EAST WHISMAN PHASE 1
MOUNTAIN VIEW, CALIFORNIA

GEOTECHNICAL REPORT
FOR HORIZONTAL IMPROVEMENTS AT R1 AND R2

SUBMITTED TO
Google, LLC
% Ms. Lisa Herrera
1600 Amphitheatre Parkway
Mountain View, CA 94043

PREPARED BY
ENGEO Incorporated

January 29, 2021
Revised February 8, 2021

PROJECT NO.
17954.000.001

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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

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1600 Amphitheatre Parkway
Mountain View, CA 94043

Subject: East Whisman Phase 1
Mountain View, California

GEOTECHNICAL REPORT
FOR HORIZONTAL IMPROVEMENTS AT R1 AND R2

Dear Ms. Herrera:

We are pleased to present this geotechnical report for horizontal improvements for the proposed East Whisman Phase 1 project located in Mountain View, California. This report presents our preliminary geotechnical observations, as well as our conclusions and preliminary recommendations for the project.

Based on the results of our exploration, the planned development at the site is feasible from a geotechnical standpoint. Recommendations presented in this report should be considered during the schematic design. We performed a preliminary study including laboratory testing and detailed engineering analyses under a separate cover.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Anne Robertson

Pedro Espinosa, GE

Bofei Xu, PE

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ar/pe/bx/ue/jf

REVISION RECORD

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SIGNATURES

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical report for horizontal improvements is to provide an assessment of geotechnical conditions and concerns associated with the proposed site redevelopment and provide preliminary recommendations to support development plans of the East Whisman Phase 1 project. Our services included the following tasks.

- Review available literature and geologic maps.
- Review historic aerial photos.
- Review available geotechnical explorations and geophysical data.
- Obtain appropriate Santa Clara Valley Water District permits.
- Notify Underground Services Alert a minimum of 48 hours prior to our exploration.
- Retain a private utility locator to clear the proposed exploration locations of existing utilities.
- Prepare a work plan, including proposed locations for our explorations, as well as excavation checklists showing their proximity to existing utilities.
- Perform subsurface field exploration.
- Install three vibrating-wire piezometers to monitor groundwater levels.
- Install a closed-loop geothermal pump for analysis of geothermal potential.
- Perform one percolation test in one of the borehole locations.
- Perform laboratory testing on soil samples collected.
- Analyze geotechnical data collected.
- Evaluate potential geotechnical concerns.
- Perform preliminary dewatering analysis for proposed utility retrofits and new corridors to evaluate potential settlements.
- Provide preliminary foundation recommendations for planning.
- Provide preliminary earthwork and horizontal improvement recommendations.

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1.2 SITE LOCATION AND DESCRIPTION

The East Whisman project is located in Mountain View, California, as shown on the Vicinity Map (Figure 1). It is approximately 2 miles south of the San Francisco Bay. The Site Plan (Figure 2) shows the boundaries of the site and the locations of our explorations.

The project site is located in the southwestern portion of the Middlefield Park Planning area, which has a combined area of approximately 40 acres. The site is currently occupied by a mid-rise office building and its associated parking lot. It is bounded on the west by Ellis Street, on the east by the VTA Light Rail Orange Line right-of-way, on the north by a building at 401 Ellis Street, and on the south by East Middlefield Road. Access is provided via Ellis Street. The two parcels included in the project site are 401 Ellis Street, with the Assessor's Parcel Number (APN) 160-58-016, and 500 East Middlefield Road, with the APN 160-58-017 (southern portion).

The site slopes gently to the northwest, with elevations ranging from approximately 55 feet (NAVD88) in the northwest corner to approximately 62 feet (NAVD88) in the southeast corner.

1.3 PROPOSED DEVELOPMENT

Based on our discussions with the project team and review of the information provided, we understand that the project will consist of construction of two residential podium buildings, referred to as R1 (south) and R2 (north). R1 is planned with 451 units, while R2 is planned with 462 units. Both R1 and R2 will consist of three mass timber structures each, with varying numbers of stories (up to 9) that will be constructed over a common concrete podium, one for each buildings. The podium buildings may also incorporate geothermal systems.

1.4 EXISTING GEOTECHNICAL INFORMATION

Ninyo & Moore (N&M) performed a preliminary geotechnical investigation for the site and published a preliminary geotechnical report dated November 29, 2019. Their field exploration included drilling one mud-rotary boring (B-6) and advancing two cone penetration tests (CPT) (CPT-11 and CPT-12). The mud-rotary boring was drilled to depths of 44½ feet, and the CPTs were advanced to depths of up to 101 feet below existing ground surface. We provide the boring and CPT logs, and laboratory test results from this previous work in Appendix C. The approximate locations of the N&M explorations are also shown on Figure 2.

2.0 FINDINGS

2.1 SITE HISTORY

We reviewed historical aerial photographs for the site from dates ranging between 1948 and present. Aerial photographs suggest that the site was used for agriculture prior to the 1960s. In the 1960s, Ellis Street and Middlefield Road were constructed, and the site was developed with an office building and a surface parking lot. In the 1990s, the original office building was demolished and replaced with the current structure. The site is presently occupied by a mid-rise office building with four stories, asphalt concrete-paved parking areas, trees, and associated landscaping.

2.2 REGIONAL GEOLOGY

The site is located on the western side of San Francisco Bay on the eastern side of the San Francisco Peninsula, in the Coast Ranges physiographic province of California. The Coast Ranges comprise a system of northwest-trending, fault-bounded mountain ranges and intervening valleys that trend approximately parallel to the right-lateral transform boundary between the North American and Pacific Plates. The present geomorphology and geology of the Coast Ranges are the result of deformation and deposition along the tectonic boundary between the North American plate and the Pacific plate. Plate-boundary fault movements are largely concentrated along the well-known fault zones, which in the Bay Area include the San Andreas, Hayward, and Calaveras faults, as well as other lesser-order faults. Bedrock in the Coast Ranges consists of igneous, metamorphic, and sedimentary rocks that range in age from Jurassic to Pleistocene.

2.3 SITE GEOLOGY

According to published geologic mapping prepared by Brabb et al. (2000) and Witter et al. (2006), the site is underlain by Holocene alluvial fan deposits (Qhaf), as shown on Figure 3. The site is located near the distal fan edge and the alluvial deposits that are described as consisting of medium dense sand with layers of sandy or silty clay (Brabb, 2000).

According to the California Geologic Survey (CGS) seismic hazards zone map of the Mountain View Quadrangle (2006), the site is mapped within a potential liquefaction hazard zone.

2.4 SEISMICITY

Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of active faults and significant historic earthquakes recorded within the San Francisco Bay Region. The Mountain View area contains numerous active earthquake faults. The nearest active faults are the Monte Vista-Shannon, Northern San Andreas, Hayward-Rogers Creek, and Calaveras faults, which are capable of producing earthquakes with moment magnitudes of 6.5, 8.1, 7.3, and 7.0, respectively. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,700 years - CGS, 2018).

The site is not located within a designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site; as such, the risk of fault rupture through the site is considered low.

Seismicity of the site is further discussed in Section 4 of the design-level study under a separate cover.

2.5 FIELD EXPLORATION

Our field exploration included advancing four CPTs (1-CPT01 through 1-CPT04); drilling three borings (1-B01 through 1-B03); performing one percolation test in Boring 1-B01; installing and monitoring three vibrating-wire piezometers (VWPs) (two at 1-B03, and one at 1-B02); and installing one 100-foot geothermal closed-loop pipe, and performing thermal conductivity testing at 1-B01. The field explorations and geothermal testing were performed between November 13 and December 4, 2020. We will continue to monitor VWPs with quarterly site visits.

We show the locations of the explorations on Figure 2. A summary of boring locations and methods can be found in Table 2.5-1. A summary of previous explorations by Ninyo & Moore can be found in Table 2.5-2.

TABLE 2.5-1: Summary of Current Explorations

EXPLORATION LOCATION	MAXIMUM DEPTH (FEET)	GROUND SURFACE ELEV. (FEET, NAVD88)	DRILLING METHOD	DATES
1-CPT01	100.8	55	CPT	11/13/2020
1-CPT02	101.0	61	CPT	11/13/2020
1-CPT03	100.9	55	CPT	11/13/2020
1-CPT04	100.9	61	SCPT	11/13/2020
1-B01	61.5	61	RW	11/18/2020
1-B02	61.5	55	RW	11/17/2020
1-B03	102.5	61	RW	11/13/2020

RW = Rotary Wash

TABLE 2.5-2: Summary of Previous Explorations

EXPLORATION LOCATION	MAXIMUM DEPTH (FEET)	GROUND SURFACE ELEV. (FEET, NAVD88)	DRILLING METHOD	DATES
CPT-11	80.1	57	CPT	9/26/2019
CPT-12	101.2	62	CPT	9/26/2019
B-6	44.5	57	RW	9/27/2019

RW = Rotary Wash

2.5.1 Borings

We observed the drilling of three borings at the locations shown on the Site Plan, Figure 2. An ENGEO representative observed the drilling and logged the subsurface conditions at each location. We retained the services of a drilling contractor using a truck-mounted drill rig. Drilling consisted of 5-inch-diameter augers and used a mud-rotary method. We advanced the borings to depths ranging from 61½ to 102½ feet below existing grade. To address environmental concerns, we cased the upper 50 feet of each exploratory boring with steel casing to avoid cross-contamination of the upper and lower aquifers. We did not observe artesian conditions in the aquifers within the exploratory borings. We permitted and backfilled the borings in accordance with the requirements of the Santa Clara Valley Water District (SCVWD).

We obtained soil samples at various intervals using standard penetration test (SPT) samplers with a 2-inch outside diameter (O.D. split-spoon sampler) and California Modified samplers with a 2½-inch inside diameter (I.D.). We obtained the blow counts shown on our bore logs with an automatic trip, 140-pound hammer with a 30-inch free fall. We drove the sampler 18 inches and recorded the number of blows for each 6 inches of penetration. We have not converted the blow counts presented on the borelogs using any correction factors. We also obtained hydraulically pushed Shelby tubes at select locations. We present the fluid pressures recorded for the hydraulically pushed samples on the exploration logs in Appendix A.

Upon completion of Borings 1-B02 and 1-B03, we installed VWPs at various depths. The boring and the VWPs were backfilled with cement grout under the observation of a SCVWD inspector.

Soil cuttings and excess fluids were contained in 55-gallon steel drums and were sampled according to procedures described in the gSAFE document, *EHS Processes to Haul Soil off Site*. The findings and recommendations for disposal are presented under a separate cover.

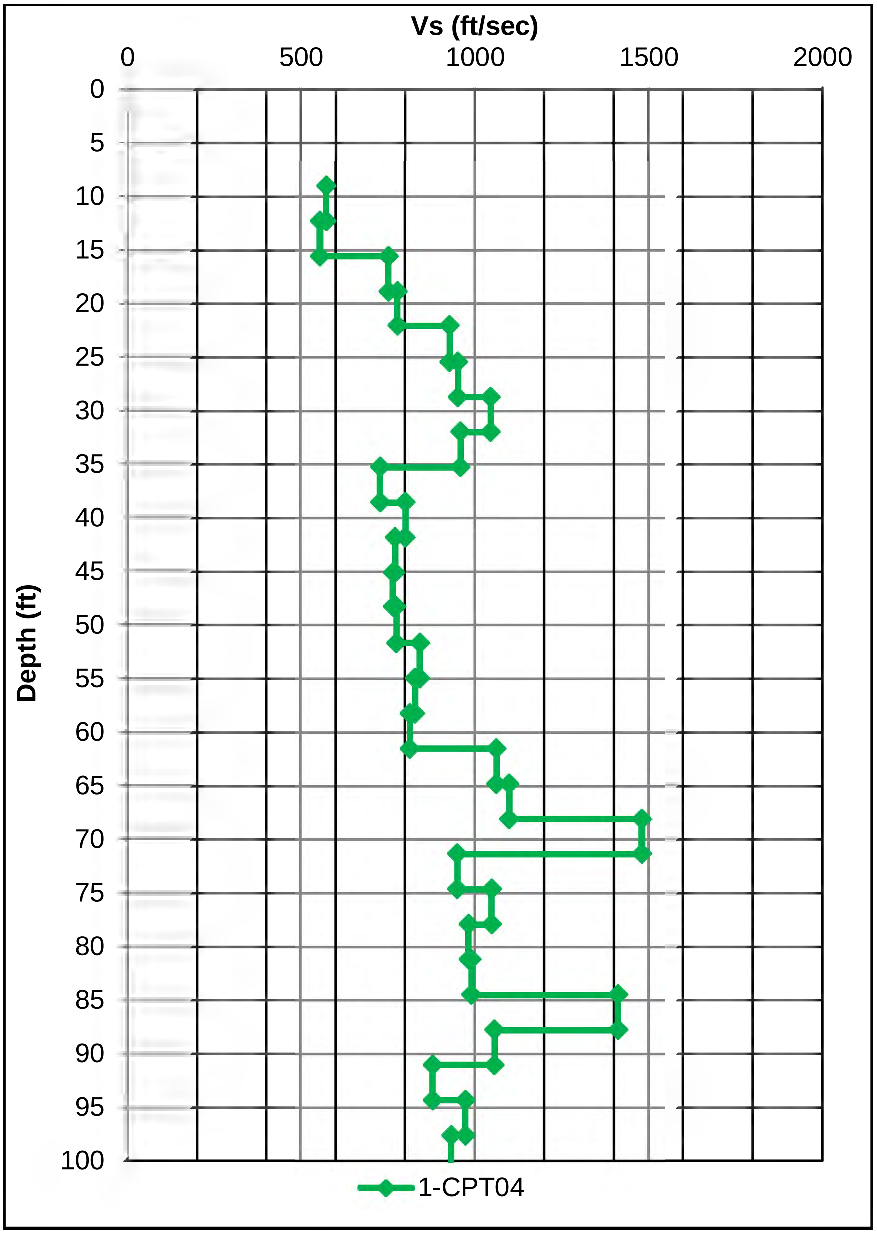
We provide additional information about specific subsurface conditions at each location in our exploration logs in Appendix A. The soil type, color, consistency, and visual classification provided in the logs are generally accordance with the Unified Soil Classification System.

2.5.2 Cone Penetration Tests

We retained the services of a contractor with a CPT rig to advance CPTs at four locations to approximately 100 feet below ground surface (bgs) in general accordance with ASTM D-5778. Mud-rotary borings were drilled in proximity to 1-CPT02 and 1-CPT03 to allow direct comparison of the data (matched pairs). Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988).

Shear-wave velocity (V_S) measurements were performed by the CPT contractor in 1-CPT04, using the downhole seismic method specified in ASTM D7400. We present the CPT logs in Appendix B. The V_S profiles obtained from this testing are shown in Exhibit 2.5.2-1. The time-averaged shear-wave velocity over the top 100 feet or 30 meters (V_{S30}) for this V_S profile is 855 feet/sec or 260 meters/sec.

EXHIBIT 2.5.2-1: Vs profile obtained from seismic CPT testing



2.6 SURFACE AND SUBSURFACE CONDITIONS

Current ground-surface elevations at the site range from Elevation 55 to 62 feet (NAVD88). The project site is currently occupied by existing structures and related improvements. Surface conditions outside of the building footprints generally consist of asphalt-paved parking areas, concrete-paved sidewalks, and landscape vegetation.

In our exploration locations, we encountered approximately 2 to 3 inches of asphalt pavement, underlain by approximately 1½ feet of aggregate base. Directly below the pavement section, we encountered existing fill up to 5 feet bgs.

Beneath the fill, we encountered basin deposits composed of lean clay and sandy lean clay interbedded with sand and gravel. The clay was generally dark yellowish brown, olive, and greenish gray, ranged from medium stiff to hard, had medium to low-plasticity, and exhibited a variety of consistencies, plasticity, and sand content. The clay was interbedded with medium-dense to very dense sand and gravel layers. The sandy and gravelly layers were up to 20 feet thick, but more typically between 5 to 10 feet thick, and ranged from isolated channel deposits to more widely extending bedded deposits.

We developed two generalized subsurface cross sections that depict our interpretation of the soil conditions based on the field explorations (Figure 6). These interpreted cross sections may assist in visualization of layering and general subsurface trends in two dimensions across the site.

2.7 PERCOLATION TESTING

We performed a percolation test on November 17, 2020, in Boring 1-B01, at the approximate location shown on figure 2.

Boring 1-B01 was drilled to an approximate depth between 4½ and 5 feet below the existing ground surface with a 3½-inch-diameter hand auger. A vertical 3-inch-diameter PVC drain pipe was temporarily set in place, with the lowermost portion of the pipe having perforations. The annulus along the perforated interval was filled with pea gravel and the hole was soaked with water up to 2 feet above the bottom of the borehole up to 24 hours before testing. During percolation testing, we measured groundwater levels using a water-level meter. Upon completion of testing, the standpipe was removed and the drilling and sampling was continued at 1-B01 as discussed in Section 2.5.1.

Percolation rates were converted to infiltration rates using the Porchet Method. We also performed gradation testing on soil collected from 5 feet bgs in Boring 1-B01, as verification of the infiltration rate. Based on our percolation test, and soil gradation, we recommend a design infiltration rate of approximately 0.1 inch per hour.

2.8 GROUNDWATER CONDITIONS

We did not observe groundwater in the current borings during drilling due to the method and casing used. However, we installed vibrating-wire piezometers after drilling at Borings 1-B02 and 1-B03. We measured groundwater at depths ranging from 10 to 15 feet, which correspond to Elevations between 45 and 49 feet (NAVD88). We also performed pore pressure dissipation tests in the CPTs. These tests suggest that the groundwater level is approximately 8 to 16 feet below ground surface, which corresponds to Elevations of 46 to 47 feet, as presented in Table 2.8-1 below.

TABLE 2.8-1: Recorded Groundwater Levels

BORING / CPT	MEASUREMENT TAKEN DEPTH (FEET BGS)	GROUNDWATER DEPTH (FEET BGS)	GROUNDWATER ELEVATION (NAVD88, FEET)	DATE OBSERVED
1-B02[1]	26	10	45	11/20/2020
1-B03[1]	26	15	46	11/20/2020
	66	12	49	11/20/2020
1-CPT01[2]	73	9	46	11/13/2020
1-CPT02[2]	26	16	46	11/13/2020
1-CPT03[2]	42	8	47	11/13/2020
1-CPT04[2]	7	14	47	11/13/2020

NOTES:
[1]Phreatic surface measured after drilling with vibrating-wire piezometer.
[2]Assumed phreatic surface based on pore pressure dissipation tests assuming hydrostatic conditions.

Previous groundwater data from the subsurface investigation performed by Ninyo & Moore (2019) are summarized in Table 2.8-2. They observed groundwater at depths ranging from 9 to 12 feet bgs, which correspond to approximately Elevation 48 to 50 feet).

TABLE 2.8-2: Previous Groundwater Levels

BORING / CPT	MEASUREMENT TAKEN DEPTH (FEET BGS)	GROUNDWATER DEPTH (FEET BGS)	GROUNDWATER ELEVATION (NAVD88, FEET)	DATE OBSERVED
B-6 (N&M)	N/A	NMDM	N/A	N/A
CPT-11 (N&M)	62	9*	48	9/26/2018
CPT-12 (N&M)	59	12*	50	9/26/2018

NOTES:
NMDM = not measured due to method
*Assumed phreatic surface based on pore pressure dissipation tests assuming hydrostatic conditions.

Plate 1.2 of the Seismic Hazard Zone Report for the Mountain View Quadrangle (2006) maps the shallowest historical groundwater within the site vicinity to be less than approximately 8 to 10 feet below the ground surface. For the purposes of our analyses and recommendations, we consider a groundwater level at Elevation 47 feet appropriate for design; this elevation corresponds to a depth range of 8 to 15 feet below ground the surface within the project site boundaries. This elevation coincides with the highest measured groundwater elevation at the upper aquifer.

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practices, and other factors not evident at the time measurements were made. Excavations for utility installation may encounter groundwater, depending upon the time of year of construction.

We will continue to monitor the groundwater level measurements from the three installed piezometers, and provide the design team with any update when available.

2.9 LABORATORY TESTING

We performed laboratory tests on select soil samples to evaluate their engineering properties. For this project, we performed laboratory testing as shown in the table below.

TABLE 2.9-1: Laboratory Testing

SOIL CHARACTERISTIC	TESTING METHOD	LOCATION OF RESULTS
R-Value	ASTM D2844	Appendix D
Plasticity Index (PI) (Wet Method)	ASTM D4318	Appendix D
Grain Size Distribution & Hydrometer	ASTM D422	Appendix D
Grain Size Distribution	ASTM D1140	Appendix D
Corrosivity	ASTM Methods	Appendix F

3.0 DISCUSSION AND CONCLUSIONS

Based on the exploration and laboratory test results, the proposed project development is feasible on the site provided the preliminary recommendations contained in this report are properly incorporated and additional design-level evaluations are performed.

The primary geotechnical concerns for the proposed site redevelopment are as follows.

- The settlement of moderately compressible layers due to building loads
- The potential for liquefaction of coarse-grained material and cyclic softening of some of the fine-grained soil material below the groundwater table during a seismic event
- The presence of shallow groundwater and its influence on below-grade construction

These and other issues are discussed below.

3.1 2019 CBC SEISMIC DESIGN PARAMETERS

The average shear-wave velocity at the project site is approximately 855 feet per second (fps), as measured during our field exploration; therefore, we classify the site as Site Class D. Based on collected CPT data and our liquefaction analysis, we do not believe that the thin lenses of potentially liquefiable soil will significantly change the natural period of the site soil profile. Hence, the project site is not classified as Class F. We discuss our liquefaction analysis further in the following sections.

We performed a site-specific seismic hazard analysis for Site Class D as required by the California Building Code (CBC), in accordance with the procedure described in Chapter 21 of ASCE 7-16. This analysis was performed for the design-level evaluations and will be incorporated in the design-level report under a different cover.

3.2 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking and liquefaction. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, lurching, landslides, tsunamis, or seiches is low to negligible at the site.

3.2.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, ground rupture is unlikely at the subject property.

3.2.2 Ground Shaking

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead and live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the actual forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse, but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.2.3 Liquefaction / Cyclic Softening

The site is located within a State of California Seismic Hazard Zone (CGS, 2006) for areas that may be susceptible to liquefaction (Figure 5).

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose silty sand is also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. If excess hydrostatic pressures exceed the effective confining stress from the overlying soil, the sand may undergo deformation. If the sand undergoes virtually unlimited deformation without developing significant resistance, it is said to have liquefied, and if the sand consolidates or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur. In addition to liquefaction of sandy material, clayey soil can also undergo “cyclic-softening” or strength loss as a result of cyclic loading.

3.2.3.1 Liquefaction Analysis Overview

We divided the soil into “sand-like” and “clay-like” behaviors using procedures presented in Boulanger and Idriss (2008). We then performed an initial liquefaction susceptibility assessment based on the methodologies presented by Bray and Sancio (2006). Section 3.2.3.2 presents the details of screening of soil samples for liquefaction susceptibility.

We then performed an analysis of liquefaction potential based on the CPT data using the computer software CLiq (Version 2.2.1.4) developed by GeoLogismiki. The software incorporates the procedure introduced by the 1996 National Center for Earthquake Engineering Research

(NCEER) workshop and the 1998 NCEER/National Science Foundation (NSF) workshop. The workshops are summarized by Youd et al. (2001) and updated by Robertson (2009).

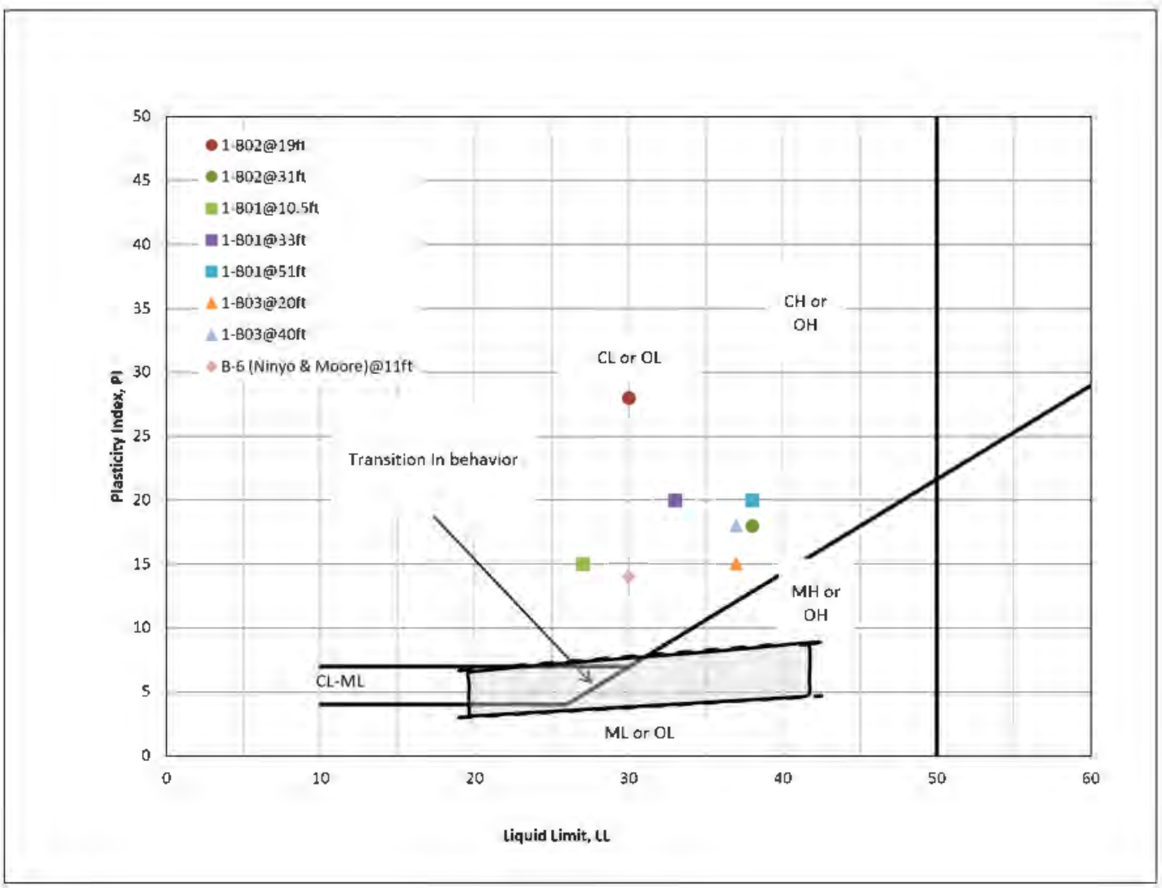
We used the in situ data (blow counts and soil descriptions), laboratory data (plasticity index, moisture content, fines content), and Boulanger and Idriss (2008) and Bray and Sancio (2006) methodologies to establish a relationship between soil that is potentially liquefiable in the CPTs by comparing it to adjacent "matched-pair" borings. To assess seismically induced settlements, we considered the methodology presented by Zhang et al. (2002). The details and results of our analyses are presented in the following sections.

3.2.3.2 Liquefaction Susceptibility Screening of Soil Samples

Boulanger and Idriss (2008) found that for practical purposes, soil can be divided into either 'sand-like' or 'clay-like' behavior. Where sand-like soil can experience 'liquefaction' and clay-like soil can experience 'cyclic failure or softening'. In general, sand-like soil is gravel, sand and very low plasticity silt, whereas clay-like soil comprises clay and plastic silt.

In order to evaluate the clay-like, intermediate, and sand-like behavior of the fine-grained soil at the site, we plotted the PI and liquid limit (LL) of the on-site soil relative to the soil behavior limits. These results are presented below (Exhibit 3.2.3.2-1). Based on Idriss and Boulanger (2008), we conclude the fine-grained soil at the site should be considered as 'clay-like'.

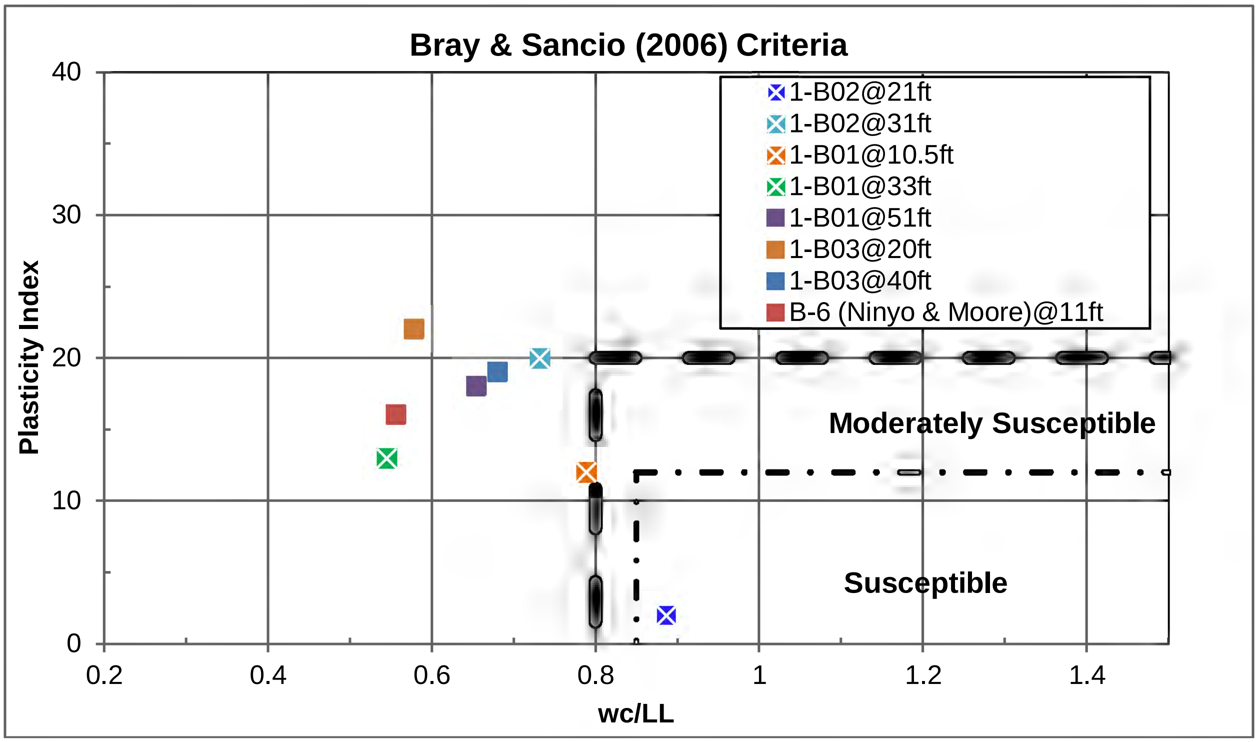
EXHIBIT 3.2.3.2-1: Idriss and Boulanger (2008) Methodology for Differentiating between Clay-like and Sand-like Behavior Based on Atterberg Limits



We then considered the criteria presented by Bray and Sancio to assess the potential for liquefaction triggering on the site fine-grained soil. Bray and Sancio observed that fine-grained soil with a PI less than 12 and a water content (w_c) to liquid limit (LL) ratio of more than 0.85 is susceptible to liquefaction/cyclic-softening. Soil with PI greater than 18 and/or w_c/LL less than 0.8 was deemed to be not susceptible to liquefaction because it is too plastic and/or its water contents are too low.

We considered the Bray and Sancio criteria at this site and plotted w_c/LL versus PI for our available laboratory data. As shown in Exhibit 3.2.3.2-2, the majority of the laboratory data plot as not susceptible to liquefaction based on these criteria. One clayey silt sample (Boring 1-B02 at 21 feet) plots as marginally susceptible to liquefaction. Based on our evaluation of the subsurface soil profile, this layer is not continuous and is only present at the northwestern portion of the site. We will evaluate this layer further in our design-level study.

EXHIBIT 3.2.3.2-2: Assessment of the Liquefaction/Cyclic-Softening Potential of Fine-Grained based on the Bray and Sancio (2006) Criteria



3.2.3.3 Liquefaction Analysis of CPT Data and Matched-Pair Borings

We estimated the Cyclic Stress Ratio (CSR) for a Maximum Considered Earthquake (MCE) Peak Ground Acceleration (PGA_M) value of 0.67g as outlined in the latest California building code with an earthquake magnitude of 7.9. We used a groundwater elevation of 47 feet (NAVD88) for this analysis. We also considered the depth of excavation in the CLiq analysis.

We then compared the calculated soil behavior Type Index (I_c) to soil zones that were not susceptible to liquefaction or cyclic softening according to Bray and Sancio (2006) in the adjacent borings. From this comparison, we established that soil with an I_c greater than 2.5 is mainly "clay-like" behavior-type soil, and as previously described, has a low susceptibility to liquefaction.

With the same comparison, given the prevalent conditions of interbedded fine-grained and coarse-grained granular soil layers, it is appropriate to turn on "auto transition layer detection." This allowed us to minimize over-prediction of liquefaction-induced settlement due to thin soil layer transition. We present the matched-pair lab data (from borings) and I_c (from CPTs) in Table 3.2.3.3-1. Appendix E presents the results of the CLiq analyses.

TABLE 3.2.3.3-1: Liquefaction/Cyclic Softening Susceptibility Evaluation based on Matched Pair Borings and CPTs – Bray and Sancio (2006)

CPT	DEPTH (ft)	I_C	MATCHED-PAIR BORING	PLASTIC INDEX (PI)	W_C/LL	TRIGERRING OF LIQUEFACTION/CYCLIC SOFTENING
1-CPT02	10.5	2.68	1-B01	12	0.79	No
1-CPT02	33	2.67	1-B01	13	0.55	No
1-CPT02	51	2.83	1-B01	18	0.66	No
1-CPT03	21	2.23	1-B02	2	0.89	Yes
1-CPT03	31	2.44	1-B02	20	0.73	No

Based on our evaluations, most of the fine-grained soil at this site should not be considered liquefiable. One clayey silt sample (Boring 1-B02 at 21 feet) plots as susceptible to liquefaction. Based on our evaluation of the subsurface soil profile, this layer is not continuous and is only present at the northwestern portion of the site. We will evaluate this layer further in our design-level study.

3.2.3.4 Liquefaction Analysis Conclusion

Based on site-specific study of the liquefaction hazard, we estimate the overall total liquefaction-induced settlement at the project site to be less than ¾ inch. In some isolated areas, the settlement value can be up to 1 inch.

3.2.4 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. The closest free face to the project site is 0.9 mile to the west. Therefore, the risk of lateral spreading at the project site is negligible.

3.2.5 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soil. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the Bay Area Region, but based on the site location, it is our opinion that the offset is expected to be minor. We provide recommendations for foundation and pavement design in this report that are intended to reduce the potential for adverse impacts from lurch cracking.

3.2.6 Flooding

Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) 06085C0045H (Figure 7) indicates that the site is within Zone X: an area protected by levees from the 1% annual chance flood. The Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project.

3.3 SHALLOW GROUNDWATER AND EXCAVATION CONSIDERATIONS

Based on our findings described in Section 2.7 of this report and the proposed development, groundwater may impact basement design and construction at the site. Shallow groundwater conditions may result in the following impacts.

1. Require construction dewatering

2. Result in unstable conditions at the base of excavation requiring stabilization prior to improvement construction

3. Develop hydrostatic uplift pressures below proposed basement foundations

4. Cause moisture damage to sensitive floor coverings

5. Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment

6. Require tie-downs due to hydrostatic uplift for the proposed basement structures, if any

7. Require waterproofing for the proposed basement structures, if any

3.4 SOIL CORROSION POTENTIAL

Corrosive soil and corrosive saline groundwater can cause damage to structures, foundations and buried utilities and can also increase required maintenance. Depending on the degree of corrosivity of subsurface soil, concrete and reinforcing steel in concrete structures and bare metal structures exposed to this soil can deteriorate, eventually leading to structural failure.

In general, ground environments may be classified as corrosive to buried concrete structural elements if any of the following conditions is present in the ground or may be present during the service life of a facility (Caltrans, 2018).

- The pH of the soil or groundwater is less than 5.5,
- The sulfate concentration is 1,500 ppm or greater, or
- The chloride concentration is 500 ppm or greater.

Additionally, a correlation between electrical resistivity and corrosivity to ferrous metals is provided in Table 3.4-1.

TABLE 3.4-1: Soil Resistivity and Corrosivity Correlation

SATURATED SOIL RESISTIVITY (OHM-CM)	SOIL CORROSIVITY TO FERROUS METALS
>10,000	Mildly Corrosive
2,000 – 10,000	Moderately Corrosive
1,000 – 2,000	Corrosive
< 1,000	Severely Corrosive

As part of this study, we collected two soil samples and submitted them to Sunland Analytical lab for determination of redox potential, pH, resistivity, sulfate, and chloride. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures and metal pipes. The results are included in Appendix F and summarized in the table below.

TABLE 3.4-2: Corrosivity Test Results

SAMPLE LOCATION	DEPTH (feet)	REDOX (mV)	pH	RESISTIVITY (OHMS-CM)	CHLORIDE* (mg/kg)	SULFATE* (mg/kg)
1-B2	11.0	219	7.34	1,800	6.7	34.1
1-B3	51.0	82	7.73	1,100	9.6	80.9

* ASTM D4327

The 2019 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Section 19.3.1 for concrete durability requirements. ACI Table 19.3.1.1 provides the exposure categories and classes, and Table 19.3.2.1 provides requirements for concrete in contact with soil based upon the exposure class.

Based on the test results and ACI criteria, the tested soil would classify as 'Not Applicable' for sulfate exposure; there is no requirement for cement type or water-cement ratio for this category; however, a minimum concrete compressive strength of 2,500 psi is specified by the building code. For this sulfate range, we recommend Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporates a maximum water-cement ratio of 0.50. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

Soil with a pH less than 6.0 is considered to be corrosive to buried metal piping and reinforced concrete structures. The samples had a pH of above 7.0, which does not present corrosion concerns for buried iron, steel, mortar-coated steel, or reinforced concrete structures.

Based on the resistivity measurements, both samples are classified as "corrosive" to buried metal piping. We recommend that in locations where corrosive soil is expected, buried structural elements that expose ferrous materials to the surrounding soil (utilities, rebar, etc.) are provided with suitable corrosion protection.

Values tested for chloride do not pose a significant impact to metals or concrete.

If it is desired to investigate this further, we recommend a corrosion consultant be retained to evaluate whether specific corrosion recommendations are advised for the project.

3.5 INFILTRATION CONSIDERATIONS

The geotechnical explorations generally indicate non-engineered fill in the upper 3 to 5 feet below ground surface across the site. As discussed in Section 2.6, non-engineered fill consisted of predominantly lean clay with sand, organics, and debris of brick and artificial fibrous material. Underlying the surficial non-engineered fill is native lean clay with varying amounts of sand.

We recommend infiltration rates be consistent with native clay soil with infiltration rates at approximately 0.1 inch per hour.

Reliance of the non-engineered fill for infiltration of stormwater is not recommended as rates are expected to vary dramatically across the site. If infiltration rates are desired to be higher, we recommend subsurface drainage systems be installed or local removal and replacement with granular material with consideration of the native clay below.

4.0 PRELIMINARY EARTHWORK RECOMMENDATIONS

4.1 DEMOLITION AND STRIPPING

Site development should commence with the removal of existing pavement and buildings as well as excavation and removal of buried structures, including utilities and foundations.

Existing vegetation should be removed from areas to receive fill or improvements and those areas to serve for borrow. Tree roots should be removed to a depth of at least 3 feet below existing grade. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations. All excavations from demolition below design grades should be cleaned to a firm undisturbed native soil surface as determined by our representative. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. All backfill material should be placed and compacted as engineered fill according to the recommendations in Sections 4.4 and 4.5.

4.2 EXISTING FILL REMOVAL

If existing fill is encountered during construction, we recommend removal of the fill to competent native soil, as evaluated by our field representative. If in a fill area, the base of the subexcavations should be processed, moisture conditioned (as needed), and compacted in accordance with the recommendations for engineered fill.

If existing fill is left in place in portions of the site that are being developed with walkways or other improvements that are not sensitive to settlement, ongoing maintenance should be anticipated.

If on-site recycled materials are being considered for reuse as engineered fill in SCVWD improvement areas, we recommend discussing suitability with the SCVWD prior to placing fill.

4.3 FILL COMPACTION

4.3.1 Grading in Structural Areas

After removing the loose soil, the contractor should scarify to a depth of at least 8 inches then moisture condition and compact the subgrade in accordance with the table below. The loose lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less.

TABLE 4.3.1-1: Fill Placement Requirements

MATERIALS		FILL LOCATION	MINIMUM RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT (PERCENTAGE POINTS ABOVE OPTIMUM)
Low-Expansive	PI < 25	General Fill	90	3
		Upper 6 inches in Pavement Areas	95	1

The contractor should compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557), at a moisture content above the optimum.

4.3.2 Landscape Fill

In landscaping areas, the contractor should process, place, and compact fill in accordance with Section 4.5.1, but to at least 85 percent relative compaction.

4.3.3 Underground Utility Backfill

The contractor is responsible for conducting trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe-bedding materials.

Utility trench backfill should conform to the recommendations in Section 4.5.1. Where utility trenches cross underneath buildings, we recommend that a plug be placed within the trench backfill to help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath or into the building. The plug should be constructed using a sand-cement slurry (minimum 28-day compressive strength of 500 psi) or relatively impermeable native soil for pipe bedding and backfill. We recommend that the plug extend a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

Jetting of backfill is not an acceptable means of compaction. Thicker loose lift thicknesses may be allowed based on acceptable density test results, where increased effort is applied to rocky fill, or for the first lift of fill over pipe bedding.

4.4 SITE DRAINAGE

The project Civil Engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, finish grades should be sloped away from buildings and pavements to the maximum extent practical. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations.

If landscaped areas are planned at finished grade elevations or on top of structures, proper subsurface drainage will be required to prevent ponding on covered roofs or along walls. The roofs and drainage systems should be designed with appropriate slopes to expediently transfer moisture across and off the roofs.

5.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

The main consideration in foundation design for this project is the potential for statically and seismically induced settlement. We developed preliminary foundation recommendations using data obtained from our exploration.

5.1 STRUCTURAL MAT FOUNDATIONS

A combination of a structural mat foundation and waterproofing is a common system for structures founded below the groundwater table. This option avoids the need for permanent dewatering. Based on the depth of the excavation and groundwater depths, the mat foundation will have to be designed to resist hydrostatic uplift forces. In addition, and based on the potential loading conditions of the structure, ground improvement under the mat foundation may be required. Design-level geotechnical evaluations should be performed for final design.

6.0 SECONDARY SLABS-ON-GRADE

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor plazas exposed to foot traffic only. Concrete flatwork should have a minimum thickness of 4 inches and include control and construction joints in accordance with current Portland Cement Association Guidelines.

Exterior slabs should slope away from the buildings to prevent water from flowing toward the foundations. Site soil should be moistened just prior to concrete placement.

We recommend that flatwork leading to a building entrance area be structurally independent of the building foundation to allow for differential movement between the flatwork and the building. Where smooth transition to provide access is necessary (ADA ramps), a hinged slab should be designed to accommodate movements of approximately 1 inch. Flatwork should be reinforced to allow for the appropriate span in the event of settlement. Maintenance or replacement of entry slabs should also be expected following a seismic event as the ground settles at the perimeter of buildings.

7.0 PRELIMINARY RECOMMENDATIONS FOR NON-BUILDING WALLS

7.1 PRELIMINARY SOIL PRESSURES

Non-building retaining walls may be required and can be designed for active lateral loading conditions. The recommended lateral equivalent fluid pressures (static case) are presented below.

TABLE 7.1-1: Lateral Earth Pressures

LOADING CONDITION	EQUIVALENT FLUID PRESSURES (PCF)	
	WITHOUT HYDROSTATIC PRESSURES (PCF)	WITH HYDROSTATIC PRESSURES (PCF)
Cantilevered (Active)	50	90

The above lateral earth pressures assume level backfill conditions. The design groundwater level should be assumed to be located at Elevation 47 feet. We recommend placing a drain behind all walls above the design groundwater level to reduce hydrostatic pressure; if a drain is not feasible, hydrostatic pressure should be added to the equivalent fluid pressure. Recommendations for wall drainage follow in the next section.

Where surcharge loads from vehicles or other loads are expected within a horizontal distance equal to the height of the walls, the walls should be designed for an additional uniform lateral pressure of 100 psf to be applied over the entire height of the wall or 10 feet, whichever is less.

7.2 RETAINING WALL DRAINAGE

Unless the full height of the basement walls is designed for hydrostatic pressures, these walls should be provided with wall drainage. Wall drainage may be provided using a 4-inch-diameter perforated pipe embedded in Class 2 permeable material, free-draining gravel surrounded by synthetic filter fabric, or prefabricated wall panels. The width of the drain blanket should be at least 12 inches. The drain blanket should extend from about 1 foot below the finished grades down to the design groundwater level elevation. The upper 1 foot of wall backfill should consist of clayey soil. Drainage should be allowed to equilibrate with the groundwater at the design level; no sumps or outfalls are necessary.

The foundation details and structural calculations for retaining walls should be submitted for our review.

7.3 SEISMIC DESIGN CONSIDERATIONS

Seismic conditions need to be considered in the design of the basement retaining walls. Under seismic conditions, the seismic force along the retaining wall should be added to the static active pressures, and can be calculated as follows.

$$\Delta P = 14 \text{ x } H^2$$

H is the design height of the wall (in feet) and ΔP is the seismic force in pounds per foot of wall. This force has a horizontal direction and should be applied at 0.3 x H from the base of the wall. Since seismic loading requires soil movement, evaluation of the seismic case should include adding the seismic increment to the active soil pressure for all wall types.

8.0 PAVEMENT DESIGN

We prepared a preliminary pavement design recommendations based on assumed Traffic Index and tested subgrade resistance values (R-value) of a sample collected within the upper 5 feet of soil in Boring 1-B01. The laboratory test result is attached in Appendix D and indicates that an R-value of 5 is appropriate for the pavement design. The TI should be determined by the Civil Engineer or appropriate public agency.

Due to variability in subsurface conditions, we recommend that if the subgrade material encountered is significantly different from the tested soil sample during this study, representative bulk samples of subgrade soil be obtained during rough grading to allow confirmation R-value testing for the design R-value used. Actual sections should be based on R-Value tests performed on samples of actual subgrade materials recovered on-site during construction.

8.1 FLEXIBLE PAVEMENTS

We developed the following pavement sections for parking areas and access streets using Traffic Indexes of 5 to 9, based on Topic 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety). This is for a 30-year design pavement life.

TABLE 8.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (AC) (INCHES)	CLASS 2 AGGREGATE BASE (AB) (INCHES)
5	4	9½
6	4	13½
7	4	17½
8	4½	20½
9	5	23½

The civil engineer should determine the appropriate traffic indexes based on the estimated traffic loads and frequencies.

8.2 RIGID PAVEMENTS

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections and reinforcement should be performed based on estimated traffic loads and frequencies.

Rigid pavement section should consist of Portland cement concrete paving (PCCP) over Class 2 aggregate base over prepared subgrade. The PCCP should achieve a minimum 28-day concrete compressive strength of 3,500 psi. Control joints, spaced in accordance with Caltrans guidelines, should also be considered. To reduce concrete cracking, No. 4 bars at 16 inches on center each way placed at mid-depth of the concrete section may be considered.

TABLE 8.2-1: Rigid Pavement Design Recommendations

TRAFFIC INDEX (TI)	R-VALUE OF 5 (UNTREATED SUBGRADE)	
	PCCP (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	6	6
6	6	8
7	6	10

8.3 PAVEMENT SUBGRADE PREPARATION

The contractor should compact finished subgrade and aggregate base in accordance with Section 5.5.1. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02b of the latest Caltrans Standard Specifications.

8.4 PERVIOUS PAVERS

We provide preliminary recommendations for vehicular pavers assuming a Traffic Index of 7.

In accordance with the guidelines provided by the Interlocking Concrete Pavement Institute (ICPI), the paver section may consist of 3.15-inch (80-millimeter) thick pavers on 1 inch of compacted bedding over 18 inches of AB. This section applies for a pervious or impermeable system. Concrete edge restraints should also be constructed to provide lateral constraint for the pavers. Construction and materials should follow the recommendations presented herein and within the ICPI specifications. Impacts from manmade factors such as over-irrigation, poor drainage, and/or leaking utilities may prematurely impact the subgrade soil and/or trench backfill under the paver areas, causing surface irregularities in the paver not associated with section design protocols.

Based on subsurface soil conditions and our performed infiltration test, water infiltration at the site is likely insufficient, as discussed in Section 3.5. Paver areas should be underlain by a subdrainage system to allow for rapid removal of water. The surface of the prepared subgrade should be sloped to drain toward the subdrain system and the top of pipe should be at or below the design rock section. The subdrain system should comprise 4-inch-diameter (SDR 35 or stronger) perforated pipe (perforations facing down), with glued joints and end caps. Prior to installation, the pipe should be wrapped in a 6-ounce filter fabric "sock." The pipe should be sloped a minimum of ½ percent to drain towards an outlet approved by the Civil Engineer. We can perform additional site-specific infiltration testing if desired to refine these recommendations.

We recommend a slope of 1 percent for pavement surfaces. Slopes of grid pavements should not exceed 5 percent. Slopes exceeding 3 percent typically require berms or check dams placed laterally over the soil subgrade to slow the flow of water and provide some infiltration.

8.5 CUT-OFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain towards pavement. If it is desired to install pavement cutoff barriers, they should be placed where pavement areas lie downslope of any landscape areas that are to be irrigated, and should extend to a depth of at least 6 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, the cutoff barrier may be eliminated.

9.0 GROUND HEAT EXCHANGE

Based on our findings and review of the proposed development, we consider the site to be highly suitable for using a Ground Heat-Exchange (GHX) system to achieve energy savings and to potentially eliminate the need for outdoor air conditioner units.

During our field investigation, we installed a closed-loop GHX system to test the thermal properties of the soil and groundwater conditions at the site.

For the thermal properties of the soil and groundwater conditions at the site, a closed-loop GHX system would likely be well suited and could be implemented on select buildings, or integrated into a project-wide system.

The TC testing was successfully completed in accordance with our conversations with you and with Geothermal Resource Technologies Inc. (GTRI) standard procedures. To perform the TC testing, we increased the depth of one of our geotechnical borings on the project site, oversaw the installation of the closed-loop geothermal system, observed the recommended minimum waiting period of 2 days, and coordinated with Air Connection to perform the testing over a 46-hour period. The overall procedure took place between November 18 and December 4, 2020. The boring is identified as 1-B01 (Figure 2).

Boring 1-B01 was advanced to a total depth of 103 feet below the ground surface (bgs). We sampled the boring to collect geotechnical specimens in the upper 61½ feet, and straight drilled to the final depth of 103 feet. We logged soil stratigraphy based on the retrieved soil samples and by observing the drilling cuttings, and checked them against previous ENGEO site observations.

At the completion of drilling, the drilling subcontractor inserted a 1-inch High Density Polyethylene (HDPE) U-bend loop into the borehole and grouted the borehole to the surface elevation using a tremie pipe. We left the test bore idle to equalize for longer than the recommended minimum waiting period of 2 days between grouting and testing. We started the Thermal Conductivity test on November 30, 2020. One Air Connection representative was on site to prepare the testing equipment. The testing duration was 46 hours.

The Air Connection test data and analysis report are attached in Appendix G. The test report provides a summary of the test procedure, analysis process, plots of loop temperature and input heat rate data. The results of formation thermal conductivity, thermal diffusivity, and undisturbed formation temperature for the borehole are summarized below in Table 9.0-1.

TABLE 9.0-1: Thermal Conductivity Test Results Summary

DESCRIPTION	FORMATION THERMAL CONDUCTIVITY	FORMATION THERMAL DIFFUSIVITY	UNDISTURBED FORMATION TEMPERATURE
1-B01	1.00 Btu/hr-ft-˚F	0.70 ft2/day	Approx. 66.1 ˚F

Drill logs show interbedded poorly graded gravel, poorly graded sand with gravel, sandy silt, silt, and lean clay. Typical values of thermal conductivity for these strata range between 0.8 and 1.2 Btu/hr-ft-°F (Kavanaugh and Rafferty, Geothermal heating and cooling: Design of ground-source heat pump systems published by ASHRAE, 2014). Values from this site investigation can be said to be within anticipated typical thermal characteristics for these strata.

As project planning progresses into architectural design, we can meet with you, your architect, and your MEP designer to further assess and develop GHX energy saving opportunities and efficiencies.

10.0 PRELIMINARY DEWATERING-INDUCED SETTLEMENT ASSESSMENT

As requested, we performed an assessment to evaluate potential settlement as a result of possible dewatering activities for the Ellis Street sewer line augmentation trench, and District System utility installation trench.

We used MODFLOW to estimate groundwater drawdown and pumping rate for the proposed excavations. MODFLOW is a three-dimensional finite-difference groundwater modeling software developed by the United States Geological Survey (USGS), and is considered to be the international standard for simulating and predicting groundwater conditions.

Our MODFLOW model consists of a horizontal network of 10-foot-by-10-foot grid cells, and is vertically discretized into three layers based on our interpretation of exploration logs of the underlying soil stratigraphy. We selected hydraulic conductivity and vertical anisotropy values based on grain-size data from our recent exploration, previous experience, and relevant literature. The model layers and parameters are summarized in Table 10.0-1. We varied the thickness of model layer 2 (aquifer layer) between 10 feet and 17 feet to provide a range of the estimated pumping rate for each excavation.

TABLE 10.0-1: MODFLOW Model Layers and Parameters

MODEL LAYER	HYDROGEOLOGIC UNIT	ELEVATION (feet, NAVD88)	HORIZONTAL HYDRAULIC CONDUCTIVITY, K_x (ft/day)	VERTICAL ANISOTROPY, K_x/K_z
1	CL	58 to 45	0.028	4
2	SP	45 to 28	28	1
3	CL	28 to 0	0.028	4

We assumed the ground surface to be at Elevation 58 feet (NAVD 88) and the initial groundwater table to be at Elevation 47 feet (11 feet below ground surface) for each excavation dewatering analysis. We modeled two proposed excavations: Ellis Street sewer line trench, and District System lines trench. The Ellis Street sewer line trench excavation elevation was based on our review of existing utility plans, which showed sewer line invert elevations at nearby manholes. The District System Line installation trench excavation elevation is to be determined, so we modeled scenarios where it ranges between 15 and 20 feet below ground surface. The excavations were dewatered separately based on our understanding of the project schedule and sequencing. We assumed the desired drawdown elevation to be approximately 3 feet below the bottom of excavation elevation. In addition, we conservatively assumed the excavation shoring to be 100% permeable.

We modeled a steady-state dewatering condition, and assumed a constant recharge rate based on the region's average monthly rainfall depths. Our preliminary modeling results for each excavation are summarized in Table 10.0-2. Figures of the dewatering zone of influence for each excavation are included as Figures 9A to 9C. While our analysis is generally conservative, variations in local hydrogeologic conditions may require higher pumping rates to achieve dry working conditions in all regions of each excavation. We recommend a comprehensive dewatering analysis be performed once the project progresses into the final design phase.

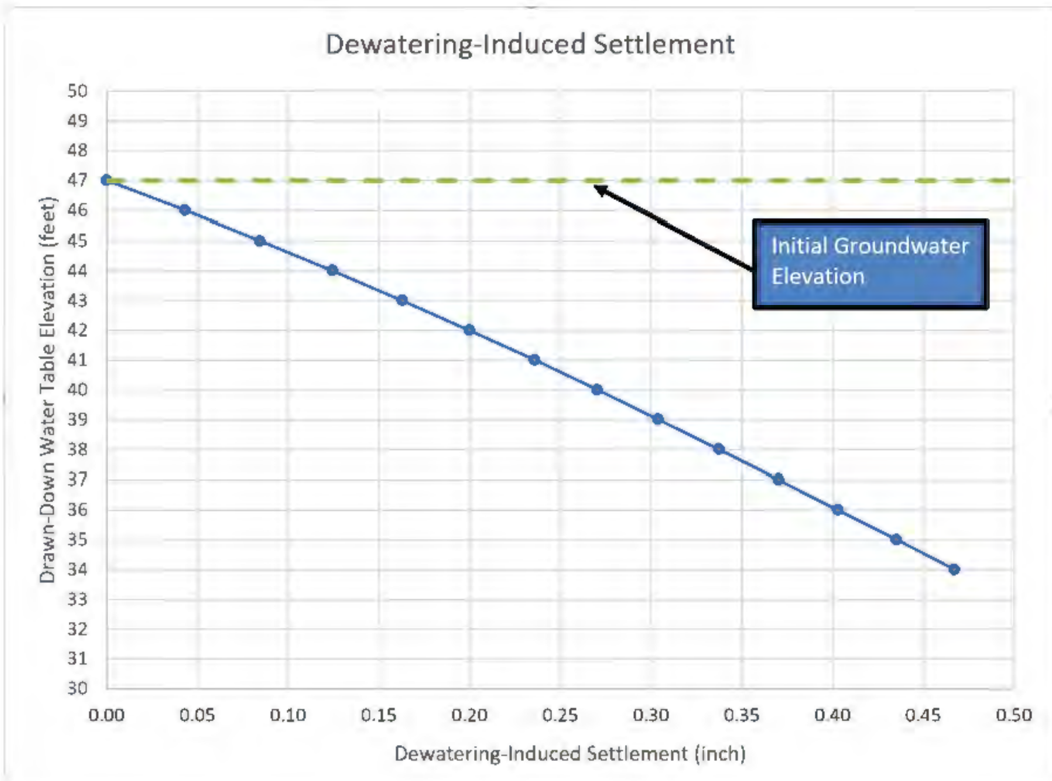
TABLE 10.0-2: Excavation Details and Preliminary Pumping Rates

EXCAVATION	APPROXIMATE EXCAVATION DEPTH (feet)	BOTTOM OF EXCAVATION ELEVATION (feet, NAVD 88)	DESIRED DRAWDOWN ELEVATION (feet, NAVD 88)	ESTIMATED PUMPING RATE (gpm)
Ellis St Sewer Line Trench	11 to 17	47 to 41	44 to 38	40 to 70
District System Lines Trench – Scenario 1	15	43	40	45 to 60
District System Lines Trench – Scenario 2	20	38	35	60 to 80

Based on the results of our analysis, we anticipate vertical settlements within influenced areas, as shown in Figures 9A to 9C, to be ½ inch or less due to groundwater level drawdown resulting from all aforementioned dewatering activities. If the predicted settlements are unacceptable, a cutoff wall and internal dewatering may be considered along the excavation perimeters to mitigate dewatering-induced settlement impacts.

The results of our settlement analyses are presented in Exhibit 10.0-1. Figures 9A to 9C also present the dewatering analysis results in terms of drawdown elevation and associated settlement induced for the discussed cases.

EXHIBIT 10.0-1: Settlement Analysis Results



11.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents preliminary geotechnical recommendations for the East Whisman Phase 1 project discussed in Section 1.3. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, designers, and contractors. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; there is no warranty, express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, fill, and groundwater, additional unexpected costs may be incurred in completing the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO should be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include soil volume change factors or flood potential. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials should be notified immediately.

This document must not be subject to unauthorized reuse, that is, reuse without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document’s applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO’s recommendations. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO’s scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We determined the boundaries designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

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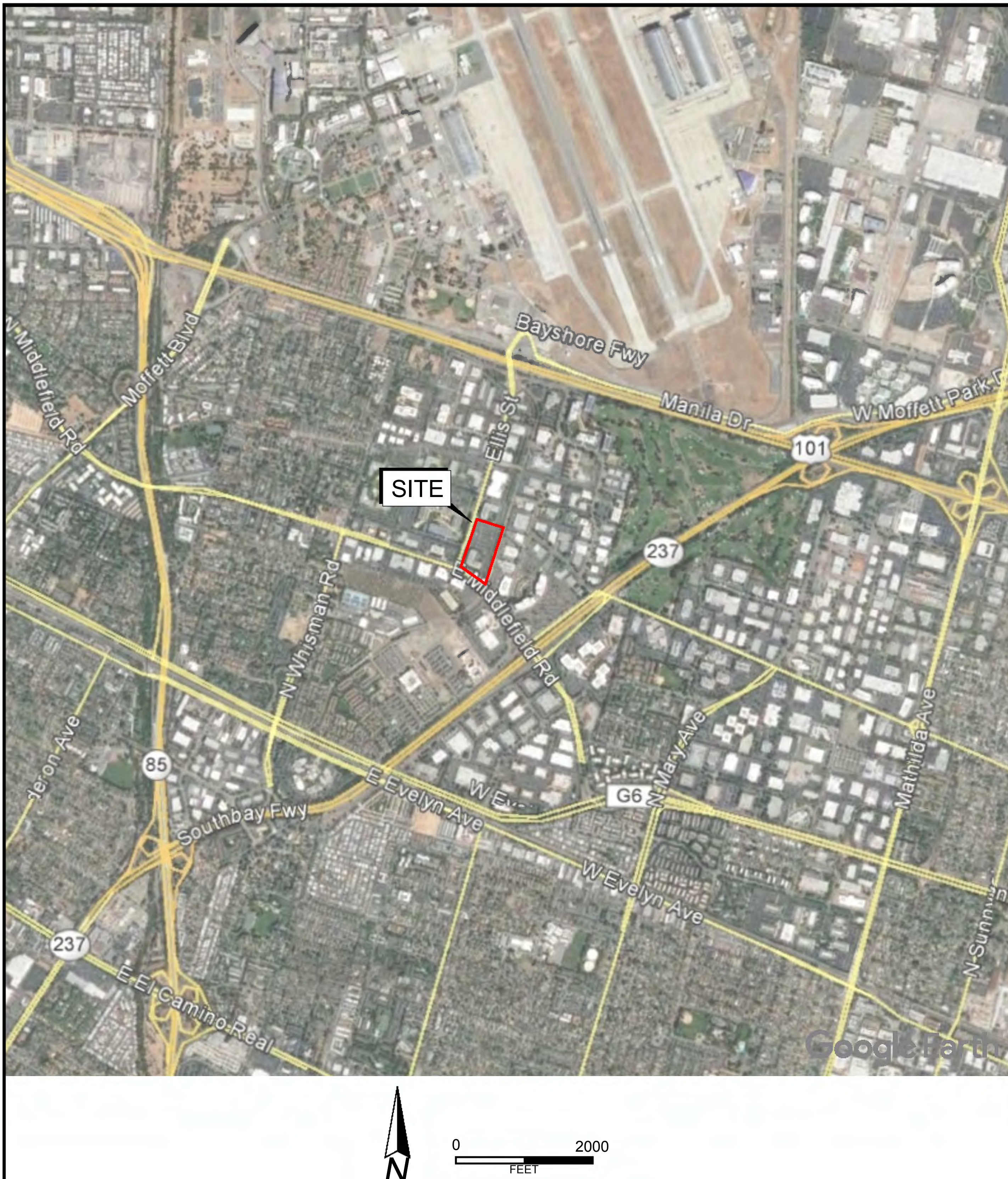
Zhang, G. Robertson. P.K, Brachman, R., (2002), Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180.

FIGURES



BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence

VICINITY MAP	PROJECT NO.: 17954.000.001	FIGURE NO.
EAST WHISMAN PHASE 1	SCALE: AS SHOWN	1
MOUNTAIN VIEW, CALIFORNIA	DRAWN BY: LL / CHECKED BY: PE	

ORIGINAL FIGURE PRINTED IN COLOR

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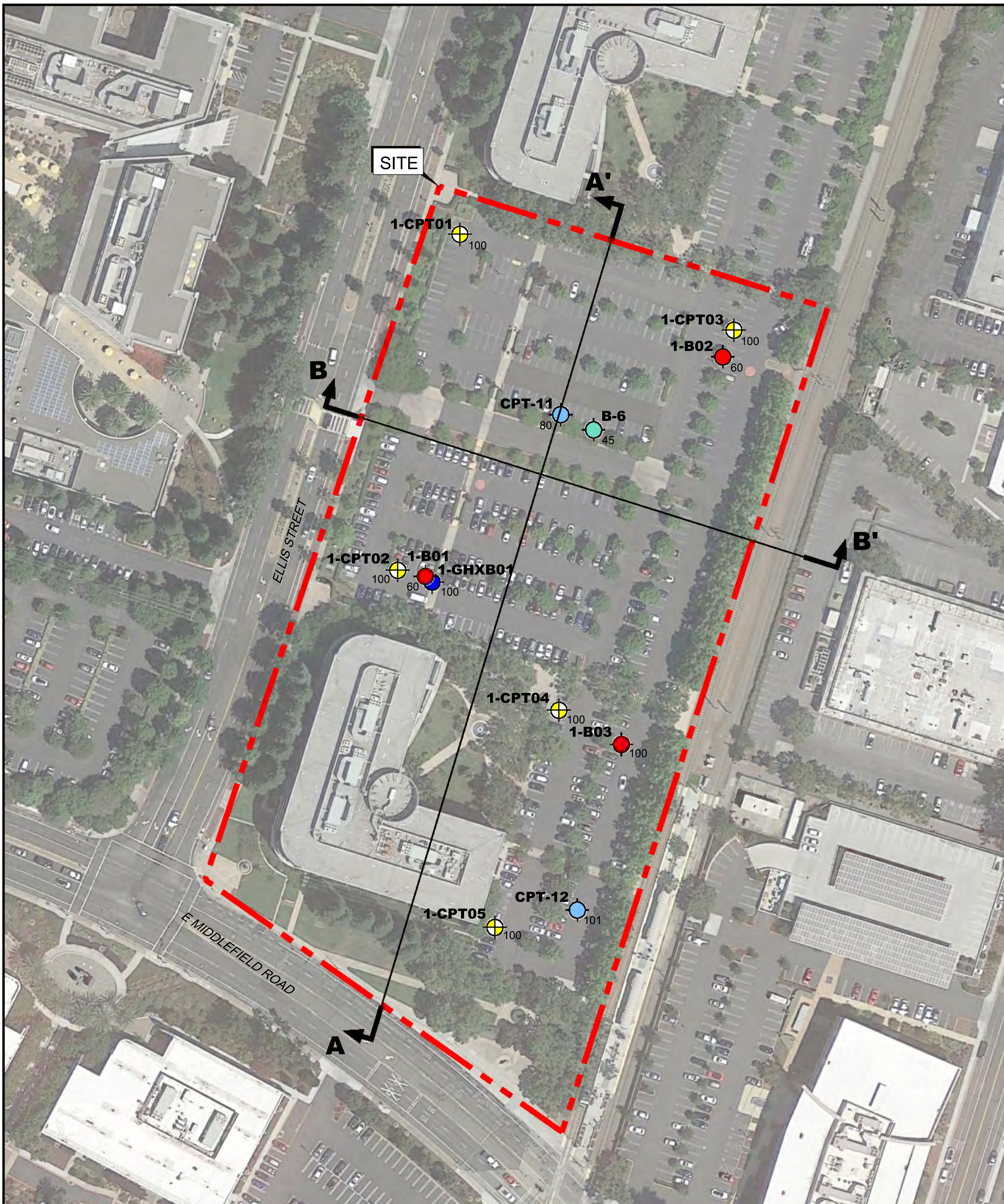


EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- BORING, WITH DEPTH SHOWN IN FEET (ENGEO, 2020)
- BORING (NINYO & MOORE, 2019)
- CONE PENETRATION TEST, WITH DEPTH SHOWN IN FEET (ENGEO, 2020)
- CONE PENETRATION TEST (NINYO & MOORE, 2019)
- GROUND HEAT EXCHANGE TESTING, WITH DEPTH SHOWN IN FEET (ENGEO, 2020)

B B' — CROSS SECTION LOCATION

N

0 100
FEET

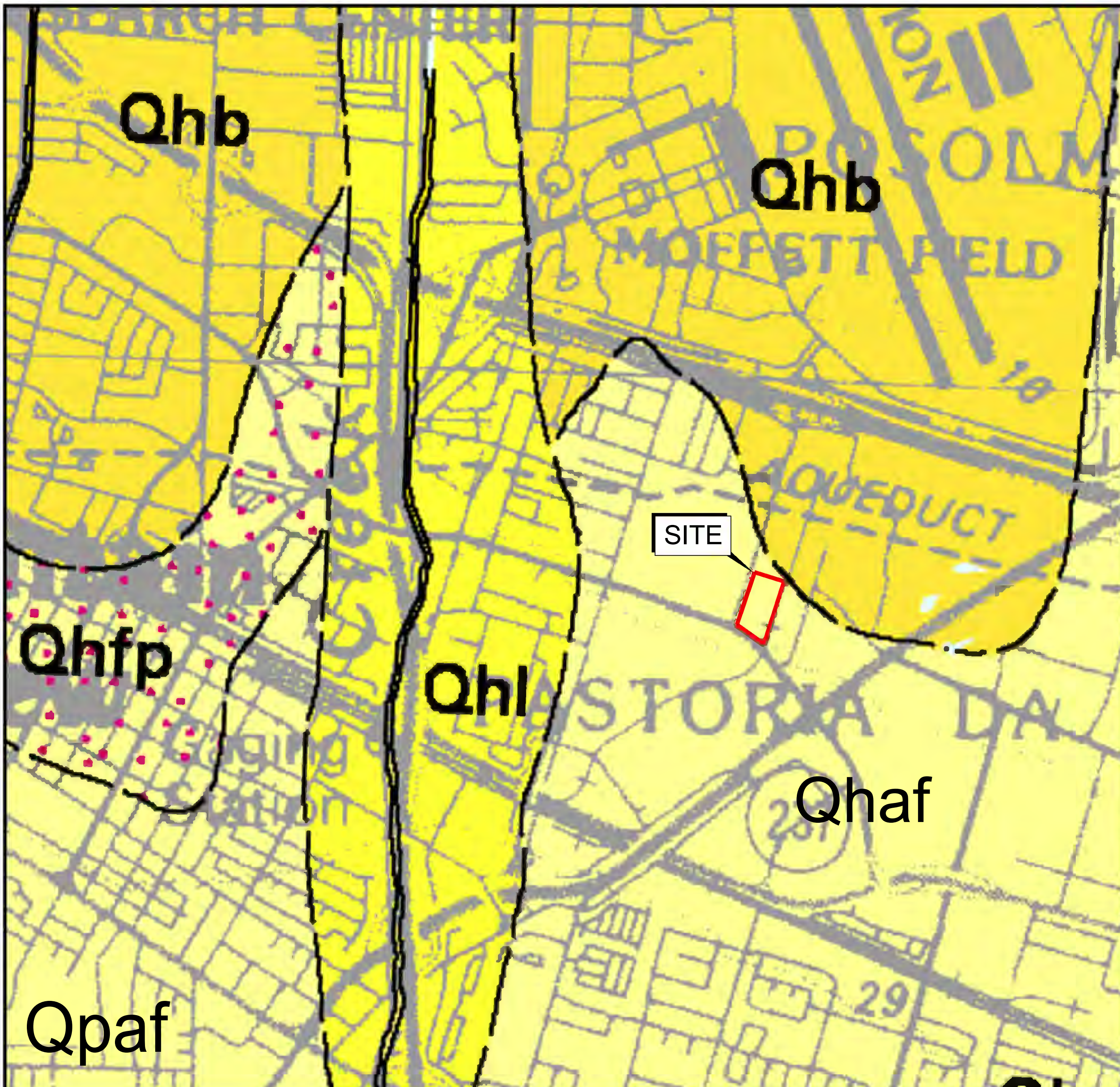
BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence

SITE PLAN
EAST WHISMAN PHASE 1
MOUNTAIN VIEW, CALIFORNIA

PROJECT NO.: 17954.000.001
SCALE: AS SHOWN
DRAWN BY: LL
CHECKED BY: PE
FIGURE NO. 2

ORIGINAL FIGURE PRINTED IN COLOR



N

0 2000
FEET

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qhb	BASIN DEPOSITS (HOLOCENE)
Qhbs	BASIN DEPOSITS, SALT-AFFECTED (HOLOCENE)
Qhfp	FLOOD-PLAIN DEPOSITS (HOLOCENE)
Qhl	NATURAL LEVEE DEPOSITS (HOLOCENE)
Qhaf	ALLUVIAL AND FLUVIAL FAN DEPOSITS (HOLOCENE)
Qpaf	ALLUVIAL FAN AND FLUVIAL DEPOSITS (PLEISTOCENE)

BASE MAP SOURCE: BRABB, GRAYMER, AND JONES, 2000

ENGEO — Expect Excellence

REGIONAL GEOLOGIC MAP
EAST WHISMAN PHASE 1
MOUNTAIN VIEW, CALIFORNIA

PROJECT NO.: 17954.000.001	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: PE

ORIGINAL FIGURE PRINTED IN COLOR

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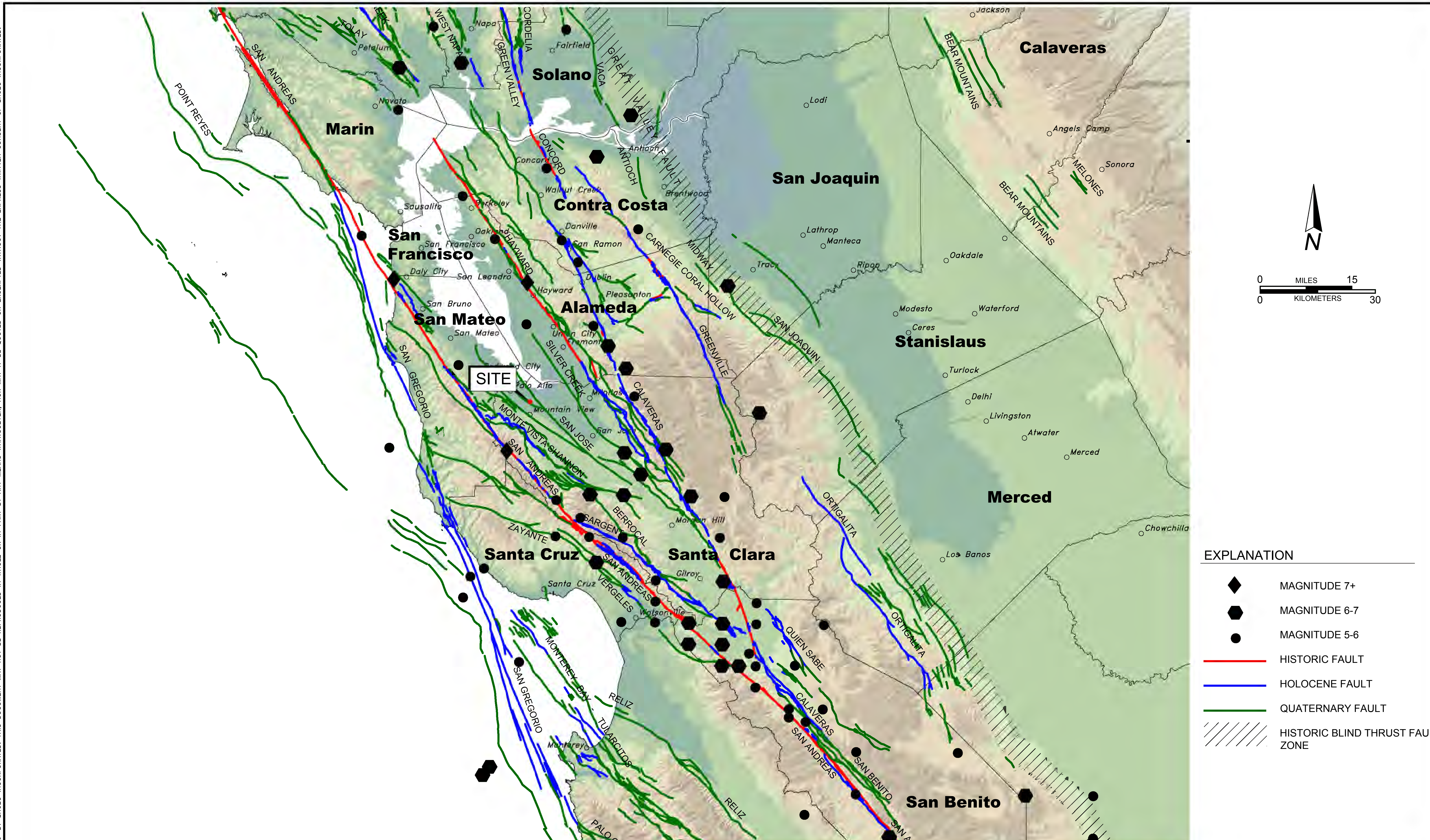

BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2016

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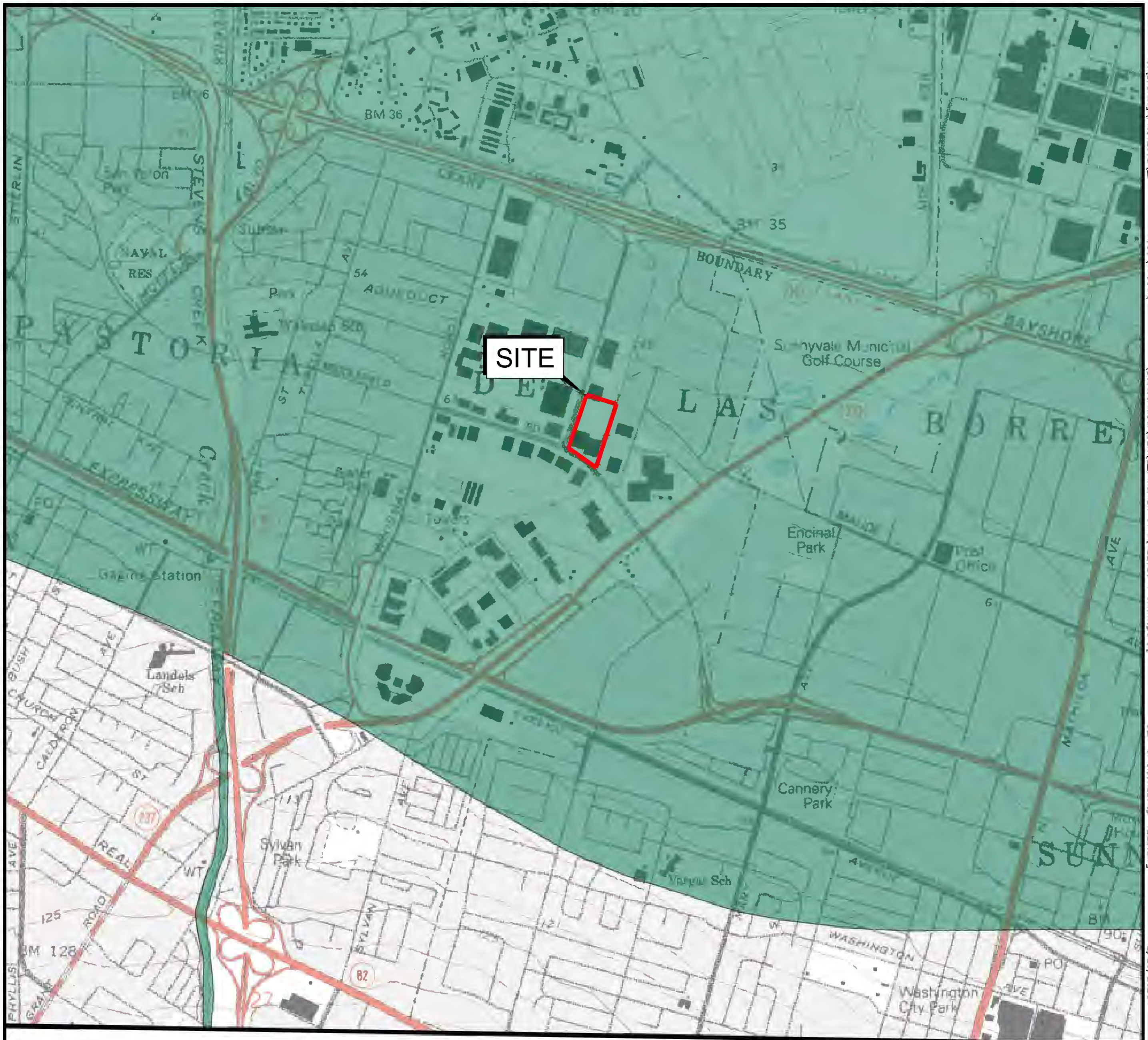
REGIONAL FAULTING AND SEISMICITY
EAST WHISMAN PHASE 1
MOUNTAIN VIEW, CALIFORNIA

PROJECT NO.: 17954.000.001		FIGURE NO. 4
SCALE: AS SHOWN		
DRAWN BY: LL	CHECKED BY: PE	

FILE PATH: G:\Drafting\PROJECTS_16000 to 17999\17954\17954000001\Geotechnical\GEX\E_Whisman-PH1\122920-PrelimReport\17954000001-GEX-4-FAULTMAP-1220.dwg SAVE DATE: 12/29/2020 11:24:54 AM SAVED BY: llee

ORIGINAL FIGURE PRINTED IN COLOR



(CUPERTINO)

0 2000
FEET

N

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

LIQUEFACTION

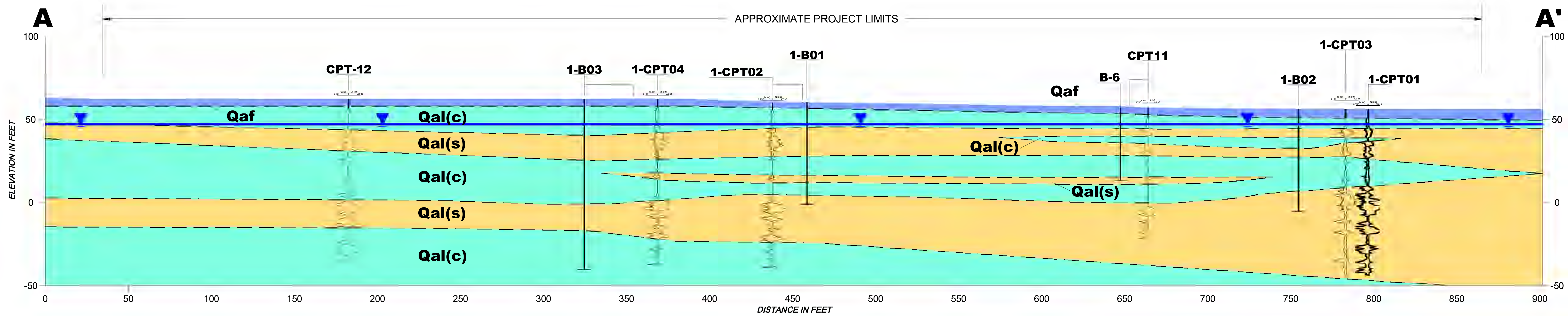
AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2016

ENGEO Expect Excellence	SEISMIC HAZARD ZONES MAP EAST WHISMAN PHASE 1 MOUNTAIN VIEW, CALIFORNIA	PROJECT NO.: 17954.000.001		FIGURE NO. 5
		SCALE: AS SHOWN		
		DRAWN BY: LL	CHECKED BY: PE	

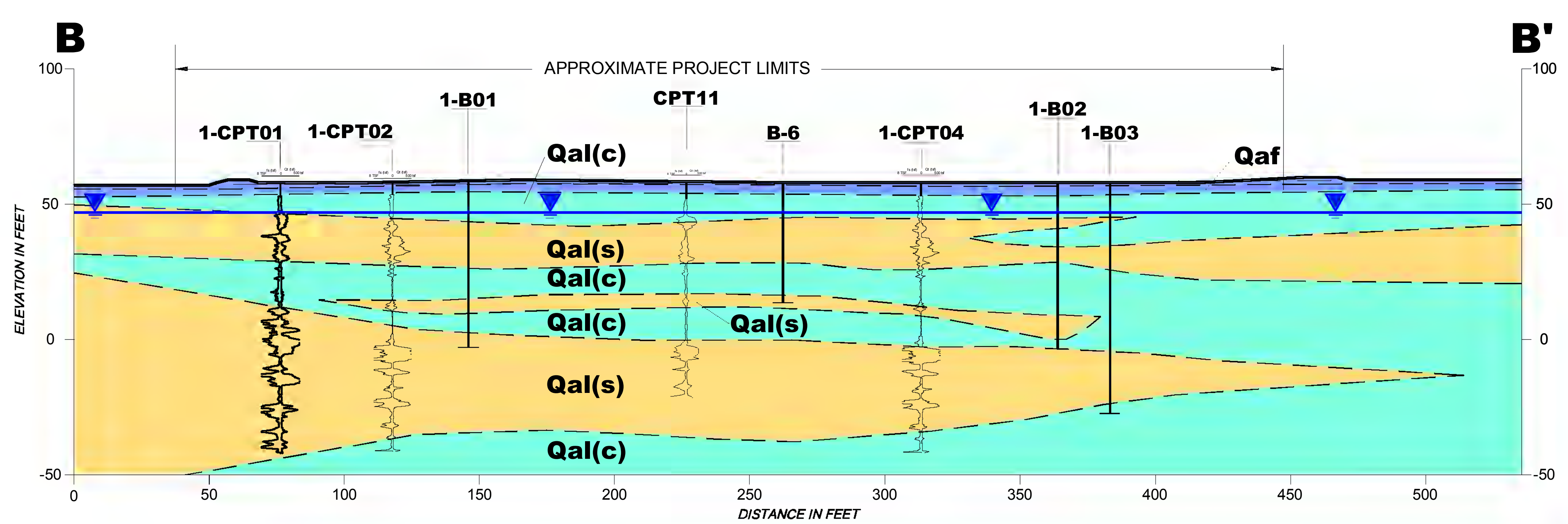
ORIGINAL FIGURE PRINTED IN COLOR

FILE PATH: G:\Drafting\PROJECTS_16000 to 17999\17954\179540000001\Geotechnical\GEX\E_Whisman-PH1\122920-PrelimReport\179540000001-GEX-5-SeisHazMap-1220.dwg SAVE DATE: 12/29/2020 11:24:58 AM SAVED BY: llee



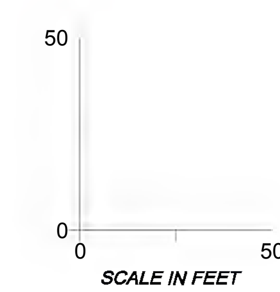
SECTION A-A'

1"=50'



SECTION B-B'

1"=50'



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qaf	FILL
Qal(s)	SANDY ALLUVIUM
Qal(c)	CLAYEY ALLUVIUM
1-B03	BORING (ENGEO, 2020)
B-6	BORING (NINYO & MOORE, 2019)
1-CPT05	CONE PENETRATION TEST (ENGEO, 2020)
CPT12	CONE PENETRATION TEST (NINYO & MOORE, 2019)
▼	GROUNDWATER LEVEL AT ELEVATION 47

ENGEO Expect Excellence	CROSS SECTION A-A' AND B-B' EAST WHISMAN PHASE 1 MOUNTAIN VIEW, CALIFORNIA	PROJECT NO.: 17954.000.001 SCALE: AS SHOWN DRAWN BY: LL / CHECKED BY: PE	FIGURE NO. 6

ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

SPECI[redacted]D HAZARD AREAS SUBJECT TO INUNDATION BY TH[redacted]NUAL CHANCE FLOOD

The 1% annual flood (100-[redacted], also known as the base flood, is the flood that has a 1% chance of being equaled o[redacted] in any given year. The Special Flood Hazard Area is the area subject to flooding by[redacted]nnual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR[redacted]and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X Areas determined to be outside the 0.2% annual chance floodplain.

ZONE D Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

1% annual chance floodplain boundary

0.2% annual chance floodplain boundary

Floodway boundary

Zone D boundary

CBRS and OPA boundary

Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

513	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988

A — A	Cross section line
23 - - - 23	Transect line
87°07'45", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
$^{24}76^{000m}$N	1000-meter Universal Transverse Mercator grid values, zone 10N
600000 FT	5000-foot grid ticks: California State Plane coordinate system, zone III (FIPSZONE 0403), Lambert Conformal Conic projection
DX5510 ×	Bench mark (see explanation in Notes to Users section of this FIRM panel)
● M1.5	River Mile

N

0 800
FEET

BASE MAP SOURCE: FEMA FIRM MAP

ENGEO Expect Excellence	FEMA FLOOD INSURANCE MAP EAST WHISMAN PHASE 1 MOUNTAIN VIEW, CALIFORNIA	PROJECT NO.: 17954.000.001 SCALE: AS SHOWN DRAWN BY: LL / CHECKED BY: PE	FIGURE NO. 7

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ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

Tsunami Inundation Line

Tsunami Inundation Area

N

0 800
FEET

BASE MAP SOURCE: STATE OF CALIFORNIA – TSUNAMI INUNDATION MAP

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TSUNAMI INUNDATION MAP
EAST WHISMAN PHASE 1
MOUNTAIN VIEW, CALIFORNIA

PROJECT NO.: 17954.000.001	
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: PE

FIGURE NO.
8

ORIGINAL FIGURE PRINTED IN COLOR

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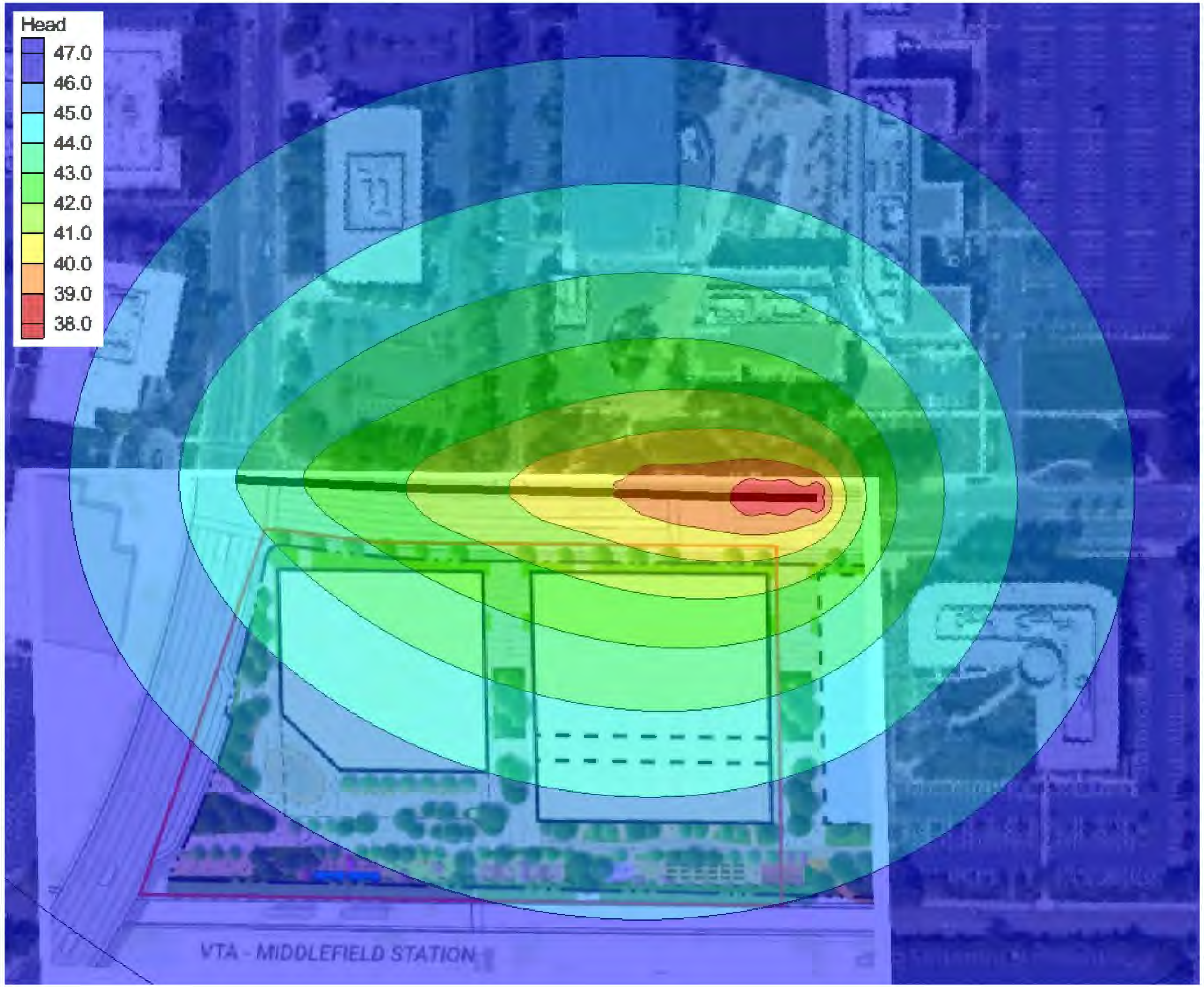

ELLIS STREET SEWER

(INVERT STARTS AT ELEVATION 47' AND ENDS AT ELEAVTION 41')

DATA TABLE

Drawn-down WT elevation (ft NAVD88)	Dewatering Induced Settlement (inch)
47	0.00
46	0.04
45	0.08
44	0.12
43	0.16
42	0.20
41	0.24
40	0.27
39	0.30
38	0.34

* ASSUMING FREE GROUND CONDITIONS, WITH NO EFFECT FROM BUILDING LOADS

N

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

ELLIS STREET SEWER LINE

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DEWATERING DRAW-DOWN
AND INDUCED SETTLEMENT MAP
EAST WHISMAN PHASE 1
MOUNTAIN VIEW, CALIFORNIA

PROJECT NO.: 17954.000.001	FIGURE NO. 9A
SCALE: NO SCALE	
DRAWN BY: LL	CHECKED BY: PD

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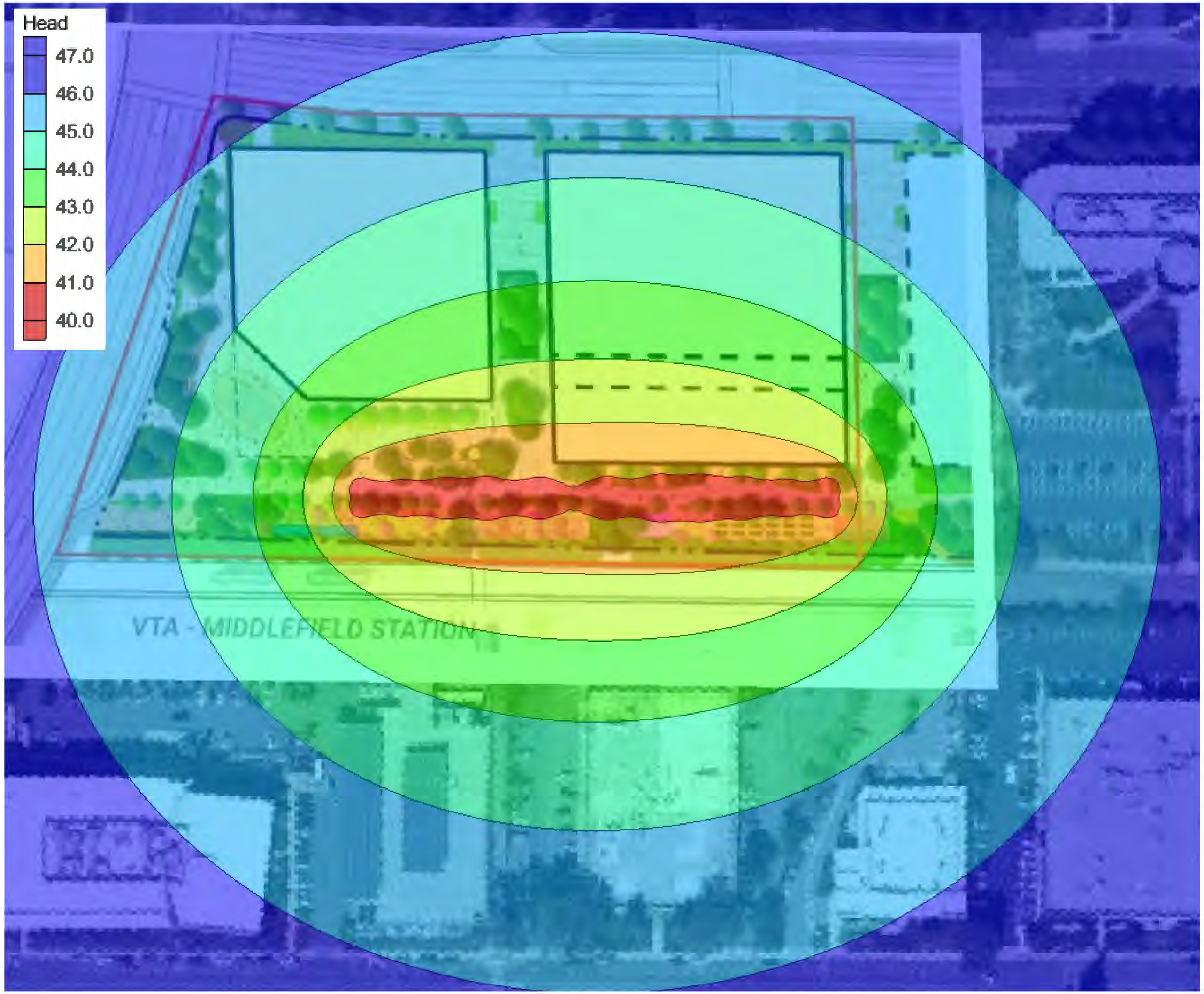
ORIGINAL FIGURE PRINTED IN COLOR



DISTRICT SYSTEM UTILITY TRENCH

(SCENARIO 1, TRENCH DEPTH = 15' bgs)

DATA TABLE

Drawn-down WT elevation (ft NAVD88)	Dewatering Induced Settlement (inch)
47	0.00
46	0.04
45	0.08
44	0.12
43	0.16
42	0.20
41	0.24
40	0.27

* ASSUMING FREE GROUND CONDITIONS, WITH NO EFFECT FROM BUILDING LOADS

N

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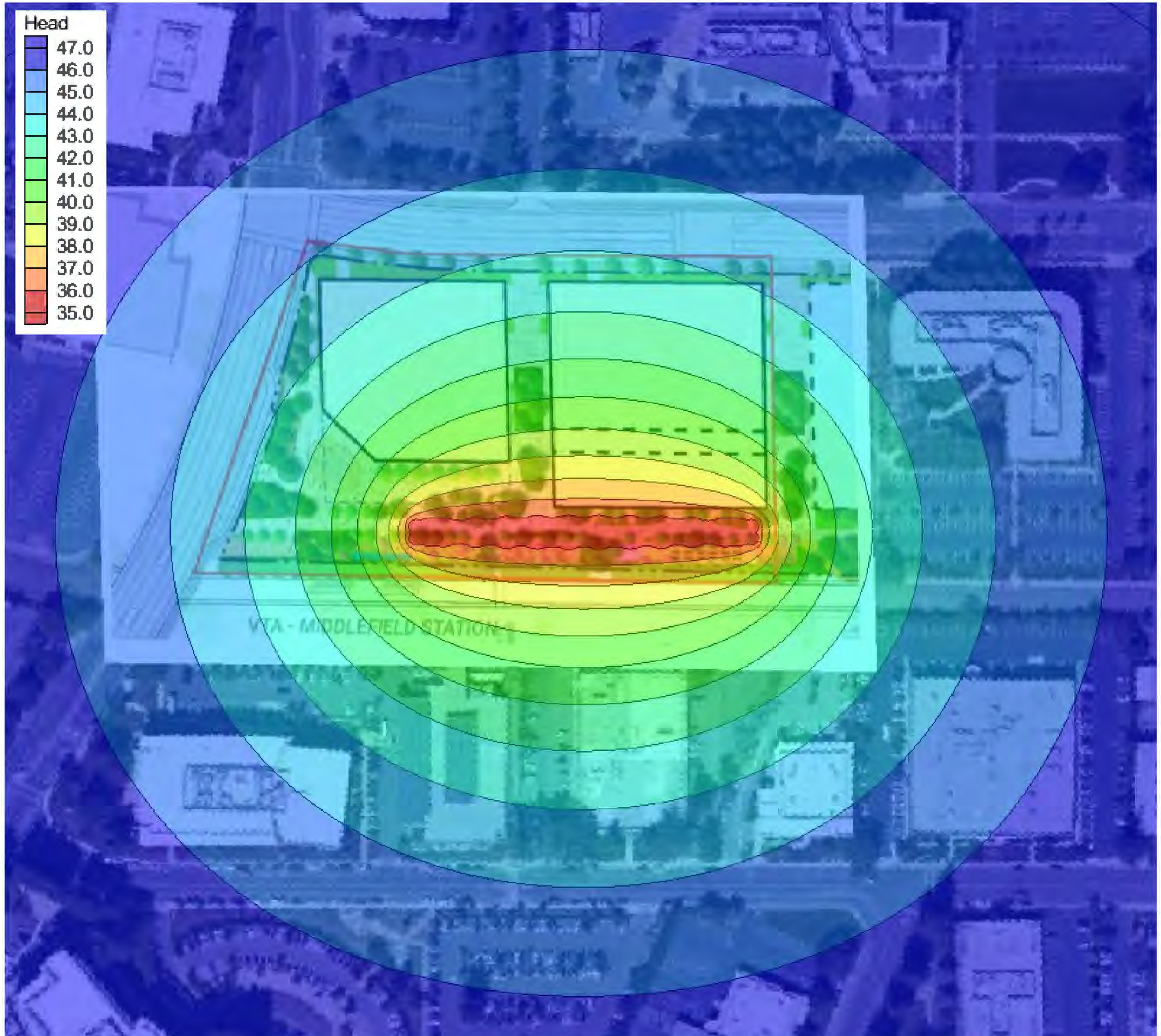
DEWATERING DRAW-DOWN
AND INDUCED SETTLEMENT MAP
EAST WHISMAN PHASE 1
MOUNTAIN VIEW, CALIFORNIA

PROJECT NO.: 17954.000.001
SCALE: NO SCALE
DRAWN BY: LL
CHECKED BY: PD
FIGURE NO. 9B



DISTRICT SYSTEM UTILITY TRENCH

(SCENARIO 2, TRENCH DEPTH = 20' bgs)

DATA TABLE

Drawn-down WT elevation (ft NAVD88)	Dewatering Induced Settlement (inch)
47	0.00
46	0.04
45	0.08
44	0.12
43	0.16
42	0.20
41	0.24
40	0.27
39	0.30
38	0.34
37	0.37
36	0.40
35	0.44

* ASSUMING FREE GROUND CONDITIONS, WITH NO EFFECT FROM BUILDING LOADS

N

ENGEO Expect Excellence	DEWATERING DRAW-DOWN AND INDUCED SETTLEMENT MAP EAST WHISMAN PHASE 1 MOUNTAIN VIEW, CALIFORNIA	PROJECT NO.: 17954.000.001 SCALE: NO SCALE DRAWN BY: LL / CHECKED BY: PD	FIGURE NO. 9C

FILE PATH: G:\Drafting\PROJECTS_16000 to 17999\17954\17954000001\Geotechnical\GEX\E_Whisman-PH1\122920-PrelimReport\17954000001-GEX-9-EstStatic-1220.dwg SAVE DATE: 12/29/2020 10:34:45 AM SAVED BY: llee

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

EXPLORATION LOGS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity
			CH - Fat clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Dames and Moore Piston
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

________	Solid - Layer Break
– – – – – –	Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- ▽ Groundwater level during drilling
- ▼ Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

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LOG OF BORING 1-B01

ENGEO
Expect Excellence

LATITUDE: 37.396749 | LONGITUDE: -122.052878

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/18/2020
HOLE DEPTH: 61.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 61 ft.

LOGGED / REVIEWED BY: A. Robertson / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		3" Asphalt concrete (AC)										
	60	15" Aggregate Base (AB), brown to dark brown, dry to moist										
		LEAN CLAY (CL), dark brown to black, moist, approximately 0 to 5% fine- to coarse-grained sand, rootlets, trace brick debris [FILL]										
		Cement-soil mix										
5		LEAN CLAY WITH SAND (CL), dark yellowish brown, medium stiff, moist, low plasticity, carbonates, iron oxide staining [NATIVE]										
	55		12	34	18	16	79			640*	1.0*	PP+TV
		Medium-grained sand, reduced sand content	9							700*	0.9*	PP+TV
10	50	Becomes stiff, approximately 0 to 5% coarse-grained sand, approximately 0 to 5% rounded fine gravel	17					25.6	99.3	2801		UU
		Becomes olive gray, very stiff, wet, low plasticity, carbonates, iron oxide staining	9								2.5*	PP
15			18									
	45	POORLY GRADED SAND WITH GRAVEL (SP), olive gray mottled with yellow, medium dense to dense, moist, approximately 5 to 10% silt, approximately 5 to 10% subrounded fine gravel										
		Rounded to coarse gravel, pockets of yellow fine-grained sand	37				10	9.2				
20	40											
			29									
25												

LOG OF BORING 1-B01

LATITUDE: 37.396749 LONGITUDE: -122.052878

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/18/2020
HOLE DEPTH: 61.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 61 ft.

LOGGED / REVIEWED BY: A. Robertson / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet: 30, 35, 40, 45, 50
Elevation in Feet: 35, 30, 25, 20, 15

DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
POORLY GRADED SAND WITH GRAVEL (SP), olive gray mottled with yellow, medium dense to dense, moist, approximately 5 to 10% silt, approximately 5 to 10% subrounded fine gravel										
	65				11	8.5				
Becomes very dense, weakly stratified, with layers of fine-grained sand and medium- to coarse-grained sand with fine to coarse gravel, iron oxide staining										
LEAN CLAY TO SILT (CL-ML), gray, very stiff, moist to wet, approximately 0 to 5% fine- to medium-grained sand										
	95 to 200 psi	23	16	7		16.4	111.8	1060*	2.75*	PP+TV
LEAN CLAY (CL), light bluish gray, very stiff, moist, high plasticity, approximately 0 to 5% coarse-grained sand and fine fravel-sized angular calcitic concretions										
	95 to 225 psi					25.3	98.5	1784.7		UU
SANDY SILT (ML), dark bluish gray, loose to medium dense, moist, approximately 0 to 5% rounded fine gravel, approximately 0 to 5% rounded coarse-grained sand, trace dark reddish brown woody fibers, pockets of greenish gray fine sand										
	22				65	18.2	115.1		2*	PP
	11									

ENGEO
Expect Excellence

LOG OF BORING 1-B01

LATITUDE: 37.396749 LONGITUDE: -122.052878

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/18/2020
HOLE DEPTH: 61.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 61 ft.

LOGGED / REVIEWED BY: A. Robertson / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	10		LEAN CLAY WITH SAND (CL), pale bluish gray, soft, moist, carbonates, approximately 0 to 5% rounded fine gravel, approximately 0 to 5% rounded coarse-grained sand, trace dark reddish brown woody fibers, pockets of greenish gray fine sand Becomes pale olive brown with iron oxide staining, approximately 20-30% sand content			3	38	20	18				240*	0.5*	PP+TV
55	5		POORLY GRADED GRAVEL (GP), brownish gray, wet, fine, angular, identified in cuttings												
60	0		SILT (ML), greenish gray mottled with olive brown, stiff, moist, carbonates, approximately 0 to 5% medium- to coarse-grained sand			20	31	23	8		25.6	101	810		UU
			Geotechnical logging terminated at approximately 61.5 feet below ground surface. Groundwater not observed during drilling due to drilling method. Boring advanced to approximately 103 feet below grounds surface for geothermal pump installation.												

LOG OF BORING 1-B02

ENGEO — Expect Excellence

LATITUDE: 37.397348 LONGITUDE: -122.05185

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/17/2020
HOLE DEPTH: 61.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 55 ft.

LOGGED / REVIEWED BY: A. Robertson / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		2" Asphalt Concrete (AC)										
		16" Aggregate Base (AB), brown, dry										
		LEAN CLAY WITH SAND (CL), dark brown, moist, angular medium- to coarse-grained sand, fine to coarse gravel, quartz [FILL] Becomes brown, fine-grained sand, subrounded fine to medium gravel, high plasticity, pockets of red-brown fibrous organics Becomes light yellowish brown mottled with gray, pockets of reddish yellow, weak lamination										
5	50	LEAN CLAY (CL), gray mottled with dark yellowish brown, hard, moist, low plasticity, white to light gray calcite stringers, weak lamination [NATIVE]	22				79			2750*	>4.5*	PP+TV
10	45	Becomes pale olive brown mottled with light yellowish red, very stiff to hard, moist, fine- to coarse-grained sand, iron oxide staining, approximately 0 to 5% subrounded fine gravel	19							2250*	3.5*	PP+TV
15	40	POORLY GRADED SAND WITH GRAVEL (SP), brown, dense, moist to wet, approximately 30 to 40% fine to coarse subrounded to subangular gravel	37				10	28.4	97			
20	35	LEAN CLAY TO SILT (CL-ML), pale olive, stiff to very stiff, moist to wet	95 to 250 psi	30	28	2	92	27.8	99.3	500*	2.0*	PP+TV
25	30											

ENGEO *Expect Excellence*

LOG OF BORING 1-B02

LATITUDE: 37.397348 LONGITUDE: -122.05185

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/17/2020
HOLE DEPTH: 61.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 55 ft.

LOGGED / REVIEWED BY: A. Robertson / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			POORLY GRADED SAND (SP), gray, dense, moist to wet, approximately 10 to 15% rounded to subangular fine to coarse gravel			34									
30	25		LEAN CLAY (CL), gray, very stiff, moist, approximately 0 to 5% coarse-grained sand, approximately 0 to 5% rounded coarse gravel			16	38	18	20		27.8	96.8	1320*	2.5*	PP+TV
35	20		Becomes stiff, increased silt content			50 to 175 psi							700*	.75*	PP+TV
40	15		Becomes low plastic, decreased silt content, volcanic and metamorphic coarse rounded gravel			17					24.9	101.2	924.68		UC
45	10		Becomes very stiff, approximately 5 to 10% fine-grained sand, increased silt content, iron oxide staining, trace black medium- to coarse-grained sand			19								2.75*	PP
			SILTY SAND WITH GRAVEL (SM), yellowish brown, very dense, moist to wet, subrounded medium to coarse gravel												
50	5														

ENGEO
Expect Excellence

LOG OF BORING 1-B02

LATITUDE: 37.397348 LONGITUDE: -122.05185

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/17/2020
HOLE DEPTH: 61.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 55 ft.

LOGGED / REVIEWED BY: A. Robertson / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SILTY SAND WITH GRAVEL (SM), yellowish brown, very dense, moist to wet, subrounded medium to coarse gravel			53				23					
55	0		Increased gravel content			60									
			SANDY SILT WITH GRAVEL (ML), gray, very stiff, moist to wet, fine to medium gravel												
60	-5					37					14.5	118.8	443		UC
			Boring terminated at approximately 61.5 feet below ground surface. Groundwater was not observed during drilling due to drilling method. Casing installed to approximately 50 feet below ground surface.												

ENGEO
Expect Excellence

LOG OF BORING 1-B03

LATITUDE: 37.396285 LONGITUDE: -122.052225

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/13/2020
HOLE DEPTH: 102.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 61 ft.

LOGGED / REVIEWED BY: C. Nicas / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			2" Asphalt concrete (AC)												
	60		10" Aggregate Base (AB), brown, dry												
		X	SANDY LEAN CLAY (CL), dark brown, moist, fine- to medium-grained sand, trace rootlets, trace brick debris [FILL]				41	17	24						
5	55		Becomes dark yellowish brown mottled with pale olive, medium dense, moist, fine- to medium-grained sand [NATIVE]			12									
10	50		LEAN CLAY (CL), olive brown mottled with pale olive, hard, moist, iron oxide staining			21								>4.5*	PP
15	45		SANDY LEAN CLAY (CL), pale olive, medium stiff, moist, high plasticity, fine-grained sand, iron oxide staining			7					21.4	111.2	365.5		UC
20	40		Becomes very soft to soft, fine- to coarse-grained sand, angular fine to coarse gravel, trace shell fragments			2	37	15	22						
25			POORLY GRADED SAND WITH GRAVEL (SP), dark olive brown, medium dense, wet, medium-grained sand, angular fine to coarse gravel												

ENGEO
Expect Excellence

LOG OF BORING 1-B03

LATITUDE: 37.396285   LONGITUDE: -122.052225

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/13/2020
HOLE DEPTH: 102.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 61 ft.

LOGGED / REVIEWED BY: C. Nicas / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
POORLY GRADED SAND WITH GRAVEL (SP), dark olive brown, medium dense, wet, medium-grained sand, angular fine to coarse gravel	41				6					
	22				12	13.2				
CLAYEY SAND WITH GRAVEL (SC), dark yellowish brown, medium dense, moist, fine- to medium-grained sand, subrounded fine to medium gravel	29				36					
SANDY LEAN CLAY WITH GRAVEL (CL), dark gray, stiff, moist, medium- to coarse-grained sand, fine gravel, trace rootlets, trace shell fragments	75 to 200 psi	37	18	19	77	25.2	100.3		2.0*	PP
	17					19.1	114.6	1036.3		UC

Depth in Feet: 30, 35, 40, 45, 50
Elevation in Feet: 35, 30, 25, 20, 15

LOG - GEOTECHNICAL_SU+QU W/ ELEV BORING LOGS_11-23-2020.GPJ ENGEO INC.GDT 12/22/20

LOG OF BORING 1-B03

ENGEO
Expect Excellence

LATITUDE: 37.396285 LONGITUDE: -122.052225

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/13/2020
HOLE DEPTH: 102.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 61 ft.

LOGGED / REVIEWED BY: C. Nicas / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Liquid Limit (Atterberg Limits)	Plastic Limit (Atterberg Limits)	Plasticity Index (Atterberg Limits)	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		LEAN CLAY (CL), greenish gray, stiff, moist, trace rootlets, trace shell fragments	19							1400*	1.75*	PP+TV
	10											
55												
	5							9				
60												
		Becomes pale olive, stiff to very stiff, fine- to medium-grained sand, iron oxide staining	22					22.6	104.8	1700*	2.5*	PP+TV
	0											
		POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM), olive brown, very dense, moist to wet, fine- to coarse-grained sand, fine gravel										
65												
			58				10					
	-5											
		SANDY SILT WITH GRAVEL (ML), pale olive, very stiff, wet, fine- to coarse-grained sand, fine to coarse gravel										
70												
			55									
	-10											
		SILTY SAND WITH GRAVEL (SM), olive brown, very dense, moist to wet, fine- to coarse-grained sand, fine gravel	69									
75												

LOG OF BORING 1-B03

ENGEO *Expect Excellence*

LATITUDE: 37.396285 LONGITUDE: -122.052225

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/13/2020
HOLE DEPTH: 102.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 61 ft.

LOGGED / REVIEWED BY: C. Nicas / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-15		SILTY SAND WITH GRAVEL (SM), olive brown, very dense, moist to wet, fine- to coarse-grained sand, fine gravel												
			SANDY LEAN CLAY WITH GRAVEL (CL), dark gray, stiff to very stiff, moist, medium- to coarse-grained sand, rounded fine to coarse gravel, trace rootlets												
80	-20					26	36	15	21		19.2	111.5	1349.4		UC
85	-25														
90	-30					24									
			SANDY SILT (ML), olive brown, very stiff, moist, fine- to medium-grained sand												
			LEAN CLAY (CL), dark gray, stiff to very stiff, moist, trace rootlets												
95	-35														
100															

LOG OF BORING 1-B03

Expect Excellence

LATITUDE: 37.396285 LONGITUDE: -122.052225

Geotechnical Exploration
East Whisman Phase 1
Mountain View, CA
17954.000.001

DATE DRILLED: 11/13/2020
HOLE DEPTH: 102.5 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NAVD88): 61 ft.

LOGGED / REVIEWED BY: C. Nicas / PE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-40		LEAN CLAY (CL), dark gray, stiff to very stiff, moist, trace rootlets			32					23.4	103.8	1060.3		UC
			Boring terminated at 102.5 feet below ground surface. Groundwater was not observed during drilling due to drilling method. Casing installed to approximately 50 feet below ground surface.												

APPENDIX B

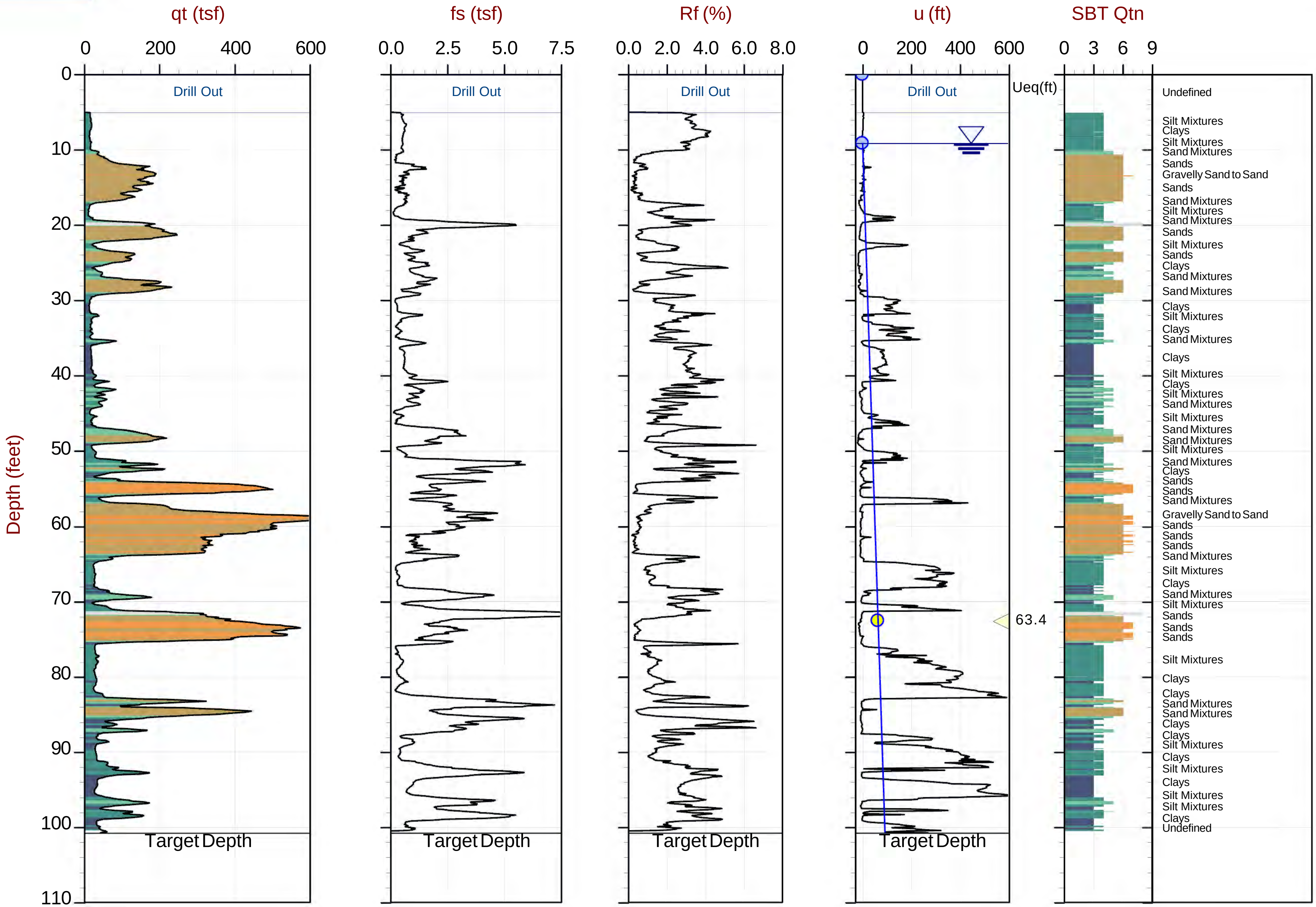
CONE PENETRATION TEST LOGS

CONETEC

ENGEO

Job No: 20-56-21609
Date: 2020-11-13 14:42
Site: East Whisman Phase 1

Sounding: 1-CPT01
Cone: 537:T1500F15U500



Max Depth: 30.725 m / 100.80 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21609_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139409m E: 583839m

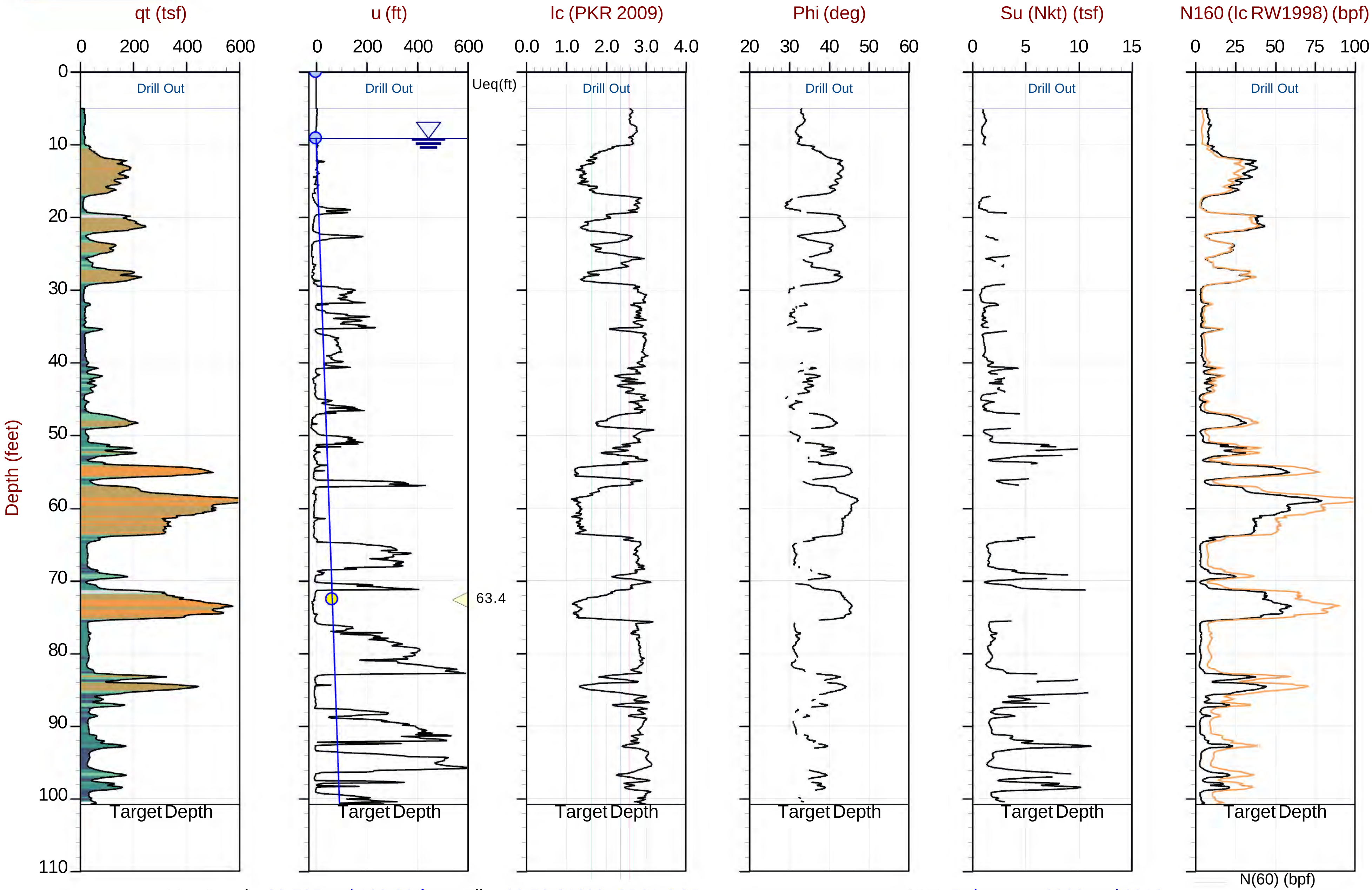
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

ENGEO

Job No: 20-56-21609
Date: 2020-11-13 14:42
Site: East Whisman Phase 1

Sounding: 1-CPT01
Cone: 537:T1500F15U500



Max Depth: 30.725 m / 100.80 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21609_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139409m E: 583839m

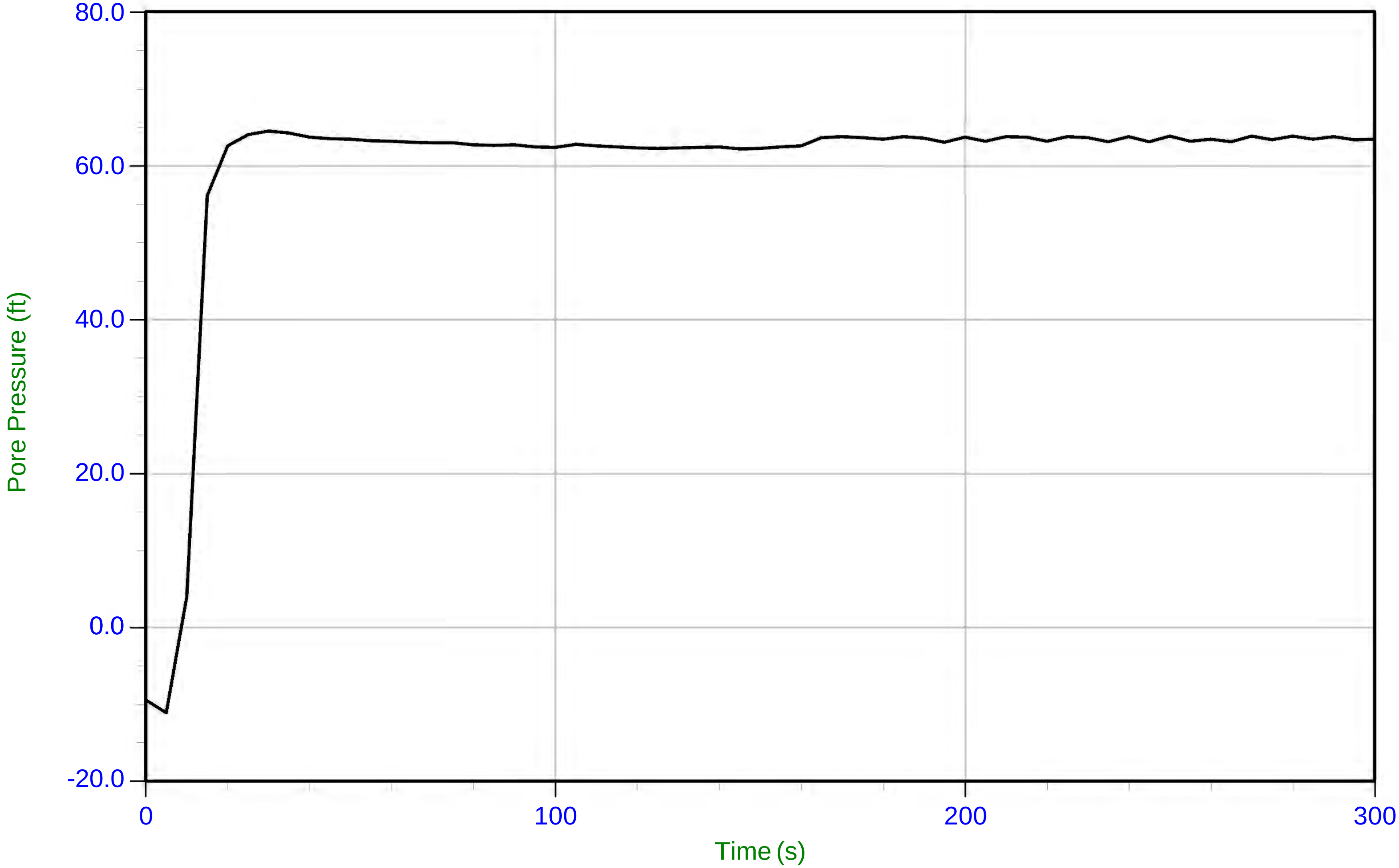
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

ENGEO

Job No: 20-56-21609
Date: 11/13/2020 14:42
Site: East Whisman Phase 1

Sounding: 1-CPT01
Cone: 537:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 20-56-21609_CP01.PPF
Depth: 22.125 m / 72.588 ft
Duration: 300.0 s

u Min: -11.1 ft
u Max: 64.5 ft
u Final: 63.4 ft

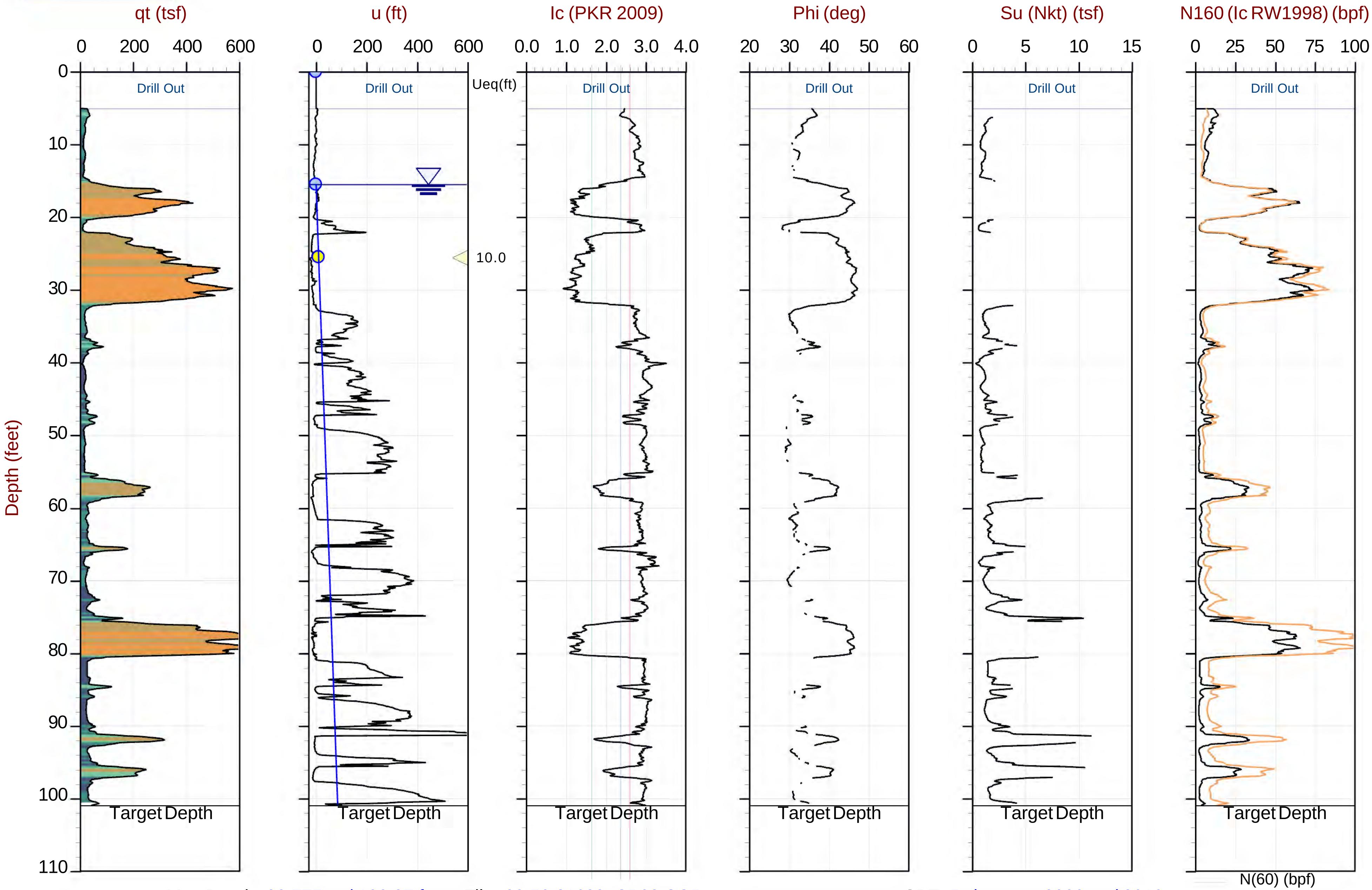
WT: 2.789 m / 9.149 ft
Ueq: 63.4 ft

CONETEC

ENGEO

Job No: 20-56-21609
Date: 2020-11-13 13:10
Site: East Whisman Phase 1

Sounding: 1-CPT02
Cone: 537:T1500F15U500



Max Depth: 30.775 m / 100.97 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21609_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139308m E: 583828m

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

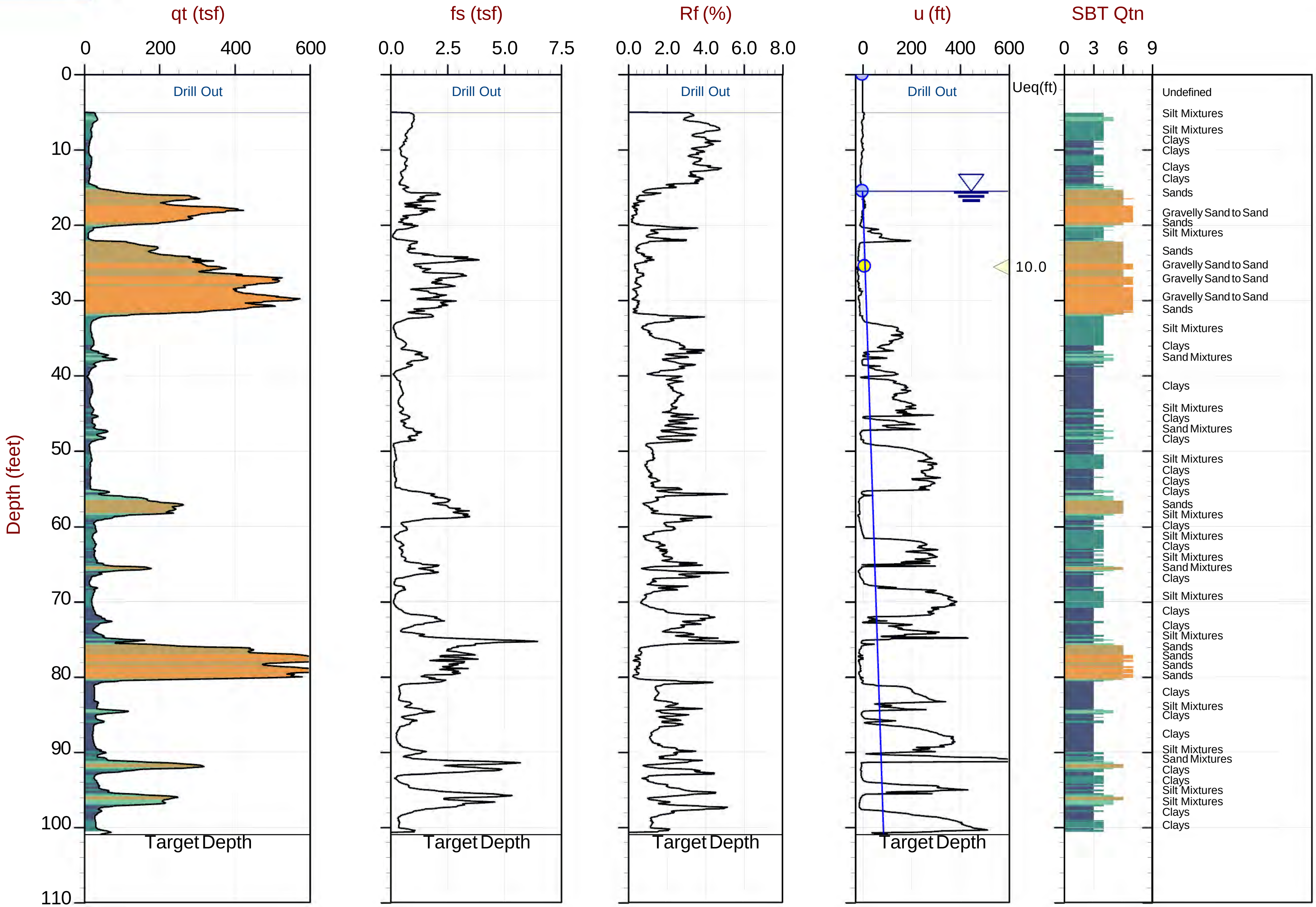
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 20-56-21609
Date: 2020-11-13 13:10
Site: East Whisman Phase 1

Sounding: 1-CPT02
Cone: 537:T1500F15U500



Max Depth: 30.775 m / 100.97 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21609_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139308m E: 583828m

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

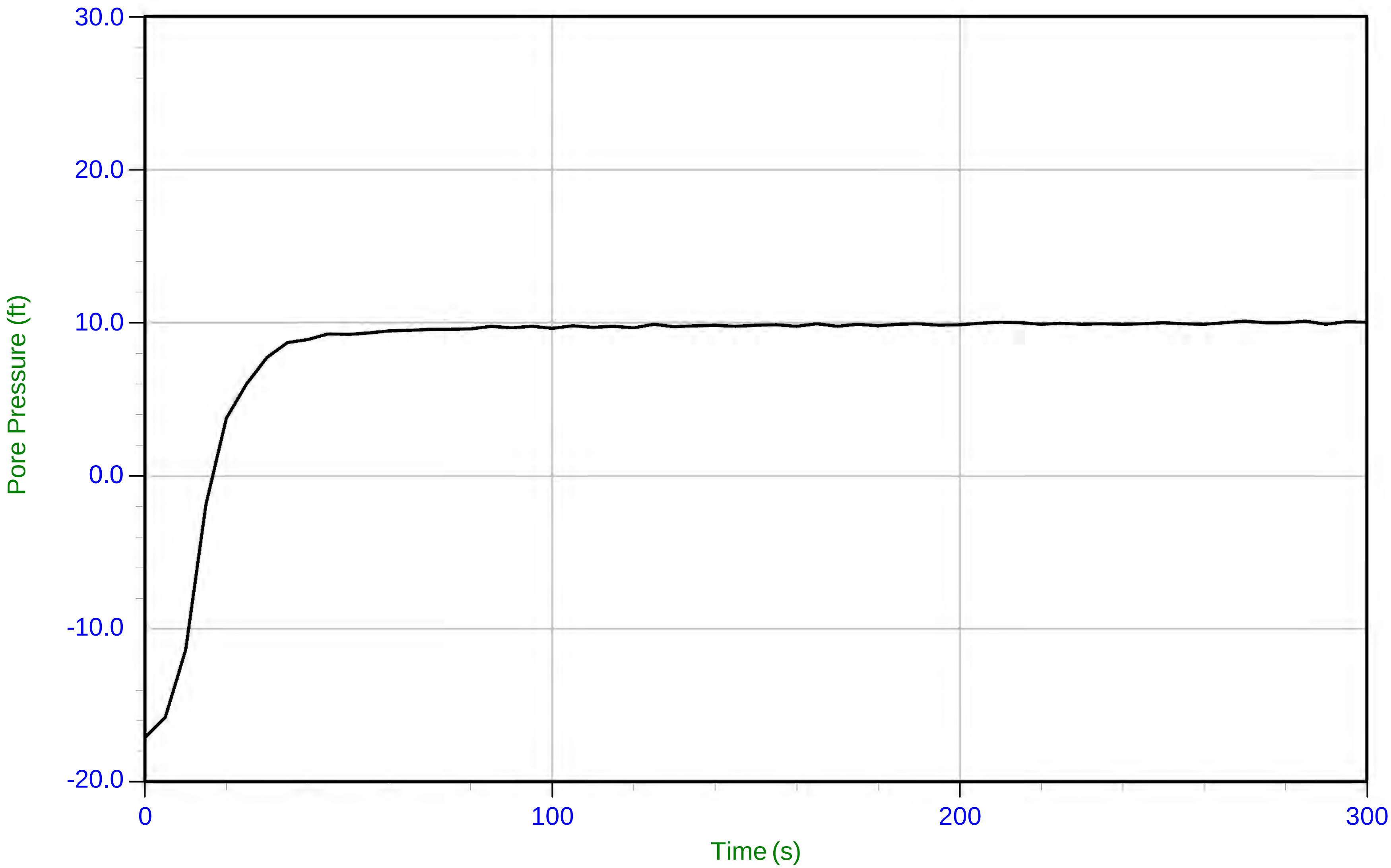
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 20-56-21609
Date: 11/13/2020 13:10
Site: East Whisman Phase 1

Sounding: 1-CPT02
Cone: 537:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 20-56-21609_CP02.PPF
Depth: 7.775 m / 25.508 ft
Duration: 300.0 s

u Min: -17.1 ft
u Max: 10.1 ft
u Final: 10.0 ft

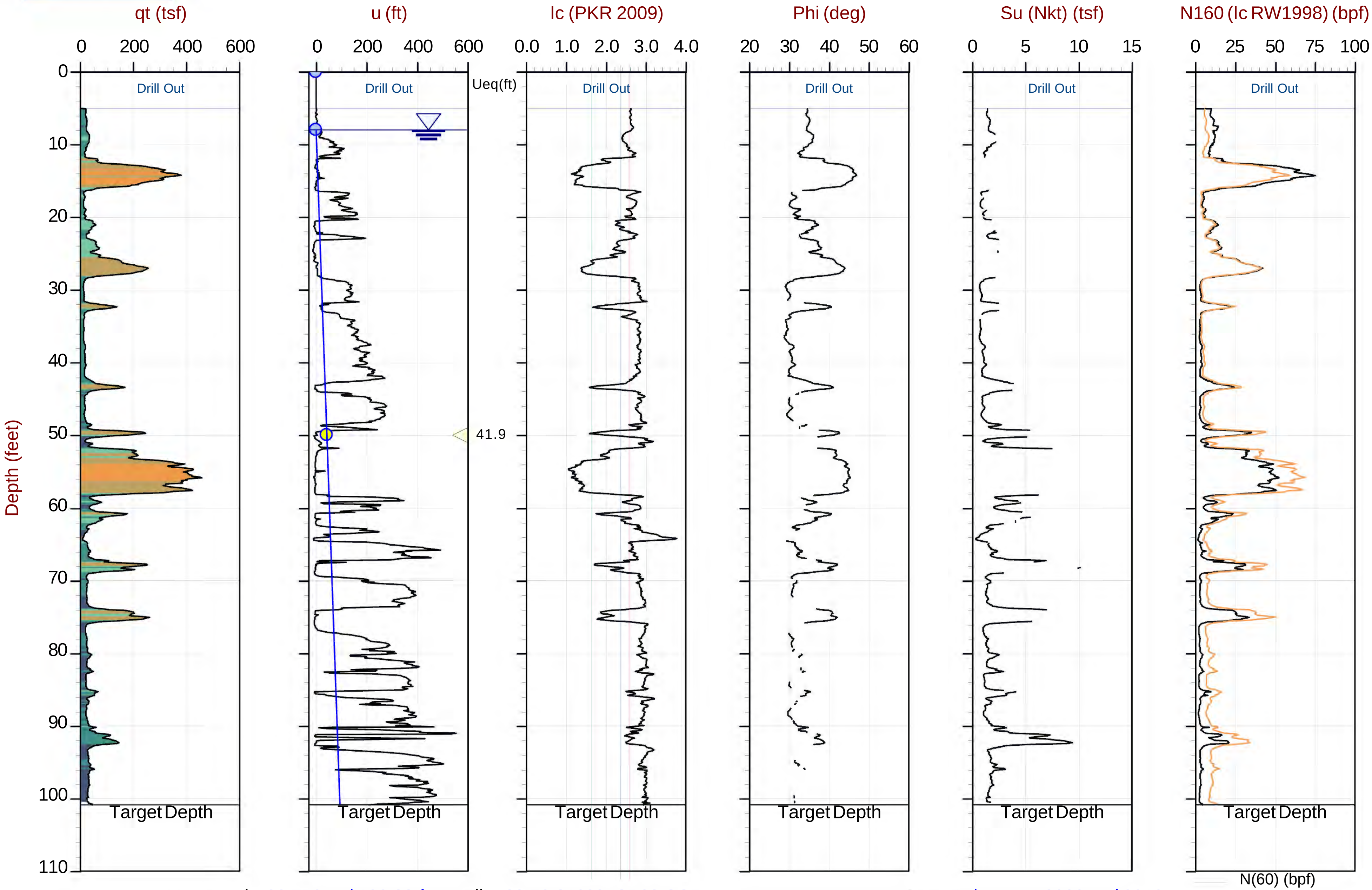
WT: 4.721 m / 15.487 ft
Ueq: 10.0 ft

CONETEC

ENGEO

Job No: 20-56-21609
Date: 2020-11-13 11:25
Site: East Whisman Phase 1

Sounding: 1-CPT03
Cone: 537:T1500F15U500



Max Depth: 30.750 m / 100.88 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21609_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139389m E: 583927m

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

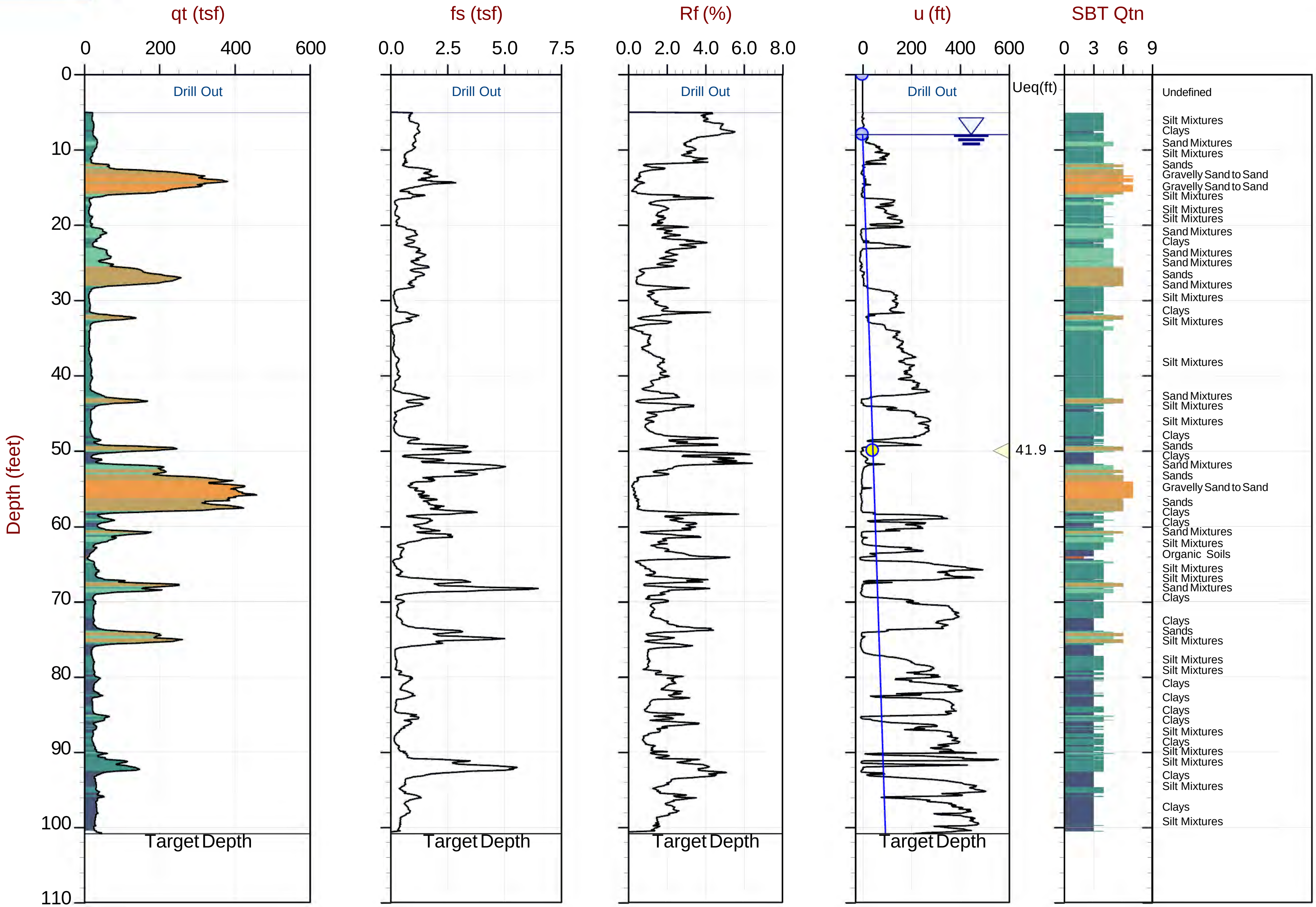
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 20-56-21609
Date: 2020-11-13 11:25
Site: East Whisman Phase 1

Sounding: 1-CPT03
Cone: 537:T1500F15U500



Max Depth: 30.750 m / 100.88 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21609_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139389m E: 583927m

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

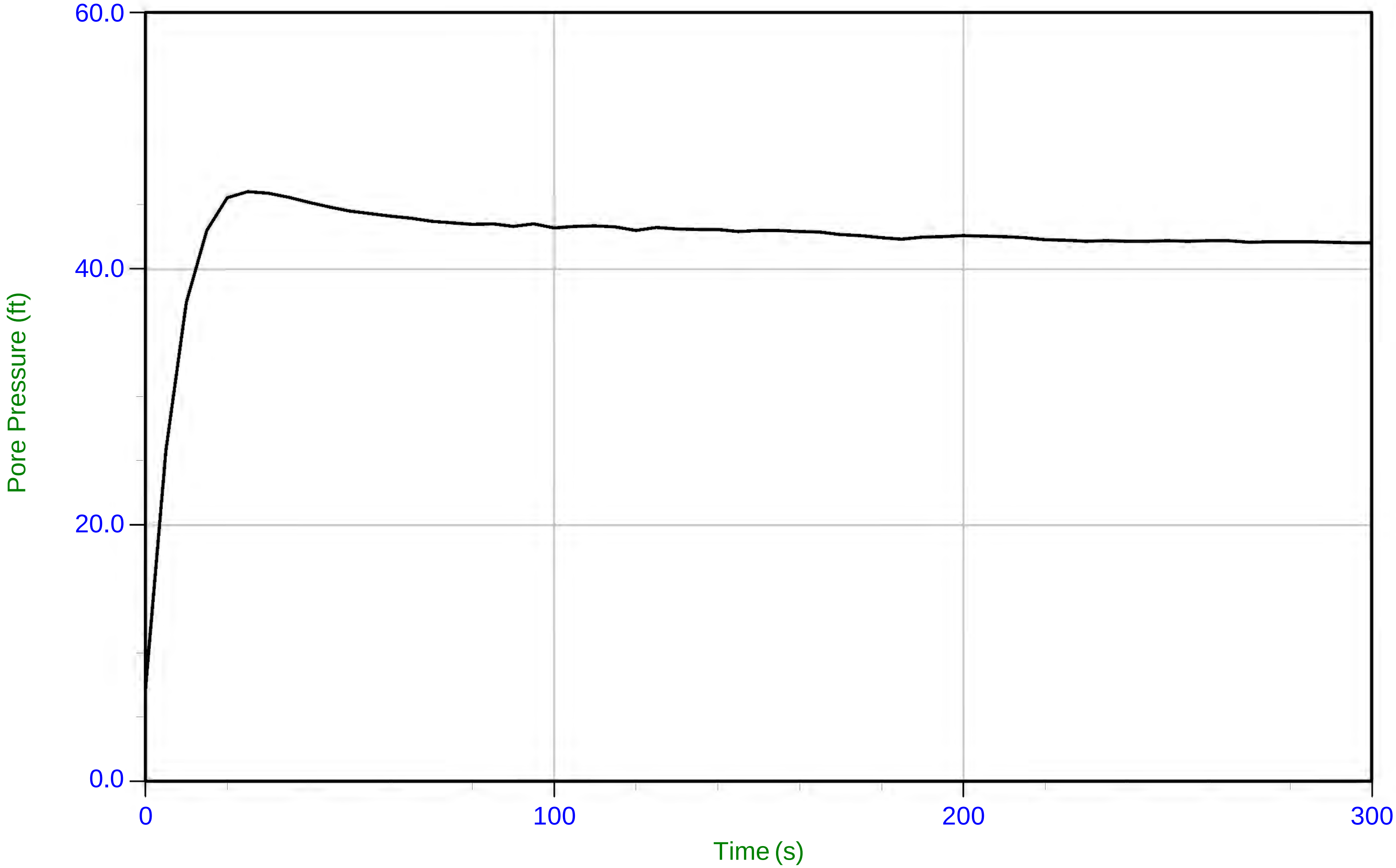
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 20-56-21609
Date: 11/13/2020 11:25
Site: East Whisman Phase 1

Sounding: 1-CPT03
Cone: 537:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 20-56-21609_CP03.PPF
Depth: 15.225 m / 49.950 ft
Duration: 300.0 s

u Min: 7.2 ft
u Max: 46.0 ft
u Final: 42.0 ft

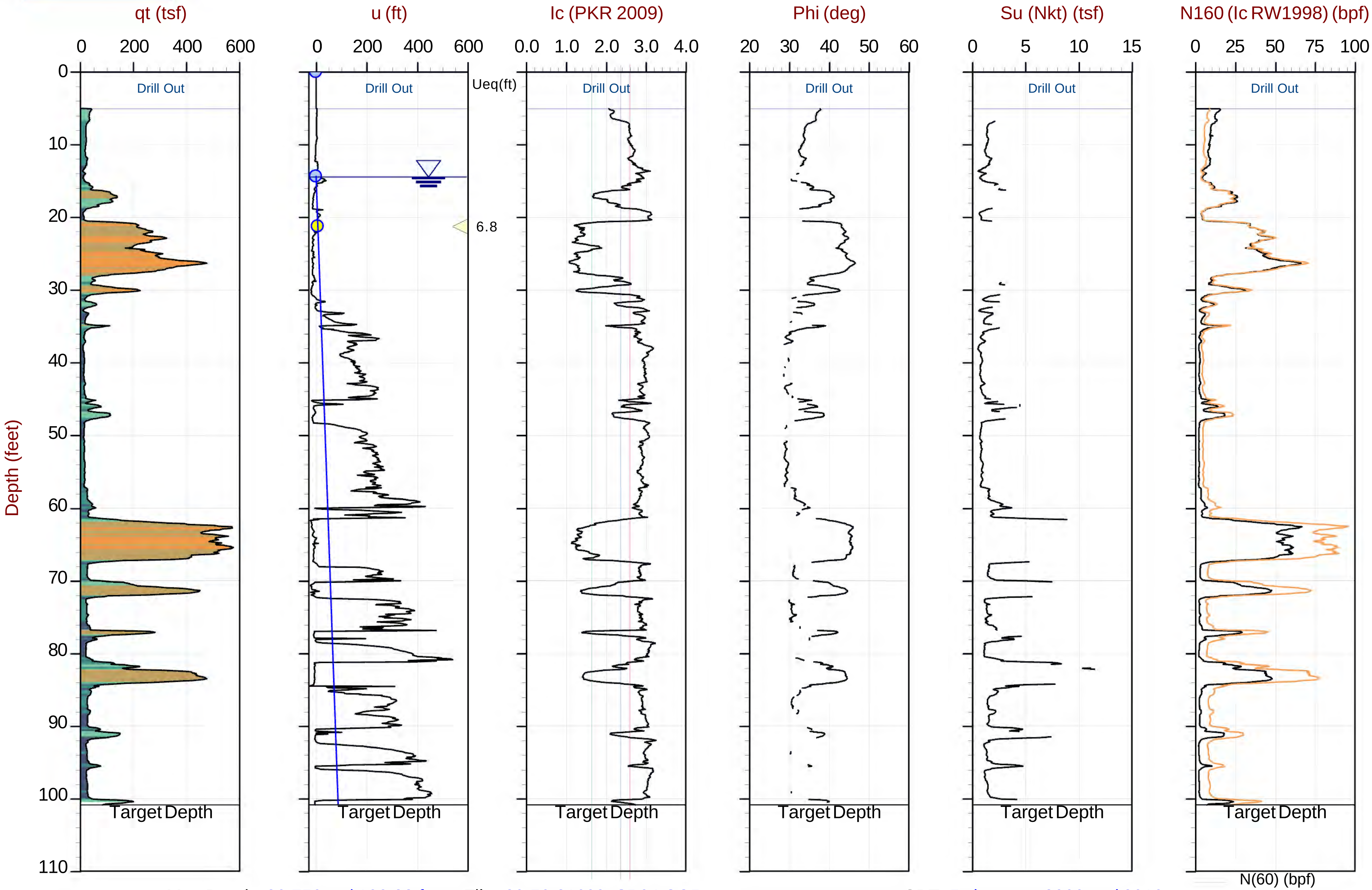
WT: 2.431 m / 7.976 ft
Ueq: 42.0 ft

CONETEC

ENGEO

Job No: 20-56-21609
Date: 2020-11-13 08:49
Site: East Whisman Phase 1

Sounding: 1-SCPT04
Cone: 537:T1500F15U500



Max Depth: 30.750 m / 100.88 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21609_SP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139267m E: 583873m

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

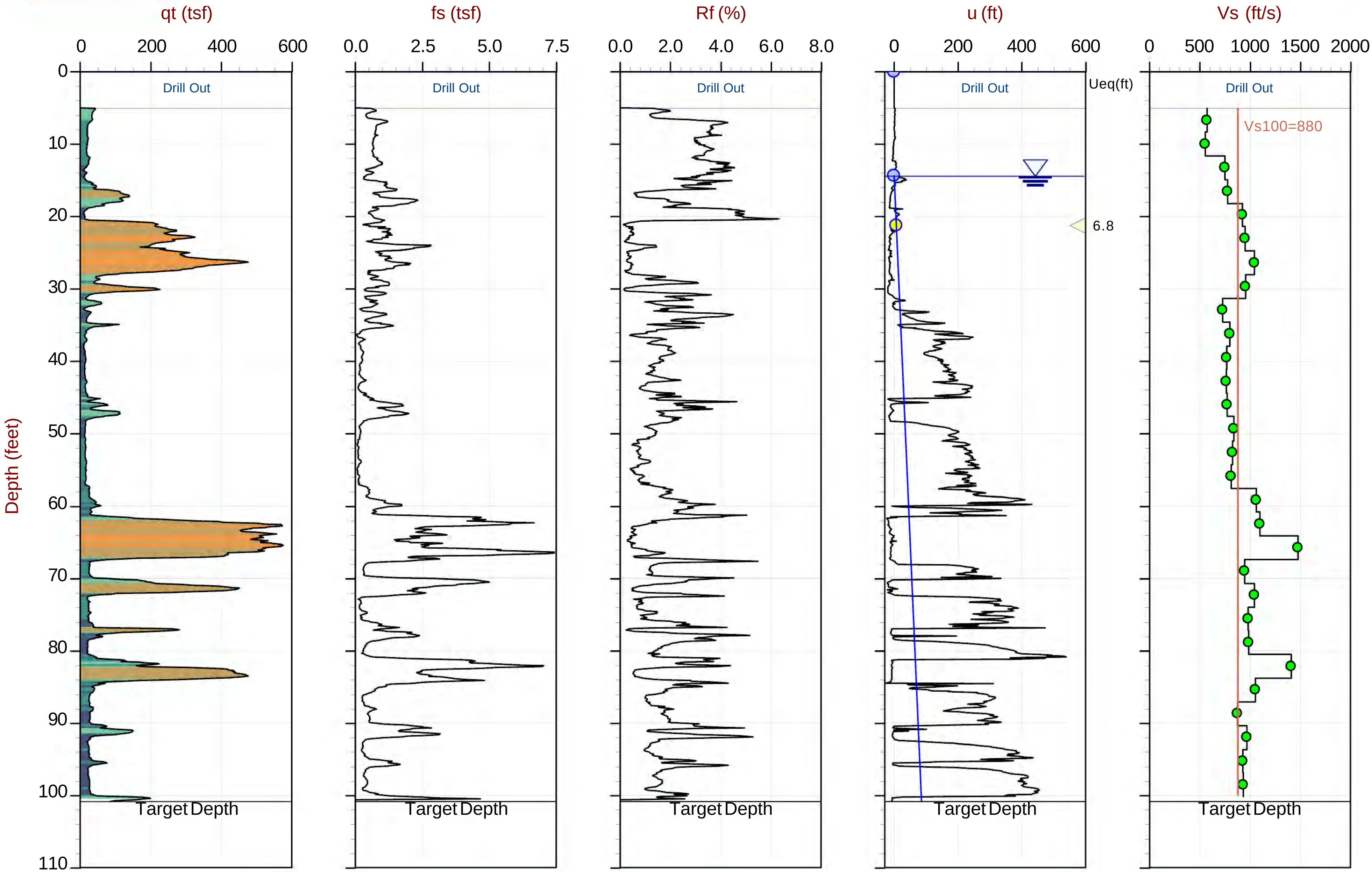
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 20-56-21609
Date: 2020-11-13 08:49
Site: East Whisman Phase 1

Sounding: 1-SCPT04
Cone: 537:T1500F15U500



Max Depth: 30.750 m / 100.88 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21609_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139267m E: 583873m

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

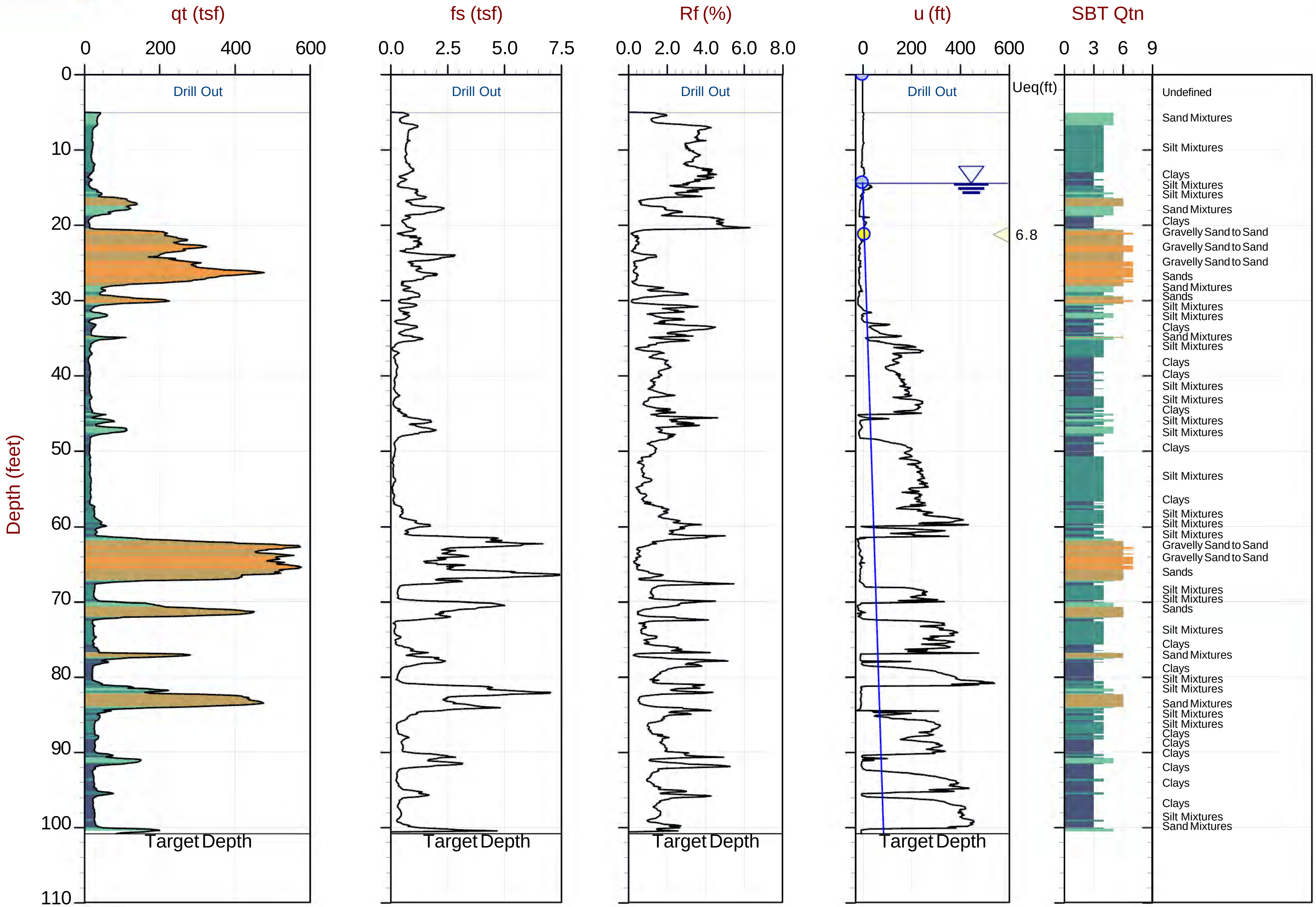
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 20-56-21609
Date: 2020-11-13 08:49
Site: East Whisman Phase 1

Sounding: 1-SCPT04
Cone: 537:T1500F15U500



Max Depth: 30.750 m / 100.88 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21609_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139267m E: 583873m

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.

Job No: 20-56-21609
Client: ENGEO
Project: East Whisman Phase 1
Sounding ID: 1-SCPT04
Date: 11:13:20 08:49

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

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

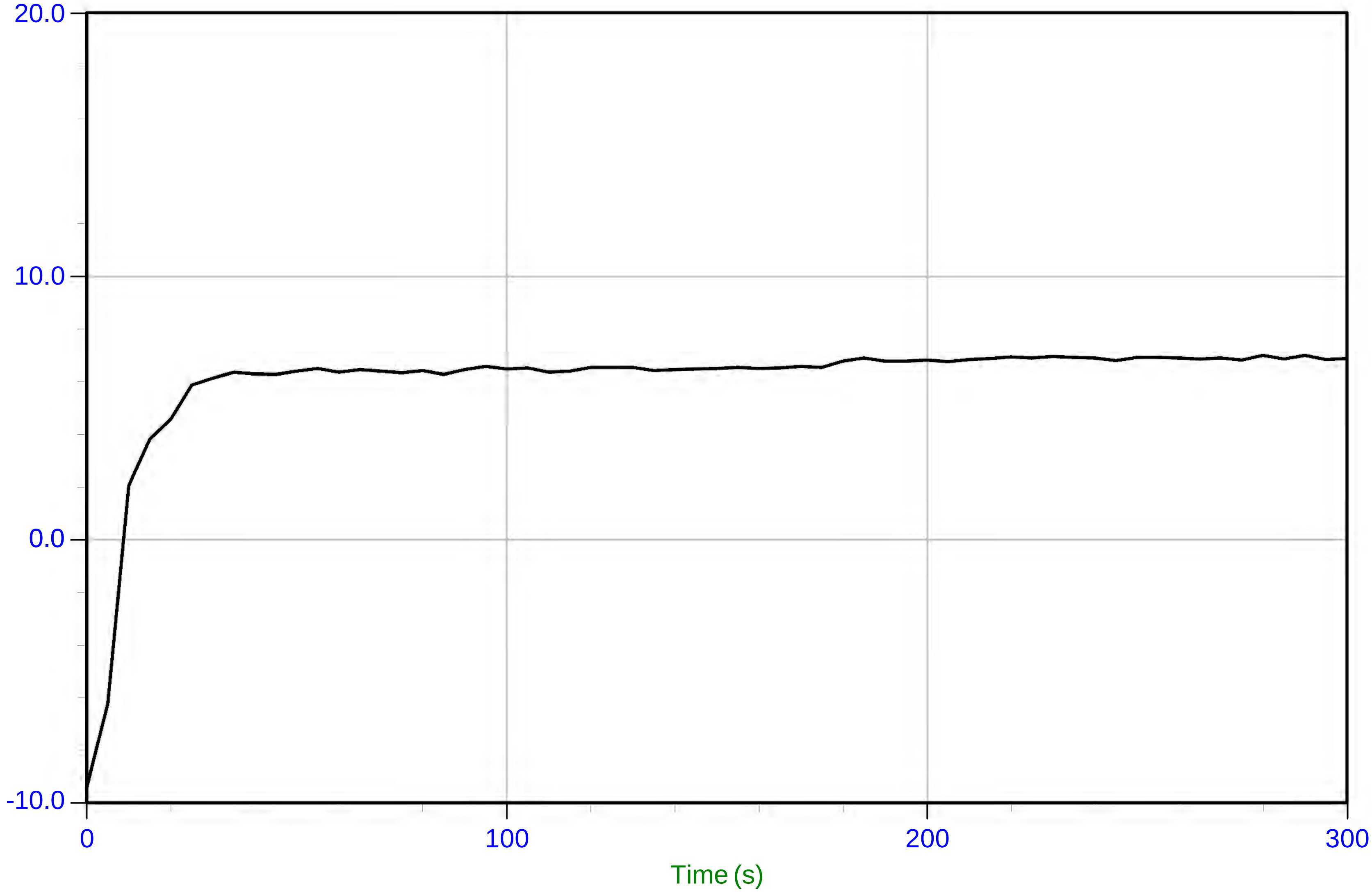
Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
5.74	5.09	5.50			
9.02	8.37	8.63	3.12	5.44	574
12.30	11.65	11.84	3.21	5.78	556
15.58	14.93	15.08	3.24	4.31	752
18.87	18.21	18.33	3.26	4.18	778
22.05	21.39	21.49	3.16	3.41	928
25.43	24.77	24.86	3.37	3.54	952
28.71	28.05	28.13	3.27	3.13	1046
31.99	31.33	31.40	3.27	3.41	959
35.27	34.61	34.68	3.28	4.50	728
38.55	37.89	37.95	3.28	4.09	801
41.83	41.18	41.23	3.28	4.25	771
45.11	44.46	44.51	3.28	4.28	765
48.29	47.64	47.68	3.18	4.10	775
51.67	51.02	51.06	3.38	4.01	843
54.95	54.30	54.34	3.28	3.96	829
58.24	57.58	57.62	3.28	4.03	814
61.52	60.86	60.90	3.28	3.09	1063
64.80	64.14	64.17	3.28	2.98	1100
68.08	67.42	67.45	3.28	2.22	1481
71.36	70.70	70.73	3.28	3.45	950
74.64	73.98	74.01	3.28	3.13	1049
77.92	77.26	77.29	3.28	3.34	983
81.20	80.55	80.57	3.28	3.31	991
84.48	83.83	83.85	3.28	2.32	1412
87.76	87.11	87.13	3.28	3.10	1057
91.04	90.39	90.41	3.28	3.73	879
94.32	93.67	93.69	3.28	3.37	973
97.61	96.95	96.97	3.28	3.52	932
100.89	100.23	100.25	3.28	3.50	937

CONETEC

ENGEO

Job No: 20-56-21609
Date: 11/13/2020 08:49
Site: East Whisman Phase 1

Sounding: 1-SCPT04
Cone: 537:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 20-56-21609_SP04.PPF
Depth: 6.475 m / 21.243 ft
Duration: 300.0 s

u Min: -9.4 ft
u Max: 7.0 ft
u Final: 6.9 ft

WT: 4.386 m / 14.390 ft
Ueq: 6.9 ft

APPENDIX C

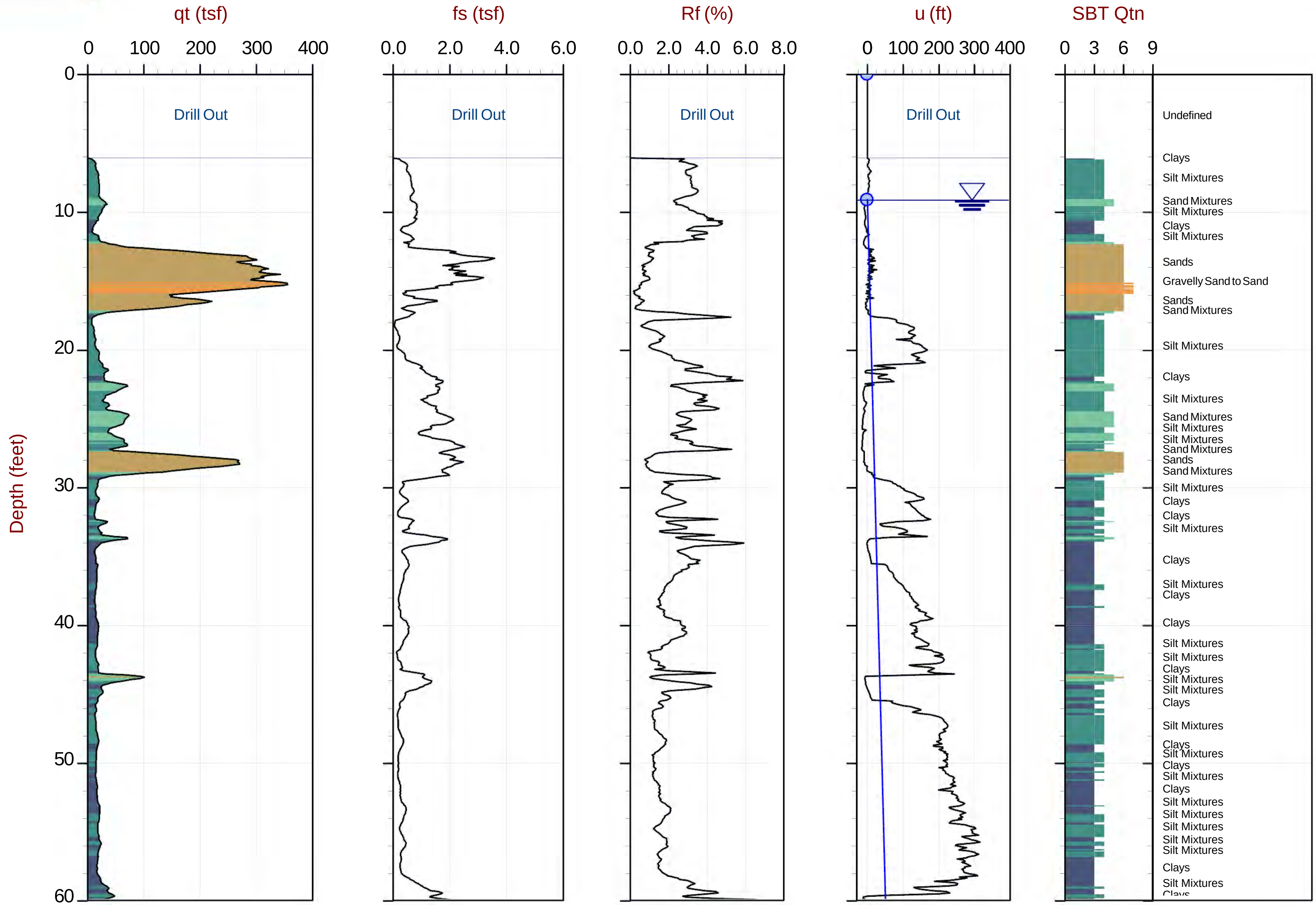
EXPLORATION AND CPT LOGS
AND LABORATORY TEST DATA
BY NINYO & MOORE

CONETEC

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-26 10:01
Site: East Whisman

Sounding: CPT-11
Cone: 446:T1500F15U500



Max Depth: 24.400 m / 80.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP11.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139360m E: 583892m

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

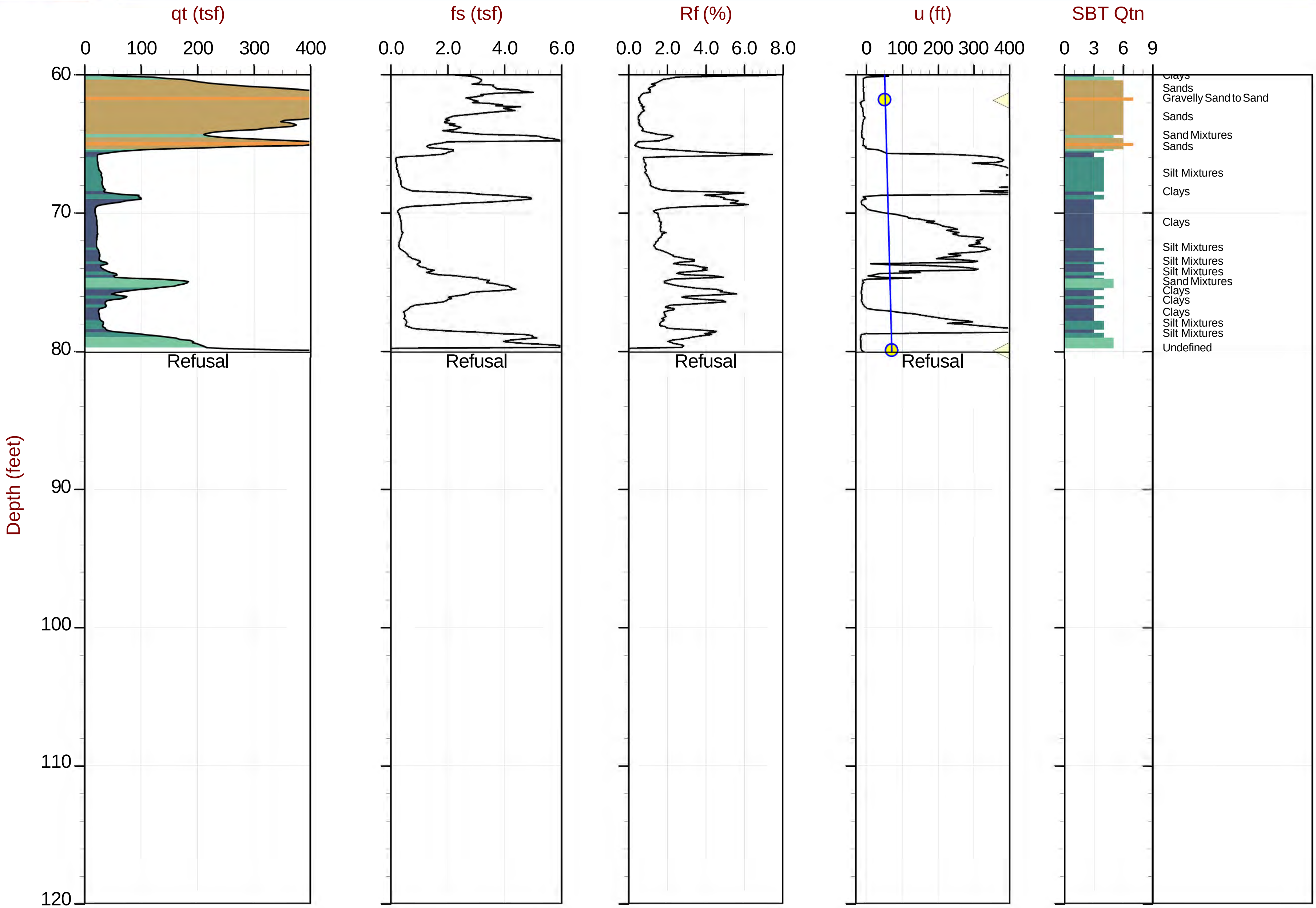
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-26 10:01
Site: East Whisman

Sounding: CPT-11
Cone: 446:T1500F15U500



Max Depth: 24.400 m / 80.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP11.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139360m E: 583892m

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

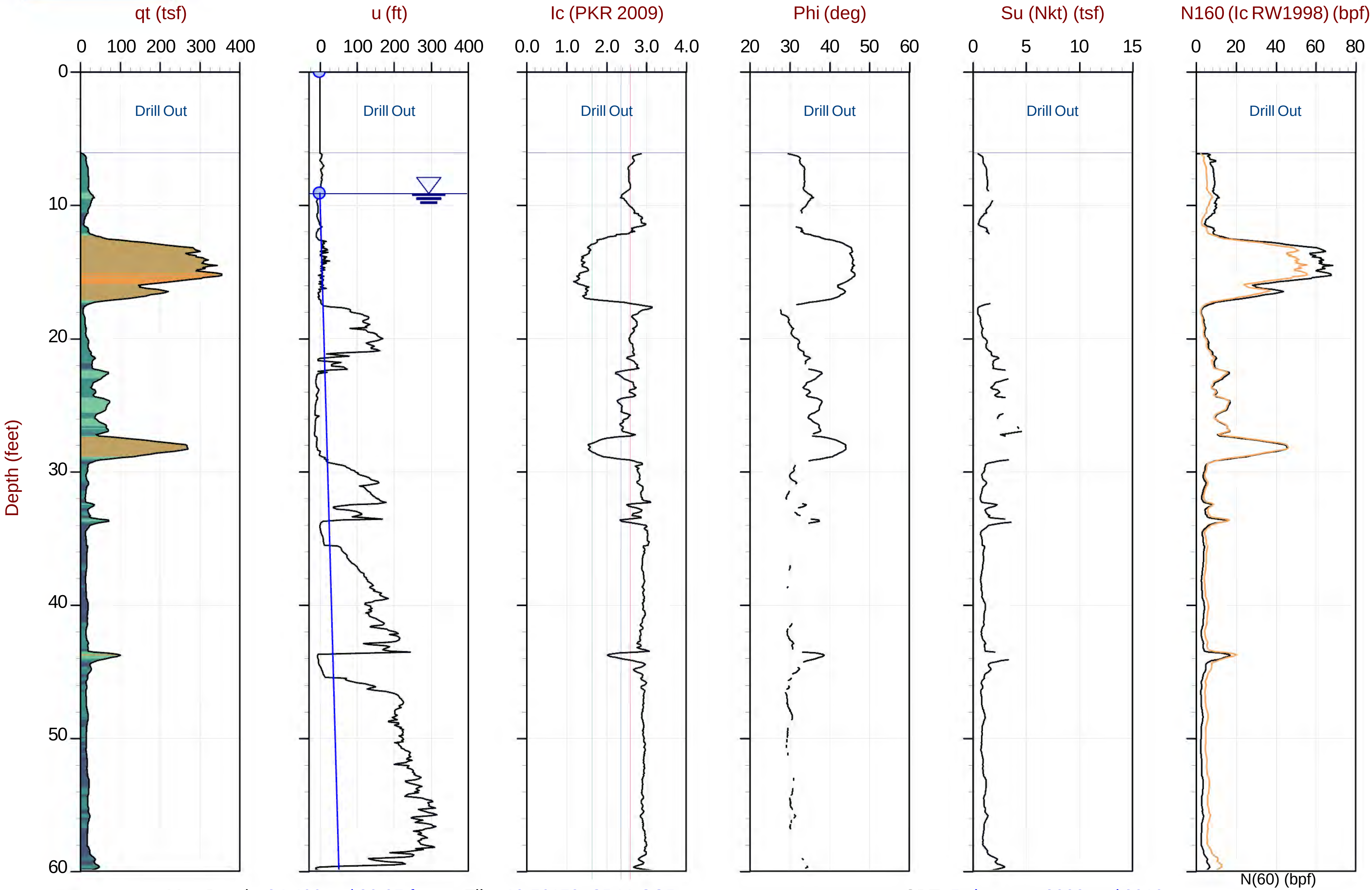
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-26 10:01
Site: East Whisman

Sounding: CPT-11
Cone: 446:T1500F15U500



Max Depth: 24.400 m / 80.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP11.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139360m E: 583892m

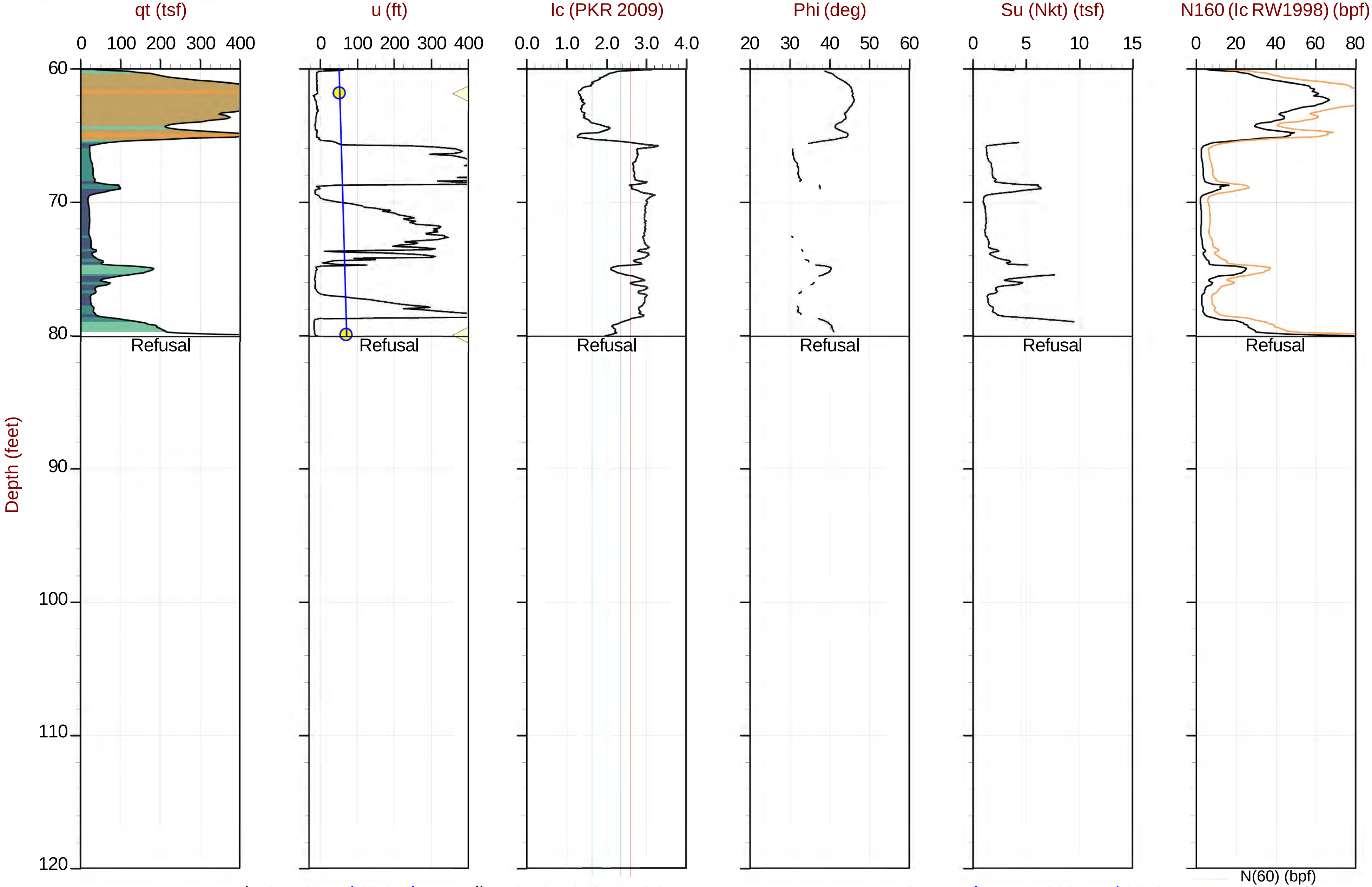
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-26 10:01
Site: East Whisman

Sounding: CPT-11
Cone: 446:T1500F15U500



Max Depth: 24.400 m / 80.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP11.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139360m E: 583892m

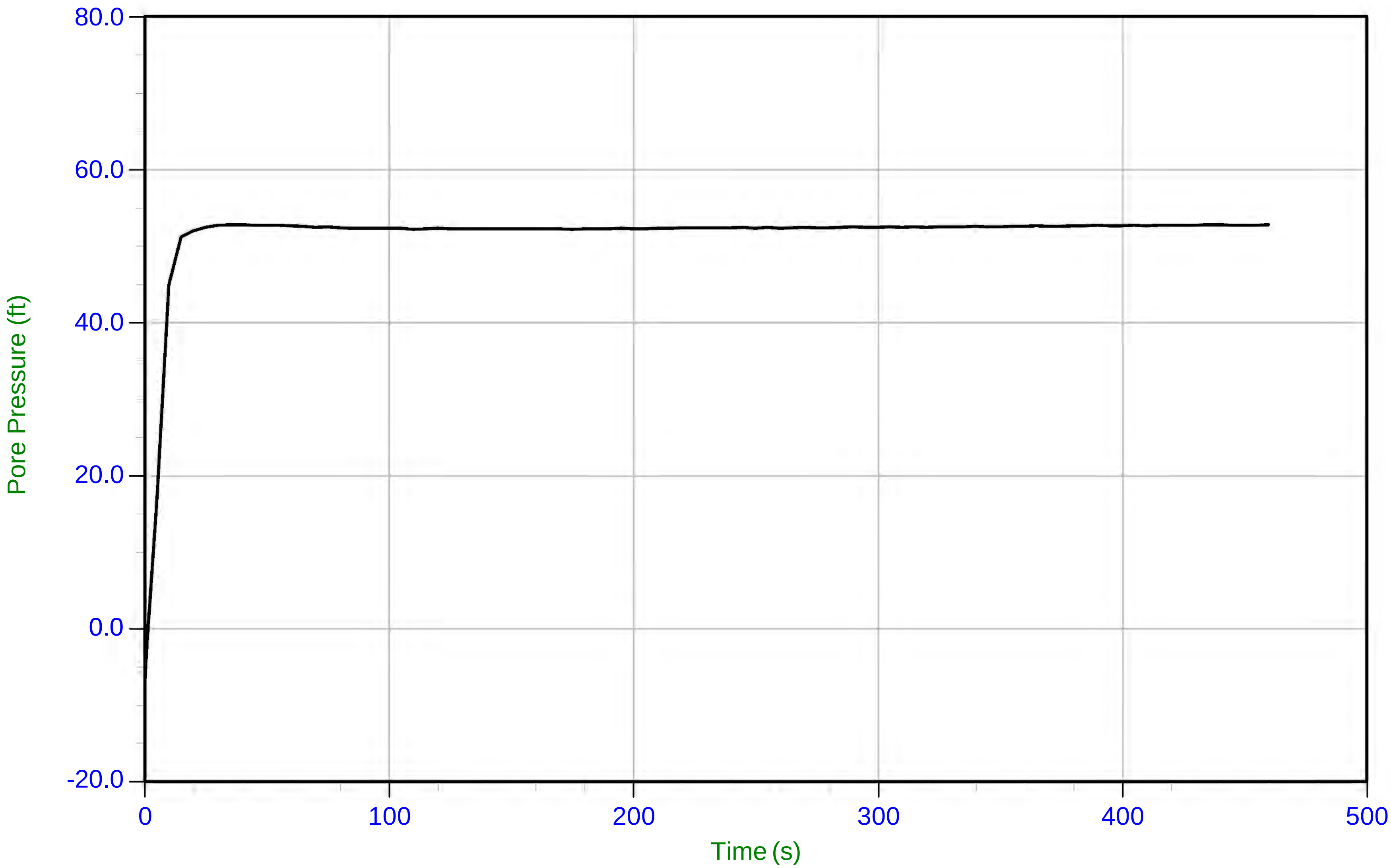
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

Ninyo & Moore

Job No: 19-56150
Date: 09/26/2019 10:01
Site: East Whisman

Sounding: CPT-11
Cone: 446:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56150_CP11.PPD
Depth: 18.850 m / 61.843 ft
Duration: 460.0 s

u Min: -6.6 ft
u Max: 52.8 ft
u Final: 52.8 ft

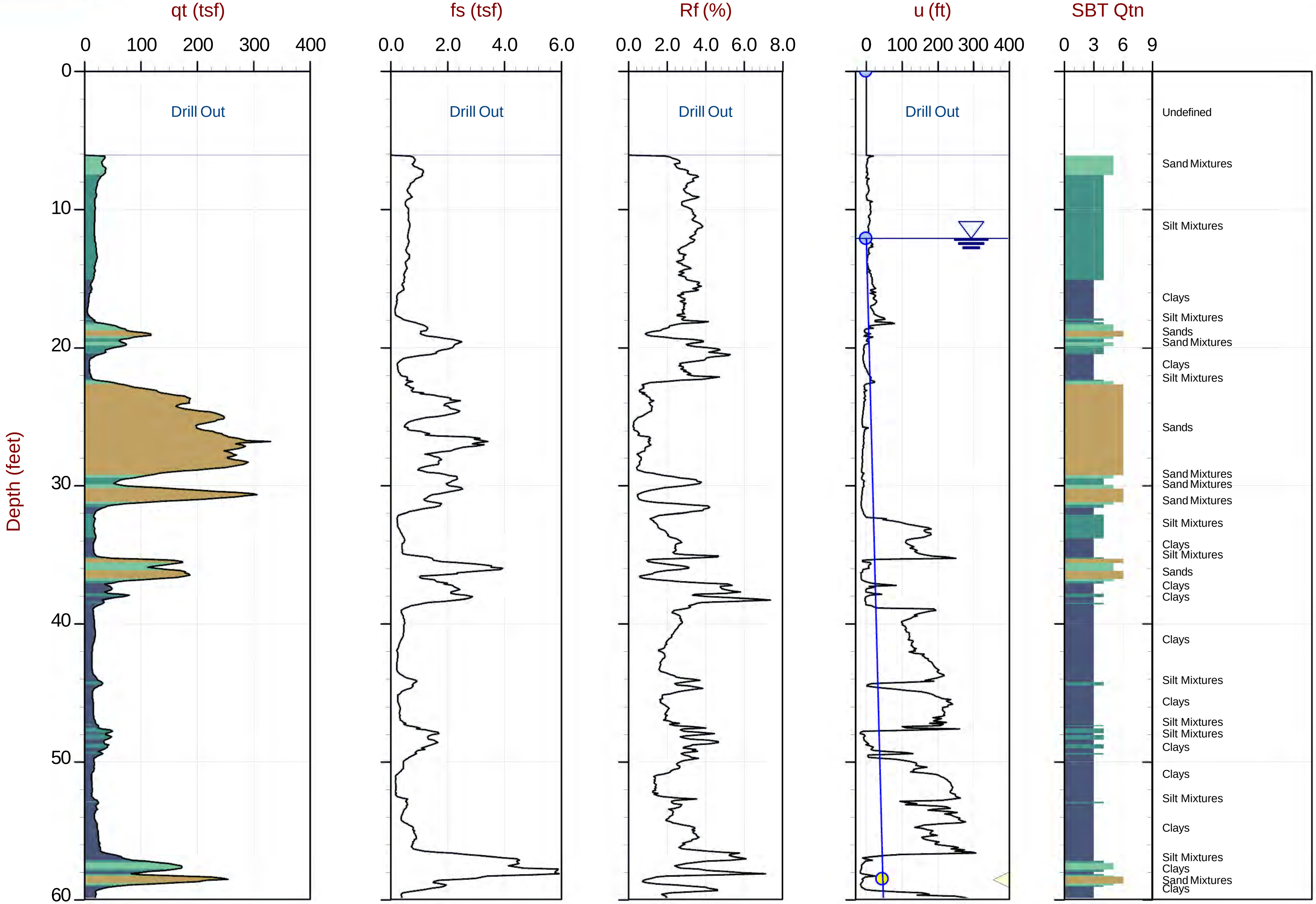
WT: 2.788 m / 9.147 ft
Ueq: 52.7 ft

CONETEC

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-26 11:23
Site: East Whisman

Sounding: CPT-12
Cone: 446:T1500F15U500



Max Depth: 30.850 m / 101.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP12.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139230m E: 583883m

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

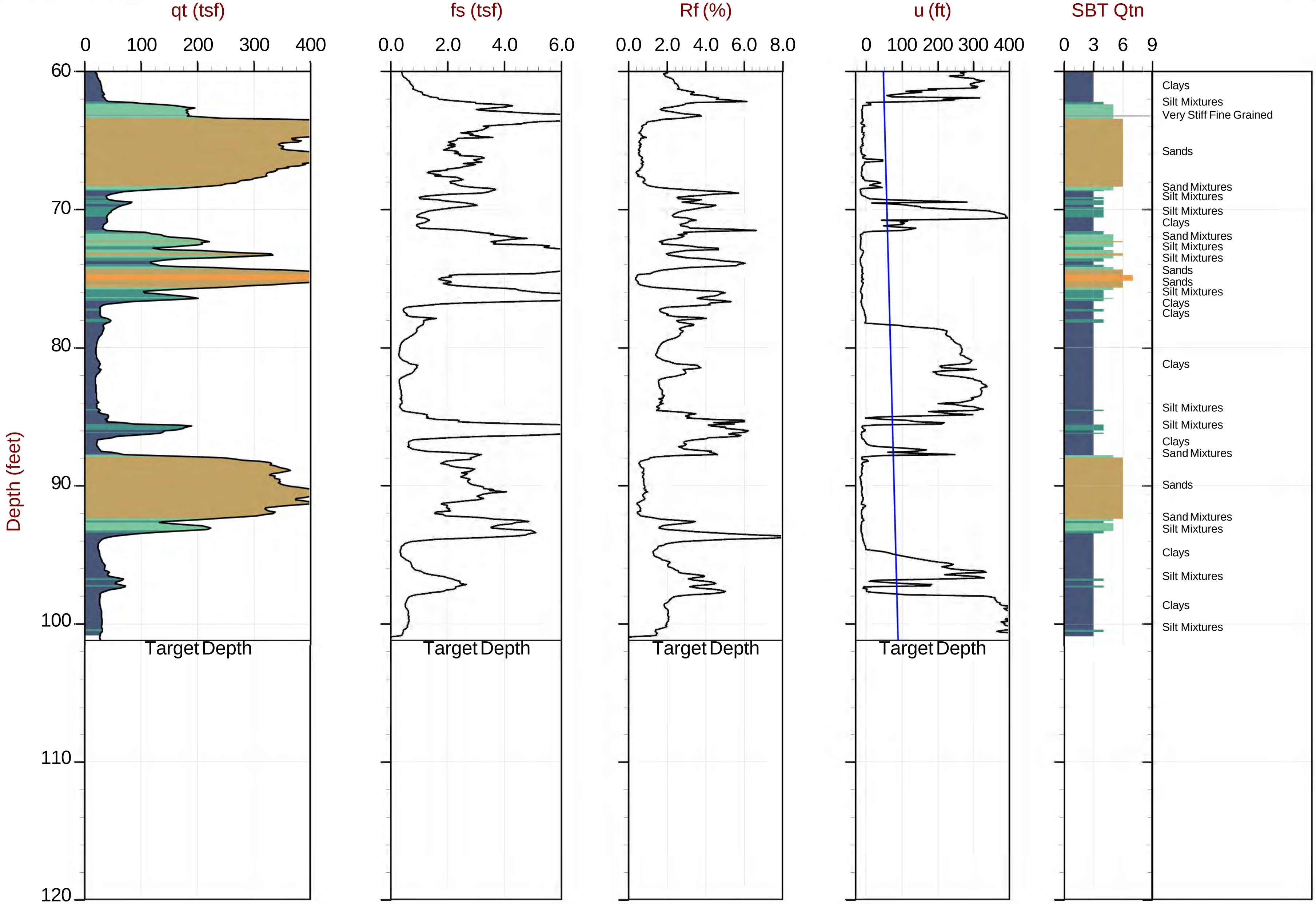
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-26 11:23
Site: East Whisman

Sounding: CPT-12
Cone: 446:T1500F15U500



Max Depth: 30.850 m / 101.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP12.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139230m E: 583883m

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

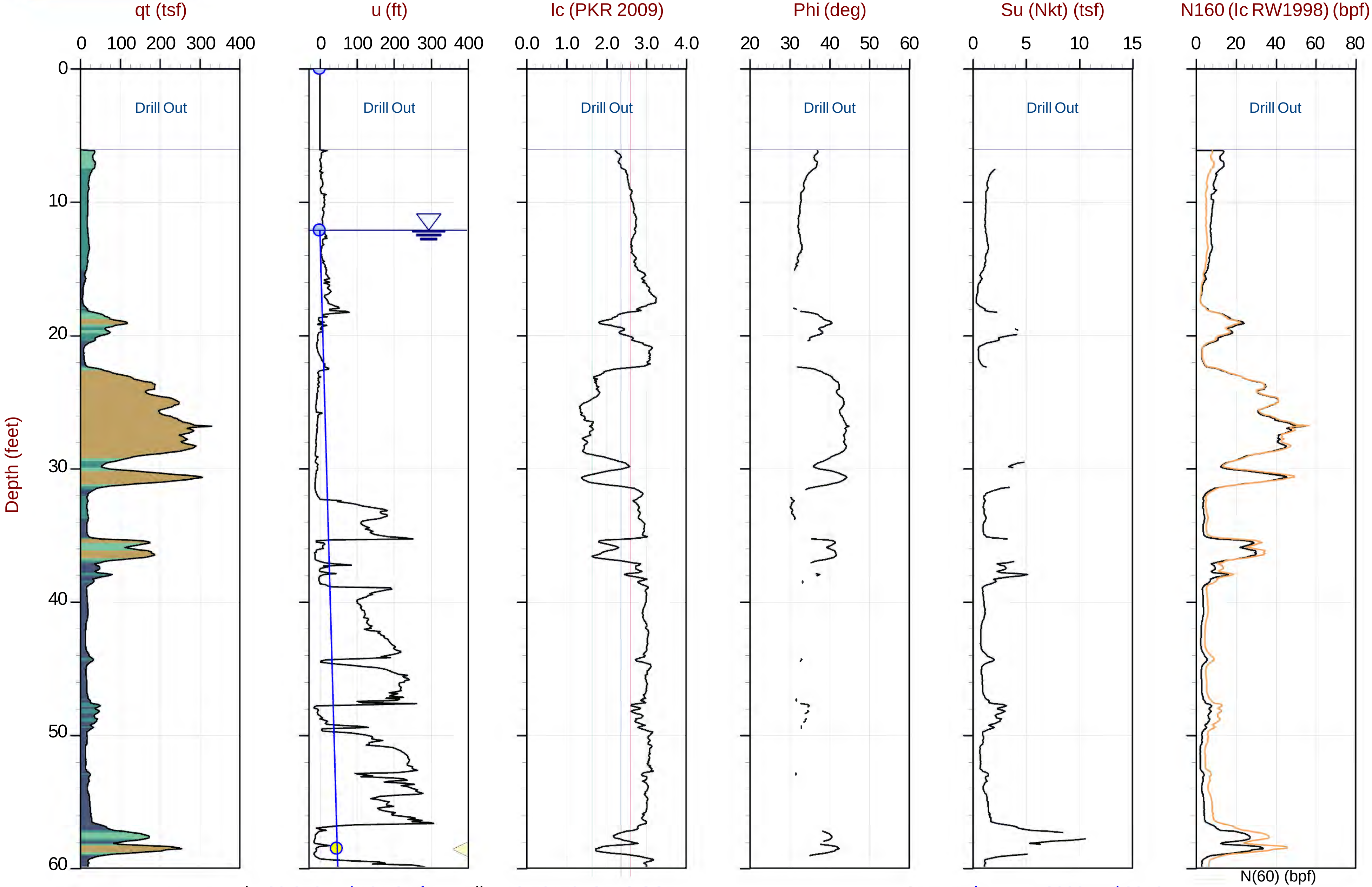
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-26 11:23
Site: East Whisman

Sounding: CPT-12
Cone: 446:T1500F15U500



Max Depth: 30.850 m / 101.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP12.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139230m E: 583883m

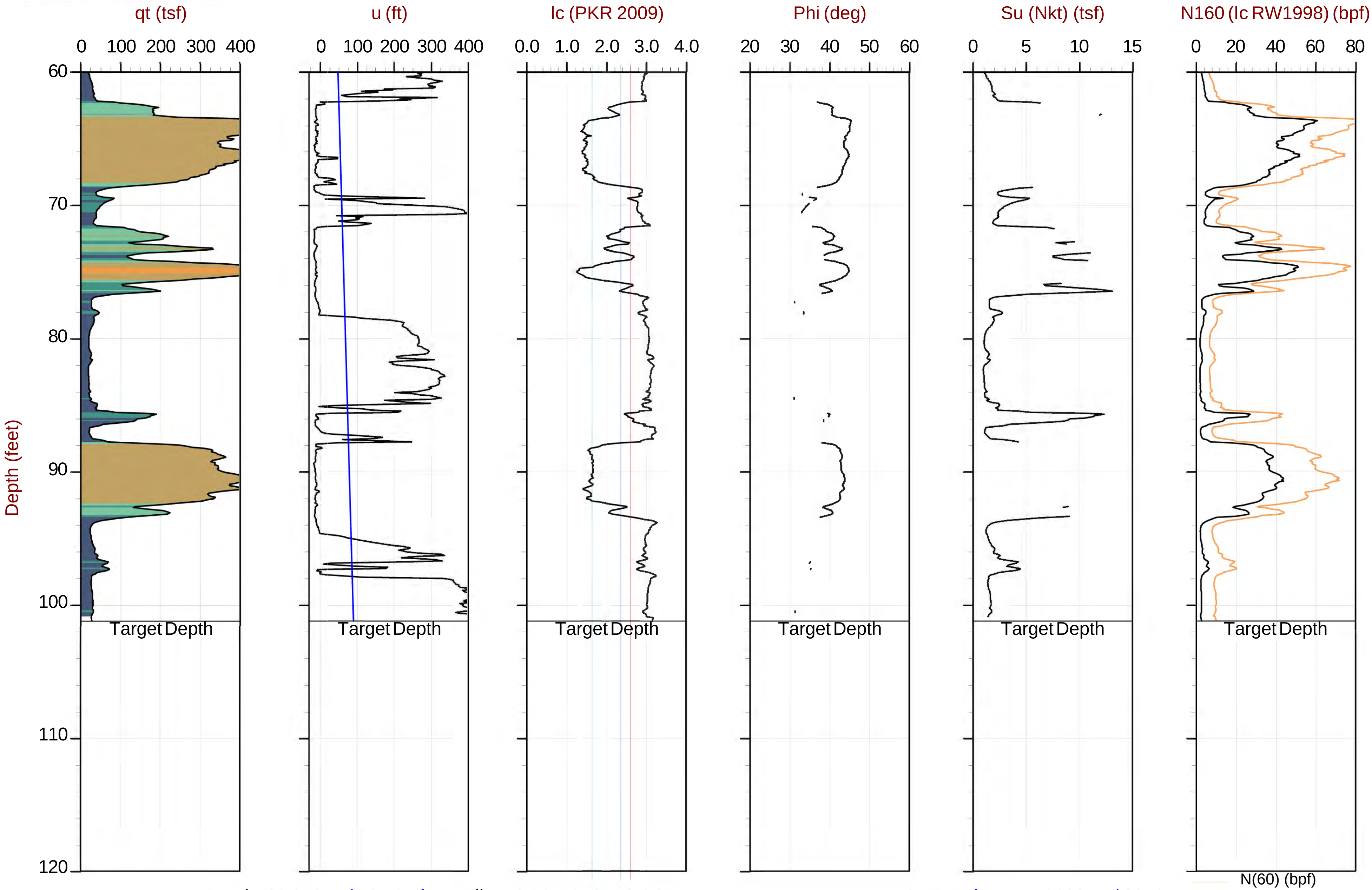
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-26 11:23
Site: East Whisman

Sounding: CPT-12
Cone: 446:T1500F15U500



Max Depth: 30.850 m / 101.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP12.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139230m E: 583883m

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

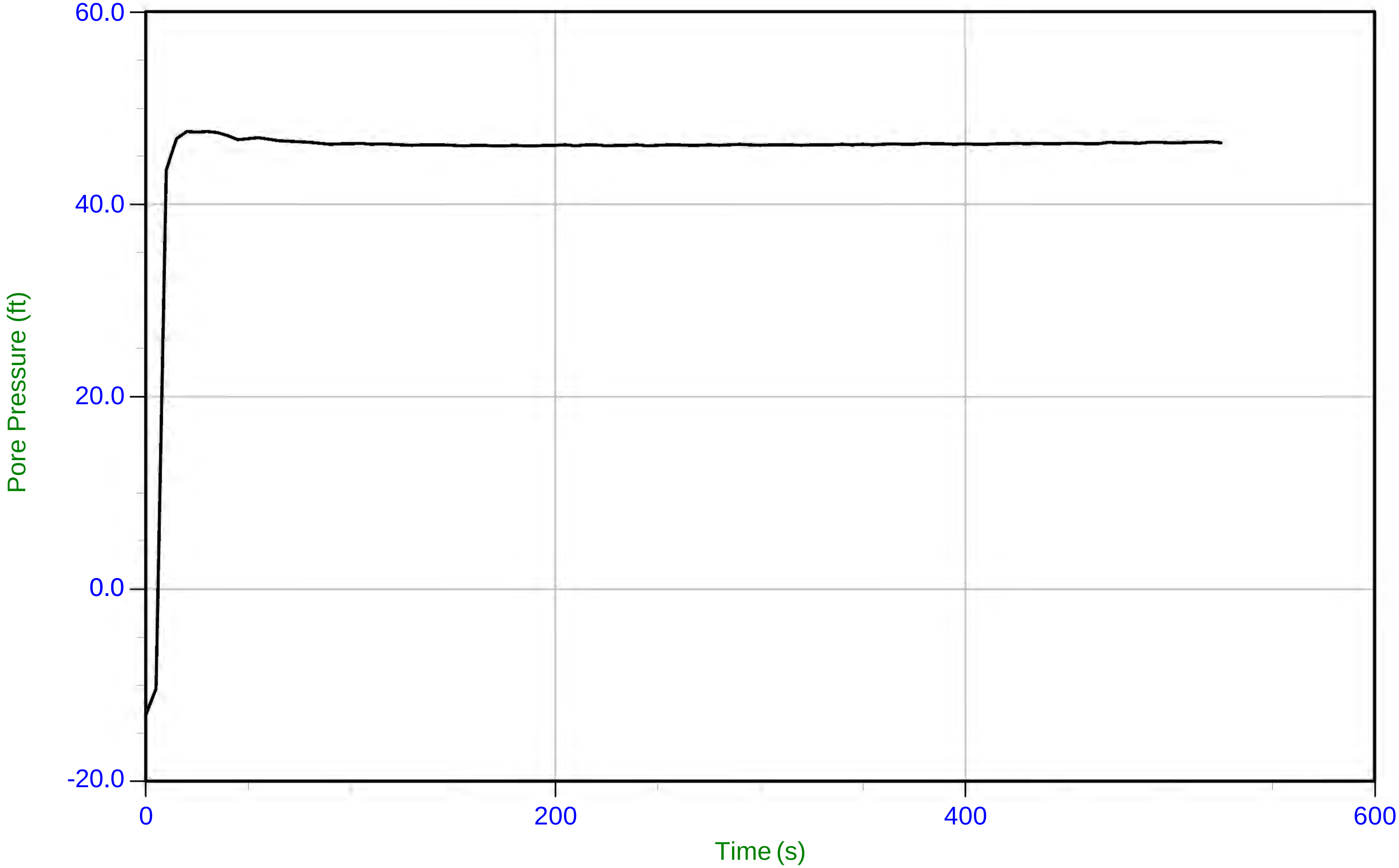
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56150
Date: 09/26/2019 11:23
Site: East Whisman

Sounding: CPT-12
Cone: 446:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56150_CP12.PPD
Depth: 17.850 m / 58.562 ft
Duration: 525.0 s

u Min: -13.1 ft
u Max: 47.6 ft
u Final: 46.4 ft

WT: 3.697 m / 12.130 ft
Ueq: 46.4 ft

DATE DRILLED 9/27/2019 BORING NO. B-6

GROUND ELEVATION 58'±(MSL) SHEET 1 OF 2

METHOD OF DRILLING 4" Mud Rotary, PD Failing 1500 (Pitcher), 3" HA top 6'

DRIVE WEIGHT 140 lbs (automatic trip hammer) DROP 30 inches

SAMPLED BY KCC LOGGED BY KCC REVIEWED BY PCC

DEPTH (feet)	SAMPLES	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						ASPHALT CONCRETE: Approximately 4.5 inches thick.
	Bulk				CL	AGGREGATE BASE: Approximately 4 inches thick.
	Bulk				CL	FILL: Dark brown, moist, stiff, lean CLAY; trace gravel.
	Bulk	12/7				ALLUVIUM: Brown, moist, stiff, lean CLAY.
	Driven	30/28	23.3	100.1		Very stiff.
10	Driven	26	16.7	108.2		Gray; very stiff; increase in sand content.
					SW-SC	Gray, wet, very dense, well-graded SAND with clay and gravel.
	Driven	35				
					CL	Brown, wet, stiff, sandy lean CLAY.
20	Driven	10	21.4	107.9		
					ML	Brown, wet, loose, sandy SILT.
	Driven	16	23.8	99.0		Gray, medium dense.
	Driven	20			SC	Gray, wet, medium dense, clayey SAND with gravel.
30					CL	Gray, wet, very stiff, sandy lean CLAY.
	Driven	14	30.0	90.1		
40						

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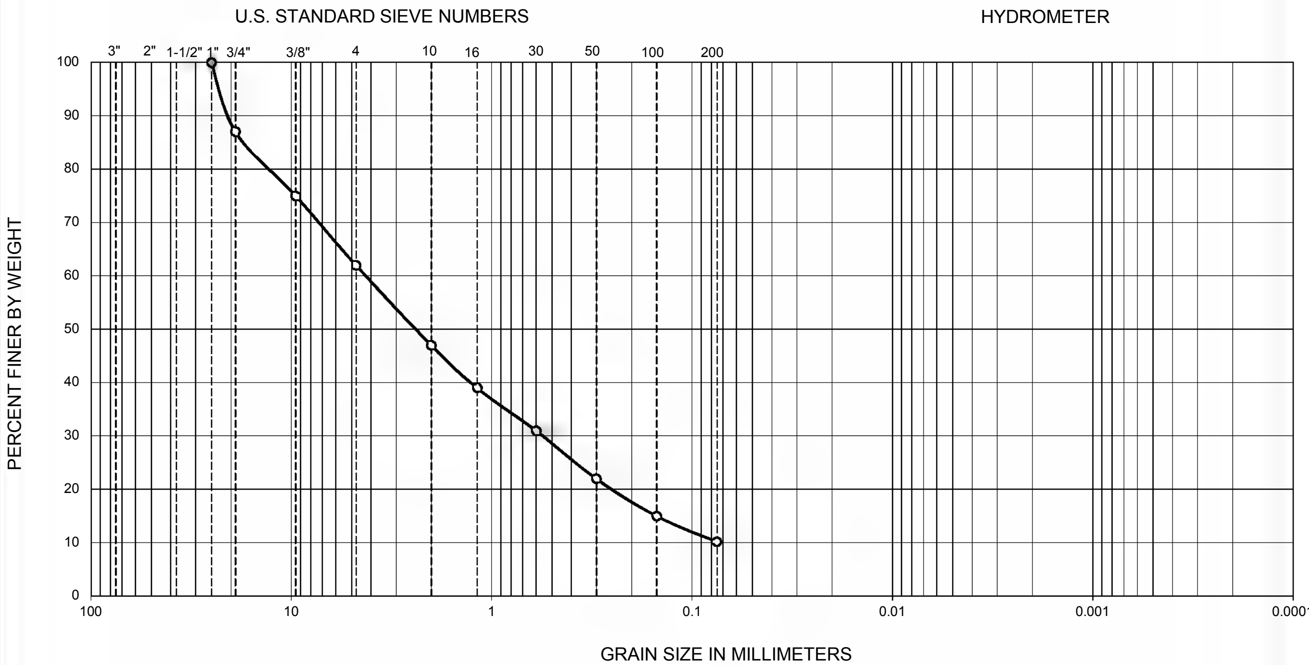
DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 9/27/2019 BORING NO. B-6 GROUND ELEVATION 58'±(MSL) SHEET 2 OF 2 METHOD OF DRILLING 4" Mud Rotary, PD Failing 1500 (Pitcher), 3" HA top 6' DRIVE WEIGHT 140 lbs (automatic trip hammer) DROP 30 inches SAMPLED BY KCC LOGGED BY KCC REVIEWED BY PCC DESCRIPTION/INTERPRETATION
40			25				CL	ALLUVIUM:(continued) Olive gray, wet, very stiff, sandy lean CLAY.
							SC	Olive gray, wet, medium dense, clayey SAND with gravel.
			41	11.1	125.2		SW	Olive gray, wet, medium dense, well-graded SAND with gravel. Total depth = 44.5 feet. Backfilled with cement grout on 9/27/2019. Notes: Depth to groundwater obscured by method of drilling. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
50								
60								
70								
80								

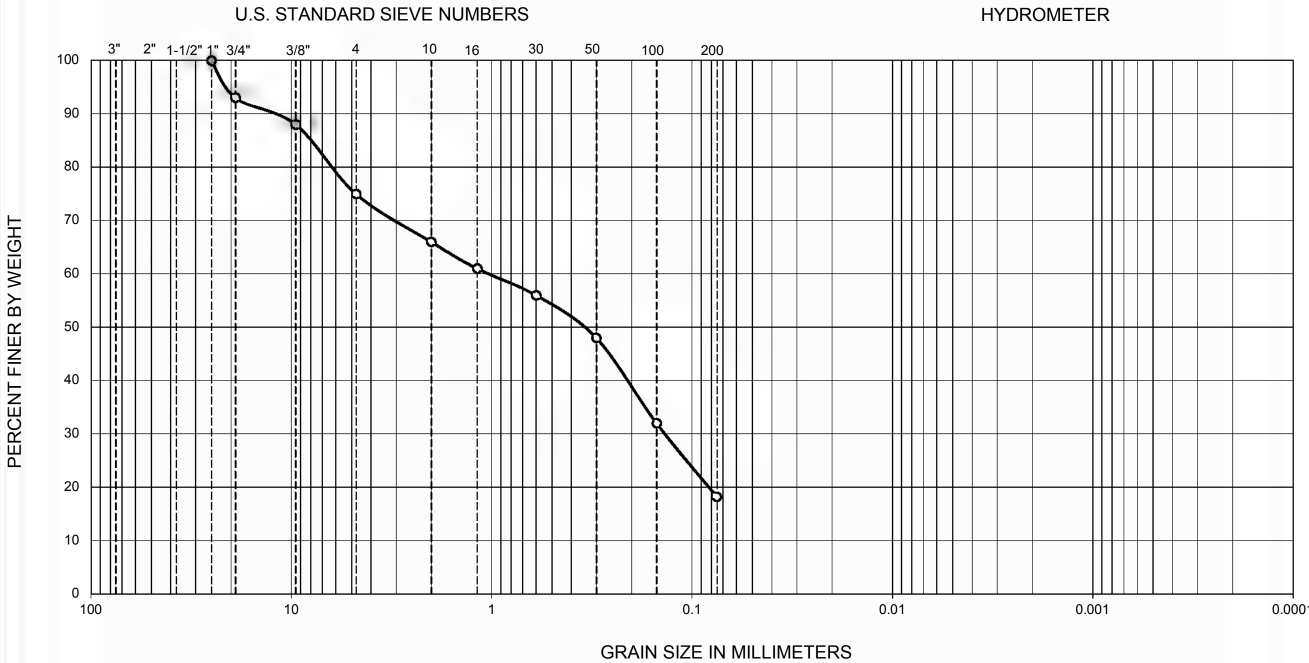
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-6	15.0-16.5	--	--	--	0.08	0.56	4.30	57.3	1.0	10	SW-SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-6	28.5-30.0	--	--	--	--	--	--	--	--	18	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-2

GRADATION TEST RESULTS

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EAST WHISMAN
MOUNTAIN VIEW, CALIFORNIA

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GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-6	43.5-44.0	--	--	--	--	--	--	--	--	32	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

GRADATION TEST RESULTS

EAST WHISMAN
MOUNTAIN VIEW, CALIFORNIA

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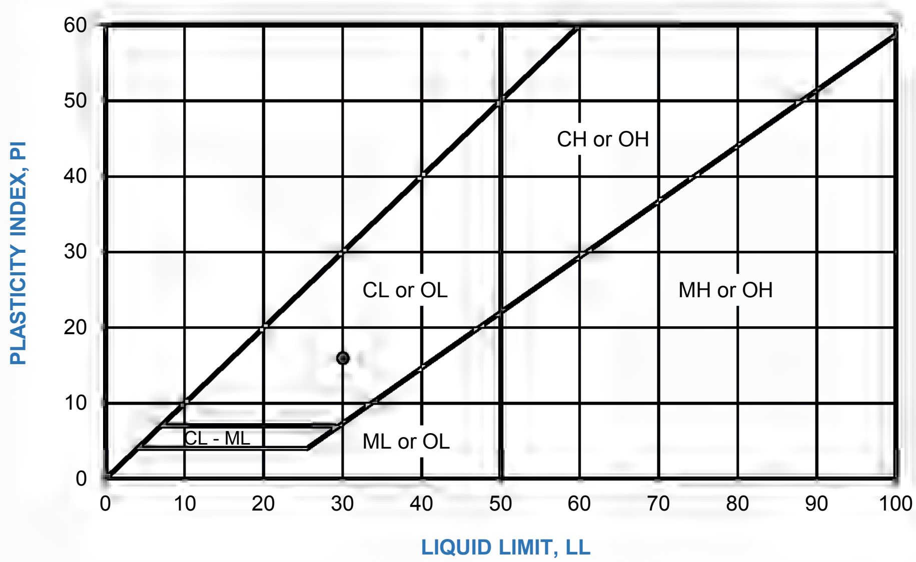
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-6	11.0-11.5	30	14	16	CL	CL
■	B-6	25.5-26.0	--	--	NP	ML	ML

NP - INDICATES NON-PLASTIC



PERFORMED IN ACCORDANCE WITH ASTM D 4318

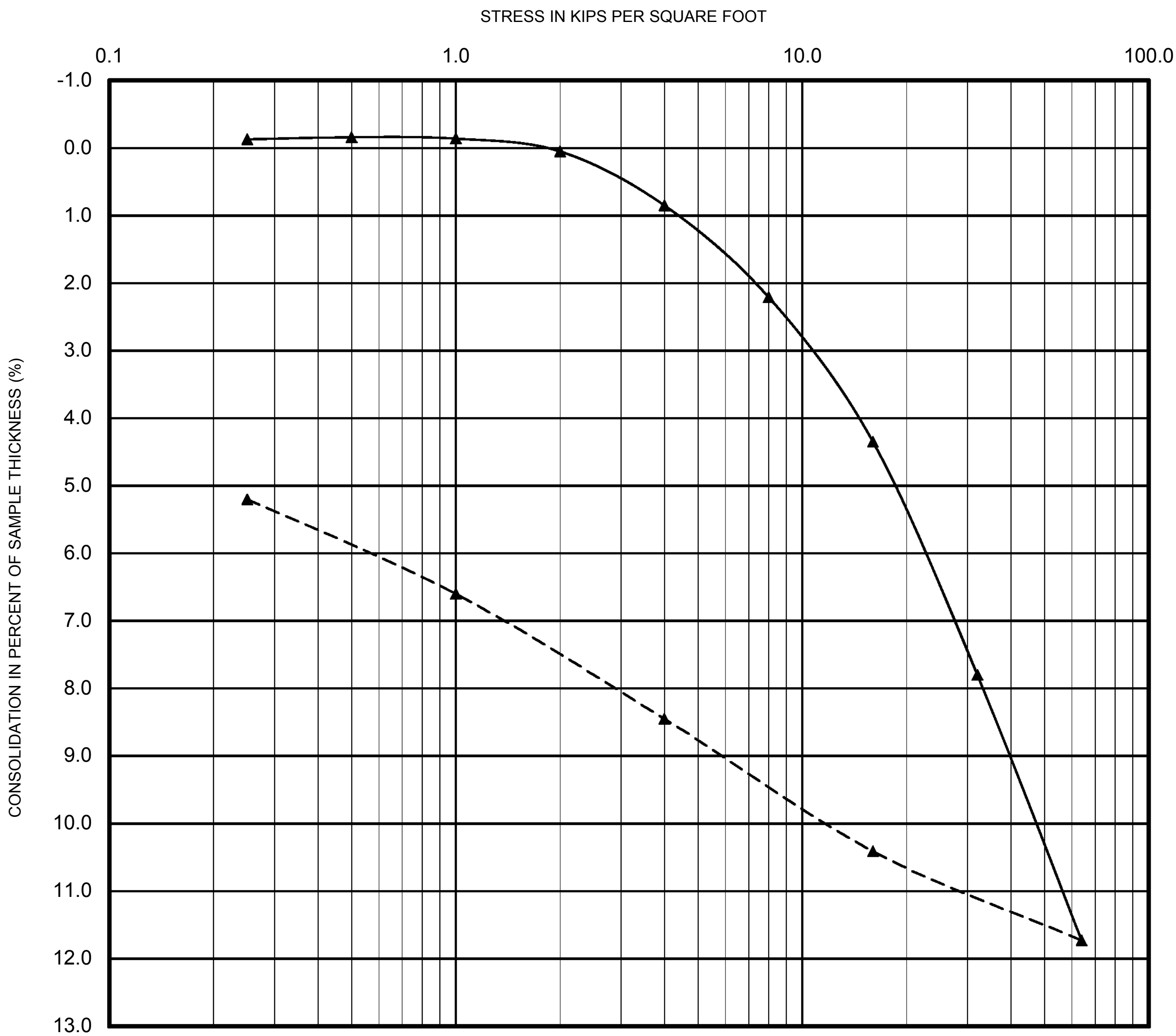
FIGURE C-4

ATTERBERG LIMITS TEST RESULTS

EAST WHISMAN
MOUNTAIN VIEW, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

403253010 | 11/19



Loading After Inundation
Rebound Cycle

Sample Location B-6
Depth (ft) 6.0-8.5
Soil Type CL

PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-5

CONSOLIDATION TEST RESULTS

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MOUNTAIN VIEW, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-6	1.0-6.0	12.0	101.1	27.5	0.072	72	Medium

PERFORMED IN ACCORDANCE WITH ASTM D 4829

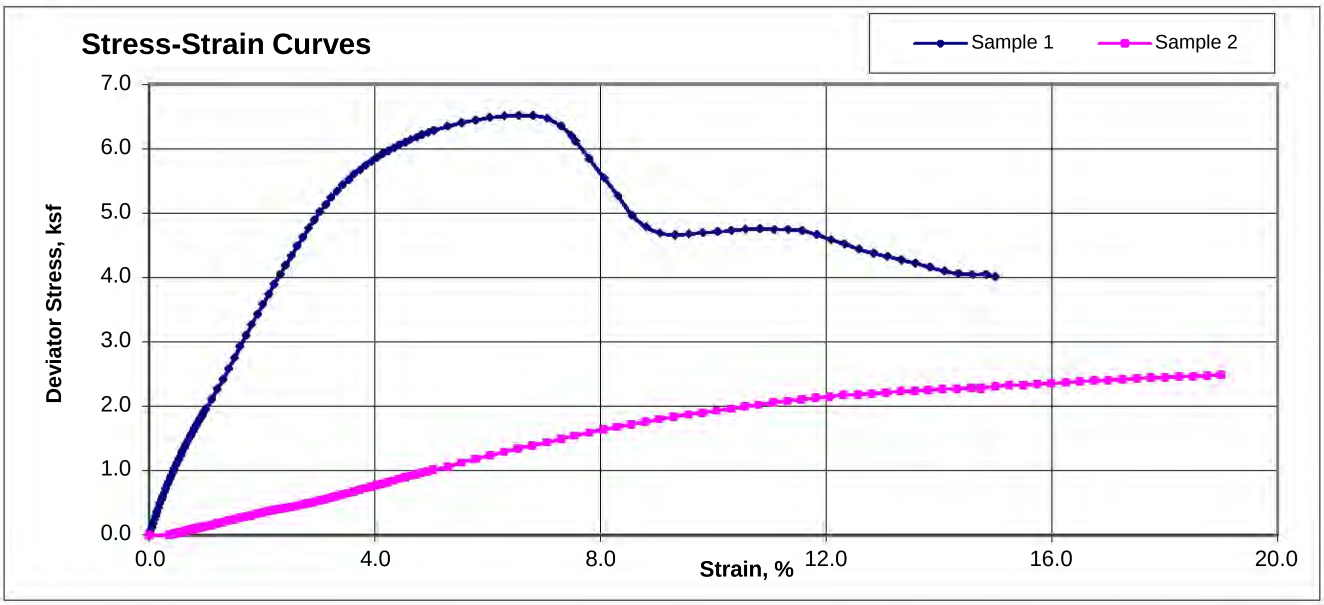
SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-6	1.0-6.0	6.6	900	1,000	0.100	650

[1] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 422

CORROSIVITY TEST RESULTS

EAST WHISMAN
MOUNTAIN VIEW, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	CELL PRESSURE (ksf)	UNDRAINED SHEAR STRENGTH (ksf)
◆	Brown Lean CLAY	CL	B-6	6.0-6.5	23.3	100.1	0.50	3.26
■	Brown Sandy Lean CLAY	CL	B-6	20.5-21.0	21.4	107.9	2.00	1.26

PERFORMED IN ACCORDANCE WITH ASTM D 2850

STRAIN RATE: 1.0%/MIN

FIGURE C-8

Ninyo & Moore

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UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST

EAST WHISMAN
MOUNTAIN VIEW, CALIFORNIA

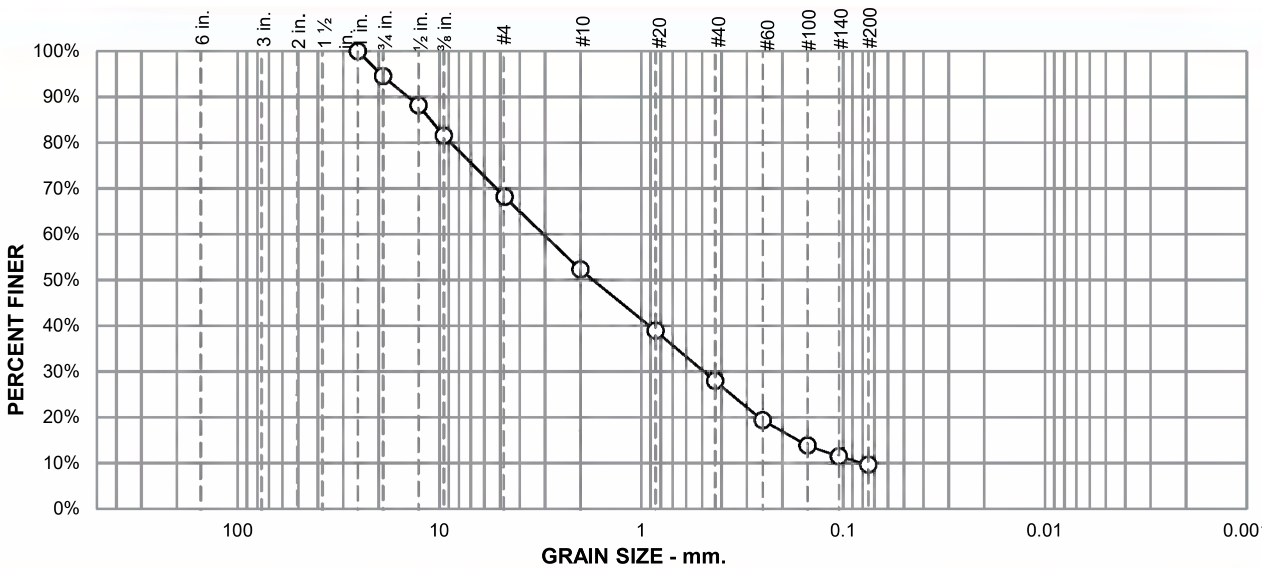
403253010 | 11/19

APPENDIX D

LABORATORY TEST DATA

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B01@18

DEPTH (ft): 18

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	5.4	26.4	15.9	24.3	18.4	9.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
¾ in.	94.6		
½ in.	88.2		
⅜ in.	81.5		
#4	68.2		
#10	52.3		
#20	39.0		
#40	28.0		
#60	19.4		
#100	13.9		
#140	11.5		
#200	9.6		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 14.2341 mm D_{85} = 11.0696 mm D_{60} = 3.0406 mm

D_{50} = 1.7249 mm D_{30} = 0.4867 mm D_{15} = 0.1661 mm

D_{10} = 0.0805 mm C_u = 37.77 C_c = 0.97

CLASSIFICATION

USCS =

REMARKS

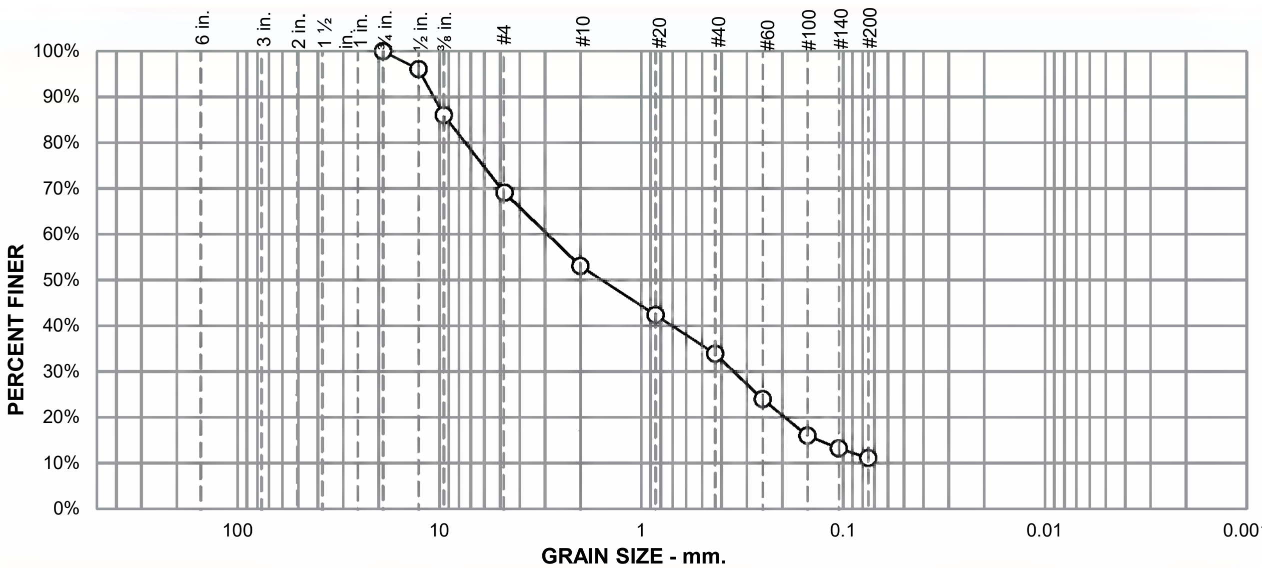
* (no specification provided)

CLIENT: Google LLC
PROJECT NAME: East Whisman Phase 1
PROJECT NO: 17954.000.001 PH002
PROJECT LOCATION: Mountain View, CA
REPORT DATE: 12/4/2020
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

ENGEO — Expect Excellence

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B01@28

DEPTH (ft): 28

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		30.9	16.0	19.2	22.8	11.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
¾ in.	100.0		
½ in.	96.1		
⅜ in.	86.1		
#4	69.1		
#10	53.1		
#20	42.4		
#40	33.9		
#60	24.0		
#100	16.0		
#140	13.2		
#200	11.1		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 10.6559 mm D_{85} = 9.1057 mm D_{60} = 2.9043 mm

D_{50} = 1.5609 mm D_{30} = 0.3473 mm D_{15} = 0.1321 mm

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

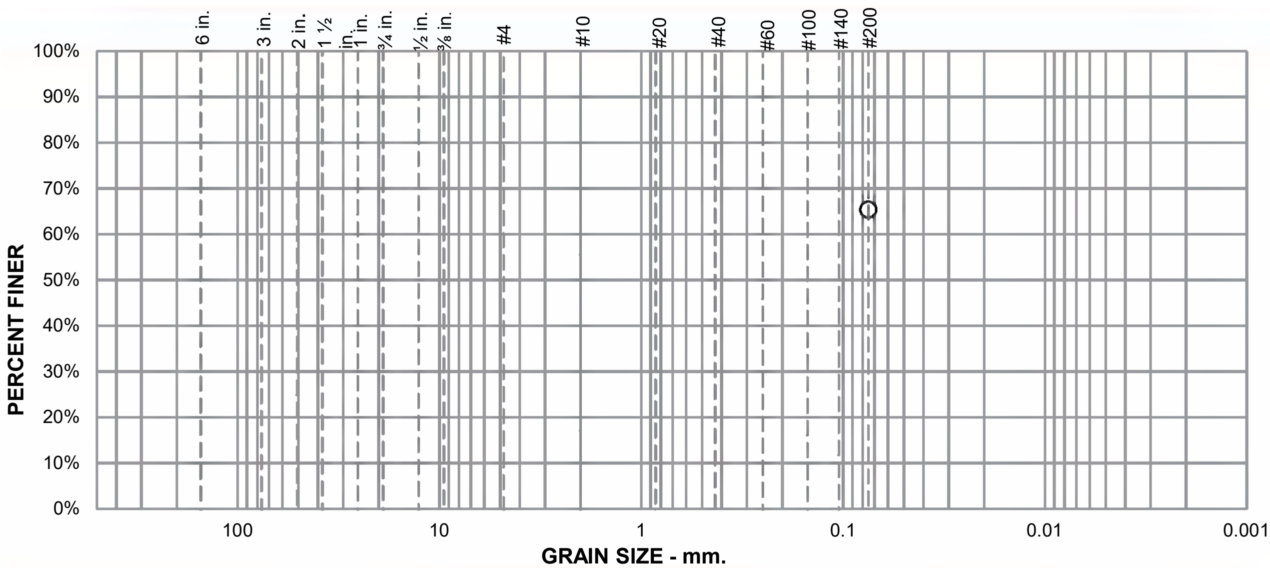
REMARKS

* (no specification provided)

CLIENT: Google LLC
PROJECT NAME: East Whisman Phase 1
PROJECT NO: 17954.000.001 PH002
PROJECT LOCATION: Mountain View, CA
REPORT DATE: 12/4/2020
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B01@46

DEPTH (ft): 46

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						65.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	65.4		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 176.3 g

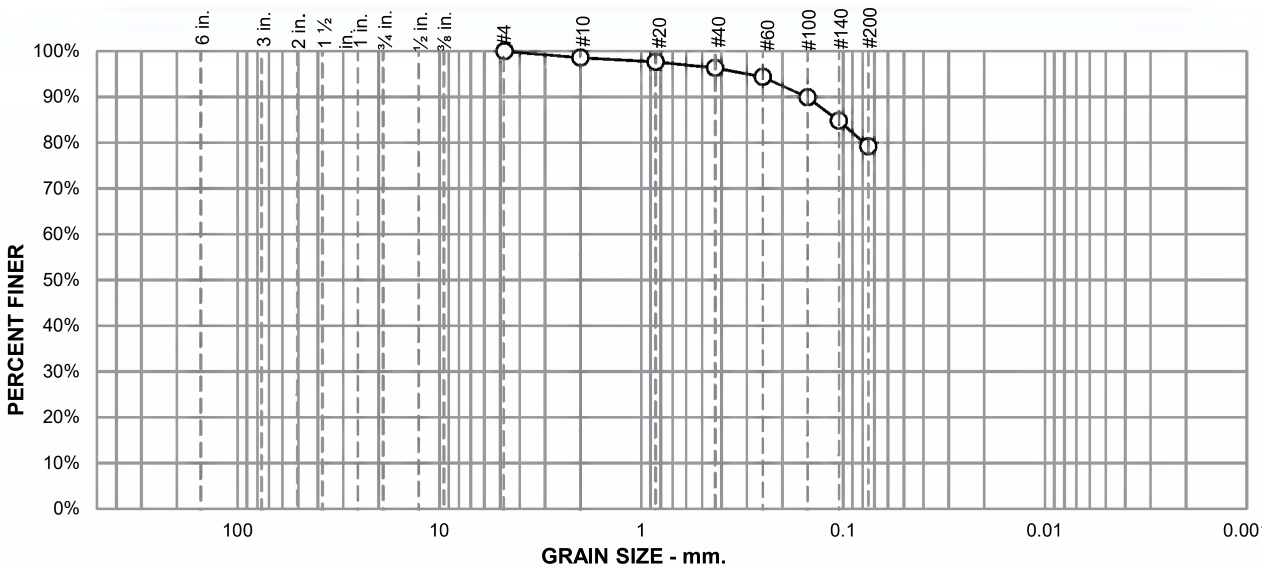
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: Google LLC
PROJECT NAME: East Whisman Phase 1
PROJECT NO: 17954.000.001 PH002
PROJECT LOCATION: Mountain View, CA
REPORT DATE: 12/4/2020
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B01@5.5

DEPTH (ft): 5.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
			1.4	2.2	17.2	79.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.6		
#20	97.7		
#40	96.4		
#60	94.4		
#100	90.0		
#140	84.8		
#200	79.2		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 0.1500 mm	D_{85} = 0.1065 mm	D_{60} =
D_{50} =	D_{30} =	D_{15} =
D_{10} =	C_u =	C_c =

CLASSIFICATION

USCS =

REMARKS

ENGEO Expect Excellence

CLIENT: Google LLC

PROJECT NAME: East Whisman Phase 1

PROJECT NO: 17954.000.001 PH002

PROJECT LOCATION: Mountain View, CA

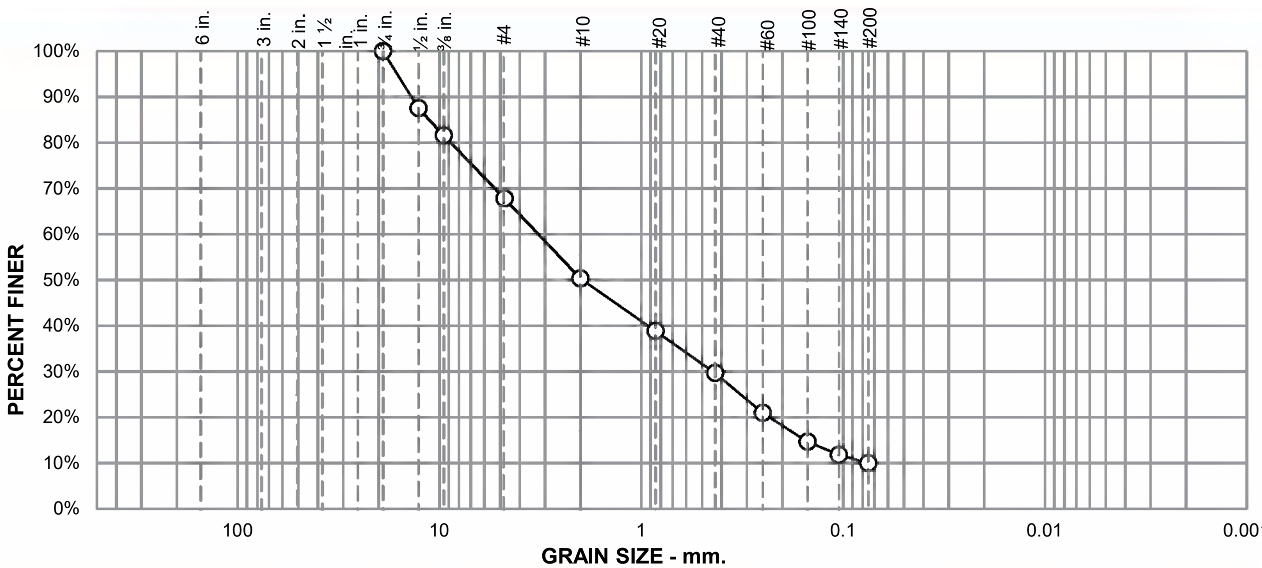
REPORT DATE: 12/4/2020

TESTED BY: M. Quasem

REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B02@14

DEPTH (ft): 14

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		32.2	17.4	20.6	19.8	10.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
¾ in.	100.0		
½ in.	87.6		
⅜ in.	81.6		
#4	67.8		
#10	50.4		
#20	38.9		
#40	29.8		
#60	21.0		
#100	14.7		
#140	11.8		
#200	10.0		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 13.7368 mm　D_{85} = 11.2115 mm　D_{60} = 3.2232 mm

D_{50} = 1.9414 mm　D_{30} = 0.4365 mm　D_{15} = 0.1537 mm

D_{10} = 0.0750 mm　C_u = 42.98　C_c = 0.79

CLASSIFICATION

USCS =

REMARKS

* (no specification provided)

ENGEO — Expect Excellence

CLIENT: Google LLC

PROJECT NAME: East Whisman Phase 1

PROJECT NO: 17954.000.001 PH002

PROJECT LOCATION: Mountain View, CA

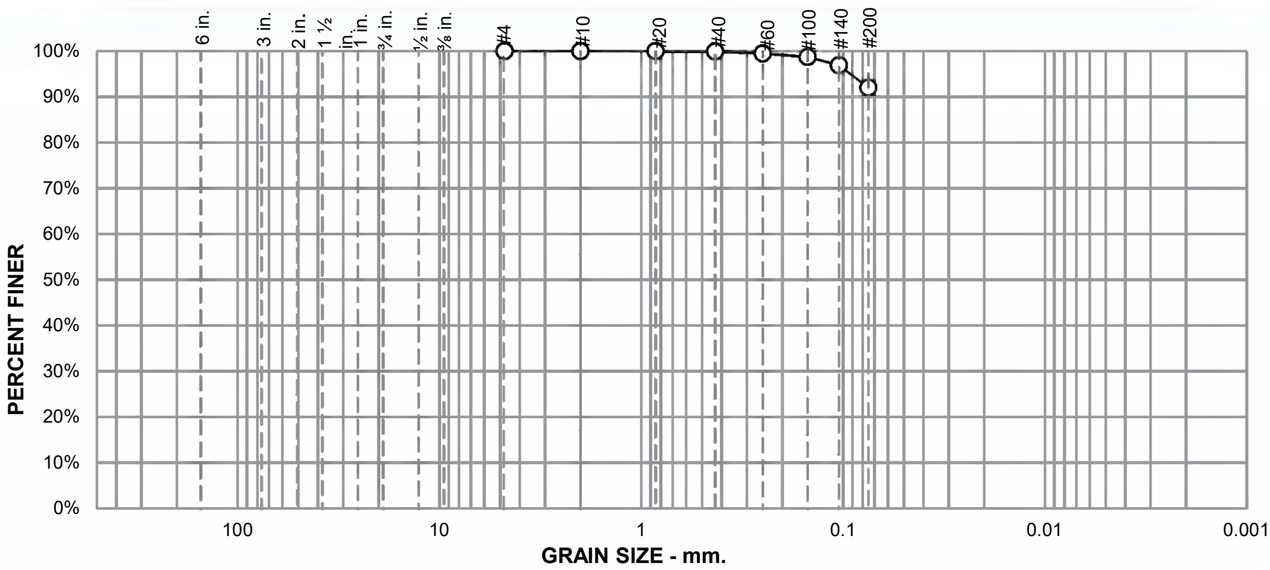
REPORT DATE: 12/9/2020

TESTED BY: M. Quasem

REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B02@19

DEPTH (ft): 19 (21.5-22.0)

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
				0.1	7.8	92.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	100.0		
#40	99.9		
#60	99.5		
#100	98.8		
#140	96.9		
#200	92.1		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 28 LL = 30 PI = 2

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS = ML

REMARKS

PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Sampled between (21.5-22.0')

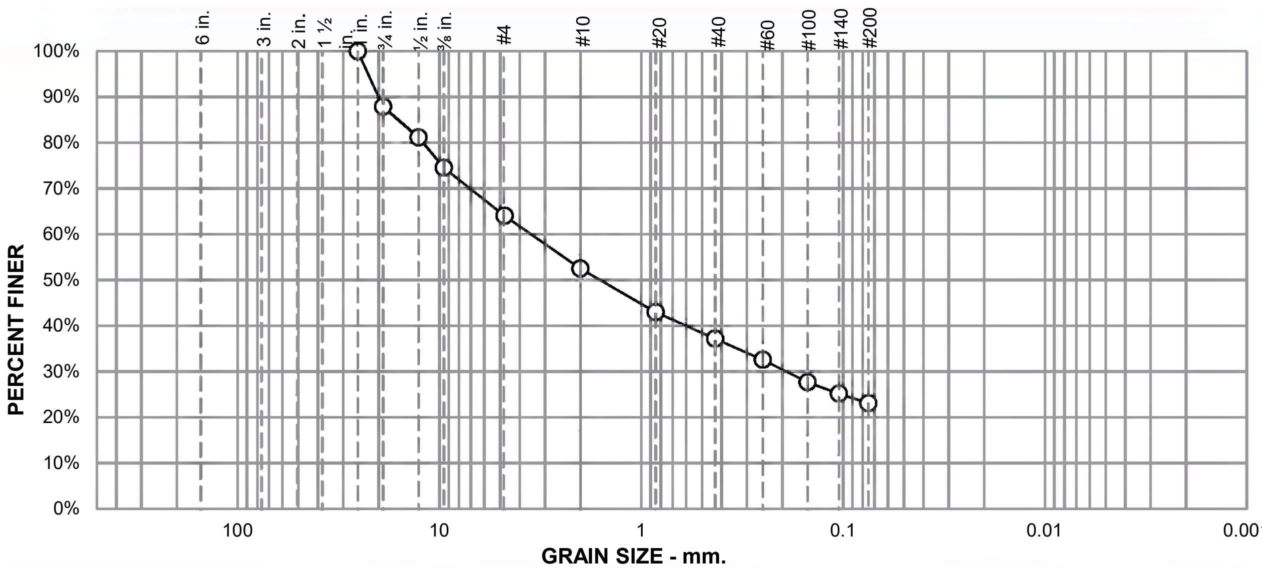
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: Google LLC
PROJECT NAME: East Whisman Phase 1
PROJECT NO: 17954.000.001 PH002
PROJECT LOCATION: Mountain View, CA
REPORT DATE: 12/14/2020
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B02@50

DEPTH (ft): 50

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	12.1	23.8	11.5	15.4	14.1	23.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
¾ in.	87.9		
½ in.	81.2		
⅜ in.	74.5		
#4	64.1		
#10	52.6		
#20	43.0		
#40	37.2		
#60	32.6		
#100	27.7		
#140	25.2		
#200	23.1		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 20.0253 mm D_{85} = 15.9837 mm D_{60} = 3.4895 mm

D_{50} = 1.5863 mm D_{30} = 0.1906 mm D_{15} =

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

* (no specification provided)

CLIENT: Google LLC

PROJECT NAME: East Whisman Phase 1

PROJECT NO: 17954.000.001 PH002

PROJECT LOCATION: Mountain View, CA

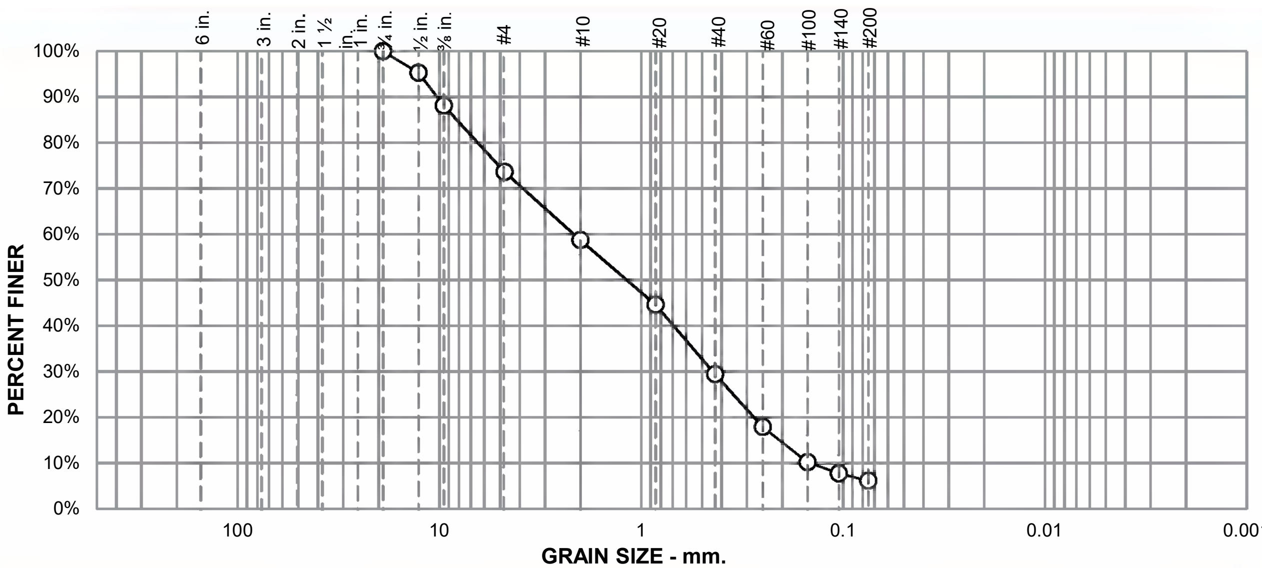
REPORT DATE: 12/4/2020

TESTED BY: M. Quasem

REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B03@26

DEPTH (ft): 26

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		26.3	14.9	29.4	23.2	6.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
¾ in.	100.0		
½ in.	95.3		
⅜ in.	88.1		
#4	73.7		
#10	58.8		
#20	44.6		
#40	29.4		
#60	17.9		
#100	10.3		
#140	7.8		
#200	6.2		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 10.2763 mm D_{85} = 8.2000 mm D_{60} = 2.1443 mm

D_{50} = 1.1769 mm D_{30} = 0.4417 mm D_{15} = 0.2057 mm

D_{10} = 0.1437 mm C_u = 14.92 C_c = 0.63

CLASSIFICATION

USCS =

REMARKS

ENGEO — Expect Excellence

CLIENT: Google LLC

PROJECT NAME: East Whisman Phase 1

PROJECT NO: 17954.000.001 PH002

PROJECT LOCATION: Mountain View, CA

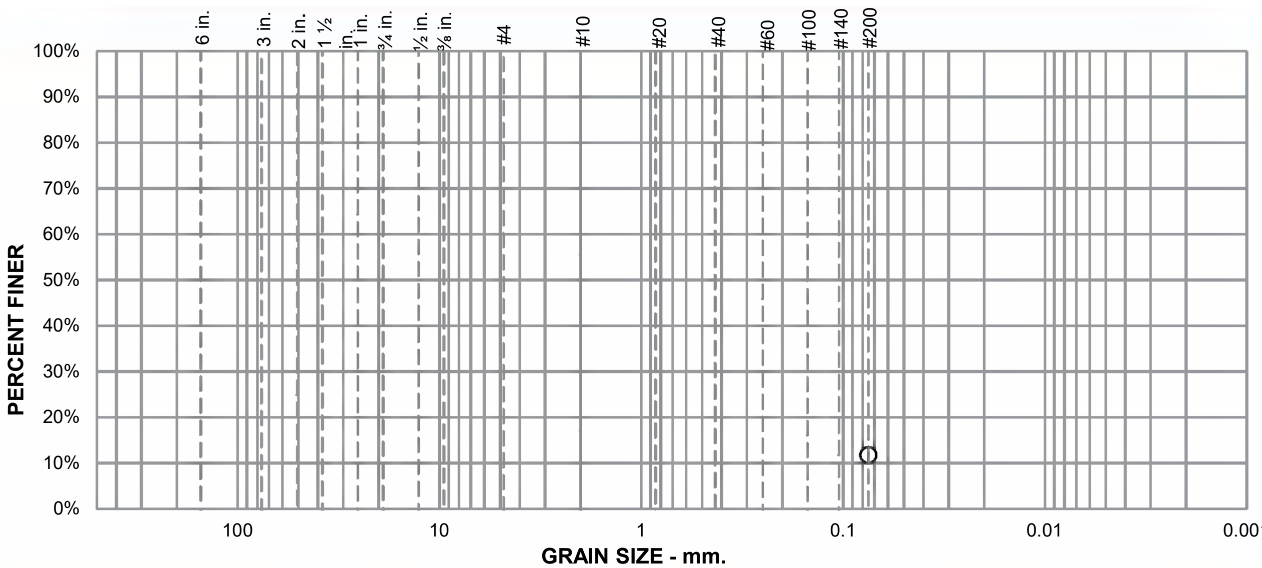
REPORT DATE: 12/4/2020

TESTED BY: M. Quasem

REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B03@30

DEPTH (ft): 30

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						11.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	11.8		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

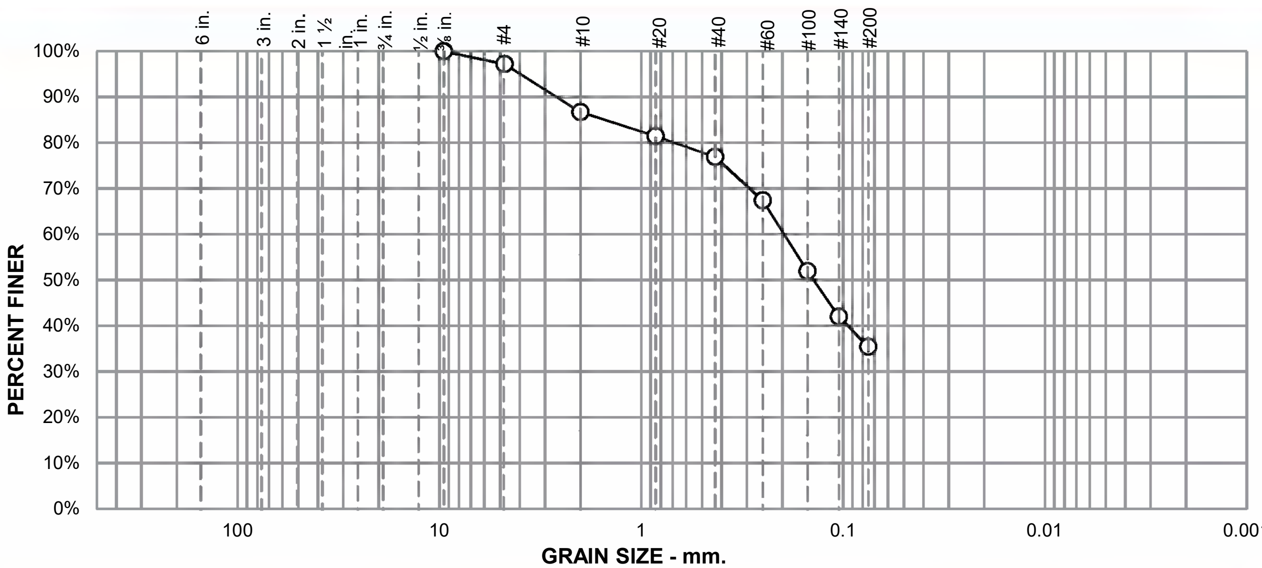
Soak time = 180 min
Dry sample weight = 362.2 g

ENGEO Expect Excellence

CLIENT: Google LLC
PROJECT NAME: East Whisman Phase 1
PROJECT NO: 17954.000.001 PH002
PROJECT LOCATION: Mountain View, CA
REPORT DATE: 12/4/2020
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B03@36

DEPTH (ft): 36

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		2.8	10.4	9.8	41.5	35.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
⅜ in.	100.0		
#4	97.2		
#10	86.8		
#20	81.5		
#40	77.0		
#60	67.4		
#100	52.0		
#140	42.0		
#200	35.5		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 2.6099 mm D_{85} = 1.4956 mm D_{60} = 0.1956 mm

D_{50} = 0.1397 mm D_{30} = D_{15} =

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

* (no specification provided)

CLIENT: Google LLC

PROJECT NAME: East Whisman Phase 1

PROJECT NO: 17954.000.001 PH002

PROJECT LOCATION: Mountain View, CA

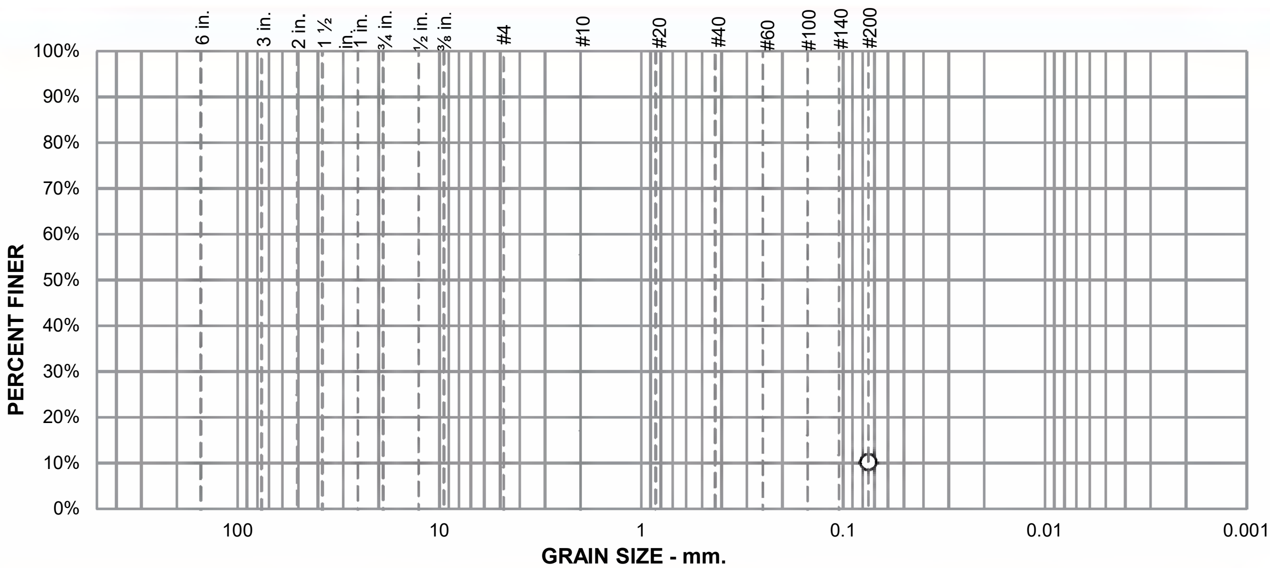
REPORT DATE: 12/4/2020

TESTED BY: M. Quasem

REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B03@65

DEPTH (ft): 65

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						10.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	10.2		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

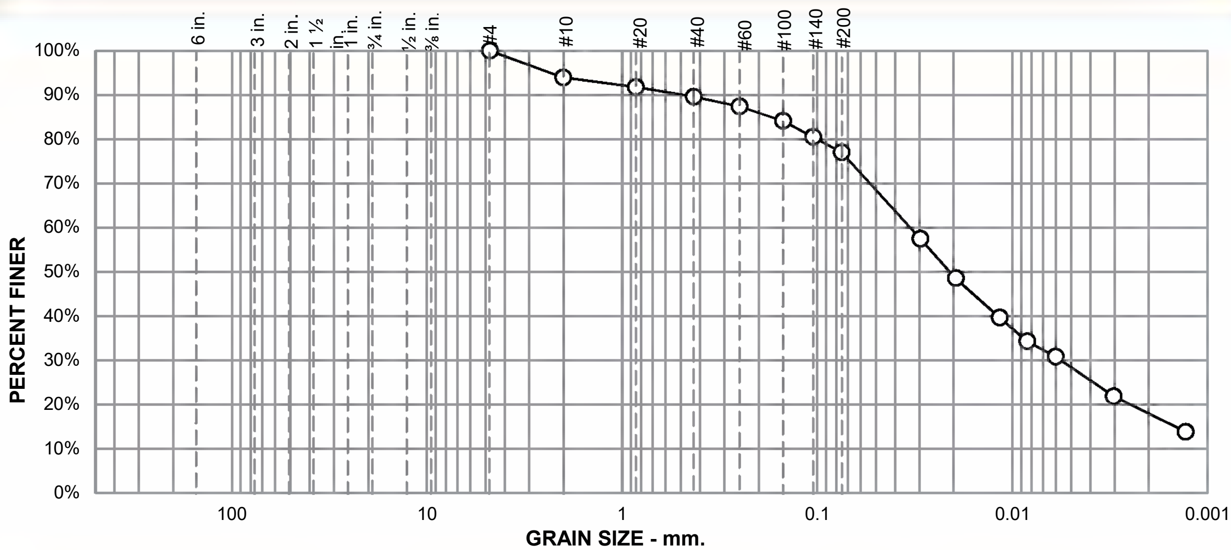
Soak time = 180 min
Dry sample weight = 563 g

* (no specification provided)

ENGEO Expect Excellence

CLIENT: Google LLC
PROJECT NAME: East Whisman Phase 1
PROJECT NO: 17954.000.001 PH002
PROJECT LOCATION: Mountain View, CA
REPORT DATE: 12/4/2020
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D422



SAMPLE ID: 1-B03@40

DEPTH (ft): 40

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
			6.0	4.4	12.5	59.1	18.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	94.0		
#20	91.8		
#40	89.6		
#60	87.4		
#100	84.2		
#140	80.5		
#200	77.1		
0.0296 mm.	57.5		
0.0195 mm.	48.6		
0.0116 mm.	39.7		
0.0084 mm.	34.3		
0.0060 mm.	30.8		
0.0030 mm.	21.9		
0.0013 mm.	13.9		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 0.4867 mm D_{85} = 0.1704 mm D_{60} = 0.0334 mm

D_{50} = 0.0208 mm D_{30} = 0.0056 mm D_{15} = 0.0015 mm

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used

Specific Gravity (ASTM D854) = 2.716

ENGEO
Expect Excellence

CLIENT: Google LLC

PROJECT NAME: East Whisman Phase 1

PROJECT NO: 17954.000.001 PH002

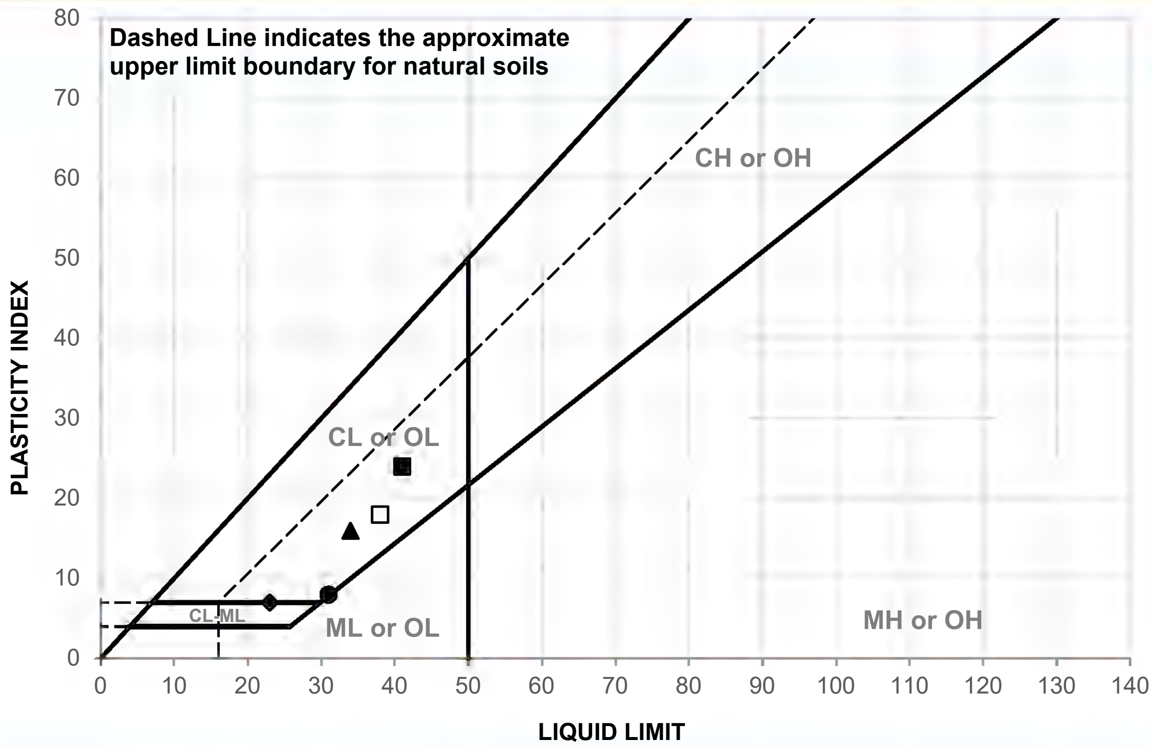
PROJECT LOCATION: Mountain View, CA

REPORT DATE: 12/4/2020

TESTED BY: M. Quasem

REVIEWED BY: W. Miller

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B01@6	6.0 feet	See exploration logs	34	18	16
◆	1-B01@33	33.0 feet	See exploration logs	23	16	7
□	1-B01@51	51.0 feet	See exploration logs	38	20	18
●	1-B01@60.5	60.5 feet	See exploration logs	31	23	8
■	1-B03@2.5	20.0 feet	See exploration logs	41	17	24

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B01@6	PI: ASTM D4318, Wet Method	
◆	1-B01@33	PI: ASTM D4318, Wet Method	
□	1-B01@51	PI: ASTM D4318, Wet Method	
●	1-B01@60.5	PI: ASTM D4318, Wet Method	
■	1-B03@2.5	PI: ASTM D4318, Wet Method	

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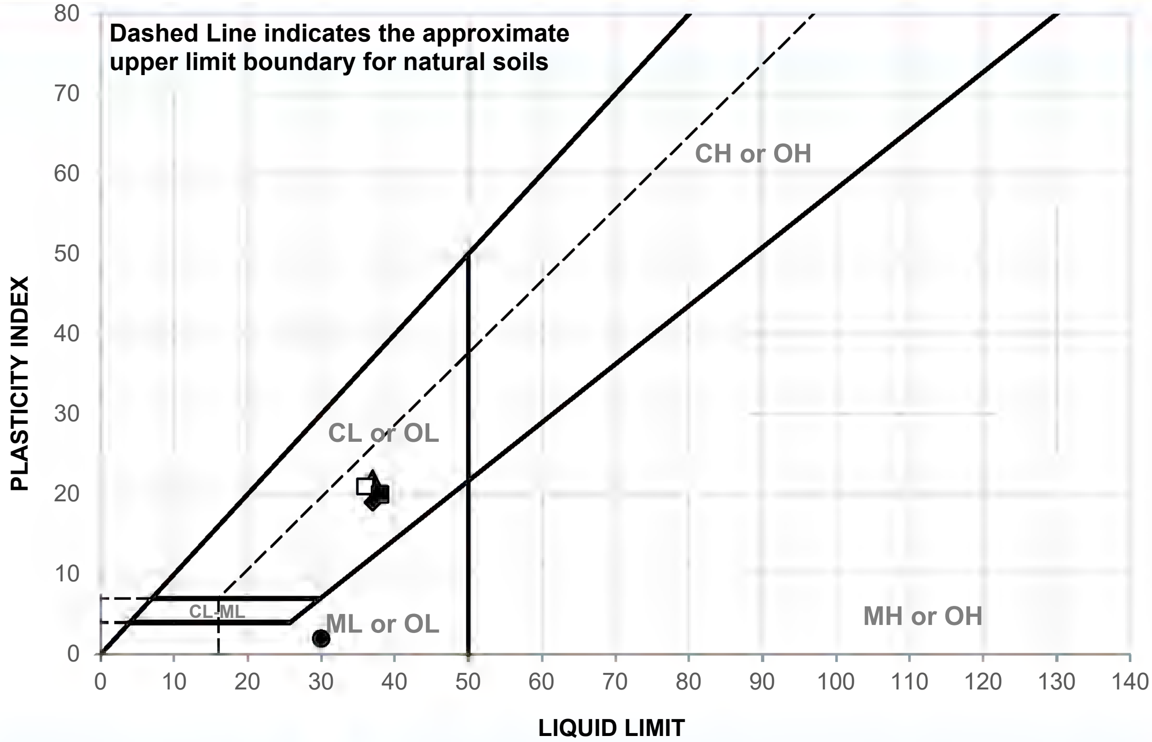
CLIENT: Google LLC
PROJECT NAME: East Whisman Phase 1
PROJECT NO: 17954.000.001 PH002
PROJECT LOCATION: Mountain View, California
REPORT DATE: 12/7/2020
TESTED BY: M. Quasem, G. Criste
REVIEWED BY: K. Lecce

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



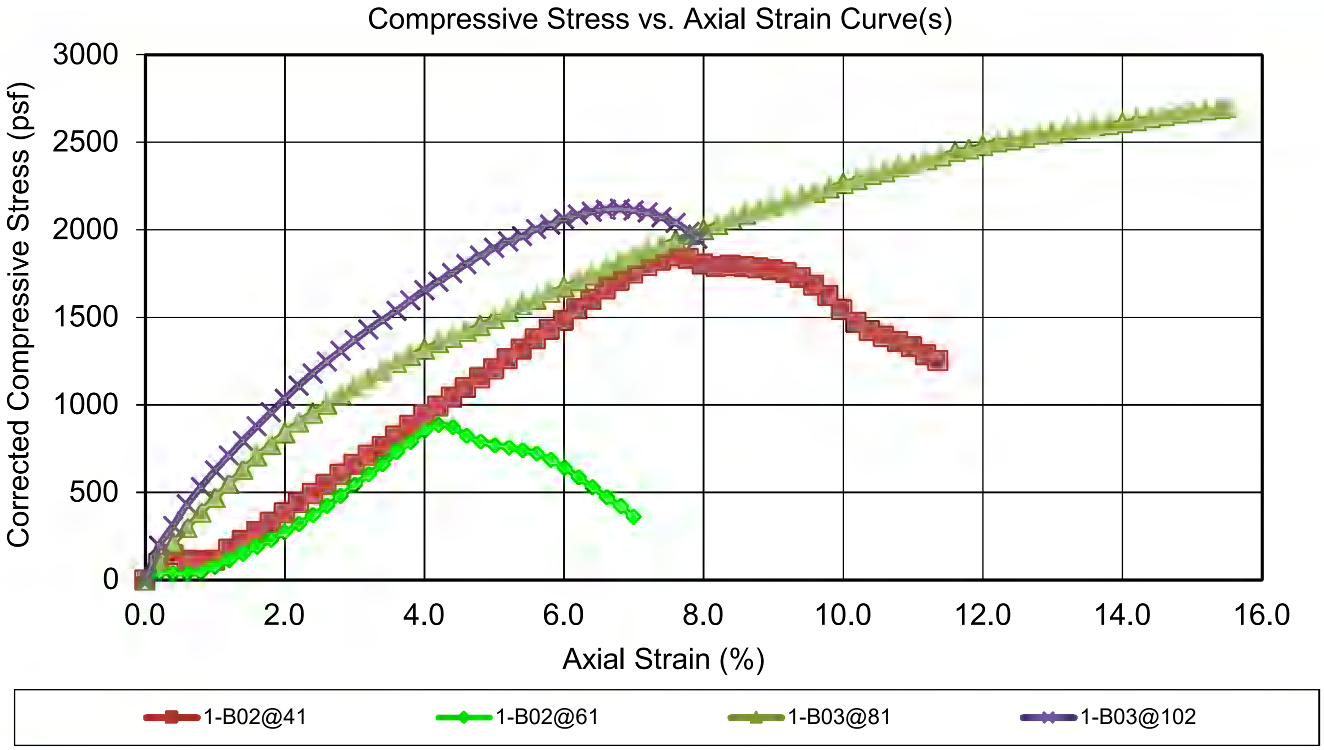
	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B03@20	20.0 feet	See exploration logs	37	15	22
◆	1-B03@40	40.0 feet	See exploration logs	37	18	19
□	1-B03@81	81.0 feet	See exploration logs	36	15	21
●	1-B02@19	19.0 feet	See exploration logs	30	28	2
■	1-B02@31	31.0 feet	See exploration logs	38	18	20

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B03@20	PI: ASTM D4318, Wet Method	
◆	1-B03@40	PI: ASTM D4318, Wet Method	
□	1-B03@81	PI: ASTM D4318, Wet Method	
●	1-B02@19	PI: ASTM D4318, Wet Method	
■	1-B02@31	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: Google LLC
PROJECT NAME: East Whisman Phase 1
PROJECT NO: 17954.000.001 PH002
PROJECT LOCATION: Mountain View, California
REPORT DATE: 12/9/2020
TESTED BY: G. Criste
REVIEWED BY: K. Lecce

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B02@41	SPECIMEN 1-B02@61	SPECIMEN 1-B03@81	SPECIMEN 1-B03@102
Test Moisture Content (%)	24.91	14.52	19.21	23.38
Dry Density (pcf)	101.2	118.8	111.5	103.8
Saturation (%)	100.0	92.1	100.0	99.9
Void Ratio	0.68	0.43	0.52	0.64
Diameter (in)	2.400	2.410	2.380	2.390
Height (in)	5.010	4.990	5.000	5.000
Height-To-Diameter Ratio	2.09	2.07	2.10	2.09
TEST DATA				
Unconfined Compressive Strength (psf)	1849	886	2699	2121
Undrained Shear Strength (psf)	924.68	443.00	1349.41	1060.30
Strain Rate (in/min)	0.050	0.050	0.050	0.050
Specific Gravity (ASSUMED)	2.720	2.720	2.720	2.720
Strain at Failure(%)	7.58	4.21	15.48	6.80
Test Remarks				

SPECIMEN	DESCRIPTION
1-B02@41	See exploration logs
1-B02@61	See exploration logs
1-B03@81	See exploration logs
1-B03@102	See exploration logs

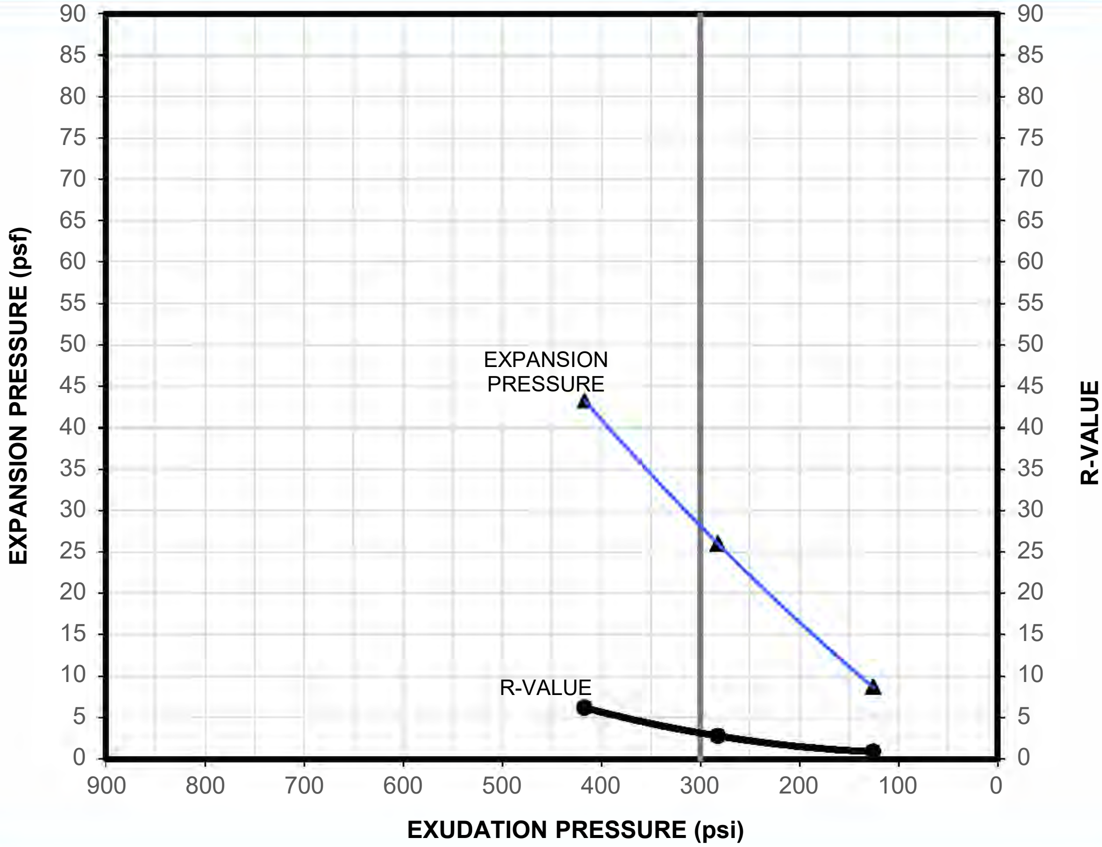
PROJECT NAME: East Whisman Phase 1
PROJECT NO: 17954.000.001 PH002
CLIENT: Google LLC
LOCATION: Mountain View, CA

Test Date: 12/2/2020
Tested By: M. Quasem
Reviewed By: W. Miller

ENGEO — Expect Excellence —

R-VALUE TEST REPORT

CTM 301



SAMPLE ID	MATERIAL DESCRIPTION	SAMPLE LOCATION		
RV-01	Composite of 1-B01 @ 2.5 + 4.0 - See Exploration Logs	1-B01		
SPECIMENS		**1**	**2**	**3**
EXUDATION PRESSURE (psi)		417	282	126
EXPANSION PRESSURE (psf)		43	26	9
R-VALUE		6	3	1
MOISTURE CONTENT (%)		21.0	23.3	26.1
DRY DENSITY (pcf)		113.2	100.1	95.9
EXPANSION PRESSURE (psf) AT EXUDATION PRESSURE OF 300 psi			28	
R-VALUE AT EXUDATION PRESSURE OF 300 psi			**TEST RESULT**	
			<5	

ENGEO
Expect Excellence

CLIENT: Google, LLC

PROJECT NAME: East Whisman Phase 1 Prelim Study

PROJECT NO: 17954.000.001

PROJECT LOCATION: Mountain View, CA

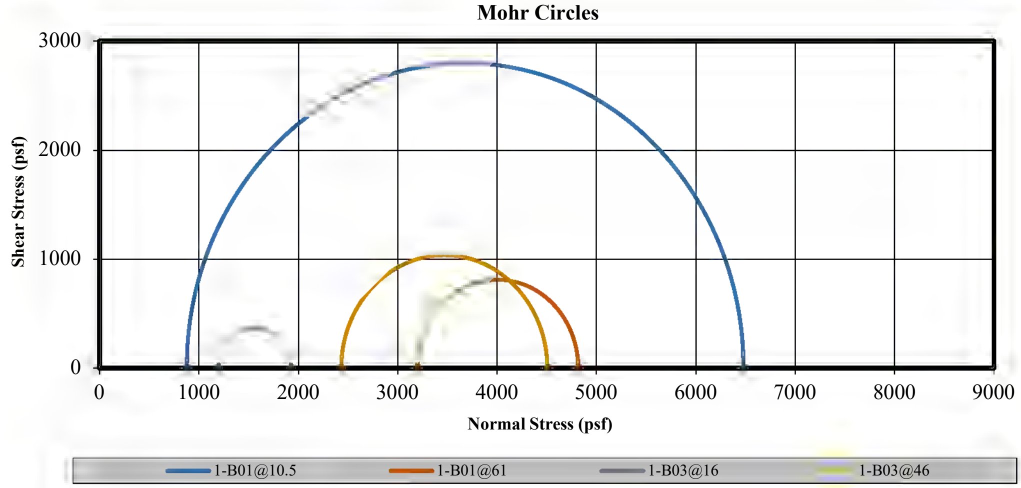
REPORT DATE: 12/8/2020

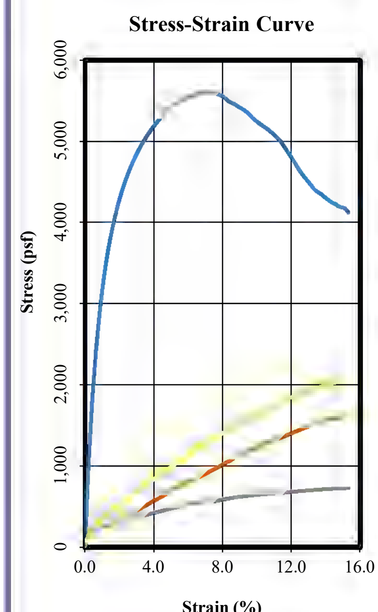
TESTED BY: W. Miller

REVIEWED BY: M. Quasem

Isotropic Unconsolidated Undrained Triaxial Test

ASTM D2850





Before Test	Specimen 1-B01@10.5	1-B01@61	1-B03@16	1-B03@46
Water Content (%)	25.60	25.62	21.40	19.05
Dry Density (pcf)	99.30	101.00	111.20	114.60
Saturation (%)	98.09	99.91	110.44	99.77
Void Ratio	0.71	0.71	0.53	0.54
Diameter (in)	2.415	2.397	2.387	2.384
Height (in)	5.019	4.960	4.987	4.950
Height-to-Diameter Ratio	2.078	2.069	2.089	2.076
ASTM D4318 - Wet Method				
Liquid Limit				
Plastic Limit				
ASTM D854 - Assumed				
Specific Gravity	2.720	2.767	2.720	2.828
After Test	**1-B01@10.5**	**1-B01@61**	**1-B03@16**	**1-B03@46**
Water Content (%)	25.60	25.62	21.40	19.05
Saturation (%)	98.09	99.91	100.00	99.77
Strain Rate (in/min)		0.05	0.05	0.05
Peak Deviator Stress (psf)	5602.0	1620.7	730.9	2072.7
Axial Strain @ Failure (%)	6.974	15.322	15.240	15.321
	Cell Pressure			
Cell (psf)	878.4	3196.8	1195.2	2433.6
Back (psf)	n/a	n/a	n/a	n/a
	Principle Stresses at Failure			
σ1 (psf)	6480.4	4817.5	1926.1	4506.3
σ3 (psf)	878.4	3196.8	1195.2	2433.6
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	n/a		2801.0	810.4	365.5	1036.3
Friction Angle Ø	n/a		n/a	n/a	n/a	n/a

Project Information	
Project Name:	East Whisman Phase 1
Project Number:	17954.000.001 PH002
Project Location:	Mountain View, California
Client:	Google LLC
Description:	See exploration logs
Test Remarks:	

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Date: 12/02/20
Checked By: K. Lecce
Date: 12/1/2020
Tested By: G. Criste

Isotropic Unconsolidated Undrained Triaxial Test

ASTM D2850

Tested By: G. Criste Date: 12/1/2020 Checked By: K. Lecce Date: 12/02/20

SPECIMEN PHOTOS

SAMPLE NUMBER: 1-B01@10.5

SAMPLE NUMBER: 1-B01@61

SAMPLE NUMBER: 1-B03@16

SAMPLE NUMBER: 1-B03@46

Project Information	
Project Name:	East Whisman Phase 1
Project Number:	17954.000.001 PH002
Project Location:	Mountain View, California
Client:	Google LLC
Description:	See exploration logs
Test Remarks:	

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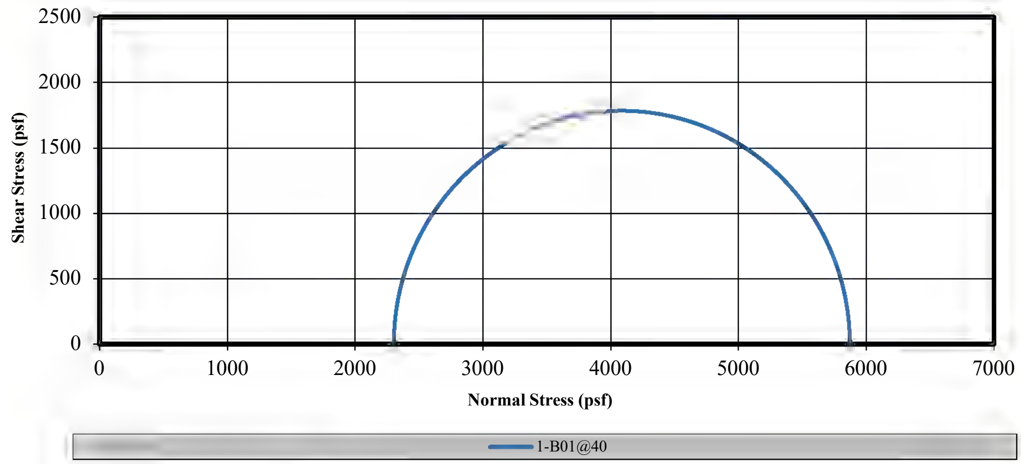
Isotropic Unconsolidated Undrained Triaxial Test

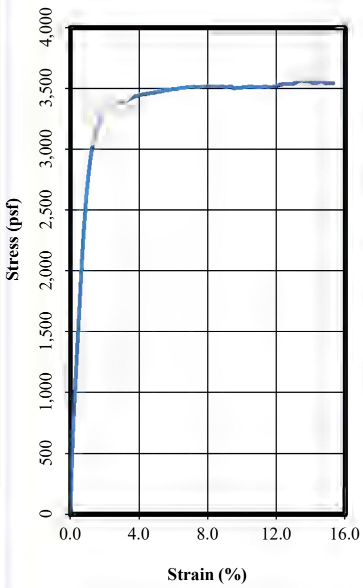
ASTM D2850

Tested By: G. Criste   Date: 12/2/2020   Checked By: K. Lecce   Date: 12/03/20

Mohr Circles



Stress-Strain Curve



	Specimen			
Before Test	**1-B01@40**			
Water Content (%)	24.98			
Dry Density (pcf)	99.20			
Saturation (%)	95.38			
Void Ratio	0.71			
Diameter (in)	2.857			
Height (in)	6.082			
Height-to-Diameter Ratio	2.129			
ASTM D4318 - Wet Method				
Liquid Limit				
Plastic Limit				
ASTM D854 - Measured				
Specific Gravity	2.726			
After Test	**1-B01@40**			
Water Content (%)	24.98			
Saturation (%)	95.59			
Strain Rate (in/min)	0.06			
Peak Deviator Stress (psf)	3569.3			
Axial Strain @ Failure (%)	13.482			
	Cell Pressure			
Cell (psf)	2304.0			
Back (psf)	n/a			
	Principle Stresses at Failure			
σ1 (psf)	5873.3			
σ3 (psf)	2304.0			
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			**Cohesion at Failure with a Zero Friction Angle (Ø=0)**		
Cohesion, c (psf)	n/a		1784.7		
Friction Angle Ø	n/a		n/a		

Project Information	
Project Name:	East Whisman Phase 1
Project Number:	17954.000.001 PH002
Project Location:	Mountain View, California
Client:	Google LLC
Description:	See exploration logs
Test Remarks:	

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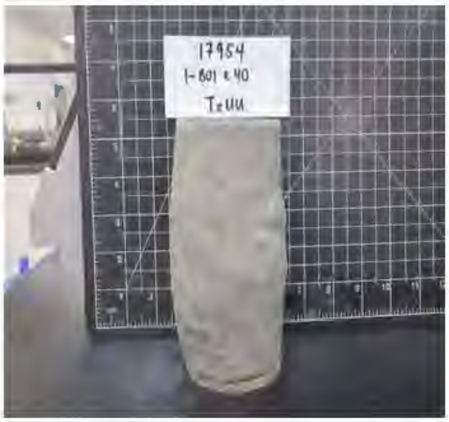
Isotropic Unconsolidated Undrained Triaxial Test

ASTM D2850

Tested By: G. Criste   Date: 12/2/2020   Checked By: K. Lecce   Date: 12/03/20

SPECIMEN PHOTOS

SAMPLE NUMBER: 1-B01@40



Project Information	
Project Name:	East Whisman Phase 1
Project Number:	17954.000.001 PH002
Project Location:	Mountain View, California
Client:	Google LLC
Description:	See exploration logs
Test Remarks:	

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APPENDIX E

LIQUEFACTION ANALYSIS

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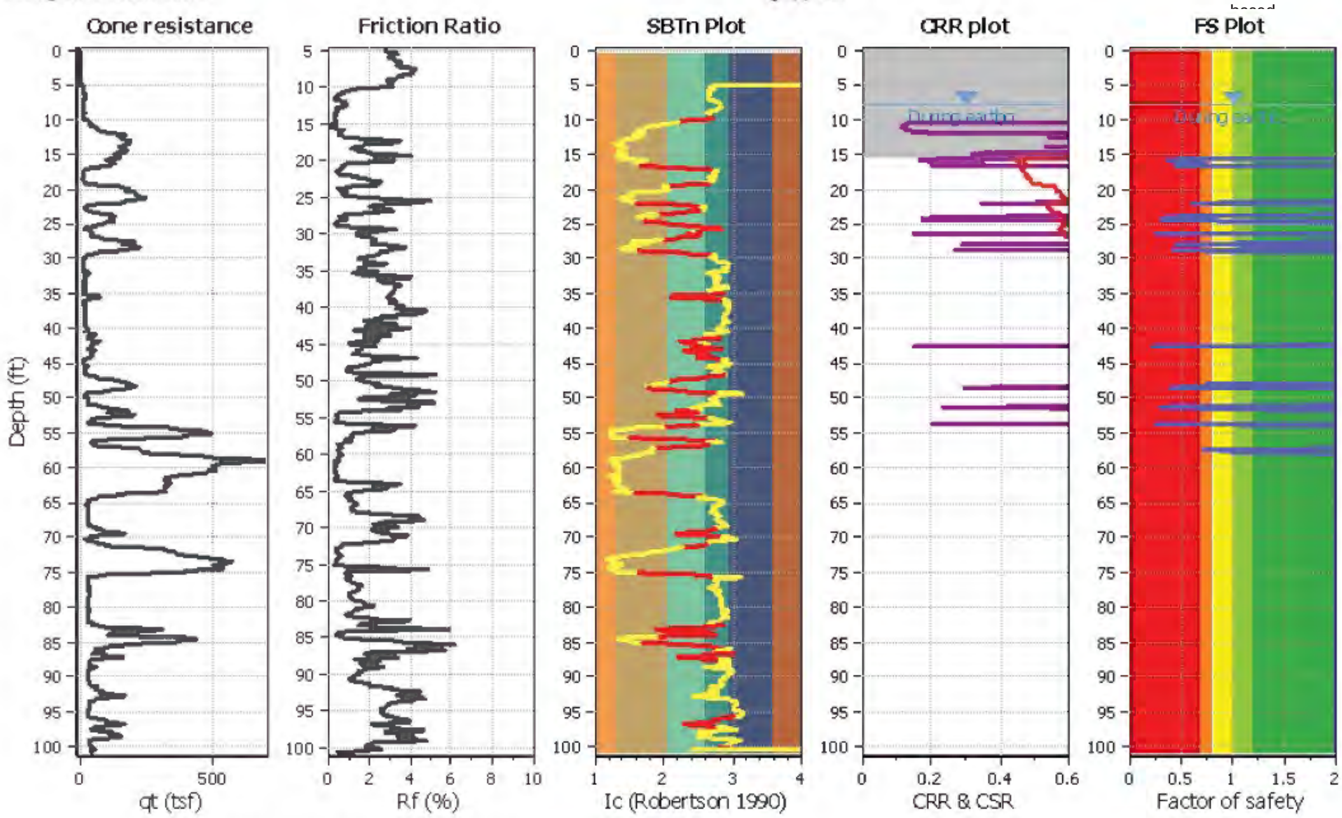
LIQUEFACTION ANALYSIS REPORT

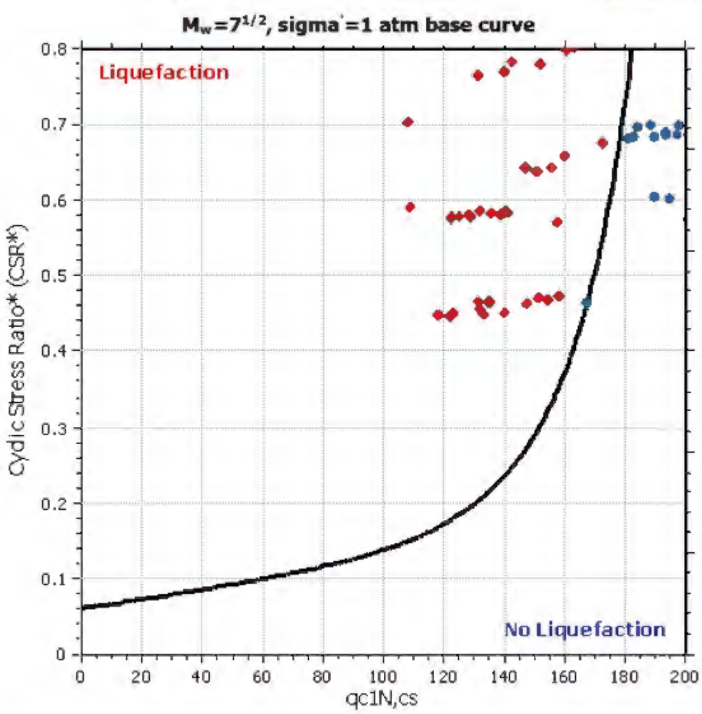
Project title : East Whisman Phase 1 **Location : Mountain View, CA**

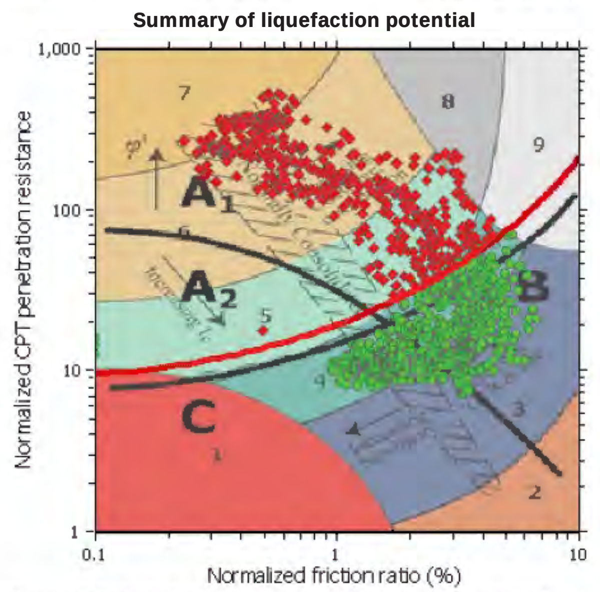
CPT file : 1-CPT01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.00 ft	Excavation:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 ft	Excavation depth:	15.50 ft	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.75 tsf	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.67	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

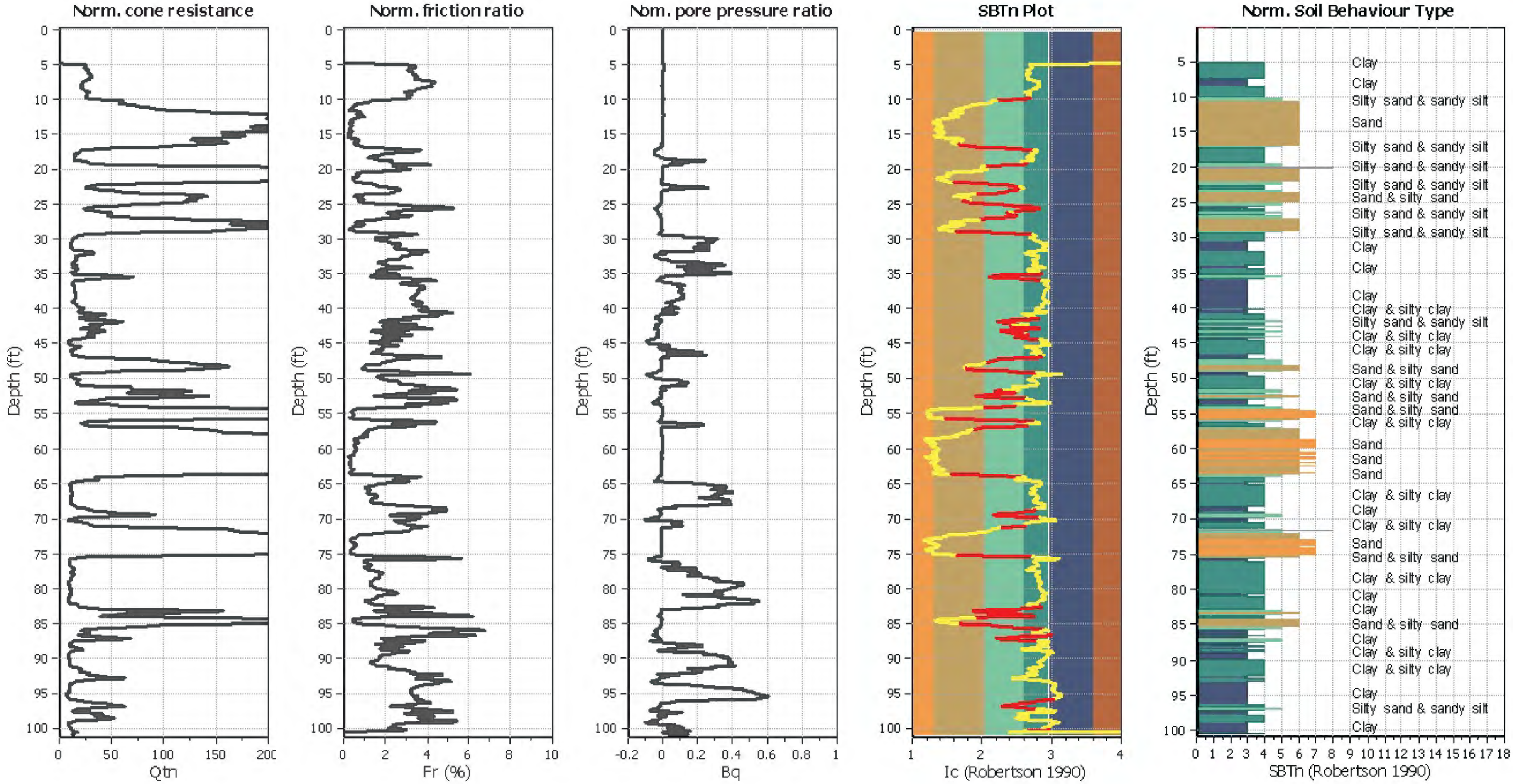






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



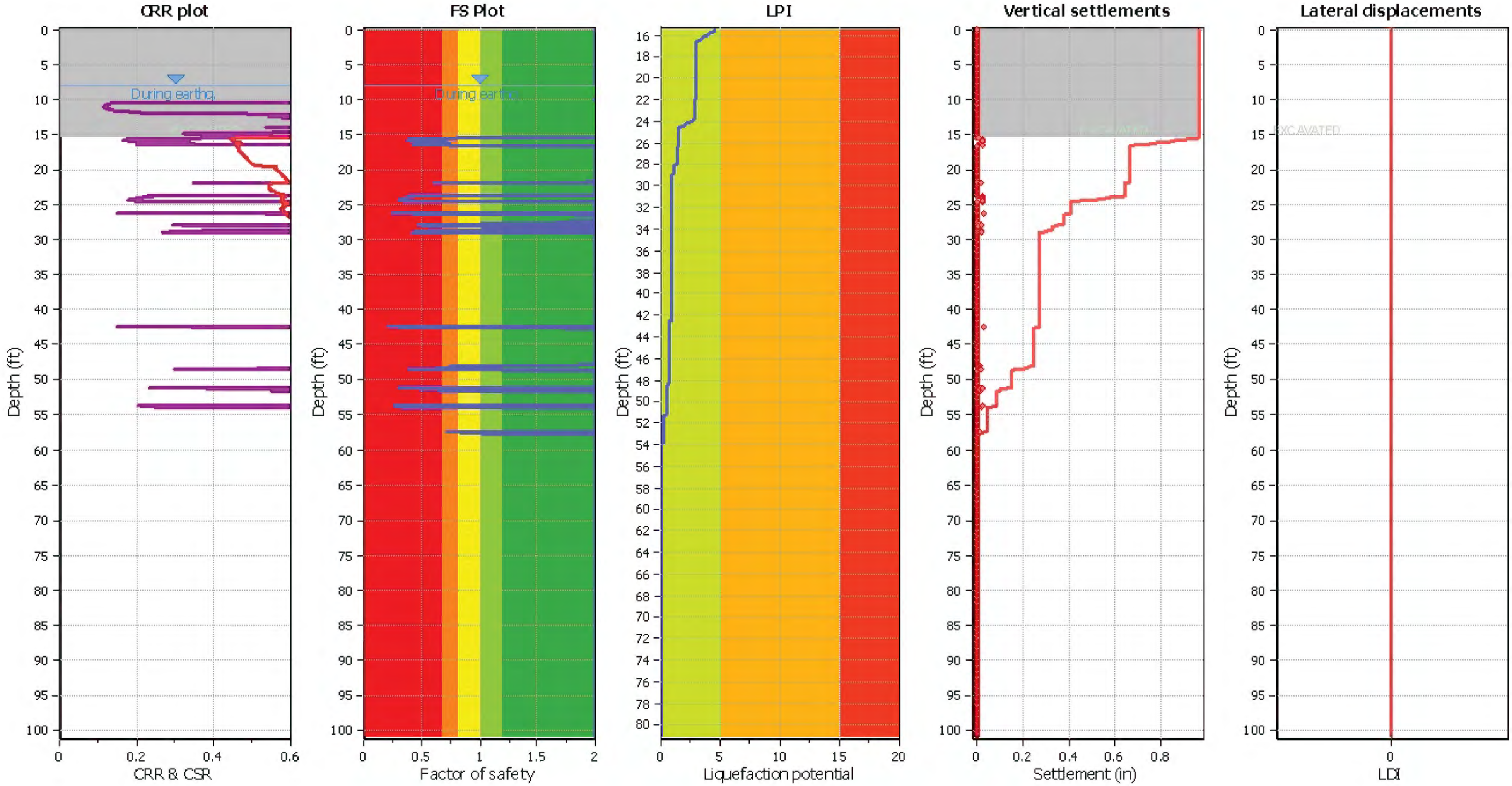
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Footing load:	0.75 tsf
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.67	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Excavation depth:	15.50 ft	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Footing load:	0.75 tsf
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.67	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Excavation depth:	15.50 ft	Limit depth:	60.00 ft

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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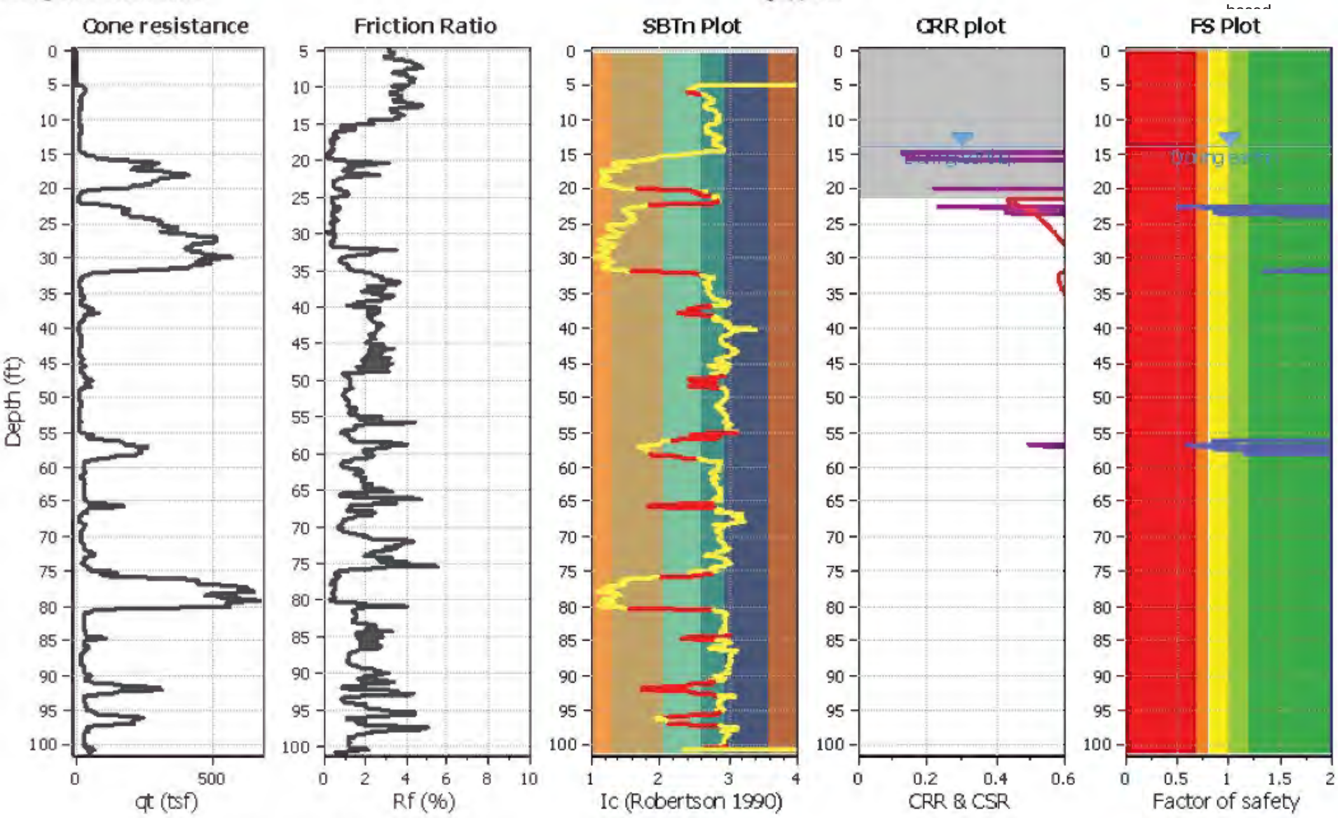
LIQUEFACTION ANALYSIS REPORT

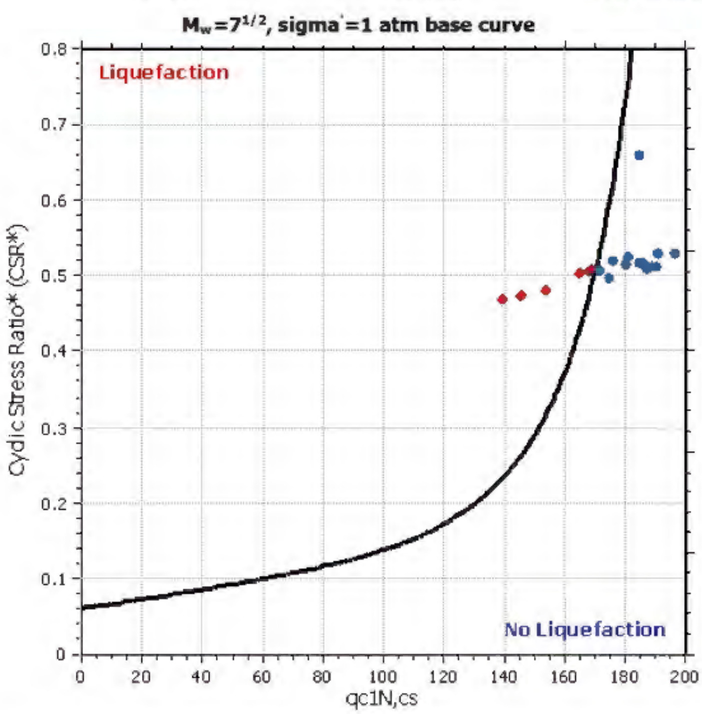
Project title : East Whisman Phase 1 **Location : Mountain View, CA**

CPT file : 1-CPT02

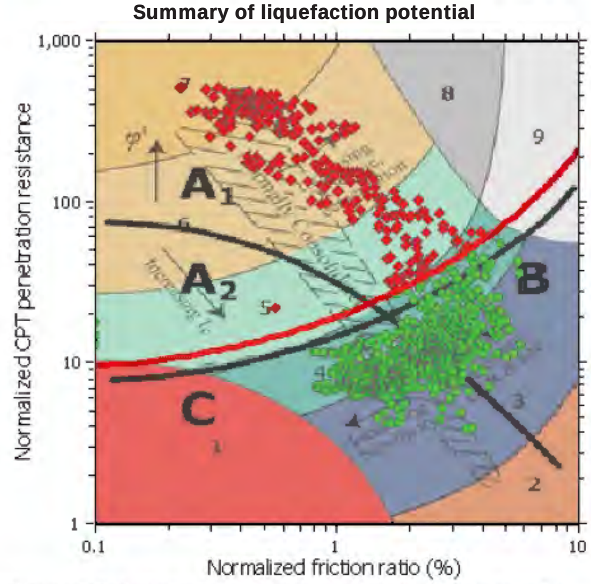
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	14.00 ft	Excavation:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	14.00 ft	Excavation depth:	21.50 ft	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.75 tsf	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.67	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



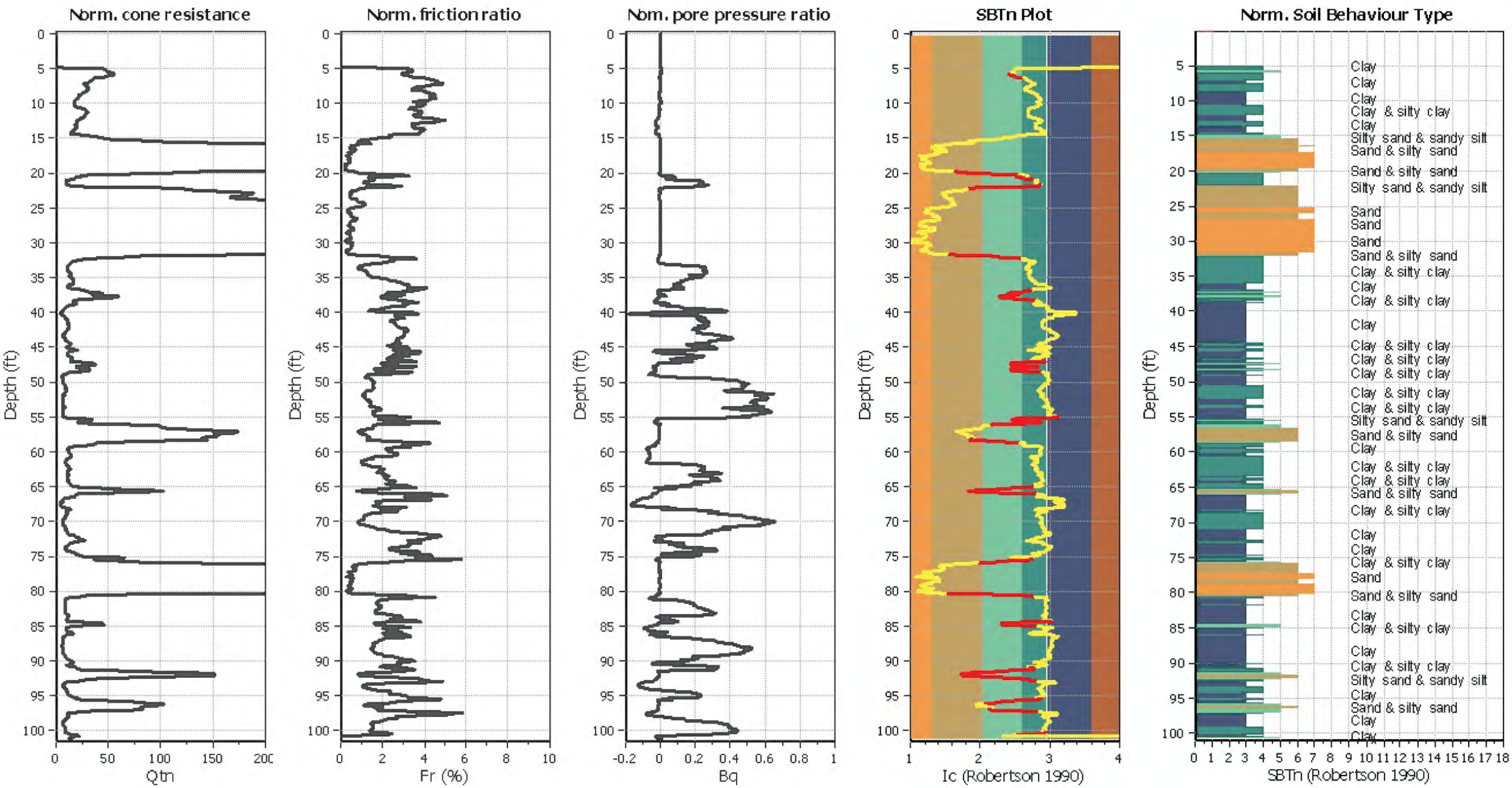


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



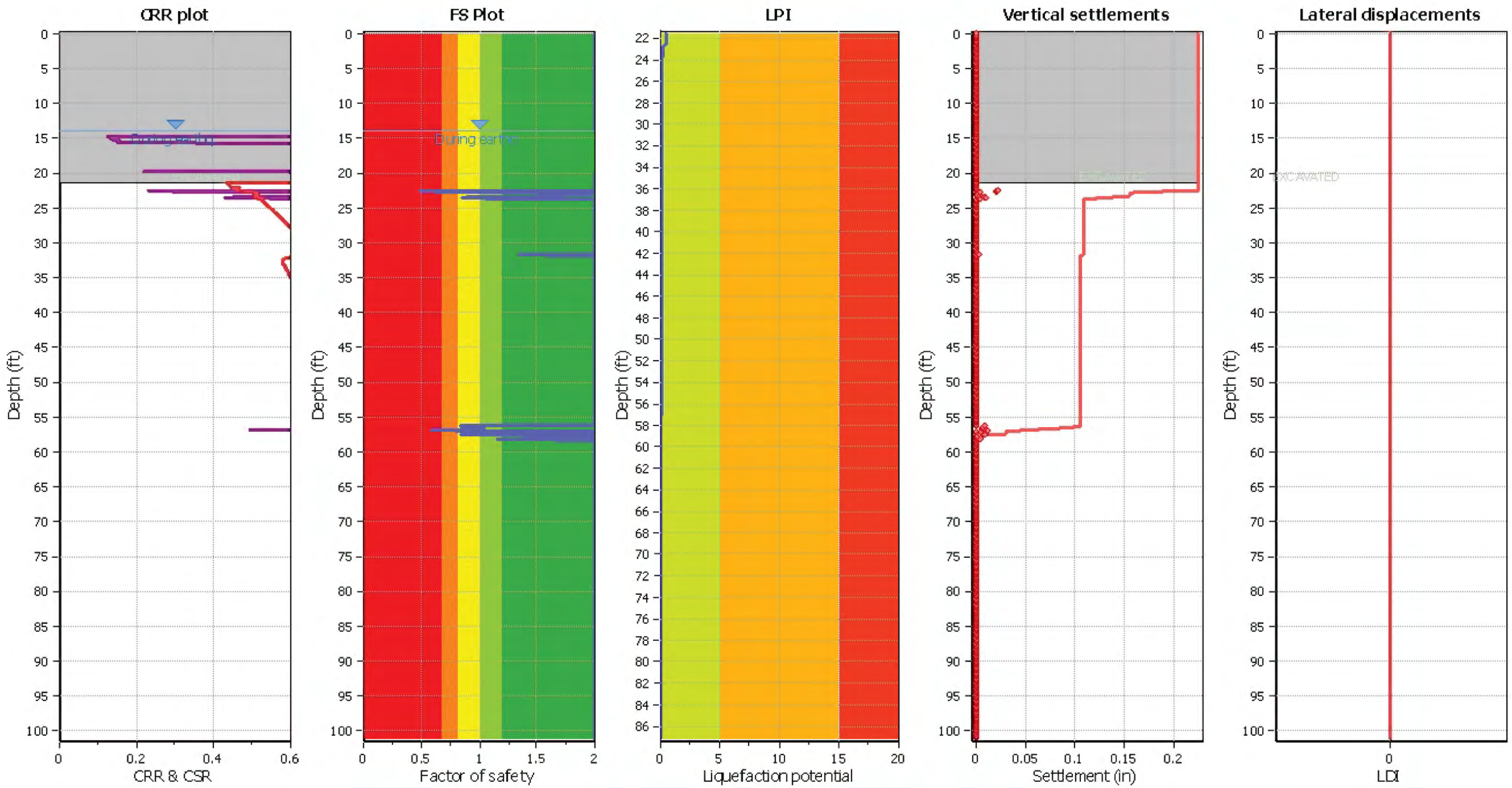
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.00 ft	Footing load:	0.75 tsf
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.67	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	14.00 ft	Excavation depth:	21.50 ft	Limit depth:	60.00 ft

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.00 ft	Footing load:	0.75 tsf
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.67	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	14.00 ft	Excavation depth:	21.50 ft	Limit depth:	60.00 ft

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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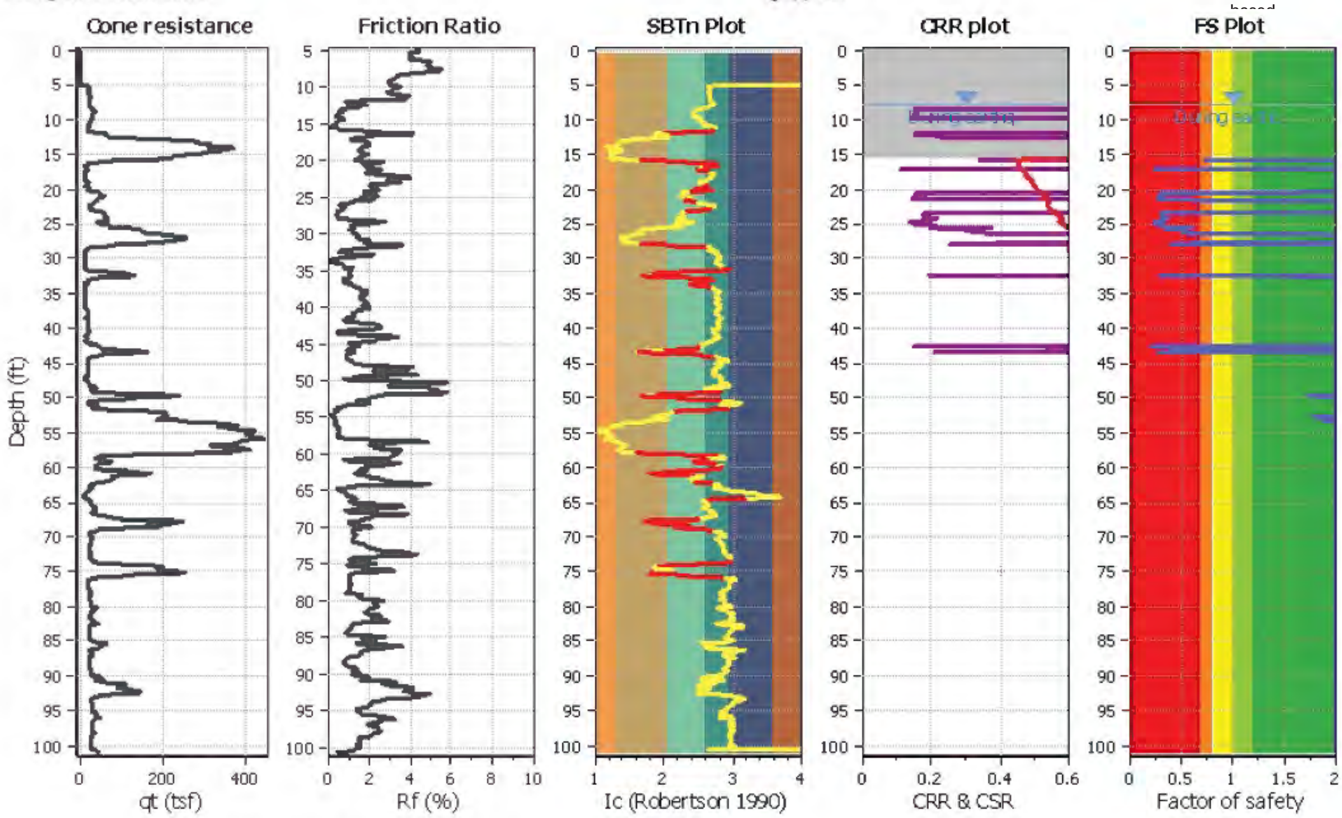
LIQUEFACTION ANALYSIS REPORT

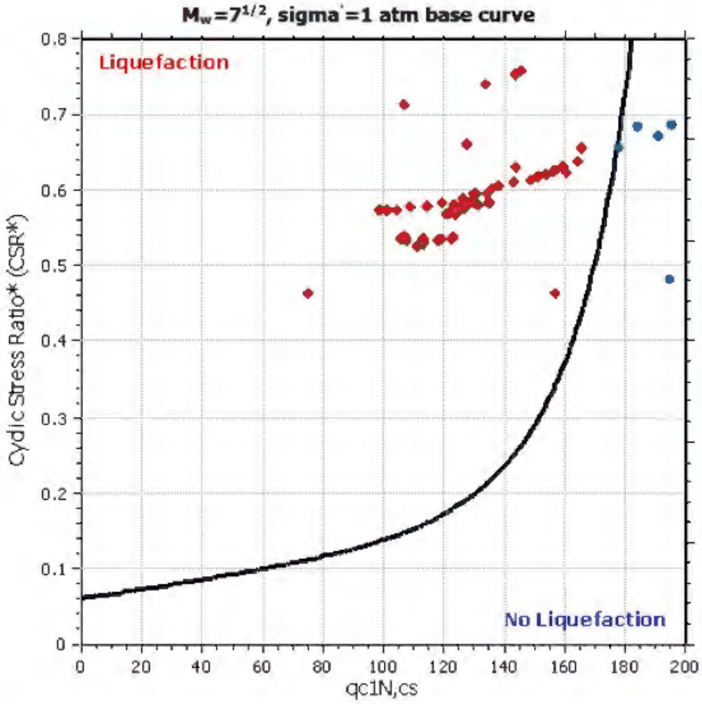
Project title : East Whisman Phase 1 **Location : Mountain View, CA**

CPT file : 1-CPT03

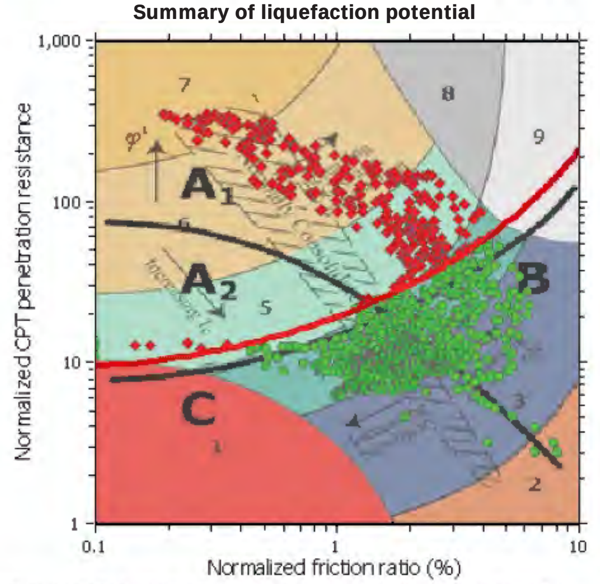
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.00 ft	Excavation:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 ft	Excavation depth:	15.50 ft	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.75 tsf	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.67	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



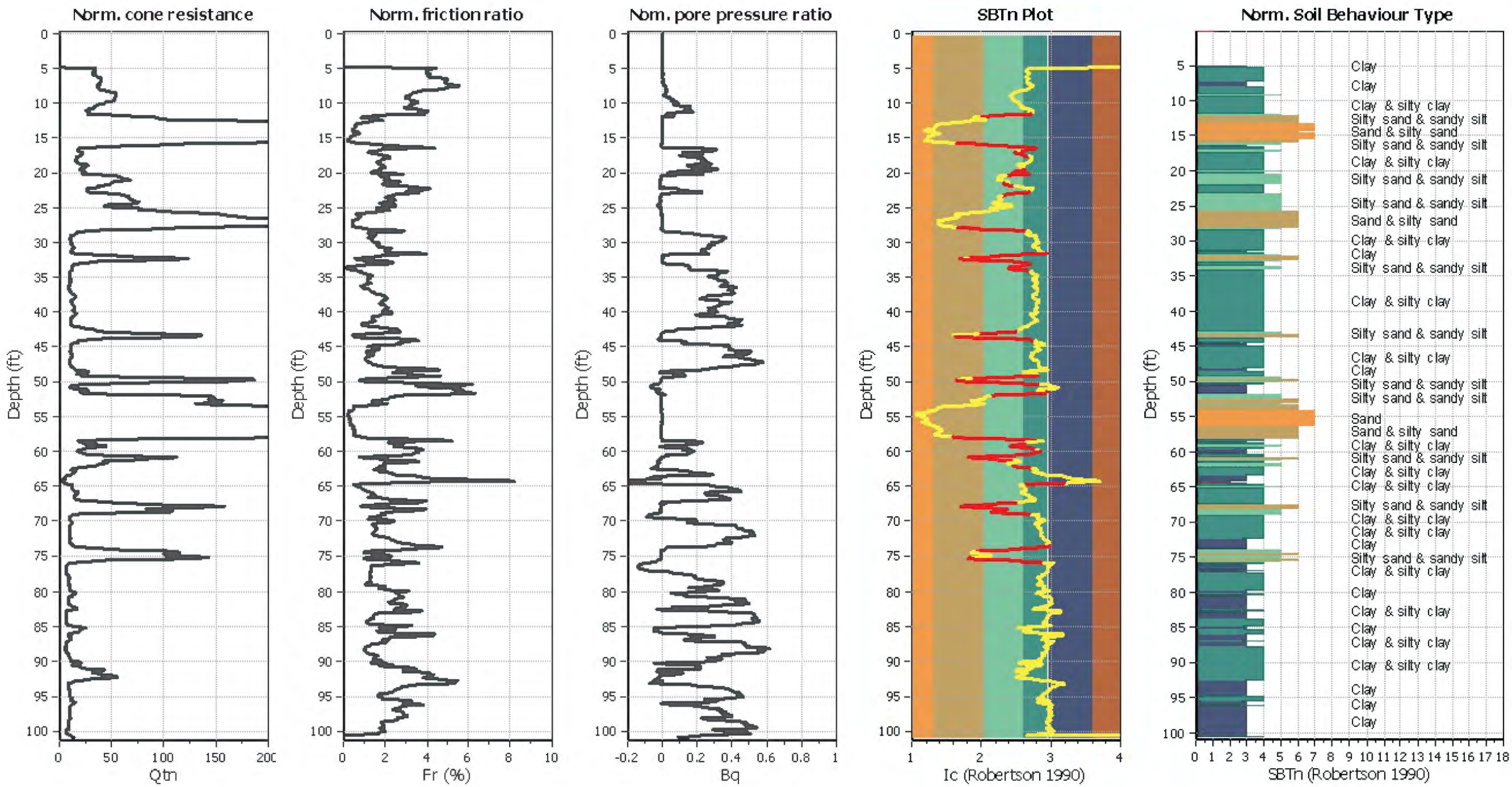


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



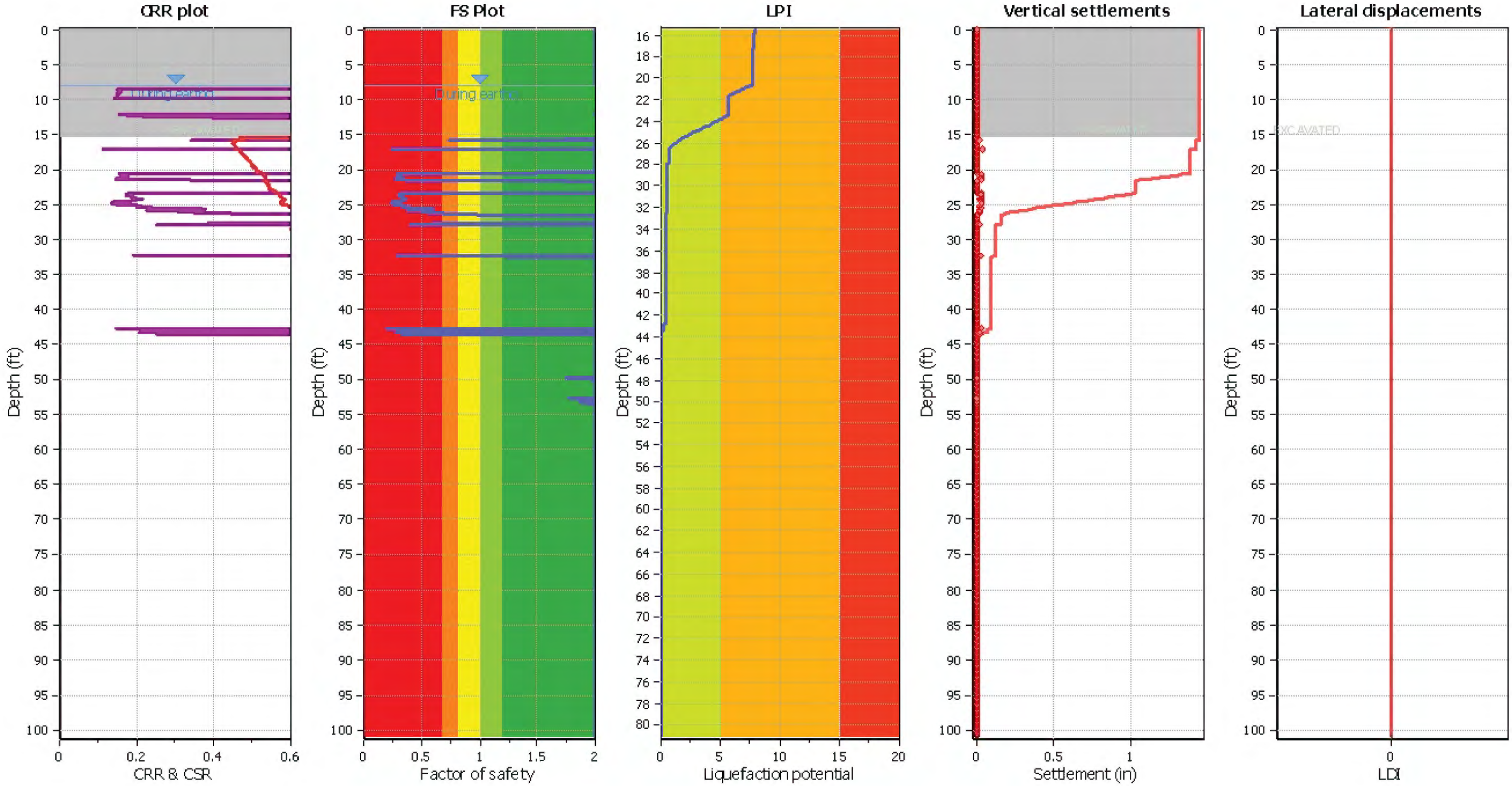
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Footing load:	0.75 tsf
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.67	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Excavation depth:	15.50 ft	Limit depth:	60.00 ft

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Footing load:	0.75 tsf
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.67	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Excavation depth:	15.50 ft	Limit depth:	60.00 ft

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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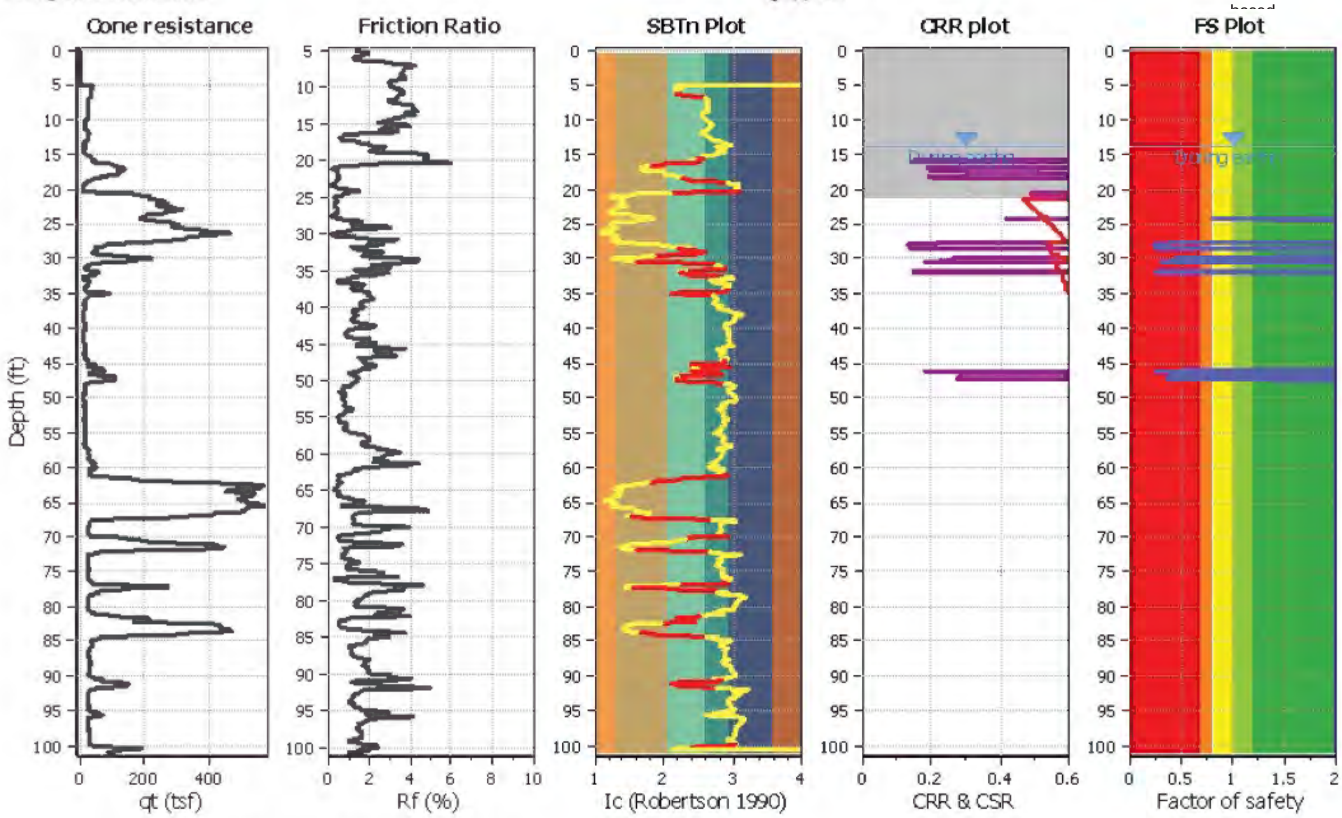
LIQUEFACTION ANALYSIS REPORT

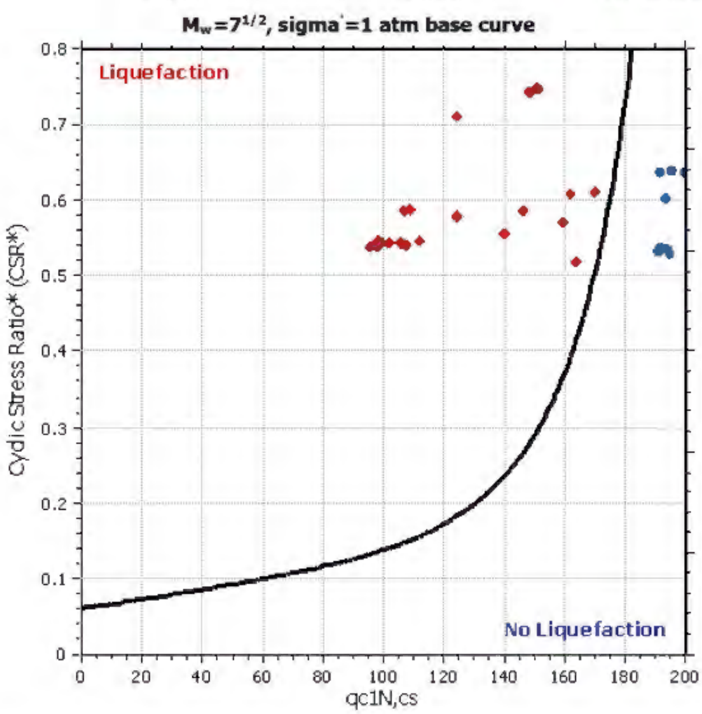
Project title : East Whisman Phase 1 **Location : Mountain View, CA**

CPT file : 1-CPT04

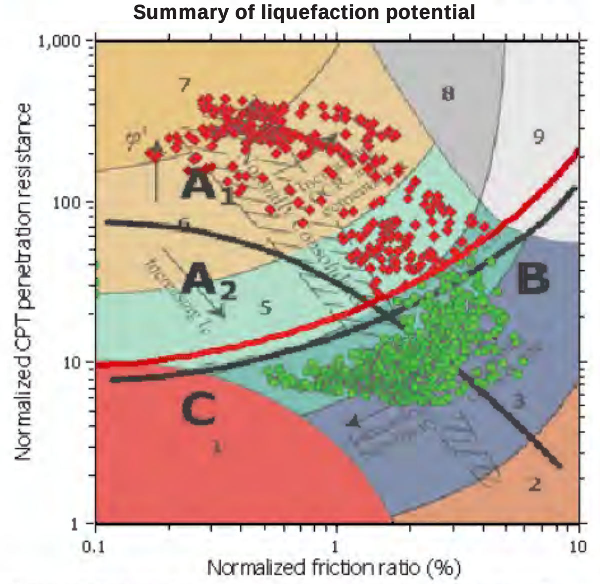
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	14.00 ft	Excavation:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	14.00 ft	Excavation depth:	21.50 ft	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.75 tsf	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.67	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



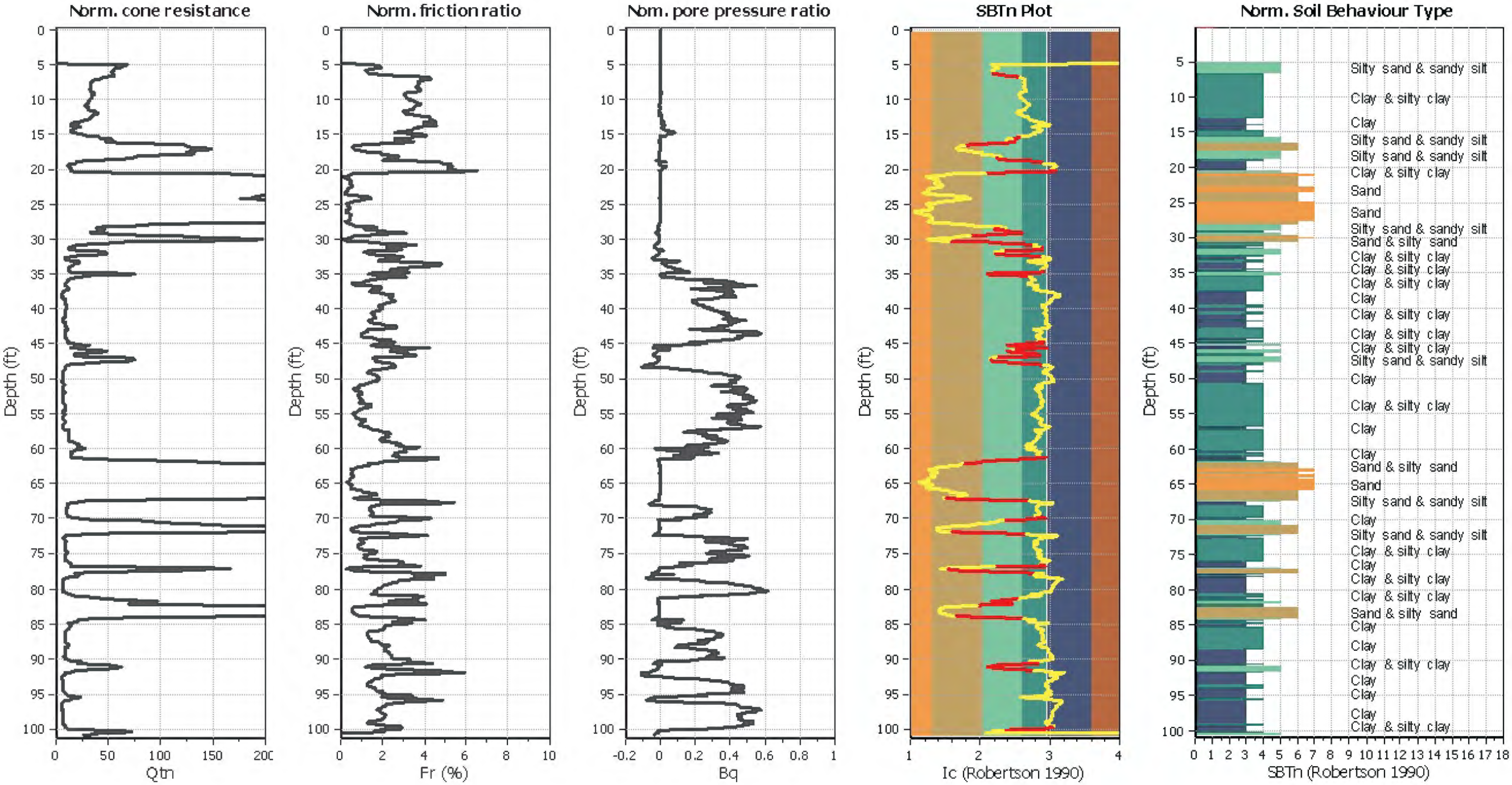


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



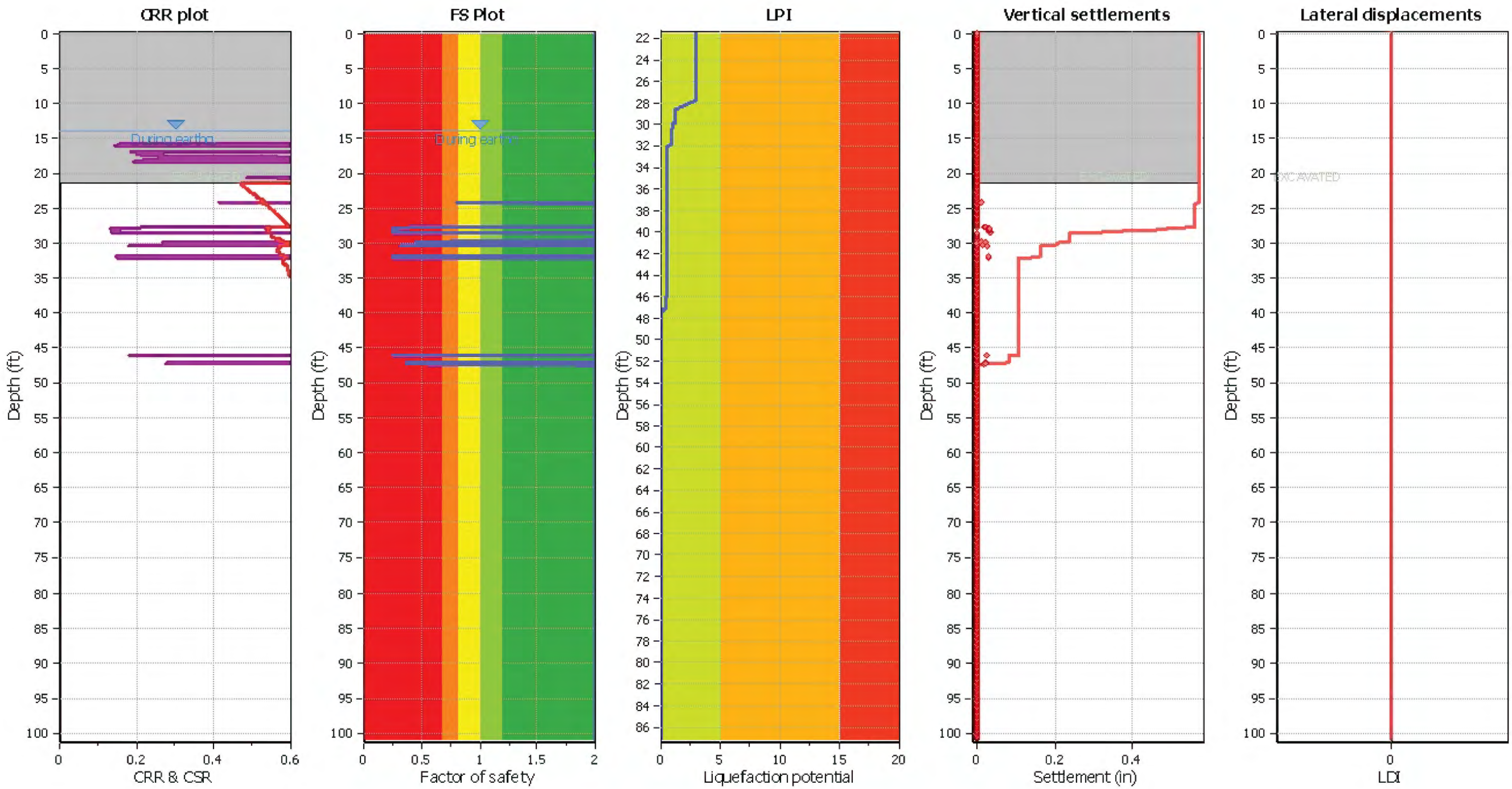
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.00 ft	Footing load:	0.75 tsf
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.67	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	14.00 ft	Excavation depth:	21.50 ft	Limit depth:	60.00 ft

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.00 ft	Footing load:	0.75 tsf
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.67	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	14.00 ft	Excavation depth:	21.50 ft	Limit depth:	60.00 ft

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

APPENDIX F

CORROSIVITY TEST RESULTS BY SUNLAND ANALYTICAL

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 12/02/2020
Date Submitted 11/25/2020

To: Bofei Xu
Engeo, Inc.
2010 Crow Canyon PL. Ste #250
San Ramon, CA 94583

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : EAST WHISMAN Site ID : 1-B02@11.
Thank you for your business.

* For future reference to this analysis please use SUN # 83556-174325.

EVALUATION FOR SOIL CORROSION

Soil pH	7.34	
Moisture	14.3 %	
Minimum Resistivity	1.80 ohm-cm (x1000)	
Chloride	6.7 ppm	00.00067 %
Sulfate	34.1 ppm	00.00341 %
Redox Potential	(+) 219 mv	
Sulfides	Presence - NEGATIVE	

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m
Redox Potential ASTM G-200m, Sulfides AWWA C105/A25.5

Sunland Analytical
11419 Sunrise Gold Circle. #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 12/02/2020
Date Submitted 11/25/2020

To: Bofei Xu
Engeo, Inc.
2010 Crow Canyon PL. Ste #250
San Ramon, CA 94583

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : EAST WHISMAN Site ID : 1-B3@51.
Thank you for your business.

* For future reference to this analysis please use SUN # 83556-174326.

EVALUATION FOR SOIL CORROSION

Soil pH	7.73		
Moisture	16.9 %		
Minimum Resistivity	1.10 ohm-cm (x1000)		
Chloride	9.6 ppm	00.00096 %	
Sulfate	80.9 ppm	00.00809 %	
Redox Potential	(+) 82 mv		
Sulfides	Presence - NEGATIVE		

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m
Redox Potential ASTM G-200m, Sulfides AWWA C105/A25.5

APPENDIX G

THERMAL CONDUCTIVITY TEST RESULTS BY AIR CONNECTION

AIR CONNECTION

500 E Middlefield Road

Mountain View CA

Geothermal Test Bore Documentation

Contractor: ENGEO Incorporated

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Proposed Borings, Preliminary Study, Additional Scope
Proposed CPTs, Preliminary Study
Proposed CPTs, Preliminary Study, Additional Scope
Proposed Geothermal Boring, Preliminary Study
1-CPT01 (100')
1-CPT03 (100')
1-B02 (60')
B-6 (44.5') N&M 2019
CPT-11 (80.1') N&M 2019
1-SCPT02 (100')
1-B01 (60')
1-GHXB01 (100')
1-GHXB01 (alternative location)
1-CPT04 (100')
1-B03 (100')
CPT-12 (101.2') N&M 2019
1-CPT05 (100')
Ellis St
E Middlefield Rd
W Maude Ave
Logue Ave
N
500 ft
Google Earth
© 2020 Google

Formation Thermal Conductivity
Test & Data Analysis

Test location **East Whisman Phase 1**
Mountain View, CA

Test Date November 30 – December 4, 2020

Analysis For Air Connection
1375 Central Ave.
Santa Rosa, CA 95401
Phone: (707) 571-8384

Test Performed By Air Connection

Executive Summary

A formation thermal conductivity test was performed on the geothermal bore with a GPS location of N 37.396749°, W 122.052878° at the East Whisman Phase 1 site in Mountain View, California. The vertical bore was completed on November 18, 2020 by Pitcher Drilling. Geothermal Resource Technologies' (GRTI) test unit was attached to the vertical bore on the morning of November 30, 2020.

This report provides an overview of the test procedures and analysis process, along with plots of the loop temperature and input heat rate data. The collected data was analyzed using the "line source" method and the following average formation thermal conductivity was determined.

Formation Thermal Conductivity = 1.00 Btu/hr-ft-°F

Due to the necessity of a thermal diffusivity value in the design calculation process, an estimate of the average thermal diffusivity was made for the encountered formation.

Formation Thermal Diffusivity ≈ 0.70 ft^2/day

The undisturbed formation temperature for the tested bore was established from the initial loop temperature data collected at startup.

Undisturbed Formation Temperature ≈ 66.1°F

The formation thermal properties determined by this test do not directly translate into a loop length requirement (i.e. feet of bore per ton). These parameters, along with many others, are inputs to commercially available loop-field design software to determine the required loop length. Additional questions concerning the use of these results are discussed in the frequently asked question (FAQ) section at www.grti.com.

TEST PROCEDURES

The American Society of Heating, Refrigeration, and Air-Conditioning Engineers (ASHRAE) has published recommended procedures for performing formation thermal conductivity tests in the ASHRAE HVAC Applications Handbook, Geothermal Energy Chapter. The International Ground Source Heat Pump Association (IGSHPA) also lists test procedures in their Design and Installation Standards. GRTI's test procedures meet or exceed those recommended by ASHRAE and IGSHPA, with the specific procedures described below:

Grouting Procedure for Test Loops – To ensure against bridging and voids, it is recommended that the bore annulus is uniformly grouted from the bottom to the top via tremie pipe.

Time Between Loop Installation and Testing – A minimum delay of five days between loop installation and test startup is recommended for bores that are air drilled, and a minimum waiting period of two days for mud rotary drilling.

Undisturbed Formation Temperature Measurement – The undisturbed formation temperature should be determined by recording the loop temperature as the water returns from the u-bend at test startup.

Required Test Duration – A minimum test duration of 36 hours is recommended, with a preference toward 48 hours**.**

Data Acquisition Frequency - Test data is recorded at five minute intervals.

Equipment Calibration/Accuracy – Transducers and datalogger are calibrated per manufacturer recommendations. Manufacturer stated accuracy of power transducers is less than ±2%. Temperature sensor accuracy is periodically checked via ice water bath.

Power Quality – The standard deviation of the power should be less than or equal to 1.5% of the average power, with maximum power variation of less than or equal to 10% of the average power.

Input Heat Rate – The heat flux rate should be 51 Btu/hr (15 W) to 85 Btu/hr (25 W) per foot of installed bore depth to best simulate the expected peak loads on the u-bend.

Insulation – GRTI's equipment has 1 inch of foam insulation on the FTC unit and 1/2 inch of insulation on the hose kit connection. An additional 2 inches of insulation is provided for both the FTC unit and loop connections by insulating blankets.

Retesting in the Event of Failure – In the event that a test fails prematurely, a retest may not be performed until the bore temperature is within 0.5°F of the original undisturbed formation temperature or until a period of 14 days has elapsed.

Data Analysis

Geothermal Resource Technologies, Inc. (GRTI) uses the "line source" method of data analysis to determine the thermal conductivity of the formation. The line source method assumes an infinitely thin line source of heat in a continuous medium. A plot of the late-time temperature rise of the line source temperature versus the natural log of elapsed time will follow a linear trend. The linear slope is inversely proportional to the thermal conductivity of the medium. Applying the line source method to a u-bend grouted in a borehole, the test must be run long enough to allow the finite dimensions of the u-bend pipes and the grout to become insignificant. Experience has shown that approximately ten hours is required to allow the error of early test times and the effects of finite borehole dimensions to become insignificant.

In the analysis of the data from the formation thermal conductivity test, the average temperature of the water entering and exiting the u-bend heat exchanger was plotted versus the natural log of elapsed testing time. Using the Method of Least Squares, linear coefficients were calculated that produce a line that fit the data. This procedure was repeated for various time intervals to ensure that variations in the power or other effects did not produce inaccurate results.

The calculated results are based on test bore information submitted by the driller/testing agency. GRTI is not responsible for inaccuracies in the results due to erroneous bore information. All data analysis is performed by personnel that have an engineering degree from an accredited university with a background in heat transfer and experience with line source theory. The test results apply specifically to the tested bore. Additional bores at the site may have significantly different results depending upon variations in geology and hydrology.

Through the analysis process, the collected raw data is converted to spreadsheet format (Microsoft Excel®) for final analysis. If desired, please contact GRTI and a copy of the data will be made available in either a hard copy or electronic format.

Contact: Galen Streich
Regional Managing Engineer
Elkton, SD
Ph: 866-991-4784
gstreich@grti.com

Test Bore Details
(As Provided by Pitcher Drilling)

Site Name	East Whisman Phase 1
Location	Mountain View, CA
Driller	Pitcher Drilling
Installed Date	November 18, 2020
Borehole Diameter	5 inches
U-Bend Size	1 inch HDPE
U-Bend Depth Below Grade	103 ft
Grout Type	Wyo-Ben Therm-Ex
Grout Mixture	200 lb sand per 50 lb bentonite
Grouted Portion	Entire bore

Drill Log

Formation Description	Depth (ft)
Asphalt, aggregate base	0-3"
Fat clay	3"-4'
Sandy lean clay	4'-15.5'
Poorly graded sand with gravel	15.5'-30.5'
Lean clay	30.5'-39'
Fat clay	39'-44'
Silty sand	44'-49'
Fat clay with sand	49'-56'
Poorly graded gravel	56'-59'
Silt	59'-69'
Sandy lean clay	69'-76.5'
Poorly graded gravel	76.5'-80'
Lean clay	80'-84'
Poorly graded sand with gravel	84'-86.5'
Lean clay	86.5'-92'
Sandy clay	92'-96'
Sand and gravel	96'-97.5'
Clay	97.5'-103'

Thermal Conductivity Test Data

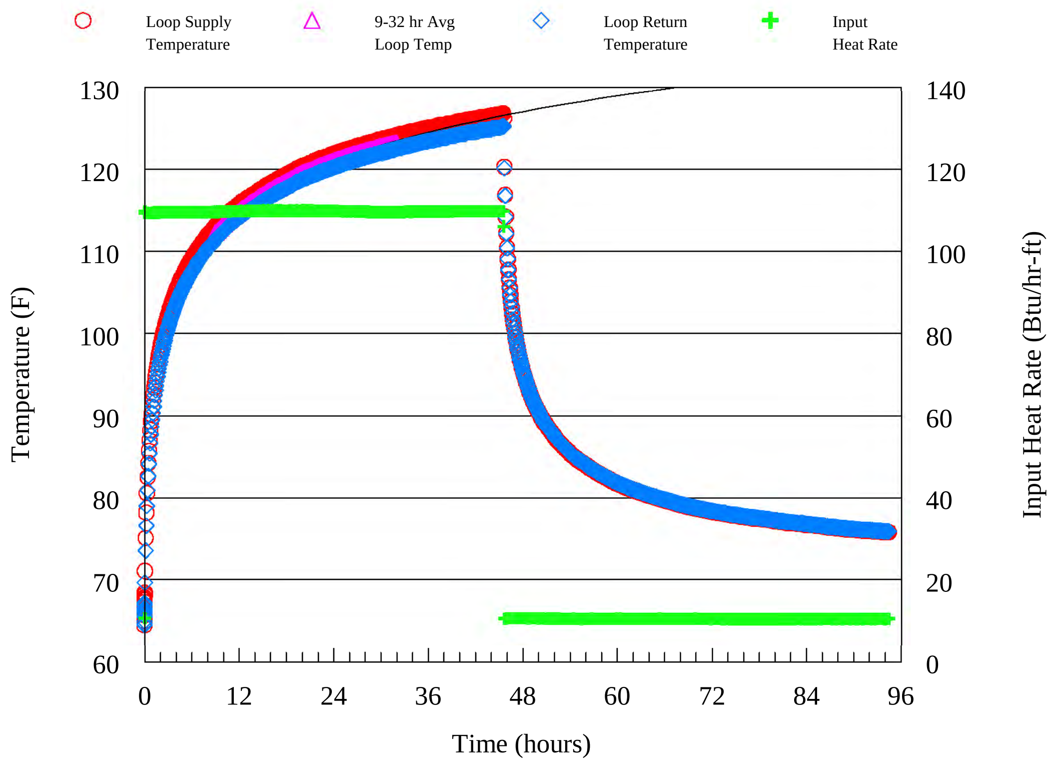

Fig. 1: Temperature & Heat Rate Data Vs Time

Figure 1 above shows the loop temperature and heat input rate data versus the elapsed time of the test. The temperature of the fluid supplied to and returning from the U-bend are plotted on the left axis, while the amount of heat supplied to the fluid is plotted on the right axis on a per foot of bore basis. In the test statistics below, calculations on the power data were performed over the analysis time period listed in the Line Source Data Analysis section.

Summary Test Statistics

Test Date	November 30 – December 4, 2020
Undisturbed Formation Temperature	Approx. 66.1°F
Heating Duration	45.5 hr
Average Voltage	240.9 V
Average Heat Input Rate	11,305 Btu/hr (3,313 W)
Avg Heat Input Rate per Foot of Bore	109.8 Btu/hr-ft (32.2 W/ft)
Circulator Flow Rate	14.2 gpm
Standard Deviation of Power	0.11%
Maximum Variation in Power	0.24%

LINE SOURCE DATA ANALYSIS

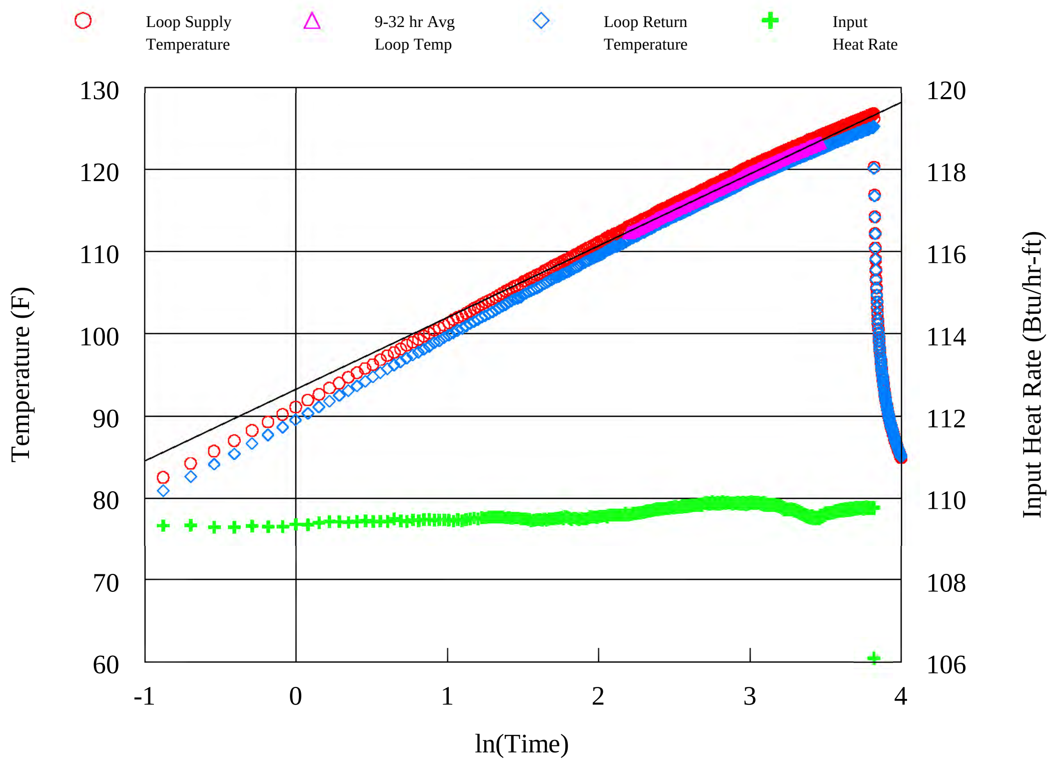

FIG. 2: TEMPERATURE & HEAT RATE VS NATURAL LOG OF TIME

The loop temperature and input heat rate data versus the natural log of elapsed time are shown above in Figure 2. The temperature versus time data was analyzed using the line source method (see page 3) in conformity with ASHRAE and IGSHPA guidelines. A linear curve fit was applied to the average of the supply and return loop temperature data between 9 and 32.0 hours. The slope of the curve fit was found to be 8.73. The resulting thermal conductivity was found to be **1.00 Btu/hr-ft-°F**.

Thermal Diffusivity

The reported drilling log for this test borehole indicated that the formation consisted of clay, silt, sand and gravel. A weighted average of heat capacity values based on the indicated formation was used to determine an average heat capacity of 34.4 Btu/ft^3-°F for the formation. A diffusivity value was then found using the calculated formation thermal conductivity and the estimated heat capacity. The thermal diffusivity for this formation was estimated to be **0.70 ft^2/day**.

CERTIFICATE OF CALIBRATION

GRTI maintains calibration of the datalogger, current transducer and voltage transducer on a regular schedule. The components are calibrated by the manufacturer using recognized national or international measurement standards such as those maintained by the National Institute of Standards and Technology (NIST).

FTC Unit 201

DA Unit 70

Primary Equipment		
Component	Calibration Date	Calibration Due Date
Datalogger	7/20/2018	7/20/2021
Current Transducer	7/23/2018	7/23/2021
Voltage Transducer	7/23/2018	7/23/2021

GRTI periodically verifies the combined temperature sensor/datalogger accuracy via a water bath. Temperature readings are simultaneously taken with a digital thermometer that has been calibrated using instruments traceable to NIST.

Date	9/21/2020			
Thermocouple 1 (°F)	32.1 32.1 32.1			
Thermocouple 2 (°F)	32.1 32.0 32.1			
Thermocouple 3 (°F)	32.1 32.1 32.1			
Thermocouple 4 (°F)	32.2 32.2 32.2			
Digital Thermometer (°F)	32.3 32.2 32.2			


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