-ton truck rig with an electronic piezocone penetrometer. In-situ shear wave velocity measurements were also performed in one of the CPT. The approximate locations of the borings and CPTs are shown on Figure 2.

Prior to commencement of the fieldwork, Underground Service Alert (USA) was notified, and a private utility locator was hired to identify potential conflicts with subsurface structures and utilities at the boring and CPT locations. A Kleinfelder field engineer maintained logs of the borings, visually classified the soils encountered per the Unified Soil Classification System, and obtained samples of the subsurface materials. Soil classifications made in the field from samples and auger cuttings were made in accordance with ASTM D2488. These classifications were re-evaluated in the laboratory after further examination and testing in general accordance with ASTM D2487. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the boring logs. The blow counts listed on the boring logs are raw values and have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency. Hammer efficiency and calibration date shown on the logs are provided by the driller.

Keys to the soil descriptions and symbols, and logs of the borings are included in Appendix A. Soil stratification lines shown on the logs represent the estimated boundaries between different soil types, and the actual transition may vary and can be gradual. A CPT result report prepared by ConeTec is also included in the Appendix A.

Soil samples were collected from the borings at selected depths by using either a Standard Penetration Test (SPT) split-barrel drive sampler (ASTM D1586, outside diameter of 2 inches, inside diameter of 1-3/8 inches) or a California-type split barrel drive sampler (ASTM D3550, outside diameter of 3 inches, inside diameter of 2-1/2 inches). The samplers were driven 18 inches (unless otherwise noted) into soils using a 140-pound wireline hammer with a 30-inch free fall in general accordance with ASTM D1586.

The samplers were driven 18 inches, or a shorter distance where hard resistance was encountered, and the number of blows were recorded for each 6 inches of penetration. The SPT sampler had room for liners, but liners were not used in these samplers. Brass or stainless steel liners were used with the California-type sampler. Bulk soil samples were also collected from drill cuttings. Soil samples obtained from the site were packaged and transported to our geotechnical laboratory for further evaluation and testing.

Upon completion of the borings and CPTs, the explored holes were backfilled with lean cement grout in accordance with Santa Clara Valley Water District drilling permit requirements. The tops of the holes were patched with either quick setting concrete mix or asphalt cold patch material. Excess soil cuttings were disposed of offsite by the drilling contractor.

1.2.3 Task 3 – Laboratory Testing

Laboratory testing was performed on representative bulk and relatively undisturbed soil samples from the site to aid in soil classification and development of engineering parameters for geotechnical analysis and design recommendation. The tests were performed in general conformance with the current ASTM or Caltrans standards, which include:

- Moisture content and dry unit weight determinations per ASTM D2937;
- Sieve analysis per ASTM D6913;
- Atterberg Limits (plastic and liquid limits) per ASTM D4318;
- Unconsolidated undrained triaxial shear strength test (TXUU) per ASTM D2850;
- Unconfined compressive strength test per ASTM D2166; and
- Resistance value (R-value) test per Caltrans Standard Test Method 301.

All tests were performed by Kleinfelder's geotechnical laboratory in Hayward, California. The results of the geotechnical laboratory testing performed for this study are presented in Appendix B.

Corrosivity tests that include redox, pH, chloride, sulfate, sulfide, and resistivity were performed by CERCO Analytical, Inc. in Concord, California, on one representative onsite soil sample (from Boring KB-2 within a depth of about 5 feet). The test results and a brief evaluation report prepared by CERCO regarding the onsite near-surface soil corrosivity are in included in Appendix B.

1.2.4 Task 4 – Geotechnical Analyses

We analyzed field and laboratory data based on the assumed finished grades, new structures information and layouts, and anticipated structural loads to provide geotechnical recommendations for the project design and construction. Engineering analyses were performed to evaluate feasible foundation systems and bearing capacities, concrete slab support, pavement design, and earthwork. A site-specific ground motion hazard analysis (GMHA) was also performed in accordance with the 2019 California Building Code (CBC).

1.2.5 Task 5 – Report Preparation

This report summarizes the services performed, data acquired, and our findings, conclusions, and geotechnical recommendations for the design and construction of the proposed structures and improvements. Our report includes the following:

- A site vicinity map and a site plan showing the proposed project structures and improvements, and approximate locations of borings and CPTs;
- Boring and CPT logs (Appendix A);
- Results of laboratory testing (Appendix B);
- Previous exploration and laboratory testing by others (Appendix C);
- Discussion of general site surface and subsurface conditions as encountered by our field explorations, including estimated depth to groundwater level;
- Discussion of regional and local geology, and geologic and seismic hazards, including soil liquefaction, dynamic densification, and lateral spreading potentials;
- Recommendations for seismic design parameters in accordance with the 2019 CBC (detailed site-specific ground motion hazard analysis results in Appendix D), site preparation, earthwork, fill placement and compaction, trench backfill, and temporary excavation and shoring;
- Recommendations for the site drainage, landscaping, and for stormwater management bio-retention system design;
- Recommendations for foundation design, allowable bearing pressures, embedment depths, passive resistances, and compatibility constraints under various loading conditions;

- Recommendations for design of retaining structures and basements, including active and at-rest lateral earth pressures, and applicable surcharge loads; and
- Recommendations for flexible and ridged pavement structural sections based on anticipated traffic loading of typical office development.

2 SITE CONDITIONS

2.1 SITE DESCRIPTION

The site is located at 19191 Vallco Parkway in Cupertino, California, as shown on Figure 1. The site is irregular in shape and is bounded by Vallco Parkway to the south, North Tantau Avenue to the east, Highway I-280 to the northeast, and Calabazas Creek to the northwest. Total area of the site is about 8 acres with maximum plan dimensions of about 750 feet by 630 feet. The site grade slopes very slightly downward toward the north. An about 2H:1V to 3H:1V roadway embankment slope up to about 20 feet high at the north is located between the site and North Tantau Avenue.

An existing at-grade two-story office building of about 77,250 sf in footprint generally occupies the southern two-third of the site. The building is surrounded by asphalt concrete paved parking and drive isles, and landscaping features. The existing asphalt concrete pavement are generally in fair to good condition. The site surface drainage appears to sheet flow into onsite catch basins and storm drain system.

The unlined, straightened Calabazas Creek channel bank along the northwestern boundary of the site is estimated to be about 15 to 20 feet high and has slope inclinations of about 2H:1V to 4H:1V. The bottom of the creek is about 20 feet wide and the creek banks are well vegetated. According to available historic topographic maps and aerial photographs, the historic alignment of the Calabazas Creek was located within the site prior to relocating to its current location probably in the 1970s. Our estimated previous creek alignment and top of creek banks based on a 1950 aerial photograph are shown on Figure 2 for reference. The actual alignment and bank locations may vary from our estimate. The previous creek channel was probably backfilled with compacted fills that may extend to depths up to about 15 to 20 feet. The actual extents and details of the channel backfill are unknown.

2.2 EXISTING BUILDING STRCUTRAL AND FOUNDATION INFORMATION

The existing office development (originally known as Tandem Computer Building 4) at the site was constructed in the 1980s. Prior to the existing development, the site was used as an orchard. Based on the existing development record drawings prepared by Cabak Randall Jasper Griffiths Associates and dated March 23, 1981, cut and fill grading of up to about 5 feet had been performed at the site to create the current grade. No grading or fill compaction record was available for our review.

The existing office building is supported on spread footings with interior concrete slab-on-grade. Isolated exterior and interior footings range from 3 feet by 3 feet to 6 feet by 6 feet in size and have a thickness of 16 inches. A continuous footing/grade beam 18 inches wide and 16 inches thick also encloses the building perimeter. The top of these footings is typically at 12 inches below the finish floor elevation. The bottom of footings is founded at about 2½ feet deep. Larger pad footings that support shear walls are founded at about 4 feet deep. The floor slab is 5 inches thick with wire mesh reinforcing. Reported underlayment below the concrete floor slab includes 2 inches of sand, 6-mil membrane, 4 inches of drain rock, and 6 inches of import fill.

It is our understanding that the existing building had experienced differential movements/settlements in the past. This caused widespread cracking of the interior walls and movement/cracking of the concrete floor in areas mostly on the perimeter of the northwestern, northern, northeastern, and eastern edges of the building. According to a previous floor level survey conducted by Kier & Wright in 2014 on both the ground and the second floors of the building, differential floor movements with up to about 5¼" inches were identified. A geotechnical investigation was also performed by BAGG Engineers in 2014 to evaluate the potential causes of the building movement, which included drilling of 4 borings to depths of about 15 to 20 feet and laboratory testing of retrieved onsite soil samples within or near the affected floor areas. According to BAGG, it appeared that the soils below the perimeter of the building had dried out and consolidated (settled), whereas floor areas at about 70 feet away from the perimeter have remained moist, and either have not moved at all or have heaved somewhat. As remedial measures, they recommended mature trees on the northern and eastern building corners be removed and building underpinning and floor re-leveling be performed if needed.

As indicated on the foundation underpinning plan (Sheet S1.0 dated May 13, 2015) and details (SK-1 and SK-2 dated June 22, 2015) prepared by SEI Engineers, drilled cast-in-place concrete piers of 2 feet in diameter and 20 feet in length had been installed in 2015 along the west, north, and east building perimeter to support the bottom of the existing perimeter grade beam via new concrete haunches. According to SEI's letter dated July 30, 2020, based on their limited inspection of the building exterior and areas adjacent to the building, settlement documented in the 2014 floor level survey has stabilized and any subsequent ground floor slab movement is likely to be minimal. They also concluded Life Safety performance of the building has not been compromised by the previous settlement.

Based on our review of available information, we generally agree that the differential building movements/settlements were most likely resulted from shrinkage and expansion of the underlying soils and fills due to near-surface moisture changes caused by trees and the lack of a sufficient layer of non-expansive fills below the floor slab.

3 GEOLOGY

3.1 GEOLOGIC SETTING

3.1.1 Regional Geology

The San Francisco Bay Area lies within the Coast Ranges geomorphic province, a series of discontinuous northwest trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. Geologic and geomorphic structures within the San Francisco Bay Area are dominated by the San Andreas Fault (SAF), a right-lateral strike-slip fault that extends from the Gulf of California in Mexico, to Cape Mendocino, on the Coast of Humboldt County in northern California. It forms a portion of the boundary between two independent tectonic plates on the surface of the earth. To the west of the SAF is the Pacific plate, which moves north relative to the North American plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF; however, it is also distributed, to a lesser extent across a number of other faults that include the Hayward, Calaveras, and Concord, among others. Together, these faults are referred to as the SAF system. Movement along the SAF system has been ongoing for about the last 25 million years. The northwest trend of the faults within this fault system is largely responsible for the strong northwest structural orientation of geologic and geomorphic features in the San Francisco Bay Area. Currently, active compressional forces normal to the northwest structural trend of the Coast Range province are also partially responsible for the strong structural trend and uplift of the mountains within the province. These compressional forces are responsible for the movements associated with the Great Valley fault system, a series of blind (no surface expressions of the faults are evident) thrust faults along the eastern margin of the Coast Range province and for the folding of the younger rocks within the region.

Basement rocks west of the SAF are generally granitic, while to the east they 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 (approximately 200-65.5 million years old [USGS, 2010]). Overlying the basement rocks are Cretaceous (approximately 145.5 to 65.5 million years old) marine, as well as Tertiary (approximately 65 to 2.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted as a result of Late Tertiary and Quaternary age (approximately 2.6 million years old to present day) regional compressional forces. The inland valleys as well as the structural depression within which the San Francisco Bay is located are filled with unconsolidated to semi-consolidated deposits of Quaternary age (about the last 2.6 million years). Continental surficial deposits (alluvium, colluvium, and landslide deposits) generally 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 (Bay Mud) and sand.

3.1.2 Site Geology

The project site has been mapped by the CGS (2002), Witter et al. (2006), and Dibblee and Minch (2007) among others. The CGS (2002) indicates the site is underlain Holocene age (approximately 11,700 years old to present day) alluvial fan deposits, consisting of clay, silt, silty sand, and clayey sand. Witter et al. (2006) indicate the majority of the site is underlain by Holocene age alluvial fan deposits, comprised of moderately to poorly sorted and moderately to poorly bedded sand, gravel, silt and clay. Witter et al. (2006) have mapped the Calabazas Creek alignment as being underlain by historic stream channel deposits, consisting of unconsolidated sand, gravel, and cobbles, with minor silt and clay. Dibblee and Minch (2007) identify two geologic units underlying the site. The majority of the site is shown by the authors to be underlain by younger Holocene age stream alluvium within alluvial fan deposits, consisting of gravel, sand, silt, and clay. Dibblee and Minch (2007) indicate the south end of the site is underlain by older Holocene age alluvial fan deposits, comprised of fine-grained sand, silt and gravel.

3.2 SUBSURFACE CONDITIONS

The subsurface conditions encountered during our field explorations are consistent with geologic mapping of the project site vicinity and the site development history.

Our borings and CPTs were performed within the existing asphalt concrete paved parking lot and drive isles. We estimate the average pavement section consist of about 3 inches of asphalt concrete and about 6 inches of aggregate base. Below the pavement section, various fills were encountered within Borings KB-1, KB-2, and KB-3, which appeared to be dense/hard and compacted in general. The fills extend to about 5 feet deep in Borings KB-1 and KB-3, and to about 11½ feet deep in Boring KB-2. This may be where the previous creek channel was located. No fill was identified in Boring KB-4. Below the fill layer, stiff to hard clays with interbeds of medium dense to very dense sands and gravels were encountered to depths of about 20 to 40 feet.

Underlying this mostly clay layer, dense to very dense sands and gravels with occasional clay layers were encountered to the maximum depth explored of about 76½. CPT penetration refusal (with tip resistance up to about 600 tsf) was encountered by CPTs at depths of about 51 to 70 feet. Similar subsurface conditions were also reportedly encountered by BAGG at the site in 2014.

According to our lab testing results, the near-surface fills and clays have a medium to high plasticity and moderate to high expansion potential.

3.3 GROUNDWATER

Groundwater was not encountered in any of our borings or CPTs to the maximum depth explored of about 76½ feet. The California Geological Survey (2002) indicates the historic high groundwater level at the site is deeper than 50 feet.

Fluctuations in groundwater level could occur due to seasonal variations, water level in the creek, variations in rainfall and runoff, regional groundwater withdrawal or recharge, construction activities, and other factors. Due to the presence of porous sand and gravel layers at about the creek bottom elevation, the groundwater level below and along the creek channel could be influenced by the seasonal water flow in the creek.

In addition, localized zones of perched water and high soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas on or immediately adjacent to the site can also cause water seepage at shallow depth below the site. In general, late fall to early summer construction can experience extra earthwork costs related to the presence of groundwater or seepage, depending on the magnitude of prior seasonal rainfall.

3.4 ASSESSMENT OF POTENTIAL GEOLOGIC HAZARDS

3.4.1 Localized Faulting

The site is not located within an Earthquake Fault Zone designated by California State (CGS 2021) in accordance with the Alquist-Priolo Earthquake Fault Zone Act of 1972. The nearest zoned active fault is the San Andreas fault, which is located approximately 6.5 miles southwest of the site. The Working Group on California Earthquake Probabilities (WGCEP, 2015) however indicate the Monte Vista-Shannon fault is the most proximal fault, located approximately 2.8 miles southwest of the site location. It should be noted the Monte Vista-Shannon Fault is not zoned as active by the CGS2021. However, the Santa Clara County Geologic Hazard Zones Map (2012) and City of Cupertino General Plan (2014) considers the Monte Vista-Shannon fault a potential surface rupture and seismic shaking hazard. That said, moderate to major earthquakes generated on the San Andreas, the Monte Vista-Shannon and other faults in the region can be expected to cause strong ground shaking at the site.

The proximities and seismic parameters of significant faults in the vicinity of the crossing location are listed in Table 2. The locations of the faults presented on Table 2 are based on the WGCEP (2015), and the CGS (1974), where noted.

Table 2
Significant Faults

Fault Name	Closest Distance to Site* (miles)	Slip Sense**
Monte Vista-Shannon	2.8	R
Silver Creek	6.5	SS
San Andreas	6.6 (CGS, 6.5)	SS
Pilarcitos	8.6	Unspecified
Butano	9.7	SS
Hayward	11.4	SS
Sargent	13.2	R
Calaveras	13.6	SS
Mission	13.9	R
Zayante-Vergales	14.8	SS
San Gregorio	20.1	SS
Greenville	28.1	SS
Ortigalita	39.0	SS
Quien Sabe	44.0	SS

* Closest distance to the potential rupture.

** SS – Strike Slip; R – Reverse/Thrust; N - Normal

The United States Geological Survey (2020 Update) identifies two faults in close proximity to the site. The Stanford fault, located approximately 1.1 miles northeast of the site, is characterized as an undifferentiated Quaternary fault (fault activity within the last 1.6 million years). The Cascade fault, located 1.2 miles southwest of the site, is also characterized as an undifferentiated Quaternary fault. Both the Stanford and Cascade faults are not zoned as active by the CGS (2021) and are not considered a source of seismic shaking by the WGCEP (2015).

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

3.4.2 Soil Liquefaction, Dynamic Densification, and Lateral Spreading

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers. These soils can dramatically lose strength due to increased pore water pressure during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soils acquire mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated sands that lie close to the ground surface; although, liquefaction can also occur in fine-grained soils, such as low-plasticity silts. Densification can occur where the loose cohesionless soils are unsaturated. Lateral spreading occurs when soils liquefy during an earthquake event and the liquefied soils with the overlying soils move laterally to unconfined surfaces (i.e., the creek channel to the west of the site), which causes significant horizontal ground displacements.

According to Witter et al. (2006), the site, besides the area of the previous Calabazas Creek channel, is located in an area that has been characterized as having a moderate liquefaction susceptibility. The previous Calabazas Creek channel is in an area mapped as having a very high liquefaction susceptibility. The Seismic Hazard Zones Map of the Cupertino Quadrangle (CGS, 2002) also indicates that the previous Calabazas Creek channel is located in a liquefaction seismic hazard zone as designated by California State.

Due to the lack of groundwater within the maximum depth explored of about 76½ feet and the encountered cohesionless soils are generally dense to very dense in composition, it is our opinion that the potential for soil liquefaction to impact the site is low. In addition, based on the types and densities of the sols encountered at the site, the potential for dynamic densification of unsaturated soils is low. It is also our opinion that, based on Youd et al. (2002) empirical correlations, the potential for ground surface damage at the site resulting from lateral spreading adjacent to the creek channel is low since the encountered cohesionless soils at and around the creek bottom elevation have SPT N-value more than 15.

3.4.3 Landsliding

Landslides and other forms of mass wasting, including mud flows, debris flows, soil slips, and rock falls occur as soil or rock moves down slope under the influence of gravity. Landslides are frequently triggered by intense rainfall or seismic shaking.

The project site is generally level. However, the North Tantau Avenue roadway embankment slope up to about 20 feet high is located along the eastern site boundary. In addition, the unlined Calabazas Creek bank slope of about 15 to 20 feet high is located along the northwestern site boundary. At the time of our field reconnaissance, we did not observe adverse slope and drainage conditions on these slopes. In our opinion, the potential for landslide hazards to impact the proposed development is low provided these slopes are regularly maintained by the governing agencies.

3.4.4 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade.

The surficial fills and soils at the site generally consist of cohesive clay. Our laboratory testing results of the upper 5 feet of fills and soils indicate Plastic Index (PI) of about 19 to 21, which are considered as having a moderate expansion potential. In addition, some fills, and soils with PI up to about 34 were also reported from previous laboratory testing by others, which are considered as having a high expansion potential. The potential adverse effects of expansive soil hazard at the site can be reduced provided our recommendations in the report are followed.

3.4.5 Subsidence

The site is located near the southwestern end of the Santa Clara Valley. Land subsidence has occurred within the Santa Clara Valley as a result of groundwater overdraft in the past. Extensive measures had been undertaken by Santa Clara Valley Water District and other agencies to re-establish the original groundwater levels. By about 1970, groundwater levels were gradually recovered, and ground subsidence largely ceased. Provided the current recharge and conservation programs remain in place, the potential for subsidence to occur at the site is considered low.

3.4.6 Flooding

The Flood Insurance Rate Map (FIRM) prepared by the Federal Emergency Management Agency (FEMA) was reviewed to identify the potential flood hazard for the project. According to FIRM No. 06085C2009H dated May 18, 2009, the site is within Zone X which is defined as areas having 0.2% annual chance flood hazard or areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile. Based on this information the potential for the project site to be impacted by regional flooding is considered low. However, the accuracy of this information should be confirmed by a qualified civil engineer/hydrologist. The need and/or method for mitigation of potential flooding should also be addressed if needed.

4 GEOTECHNICAL DESIGN RECOMMENDATIONS

4.1 GENERAL

Based on the results of our data review, field reconnaissance, subsurface exploration, lab testing, and engineering analyses, it is our professional opinion that the proposed project is geotechnically feasible, provided the recommendations presented in this report are incorporated into the project design and construction. The primary geotechnical considerations for the project design and construction include demolition of the existing building and its foundation system, presence of the historical creek alignment within the site, expansion/shrinkage potential of the onsite soils and excavation and retaining for the new parking garage basement. The following sections discuss our conclusions and recommendations with respect to current California Building Code (CBC) seismic design considerations, and foundation, improvement, and pavement designs.

4.2 2019 CBC SEISMIC DESIGN PARAMETERS

According to the 2019 CBC, every structure, and portion thereof, including non-structural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7-16, excluding Chapter 14 and Appendix 11A. The Seismic Design Category for a structure may be determined in accordance with Section 1613.3.5 of the 2019 CBC.

Based on geologic mapping of the site region and results of borings and CPTs (as well as in-situ shear wave velocity measurements) performed at the site, in our opinion the onsite soils can be classified as Site Class D, Stiff Soil, per Table 20.3-1 of ASCE 7-16. Site Class D is defined as a soil profile consisting of stiff soil with an average shear wave velocity between 600 ft/sec and 1,200 ft/sec, standard penetration test (SPT) blow counts (N-value) between 15 blows per foot and 50 blows per foot, or undrained shear strength between 1,000 psf and 2,000 psf in the top 100 feet. In addition, based on shear wave velocity (V_S) measurements in the seismic CPT at the site (CPT-1) and correlations with cone tip resistance and boring SPT blowcount, we have estimated a V_S30 of about 1,155 feet/sec (352 m/s) as reasonably representative of the site, which is consistent with a Site Class D profile (near C/D boundary).

Approximate coordinates of the site are noted below.

- Latitude: 37.3253°N
- Longitude: 122.0077°W

A site-specific ground motion hazard analysis (GMHA) was performed for the proposed new office development in accordance with the requirements of the 2019 California Building Code (CBC) which adopts the procedures outlined in ASCE 7-16 and Supplement 1 of that standard. The site-specific horizontal spectral accelerations and design acceleration parameters are summarized in the tables below. Detailed analysis results are included in Appendix D.

Table 3

Site-Specific Horizontal Spectral Accelerations (g)

Period (second)	Design Earthquake (DE) Spectrum	Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectrum
	5% Damping	
0.010	0.639	0.958
0.020	0.645	0.967
0.030	0.666	0.999
0.050	0.752	1.128
0.075	0.909	1.364
0.100	1.058	1.586
0.150	1.292	1.939
0.200	1.449	2.174
0.250	1.576	2.364
0.300	1.640	2.461
0.400	1.636	2.454
0.500	1.527	2.291
0.750	1.222	1.833
1.000	0.986	1.479
1.500	0.654	0.981
2.000	0.479	0.718
3.000	0.339	0.509
4.000	0.256	0.384
5.000	0.198	0.297

Table 4
Site-Specific Design Acceleration Parameters

Parameter	Value (5% Damping)
S_{DS}	1.476g
S_{D1}	1.025g
S_{MS}	2.215g
S_{M1}	1.538g

The site-specific PGA_M value is estimated to be about 0.805g and is controlled by the deterministic geometric mean peak ground acceleration from the Monte Vista - Shannon fault with a magnitude of about 7.1

4.3 FOUNDATIONS

4.3.1 General

Based on the results of our field exploration, laboratory testing, and geotechnical analyses, the proposed at-grade office building can be supported on conventional shallow spread footing foundations and the parking garage can be supported on below-grade mat slab foundations. Recommendations for the design and construction of foundations are presented below.

4.3.2 Footing Foundations

The at-grade office building can be supported on conventional continuous and isolated spread footings that bear on engineered fills or competent native soils. To provide structural continuity and permit spanning of local irregularities, we recommend exterior walls be underlain by a continuous spread footing.

Footings should be embedded at least 36 inches below the lowest adjacent finished grade. The footing dimension and reinforcement should be designed by the Structural Engineer; however, continuous, and isolated spread footings should have minimum widths of 18 and 24 inches, respectively. Portions of the foundations located above an imaginary 1H:1V extending upward from the bottom edges of any adjacent footings or utility trenches should be neglected in the vertical bearing and lateral resistance analyses. Alternatively, the foundation reinforcing could be increased to span the area defined above assuming no soil support is provided. Our recommended allowable spread footing bearing pressures are provided below. These allowable bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes.

Table 5
Allowable Spread Footing Bearing Pressures

Load Condition	Allowable Bearing Pressure (psf)	Factor of Safety
Dead Load	2,500	3.0
Dead plus Live Loads	3,750	2.0
Total Loads (including Wind or Seismic)	7,500	1.5

We estimate maximum total settlement of foundations under the above recommended allowable bearing pressures to be on the order of 1 inch or less. Differential static settlement between similarly loaded footings is estimated to be approximately ½ inch.

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade and by passive resistance acting against the vertical faces of the foundations. An allowable coefficient of sliding friction of 0.3 between the foundation and the supporting subgrade may be used for design. This value includes a safety factor of at least 1.5. For allowable passive resistance, an equivalent fluid weight of 350 pounds per cubic foot (pcf) acting against the side of the foundation may be used where the foundation concrete is poured neat against undisturbed subgrade. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than ½ inch. Passive resistance in the upper 12 inches of soil should be neglected unless the area in front of the footing is protected by concrete or pavement from disturbance. The allowable friction coefficient and passive resistance may be used concurrently without reduction.

Any visible cracks in the bottoms of the footing excavations should be closed by wetting prior to construction of the foundations. We recommend project Geotechnical Engineer observe the footing excavations prior to placing reinforcing steel or concrete to check that footings are founded on appropriate materials. All foundation excavations should be cleaned of loose materials and should be free of water. The footing excavations should be kept moist prior to concrete placement.

4.3.3 Interior Slab-on-Grade

Where interior slabs-on-grade will be used in conjunction with footings, we recommend the interior slabs be at least 5 inches thick, reinforced with a minimum of #4 bars on 18-inch centers (both ways), and supported on an at least 18-inch thick layer of imported, predominantly granular, “non-expansive” engineered fills that meet the requirements presented in this report. Alternatively, the onsite fills and soils can be lime-treated to create the non-expansive fill layer. Slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support. The actual thickness and reinforcing of the slabs should be designed by the project Structural Engineer based upon the actual use and loading of the slabs. We recommend that the interior slabs-on-grade be poured monolithically with the footings.

Floor slab control joints should be used to reduce damage due to shrinkage cracking. Control joint spacing is a function of slab thickness, aggregate size, slump and curing conditions. The requirements for concrete slab thickness, joint spacing, and reinforcement should be established by the designer, based on experience, recognized design guidelines and the intended slab use. Placement and curing conditions will have a strong impact on the final concrete slab integrity.

If migration of water vapor through the slab is undesirable, we recommend a vapor retarder and an underlying 4-inch layer of ¾-inch, clean, crushed, uniformly graded gravel/drain rock be placed between the bottom of the slab and the recommended non-expansive engineered fill layer. The gravel/drain rock layer can be considered as part of the non-expansive engineered fill layer.

We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil or equivalent provided the equivalent satisfies the following criteria: a permeance less than 0.01 perms as guided by ACI 302.2R, Class A strength as determined by ASTM E1745, and a thickness of at least 15 mils. Installation of the vapor retarder, including protrusions where pipes or conduit penetrate the membrane, should conform to ASTM E1643 and the manufacturer’s requirements. Care must be taken to protect the membrane from tears and punctures during construction. The edges of the vapor retarder membrane should be draped over the interior side of the footing excavations.

Normally, 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 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 curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

We recommend the foundation designer to determine the maximum water/cement ratio for the concrete mix design and whether corrosion protection is needed for the foundation concrete and reinforcing steel. The soil corrosivity test results and a brief evaluation report are in included in Appendix B; the foundation designer should determine if additional testing is needed. In addition, the foundation designers should provide recommendations to reduce the potential for differential concrete curing if necessary.

4.3.4 Mat Slab

The below-grade parking garage can be supported on structural mat slabs foundations that bear on properly prepared subgrade. The subgrade should be prepared in accordance with recommendations in this report and proof-rolled to provide a smooth, unyielding surface for slab support. A layer of 6 to 12 inches thick of Caltrans Class 2 aggregate base can be placed at the bottom to aid in base stabilization and to protect the subgrade from disturbance during construction if necessary.

We recommend the mat slabs be at least 8 inches thick, reinforced with a minimum of #4 bars at 18-inch on center, top and bottom, and in both directions. The mat should be appropriately reinforced to spread the loads uniformly across the mat. The actual thickness and reinforcing of the slabs should be designed by the project Structural Engineer based upon the actual use and loading of the slabs.

Our recommended allowable bearing pressures are provided below. These allowable bearing pressures are net values; therefore, the weight of the slab can be neglected for design purposes. Based on these bearing pressures, we estimate that total settlement of the structure will be less than 1 inch.

Table 6
Allowable Mat Slab Bearing Pressures

Load Condition	Allowable Bearing Pressure (psf)	Factor of Safety
Dead Load	2,500	3.0
Dead plus Live Loads	3,750	2.0
Total Loads (including Wind or Seismic)	5,000	1.5

We recommend a modulus of subgrade reaction k1 (1 foot by 1 foot) of 200 pounds per cubic inch (pci) be used for the design of the structural mat foundation. If needed, this value should be adjusted to generate the modulus of subgrade reaction k based on the slab dimensions. We recommend mat slab structural calculations be provided for our review to confirm the modulus value.

Addition recommendations regarding slab control joint, vapor retarder and slab design and construction are included in Section 4.3.3. Considerations should be given to waterproof the below-grade basement floor and walls if underground seepage is a concern.

4.4 RETAINING WALLS AND BELOW-GRADE WALLS

Building retaining walls could include basement walls, low loading dock walls and below-grade walls for elevator pits and other utility vaults. Any walls that retains soils should be designed to resist both lateral earth pressures and any additional lateral loads caused by roadway surcharging, earthquake loading, and hydrostatic pressure if wall back-drainage is not provided.

If no movement is allowed at the top of the walls, at-rest pressures need to be resisted. If the wall is allowed to deflect outward at the top at least 0.002 H, where H is the wall height, it may be designed to resist active pressures. The recommended active and at-rest lateral earth pressures under both drained and undrained conditions are provided in Table 5, which are expressed as equivalent fluid pressures in pounds per cubic foot (pcf).

Table 7

Lateral Earth Pressures for Retaining Structures

Wall Movement	Backfill Condition	Drained Equivalent Fluid Pressure (pcf)	Undrained Equivalent Fluid Pressure (pcf)	Incremental Seismic Pressure (pcf)
Free to Deflect (Active Condition)	Level	40	80	38*
Restrained (At-rest Condition)		60	90	N/A**

Note: * Walls more than 6 feet high should be designed to support an incremental seismic lateral pressure, which is applied as a triangular fluid pressure distribution (not inverted).

** For restrained wall, use the static active earth pressure and seismic increment to check the seismic condition; 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 wall.

Walls retaining more than 6 feet of backfill should be designed to support an incremental seismic lateral pressure noted in the above table, using a triangular fluid pressure distribution (not inverted). The seismic lateral earth pressure was estimated based on the half of the peak ground acceleration from a MCE earthquake (0.5 x PGA_M). Due to the transient nature of the seismic loading, a factor of safety of at least 1.1 can be used in the design of the walls when they resist seismic lateral loads.

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, including traffic, or dead load) located within an imaginary 1H:1V plane projected 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.

The recommended drained lateral pressures assume walls are fully back drained to prevent the build-up of hydrostatic pressures. If drainage behind the wall is omitted, the wall should be designed for the undrained condition. Considerations should be given to applying waterproofing to backside of the wall to reduce water/vapor transmitting and efflorescence forming on the front wall face. Wall back-drainage can be accomplished by using ½ to ¾ inch crushed, uniformly graded gravel entirely wrapped in filter fabric, such as Mirafi 140N or equal (an overlap of at least 12 inches should be provided at all fabric joints). The gravel and fabric should be at least 12 inches wide and extend from the base of the wall to within about 2 feet of the finished grade at the top (Class 2 permeable material per Caltrans Specification Section 68 may be used in lieu of gravel and filter fabric). The upper 2 feet of cover backfill should consist of relatively impervious material.

A 4-inch diameter, perforated pipe should be installed at the base and centered within the gravel. The perforated pipe should be connected to a solid collector pipe that transmits the water directly to suitable discharge facilities. If weep holes are used in the wall, the perforated pipe within the gravel is not necessary provided the weep holes are kept free of animals and debris, are located no higher than approximately 6 inches from the lowest adjacent grade and are able to function properly. Weepholes should be spaced at about 10 to 15 feet apart.

As an alternative to using gravel, pre-fabricated drainage panels (such as AWD SITEDRAIN Sheet 94 for walls or equal) may be used behind the walls in conjunction with perforated pipe (connected to solid collector pipe), weep holes, or strip drains (such as SITEDRAIN Strip 6000 or equal).

Design of retaining walls and back-drainage systems 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 observed and documented by a Kleinfelder representative.

4.5 SHALLOW DRILLED PIERS FOR LIGHT POLES

The compressive axial capacity of drilled pier for light poles may be estimated based on an allowable skin friction capacity of 500 pounds per square foot (which includes a factor of safety of 2) within engineered fills or competent soils. The upper two feet of the skin friction capacity should be ignored. The uplift capacity may be estimated as 80 percent of the allowable compressive axial capacity. A one-third increase in the allowable capacities may be used for transient loading conditions such as wind or seismic loads. These allowable skin friction capacities are net values; therefore, the weight of the pier can be neglected in analyses. Settlement of the drilled piers under the recommended bearing capacity is estimated to be less than ½ inch.

For use in conjunction with pole or post foundation formula listed in 2019 CBC Section 1807.3, we recommend an allowable soil passive resistance (which includes a factor of safety of 1.5) equal to an equivalent fluid weighing 300 pounds per cubic foot be used for pole foundations. This value can be used up to a maximum value of 3,000 psf. The passive resistance can be applied against twice the projected diameter of pier shaft if the piers are spaced center-on-center at least 3 times of the pier shaft diameter.

We recommend the piers be at least 6 feet deep. The actual pier length should be determined based upon the actual vertical and lateral loadings of the pier. The upper one foot of pier shaft should be neglected in passive resistance design unless it is confined by a pavement or concrete slab.

4.6 EXTERIOR FLATWORK

Due to the moderate to high expansion potential of onsite fills and soils, we recommend exterior concrete slabs for pedestrian use or landscape be at least 4 inches thick and underlain by a layer of 18 inches non-expansive engineered fills or lime-treated subgrade. The non-expansive fills or lime-treatment should extend at least 3 feet laterally beyond edges of flatwork.

The expansive clayey soils and fills at the site could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as doors that open outward. We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, considerations should be given to installing of #4 bars spaced at approximately 18 inches on center in both directions. Both score joints and expansion joints can be used to control cracking and allow for expansion and contraction of the concrete slabs. Actual design for exterior slabs should be provided by the project structural engineer.

We recommend appropriate flexible, relatively impermeable fillers be used at all expansion and cold joints. The installation of dowels at all expansion and cold joints will reduce differential slab movements; if used, the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 18 inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced.

4.7 SITE DRAINAGE AND LANDSCAPING

We recommend positive surface gradients of at least 2 percent be provided adjacent to structure foundations to direct surface water away from structure foundations and toward suitable discharge facilities. Surface gradients should also conform to the project requirements and the 2019 CBC. Roof downspouts and landscaping drainage inlets should be connected to solid pipes that discharge into closed storm drain systems. Ponding of surface water must not be allowed adjacent to structure foundations, exterior slabs, and pavements. In order to reduce moisture changes in the soils below and adjacent to structure foundations and exterior slabs, landscaping and irrigation systems should be designed and installed in a uniform and systematic manner as equally as possible on all sides of the foundations and adjacent to exterior slabs. If landscaping plans include trees, they should be planted a minimum distance of one-half the anticipated mature height of the trees from structures and improvements to reduce the adverse effects from the tree roots. We recommend that drought resistant plants and low flow/drip irrigation watering systems be used. All irrigation systems should be regularly maintained and inspected for leakage. Over-watering must be avoided.

Where exterior slabs or pavement areas abut landscaped areas, the aggregate base and subgrade soil should be protected against saturation. Continuous vertical cut off structures, such as deepened concrete curbs or edges, can be used to reduce potential lateral seepage under slabs and pavements from the adjacent landscaped areas. Vertical cut-off structures should extend at least three inches below the aggrege base/subgrade interface and be poured neat against undisturbed native soil or compacted fill.

Sand or gravel backfilled trench laterals that extend from irrigated landscaped areas, such as lawns or planting strips, toward pavements, exterior slabs, and building foundations, should be plugged with low strength concrete, sand-cement slurry mixture, or onsite clayey soils below the edges of pavements and exterior slabs, and under perimeters of the foundations.

4.8 STORMWATER MANAGEMENT BIO-RETENTION

If planned, we recommend bio-retention swales and basins where they are located within 10 feet from structure foundations and improvements (such as building foundations, exterior flatwork, and pavements) be completely lined with a relatively impermeable membrane to reduce water seepage and the potential for damage to the adjacent structures and improvements. The membrane can consist of a layer of at least 30-mil impermeable liner installing below and along the sides of these facilities to direct the collected water into subdrain pipes. The membrane should be lapped and sealed in accordance with the manufacturer's requirements, including sealing joints where pipes penetrate the membrane.

The bio-treatment soil mix materials within swales and basins should be considered as having no lateral load resistant. We recommend the sidewall slopes of the swales and basins not to exceed 2H:1V to reduce potential vertical and lateral movements of surrounding ground surface. In addition, we recommend either structures and improvements be setback beyond an imaginary 1H:1V plane projected upward from the bottom edges of the swales and basins or the affected areas of the structures and improvements be supported by deepening foundations or edges. Alternatively, properly designed below-grade concrete boxes/planters with a drainage system at the bottom can be used to build the swales and basins and to retain surrounding ground and structures/improvements.

4.9 PAVEMENTS

4.9.1 Asphalt Concrete

Due to the moderate to high expansion potential of the onsite fills and soils and based on the R-value test result of a bulk sample from the site, we recommend an R-value of 5 be used for the project pavement designs. We developed the following alternative preliminary pavement sections (Table 7) based on the State of California Department of Transportation Highway Design Manual (2020), typical traffic indices for the proposed development, and an assumed design life of about 20 years. The preliminary pavement sections should be revised, if necessary, when actual project design traffic indices are determined by the project Civil Engineer.

Table 8

Asphalt Concrete Pavement Section – R-Value 5

Traffic Index (T.I.)	Asphalt Concrete AC Thickness (Inches)	Aggregate Base AB Thickness (Inches)	Total Thickness (Inches)
4.0 (Auto Parking)	2.5	7.0	9.5
5.0 (Auto Access Areas)	3.0	10.0	13.0
6.5 (Heavy Truck Access)	4.0	14.0	18.0

If the pavements are planned to be placed prior to or during any construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. If the pavement sections will be used for construction access by heavy trucks or construction equipment, we should be consulted to provide recommendations for alternative pavement sections capable of supporting the heavier use and heavier loads.

The aggregate base for use in pavements should conform to Caltrans Standard Specification Section 26-1.02A for Class 2 Aggregate Base. Pavement aggregate base and asphalt concrete should be compacted to at least 95 percent relative compaction as determined, respectively, by ASTM D1557 and Caltrans Test Method 375. The asphalt concrete compacted unit weight should be determined using Caltrans Test Method 308-A or ASTM Test Method D1188. Asphalt concrete should also satisfy the S-value requirements by Caltrans.

4.9.2 Portland Cement Concrete Pavement

Rigid vehicular concrete pavement, including ADA access area and concrete slab for the trash enclosure, was designed in accordance with the method published by the Portland Cement Association (PCA, 1984). A modulus of subgrade reaction of 100 pounds per cubic inch (pci) was assigned to represent the engineered fill subgrade overlain by 12 inches of aggregate base. The modulus of rupture for concrete was assumed to be 550 pounds per square inch. Based on our analysis, we recommend the concrete pavement consist of 6 inches of concrete slab overlying 12 inches of Caltrans Class 2 aggregate base. The actual thickness and reinforcing of the slabs should be designed based on the anticipated traffic loads. The concrete and aggregate base should be constructed in accordance with the appropriate specifications for pavements. Prior to placement of aggregate base, pavement subgrade should be prepared in accordance with Section 5.1.4.

Longitudinal and transverse joint spacing should not exceed 12 feet and 15 feet, respectively. Joint details should conform to PCA guidelines. Expansion joints in concrete slabs should be sealed with petroleum resistant sealant to prevent minor releases from impacting subsurface soil.

4.9.3 Pavement Maintenance

Pavements may undergo movement due to changes in subgrade moisture content. This movement tends to accelerate pavement deterioration. We recommend regular maintenance of the pavement be performed, which may include slurry sealing, crack filling, and chip seals, as necessary. If regular maintenance is not performed, the pavement could experience premature degradation requiring more extensive repairs. A crack sealing program should be performed annually to slow pavement deterioration. Any areas where surface water stands on the surface should be remediated. Over time as cracking becomes more pronounced, a slurry seal coat should be applied.

4.10 SOIL CORROSIVITY

Corrosivity tests, that include redox, pH, chloride, sulfate, sulfide, and resistivity were performed on one composite soil sample from Boring KB-2 within a depth of about 5 feet. The test results and a brief evaluation report prepared by CERCO Analytical Laboratory of Concord, California, regarding the onsite soil corrosivity are included in Appendix B. The table below summarizes the corrosivity tests results.

Table 9

Corrosion Test Results

Sample	Redox (mV)	pH	Resistivity In-situ Moisture (ohms-cm)	Resistivity 100% Saturation (ohms-cm)	Soluble Sulfide Content (ppm)	Soluble Chloride Content (ppm)	Soluble Sulfate Content (ppm)
KB-2 at 0 to 5 feet deep	+340	8.52	2,400	1,400	None Detected	39	95

Note: *None detected.

Based on CERCO's evaluations on the soil sample from the site, the water-soluble sulfide concentration was not detected (with a detection limit of 50 ppm). The chloride ion concentration is determined to be insufficient to attached steel embedded in a concrete mortar coating. The sulfate ion concentration is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel.

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

The measured pH of the sample does not present corrosion problems for buried iron, steel, mortar-coated steel, and reinforced concrete structures. The measured redox potential indicates anaerobic soil conditions, which are potentially "slightly corrosive".

We recommend these test results and CERCO's evaluation report be forwarded to your underground contractors, and foundation designers and contractors so that they can design and install corrosion protection measures for buried concrete structures and ferrous metal if needed. We also recommend additional testing be performed if the test results in Appendix B are deemed insufficient by the designer of the corrosion protection.

5 CONSTRUCTION RECOMMENDATIONS

5.1 EARTHWORK

5.1.1 General

Site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations and other local, state, or federal specifications, and the recommendations included in this report. References to maximum dry unit weights are established in accordance with the latest version of ASTM Test Method D1557 (modified Proctor). The earthwork operations should be observed and tested by the project Geotechnical Engineer.

5.1.2 Clearing and Site Preparation

The site should be cleared of all obstructions, including existing structures and their foundation systems, concrete slabs-on grade, asphalt concrete pavements, existing utilities, and pipelines and their associated backfill, designated trees and their associated entire root systems, landscaping, and debris.

The existing building foundation underpinning piers can be left in place provided the pier caps and haunches be completely removed. The top of pier shafts should be cut down to a depth of at least 5 feet below bottom of the new building ground floor slab or at least 2 feet below the bottom of new building footing. Existing underground pipelines that are deemed suitable to be abandoned in place should be plugged and filled with controlled density fill/sand slurry.

Concrete/asphalt concrete, baserock, and trench backfill materials can be reused as new fills provided debris is removed and concrete/asphalt concrete are broken up to meet the engineered fill size requirements presented in this report. Debris produced by demolition operations, including wood, steel, piping, plastics, etc., should be separated and disposed of off-site.

Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with engineered fills and compacted to the requirements presented in this report. We recommend backfilling operations for any excavations to remove underground obstructions be performed under observations and testing of the project Geotechnical Engineer. After clearing, areas containing heavy surface vegetation should be stripped to an appropriate depth to remove these materials. We estimate the stripping depth to be about 6 to 12 inches. The amount of actual stripping should be determined in the field at the time of construction. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired

5.1.3 Existing Weak Fill and Soil Removal and Re-Compaction

The demolition of the existing building and improvements at the site will loosen and disturb the about upper 3 to 5 feet of onsite site fills and soils. The disturbed weak fills and soils located within the areas of proposed new structures and improvements (such as structural fills, exterior flatwork, and pavements) should be completely removed and re-compacted to depths where competent fills and soils are located. New landscaping and open space areas that do not support any new improvements will not need reworking other than typical clearing. The removal and re-compaction should extend at least 5 feet beyond the proposed structure footprint and at least 3 feet beyond other proposed improvements.

The over-excavation and re-compaction process can consist of over-excavating about 2 to 4 feet below existing ground surface, scarifying and re-compacting the bottom 12 inches in-place and replacing the excavation with compacted engineered fills. Deeper over-excavation may be required in local areas where thicker weak fills and soils are encountered.

There is no grading or fill compaction record available in regard to the fills placed during the construction of the existing development as well as extents and details of the backfill for the previous Calabazas Creek channel. Based on the results of borings and CPTs at the site, these fills and backfills appear to be compacted. We recommend test pits or potholing be performed during the site demolition and grading to identify any potential weak fills that may exist within the site.

Where the over-excavation limits abut adjacent structures or improvements, the project Geotechnical Engineer should be consulted to determine the actual vertical and lateral extents of over-excavation, so the adjacent structures or improvements are not adversely impacted. Over-excavations should also be performed so that no more than 5 feet of differential fill thickness exists below the proposed building foundations. The removed fills and soils can be used as new fills provided, they are placed and compacted in accordance with the engineered fill requirements presented in this report. The extent of the removal and re-compaction may vary across the site and should be determined in the field by the project Geotechnical Engineer at the time of earthwork operations.

5.1.4 Subgrade Preparation

Following the site clearing and preparation, and weak fill and soil over-excavation and re-compaction, soil subgrades in areas to receive structure foundations and improvements (such as engineered fills, slabs-on-grade, exterior flatwork, and pavements) should be proof-rolled with a fully-loaded tandem-axle dump truck or water truck. Areas identified as being soft or yielding may require additional compaction or over-excavation as determined in the field by the project Geotechnical Engineer. A non-expansive fill layer or lime treatment of subgrade is also required for building interior slab-on-grade and exterior flatwork at the site. If used, a 4 to 5 percent of lime by dry weight can be assumed initially for estimating purposes. The actual dosage should be designed by a specialty contractor based on the actual site and soil conditions and confirmed by laboratory and field testing results.

The prepared subgrade surface should be firm, unyielding, and kept moist during construction. The subgrades should be protected from damage caused by weather and construction traffic. If the subgrades are left exposed to weather for extended periods of time or are disturbed by construction traffic, the project Geotechnical Engineer should be consulted on the need for subgrade moisture reconditioning and/or scarifying and recompacting to eliminate shrinkage cracks and disturbances.

Prior to casting exterior flatwork, the prepared subgrade soils should be moisture conditioned to above optimum. Careful control of the concrete slab water/cement ratio should be performed to avoid shrinkage cracking due to excess water or poor concrete finishing or curing.

The site surficial soils are fine-grained, moisture sensitive, and susceptible to disturbance, rutting, and pumping during construction. The contractor should plan to repair subgrade conditions that become unstable/disturbed and should develop a plan to manage subgrade trafficability across the site throughout the construction period. Features of this plan may include temporary surface haul roads, limited traffic routes, etc.

5.1.5 Foundation Excavations

Following excavation to the foundation subgrade elevations, the exposed subgrade should be observed by the project Geotechnical Engineer to evaluate the presence of competent soils at the design elevations. If soft or disturbed soil, debris or otherwise unsuitable soil is present at the base of footing excavations, it should be over-excavated and replaced with structural concrete, 2-sack sand-cement slurry, or structural fill to the depth determined by the project Geotechnical Engineer.

5.1.6 Engineered Fill Materials

Any new fills placed at the site should consist of compacted engineered fills, except for landscaping materials which are placed on level ground. All engineered fills should have an organic content of less than 3 percent by volume and should not contain rocks or lumps larger than 3 inches in greatest dimension. Onsite soils and fills can be used as new fills. Imported non-expansive fills should meet the requirements listed in the below table.

Table 10

Imported Non-Expansive Engineered Fill Requirements

Fill Requirement		**Test Procedures**	
Gradation		**ASTM**	**Caltrans**
Sieve Size	**Percent Passing**		
3 inch	100	D6913	202
¾ inch	70-100	D6913	202
No. 200	20-50	D6913	202
Plasticity			
Liquid Limit	**Plasticity Index**		
<30	<12	D4318	204

All imported fills should be non-corrosive and should not contain environmental contaminants or debris. All imported fills and their laboratory testing results should be reviewed and approved by the project Geotechnical Engineer prior to transportation and use on site.

5.1.7 Fill Placement and Compaction

We recommend engineered fills be compacted to at least 90 percent relative compaction, as determined by ASTM D1557. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately eight inches in uncompacted thickness (loose measurement).

We recommend engineered fills be moisture conditioned to at least 3 percent above optimum water content. In order to achieve satisfactory compaction of fill materials, it may be necessary to adjust the water content at the time of earthwork operations. This may require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet.

The moisture content of the fill is considered very important, and therefore, both relative compaction and moisture content should be used to evaluate compaction acceptance. If both criteria are not within the specified tolerances, the fill should not be accepted, and the contractor should rework the material until the fill is placed within the specified tolerances. The prepared subgrade in paved areas should be covered with aggregate base within 24 hours to reduce drying of the subgrade soil.

5.1.8 Trench Backfill

Pipe zone backfill (i.e. material beneath and in the immediate vicinity of the pipe) should consist of imported soil less than ¾-inch in maximum dimension. Trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of onsite soil or imported fill that meets the requirements for engineered fill provided above.

If imported material is used for trench zone backfill, we recommend it consist of silty sand or Caltrans Class 2 aggregate base. In general, open-graded gravel should not be used for trench zone backfill due to the potential for soil migration into the relatively large void spaces present in this type of material.

Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local building requirements and/or bedding requirements for specific types of pipes. We recommend the project Civil Engineer develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study.

Pipeline trenches should be backfilled with engineered fills placed in lifts of approximately 8 inches in uncompacted thickness. Thicker lifts can be used provided the method of compaction is approved by the project Geotechnical Engineer and the required minimum degree of compaction is achieved. Backfill should be placed by mechanical means only; jetting is not permitted. Onsite fills and soils and imported fills when used for trench backfill should be compacted to at least 90 percent relative compaction. Imported sands and aggregate bases when used for trench backfill should be compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction.

Potential sources of water such as water pipes, drains, and the like should be frequently examined for signs of leakage or damage. Any such leakage or damage should be promptly repaired. Sewer lines beneath the structures should have a sufficient slope (at least 1 percent). Plumbing and utility lines should be provided with flexible joints or oversized sleeves where they penetrate floor slabs to prevent breakage caused by differential slab movement. In addition, utility trenches should be plugged with a low permeability cutoff collar to reduce moisture infiltration along the pipe and the bedding materials. Cutoff collars should be constructed of low strength concrete or controlled density fill (low strength sand-cement slurry mixture) that is at least 12 inches thick. The collars should extend into the trench bottom and walls at least 18 inches. They should also extend at least 18 inches above the pipe and into the overlying less permeable trench backfills.

In addition, sand or gravel backfilled trench laterals that extend from irrigated landscaped areas, such as lawns or planting strips, toward pavements and exterior slabs, should also be plugged with the cutoff collars below the edges of pavements and exterior slabs using low strength concrete, controlled density fill, or onsite clayey soils.

5.1.9 Temporary Excavation and Shoring

All excavations must comply with applicable local, state, and federal safety regulations, including OSHA requirements. The responsibility for excavation safety and stability of temporary construction slopes lies solely with the contractor. We are providing this information below solely as a service to our client. Under no circumstances should this information provided be interpreted to mean that Kleinfelder is assuming responsibility for final engineering of excavation or shoring, construction site safety, or the contractors' activities; such responsibility is not being implied and should not be inferred.

Sloughing and/or raveling of cut slopes should be anticipated as they dry out. Where space for slope or benching is not available, shoring will be necessary. In addition, excavations within a 1H:1V plane extending downward from a horizontal distance of 2 feet beyond the bottom outer edge of existing foundations/improvements should not be attempted without bracing and/or underpinning the foundations/improvements. The project Geotechnical Engineer should observe the excavations so that modifications can be made to the excavations, as necessary, based on variations in the encountered soil conditions.

All trench excavations should be braced and shored in accordance with good construction practice and all applicable safety ordinances and codes. Stockpiled (excavated) materials should be placed no closer to the edge of an excavation than a distance equal to the depth of the excavation, but no closer than 4 feet.

If used for basement excavation, shoring system should be designed by a licensed civil engineer or structural engineer who is experienced in the design of shoring for similar site subsurface conditions. The shoring designer should be responsible for the design of temporary shoring in accordance with applicable regulatory requirements. The shoring system should also be installed by an experienced shoring specialty contractor.

In addition to lateral earth pressures, the top of shoring should also be designed to account for load surcharging from typical construction traffic. Heavy construction equipment should not be allowed within a distance equal to the shoring depth unless the shoring is specifically designed for the appropriate surcharge. The anticipated deflections of the shoring system should be estimated by the shoring designer to check if they are acceptable with respect to the adjacent existing ground and structures. If the deflections of the shoring are estimated to be excessive, considerations should be given to using a more rigid shoring system, such as secant pile wall or tieback wall.

We recommend the project shoring plans be reviewed by the project Structural and Geotechnical Engineers. Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. Since the site is underlain by predominantly cohesive soils within the shoring depth, voids and gaps may occur around the sheet piles after pile extraction. The voids and gaps should be properly grouted by the contractor. Considerations can be given to welding grout pipes to the sheet piles prior to installation, so grout can be injected to fill the voids and gaps during extraction of the sheet piles.

5.2 UNSTABLE SUBGRADE CONDITIONS

Should grading be performed during or following extended periods of rainfall, the moisture content of the near-surface soils may be significantly above the optimum moisture content. These conditions could seriously impede grading by causing an unstable subgrade condition. Typical remedial measures include the following:

- Drying: Drying unstable subgrade involves disking or ripping wet subgrade to a depth of about 18 to 24 inches and allowing the exposed soil to dry. Multiple passes of the equipment (likely on a daily basis) will be needed because as the surface of the soil dries, a crust forms that reduces further evaporation. Frequent disking will help prevent the formation of a crust and will promote drying. This process could take several days to several weeks depending on the material, the depth of ripping, the number of passes, and the weather.
- Removal and Replacement with Crushed Rock and Geotextile Fabric: Unstable subgrade could be over-excavated 12 to 24 inches below existing grade and replaced with ¾- or 1-inch crushed rock underlain by geotextile fabric. The geotextile fabric should consist of a woven geotextile, such as Mirafi HP series or equivalent. The final depth of removal will depend upon the conditions observed in the field once over-excavation begins. The geotextile fabric should be placed in accordance with the manufacturer's recommendations.
- Lime Treatment: Unstable subgrade could be dried and partially stabilized by mixing the upper 12 to 18 inches of the subgrade with high-calcium quicklime. For estimating purposes, a 4 to 5 percent of lime by dry weight can be assumed initially. The actual dosage should be designed by a specialty contractor based on the actual site and soil conditions and confirmed by laboratory and field testing results. Final application rates should be determined in the field at the time of construction in consultation with the project Geotechnical Engineer. Lime treatment should be performed by a specialty contractor experienced in this work and in accordance with Caltrans Standard Specifications. Since lime treatment uses the onsite soil, the expense of importing material can be reduced.

5.3 PAVEMENTS

5.3.1 HMA Design

For typical office development pavements, we recommend that asphalt concrete materials meet the latest Caltrans Standard Specifications for ½-inch Type A. Asphalt paving materials and placement methods should meet current Caltrans specifications. Positive drainage of the paved areas should be provided since moisture infiltration into the subgrade may decrease the life of pavements.

5.3.2 Construction Considerations

The pavement sections provided in this report are contingent on the following recommendations being implemented during construction.

- Pavement subgrade should be prepared as recommended in Section 5.1.4.
- Subgrade soils should be in a stable, non-pumping condition at the time the aggregate base materials are placed and compacted.
- Aggregate base materials should be compacted to at least 95 percent relative compaction (ASTM D1557).
- Asphalt paving materials and placement methods should meet current applicable code specifications.
- Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet.

6 ADDITIONAL SERVICES

6.1 PLANS AND SPECIFICATIONS REVIEW

We recommend Kleinfelder perform a review of geotechnical related portions of the project plans and specifications before they are finalized to verify our geotechnical recommendations have been properly interpreted and implemented during design. If we are not accorded the privilege of performing this review, we can assume no responsibility for misinterpretation of our recommendations.

6.2 CONSTRUCTION OBSERVATION AND TESTING

Construction observation and testing are integral design components with respect to the geotechnical aspects of a project. Because geotechnical engineering is an inexact science due to the variability of natural processes, and because only limited portion of onsite soils can be sampled and evaluated during the project geotechnical investigation, unanticipated or changed conditions may be encountered during grading and foundation excavation. Proper geotechnical observation and testing during construction are imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, we recommend that Kleinfelder be retained during the construction of the project to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed.

Our services are typically needed during the following stages of construction:

- Site demolition and grubbing;
- Grading;
- Over-excavation and subgrade preparation;
- Utility trench backfill;
- Pavement base rock placement and site paving; and
- Excavation for foundation.

7 LIMITATIONS

This geotechnical study has been prepared for the exclusive use of Apple and their agents for specific application to the proposed VP01 project located at 19191 Vallco Parkway, Cupertino, California.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner’s budget, tolerance of risk and expectations for future performance and maintenance.

The opinions, conclusions, and recommendations presented in this report are based on our reviews of available geologic and geotechnical data, maps, reports, our site subsurface exploration and laboratory testing results, our engineering analysis results, and information provided by others. Our opinions, conclusions, and recommendations are professional opinions and were made in accordance with generally accepted local and current geotechnical engineering principles and practices. We make no warranty, either express or implied.

It should be recognized that definition and evaluation of subsurface conditions are difficult. Engineering assumptions and judgments leading to conclusions and recommendations are generally made due to incomplete knowledge of the subsurface conditions and limitations of field and lab data. Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed and at the time when services were conducted; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activities (such as construction adjacent to the site, dumping of fill, or excavating).

If soil or groundwater conditions encountered during construction are differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may re-evaluate the recommendations of this report. If the scope of the proposed project construction, including site grading, and locations, types and loadings of structures and improvements, changes from those described in this report, the conclusions and recommendations contained in this report are not considered valid until the changes are reviewed, and the conclusions of this report are modified or approved in writing by Kleinfelder.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. We recommend all geotechnical aspects of construction be monitored on a full-time basis by a representative from Kleinfelder, including site preparation and grading, excavation of foundation, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the Geotechnical Engineer of Record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

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

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than two years from the date of the report. Land use, site conditions (both on site and off site) or other factors (such as building codes) may change over time, so additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of this report and the nature of the new project, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party and the client agrees to defend, indemnify, and hold harmless Kleinfelder from any claims or liability associated with such unauthorized use or non-compliance.

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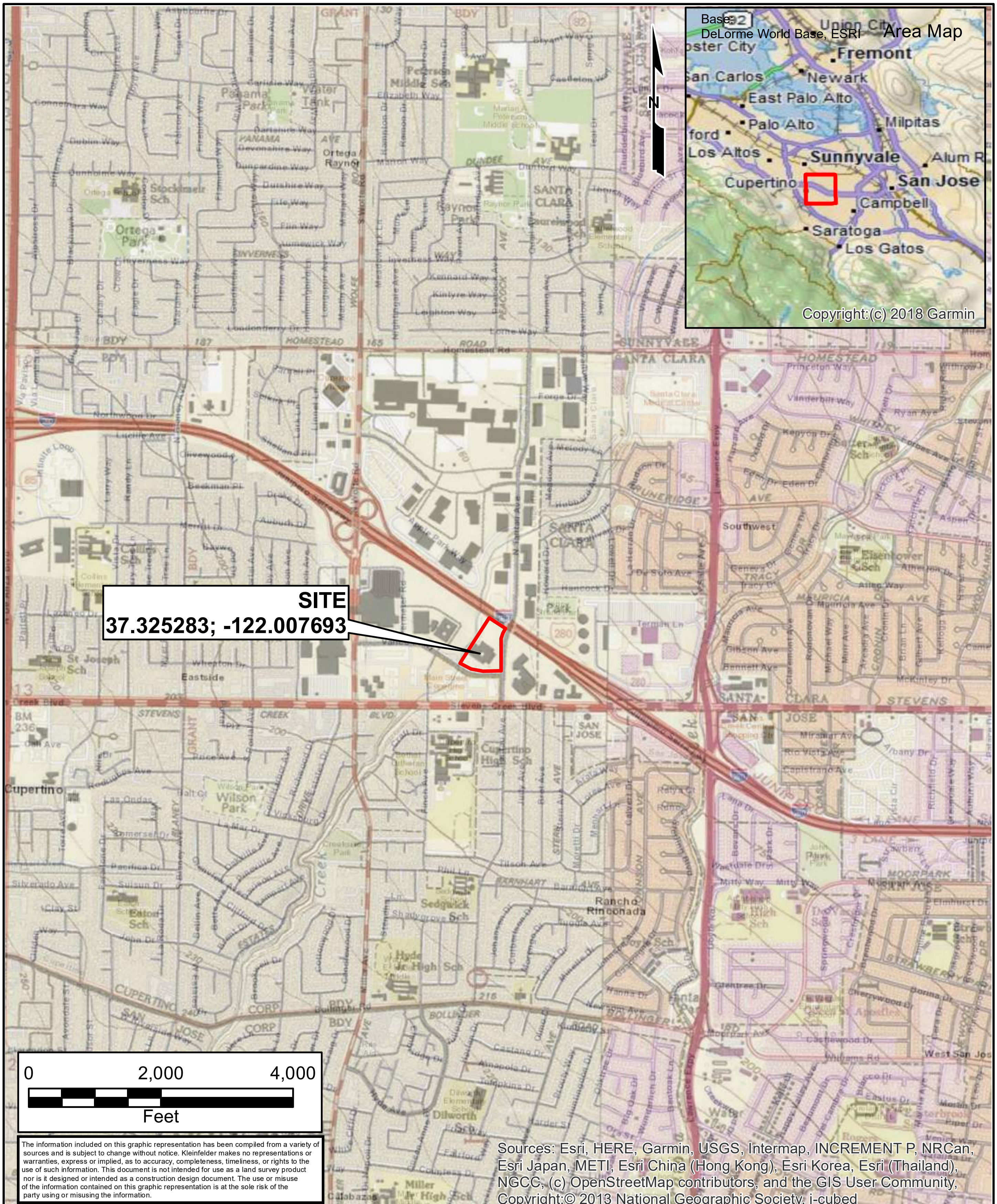
Bright People. Right Solutions.

Figures



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PROJECT NO.	20213246
DRAWN BY:	JDS
CHECKED BY:	TC
DATE:	March 2021
REVISED:	

SITE VICINITY MAP

APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CALIFORNIA

FIGURE

1

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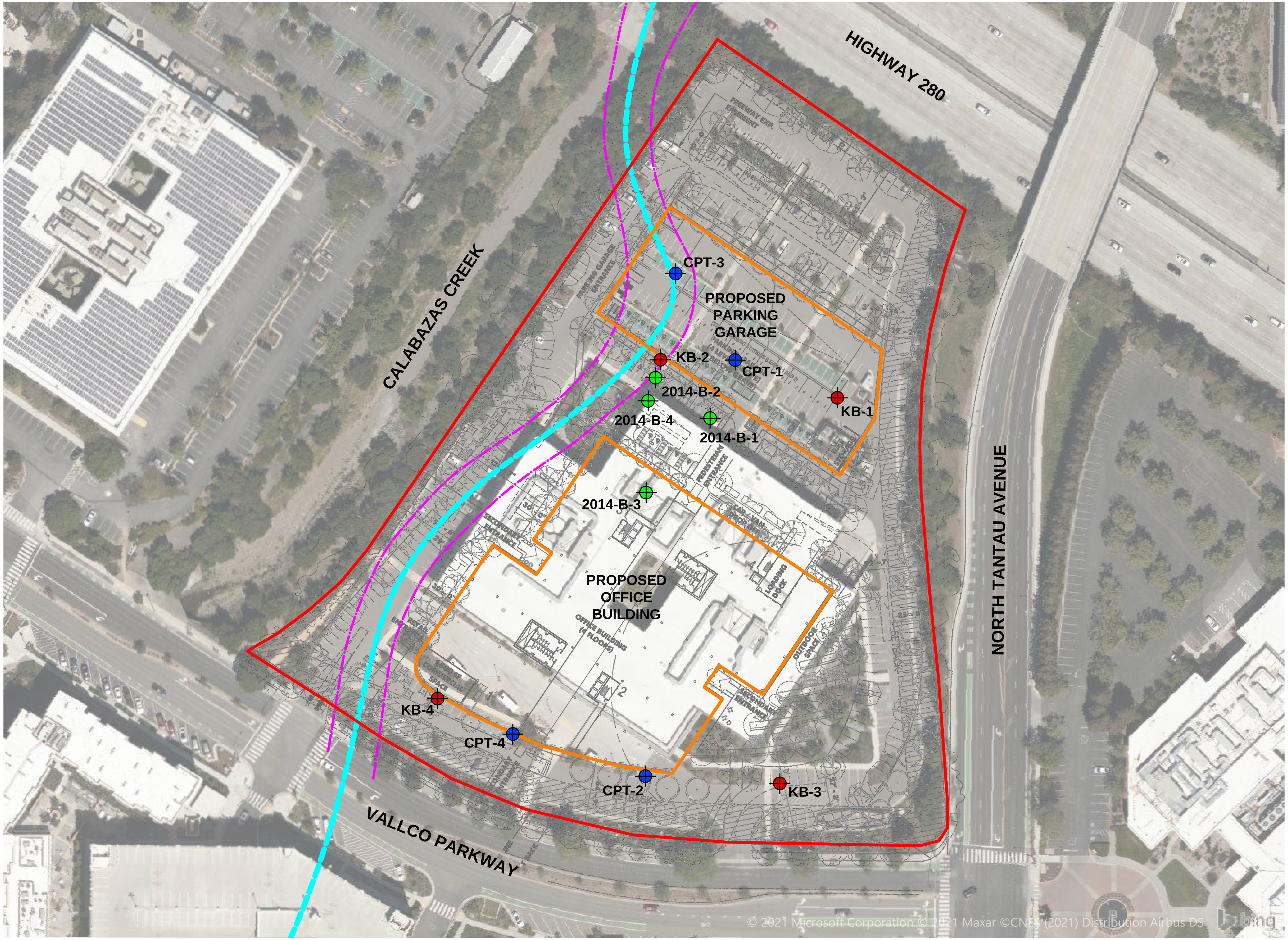

LEGEND

- BORING BY KLEINFELDER (01/26/21 TO 01/28/21,THIS STUDY)
- CPT BY KLEINFELDER (01/26/21, THIS STUDY)
- PREVIOUS BORING BY BAGG (05/21/14 & 05/31/14)
- APROXIMATE SITE BOUNDARY
- APROXIMATE CREEK ALIGNMENT
- ESTIMATED TOP OF PREVIOUS CREEK BANK

N

0 100 200

SCALE: 1" = 100' SCALE IN FEET

CAD FILE: W:\2021\20213246.001A -Apple VP01\20213246.dwg LAYOUT: F2

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NOTE:
BASEMAP FROM THE PROJECT PRELIMINARY SITE PLAN SHEET A001.1 PREPARED BY STUDIOS ARCHITECTURE AND DATED 1/30/21.

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PROJECT NO. 20213246	SITE PLAN	FIGURE
DRAWN BY JDS		2
CHECKED BY TC	APPLE VP01 19191 VALLCO PARKWAY CUPERTINO, CALIFORNIA	
DATE: March 2021		
REVISED:		

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Appendix A

SAMPLE/SAMPLER TYPE GRAPHICS

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

GROUND WATER GRAPHICS

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

NOTES

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

ABBREVIATIONS
WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

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

		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

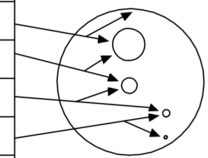
NOTE: USE MATERIAL DESCRIPTION ON THE LOG TO DEFINE A GRAPHIC THAT MAY NOT BE PROVIDED ON THIS LEGEND.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20213246.001A	GRAPHICS KEY	FIGURE
	DRAWN BY: JDS	APPLE VP01 19191 VALLCO PARKWAY CUPERTINO, CALIFORNIA	A-1
	CHECKED BY: AL		
	DATE: March 2021		

PLOTTED: 02/22/2021 11:56 PM BY: TChen
OFFICE FILTER: SAN JOSE
PROJECT NUMBER: 20213246.001A
gINT FILE: Klf_gint_master_2021
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2021.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	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	$\geq$5 to <15%	$\geq$15 to <30%
Modifier	$\geq$15%	$\geq$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 $\leq$ 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 $\leq$ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1 $\leq$ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2 $\leq$ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4 $\leq$ 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.

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PROJECT NO.: 20213246.001A		SOIL DESCRIPTION KEY	FIGURE
DRAWN BY:	JDS	APPLE VP01 19191 VALLCO PARKWAY CUPERTINO, CALIFORNIA	A-2
CHECKED BY:	AL		
DATE:	March 2021		

PLOTTED: 02/22/2021 11:57 PM BY: TChen

OFFICE FILTER: SAN JOSE

gINT FILE: Klf_gint_master_2021 PROJECT NUMBER: 20213246.001A

gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2021.GLB [_KLF_GEO-LEG 2 (SOIL DESC KEY)]

BORING LOG KB-1

Date Begin - End:	1/27/2021	**Drilling Company:**	Exploration Geoservices, Inc.		
Logged By:	A. Lin	**Drill Crew:**	John, Damion, Camillo		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Mobile B56	**Hammer Type - Drop:**	140 lb. Wire Line - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	51%
Weather:	Cloudy	**Exploration Diameter:**	8 in. O.D.	**Hammer Cal. Date:**	8/14/2020

Latitude: 37.32570°
Longitude: -122.00706°

Depth (feet)	Graphical Log	FIELD EXPLORATION: Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Asphalt Concrete (AC): about 3" thick												Hand Auger to 5'
		Aggregate Base (AB): about 6" thick												
		Lean CLAY with Sand (CL): medium plasticity, olive brown, moist, stiff, with fine- to medium-grained sand, with up to 2" gravel, some roots (FILL)	BULK											R - Value: 8
5		**Lean CLAY with Sand (CL)**: medium plasticity, olive brown, moist, very stiff to hard, with fine-grained sand	1A 1B 1C		BC=8 17 22 PP=4.5	11"		21.7	103.3					Switch to Hollow Stem Auger at 5'
			2A 2B 2C		BC=18 25 33 PP=4.5	18"		14.6	116.5					TXUU: c = 5.26 ksf
10		yellow brown	3A 3B 3C		BC=6 6 17 PP=4.5	15"								
15		**Lean CLAY (CL)**: medium plasticity, yellow brown, moist, hard	4A 4B 4C		BC=20 24 29 PP=4.5	18"								
20		**Clayey SAND with Gravel (SC)**: yellowish brown, moist, very dense, fine- to coarse-grained, clayey, with 1/2" to 1" gravel	5A 5B 5C		BC=32 33 28 PP=4.5	12"				86	33			
25		**Lean CLAY (CL)**: medium plasticity, reddish brown, moist, hard, trace fine- to medium-grained sand, trace fine gravel	6A 6B 6C		BC=12 28 30 PP=4.5	16"								

Date Begin - End: 1/27/2021	**Drilling Company:** Exploration Geoservices, Inc.		**BORING LOG KB-1**
Logged By: A. Lin	**Drill Crew:** John, Damion, Camillo		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** Mobile B56	**Hammer Type - Drop:** 140 lb. Wire Line - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger	**Hammer Efficiency:** 51%	
Weather: Cloudy	**Exploration Diameter:** 8 in. O.D.	**Hammer Cal. Date:** 8/14/2020	

Depth (feet)	Graphical Log	FIELD EXPLORATION — Latitude: 37.32570° Longitude: -122.00706° — Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	LABORATORY RESULTS — USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Lean CLAY (CL): medium plasticity, reddish brown, moist, hard, trace fine-grained sand	7		BC=7 14 23	16"								
35		stiff, no sand	8		BC=6 7 11	18"								
40		yellow brown	9		BC=7 16 26	18"								
		Gravelly CLAY with Sand (CL): low to medium plasticity, brown, moist, hard, gravelly (up to 1"), with fine- to medium-grained sand												
45		**Poorly Graded SAND with Gravel (SP)**: yellow brown, moist, dense, fine- to medium-grained, with 1/4" to 1-1/4" gravel, trace clay	10		BC=28 27 20	16"								
50		very dense, no clay	11		BC=45 40 26	18"								
55			12		BC=40 45 50/4"	18"								

Date Begin - End:	1/27/2021	**Drilling Company:**	Exploration Geoservices, Inc.		**BORING LOG KB-1**
Logged By:	A. Lin	**Drill Crew:**	John, Damion, Camillo		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Mobile B56	**Hammer Type - Drop:**	140 lb. Wire Line - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	51%
Weather:	Cloudy	**Exploration Diameter:**	8 in. O.D.	**Hammer Cal. Date:**	8/14/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.32570°
Longitude: -122.00706°

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Poorly Graded SAND with Clay and Gravel (SP-SC): yellow brown, moist, very dense, fine- to medium-grained, with clay, with up to 1" gravel	13		BC=30 30 37	18"								
65		increase in clay content	14		BC=50/6"	6"								
70		**Lean CLAY with Gravel (CL)**: medium plasticity, yellow brown, moist, hard, with fine gravel, trace fine-grained sand	15		BC=25 20 50/3"	15"								
75			16		BC=35 37 22	18"								

The boring was terminated at approximately 76.5 ft. below ground surface. The boring was backfilled with grout on January 27, 2021.

80

85

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

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PROJECT NO.: 20213246.001A		BORING LOG KB-1	FIGURE
DRAWN BY:	JDS	APPLE VP01 19191 VALLCO PARKWAY CUPERTINO, CALIFORNIA	A-3
CHECKED BY:	AL		
DATE:	March 2021		PAGE: 3 of 3

Date Begin - End:	1/28/2021	**Drilling Company:**	Exploration Geoservices, Inc.		**BORING LOG KB-2**
Logged By:	A. Lin	**Drill Crew:**	Danny, Camillo		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Mobile B61	**Hammer Type - Drop:**	140 lb. Wire Line - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	54%
Weather:	Rain	**Exploration Diameter:**	8 in. O.D.	**Hammer Cal. Date:**	8/14/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.32580°
Longitude: -122.00766°

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	Asphalt Concrete (AC): about 3" thick												Hand Auger to 5'
	Aggregate Base (AB): about 6" thick	BULK	Bulk								40	21	
	Lean CLAY with Sand (CL): medium plasticity, olive brown, moist, stiff, with fine- to medium-grained sand, with up to 2" gravel, some roots (FILL)												
5	yellow brown, hard, no gravel	1A 1B 1C		BC=25 25 28 PP=4.5	18"		14.6	120.5					Switch to Hollow Stem Auger at 5' Unconfined Compressive Strength = 2.79 ksf
		2		BC=9 14 22	18"								
10	olive gray, stiff to hard, with fine gravel	3A 3B 3C		BC=14 23 35 PP=4.5	18"		18.8 15.6	109.8 117.6					TXUU: c = 3.17 ksf
	Lean CLAY: medium plasticity, reddish brown, mosit, stiff to hard	4		BC=10 17 28	18"								
15	olive gray	5A 5B 5C		BC=12 24 48 PP=4.5	18"								
	yellow brown	6		BC=14 21 50/6"	18"								
20	**Clayey SAND with Gravel (SC)**: yellow brown, moist, very dense, fine- to coarse-grained, clayey, with 1/4" to 1" gravel	7A 7B		BC=30 50/6"	12"								
	increase in gravel, less clay	8		BC=36 30 25	16"								
25	**Lean CLAY (CL)**: medium plasticity, yellow brown, moist, hard	9		BC=14 18 36	18"								

Date Begin - End: 1/28/2021	**Drilling Company:** Exploration Geoservices, Inc.		**BORING LOG KB-2**
Logged By: A. Lin	**Drill Crew:** Danny, Camillo		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** Mobile B61	**Hammer Type - Drop:** 140 lb. Wire Line - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger	**Hammer Efficiency:** 54%	
Weather: Rain	**Exploration Diameter:** 8 in. O.D.	**Hammer Cal. Date:** 8/14/2020	

Latitude: 37.32580°
Longitude: -122.00766°

Depth (feet)	Graphical Log	FIELD EXPLORATION: Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Lean CLAY (CL): medium plasticity, yellow brown, moist, hard	10		BC=26 16 20	14"								
35		**Poorly Graded SAND with Gravel (SP)**: yellow brown to yellowish brown, moist, very dense, fine- to coarse-grained, gravelly (up to 1"), tace clay	11		BC=36 50/6"	12"								
40			12		BC=30 50/6"	14"								
45		increase in gravel content	13		BC=50/6"	6"								
50		**Clayey SAND with Gravel (SC)**: yellow brown, moist, very dense, fine- to coarse-grained, clayey, with up to 3/4" gravel	14		BC=28 50/4"	10"								
55		increase in clay content	15		BC=30 30 50/6"	18"								

The boring was terminated at approximately 56.5 ft. below ground surface. The boring was backfilled with grout on January 28, 2021.

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

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PROJECT NO.: 20213246.001A	BORING LOG KB-2	FIGURE	
DRAWN BY: JDS	APPLE VP01	A-4	
CHECKED BY: AL	19191 VALLCO PARKWAY		
DATE: March 2021	CUPERTINO, CALIFORNIA	PAGE: 2 of 2	

BORING LOG KB-3

Date Begin - End:	1/26/2021	**Drilling Company:**	Exploration Geoservices, Inc.		
Logged By:	A. Lin	**Drill Crew:**	Loren, Lyle		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Mobile B53B	**Hammer Type - Drop:**	140 lb. Wire Line - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	61%
Weather:	Clear	**Exploration Diameter:**	8 in. O.D.	**Hammer Cal. Date:**	8/14/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.32466°
Longitude: -122.00723°

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
	Asphalt Concrete (AC): about 3" thick											Hand Auger to 5'
	Aggregate Base (AB): about 6" thick											
	Clayey SAND with Gravel (CL): light brownish gray, moist, medium dense (FLL)	BULK 1										
	Silty SAND (SM): dark brown, moist, dense, fine- to medium-grained (FILL)	BULK 2										
	Fat CLAY with Sand (CH): high plasticity, dark brown, wet, soft, with fine-grained sand (FILL)	BULK 3										
5	**Lean CLAY with Sand (CL)**: medium plasticity, dark brown, moist, stiff to very stiff, with fine-grained sand	1A 1B 1C	BC=6 9 13 PP=4.0	10"		22.7	106.5					Switch to Hollow Stem Auger at 5'
		2A 2B 2C	BC=7 12 13	5"								
10		3A 3B 3C	BC=11 16 23 PP=4.5	18"								
15	**Clayey SAND (SC)**: yellowish brown, moist, medium dense, fine-grained, clayey	4A 4B 4C	BC=8 11 13	12"								
20	**Lean CLAY (CL)**: medium plasticity, yellowish brown, moist, very stiff	5A 5B 5C	BC=13 26 30 PP=4.5	18"		18.2	112.6					Unconfined Compressive Strength = 6.70 ksf
25	**Clayey SAND with Gravel (SC)**: reddish brown, moist, dense, fine- to medium-grained, clayey, with up to 1/2" gravel	6A 6B 6C	BC=28 17 27	18"								

Date Begin - End: 1/26/2021	**Drilling Company:** Exploration Geoservices, Inc.		**BORING LOG KB-3**
Logged By: A. Lin	**Drill Crew:** Loren, Lyle		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** Mobile B53B	**Hammer Type - Drop:** 140 lb. Wire Line - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger	**Hammer Efficiency:** 61%	
Weather: Clear	**Exploration Diameter:** 8 in. O.D.	**Hammer Cal. Date:** 8/14/2020	

Depth (feet)	Graphical Log	FIELD EXPLORATION Latitude: 37.32466° Longitude: -122.00723° Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	LABORATORY RESULTS USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Lean CLAY with Sand (CL): low to medium plasticity, yellowish brown, moist, very stiff, wih fine-grained sand	7		BC=9 8 12	18"								
35		**Lean CLAY (CL)**: medium plasticity, yellowish brown, moist, very stiff to hard	8A 8B 8C		BC=11 18 29 PP=4.5	15"		20.2	109.7					Unconfined Compressive Strength = 7.82 ksf
40		**Poorly Graded SAND with Clay and Gravel (SP-SC)**: yellow brown, moist, dense to very dense, fine- to coarse-grained, with clay, with fine gravel	9		BC=15 17 20	18"								
45			10		BC=50/5"	4"								
50		very dense, trace clay	11		BC=15 30 31	18"								

55

The boring was terminated at approximately 51.5 ft. below ground surface. The boring was backfilled with grout on January 26, 2021.

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

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20213246.001A DRAWN BY: JDS CHECKED BY: AL DATE: March 2021	BORING LOG KB-3 APPLE VP01 19191 VALLCO PARKWAY CUPERTINO, CALIFORNIA	FIGURE A-5 PAGE: 2 of 2

Date Begin - End: 1/26/2021	**Drilling Company:** Exploration Geoservices, Inc.		**BORING LOG KB-4**
Logged By: A. Lin	**Drill Crew:** Loren, Lyle		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** Mobile B53B	**Hammer Type - Drop:** 140 lb. Wire Line - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger	**Hammer Efficiency:** 61%	
Weather: Cloudy	**Exploration Diameter:** 8 in. O.D.	**Hammer Cal. Date:** 8/14/2020	

Latitude: 37.32487°
Longitude: -122.00840°

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
		FIELD EXPLORATION					LABORATORY RESULTS							
0		Asphalt Concrete (AC): about 3" thick												Hand Auger to 5'
		Aggregate Base (AB): about 6" thick												
		Sandy Lean CLAY (CL): medium plasticity, olive brown, moist, stiff, sandy (fine- to medium-grained)	BULK									42	21	
5		yellowish brown, hard	1A 1B 1C		BC=5 14 16 PP=4.5	10"								Switch to Hollow Stem Auger at 5'
		Lean CLAY (CL): medium plasticity, olive brown, moist, hard, trace fine-grained sand	2A 2B 2C		BC=10 19 26 PP=4.5	8"		19.4	110.0					Unconfined Compressive Strength = 8.56 ksf
10			3A 3B 3C		BC=18 28 33 PP=4.5	8"		20.1	109.7					
15			4A 4B 4C		BC=14 18 29 PP=4.5	18"		17.0	113.4					TXUU: c = 4.35 ksf
		Sandy LeanCLAY (CL): medium plasticity, yellowish brown, moist, hard, sandy (fine- to medium-grained)												
20			5A 5B 5C		BC=18 22 28	17"								
		Gravelly CLAY with Sand (CL): low to medium plasticity, yellow brown, moist, hard, gravelly (fine), with fine- to medium-grained sand												
25		**Well-Graded SAND with Clay and Gravel (SW-SC)**: yellowish brown, moist, dense, fine- to coarse-grained, gravelly (1/4" to 3/4"), with clay	6		BC=16 18 21	18"				62	7.6			

BORING LOG KB-4

Date Begin - End:	1/26/2021	**Drilling Company:**	Exploration Geoservices, Inc.		
Logged By:	A. Lin	**Drill Crew:**	Loren, Lyle		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Mobile B53B	**Hammer Type - Drop:**	140 lb. Wire Line - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	61%
Weather:	Cloudy	**Exploration Diameter:**	8 in. O.D.	**Hammer Cal. Date:**	8/14/2020

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.32487°
Longitude: -122.00840°

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
		Well-Graded SAND with Clay and Gravel (SW-SC): yellowish brown, moist, dense, fine- to coarse-grained, gravelly (1/4" to 3/4"), with clay dry, very dense	7		BC=27 36 50/6"	18"								
35			8		BC=27 33 16	18"								
40		very dense	9		BC=22 25 27	18"								
45			10		BC=22 26 30	14"								
50			11		BC=30 28 33	18"								
55														

The boring was terminated at approximately 51.5 ft. below ground surface. The boring was backfilled with grout on January 26, 2021.

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

KLEINFELDER
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PROJECT NO.: 20213246.001A		BORING LOG KB-4	FIGURE
DRAWN BY:	JDS	APPLE VP01 19191 VALLCO PARKWAY CUPERTINO, CALIFORNIA	A-6
CHECKED BY:	AL		
DATE:	March 2021		

PRESENTATION OF SITE INVESTIGATION RESULTS

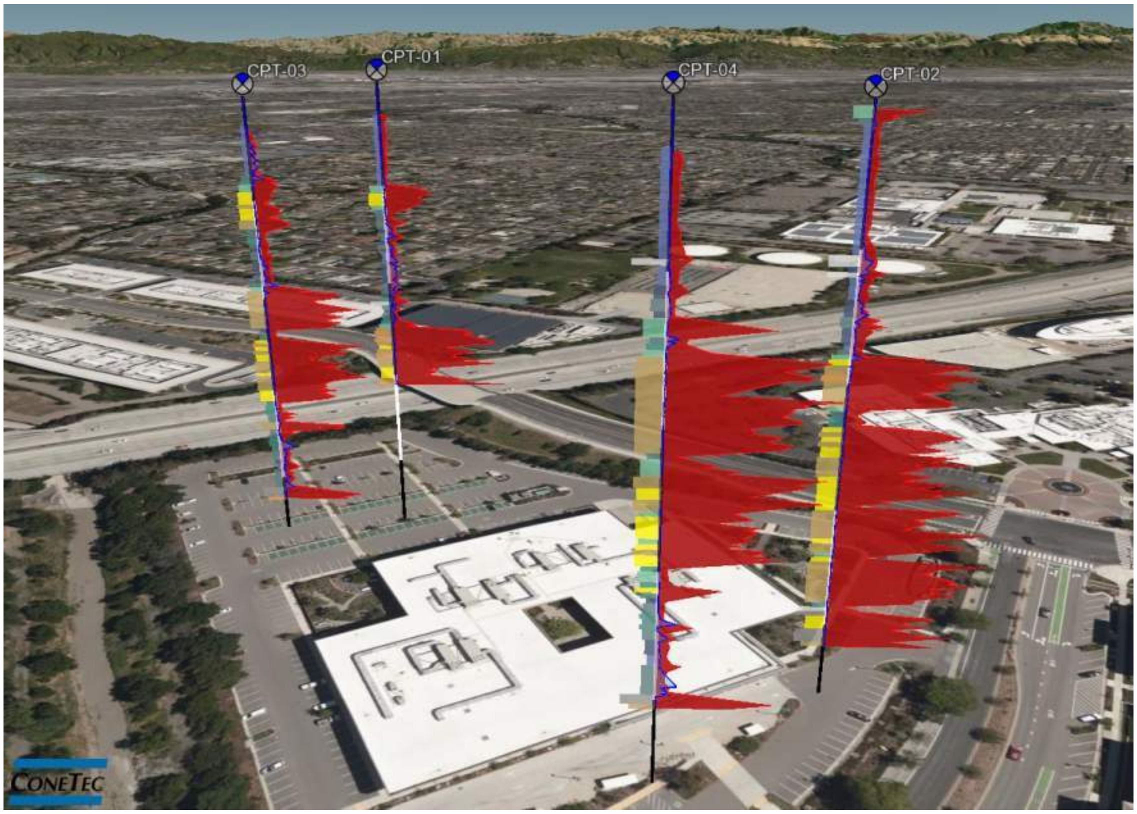
VP-01

Prepared for:

Kleinfelder

ConeTec Job No: 21-56-21869

Project Start Date: 26-Jan-2021
Project End Date: 26-Jan-2021
Report Date: 27-Jan-2021



Prepared by:

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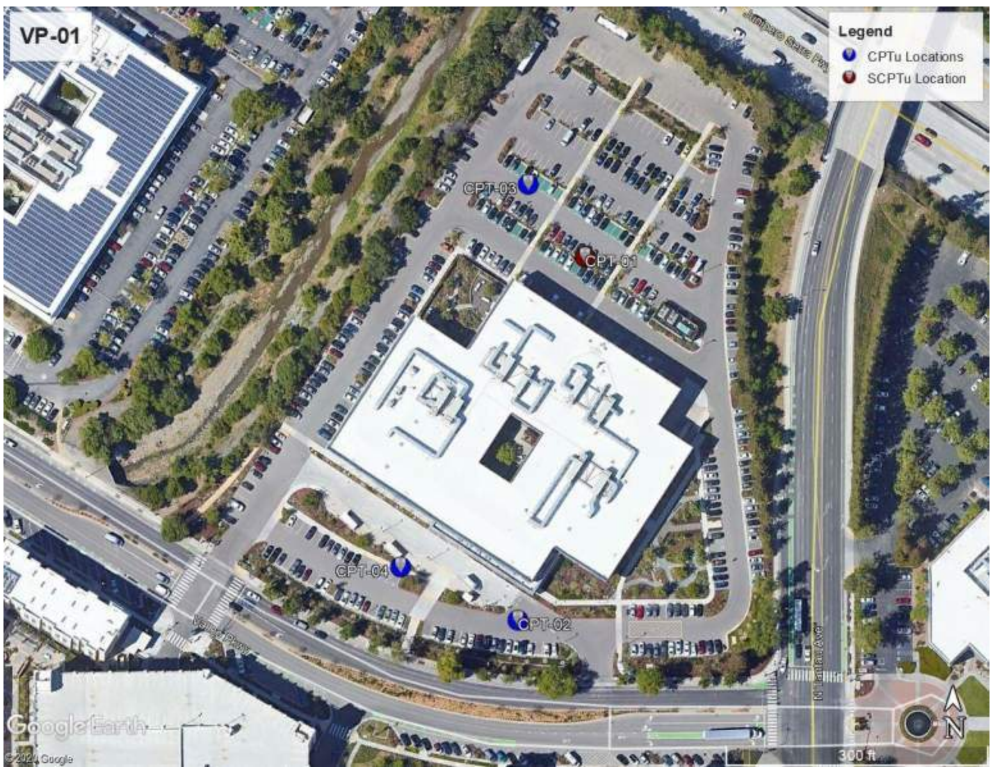
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. for Kleinfelder of Pleasanton, California. The program consisted of cone penetration testing (CPTu) at four (4) locations. Shear wave velocities were recorded in one (1) sounding. The assumed phreatic surface used for the calculated parameters is based on the shallowest pore pressure dissipation test to reach equilibrium within the or nearest to each sounding.

Project Information

Project	
Client	Kleinfelder
Project	VP-01
ConeTec Project #	21-56-21869

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



Rig Description	Deployment System	Test Type
CPT truck rig (C15)	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)
483:T1500F15U500	483	15	225	1500	15	500
Cone 483 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, Normalized, Seismic plots, Seismic results table, Seismic wave traces, and Soil Behavior Type (SBT) scatter plots are included in the data release package.
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 “VP-01”. 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.

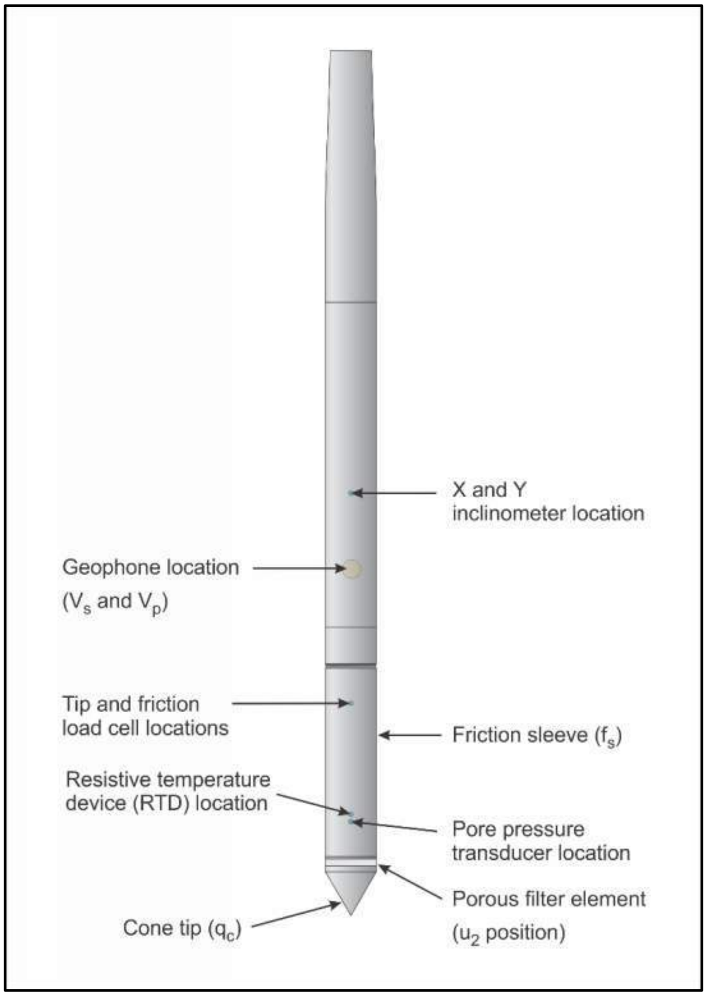


Figure CPTu. Piezocone Penetrometer (15 cm^2)

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.

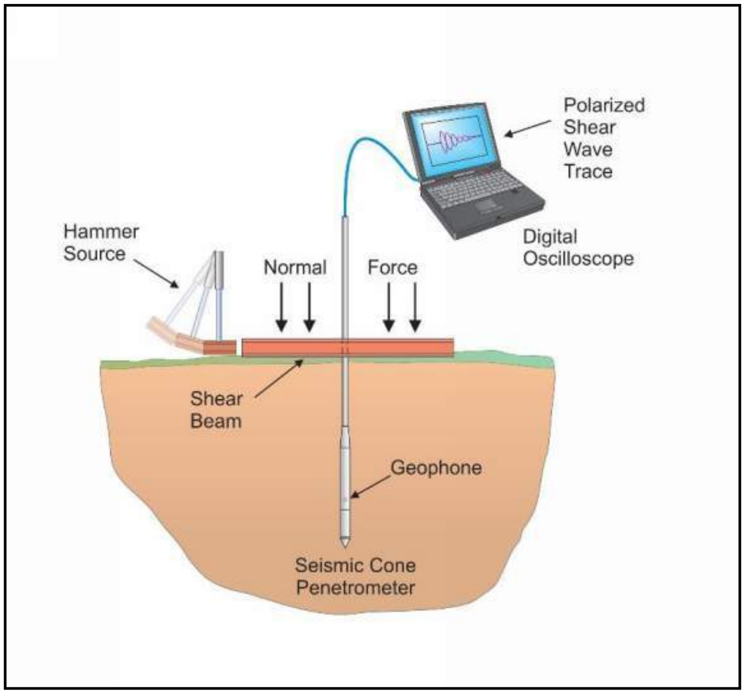


Figure SCPTu-1. Illustration of the SCPTu system

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).

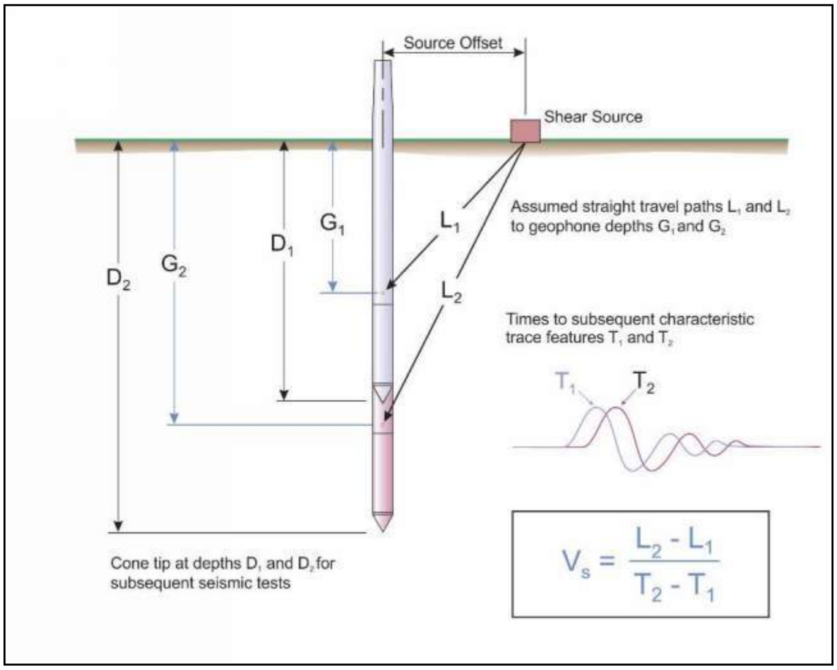


Figure SCPTu-2. Illustration of a seismic cone penetration test

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).

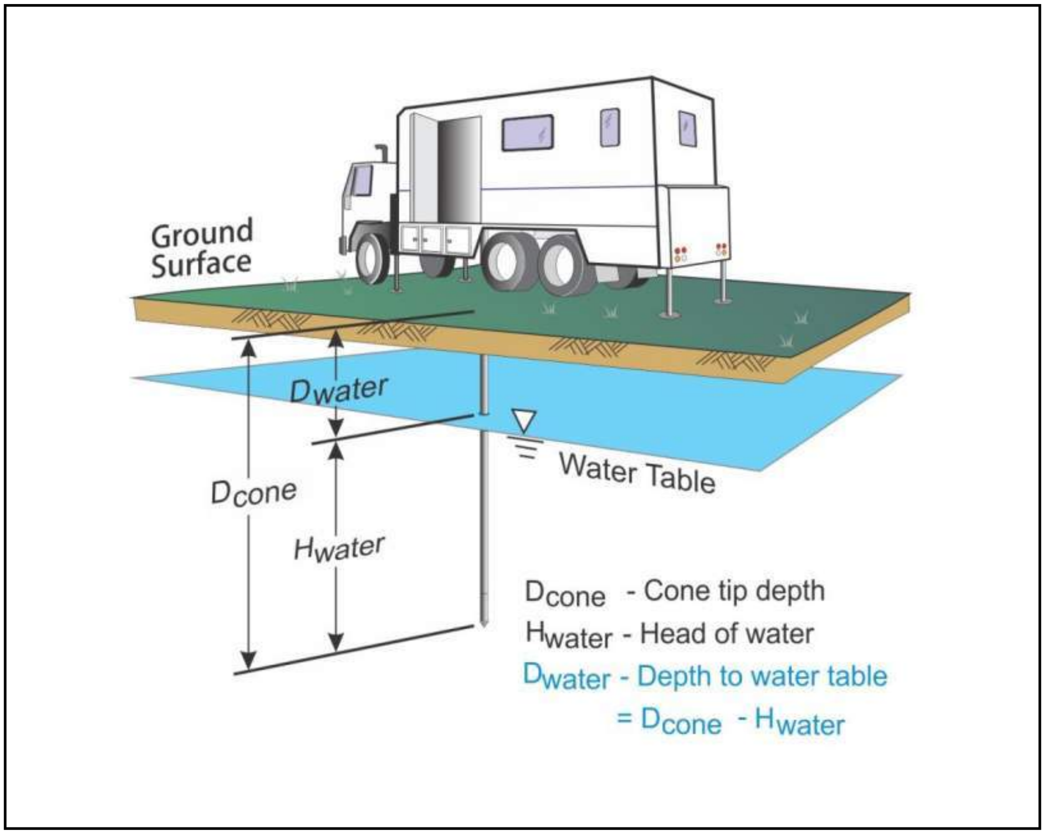


Figure PPD-1. Pore pressure dissipation test setup

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.

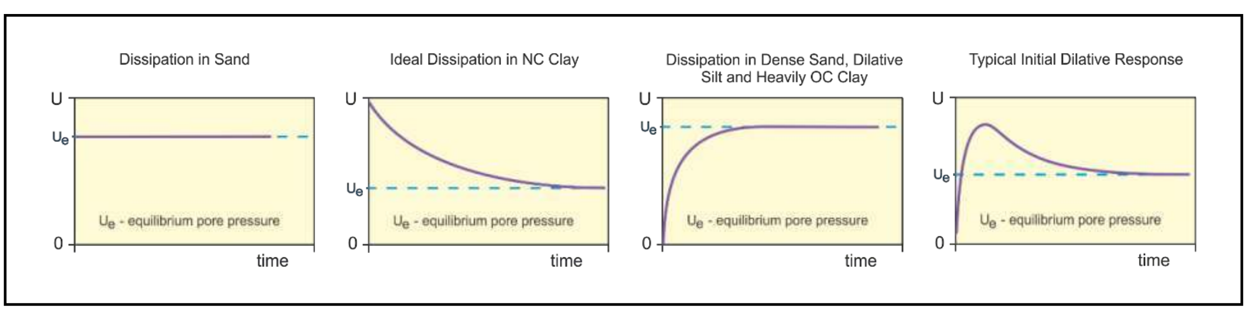


Figure PPD-2. Pore pressure dissipation curve examples

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.

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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.

APPENDICES

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 with Phi, Su(Nkt), and N1(60)Ic
- Normalized 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:	21-56-21869
Client:	Kleinfelder
Project:	VP-01
Start Date:	26-Jan-2021
End Date:	26-Jan-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
CPT-01	21-56-21869_SP01	26-Jan-2021	483:T1500F15U500	>60.0	50.93	4131475	587940	180	
CPT-02	21-56-21869_CP02	26-Jan-2021	483:T1500F15U500	>68.9	68.90	4131349	587918	179	
CPT-03	21-56-21869_CP03	26-Jan-2021	483:T1500F15U500	>69.7	69.63	4131500	587920	178	
CPT-04	21-56-21869_CP04	26-Jan-2021	483:T1500F15U500	>66.7	66.68	4131367	587877	180	4

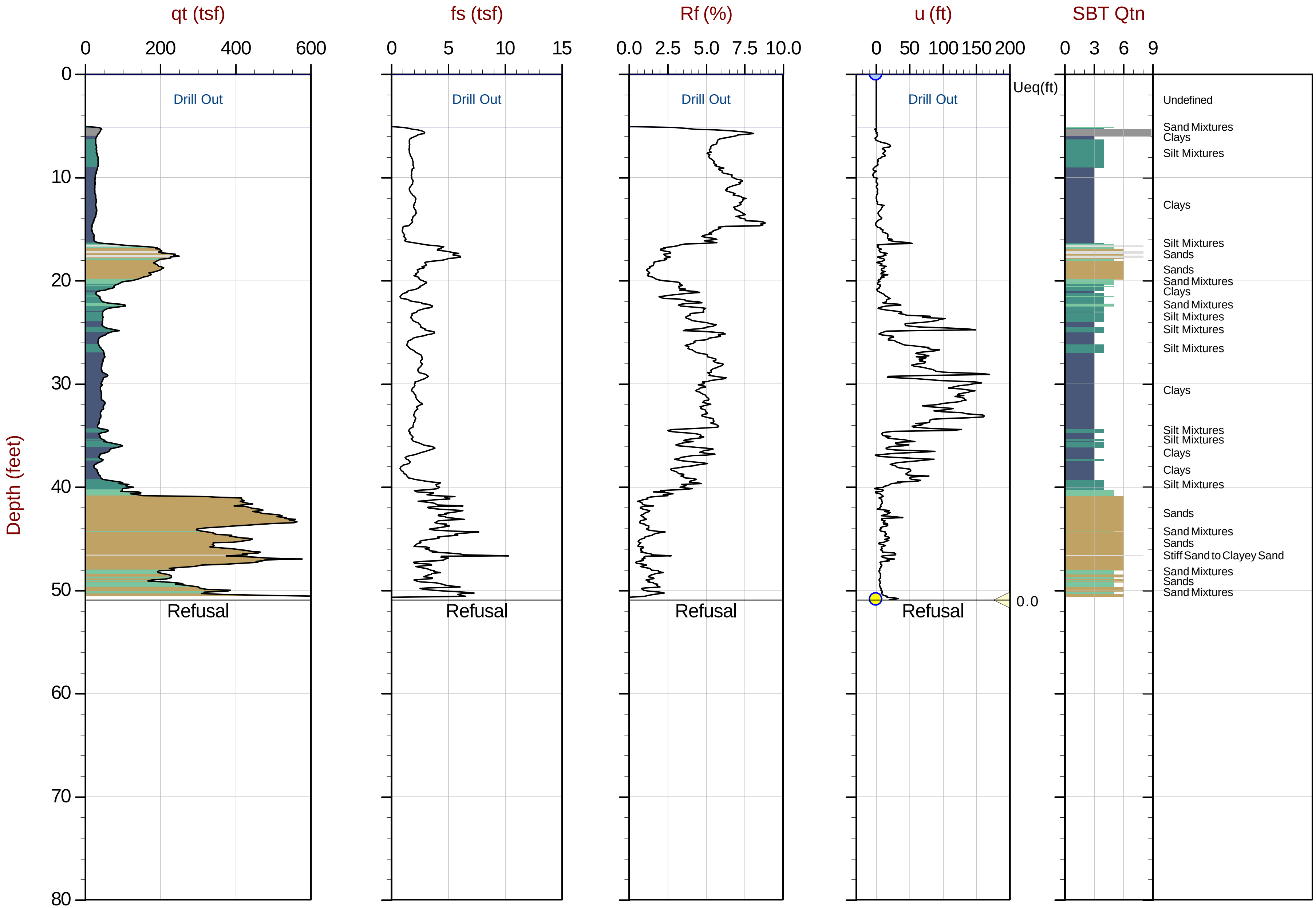
1. The assumed phreatic surface is based on the pore pressure dissipation test performed within or nearest to the sounding. The sounding assumed to be dry for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 10S.
3. Elevations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

CONETEC

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 08:03
Site: VP-01

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 15.525 m / 50.93 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131475m E: 587940m

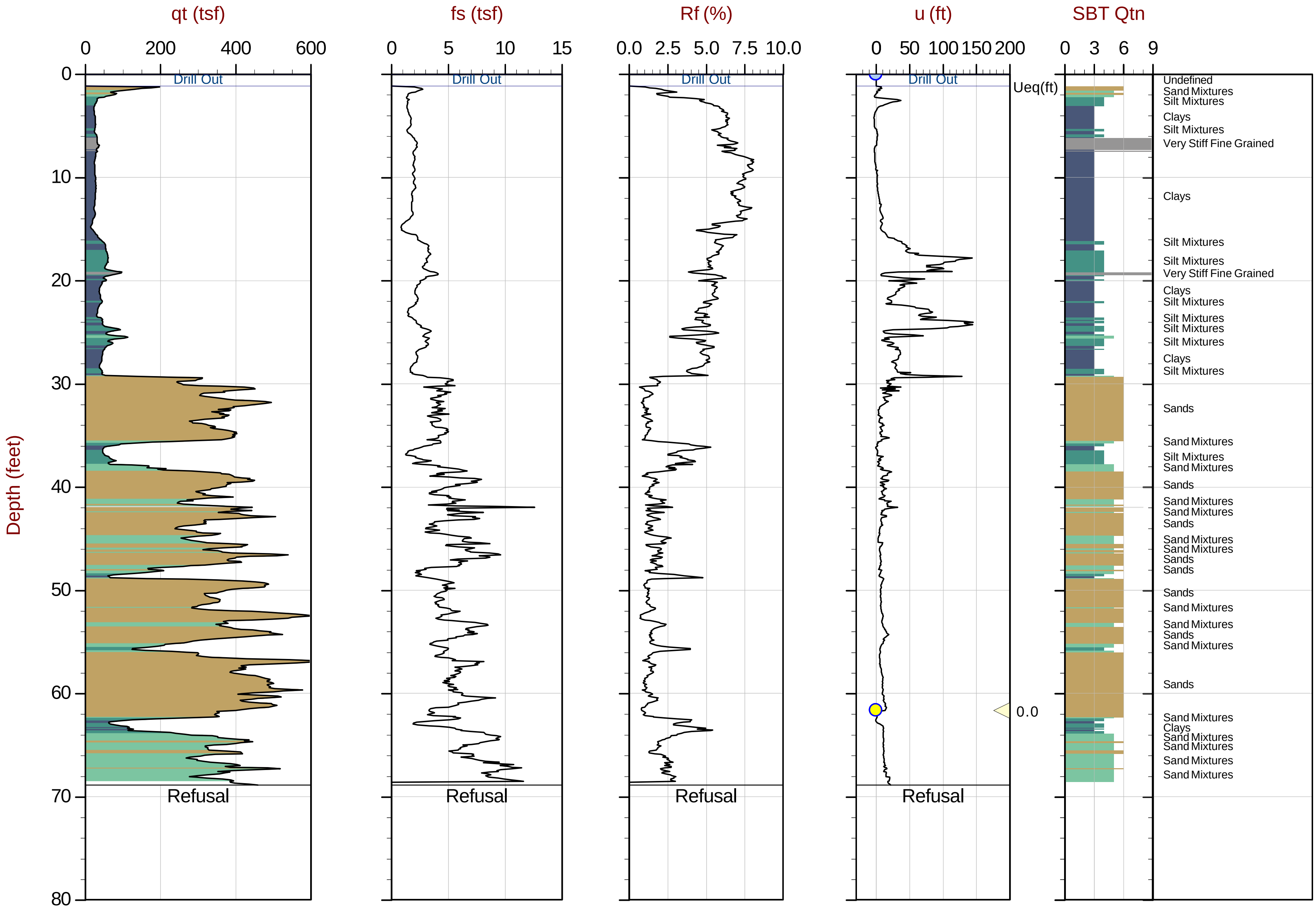
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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: 21-56-21869
Date: 2021-01-26 10:48
Site: VP-01

Sounding: CPT-02
Cone: 483:T1500F15U500



Max Depth: 21.000 m / 68.90 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131349m E: 587918m

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

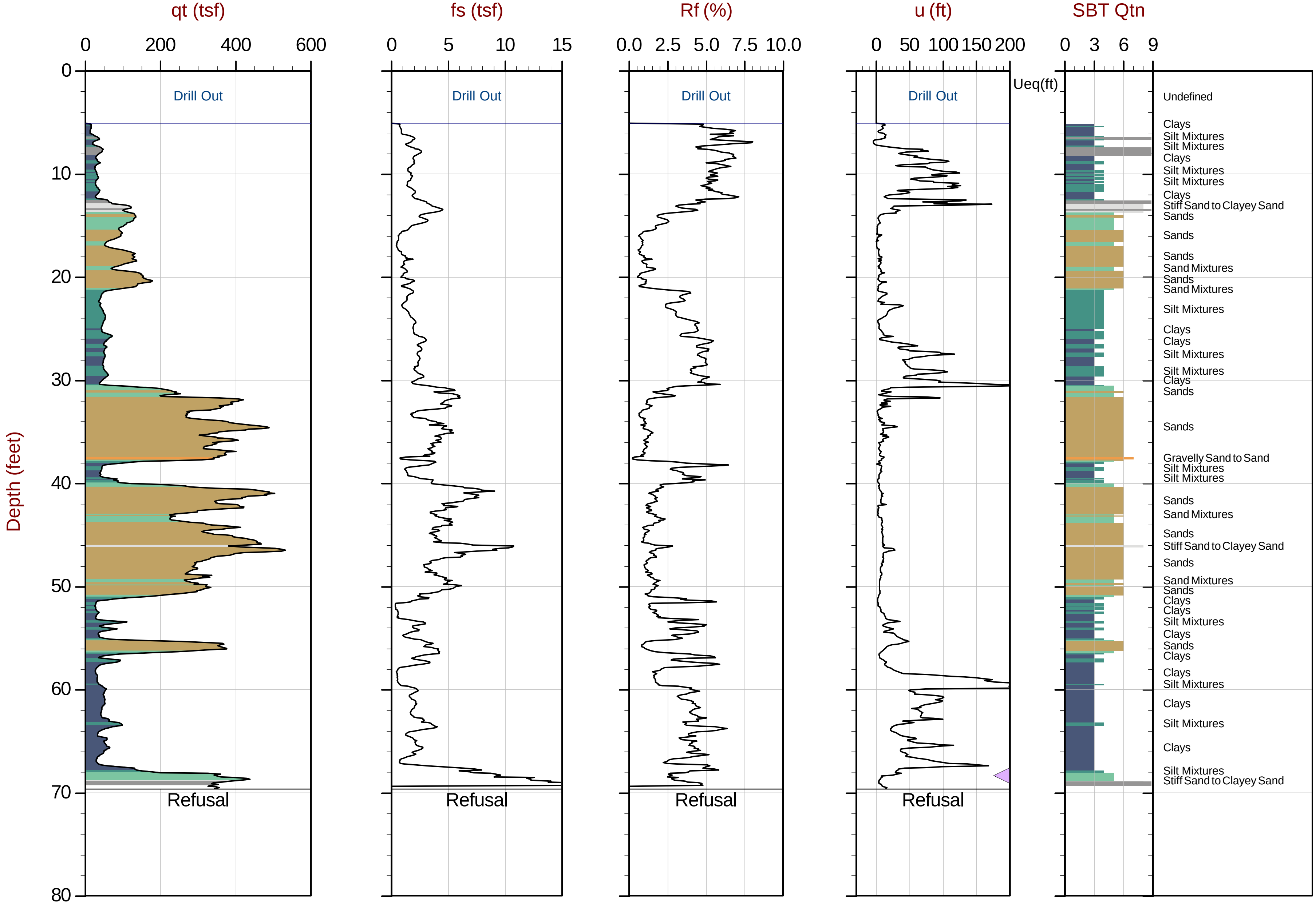
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 09:20
Site: VP-01

Sounding: CPT-03
Cone: 483:T1500F15U500



Max Depth: 21.225 m / 69.63 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131500m E: 587920m

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

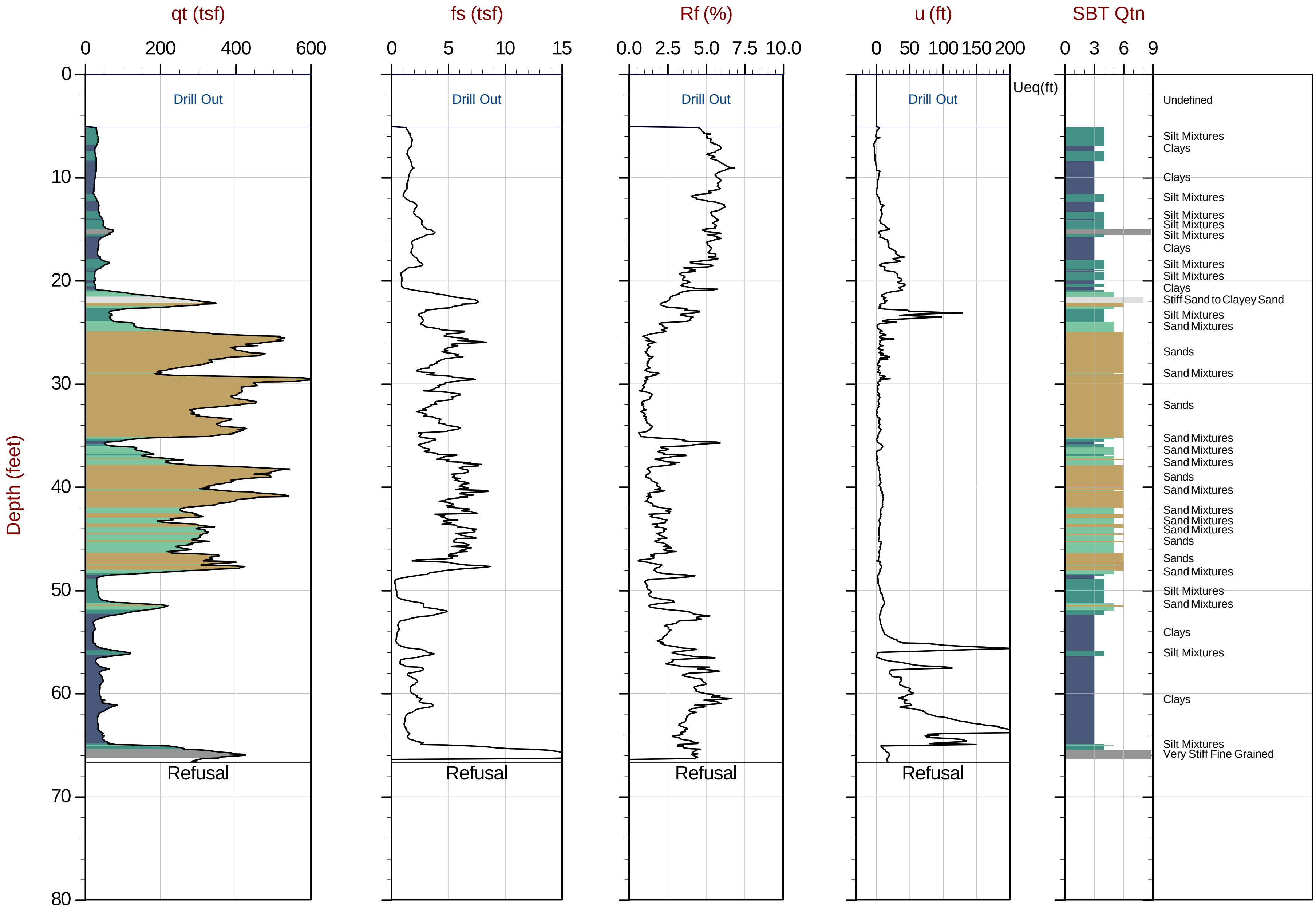
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 12:32
Site: VP-01

Sounding: CPT-04
Cone: 483:T1500F15U500



Max Depth: 20.325 m / 66.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131367m E: 587877m

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

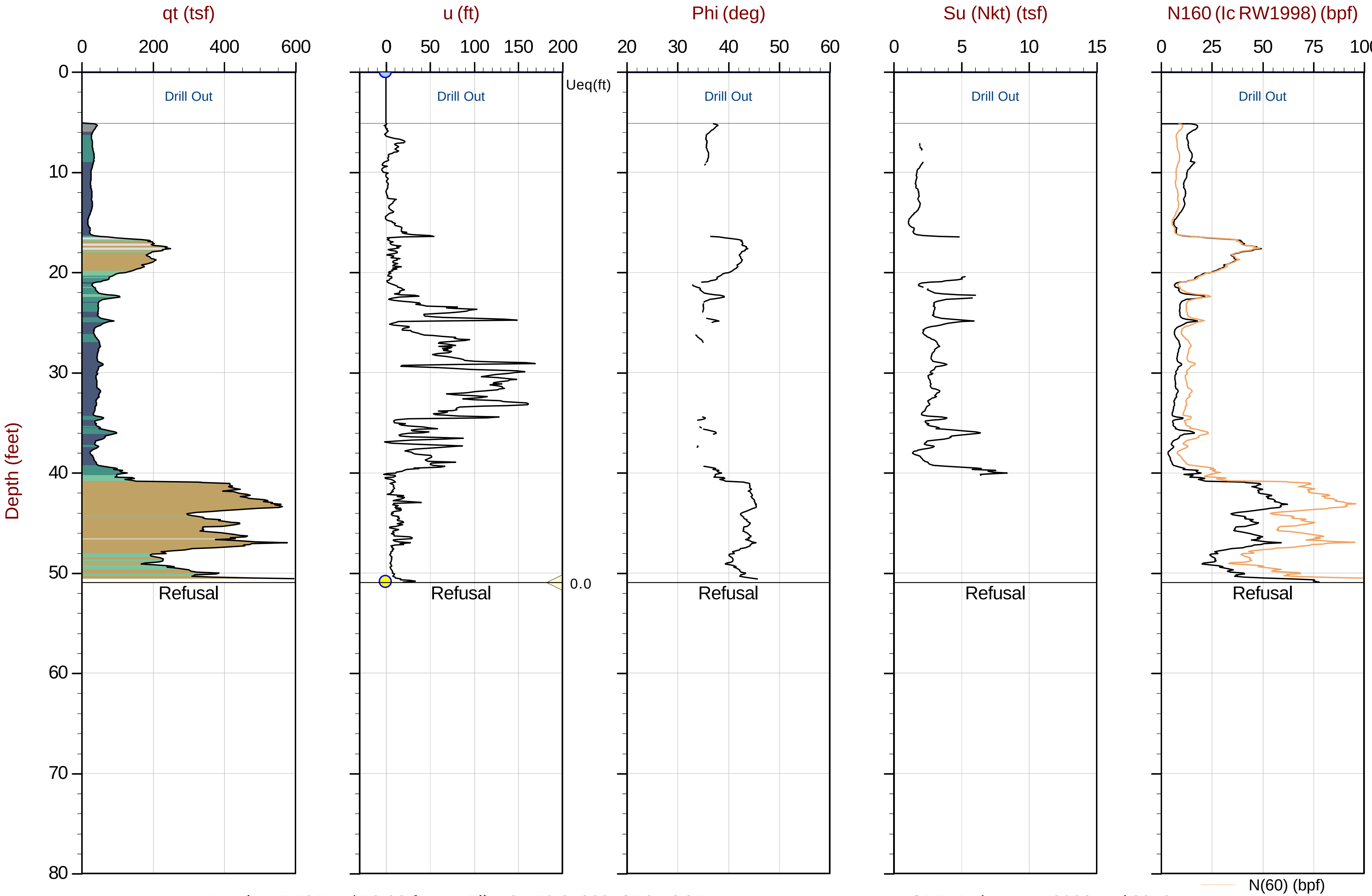
CONETEC

ConeTec

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 08:03
Site: VP-01

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 15.525 m / 50.93 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131475m E: 587940m

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

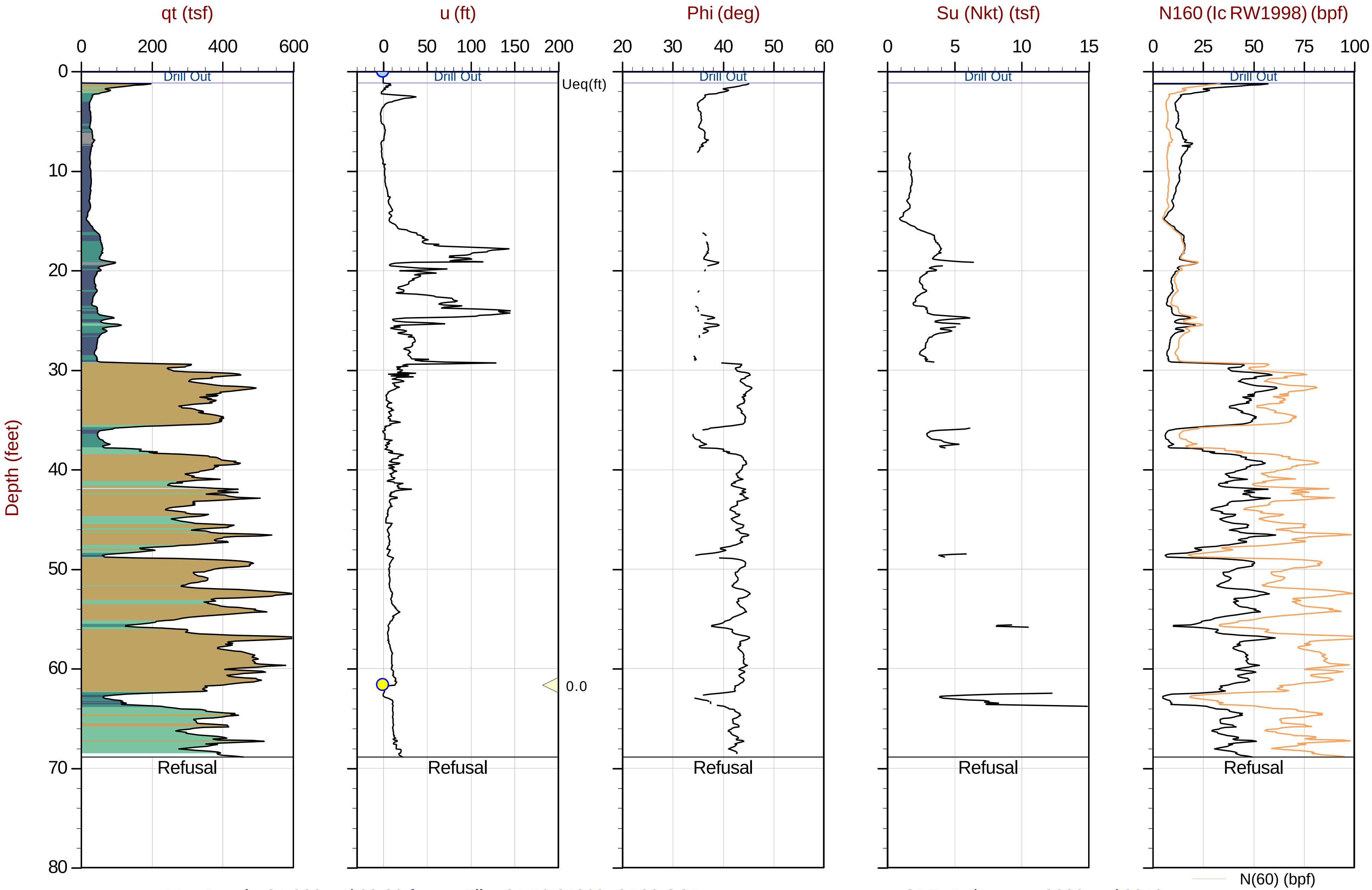
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 10:48
Site: VP-01

Sounding: CPT-02
Cone: 483:T1500F15U500



Max Depth: 21.000 m / 68.90 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131349m E: 587918m

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

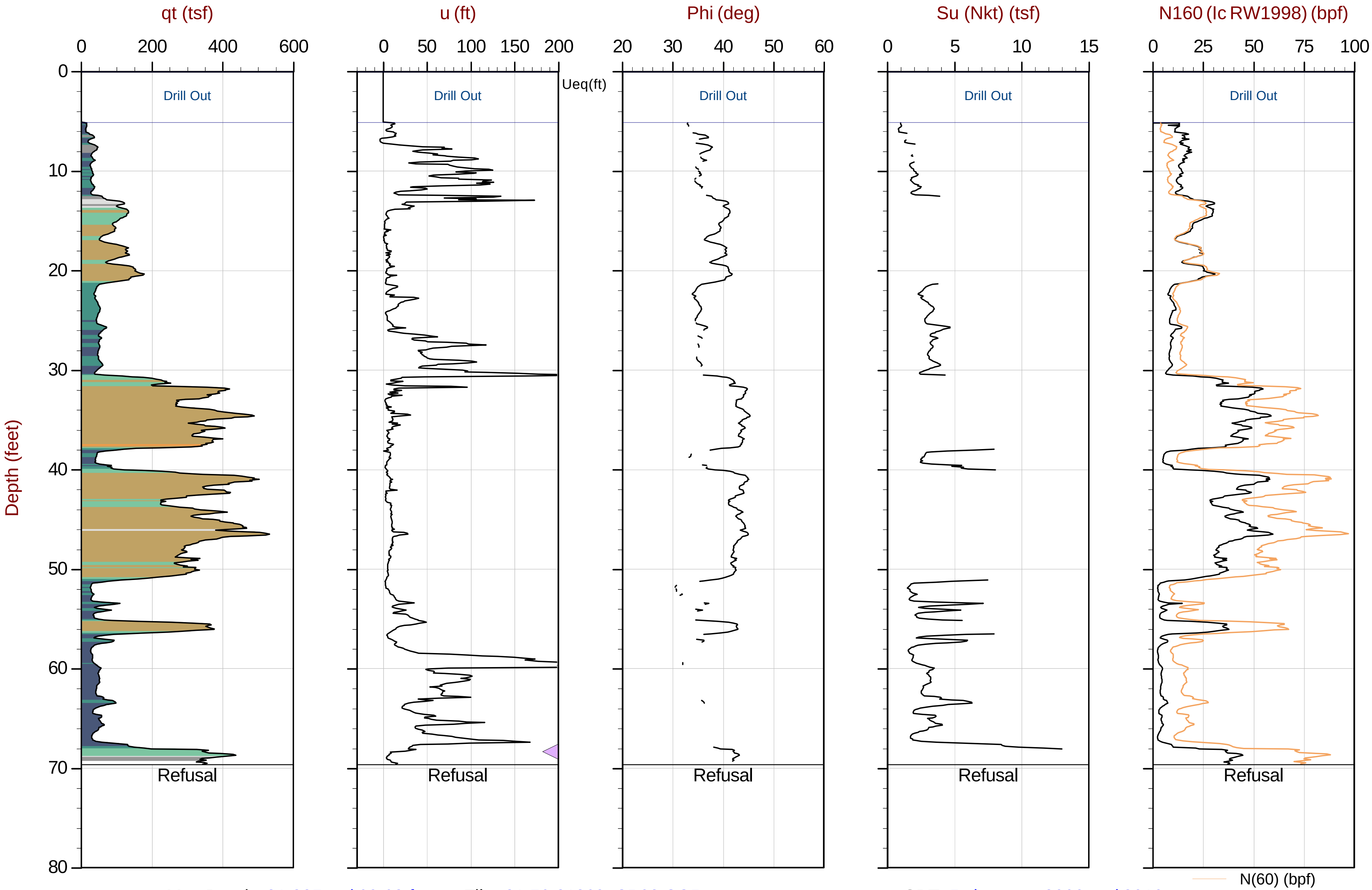
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 09:20
Site: VP-01

Sounding: CPT-03
Cone: 483:T1500F15U500



Max Depth: 21.225 m / 69.63 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131500m E: 587920m

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

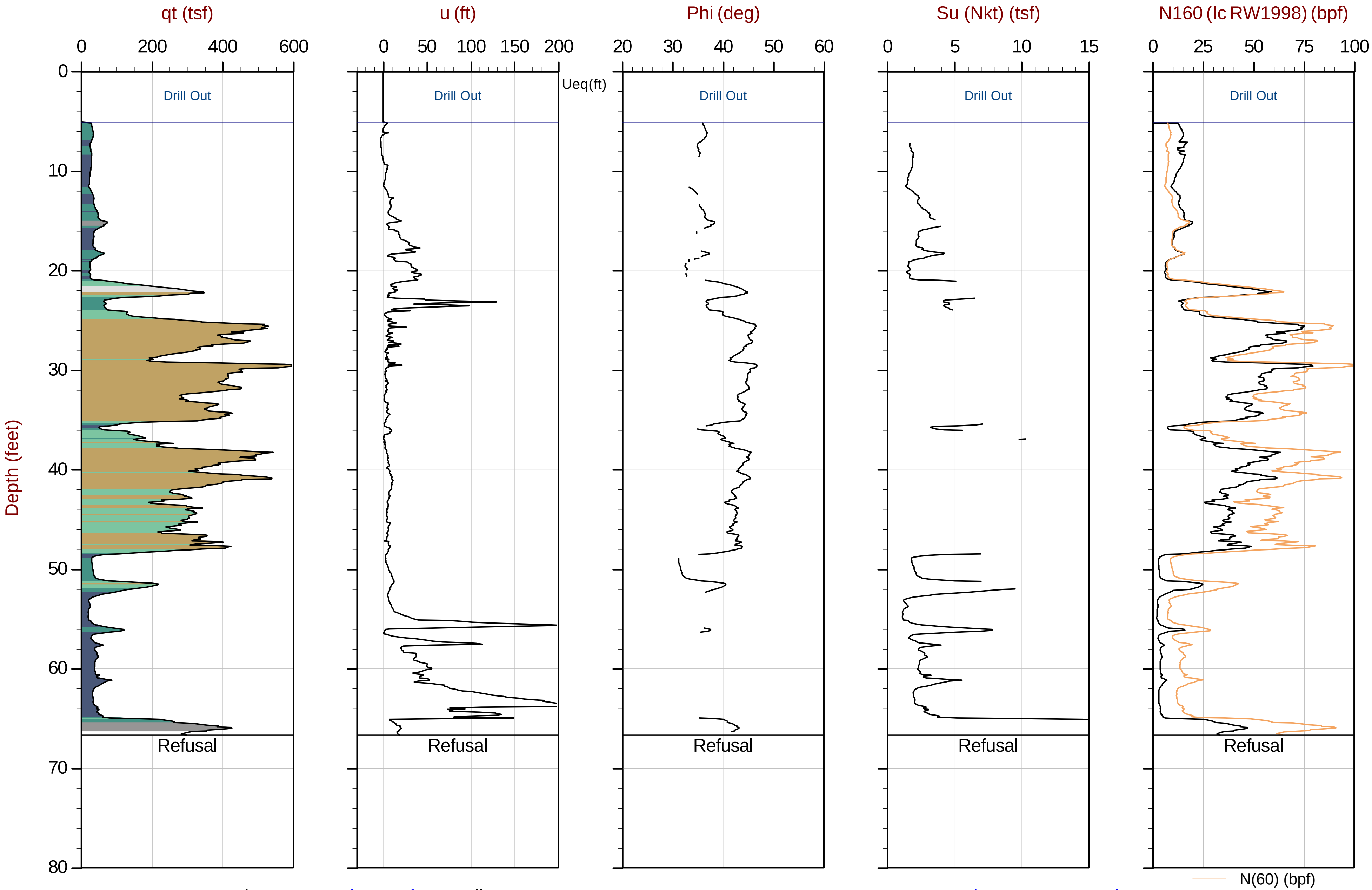
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 12:32
Site: VP-01

Sounding: CPT-04
Cone: 483:T1500F15U500



Max Depth: 20.325 m / 66.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131367m E: 587877m

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.

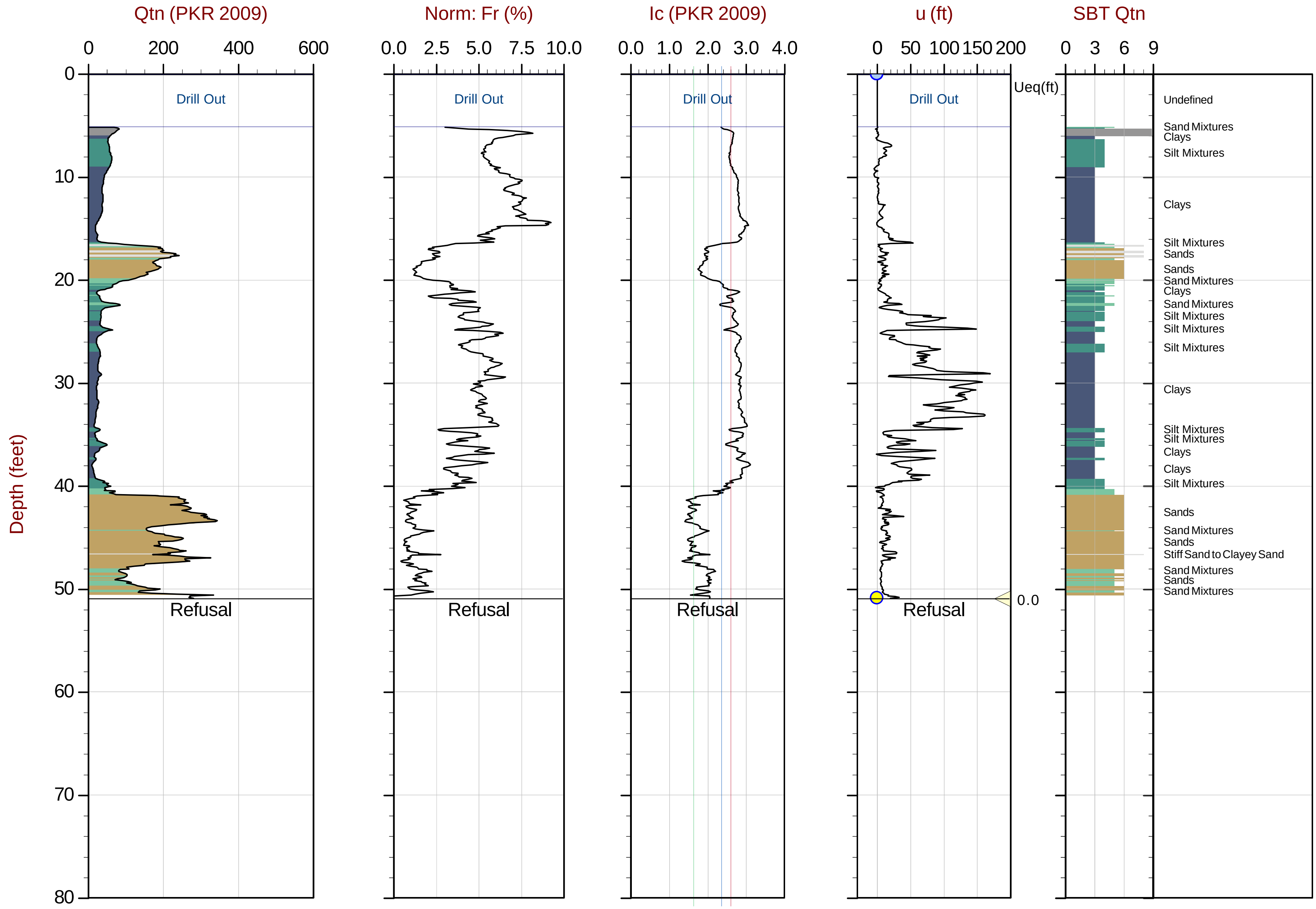
Normalized Cone Penetration Test Plots

ConeTec

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 08:03
Site: VP-01

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 15.525 m / 50.93 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131475m E: 587940m

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

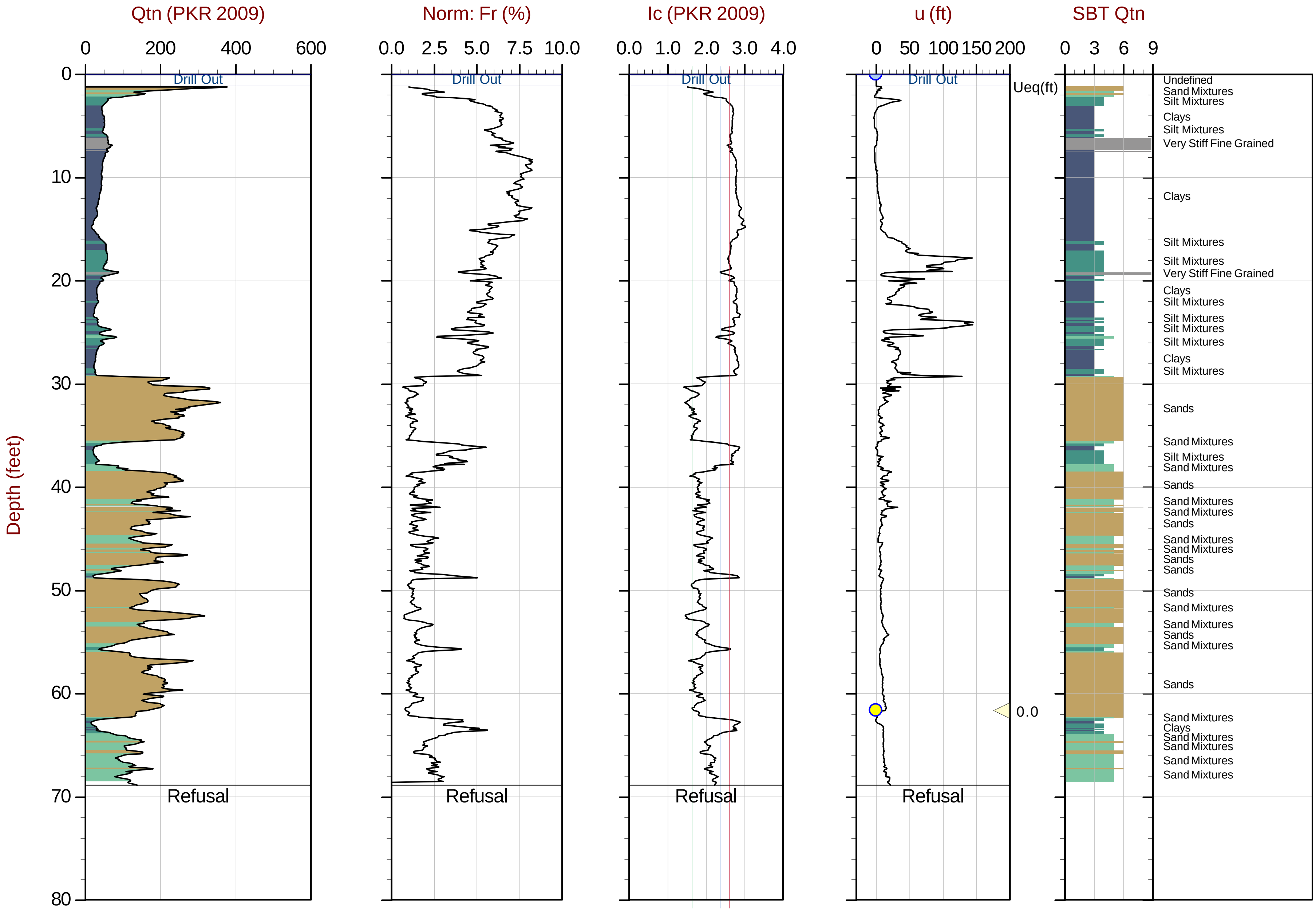
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 10:48
Site: VP-01

Sounding: CPT-02
Cone: 483:T1500F15U500



Max Depth: 21.000 m / 68.90 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131349m E: 587918m

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

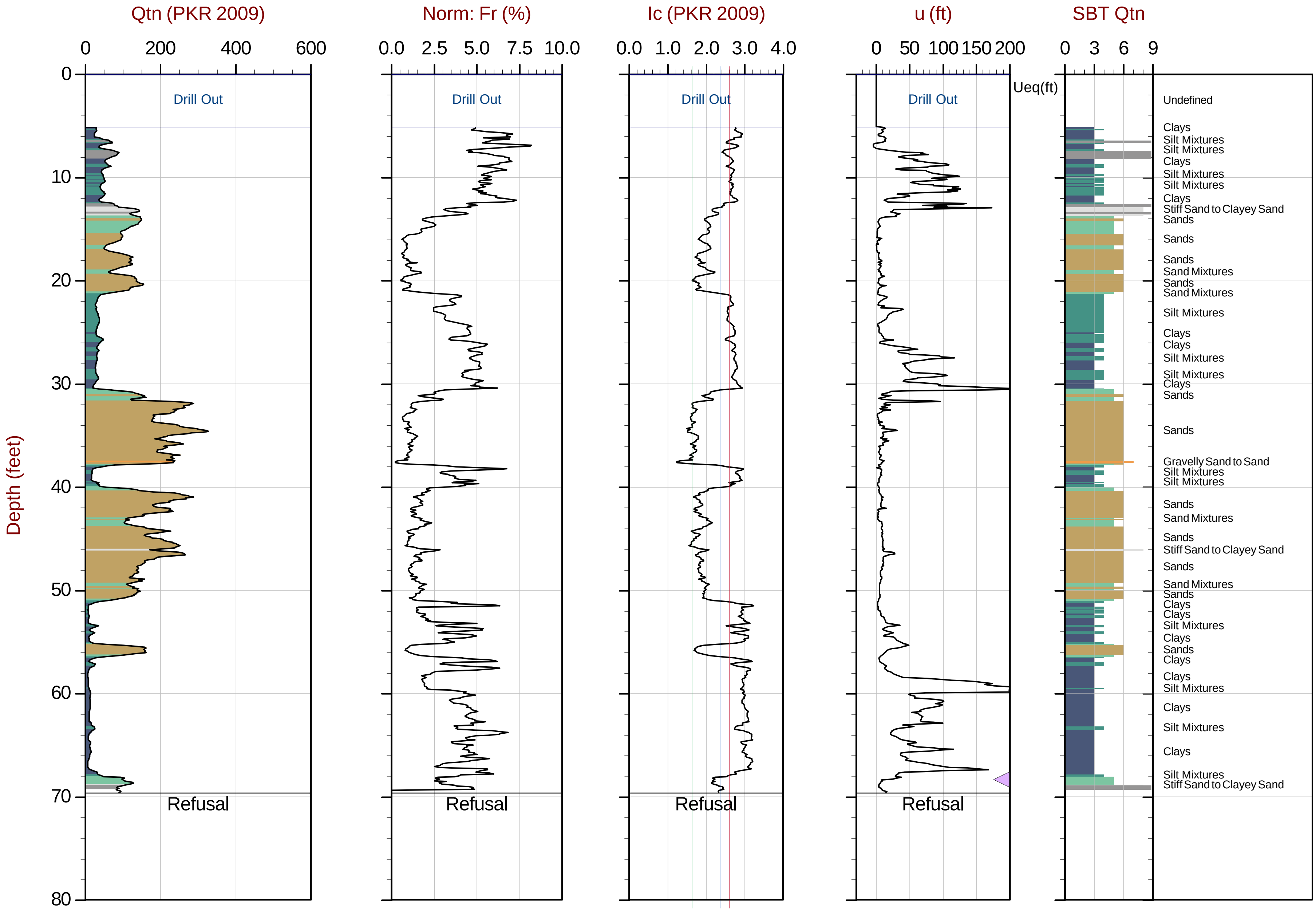
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 09:20
Site: VP-01

Sounding: CPT-03
Cone: 483:T1500F15U500



Max Depth: 21.225 m / 69.63 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131500m E: 587920m

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

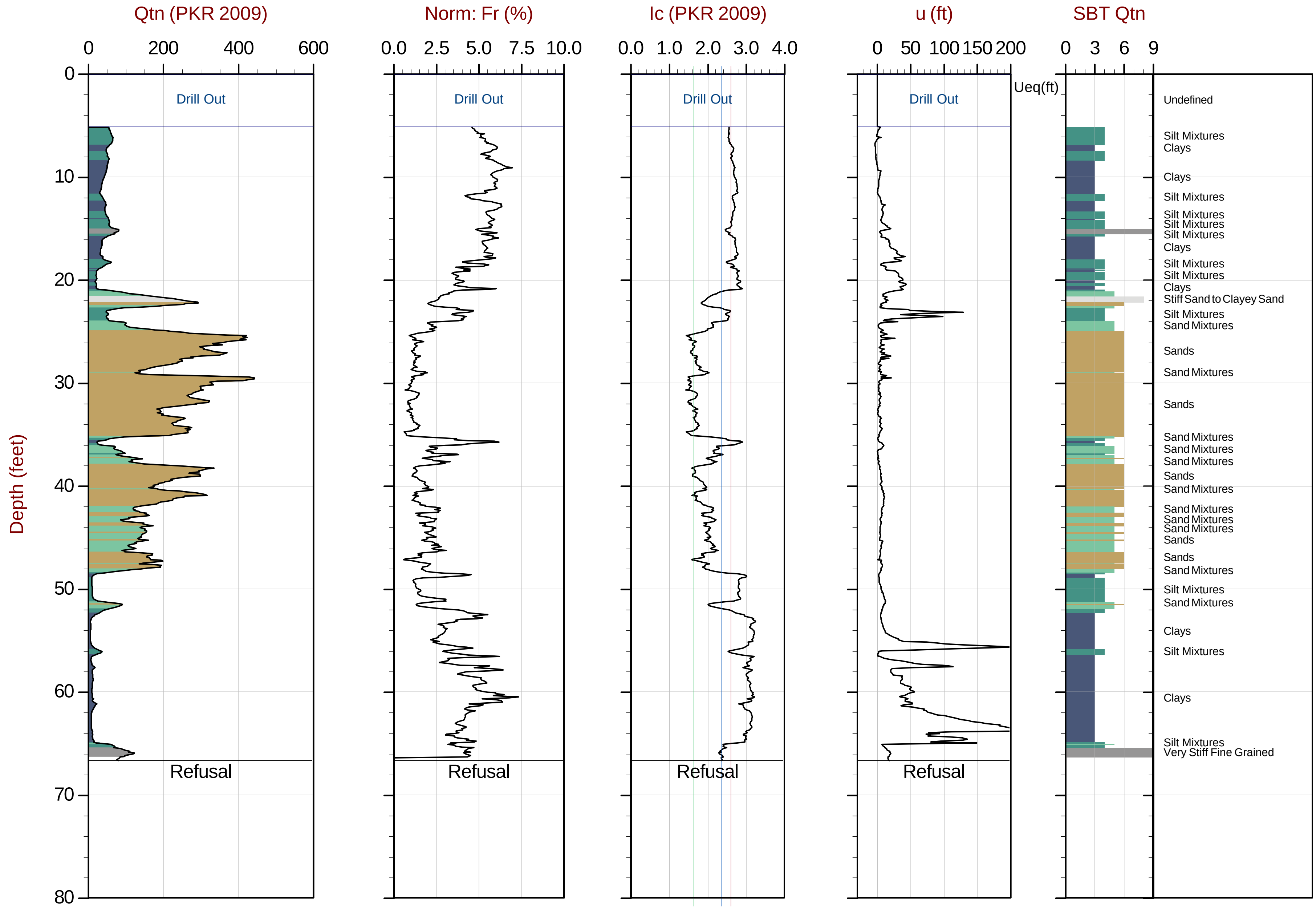
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 12:32
Site: VP-01

Sounding: CPT-04
Cone: 483:T1500F15U500



Max Depth: 20.325 m / 66.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131367m E: 587877m

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.

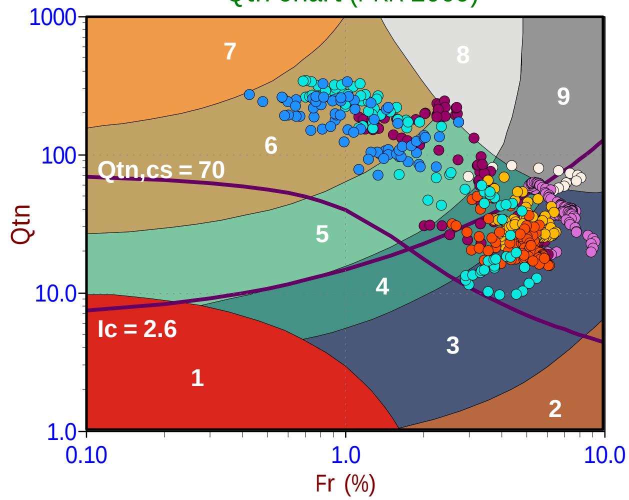
Soil Behavior Type (SBT) Scatter Plots

Kleinfelder

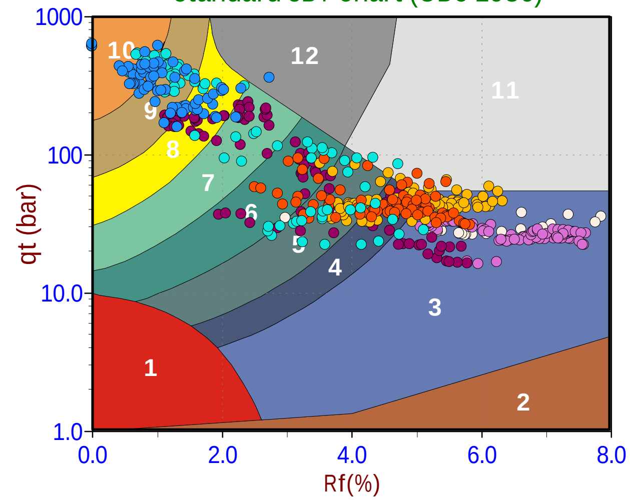
Job No: 21-56-21869
Date: 2021-01-26 08:03
Site: VP-01

Sounding: CPT-01
Cone: 483:T1500F15U500

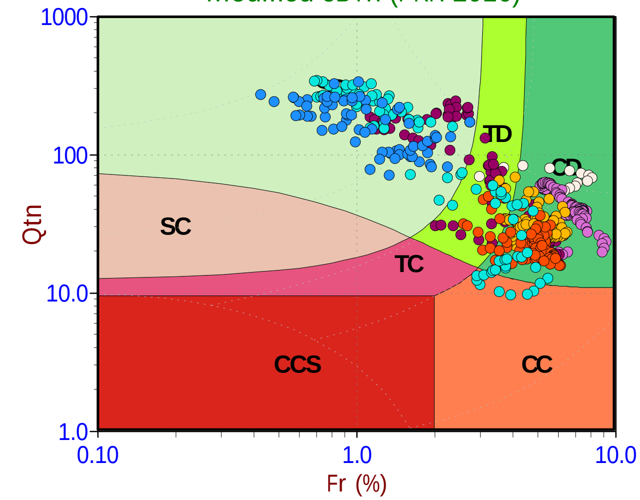
Qtn Chart (PKR 2009)



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

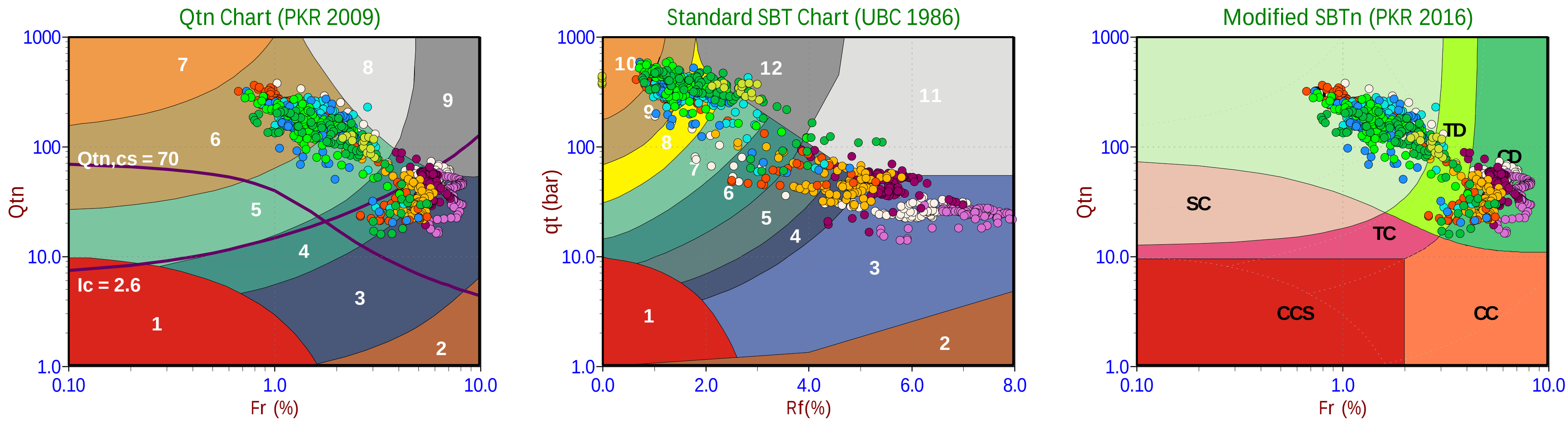
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

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)

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 10:48
Site: VP-01

Sounding: CPT-02
Cone: 483:T1500F15U500



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

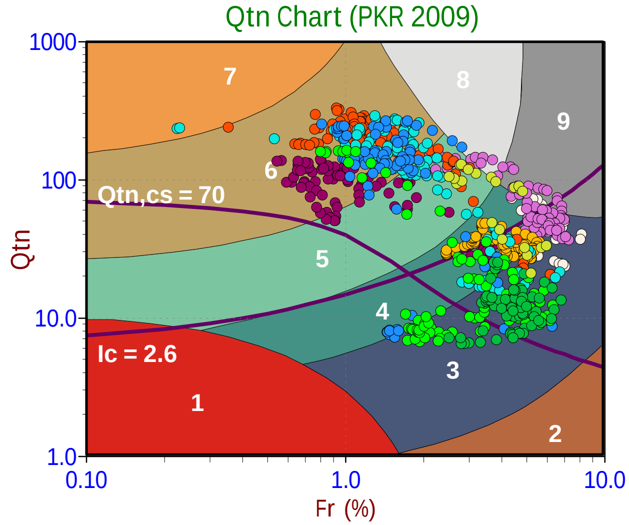
Legend

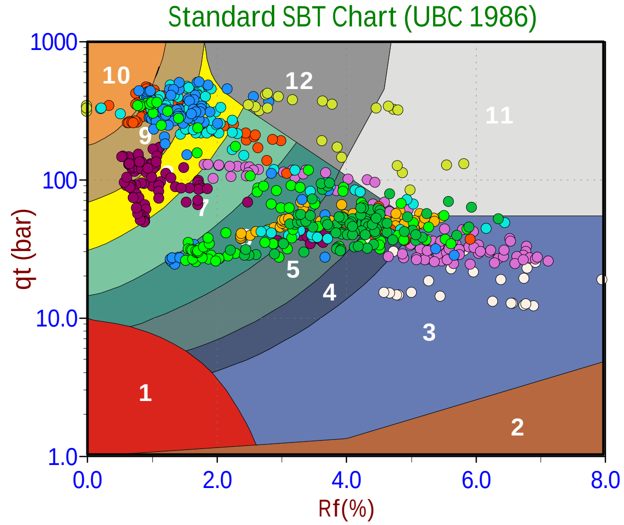
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

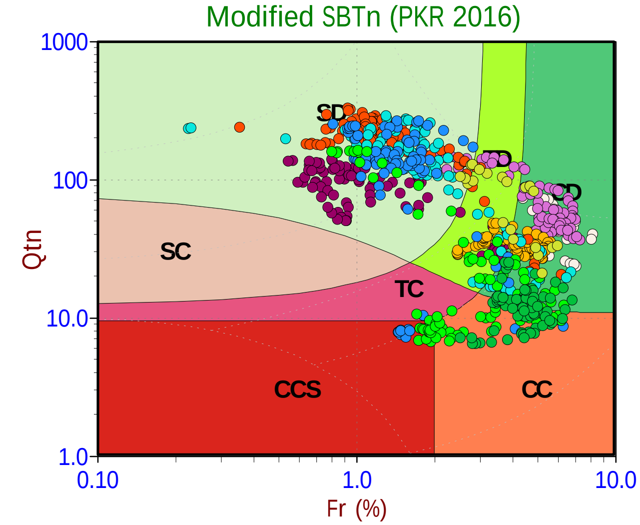
Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 09:20
Site: VP-01

Sounding: CPT-03
Cone: 483:T1500F15U500







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
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- >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
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- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

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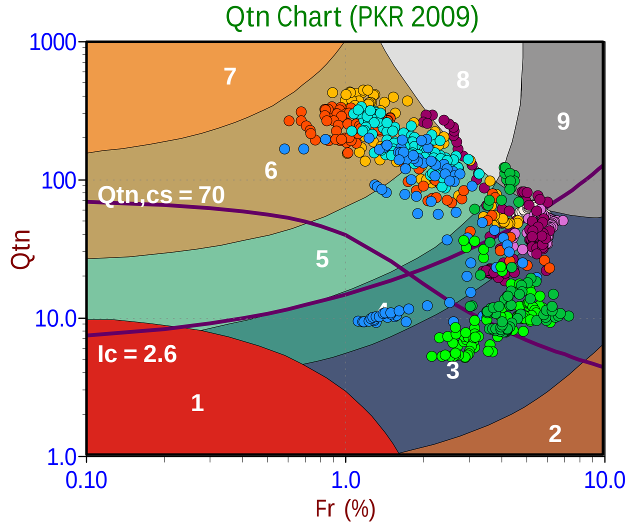
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

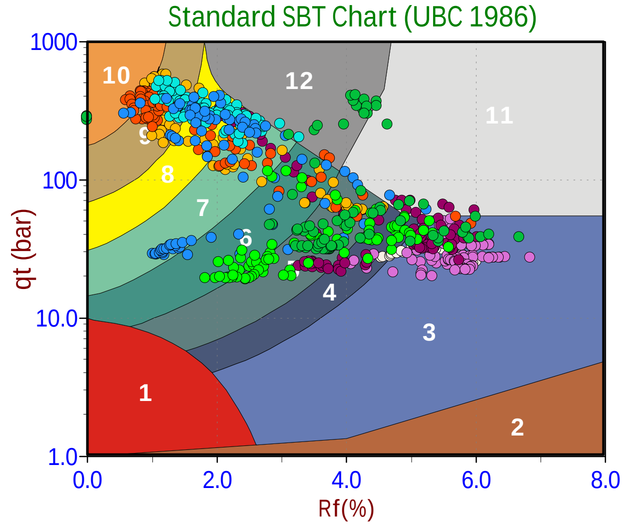
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

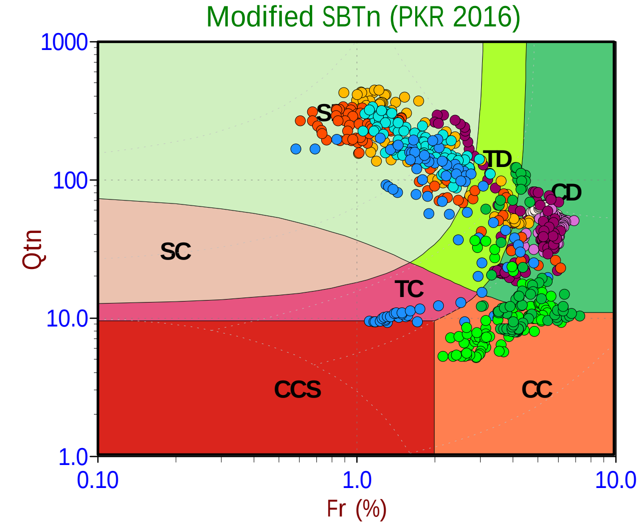
Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 12:32
Site: VP-01

Sounding: CPT-04
Cone: 483:T1500F15U500







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)

Seismic Cone Penetration Test Plots

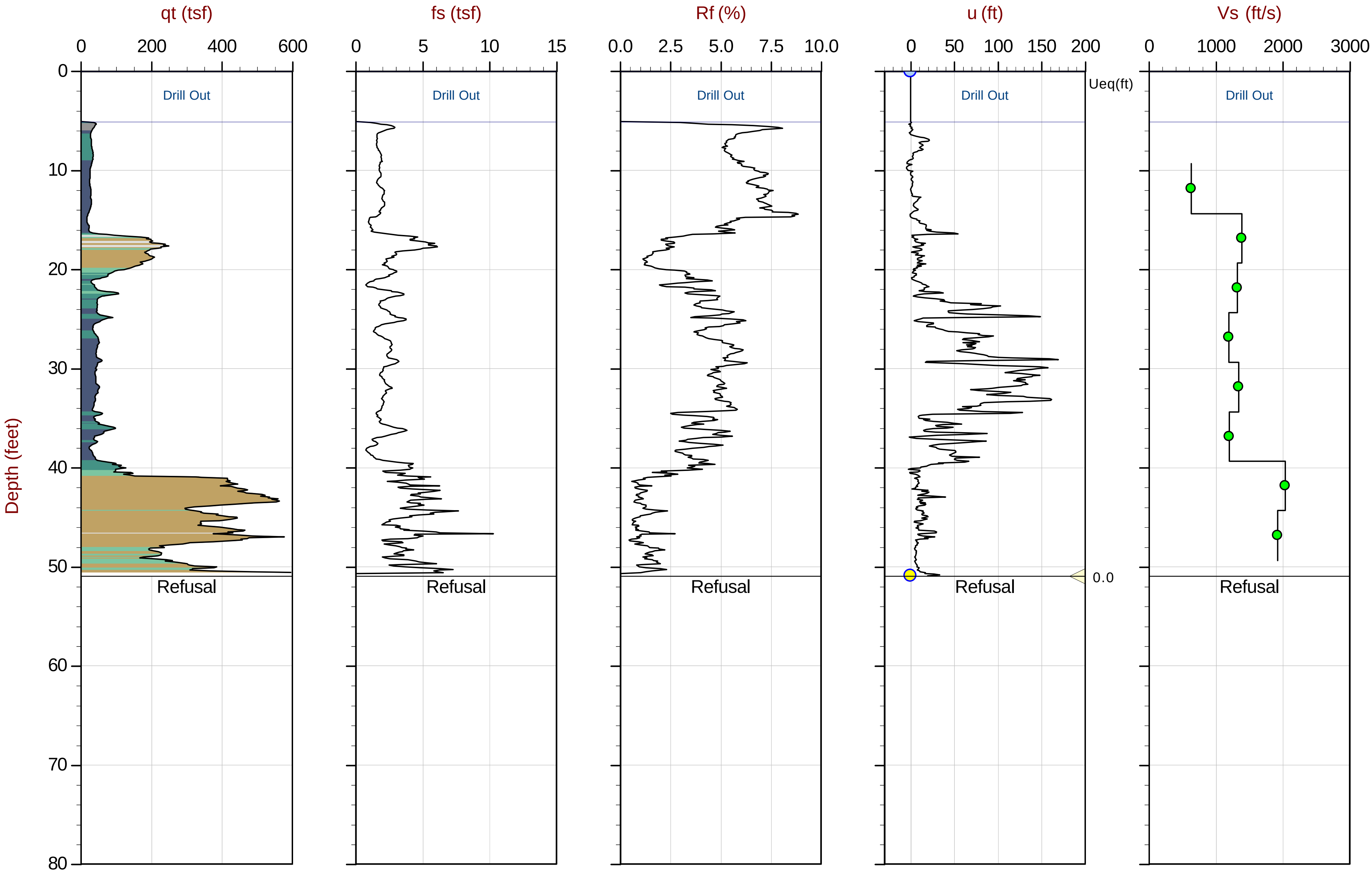
CONETEC

CONETEC

Kleinfelder

Job No: 21-56-21869
Date: 2021-01-26 08:03
Site: VP-01

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 15.525 m / 50.93 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21869_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4131475m E: 587940m

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: 21-56-21869
Client: Kleinfelder
Project: VP-01
Sounding ID: CPT-01
Date: 01:26:21 08:03

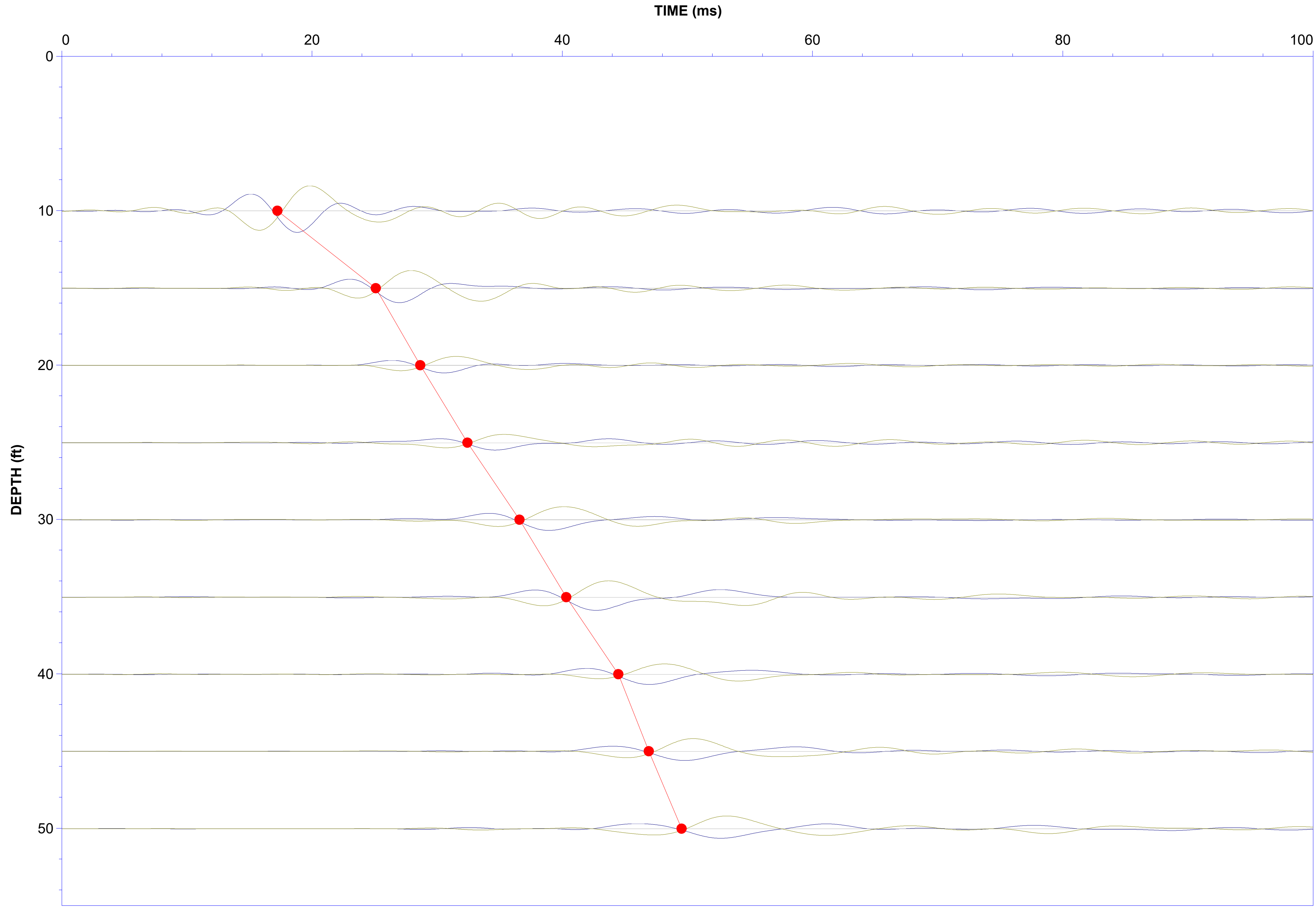
Seismic Source: Beam
Seismic Offset (ft): 1.87
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

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)
10.01	9.35	9.54			
15.03	14.37	14.49	4.96	7.85	631
20.01	19.36	19.45	4.96	3.56	1392
25.03	24.38	24.45	5.00	3.78	1325
30.02	29.36	29.42	4.97	4.16	1196
35.04	34.38	34.43	5.01	3.74	1341
40.03	39.37	39.41	4.98	4.15	1199
45.01	44.36	44.40	4.98	2.44	2038
50.03	49.38	49.41	5.02	2.61	1923

Seismic Cone Penetration Test Shear Wave (Vs) Traces

Job No: 21-56-21869	Client: Kleinfelder	Project: VP-01	Filter: BP 0-200 Hz	Sounding: CPT-01	Date: 01:26:21 08:03
Cone: 483:T1500F15U500					



Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No:	21-56-21869
Client:	Kleinfelder
Project:	VP-01
Start Date:	26-Jan-2021
End Date:	26-Jan-2021

CPTu PORE PRESSURE DISSIPATION SUMMARY

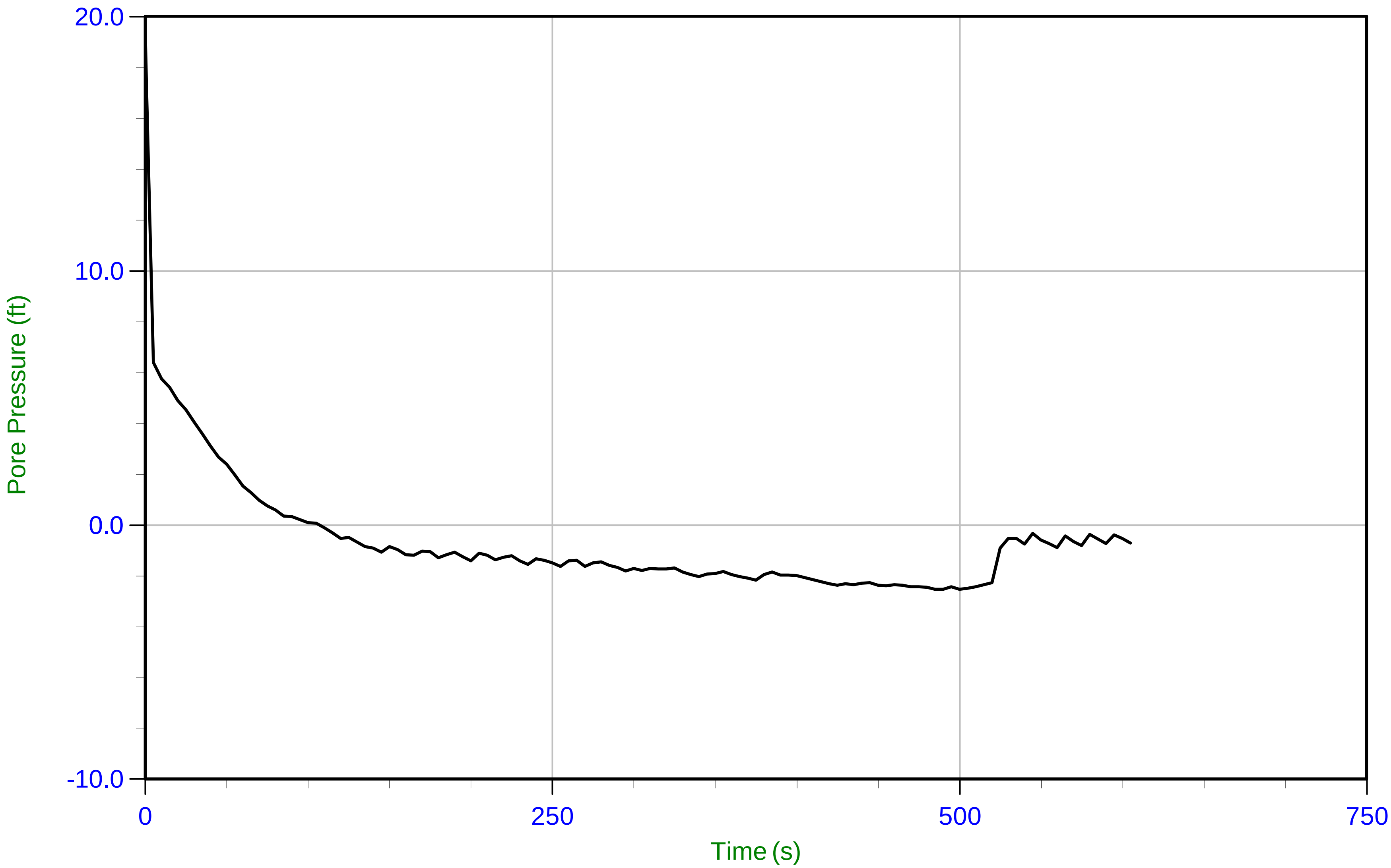
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
CPT-02	21-56-21869_CP02	15	455	61.68	0.0	
CPT-03	21-56-21869_CP03	15	325	68.32	Not Achieved	
CPT-01	21-56-21869_SP01	15	605	50.93	0.0	

CONETEC

Kleinfelder

Job No: 21-56-21869
Date: 01/26/2021 08:03
Site: VP-01

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



Trace Summary:

Filename: 21-56-21869_SP01.PPF
Depth: 15.525 m / 50.934 ft
Duration: 605.0 s

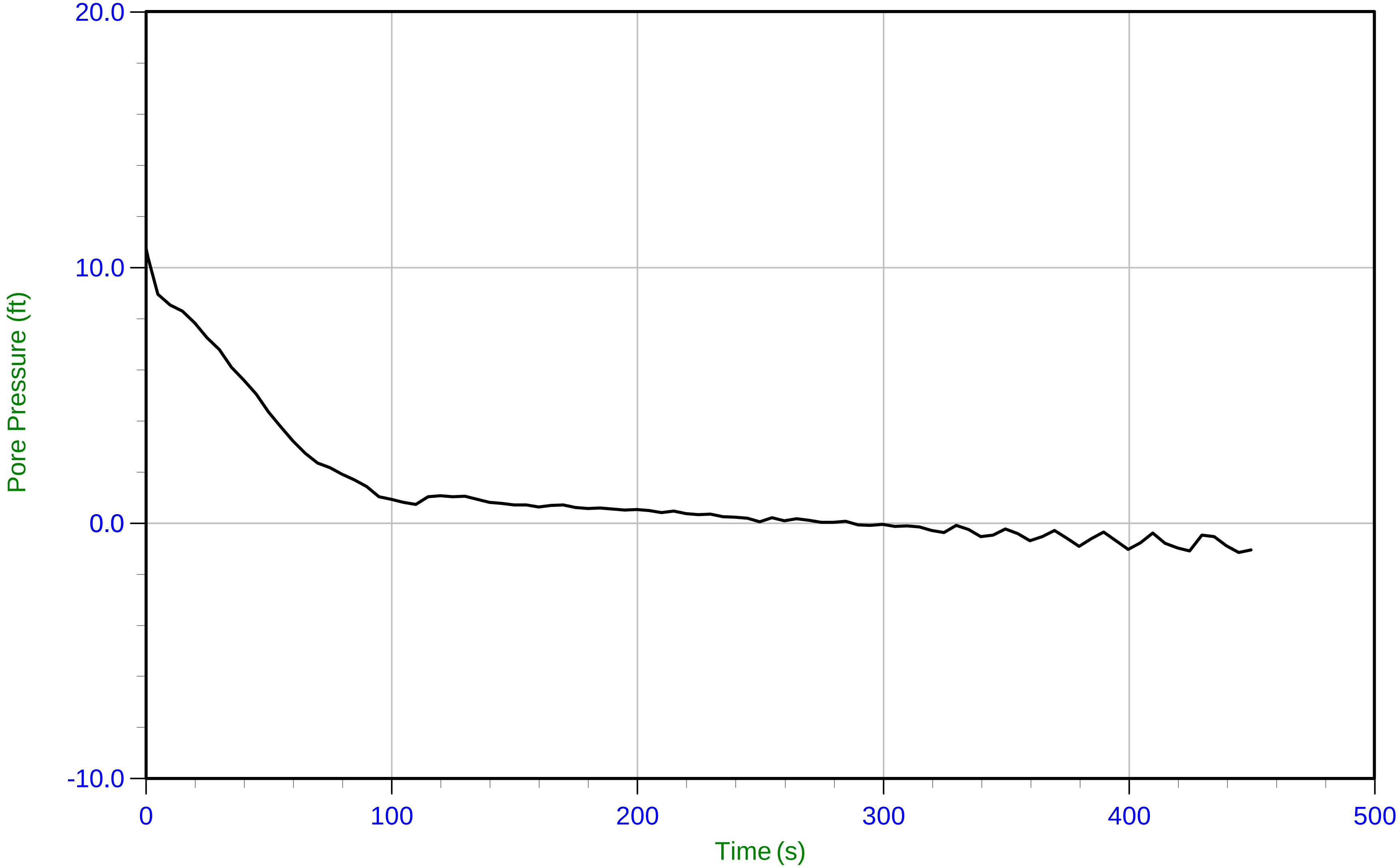
u Min: -2.5 ft
u Max: 19.4 ft
u Final: -0.7 ft

WT: 15.525 m / 50.934 ft
Ueq: 0.0 ft

Kleinfelder

Job No: 21-56-21869
Date: 01/26/2021 10:48
Site: VP-01

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



Trace Summary:

Filename: 21-56-21869_CP02.PPF
Depth: 18.800 m / 61.679 ft
Duration: 455.0 s

u Min: -1.1 ft
u Max: 10.7 ft
u Final: 1.6 ft

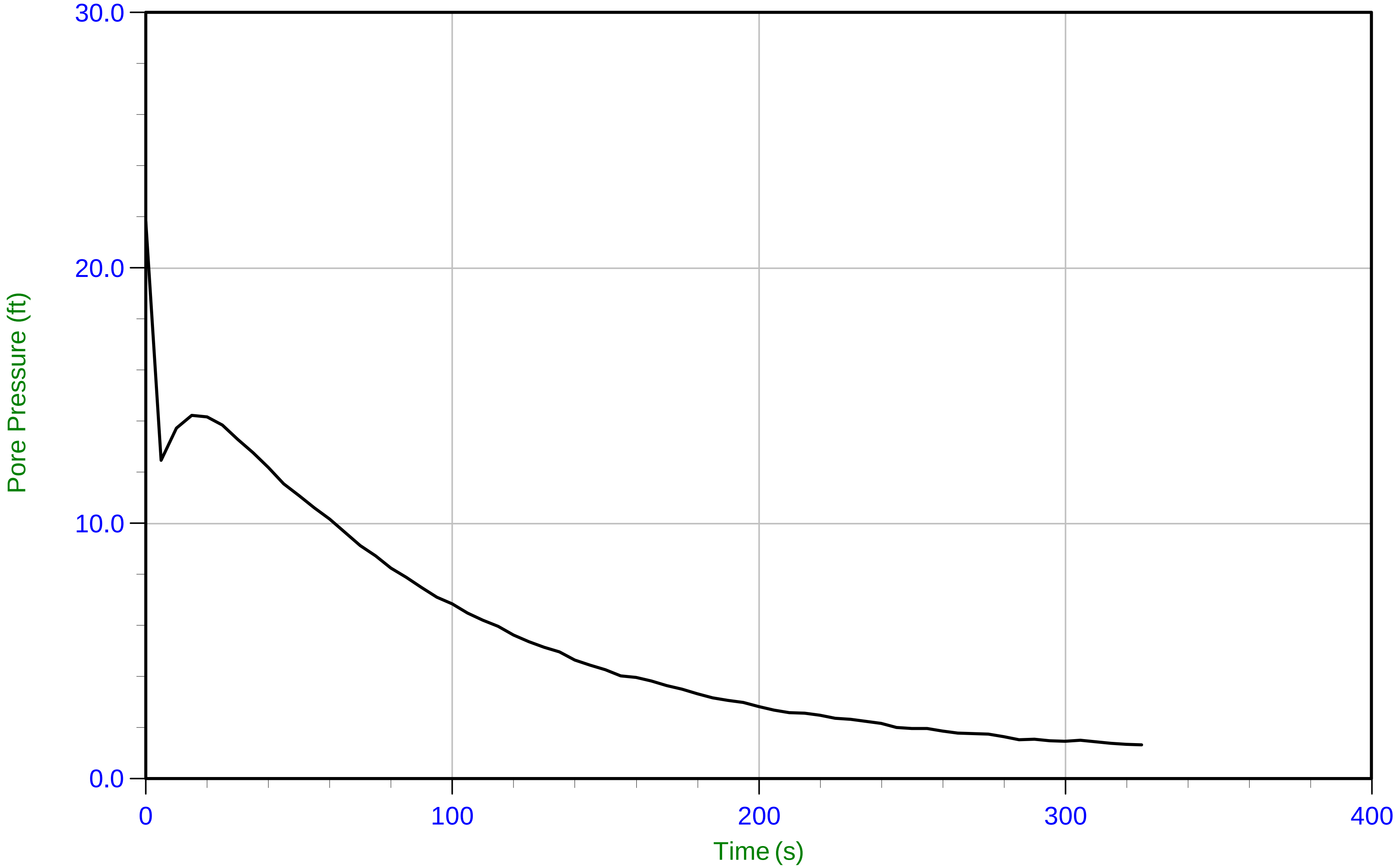
WT: 18.800 m / 61.679 ft
Ueq: 0.0 ft

ConeTec

Kleinfelder

Job No: 21-56-21869
Date: 01/26/2021 09:20
Site: VP-01

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



Trace Summary:

Filename: 21-56-21869_CP03.PPF
Depth: 20.825 m / 68.323 ft
Duration: 325.0 s

u Min: 1.3 ft
u Max: 21.8 ft
u Final: 1.3 ft

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Appendix B

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	1.0 - 5.0	BULK	LEAN CLAY WITH SAND (CL)									R - Value: 8
KB-1	6.0 - 6.5	1C	LEAN CLAY WITH SAND (CL)	21.7	103.3							
KB-1	8.0 - 8.5	2B	LEAN CLAY WITH SAND (CL)	14.6	116.5							TXUU: c = 5.26 ksf
KB-1	21.0 - 21.5	5C	CLAYEY SAND WITH GRAVEL (SC)			100	86	33				
KB-2	1.0 - 5.0	BULK	LEAN CLAY WITH SAND (CL)						40	19	21	
KB-2	6.0 - 6.5	1C	LEAN CLAY WITH SAND (CL)	14.6	120.5							Unconfined Compressive Strength = 2.79 ksf
KB-2	10.5 - 11.0	3B	LEAN CLAY WITH SAND (CL)	18.8	109.8							
KB-2	11.0 - 11.5	3C	LEAN CLAY WITH SAND (CL)	15.6	117.6							TXUU: c = 3.17 ksf
KB-3	5.5 - 6.0	1B	LEAN CLAY WITH SAND (CL)	22.7	106.5							
KB-3	21.0 - 21.5	5C	LEAN CLAY (CL)	18.2	112.6							Unconfined Compressive Strength = 6.70 ksf
KB-3	36.0 - 36.5	8C	LEAN CLAY (CL)	20.2	109.7							Unconfined Compressive Strength = 7.82 ksf
KB-4	1.0 - 5.0	BULK	LEAN CLAY (CL)						42	21	21	
KB-4	8.0 - 8.5	2B	LEAN CLAY (CL)	19.4	110.0							Unconfined Compressive Strength = 8.56 ksf
KB-4	11.0 - 11.5	3C	LEAN CLAY (CL)	20.1	109.7							
KB-4	16.0 - 16.5	4C	LEAN CLAY (CL)	17.0	113.4							TXUU: c = 4.35 ksf
KB-4	25.0 - 26.5	6	WELL GRADED SAND WITH CLAY AND GRAVEL (SW-SC)			97	62	7.6				

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

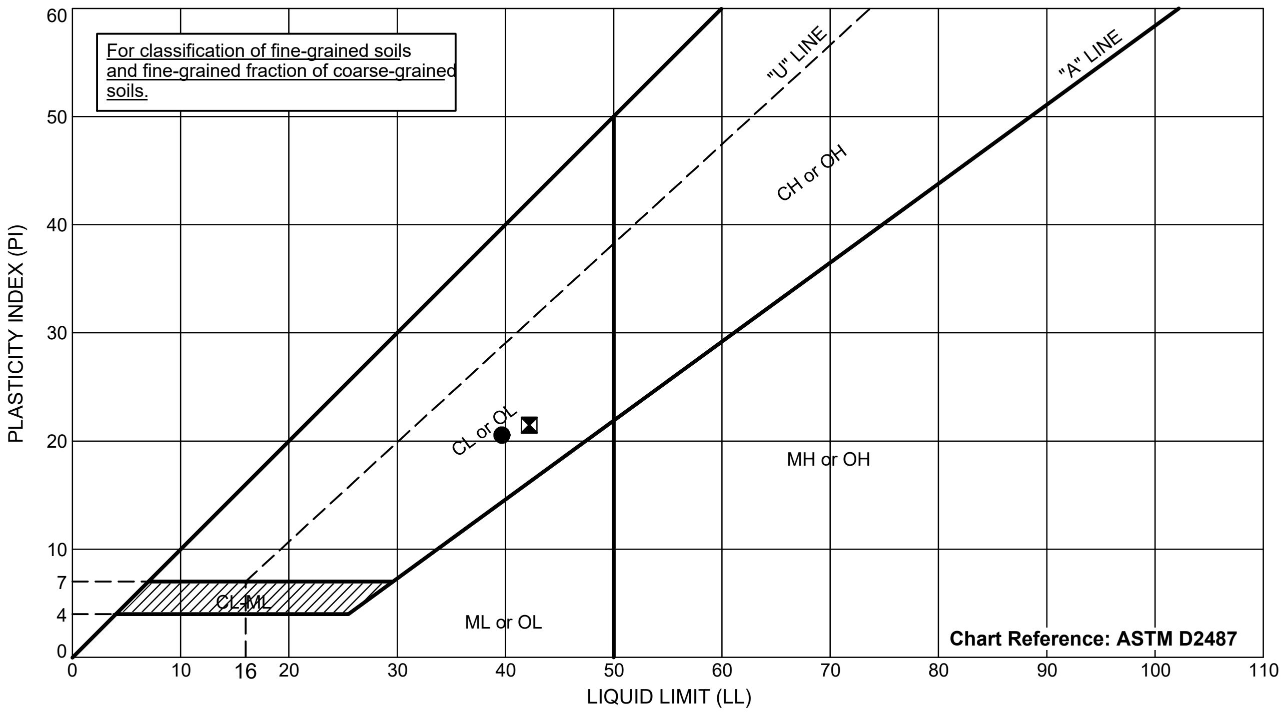
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20213246.001A
DRAWN BY: JDS
CHECKED BY: AL
DATE: February 2021

LABORATORY TEST RESULT SUMMARY

APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CALIFORNIA

FIGURE B-1



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	Passing #200	LL	PL	PI
●	KB-2	1 - 5	BULK	LEAN CLAY WITH SAND (CL)	NM	40	19	21
⊠	KB-4	1 - 5	BULK	LEAN CLAY (CL)	NM	42	21	21

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

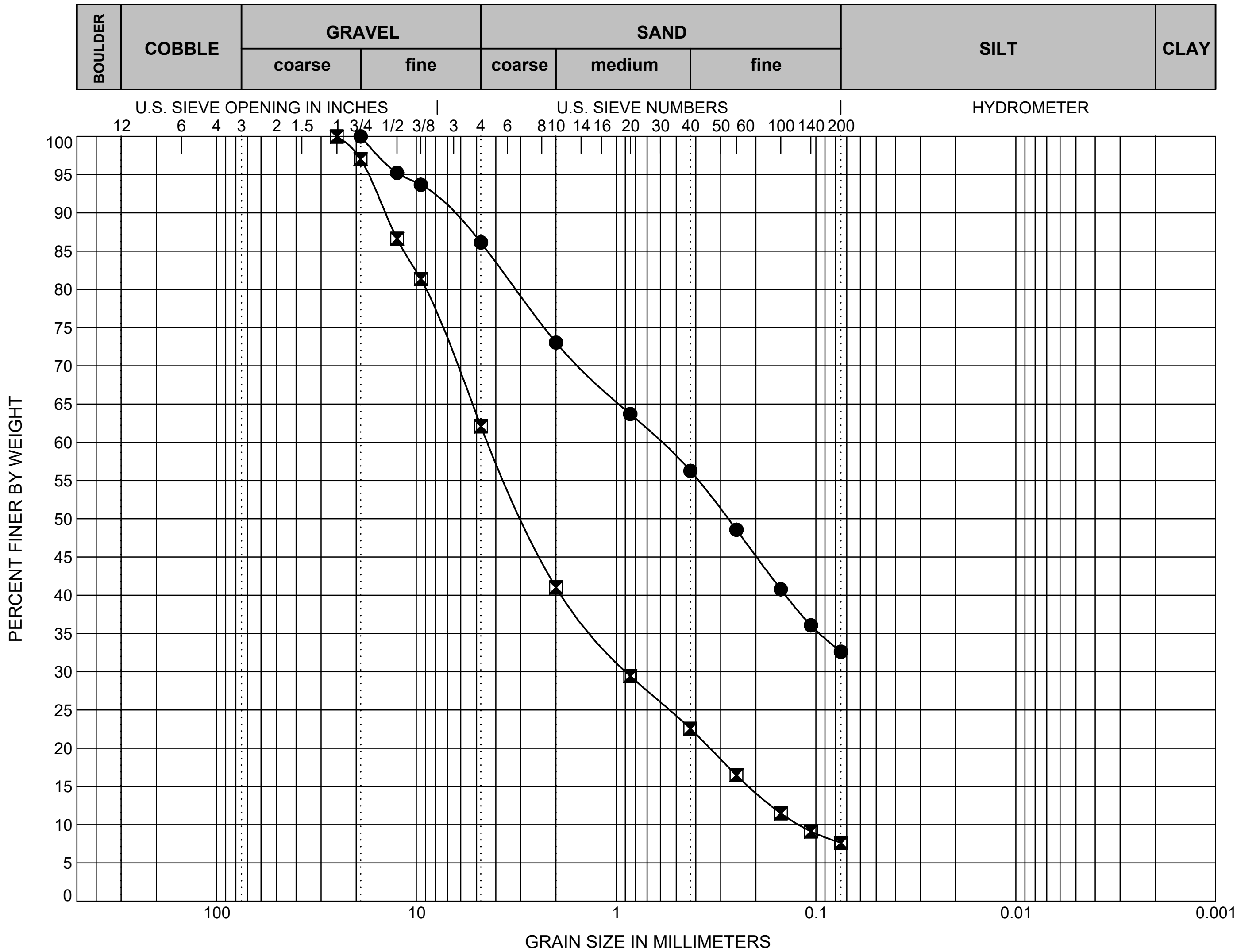
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20213246.001A
DRAWN BY: JDS
CHECKED BY: AL
DATE: February 2021

ATTERBERG LIMITS

APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CALIFORNIA

FIGURE
B-2



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	LL	PL	PI
●	KB-1	21 - 21.5	5C	CLAYEY SAND WITH GRAVEL (SC)	NM	NM	NM
⧗	KB-4	25 - 26.5	6	WELL GRADED SAND WITH CLAY AND GRAVEL (SW-SC)	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-1	21 - 21.5	19	0.602	NM	NM	NM	NM	100	86	33	NM	NM
⧗	KB-4	25 - 26.5	25	4.357	0.886	0.12	1.50	36.16	97	62	7.6	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.

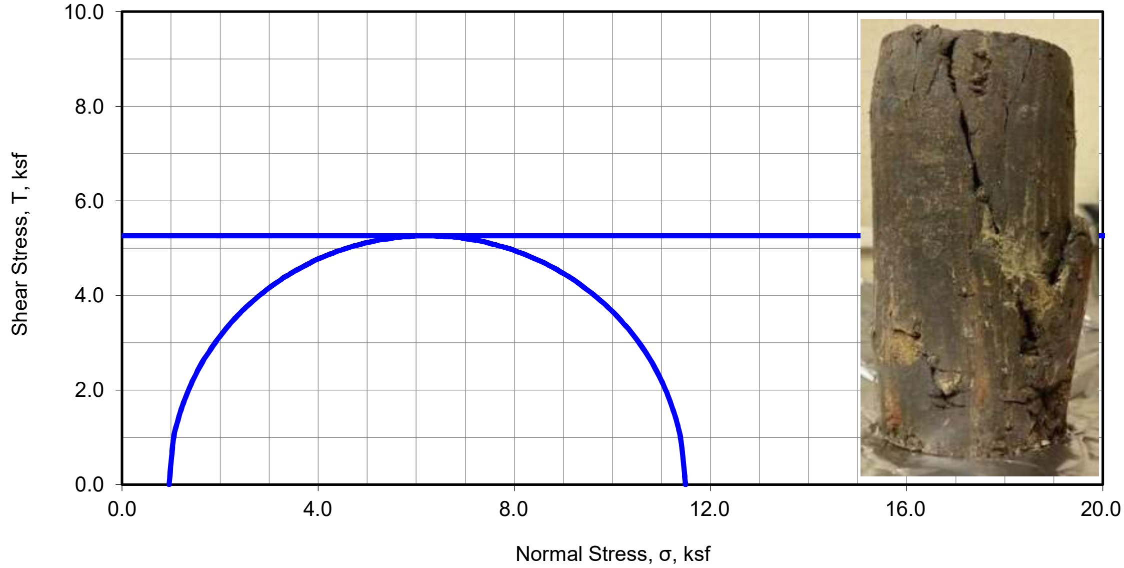
PROJECT NO.: 20213246.001A
DRAWN BY: JDS
CHECKED BY: AL
DATE: February 2021

SIEVE ANALYSIS

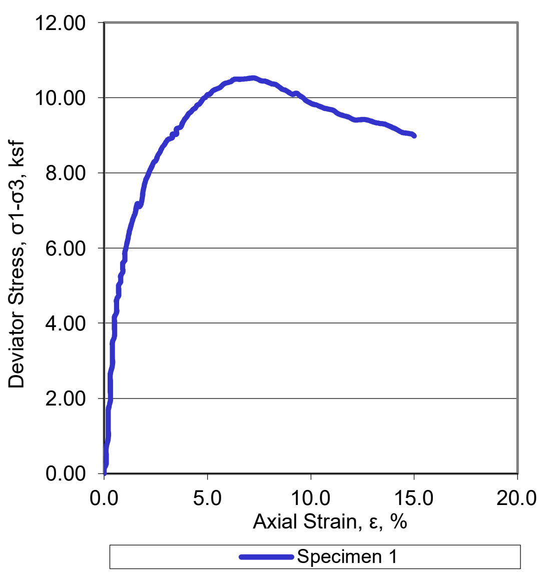
APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CALIFORNIA

FIGURE
B-3

Total		
c =	5.26	ksf



Specimen Shear Picture



Specimen No.			1
Initial	Diameter, in	D_O	2.37
	Height, in	H_O	5.84
	Water Content, %	ω_O	14.6
	Dry Density, lbs/ft^3	γd_o	116.5
	Saturation, %	S_O	92
	Void Ratio	e_O	0.420
Minor Principal Stress, ksf		σ_3	0.96
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	10.52
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	7.08
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	8.98
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	7.08

Description of Specimen: Dark Brown Sandy Clay with Gravel

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

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

Membrane correction applied

Boring:	KB-1
Sample:	2C
Depth, ft:	8.0 - 8.5
Test Date:	2/5/21

Remarks: nm= not measured, na = not applicable

KLEINFELDER
Bright People. Right Solutions.

2601 Barrington Ct, Hayward, CA 94545

Project No.:	20213246.001A
Date:	2/11/21
Entry By:	MS
Checked By:	MS
File Name:	HL13540

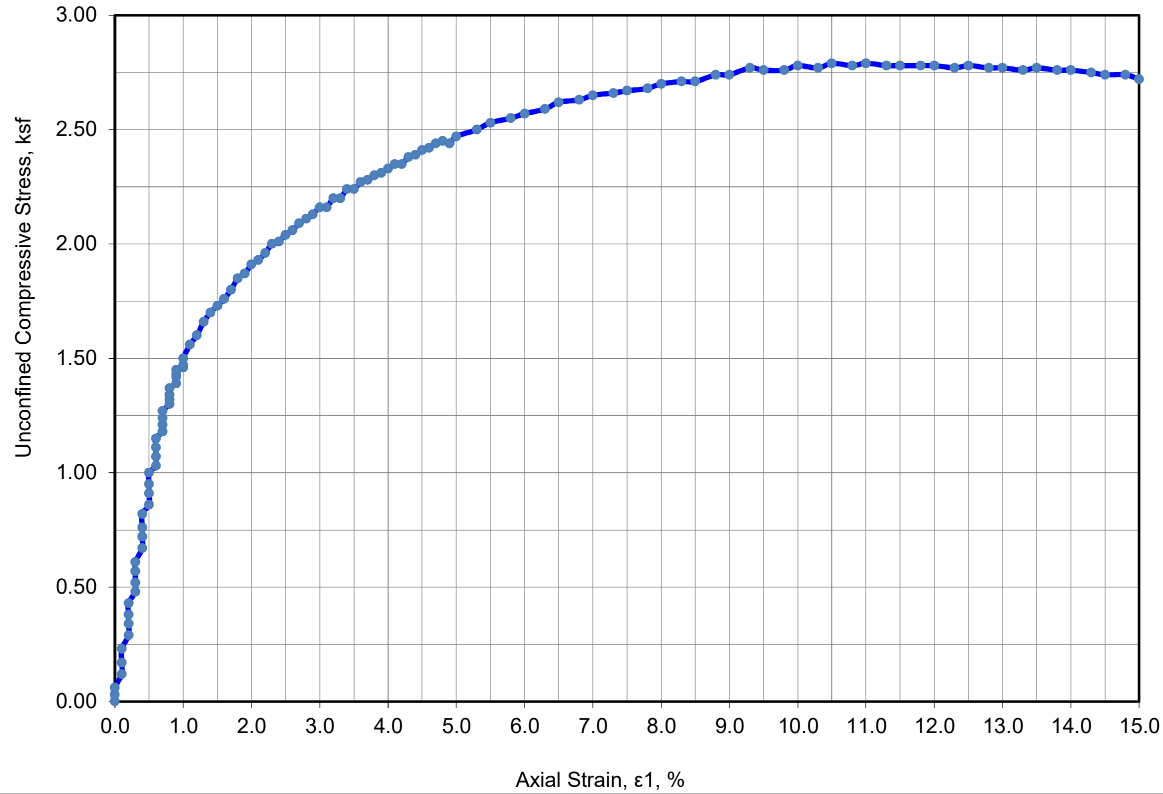
TRIAXIAL COMPRESSION TEST (UU)

Apple VP01
19191 Vallco Parkway
Cupertino, California

Figure

1 of 1

B-4



Specimen Failure Picture	Specimen No.		1
	Initial: Diameter, in	D_O	2.40
	Initial: Height, in	H_O	5.84
	Initial: Height to Diameter Ratio		2.43
	Initial: Water Content, %	ω_O	14.6
	Initial: Dry Density, lbs/ft^3	γd_o	120.5
	Initial: Saturation, %	S_O	103.8
	Initial: Void Ratio	e_O	0.372
	Time to Failure, min.	t_f	11.0
	Unconfined Compressive Strength, ksf	q_u	2.79
	Shear Strength, ksf	s_u	1.40
	Strain at Failure, %	ε_f	11.0
	Average Rate of Strain to Failure, %/min	ε	1.0

Description of Specimen: Brown Sandy Clay

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

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Intact	Test Method: ASTM D2166

Boring:	KB-2	Remarks: nm= not measured, na = not applicable
Sample:	1C	*A 3/4" piece of rock is found inside the sample.
Depth, ft:	6.0 - 6.5	
Test Date:	2/5/21	

KLEINFELDER
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2601 Barrington Court, Hayward, California 94545

Project No.:	20213246.001A
Date:	2/11/2021
Entry By:	M. Sacramento
Checked By:	M. Sacramento
File Name:	HL13540

UNCONFINED COMPRESSION TEST (UC)

Apple VP01
19191 Vallco Parkway
Cupertino, California

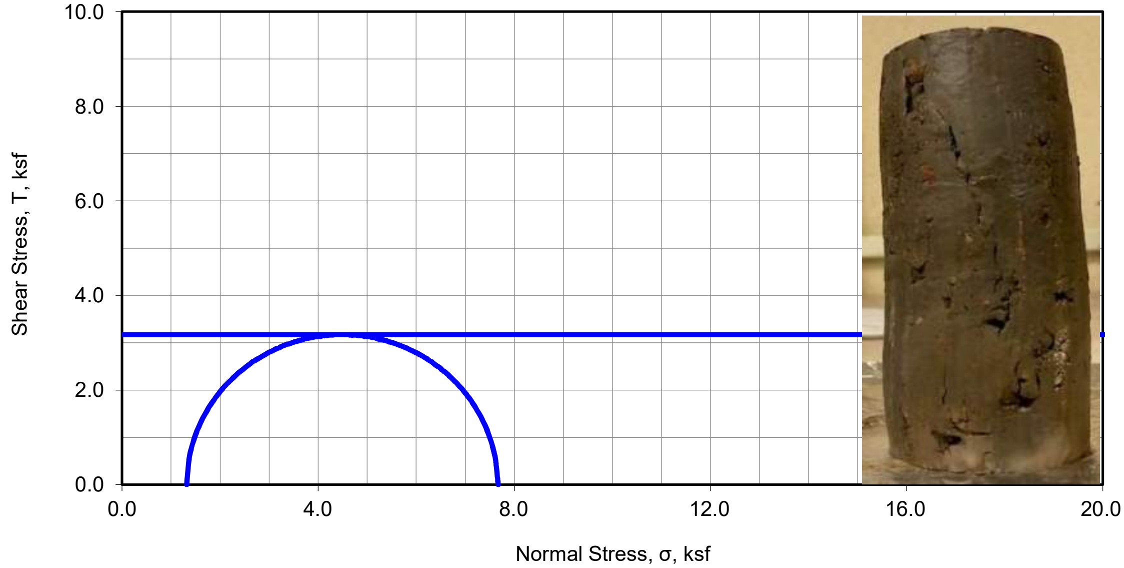
Figure

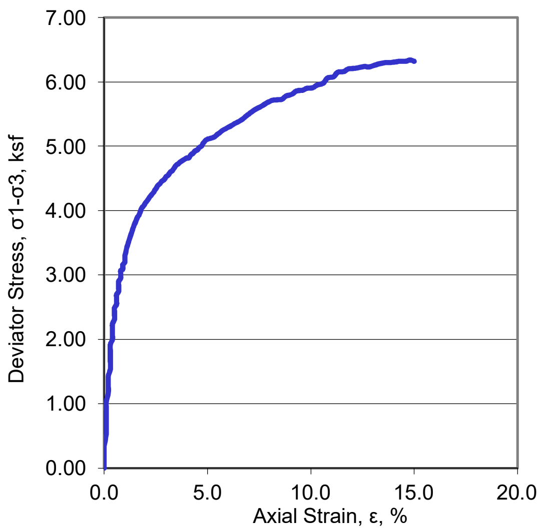
1 of 1

B-5

Total	
c = 3.17	ksf

Specimen Shear Picture





	Specimen No.		1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	6.01
	Water Content, %	ω_O	15.6
	Dry Density, lbs/ft^3	γ_{d_o}	117.6
	Saturation, %	S_O	102
	Void Ratio	e_O	0.406
	Minor Principal Stress, ksf	σ_3	1.32
	Maximum Deviator Stress, ksf	$(\sigma1-\sigma3)_{max}$	6.34
	Time to $(\sigma_1-\sigma_3)_{max}$, min	t_f	14.85
	Deviator Stress @ 15% Axial Strain, ksf	$(\sigma_1-\sigma_3)_{15\%}$	6.32
	Ultimate Deviator Stress, ksf	$(\sigma_1-\sigma_3)_{ult}$	na
	Rate of strain, %/min	$'\varepsilon$	1.00
	Axial Strain at Failure, %	ε_f	14.85

Description of Specimen: Dark Brown Sandy Clay

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

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

Membrane correction applied

Boring:	KB-2
Sample:	3C
Depth, ft:	11.0 - 11.5
Test Date:	2/5/21

Remarks: nm= not measured, na = not applicable

KLEINFELDER
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2601 Barrington Ct, Hayward, CA 94545

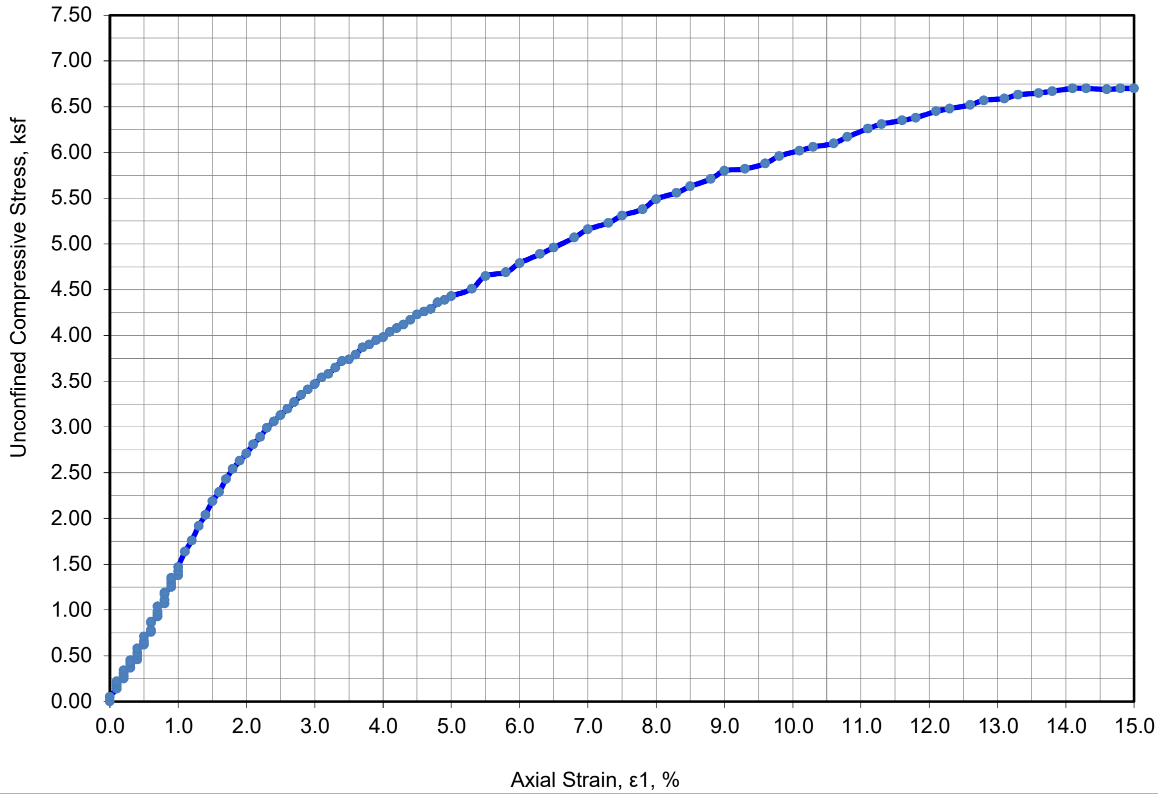
Project No.:	20213246.001A
Date:	2/11/21
Entry By:	MS
Checked By:	MS
File Name:	HL13540

TRIAXIAL COMPRESSION TEST (UU)

Apple VP01
19191 Vallco Parkway
Cupertino, California

Figure

B-6



Specimen Failure Picture

Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	5.99
	Height to Diameter Ratio		2.49
	Water Content, %	ω_O	18.2
	Dry Density, lbs/ft^3	$^{\gamma}d_o$	112.6
	Saturation, %	S_O	103.2
	Void Ratio	e_O	0.468
Time to Failure, min.		t_f	14.8
Unconfined Compressive Strength, ksf		q_u	6.70
Shear Strength, ksf		s_u	3.35
Strain at Failure, %		ε_f	14.8
Average Rate of Strain to Failure, %/min		ε	1.0

Description of Specimen: Yellowish Brown Clay

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

LL: nm | PL: nm | PI: nm | G_S: 2.65 Assumed | Specimen Type: Intact | Test Method: ASTM D2166

Boring:	KB-3
Sample:	5C
Depth, ft:	21.0
Test Date:	2/5/21

Remarks: nm= not measured, na = not applicable

KLEINFELDER
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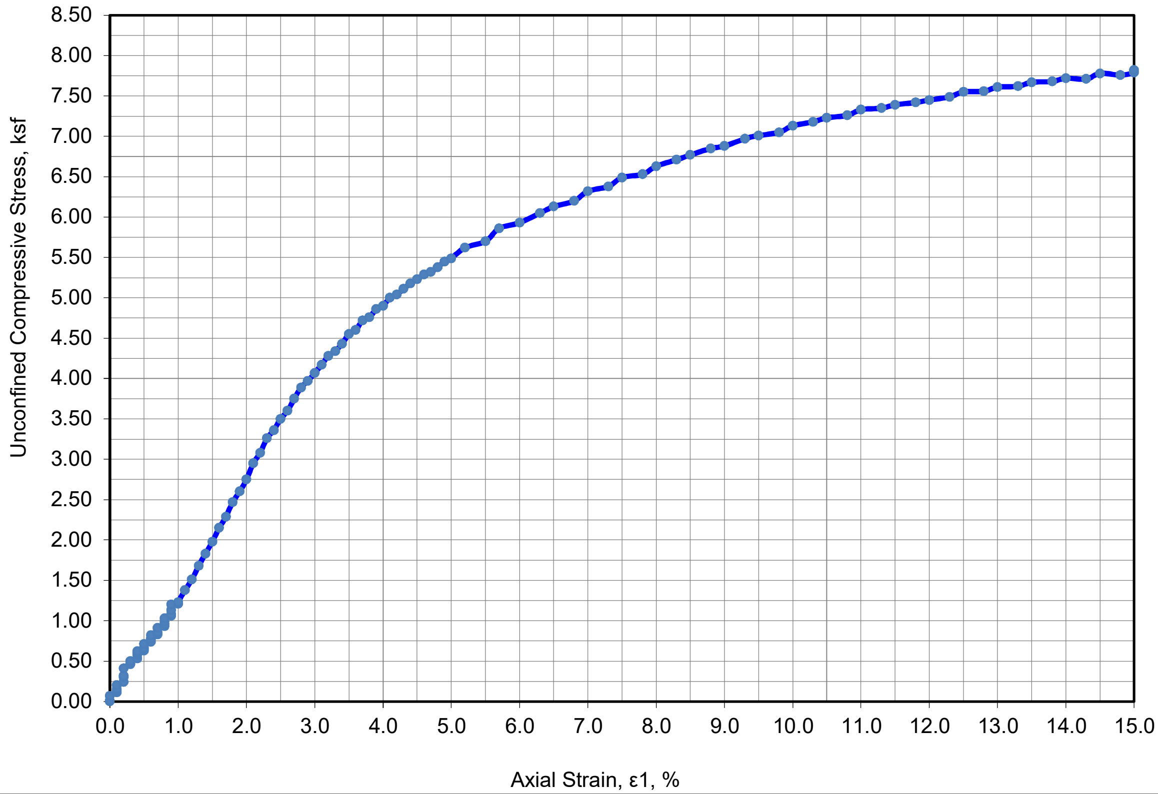
2601 Barrington Court, Hayward, California 94545

Project No.:	20213246.001A
Date:	2/11/2021
Entry By:	M. Sacramento
Checked By:	M. Sacramento
File Name:	HL13540

UNCONFINED COMPRESSION TEST (UC)

Apple VP01
19191 Vallco Parkway
Cupertino, California

Figure
1 of 1
B-7



Specimen Failure Picture	Specimen No.			1
	Initial	Diameter, in	D_O	2.42
		Height, in	H_O	6.04
		Height to Diameter Ratio		2.50
		Water Content, %	ω_O	20.2
		Dry Density, lbs/ft^3	γd_o	109.7
		Saturation, %	S_O	105.3
		Void Ratio	e_O	0.508
	Time to Failure, min.		t_f	15.0
	Unconfined Compressive Strength, ksf		q_u	7.82
	Shear Strength, ksf		s_u	3.91
	Strain at Failure, %		ε_f	15.0
	Average Rate of Strain to Failure, %/min		ε	1.0

Description of Specimen: Yellowish Brown Clay

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

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Intact	Test Method: ASTM D2166

Boring:	KB-3	Remarks: nm= not measured, na = not applicable
Sample:	8C	
Depth, ft:	41.0	
Test Date:	2/5/21	

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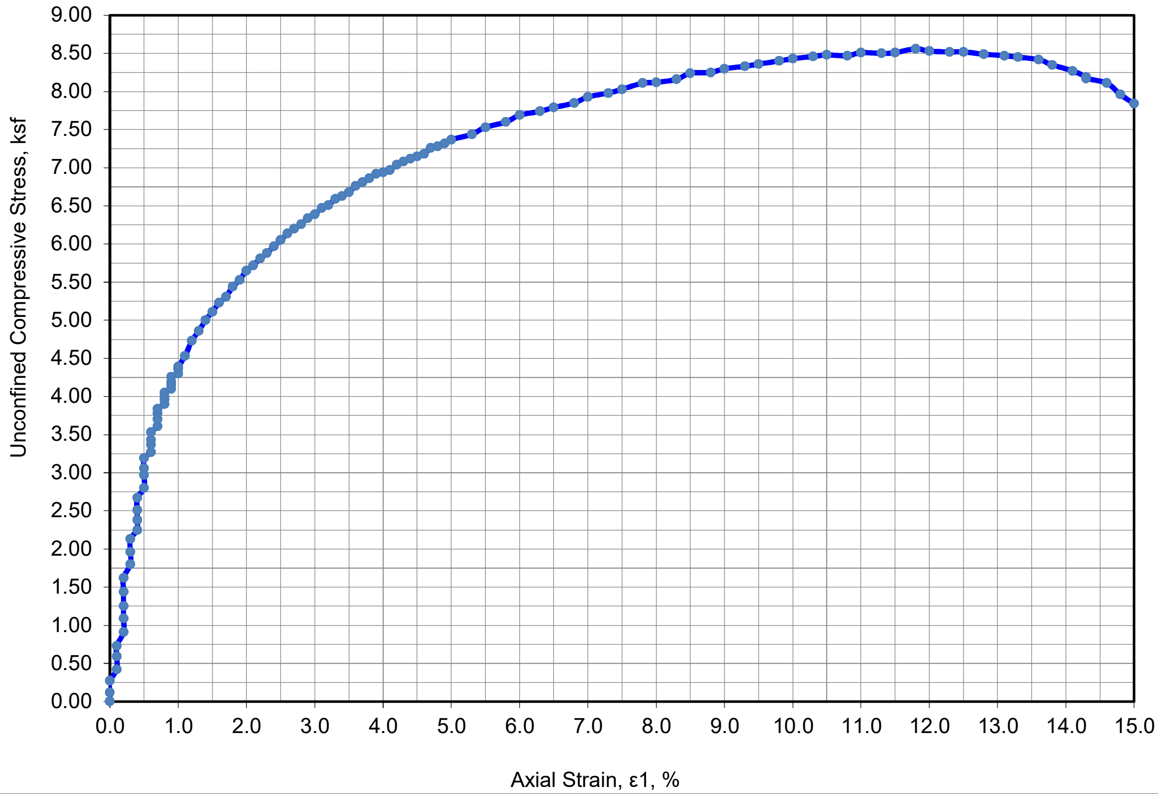
2601 Barrington Court, Hayward, California 94545

Project No.:	20213246.001A
Date:	2/11/2021
Entry By:	M. Sacramento
Checked By:	M. Sacramento
File Name:	HL13540

UNCONFINED COMPRESSION TEST (UC)

Apple VP01
19191 Vallco Parkway
Cupertino, California

Figure
1 of 1
B-8



Specimen Failure Picture	Specimen No.			1
	Initial	Diameter, in	D_O	2.42
		Height, in	H_O	6.00
		Height to Diameter Ratio		2.48
		Water Content, %	ω_O	19.4
		Dry Density, lbs/ft^3	γd_o	110.0
		Saturation, %	S_O	102.0
		Void Ratio	e_O	0.504
	Time to Failure, min.		t_f	11.8
	Unconfined Compressive Strength, ksf		q_u	8.56
	Shear Strength, ksf		s_u	4.28
	Strain at Failure, %		ε_f	11.8
	Average Rate of Strain to Failure, %/min		ε	1.0

Description of Specimen: Dark Brown Clay

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

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Intact	Test Method: ASTM D2166

Boring:	KB-4
Sample:	2C
Depth, ft:	8.0
Test Date:	2/5/21

Remarks: nm= not measured, na = not applicable

KLEINFELDER
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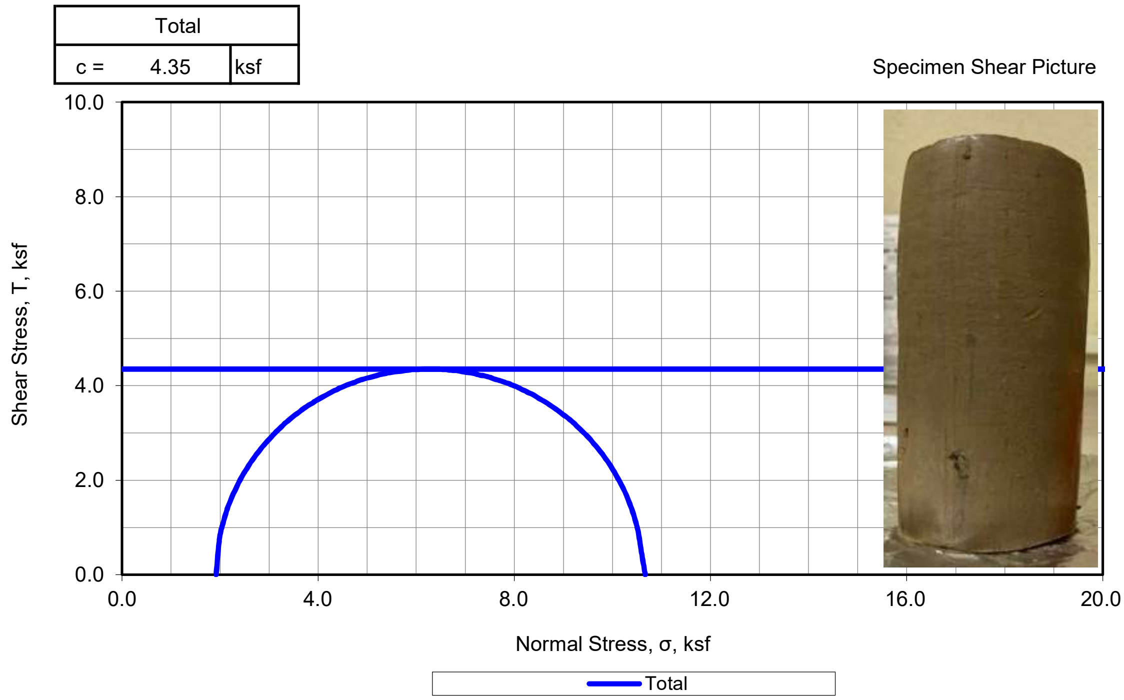
2601 Barrington Court, Hayward, California 94545

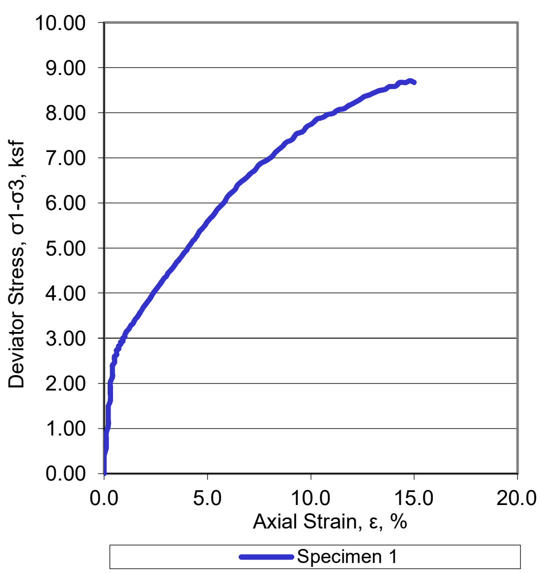
Project No.:	20213246.001A
Date:	2/11/2021
Entry By:	M. Sacramento
Checked By:	M. Sacramento
File Name:	HL13540

UNCONFINED COMPRESSION TEST (UC)

Apple VP01
19191 Vallco Parkway
Cupertino, California

Figure
1 of 1
B-9





Specimen No.			1
Initial	Diameter, in	D_O	2.42
	Height, in	H_O	6.01
	Water Content, %	ω_O	17.0
	Dry Density, lbs/ft^3	γd_o	113.4
	Saturation, %	S_O	98
	Void Ratio	e_O	0.459
Minor Principal Stress, ksf		σ_3	1.92
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	8.71
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.83
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	8.67
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.83

Description of Specimen: Brown Sandy Clay

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

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

Membrane correction applied

Boring:	KB-4
Sample:	4C
Depth, ft:	16.0 - 16.5
Test Date:	2/5/21

Remarks: nm= not measured, na = not applicable

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

Project No.:	20213246.001A
Date:	2/11/21
Entry By:	MS
Checked By:	MS
File Name:	HL13540

TRIAXIAL COMPRESSION TEST (UU)

Apple VP01
19191 Vallco Parkway
Cupertino, California

Figure
1 of 1
B-10

Laboratory Test Report

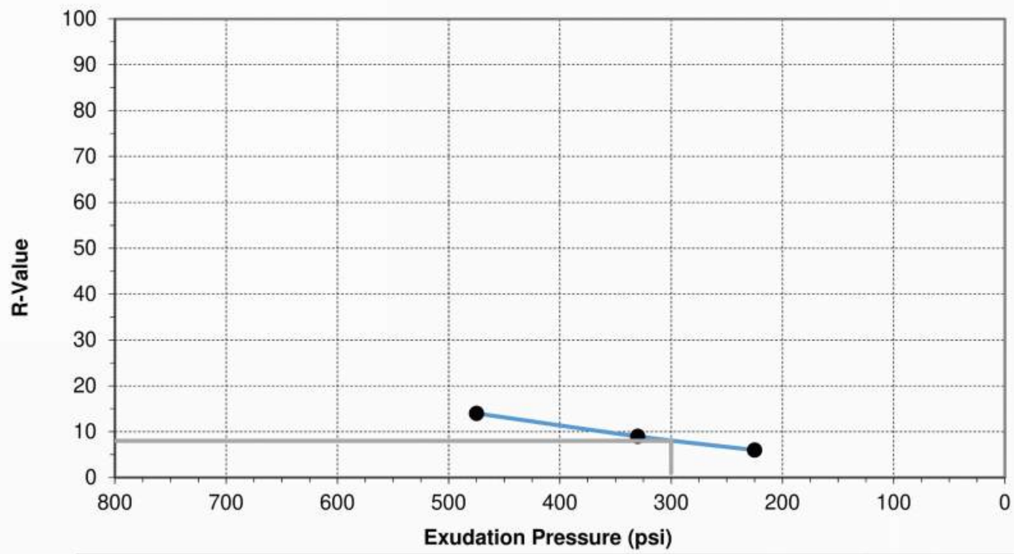
Client:	**Apple, Inc.**	Report No.:	**21-HAY-00110 Rev. 0**	Issued:	**2/12/2021**
Project:	**20213246.001A**			Field ID:	**HL13540**
	Apple VP01, Cupertino, CA GEO	Sampled by:	**A. Lin**	Date:	**1/28/2021**
	01-000L - Lab Testing	Submitted by:	**A. Lin**	Date:	**2/3/2021**

Tested on **2/9/2021** by **B. O'neil**

Test Method: **ASTM D2844**

Material Description: **Brown Clay with Gravel**

Specific Location: **KB-1 @ 0' - 5.0'**



Briquette No.	**A**	**B**	**C**
Dry Unit Weight at Test (pcf)	112.0	111.7	109.6
Expansion Pressure (psf)	48	22	74
Exudation Pressure (psi)	330	225	475
Moisture at Time of Test (%)	16.9	17.9	15.9
Resistance Value	9	6	14
R - VALUE AT 300 PSI EXUDATION PRESSURE:			**8**

Remarks:
HL13540

Reviewed on 2/12/2021 by Aaron Kidd,
Materials Manager I

California State Certified Laboratory No. 2153

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

24 February, 2021

Job No. 2102046
Cust. No. 10527

Mr. Alvin Lin
Kleinfelder
6801 Koll Center Parkway, Ste. 150
Pleasanton, CA 94566

Subject: Project No.: 20213246.001A
Project Name: Apple VP-01
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Lin:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on February 08, 2021. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

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

The chloride ion concentration is 39 mg/kg and is determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 95 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

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

The pH of the soil is 8.52 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 340-mV and is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

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

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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Client: Kleinfelder
Client's Project No.: 20213246.001A
Client's Project Name: Apple VP-01
Date Sampled: 01/26-28/21
Date Received: 8-Feb-2021
Matrix: Soil
Authorization: Chain of Custody

Date of Report: 24-Feb-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)*
2102046-001	KB-2, Bulk @ 0-5'	+340	8.52	2,400	1,400	N.D.	39	95

	Redox	pH	Resistivity (As Received)	Resistivity (100% Saturation)	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	23-Feb-2021	23-Feb-2021	22-Feb-2021	22-Feb-2021	23-Feb-2021	23-Feb-2021	23-Feb-2021

Cheryl McMillen
Laboratory Director

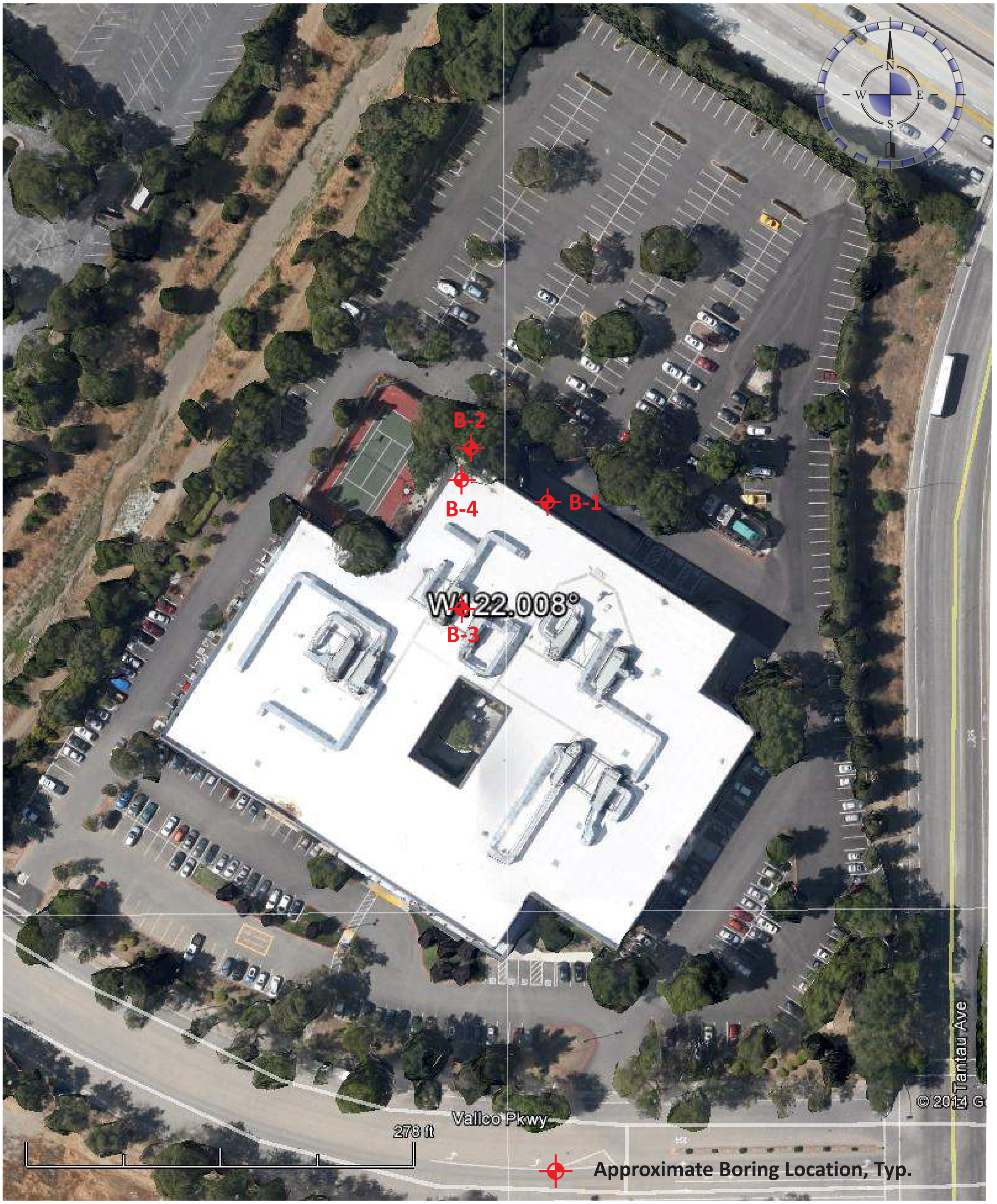
* Results Reported on "As Received" Basis
N.D. - None Detected

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

KLEINFELDER

Bright People. Right Solutions.

Appendix C

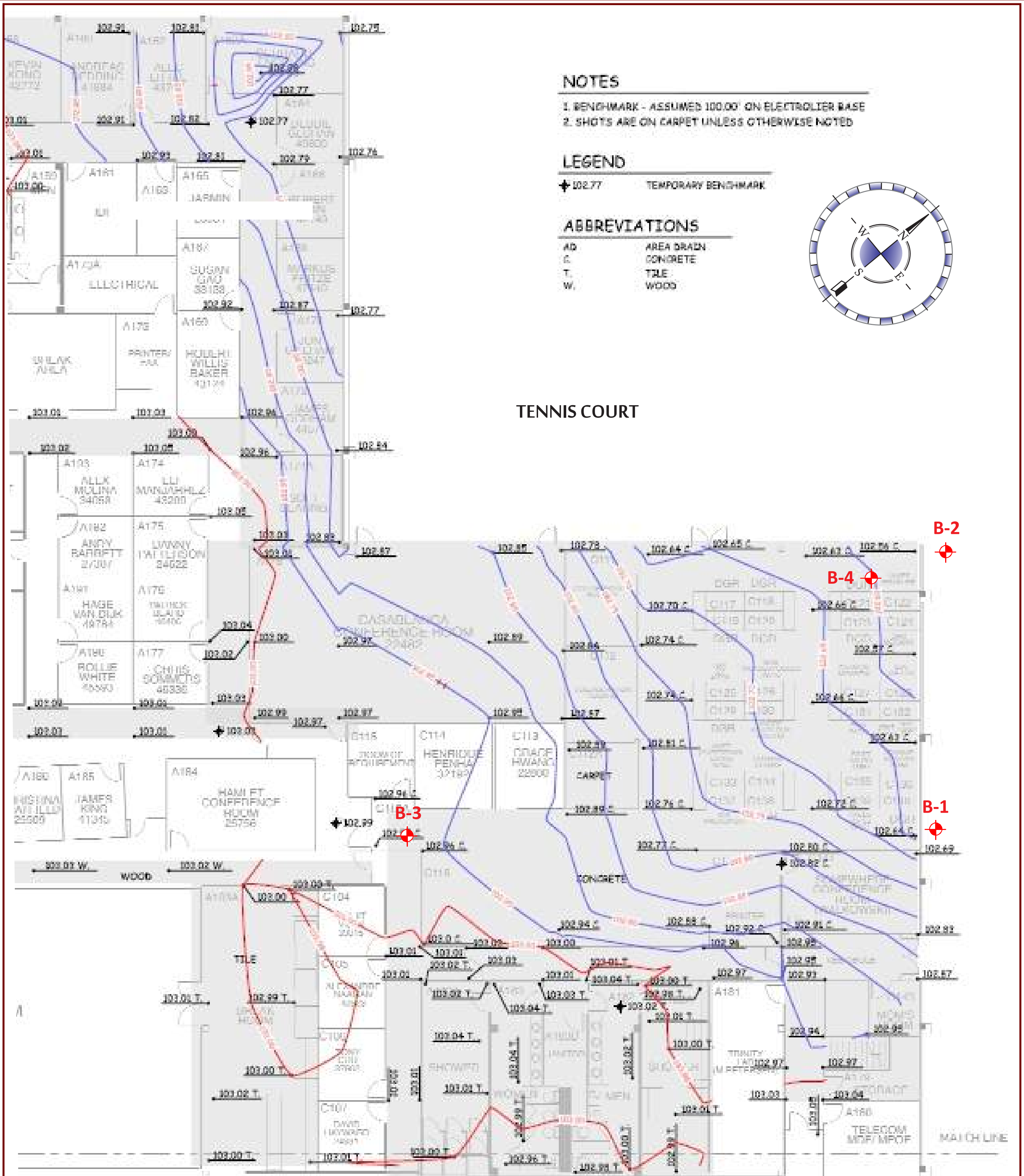


GEOTECHNICAL ENGINEERING INVESTIGATION
BUILDING MOVEMENT & DISTRESS EVALUATION
APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CALIFORNIA

SITE PLAN

DATE:	JOB NUMBER:	PLATE
July 2014	APPLE-20-00	2

BAGG ENGINEERS



BASE: "APPLE VP1-19191 Vallco Parkway, Cupetino, Floor Elevations- Floor 1 North," Page 1 of 4, by Kier & Wright, dated 4/4/2014.

Approximate Boring Location, Typ.

GEOTECHNICAL ENGINEERING INVESTIGATION BUILDING MOVEMENT & DISTRESS EVALUATION APPLE VP01 19191 VALLCO PARKWAY CUPERTINO, CALIFORNIA	PARTIAL FLOOR LEVEL SURVEY PLAN		
	DATE: July 2014	JOB NUMBER: APPLE-20-00	PLATE: 3

BAGG ENGINEERS

COARSE-GRAINED SOILS

LESS THAN 50% FINES*

GROUP SYMBOLS	ILLUSTRATIVE GROUP NAMES	MAJOR DIVISIONS
GW	Well graded gravel Well graded gravel with sand	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size
GP	Poorly graded gravel Poorly graded gravel with sand	
GM	Silty gravel Silty gravel with sand	
GC	Clayey gravel Clayey gravel with sand	
SW	Well graded sand Well graded sand with gravel	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
SP	Poorly graded sand Poorly graded sand with gravel	
SM	Silty sand Silty sand with gravel	
SC	Clayey sand Clayey sand with gravel	

NOTE: Coarse-grained soils receive dual symbols if:
(1) their fines are CL-ML (e.g. SC-SM or GC-GM) or
(2) they contain 5-12% fines (e.g. SW-SM, GP-GC, etc.)

FINE-GRAINED SOILS

MORE THAN 50% FINES*

GROUP SYMBOLS	ILLUSTRATIVE GROUP NAMES	MAJOR DIVISIONS
CL	Lean clay Sandy lean clay with gravel	SILTS AND CLAYS liquid limit less than 50
ML	Silt Sandy silt with gravel	
OL	Organic clay Sandy organic clay with gravel	
CH	Fat clay Sandy fat clay with gravel	SILTS AND CLAYS liquid limit more than 50
MH	Elastic silt Sandy elastic silt with gravel	
OH	Organic clay Sandy organic clay with gravel	
PT	Peat Highly organic silt	HIGHLY ORGANIC SOIL

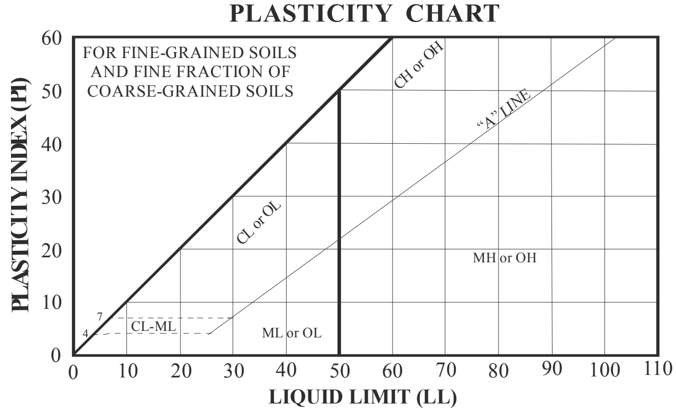
NOTE: Fine-grained soils receive dual symbols if their limits in the hatched zone on the Plasticity Chart(L-M)

SOIL SIZES

COMPONENT	SIZE RANGE
BOULDERS	ABOVE 12 in.
COBBLES	3 in. to 12 in.
GRAVEL	No. 4 to 3 in.
Coarse	¾ in to 3 in.
Fine	No. 4 to ¾ in.
SAND	No. 200 to No.4
Coarse	No. 10 to No. 4
Medium	No. 40 to No. 10
Fine	No. 200 to No. 40
*FINES:	BELOW No. 200

NOTE: Classification is based on the portion of a sample that passes the 3-inch sieve.

PLASTICITY CHART



Reference: ASTM D 2487-06, Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System).

GENERAL NOTES: The tables list 30 out of a possible 110 Group Names, all of which are assigned to unique proportions of constituent soils. Flow charts in ASTM D 2487-06 aid assignment of the Group Names. Some general rules for fine grained soils are: less than 15% sand or gravel is not mentioned; 15% to 25% sand or gravel is termed "with sand" or "with gravel", and 30% to 49% sand or gravel is termed "sandy" or "gravelly". Some general rules for coarse-grained soils are: uniformly-graded or gap-graded soils are "Poorly" graded (SP or GP); 15% or more sand or gravel is termed "with sand" or "with gravel", 15% to 25% clay and silt is termed clayey and silty and any cobbles or boulders are termed "with cobbles" or "with boulders".

UNIFIED SOIL CLASSIFICATION SYSTEM

SOIL TYPES (Ref 1)

Boulders: particles of rock that will not pass a 12-inch screen.
Cobbles: particles of rock that will pass a 12-inch screen, but not a 3-inch sieve.
Gravel: particles of rock that will pass a 3-inch sieve, but not a #4 sieve.
Sand: particles of rock that will pass a #4 sieve, but not a #200 sieve.
Silt: soil that will pass a #200 sieve, that is non-plastic or very slightly plastic, and that exhibits little or no strength when dry.
Clay: soil that will pass a #200 sieve, that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when dry.

MOISTURE AND DENSITY

Moisture Condition: an observational term; dry, moist, wet, or saturated.
Moisture Content: the weight of water in a sample divided by the weight of dry soil in the soil sample, expressed as a percentage.
Dry Density: the pounds of dry soil in a cubic foot of soil.

DESCRIPTORS OF CONSISTENCY (Ref 3)

Liquid Limit: the water content at which a soil that will pass a #40 sieve is on the boundary between exhibiting liquid and plastic characteristics. The consistency feels like soft butter.
Plastic Limit: the water content at which a soil that will pass a #40 sieve is on the boundary between exhibiting plastic and semi-solid characteristics. The consistency feels like stiff putty.
Plasticity Index: the difference between the liquid limit and the plastic limit, i.e. the range in water contents over which the soil is in a plastic state.

MEASURES OF CONSISTENCY OF COHESIVE SOILS (CLAYS) (Ref's 2 & 3)

Very Soft	N=0-1*	C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000 psf	Dented slightly by a pencil point

***N**=blows per foot in the Standard Penetration Test. In cohesive soils, with the 3-inch-diameter ring sampler, 140-pound weight, divide the blow count by 1.2 to get N (Ref 4).

MEASURES OF RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS, AND SILTS) (Ref's 2 & 3)

Very Loose	N=0-4**	RD=0-30	Easily push a ½-inch reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a ½-inch reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a ½-inch reinforcing rod
Dense	N=31-50	RD=70-90	Drive a ½-inch reinforcing rod 1 foot
Very Dense	N>50	RD=90-100	Drive a ½-inch reinforcing rod a few inches

****N**=Blows per foot in the Standard Penetration Test. In granular soils, with the 3-inch-diameter ring sampler, 140-pound weight, divide the blow count by 2 to get N (Ref 4).

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Ref 1: ASTM Designation: D 2487-06, **Standard Classification of Soils for Engineering Purposes** (Unified Soil Classification System).

Ref 2: Terzaghi, Karl, and Peck, Ralph B., **Soil Mechanics in Engineering Practice,** John Wiley & Sons, New York, 2nd Ed., 1967, pp. 30, 341, and 347.

Ref 3: Sowers, George F., **Introductory Soil Mechanics and Foundations: Geotechnical Engineering,** Macmillan Publishing Company, New York, 4th Ed., 1979, pp. 80, 81, and 312.

Ref 4: Lowe, John III, and Zaccheo, Phillip F., **Subsurface Explorations and Sampling,** Chapter 1 in "Foundation Engineering Handbook," Hsai-Yang Fang, Editor, Van Nostrand Reinhold Company, New York, 2nd Ed, 1991, p. 39.

SOIL TERMINOLOGY

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SOIL TERMINOLOGY

GENERAL NOTES FOR BORING LOGS:

The boring logs are intended for use only in conjunction with the text, and for only the purposes the text outlines for our services. The Plate "Soil Terminology" defines common terms used on the boring logs.

The plate "Unified Soil Classification System," illustrates the method used to classify the soils. The soils were visually classified in the field; the classifications were modified by visual examination of samples in the laboratory, supported, where indicated on the logs, by tests of liquid limit, plasticity index, and/or gradation. In addition to the interpretations for sample classification, there are interpretations of where stratum changes occur between samples, where gradational changes substantively occur, and where minor changes within a stratum are significant enough to log.

There may be variations in subsurface conditions between borings. Soil characteristics change with variations in moisture content, with exchange of ions, with loosening and densifying, and for other reasons. Groundwater levels change with seasons, with pumping, from leaks, and for other reasons. Thus boring logs depict interpretations of subsurface conditions only at the locations indicated, and only on the date(s) noted.

SPECIAL FIELD NOTES FOR THIS REPORT:

1. The borings for this investigation were advanced on May 21 and 31, 2014, with a portable "Minuteman" drilling rig utilizing 3½-inch-diameter continuous flight augers.

2. The boring locations were approximately located with a measuring tape from the existing site features such as curbs, walkways, trees, light fixtures, manholes, interior walls, doors, etc. Boring elevations were estimated from the floor level elevation data by Kier & Wright Civil Engineers & Surveyors, and from Google Earth.

3. The soils' Group Names [e.g. LEAN CLAY] and Group Symbols [e.g. (CL)] were determined or estimated per ASTM D 2487, Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System, see Plate 5). Other soil engineering terms used on the boring logs are defined on Plate 6, Soil Terminology.

4. Groundwater was not encountered in any of the four borings advanced at the site for this investigation.

5. The undisturbed soil samples were obtained in a 2½-inch-ID, 3-inch-OD, Modified California sampler lined with 1-inch-long brass rings (ASTM D 3550), and in 1⅜-inch-OD Standard Penetration Test sampler (ASTM D1586). The samples were driven with a 140-pound hammer free-falling approximately 30 inches (ASTM D3350 and ASTM D1586).

6. The "Blow Count" Column on the boring logs indicates the number of blows required to drive the Modified California and Standard Penetration Test samplers below the bottom of the boring, with the blow counts given for each 6 inches of sampler penetration.

7. The tabulated strength values on the boring logs are yield strength values, or where the soil begins to deform plastically.

BORING LOG NOTES

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Clayey sand
	Silty sand
	Lean clay with fine sand
	Lean Clay
	Lean to fat clay with sand
	Moderate to high plasticity clay
	Concrete
	Gravelly clay with sand
	Well graded sand
Misc. Symbols	
	Drilling refusal
	Boring continues
Soil Samplers	
	Modified California Sampler: 2.375" ID by 3" OD, split-barrel sampler driven w/ 140-pound hammer falling 30 inches

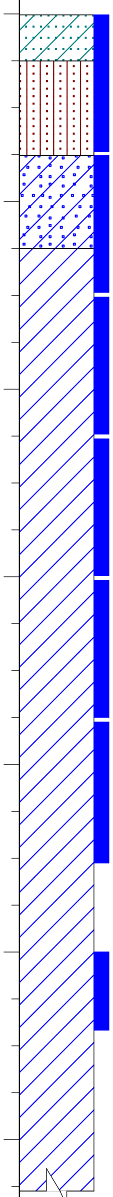
Symbol	Description
	Standard Penetration Test: 1 3/8" ID by 2" OD, split-spoon sampler driven with 140-pound hammer falling 30" (ASTM D 1586-99)
Line Types	
	Denotes a sudden, or well identified strata change
	Denotes a gradual, or poorly identified strata change
Laboratory Data	
DS	Direct shear test performed on a sample at natural moisture content (ASTM D3080 Mod).
DSX	Direct shear test performed on a sample at artificially increased moisture content (ASTM D3080 Mod).
LL	Denotes the Liquid Limit per ASTM D4318.
PI	Denotes the Plasticity Index per ASTM D4318.
Fines	Denotes a wash over Sieve No. 200 measuring percent fines per ASTM D1140.

BORING LOG

Boring No. B-1
Page 1 of 2

JOB NAME: VP01
CLIENT: Apple Inc.
LOCATION: 19191 Vallco Parkway, Cupertino, CA
DRILLER: Access Soil Drilling
DRILL METHOD: Portable Minuteman w/ 3½" Continuous Flight Augers

JOB NO.: APPLE-20-00
DATE DRILLED: 5/21/2014
ELEVATION: ≅ 182± feet
LOGGED BY: KO
CHECKED BY:



Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Blow Counts	USCS	Description	Remarks
				13.9	109	0	10	SC	CLAYEY Fine SAND, dark yellow-brown, medium dense, moist to dry	Fill LL=37, PI=23, Fines= 40%
							19	SM	SILTY SAND, gray-brown, medium desne, dry, trace gravel, medium plasticity	
DSX	320	22.0	850	14.9	115	2	16 15 20	CL	SANDY LEAN CLAY, dark yellow-brown, very stiff, dry to moist, fine-to-coarse grained sand, medium plasticity	Native Swelled 8.2%
							21 19	CL	LEAN CLAY, dark brown, stiff to very stiff, dry, some sand pockets & oxidation staining, medium plasticity	
DSX DS	500 500	19.9 Nat.	860 1670	13.8 10.5	115 116	4	35			Swelled 5% LL=33, PI=18
DSX DS DS	600 600 2000	20.0 Nat. Nat.	1500 4400 5400	14.7 16.0 13.9	115 114 116		29 15 36		...grades hard	Swelled 5.6%
							43 20		...some medium-to-coarse grained sand	
DSX DS	800 800	18.5 Nat.	2000 2900	13.2 16.4	118 113	6	45		...decrease in fine sand content & very stiff	Swelled 4.1%
							60		...hard	
				13.9	114	8	28 49 50/5"		...some medium-to-coarse-grained sand	
				11.4	125	10	38 50/4"		...grades yellow-brown	
						12				

BORING LOG

JOB NAME: VP01

JOB NO.: APPLE-20-00

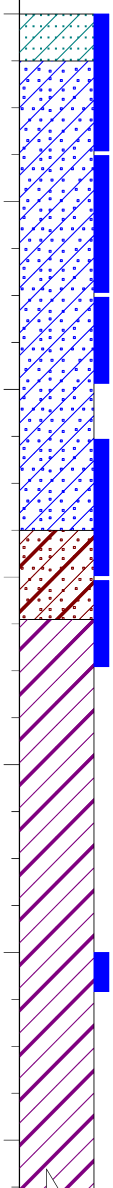
Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
				20.2	107	14	32 50/5"	CL	LEAN CLAY, yellow-brown, hard, moist	
						16 18 20 22 24 26			Boring was terminated at 15 feet due to refusal. Groundwater was not encountered. Borehole was backfilled with neat cement grout.	

BORING LOG

Boring No. B-2

JOB NAME: VP01
CLIENT: Apple Inc.
LOCATION: 19191 Vallco Parkway, Cupertino, CA
DRILLER: Access Soil Drilling
DRILL METHOD: Portable Minuteman w/ 3½" Continuous Flight Augers

JOB NO.: APPLE-20-00
DATE DRILLED: 5/21/2014
ELEVATION: ≅ 182± feet
LOGGED BY: CZ
CHECKED BY:



Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
				8.0		0	12	SC	CLAYEY SAND, brown, medium dense, dry	Fill?
							20	CL	LEAN CLAY with Fine Sand, brown, hard, dry, some fine to-medium grained & subangular gravel, trace medium-to-coarse grained sand	Fines=40% Native
DSX	320	20.0	550	13.4	118		27 30			LL=33, PI=19 Swelled 7.3%
						2	32 36		...trace oxidation stains & caliche	
				15.6	112		45 50/5"		...decreasing gravel content	Shrink-Swell
DS	600	Nat.	4000	13.6	110	4				
				12.3	116		33 41			Sand=15%
DS	900	Nat.	4800	14.3	116	6	44 44	CL/ CH	SANDY LEAN to FAT CLAY, brown, hard, moist, moderately to highly plastic	LL=45, PI=31 Fines=67%
							50/5"	CL/ CH	LEAN to FAT CLAY, yellow-brown, hard, dry, trace fine-grained sand, moderately to highly plasic	
						8				
				12.1	119	10	50/5"			
						12				

BORING LOG

JOB NAME: VP01

JOB NO.: APPLE-20-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
								CL	LEAN CLAY, yellow-brown, hard	
				17.3	110	14	43 50/3"			
				21.7			25 23 27			
						16				
									Refusal occurred at 16½ feet bgs. Groundwater was not encountered. Borehole was backfilled with neat cement grout.	
						18				
						20				
						22				
						24				
						26				

BORING LOG

Boring No. B-3
Page 1 of 2

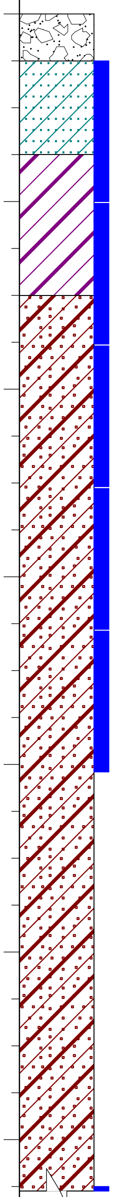
JOB NAME: VP01
CLIENT: Apple Inc.
LOCATION: 19191 Vallco Parkway, Cupertino, CA
DRILLER: Access Soil Drilling
DRILL METHOD: Portable Minuteman w/ 3½" Continuous Flight Augers

JOB NO.: APPLE-20-00
DATE DRILLED: 5/31/2014
ELEVATION:
LOGGED BY: KO
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf
DS	320	Nat.	1250	19.1	105
DSX	320	22.8	1035	21.3	105
DSX	1500	21.8	1340	21.0	104
DS	600	Nat.	1800	21.8	99
DSX	600	19.5	1040	18.4	112
DS	650	Nat.	1160	22.8	98
DSX	650	22.5	790	21.0	105
DS	2000	Nat.	1600	22.5	94
DS	750	Nat.	1300	20.2	104

Depth, ft.: 0, 2, 4, 6, 8, 10, 12

Soil Symbols, Samplers and Blow Counts



USCS	Description	Remarks
	6" thick Concrete Floor	
SC	CLAYEY SAND, gray-brown, moist, with some gravel	Fill
CL/ CH	LEAN to FAT CLAY, dark brown, trace fine grained sand and gravel, stiff, moist, medium to high plasticity	Native, LL=50, PI=32 Swelled 0.9%
CL/ CH	LEAN to FAT CLAY with Fine Sand, dark gray, stiff, moist, medium plasticity	Shrink-Swell LL=46, PI=29 Sand=23%
	...color changes to dark yellow-brown, litte fine-to-medium sand	LL=50, PI=34
	...grading stiffer	

BORING LOG

Boring No. B-3
Page 2 of 2

JOB NAME: VP01 *JOB NO.:* APPLE-20-00

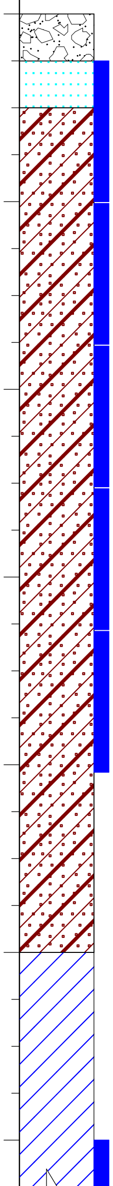
Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
DS	1600	Nat.	2500	21.6	103		15 22 32	CL/ CH	LEAN to FAT CLAY with Fine Sand, very stiff	
						14				
								CL	GRAVELLY CLAY with Sand, yellow-brown, hard, moist	
						16				
DS	2200	Nat.	2200	8.3	118	18	22 25 25			
									Boring was terminated at 19 feet & backfilled with cement grout. Top of boring was sealed with Sac-Crete. Groundwater was not encountered.	
						20				
						22				
						24				
						26				

BORING LOG

Boring No. B-4

JOB NAME: VP01
CLIENT: Apple Inc.
LOCATION: 19191 Vallco Parkway, Cupertino. CA
DRILLER: Access Soil Drilling
DRILL METHOD: Portable Minuteman w/ 3½" Continuous Flight Augers

JOB NO.: APPLE-20-00
DATE DRILLED: 5/31/2014
ELEVATION:
LOGGED BY: KO
CHECKED BY:



Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0			6" thick Concrete Floor	
							7	SW	WELL-GRADED SAND with Gravel & Silt, gray, dry	Fill
							15	CL/ CH	LEAN to FAT CLAY with Fine Sand, dark brown, hard, moist to dry, trace gravel, moderate to high plasticity	Native
DSX	320	19.9	1330	14.7	116	2	25 22			LL=49, PI=31 Swelled 5%
							25			
DSX	500	18.1	1340	12.2	113	4	32 23		...interbedded with less plastic light yellow-brown clay	LL=43, PI=26 Swelled 2%
							26			
DS DS DSX	600 2000 700	Nat. Nat. 19.3	5400 5700 2050	14.0 14.0 13.6	119 120 117		30 17			LL=45, PI=28 Swelled 5.5% Sand=18%
						6	24			
DS DS	800 2200	Nat. Nat.	4500 6000	11.4 11.6	124 124		31 24			
							30			
						8	38			
						10		CL	LEAN CLAY, yellow-brown, hard, dry to moist, trace fine sand	
						12	17			

BORING LOG

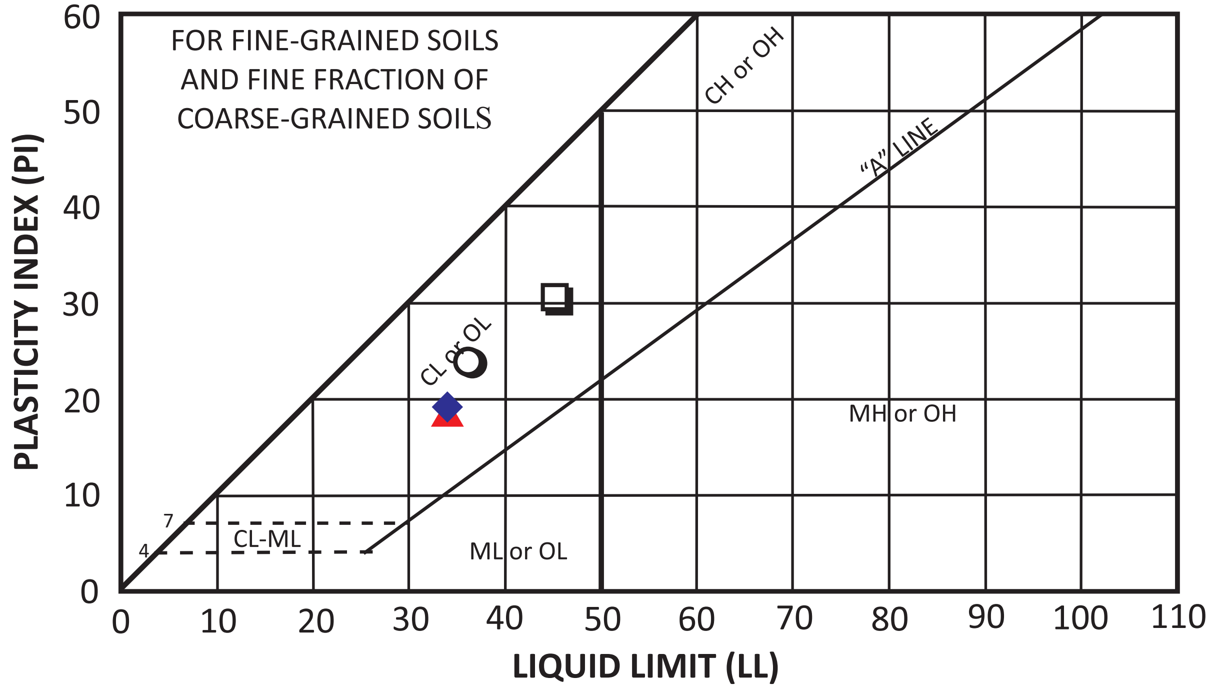
Boring No. B-4

JOB NAME: VP01

JOB NO.: APPLE-20-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
DS	1500	Nat.	5000	17.4	109		30 44		LEAN CLAY, hard ...near refusal	
						14			Boring was terminated at 13½ feet and backfilled with neat cement grout. Top of boring was sealed with Sac-Crete. Groundwater was not encountered.	
						16				
						18				
						20				
						22				
						24				
						26				

PLASTICITY CHART



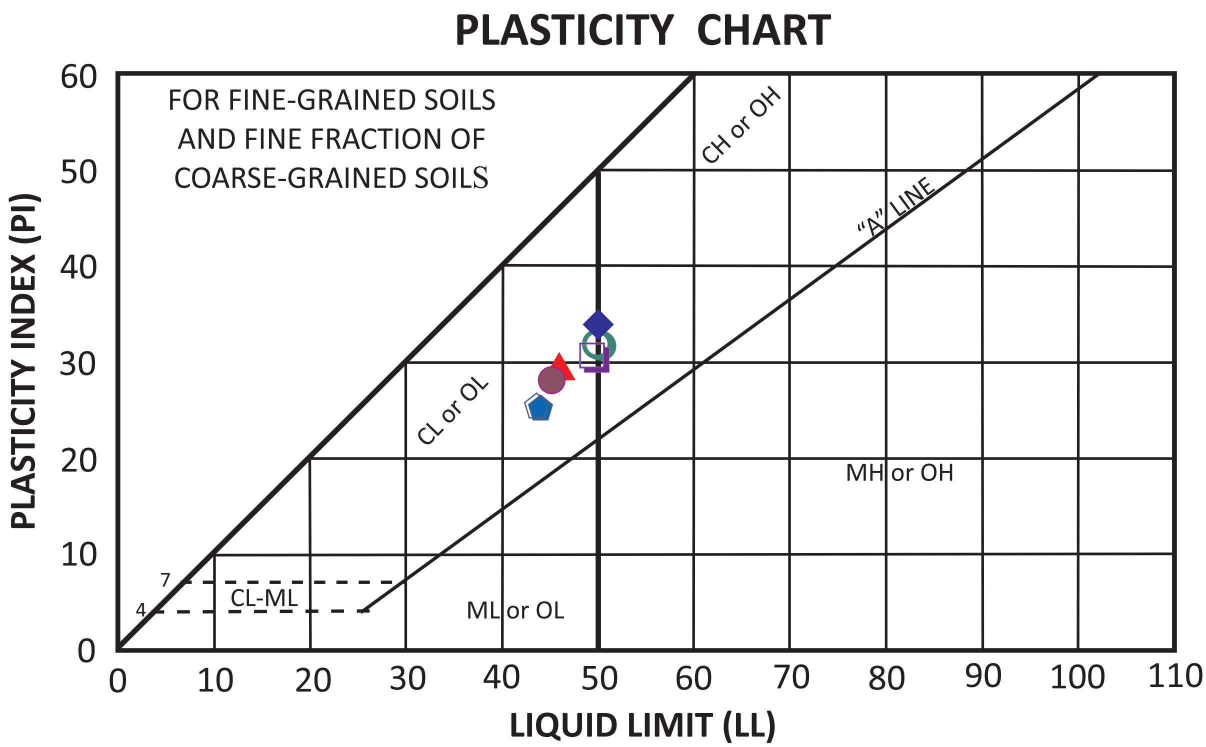
SYMBOL	SAMPLE SOURCE	DEPTH (FEET)	NATURAL WATER CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SOIL DESCRIPTION
○	Boring B-1	0	13.9	37	14	23	Dark Yellow-Brown Clayey Sand (SC)
▲	Boring B-1	3.5	10.5	33	15	18	Dark Brown Lean Clay (CL)
◆	Boring B-2	1.5	13.4	33	14	19	Brown Lean Clay with Fine Sand (CL)
□	Boring B-2	6	14.3	45	14	31	Brown Lean to Fat Clay (CL/CH)

GEOTECHNICAL ENGINEERING INVESTIGATION
BUILDING MOVEMENT & DISTRESS EVALUATION
APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CALIFORNIA

PLASTICITY DATA

DATE:	JOB NUMBER:	PLATE
July 2014	APPLE-20-00	13-A

BAGG ENGINEERS



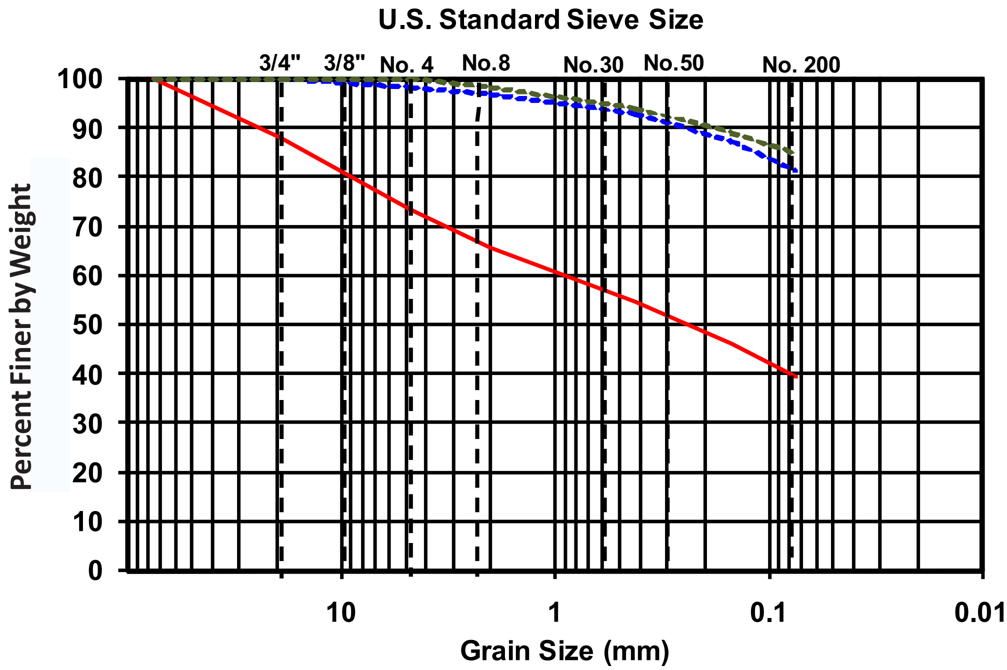
SYMBOL	SAMPLE SOURCE	DEPTH (FEET)	NATURAL WATER CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SOIL DESCRIPTION
○	Boring B-3	2	19.1	50	18	32	Dark Brown Lean to Fat Clay (CL/CH)
▲	Boring B-3	5	21.8	46	17	29	Dark Brown Lean to Fat Clay (CL/CH)
◆	Boring B-3	6.5	20.2	50	16	34	Dark Yell-Brown Lean to Fat Clay (CL/CH)
□	Boring B-4	2	14.7	49	18	31	Dark Brown Lean to Fat Clay (CL/CH)
⬟	Boring B-4	3.5	18.1	43	17	26	Dark Brown Lean to Fat Clay (CL/CH)
●	Boring B-4	5	14.0	45	17	28	Dark Brown Lean to Fat Clay (CL/CH)

GEOTECHNICAL ENGINEERING INVESTIGATION
BUILDING MOVEMENT & DISTRESS EVALUATION
APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CALIFORNIA

PLASTICITY DATA

DATE:	JOB NUMBER:	PLATE
July 2014	APPLE-20-00	13-B

BAGG ENGINEERS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	COARSE	

LEGEND	red solid line	green dashed line	blue dashed line
BORING NUMBER	B-2	B-2	B-4
DEPTH (FEET)	0	4.5'	5.5
SOIL DESCRIPTION	Clayey Sand (SC)	Lean Clay with Fine Sand (CL)	Lean to Fat Clay with Fine Sand (CL/CH)
EFFECTIVE SIZE, D_{10}			
COEFFICIENT OF UNIFORMITY D_{60}/D_{10}			
COEFFICIENT OF CURVATURE $D_{30}^2/D_{10} \times D_{60}$			

GEOTECHNICAL ENGINEERING INVESTIGATION
BUILDING MOVEMENT & DISTRESS EVALUATION
APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CALIFORNIA

GRADATION TEST DATA

DATE	JOB NO.	PLATE
July 2014	APPLE-20-00	14

BAGG ENGINEERS

SAMPLE DESCRIPTION	TEST SURCHARGE (PSF)	MOISTURE CONDITION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT EXPANSION
B-1 @ 1½ ft. SANDY LEAN CLAY (CL)	300	NATURAL	15.2	112.8	--
		SATURATION	21.9	106.5	5.9
		AIR DRY	10.4	127.7	-8.1
		OVEN DRY	0.0	128.4	-12.2
B-2 @ 3 ft. LEAN CLAY with Fine Sand (CL)	300	NATURAL	15.6	112.1	--
		AFTER SATURATION	23.4	103.7	8.2
		AIR DRY	10.7	125.0	-10.2
		OVEN DRY	0.0	130.7	-14.2
		NATURAL			
		AFTER SATURATION			
		AIR DRY			
		OVEN DRY			
		NATURAL			
		AFTER SATURATION			
		AIR DRY			
		OVEN DRY			
		NATURAL			
		AFTER SATURATION			
		AIR DRY			
		OVEN DRY			

DESCRIPTION OF SHRINK-SWELL TEST PROCEDURE

An undisturbed sample of soil, at its natural moisture content, confined in the 1-inch-high, 2.375-inch-ID cylinder in which it was obtained in the field, is immersed in water while under a surcharge pressure. Measurements of shrink or swell are taken until movement ceases. The surcharge is removed and the sample is air dried, then oven dried. By measuring the dimensions of the sample under these various conditions, it is possible to determine the soil volume under the following conditions: 1) at field moisture content, 2) when completely saturated under the given surcharge, 3) when air dry, and 4) when oven dry. The dry density is computed from the dry weight of the specimen and its volume under the various moisture conditions. The percent expansion, relative to the natural field volume of the sample, is directly related to the various volumes and inversely related to the various dry densities of the sample.

SHRINK–SWELL TEST DATA

Shrink-Swell / Expansion Pressure
ASTM D 3877m

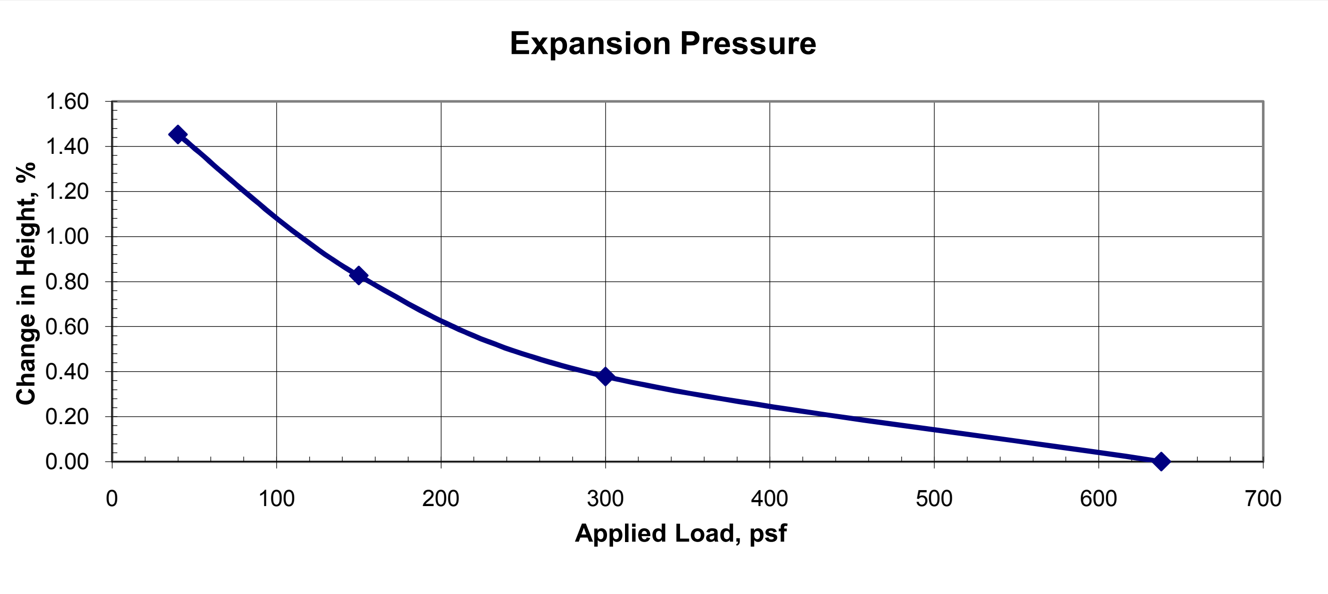
Job No.:	011-642		**LL**		**Date:**	6/18/2014	
Client:	BAGG		**PL**		**By:**	MD	
Project:	APPLE-20-00		**PI**		**Checked By:**	DC	Assumed / Determined
Boring:	B-3	**Sample:** 2	**Depth,ft:**	2.0	**Specific Gravity:**	2.75	

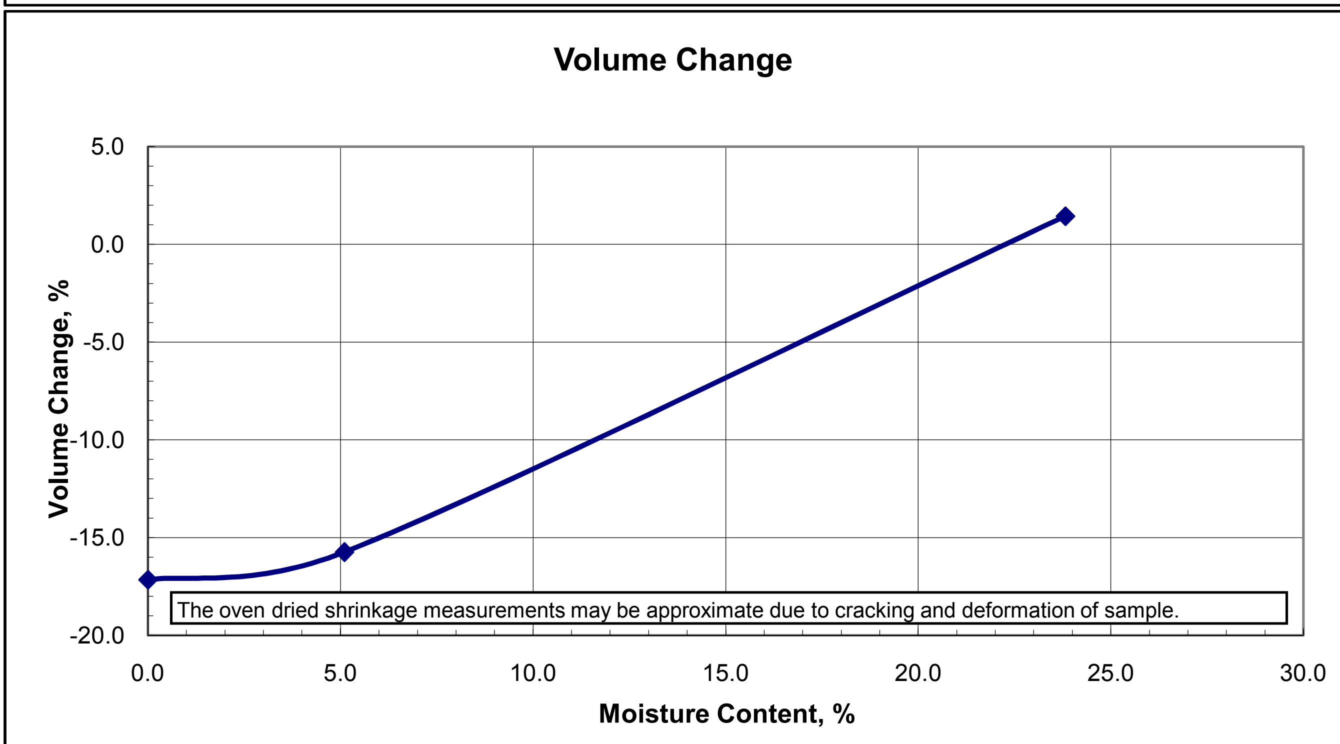
Soil Desc. -

Load, psf:	638	300	150	40
Exp., %	0.00	0.38	0.83	1.45

	Field	**Saturated**	**Air-Dry**	**Oven-Dry**
Moisture %:	21.8	23.8	5.1	0.0
Dry Density, pcf	105.3	103.8	125.0	127.1
Saturation, %	95.0	100.0	37.5	0.0
Void Ratio	0.631	0.655	0.375	0.351
Volume Change, %	0.0	1.5	-15.7	-17.2

Remarks:
Samples prepared by client prior to testing.





Shrink-Swell / Expansion Pressure
ASTM D 3877m

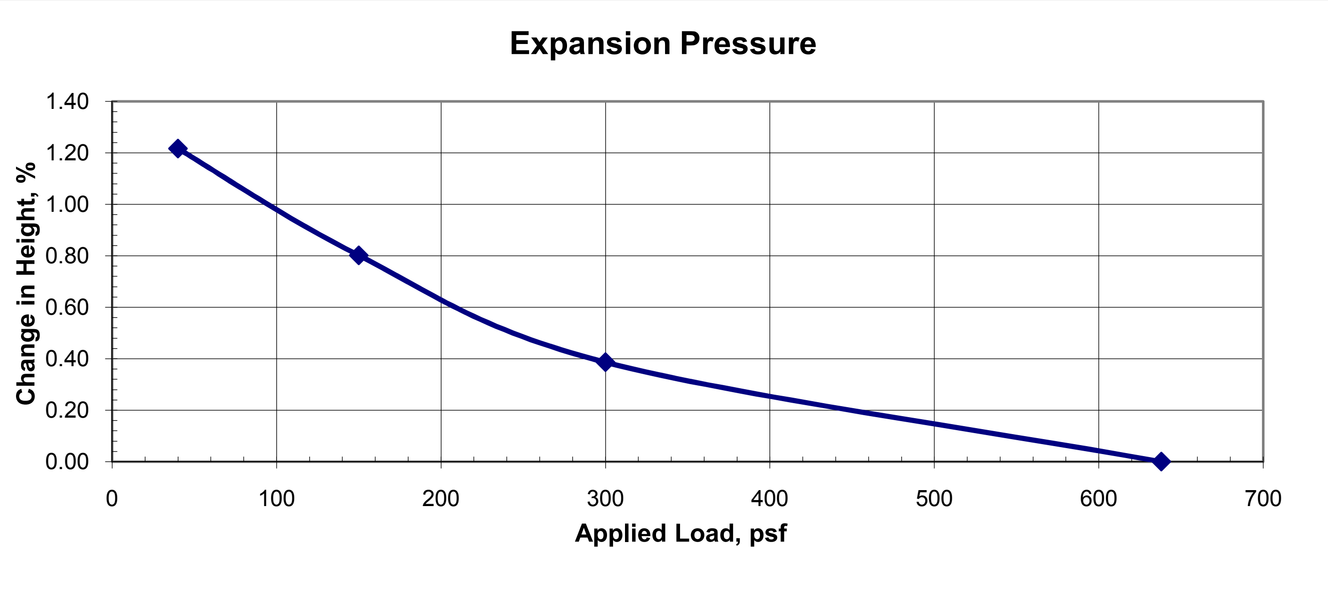
Job No.:	011-642		**LL**		**Date:**	6/18/2014	
Client:	BAGG		**PL**		**By:**	MD	
Project:	APPLE-20-01		**PI**		**Checked By:**	DC	Assumed / Determined
Boring:	B-3	**Sample:** 3a	**Depth,ft:**	4	**Specific Gravity:**	2.8	

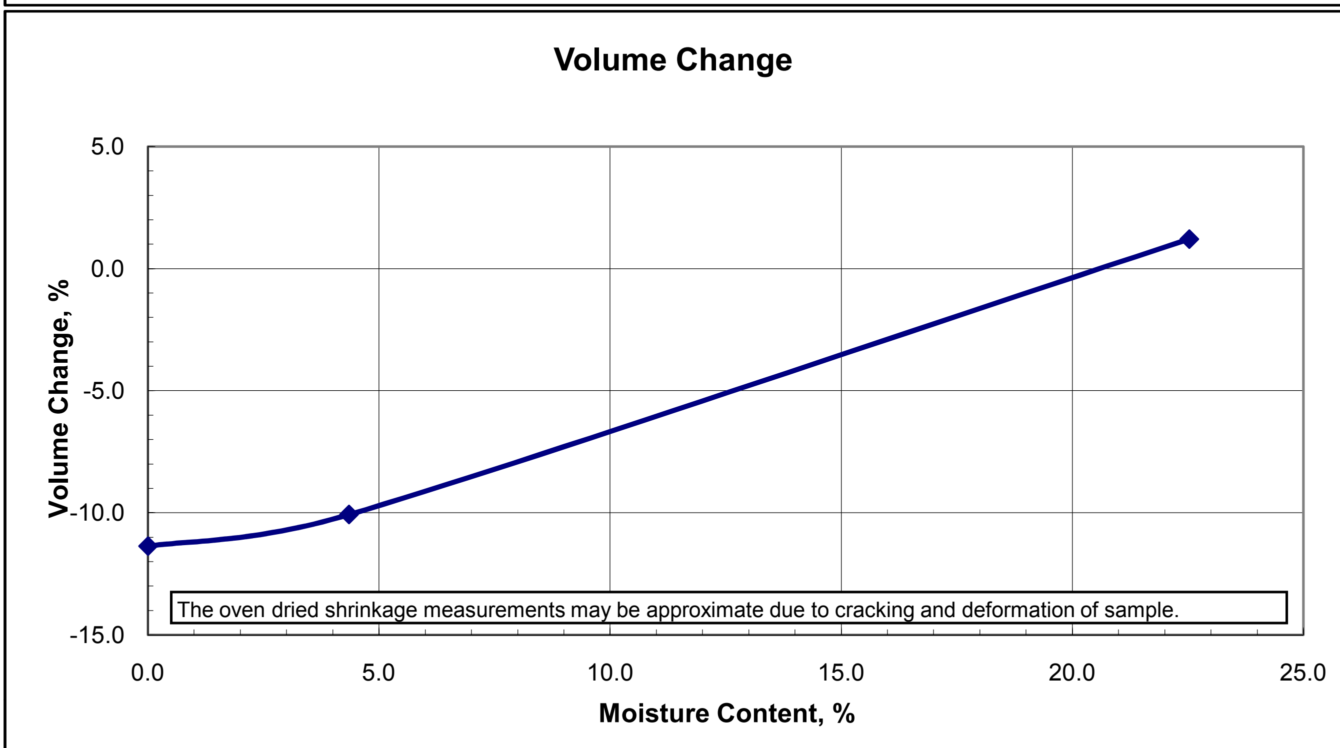
Soil Desc. -

Load, psf:	638	300	150	40
Exp., %	0.00	0.39	0.80	1.22

	Field	Saturated	Air-Dry	Oven-Dry
Moisture %:	19.4	22.5	4.4	0.0
Dry Density, pcf	108.2	106.9	120.4	122.1
Saturation, %	88.3	99.2	26.9	0.0
Void Ratio	0.616	0.636	0.453	0.432
Volume Change, %	0.0	1.2	-10.1	-11.4

Remarks:
Samples prepared by client prior to testing.





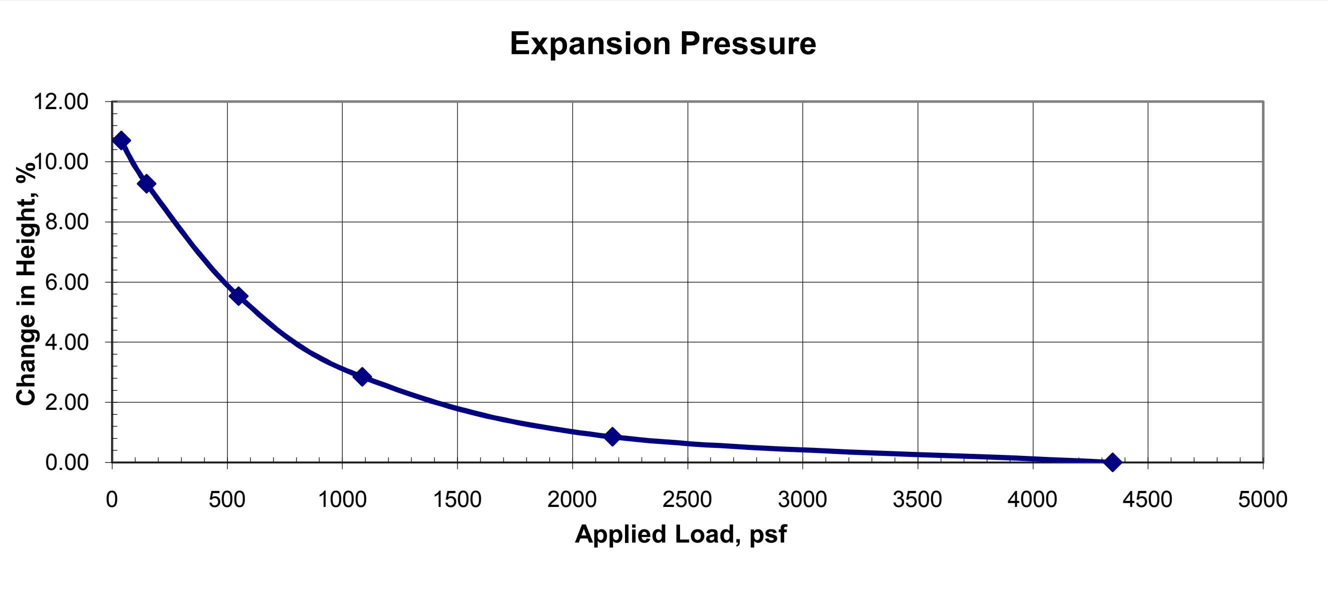
Shrink-Swell / Expansion Pressure
ASTM D 3877m

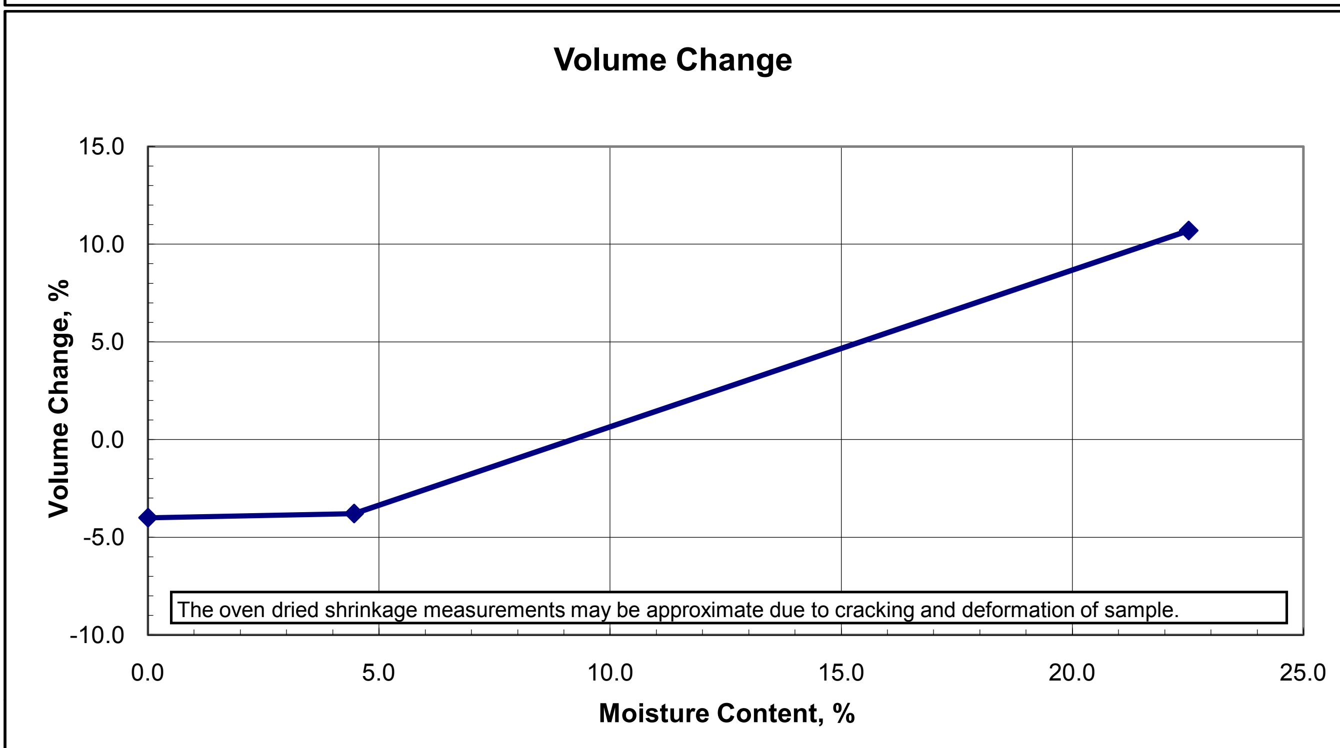
Job No.:	011-642	LL		Date:	6/24/2014
Client:	BAGG	PL		By:	MD
Project:	APPLE-02-00	PI		Checked By:	DC
Boring:	B-4	Sample: 2	Depth,ft: 2	Specific Gravity:	2.75
Soil Desc. -				Assumed	Determined

Load, psf:	4347	2174	1087	550	150	40
Exp., %	0.00	0.85	2.85	5.53	9.27	10.71

	Field	Saturated	Air-Dry	Oven-Dry
Moisture %:	14.7	22.5	4.5	0.0
Dry Density, pcf	116.4	105.2	121.0	121.3
Saturation, %	84.9	97.7	29.2	0.0
Void Ratio	0.476	0.634	0.420	0.417
Volume Change, %	0.0	10.7	-3.8	-4.0

Remarks:
Samples prepared by client prior to testing.





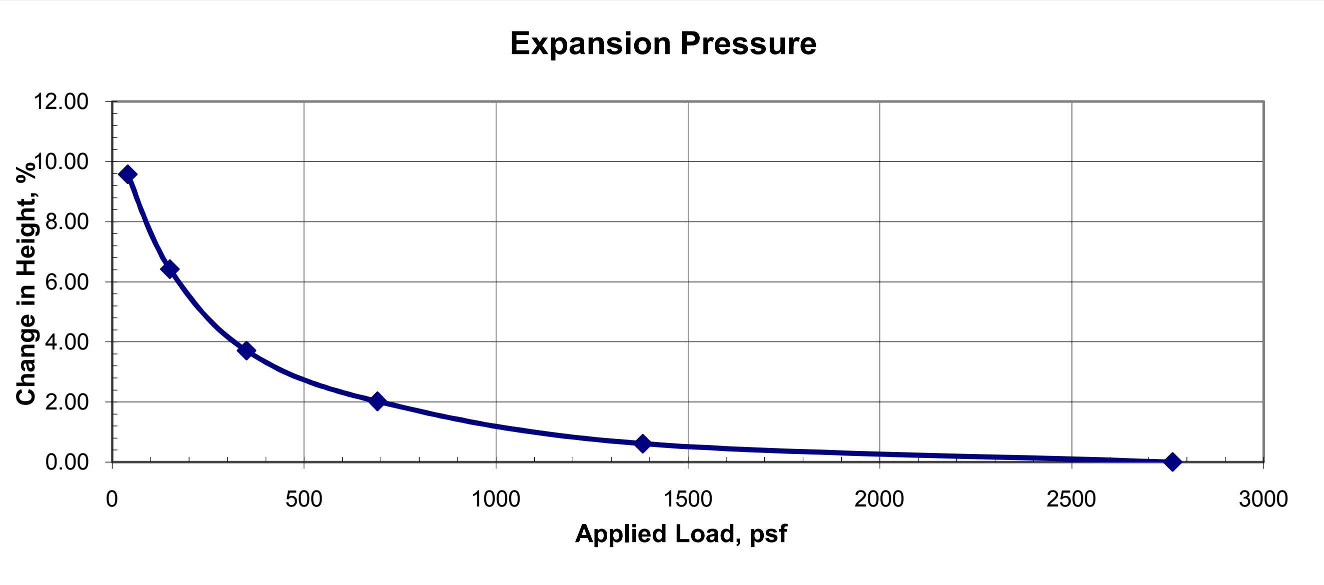
Shrink-Swell / Expansion Pressure
ASTM D 3877m

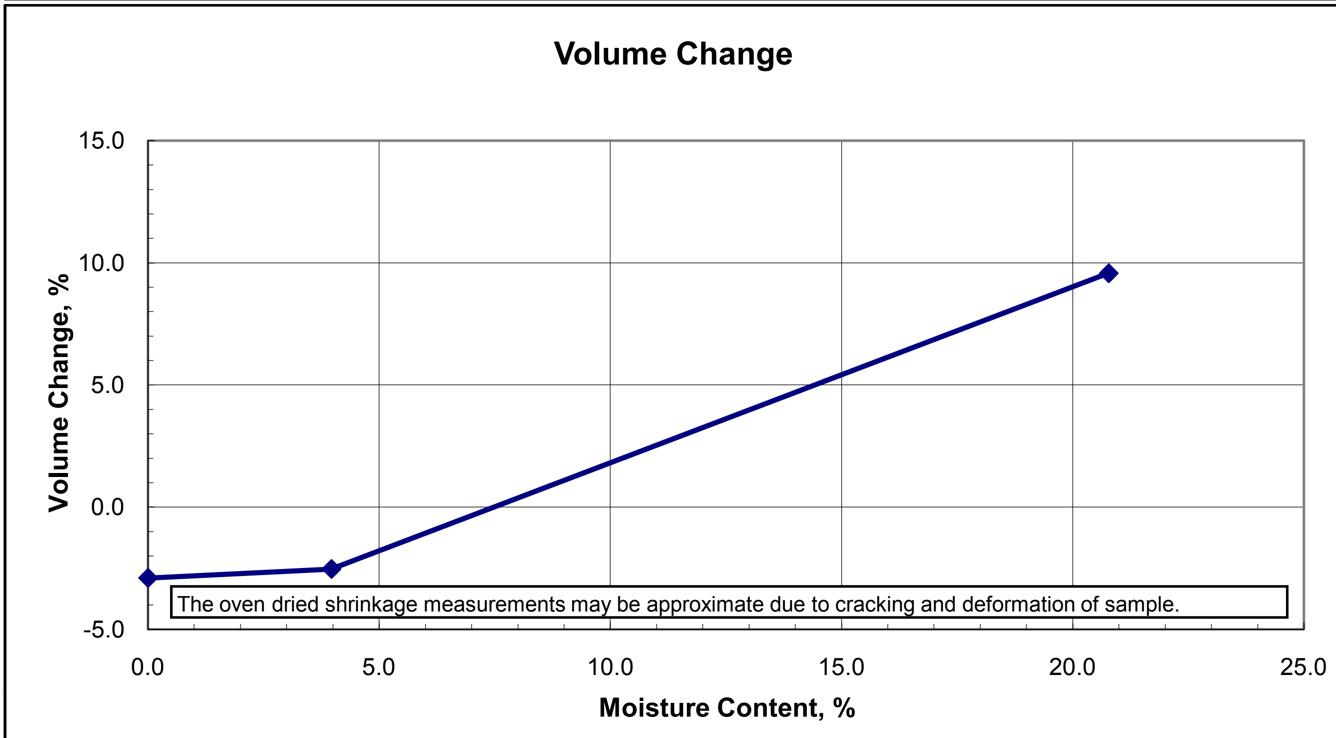
Job No.:	011-642		**LL**		**Date:**	6/24/2014	
Client:	BAGG		**PL**		**By:**	MD	
Project:	APPLE-02-00		**PI**		**Checked By:**	DC	Assumed / Determined
Boring:	B-4	**Sample:** 3	**Depth,ft:**	3.5	**Specific Gravity:**	2.7	
Soil Desc. -							

Load, psf:	2763	1382	691	350	150	40
Exp., %	0.00	0.61	2.02	3.71	6.42	9.58

	Field	Saturated	Air-Dry	Oven-Dry
Moisture %:	13.4	20.8	4.0	0.0
Dry Density, pcf	117.9	107.6	120.9	121.4
Saturation, %	84.1	98.7	27.1	0.0
Void Ratio	0.431	0.568	0.395	0.390
Volume Change, %	0.0	9.6	-2.5	-2.9

Remarks:
Samples prepared by client prior to testing.





Bright People. Right Solutions.

Appendix D

March 24, 2021
File No. 20213246.001A

APPLE, INC.
Real Estate & Development
One Apple Park Way, MS 952-31CP
Cupertino, CA 95014

Attention: Benjamin Louie
bloui@apple.com

SUBJECT: Site-Specific Ground Motion Hazard Analysis
Apple VP01
19191 Vallco Parkway
Cupertino, CA 95014

Reference: Kleinfelder, "Geotechnical Investigation Report, Apple VP01 19191 Vallco Parkway, Cupertino, CA 95014", draft report dated February 23, 2021, File No. SJO21R122528.

Benjamin Louie:

This letter presents the results of Kleinfelder's site-specific ground motion hazard analysis (GMHA) for the existing and proposed new office development located at 19191 Vallco Parkway in Cupertino, California. The scope of this study is to develop site-specific ground motion parameters using a site specific GMHA in terms of peak ground accelerations and response spectral accelerations. Analyses were conducted in accordance with the requirements of the 2019 California Building Code (CBC) which adopts the procedures outlined in ASCE 7-16 and Supplement 1 of that standard.

The scope of this analysis includes:

- Development of a site-specific earthquake source model in conformance with the current code requirements and current state of the practice.
- Performing site-specific ground motion hazard analyses per Section 21.2 of ASCE 7-16 consisting of probabilistic and deterministic seismic hazard analyses (PSHA and DSHA, respectively).
- Develop site-specific response spectra for the Risk-Targeted Maximum Considered Earthquake (MCE_R) and the Design Earthquake (DE) and to obtain seismic design parameters per Section 21.4 of ASCE 7-16.
- Preparation of this report presenting the results of the site-specific seismic hazard analyses.

KLEINFELDER 380 North 1st Street, Suite A, San Jose, CA 95112 **p |** 831.755.7900

This report is intended to support our current geotechnical study for the subject site and is subject to the same limitations as contained in the main report.

PROJECT LOCATION

The project site is located in Cupertino, California. The approximate coordinates of the project site used for the ground motion hazard analysis are:

Latitude:	37.3253° N
Longitude:	122.0077° W

SEISMOTECTONIC SETTING AND SEISMICITY

A brief discussion of the seismotectonic setting and historic seismicity is provided below. The regional seismotectonic setting and historic seismicity inform the development 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 in the Western United States (WUS) near the boundary between the Great Valley and Coast Range geomorphic provinces. Seismicity in this region is dominated by the northwest trending movement of Pacific Plate along the North American transform plate boundary. To the east, the Sierra Nevada-Great Valley block – considered an independent microplate - generally encompasses the entirety of the Sacramento Valley, beyond which as a zone of distributed shear known as the Walker Lane Belt (near California/Nevada border). Northward, in the pacific northwest, the Juan de Fuca plate is currently subducting below the North American plate in a region known as the Cascadia Subduction Zone (Humphreys and Coblentz, 2007; Unruh et al., 2003, Unruh and Humphrey, 2017).

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 system, and to a lesser extent by displacements on faults within the Walker Lane Belt (Unruh and Humphrey, 2017; Field et al., 2013).

Regional Faulting and Historic Seismicity

Figure 1 presents both active and inactive faults as mapped by Jennings and Bryant (2010). These faults were generally considered in development of independent seismogenic sources discussed in this report. Not all faults shown on the figure are considered independent seismogenic sources, with smaller or inactive faults generally excluded from consideration. A detailed discussion of the faults and associated fault hazards is provided in the referenced geotechnical report.

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. We also looked into CGS Map Sheet 49 (Toppozada et al., 2000) for M≥5 earthquakes in California.

Two faults are of particular significance to the site - the San Andreas fault, and the Hayward fault - due to their size, relatively high rate of activity, and close proximity to the site. The San Andreas fault has produced a series of earthquakes in recent history including the 1838 San Andreas Fault Earthquake (M_w 7.4) event located about 13 km from the site; and the "Great" San Francisco earthquake (M_w 7.9) located about 67 km to the northwest. The Hayward fault system also produced a series of signification events including the1868 Hayward (M_w 6.8) earthquake which was located about 42 km from the site. Other faults near the site include the Monte Vista – Shannon and Silver Creek faults, both of which have historically ruptured in the recent past. The largest recorded event on the Monte Vista – Shannon fault occurred in 1865 (M_w 6.5), and on the Silver Creek Fault in 1903 (M_w 5.8) and possibly 1911 (M_w 6.6) (USGS, 2019, 2020).

We note that some of the nearer faults, including the San Jose fault are not considered active or are not considered independent seismogenic sources in our model due to relative size or lack of activity. Historic seismicity and faulting within 100 km of the site is depicted on Figure 1.

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 (M6.7+) earthquake during the period of 2014 to 2043. This publication also shows that there is a 51 percent chance of M7+ earthquake and 20 percent chance of M7.5+ happening before 2043. These probabilities are significant and require mitigation. For individual faults, the probabilities of having an earthquake of M6.7+ are 22% and 20% for the San Andreas and the Monte Vista-Shannon faults, respectively. As has been seen in the past earthquakes such as the 1994 (M6.7) Northridge earthquake, that this level of shaking could cause significant damage to the built environment.

SUBSURFACE SITE CONDITIONS FOR SEISMIC STUDY

Site effects are typically modeled in GMHA based on the average shear wave velocity in the upper 100 feet (V_{S30}). For shear wave velocity estimates we utilized site specific measurements of shear wave velocity from seismic cone penetrometer testing. However, this data only extends to a depth of about 50 feet. As such, we supplemented this data at depth using correlations with cone tip resistance and blowcount. Based on these results we have estimated a V_{S30} of about 1,155 feet/sec (352 m/s) as reasonably representative of the site which is consistent with a Site Class D profile (near C/D boundary).

SITE SPECIFIC GROUND MOTION MODEL

A site-specific GMHA model is a useful tool in evaluation of potential ground motion hazard at a site. The model generally includes a representative seismic source model (geometry, style of faulting, magnitude, etc.), appropriate recurrence relationships, and appropriate ground motion models (aka. attenuation relationships). The model can be used to quantify the potential for strong ground shaking at a site including the mean peak ground acceleration (PGA_M) and spectral accelerations (S_a). For this work, the model used was developed consistent with the requirements of Section 21.2 of ASCE 7-16 and the 2019 CBC. Details of the model used in this study are described below.

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 as the base model for our evaluations. Within California this model utilizes the third uniform California earthquake rupture forecast model (UCERF3) which utilizes two alternative fault models (FM 3.1 and 3.2) to model on-fault seismicity. The branch averaged solution was utilized for this work. The Petersen et al. (2014) source model has been used in developing the 2014 USGS National Seismic Hazard. Off-fault seismicity (e.g. background seismicity) is modeled using gridded seismic sources.

Fault sources from the regional model within 300 km of the site have been included in the model, with intraslab subduction earthquake sources included out to 1000 km as recommended by the USGS (Petersen et al., 2014). The source model used for this work is shown on Figure 2 with significant fault sources listed in Table 1 (only within 60 km of the site for brevity).

TABLE 1: SIGNIFICANT INDEPENDENT SEISMOGENIC FAULT SOURCES WITHIN 60 KM

Segment Name	Closest Distance (km)	Length (km)	Ave Dip	FM 3.1	FM 3.2	M_{max}
Monte Vista - Shannon 2011 CFM	5	60	61	TRUE	TRUE	7.1
Silver Creek 2011 CFM	10	48	75	TRUE	TRUE	6.9
San Andreas (Peninsula) 2011 CFM	11	100	90	TRUE	TRUE	7.4
Pilarcitos 2011 CFM	14	51	81	TRUE	TRUE	6.9
Butano 2011 CFM	16	46	70	TRUE	TRUE	7.0
San Andreas (Santa Cruz Mts) 2011 CFM	17	63	79	TRUE	TRUE	7.2
Hayward (So) 2011 CFM	18	54	76	TRUE	TRUE	7.1
Hayward (So) extension 2011 CFM	20	23	48	TRUE	TRUE	6.4
Sargent 2011 CFM	21	57	90	TRUE	TRUE	7.0
Calaveras (Central) 2011 CFM	22	52	77	TRUE	TRUE	7.0
Calaveras (No) 2011 CFM	22	48	80	TRUE	TRUE	7.0
Mission (connected) 2011 CFM	22	28	90	TRUE	TRUE	6.4
Zayante-Vergeles 2011 CFM	24	90	30	TRUE	FALSE	7.5
Zayante-Vergeles	26	58	90	FALSE	TRUE	7.0
San Gregorio (North) 2011 CFM	32	129	90	TRUE	TRUE	7.4
Las Positas	36	15	90	TRUE	TRUE	6.5
Reliz 2011 CFM	43	127	58	TRUE	TRUE	7.5
Greenville (So) 2011 CFM	45	29	87	TRUE	TRUE	6.7
Greenville (No) 2011 CFM	46	51	84	TRUE	TRUE	7.1
Monterey Bay-Tularcitos	46	86	90	TRUE	TRUE	7.3
Mount Diablo Thrust	48	25	38	FALSE	TRUE	6.7
Mount Diablo Thrust South	48	11	40	TRUE	FALSE	6.5
Mount Diablo Thrust North CFM	50	19	40	TRUE	FALSE	6.8
San Gregorio (South) 2011 CFM	54	90	75	TRUE	TRUE	7.3
Franklin 2011 CFM	58	38	90	TRUE	TRUE	6.9

Contra Costa (Lafayette) 2011 CFM	59	8	90	TRUE	TRUE	6.3
Hayward (No) 2011 CFM	59	53	82	TRUE	TRUE	7.0
Contra Costa (Larkey) 2011 CFM	60	8	90	FALSE	TRUE	6.3

Notes:

1. Values shown in table are representative values presented in UCERF3 "fault section data" table.
2. Distances shown are to the modeled surface fault trace used in UCERF3 and may vary slightly from measured geologic distances based on CGS (2010) due to simplifications used in model.
3. Magnitudes are approximate maximum magnitudes based on max of relationships of Ellsworth B, Hanks and Bakun, and Shaw (see Field et al., 2013, Appendix E) using values shown in table as presented in UCERF3 documentation. Actual PSHA run with range of values representing uncertainty in hazard, relaxation of segmentation, and multi-fault ruptures. Multi-fault and multi-segment ruptures result in larger (but typically less probable) events. For example:
    a. Calaveras Combined CN + CC + CS + CE Magnitude 7.4 (BSSC, 2014)
    b. Hayward – Rodgers Creek-Healdsburg RC + HN + HS + HE Magnitude of 7.6 (BSSC, 2014)
    c. San Andreas – SAO + SAN + SAP + SAS Magnitude 8.0 (BSSC, 2014)
4. Less significant and/or distant faults not shown in table for clarity. See Figure for full system level model of fault model to 300 km.
5. CFM in table refers to the Community Fault Model

'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 3; and our model implemented the branch averaged solutions.

Faults (On-fault seismicity)

In the source model used for this work, the on-fault seismicity (e.g. seismicity along significant and/or major faults) 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 3 and discussed in detail in Field et al. (2013).

Background Seismicity (Off-fault seismicity)

Background seismicity accounts for earthquakes, both on and off identified fault sources, with generally lower magnitudes. 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.

Off-fault seismicity (e.g. background seismicity) recurrence rates are solved for simultaneously by the 'grand inversion'. 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, maximum off-fault magnitude, number of regional events per year greater than a magnitude 5, and other constraints as shown on the logic tree (see Figure 3).

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 (GMM's) used to estimate ground motion from an earthquake source need to directly or indirectly consider these effects. Many GMM's 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) has not been used as the V_{S30} for our site is outside the range of the relationship.

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

GROUND MOTION HAZARD ANALYSIS

Preceding sections described the development of the source model used in this work. This section describes the use of the source models for the current study and the resulting application to development of design ground motion parameters.

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 Chapter

21 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 (e.g. 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 84^{th} 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 design earthquake (DE) spectrum is defined as two-thirds of the MCE_R. The resulting site-specific DE 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 as is the case for this analysis. Details of our evaluation are provided below.

Probabilistic Seismic Hazard Analysis

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 □(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.

Deterministic Seismic Hazard Analysis

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 GMM's for this site. 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 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 Class A to D) with a value of S_s equal to 1.5 for purposes of these comparisons or set equal to 1.0 (Site Class E).

For the deterministic evaluations, we used the NGA West 2 spreadsheet (PEER 2015) and calculated deterministic values for some of the nearby faults including the San Andreas fault, the Hayward-Rodgers Creek-Healdsburg fault and the Monte Vista - Shannon fault. In order to estimate distances to these faults, we primarily relied on AP Earthquake Fault Zone (EFZ) map as there are some discrepancies in distances from BSSC (2014), UCERF3 and AP EFZ maps. The computed deterministic spectra from the above-mentioned nearby faults are shown in Figure 4. Figure 4 shows that the Monte Vista - Shannon fault at a distance to the surface trace of about 4.5 km (R_{RUP}, R_{JB} and R_X of about 4.5 km, 4.5 km and -4.5 km, respectively) and with a moment magnitude of about 7.1 controlled the deterministic event for periods up to about 1.8 seconds. Beyond a period of about 1.8 seconds, the San Andreas fault zone at a distance to the surface trace of about 10.6 km (R_{RUP}, R_{JB} and R_X all of about 10.6 km) and with a moment magnitude of about 8.0 controlled the deterministic event.

Site-Specific MCE_R and Design Response Spectra

To develop the site-specific spectral response accelerations, we first obtained the general seismic design parameters based on the site class, site coordinates, and the risk category based on Chapter 11 of ASCE 7-16 using online tools which access the USGS database (Table 2).

TABLE 2: GENERAL GROUND MOTION PARAMETERS BASED ON ASCE 7-16

Parameter	Value[1]	ASCE 7-16 Reference
S_S	1.752g	Fig 22-1
S_1	0.619g	Fig 22-2
Site Class	D	Table 20.3-1
F_a	1	Table 11.4-1
F_v	N/A	See Section 11.4.8
S_{MS}	1.752g	Eq. 11.4-1
S_{M1}	N/A	See Section 11.4.8
S_{DS}	1.168g	Eq. 11.4-3
S_{D1}	N/A	See Section 11.4.8
C_{RS}	0.935	Fig 22-18A
C_{R1}	0.914	Fig 22-19A
PGA	0.721g	Fig 22-9
F_{pga}	1.1	Table 11.8-1
PGA_M	0.793g	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 spectrum. 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, as was done in this report. These factors are shown in Table 3. 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} values are shown in Table 2. 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 3.

TABLE 3: RISK COEFFICIENTS AND MAXIMUM ROTATION FACTORS

Period (second)	Risk Coefficients (ASCE 7-16)	Shahi and Baker (2014) Max Rotation Factor
0.010	0.935	1.19

Period (second)	Risk Coefficients (ASCE 7-16)	Shahi and Baker (2014) Max Rotation Factor
0.020	0.935	1.19
0.030	0.935	1.19
0.050	0.935	1.19
0.075	0.935	1.19
0.100	0.935	1.19
0.150	0.935	1.20
0.200	0.935	1.21
0.250	0.934	1.22
0.300	0.932	1.22
0.400	0.930	1.23
0.500	0.927	1.23
0.750	0.921	1.24
1.000	0.914	1.24
1.500	0.914	1.24
2.000	0.914	1.24
3.000	0.914	1.25
4.000	0.914	1.26
5.000	0.914	1.26

As mentioned earlier geometric mean 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, there is no need to scale it up to estimate the deterministic lower limit and this spectrum is the governing deterministic spectrum.

Spectral acceleration values for deterministic and probabilistic are compared in Table 4 and the graphical comparison is shown on Figure 5. Table 4 and Figure 5 show that the deterministic spectrum is lower than the probabilistic spectrum for all periods and thus the deterministic spectrum controls the preliminary site-specific MCE_R spectrum. The DE spectrum was developed by taking two-thirds of the MCE_R spectrum, as shown in Table 5. Spectral acceleration values for the preliminary site-specific DE and 80% of the code DE are also compared in Table 5 with the graphical comparison shown on Figure 6. Table 5 and Figure 6 show that the preliminary DE spectrum is higher than the 80% of the code DE spectrum for all periods. Therefore, the final site-specific DE spectrum is controlled by the site-specific spectrum. The final site-specific MCE_R spectrum is taken as 1.5 times the final site-specific DE spectrum. The recommended site-specific

MCE_R and DE spectra are shown on Figure 7. Spectral acceleration values for the MCE_R and DE spectra are listed in Table 6.

TABLE 4: COMPARISON OF DETERMINISTIC AND PROBABILISTIC SPECTRAL ACCELERATIONS

Period (second)	84th-Percentile Deterministic (Sa, g)	Probabilistic RotD50 (Sa, g)	84th-Percentile Max Dir Deterministic (Sa, g)	Risk-Targeted Max Dir Probabilistic (Sa, g)
0.010	0.805	0.927	0.958	1.032
0.020	0.813	0.928	0.967	1.033
0.030	0.839	0.978	0.999	1.088
0.050	0.948	1.139	1.128	1.267
0.075	1.146	1.438	1.364	1.600
0.100	1.333	1.662	1.586	1.849
0.150	1.616	1.966	1.939	2.206
0.200	1.797	2.175	2.174	2.461
0.250	1.938	2.273	2.364	2.589
0.300	2.017	2.315	2.461	2.633
0.400	1.995	2.251	2.454	2.574
0.500	1.863	2.101	2.291	2.396
0.750	1.478	1.666	1.833	1.902
1.000	1.193	1.334	1.479	1.511
1.500	0.791	0.923	0.981	1.046
2.000	0.579	0.698	0.718	0.791
3.000	0.407	0.469	0.509	0.536
4.000	0.305	0.349	0.384	0.402
5.000	0.236	0.274	0.297	0.315

TABLE 5: COMPARISON OF SITE-SPECIFIC AND CODE SPECTRA

Period (second)	Site-Specific MCE_R (Sa, g)	Site-Specific Design Earthquake (Sa, g)	80% Code DE (Sa, g)
0.010	0.958	0.639	0.374

Period (second)	Site-Specific MCE_R (Sa, g)	Site-Specific Design Earthquake (Sa, g)	80% Code DE (Sa, g)
0.020	0.967	0.645	0.437
0.030	0.999	0.666	0.469
0.050	1.128	0.752	0.532
0.075	1.364	0.909	0.612
0.100	1.586	1.058	0.691
0.150	1.939	1.292	0.850
0.200	2.174	1.449	0.934
0.250	2.364	1.576	0.934
0.300	2.461	1.640	0.934
0.400	2.454	1.636	0.934
0.500	2.291	1.527	0.934
0.750	1.833	1.222	0.934
1.000	1.479	0.986	0.825
1.500	0.981	0.654	0.550
2.000	0.718	0.479	0.413
3.000	0.509	0.339	0.275
4.000	0.384	0.256	0.206
5.000	0.297	0.198	0.165

TABLE 6: FINAL SITE-SPECIFIC HORIZONTAL SPECTRAL ACCELERATIONS (g)

Period (second)	DE Spectrum	MCE_R Spectrum
	5% Damping	
0.010	0.639	0.958
0.020	0.645	0.967
0.030	0.666	0.999
0.050	0.752	1.128
0.075	0.909	1.364
0.100	1.058	1.586
0.150	1.292	1.939
0.200	1.449	2.174
0.250	1.576	2.364
0.300	1.640	2.461
0.400	1.636	2.454

Period (second)	DE Spectrum	MCE_R Spectrum
	5% Damping	
0.500	1.527	2.291
0.750	1.222	1.833
1.000	0.986	1.479
1.500	0.654	0.981
2.000	0.479	0.718
3.000	0.339	0.509
4.000	0.256	0.384
5.000	0.198	0.297

SITE-SPECIFIC GROUND MOTION PARAMETERS

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. For this site, S_{DS} value is governed by the spectral acceleration value at 0.3 second. 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*Sa between periods of 1 and 5 seconds, where T is the period and Sa is the corresponding spectral acceleration. For this site, the S_{D1} value is governed by the spectral acceleration value at 4 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 below in Table 7.

TABLE 7: SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Parameter	Value (5% Damping)
S_{DS}	1.476g
S_{D1}	1.025g
S_{MS}	2.215g
S_{M1}	1.538g

Site-specific maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (PGA_M) was estimated based on Section 21.5 of ASCE 7-16. According to Section 21.5 of ASCE 7-16, the site-specific PGA_M shall be taken as the lesser of the site-specific probabilistic geometric mean peak ground acceleration of Section 21.5.1 and the site-specific deterministic geometric mean peak ground acceleration of Section 21.5.2, which shall not be taken as less than one-half the F_{PGA} value determined from Table 11.8-1 using a PGA value of 0.5g. Additionally, the site-specific PGA_M shall not be taken as less than 80 percent of the PGA_M value determined from Eq. 11.8-1 (code-based). Based on this procedure, the site-specific PGA_M value is 0.805g and is controlled by the deterministic geometric mean peak ground acceleration from the Monte Vista - Shannon fault with a magnitude of about 7.1, and this magnitude may be used in geotechnical evaluations at the site.

SEISMIC DESIGN CATEGORY

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.

CLOSURE

We have completed this addendum for the exclusive use of APPLE, INC., and their consultants for specific application to the subject project. The findings and conclusions presented in this report were prepared in accordance with generally accepted geotechnical engineering practice and are subject to the limitations of the referenced Geotechnical Report prepared previously by Kleinfelder for the subject site.

We appreciate this opportunity to be of service and look forward to continuing to work with you in the future. If you have any questions about this addendum, please contact us at our office.

Sincerely,

KLEINFELDER, INC.

Alexander D. Wright, PE
Project Professional

Minxing Zhao, PhD, PE, GE
Project Professional

Reviewed by:

Zia Zafir, PhD, PE, GE
Senior Technical Manager

Attachments: Figure 1 – Regional Seismicity
Figure 2 – Seismic Source Model
Figure 3 – UCERF3 Source Model Logic Tree
Figure 4 – Comparison of Deterministic Response Spectra
Figure 5 – Comparison of Probabilistic and Deterministic Spectra
Figure 6 – Comparison of DE and 80% of Code Spectra
Figure 7 – Final Design Earthquake and MCE_R Spectra

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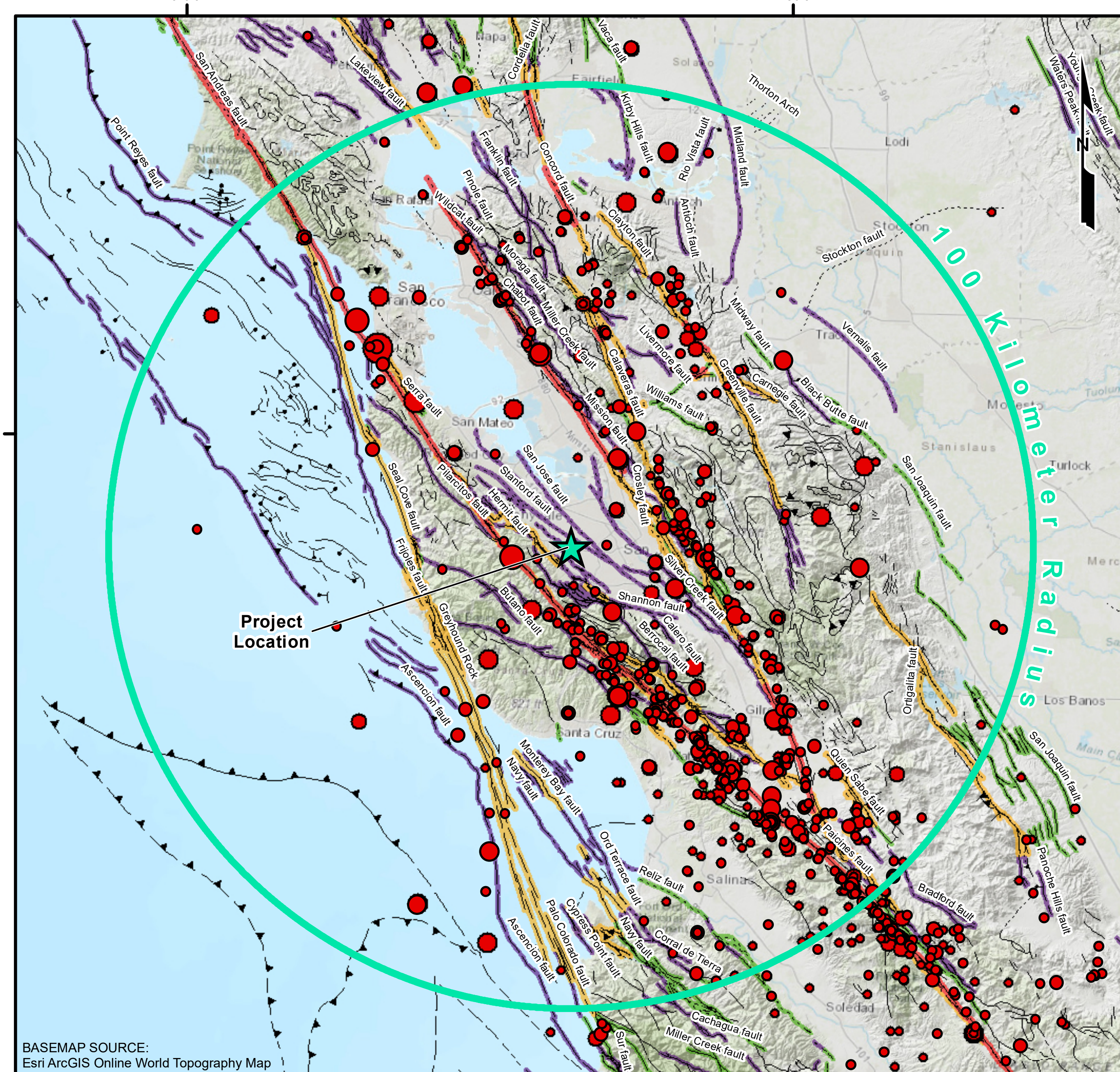
123°0'0"W
121°30'0"W
37°30'0"N



123°0'0"W
121°30'0"W
37°30'0"N

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

Date: 3/24/2021 User: ALeonard Path: \\azrgisstorp01\GIS_Projects\Client\Apple\20213246_AppleVP01\MAPS\EQ_FaultMap_FAM_100km.mxd

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CREATED:	3/24/2021
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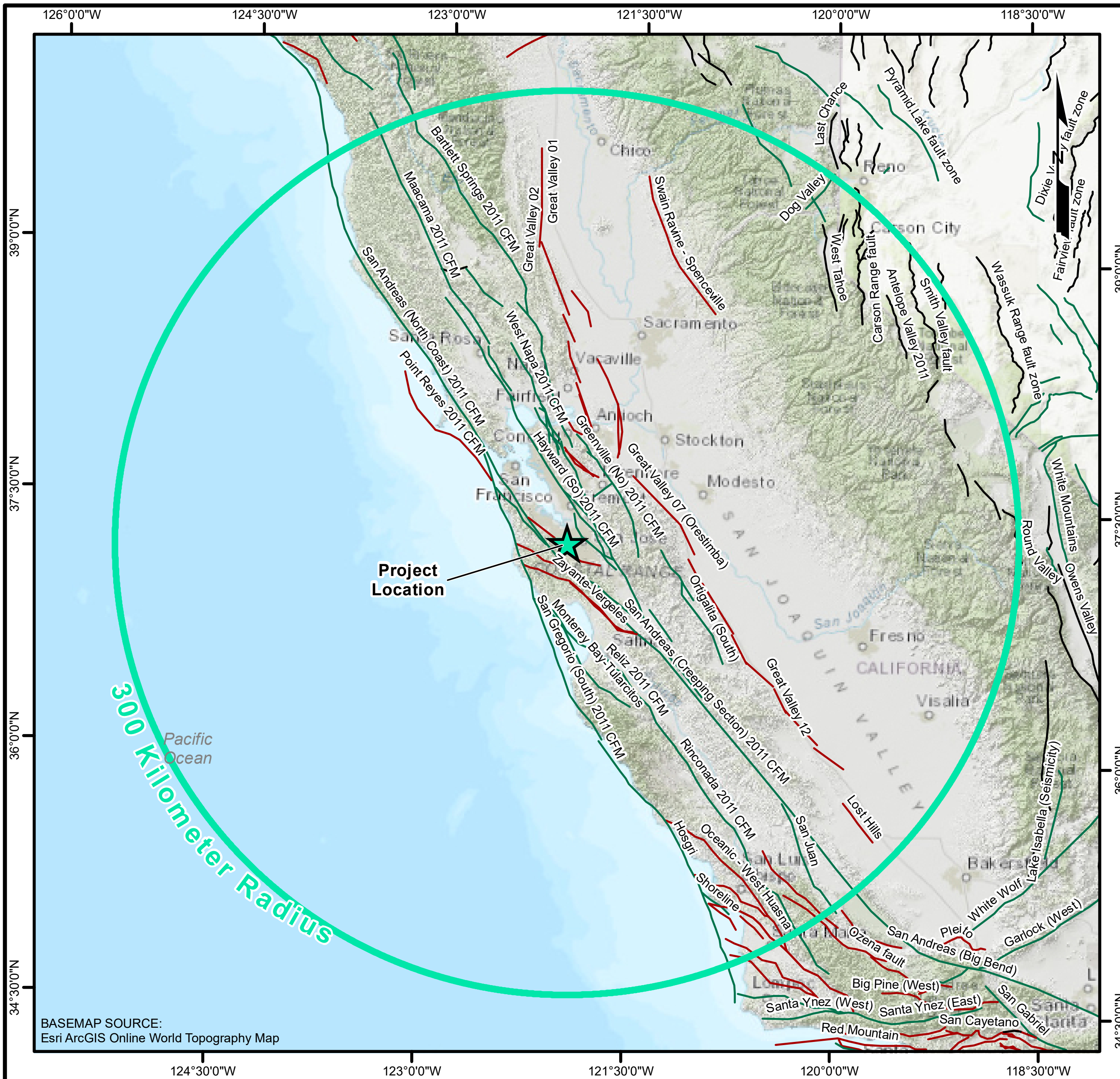
CHECKED BY:	AWright
FILE NAME:	EQ_FaultMap_FAM_100km.mxd

Regional Seismicity

SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS
APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CA

FIGURE
1



Faults, National Seismic Hazard Map (NSHM), 2014

- normal
- strike slip
- thrust
- unassigned

0 50 100 200 Kilometers

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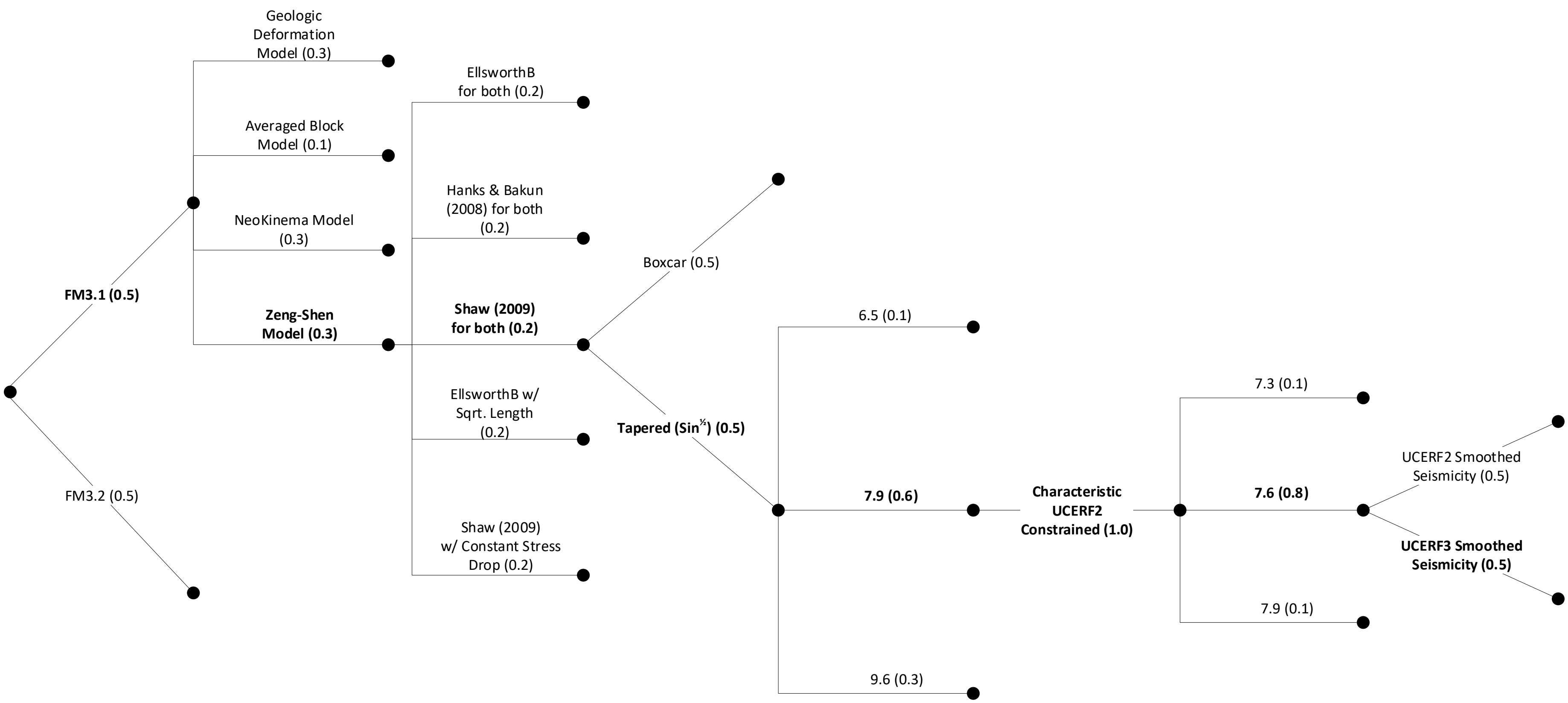
FILE NAME:	EQ_FaultModel_300km.mxd

Source Model

SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS
APPLE VP01
19191 VALLCO PARKWAY
CUPERTINO, CA

FIGURE
2

Fault Models	Deformation Models	Scaling Relationships (mag-area & slip-length relationships)	Slip Along Rupture (Dsr)	Total M ≥5 Event Rate (yr^{-1})	Inversion Model	$M_{max}^{off\text{-}fault}$	Off-Fault Spatial Seismicity PDF (aka SpatialPDF)



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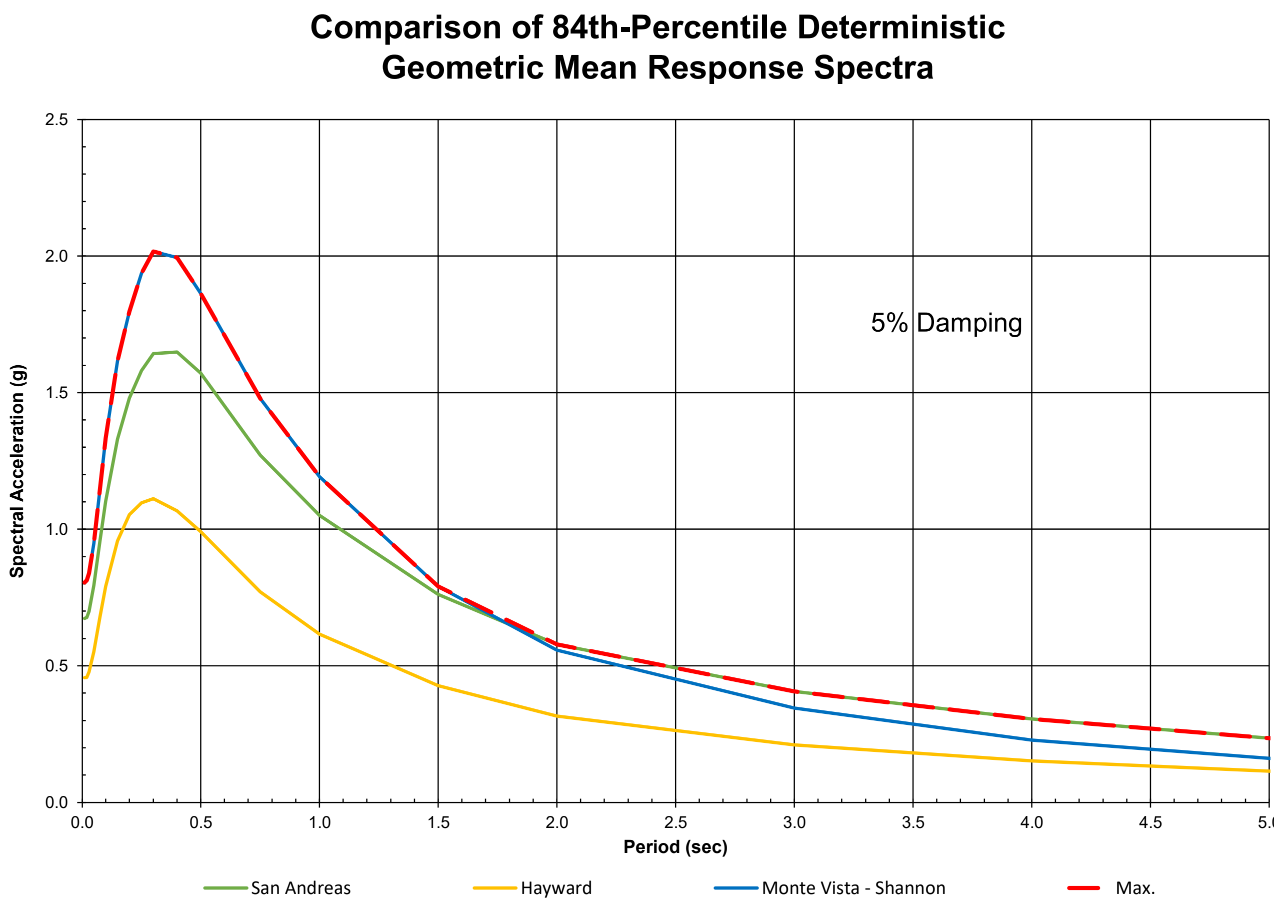
PROJECT NO.	20213246.001A
DRAWN:	01/2020
DRAWN BY:	ADW
CHECKED BY:	ZZ
FILE NAME:	UCERF3 Logic Tree

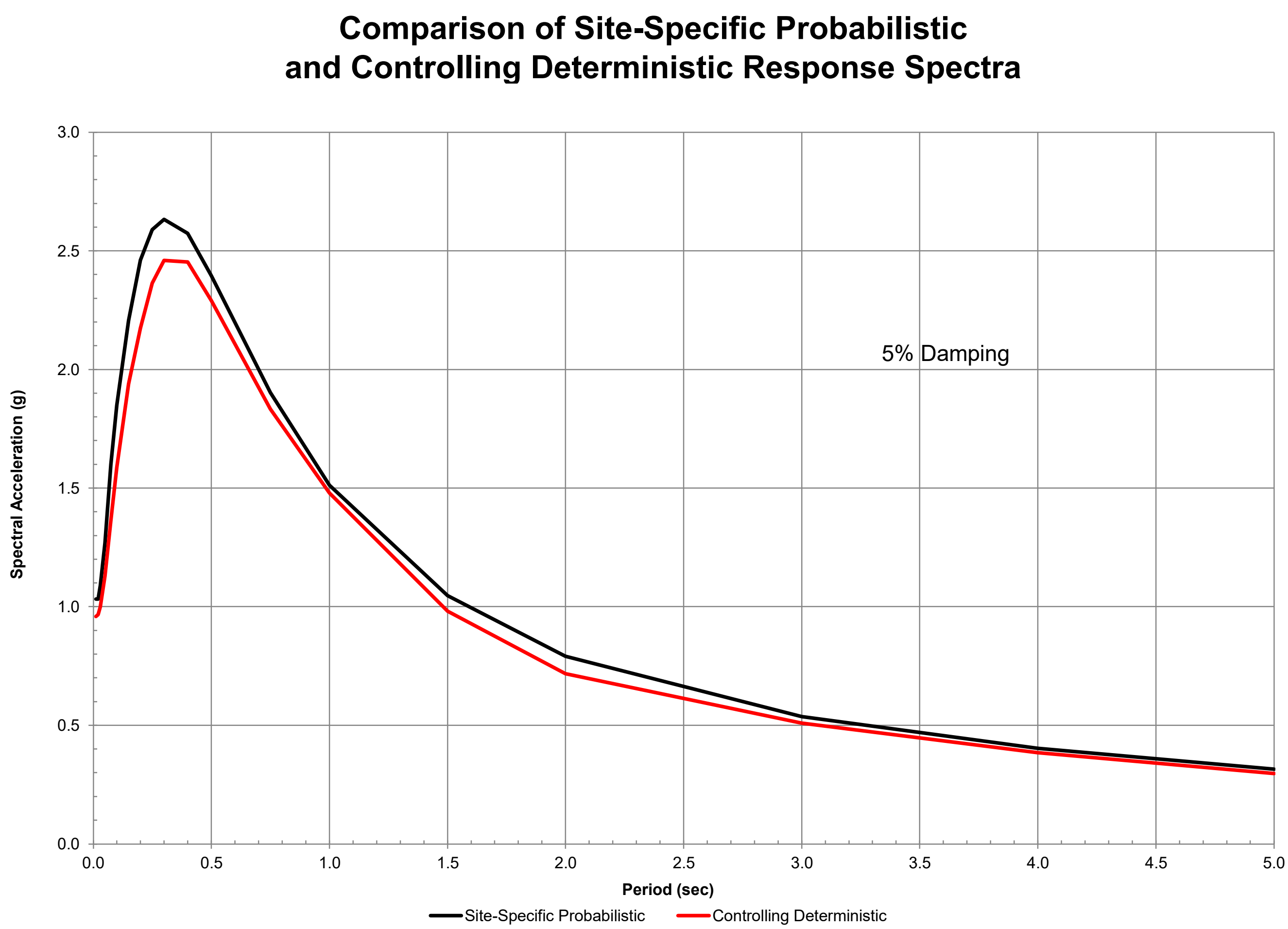
UCERF3 SOURCE MODEL
LOGIC TREE

Site-Specific Ground Motion Hazard Analysis
Apple VP01
19191 Vallco Parkway
Cupertino, CA 95014

FIGURE:
3



KLEINFELDER Bright People. Right Solutions.	Project No:	20213246.001A	**COMPARISON OF DETERMINISTIC RESPONSE SPECTRA**	**FIGURE**
	Date:	3/16/2021		
	Drawn By:	M. Zhao	Site-Specific Ground Motion Hazard Analysis Apple VP01 19191 Vallco Parkway Cupertino, CA 95014	**4**
	Checked By:	A. Wright		
	File Name:	Deterministic Spectra Comparison		



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Project No:	20213246.001A
Date:	3/17/2021
Drawn By:	M. Zhao
Checked By:	A. Wright
File Name:	GMHA-2019CBCv1.1.xlsx

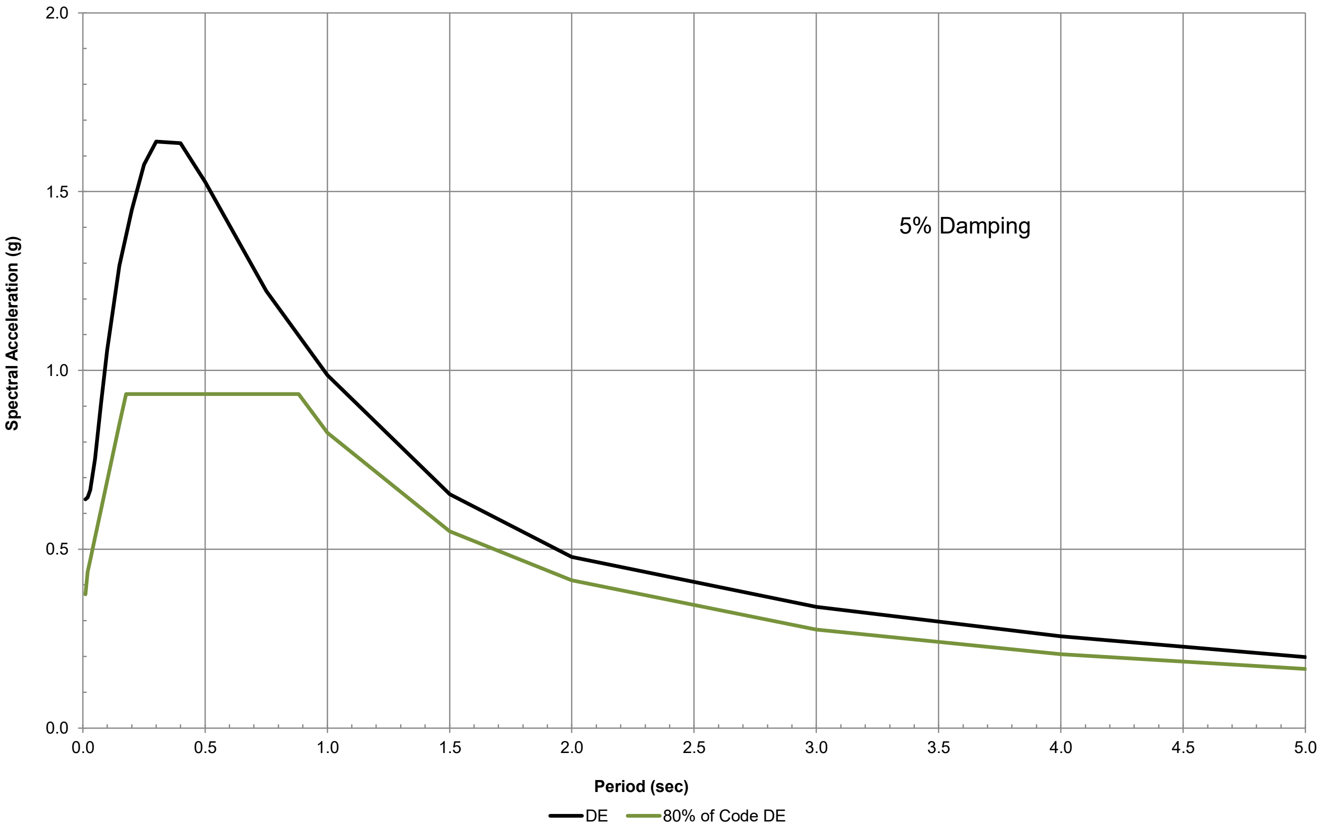
COMPARISON OF PROBABILISTIC AND DETERMINISTIC SPECTRA 2019 CBC/ASCE 7-16

Site-Specific Ground Motion Hazard Analysis
Apple VP01
19191 Vallco Parkway
Cupertino, CA 95014

FIGURE

5

Comparison of Site-Specific Design Earthquake and 80% of Code Response Spectra



Project No:	20213246.001A	
Date:	3/17/2021	
Drawn By:	M. Zhao	
Checked By:	A. Wright	
File Name:	GMHA-2019CBCv1.1.xlsx	

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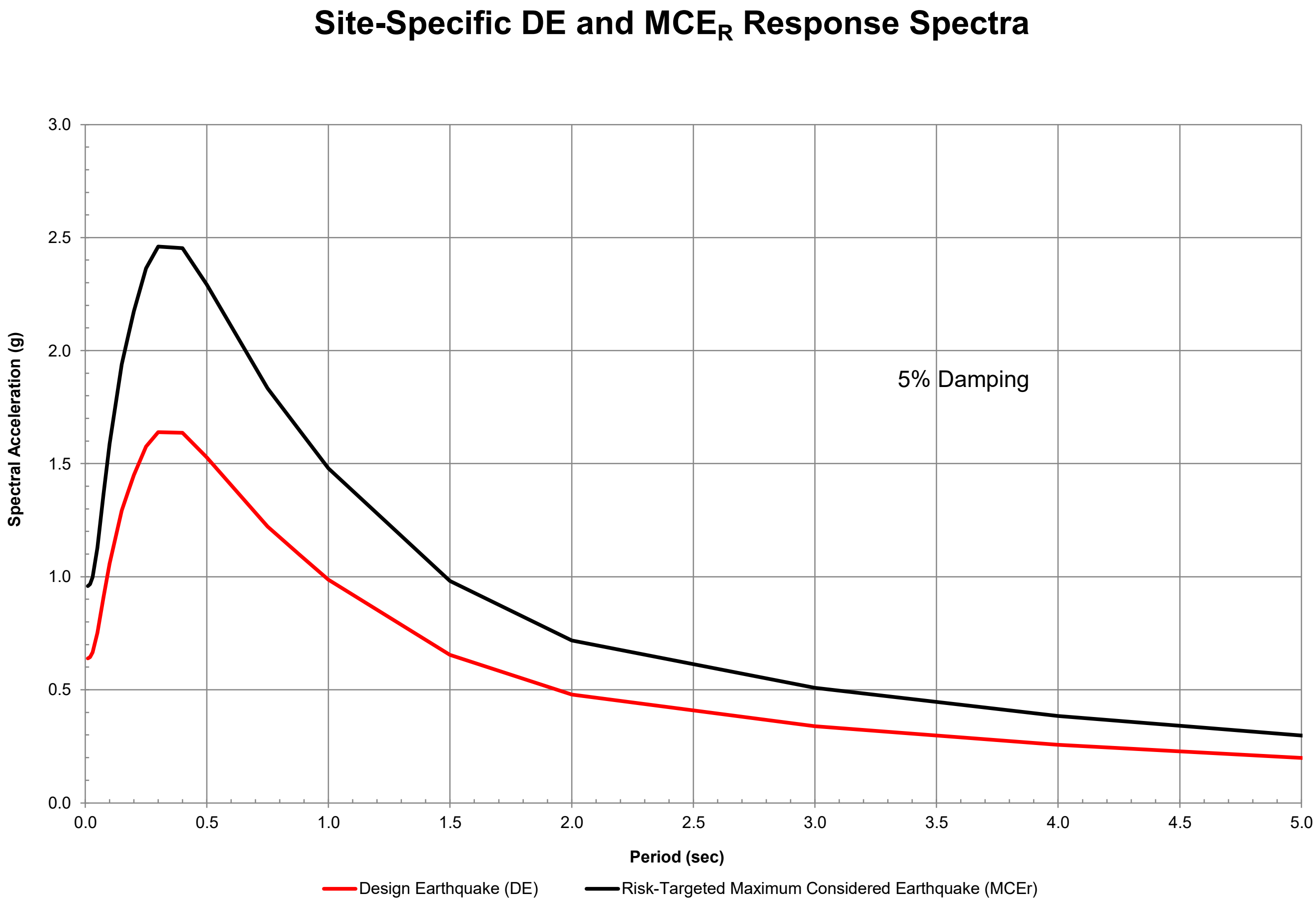
COMPARISON OF DE AND 80% OF CODE SPECTRA 2019 CBC/ASCE 7-16

Site-Specific Ground Motion Hazard Analysis
Apple VP01
19191 Vallco Parkway
Cupertino, CA 95014

FIGURE

6



KLEINFELDER Bright People. Right Solutions.	Project No:	20213246.001A	**FINAL DESIGN EARTHQUAKE AND MCE$_R$ SPECTRA 2019 CBC/ASCE 7-16**	**FIGURE**
	Date:	3/17/2021		
	Drawn By:	M. Zhao	Site-Specific Ground Motion Hazard Analysis Apple VP01 19191 Vallco Parkway Cupertino, CA 95014	**7**
	Checked By:	A. Wright		
	File Name:	GMHA-2019CBCv1.1.xlsx		

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Appendix E

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 <u>not</u> 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 <u>Not</u> 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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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e-mail: info@geoprofessional.org www.geoprofessional.org