

# Appendix GEO

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Geotechnical Investigation and Geological Hazards Study

GEOTECHNICAL-INVESTIGATION AND GEOLOGIC-HAZARDS STUDY  
MORAGA ROAD STORAGE  
2600 MORAGA ROAD STREET  
SAN PABLO, CALIFORNIA



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August 28, 2020  
2388-5A, L-32151

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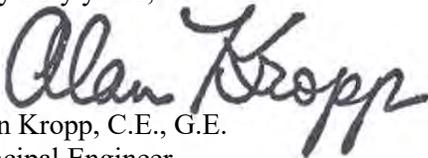
RE: Geotechnical-Studies and Fault-Hazard Investigation  
Moraga Road Storage  
San Pablo, California

Dear Mr. Thom:

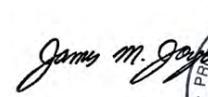
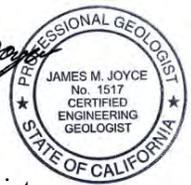
Attached is our geotechnical-investigation and geologic-hazards-study report for the planned Moraga Road Storage Facility located at the former El Portal Elementary School campus in San Pablo, California. Our services on this project are being provided in general accordance with our contract with you dated May 8, 2020.

If you have any questions regarded the report, please do not hesitate to call us.

Very truly yours,



Alan Kropp, C.E., G.E.  
Principal Engineer

James Joyce, R.G., C.E.G.  
Certified Engineering Geologist



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MJV/ab

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2388-5A Moraga Rd Storage Letter of Transmittal

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## 1.00 INTRODUCTION

This report presents the results of our geotechnical investigation and geologic-hazard study for the planned Moraga Road Storage Facility to be constructed at the former El Portal Elementary School campus in San Pablo, California. Our services on this project are being provided in accordance with our contract agreement with you dated May 8, 2020.

### 1.01 Site Location

The project site is located in western Contra Costa County on a broad, relatively level area bordering San Francisco Bay. The site address is 2600 Moraga Road in San Pablo, as indicated on the Site Location Map, Figure 1. Approximate site coordinates (latitude and longitude) for this location are:

| <b>Latitude</b> | <b>Longitude</b> |
|-----------------|------------------|
| 37.9673°        | -122.3377°       |

The site is situated within the Richmond USGS 7-½ Minute Quadrangle.

### 1.02 Project Description

Based on our review of conceptual-design documents and discussions with you, the proposed project will consist of self-storage buildings (up to three-story) and a small office. The project will also include asphalt concrete (AC) paving and other appurtenant structures. With the exception of the office, we do not consider the other structures to be “structures for human occupancy,” as defined in the Alquist-Priolo Act (APA). For the purposes of this report, we will assume that the planned buildings will be designed using light, wood-frame construction.

### 1.03 Applicable Regulations Pertaining to Geologic Hazards

The subject property is within the Alquist-Priolo Earthquake Fault Zone (APEFZ) established by the State of California around the Hayward-Rodgers Creek fault. The APA restricts the placement of structures for human occupancy, such as commercial or residential buildings, near active fault traces. Other facilities, such as non-occupied buildings, roads, utilities, or parking areas, are not subject to this restriction. A structure for human occupancy is defined in the APA as a structure that is occupied a minimum of 2,000 man/hours per year.

### 1.04 Purpose and Scope of Services

The purpose of our geotechnical-engineering investigation and geologic-hazards assessment was to identify geo-hazards and geotechnical concerns that may be present at the site and to provide geotechnical-design and construction recommendations for the project. The scope of our investigation included the following tasks:

- Review of relevant published geologic maps and data;
- Review of historic aerial photographs;
- Geologic/geotechnical site-reconnaissance visits;
- Drilling of four exploratory conventional borings at the site;

- Advancing of nine cone-penetrometer-test (CPT) soundings;
- Performing geotechnical laboratory tests on samples recovered from the borings;
- Excavation of one exploratory trench approximately 280 feet in length;
- Geologic logging of the trench by our Certified Engineering Geologist;
- Performing geotechnical- and geological-engineering analyses, including a CPT-based liquefaction analysis; and
- Preparing this combined geotechnical-investigation and geologic-hazards-study report.

The scope of our services did not include an environmental assessment or investigation for the presence of hazardous or toxic materials in the soil, groundwater, or air on, below, or around this site. An evaluation of the potential presence of sulfates in the soil, or other possibly corrosive, naturally occurring elements, was beyond our scope.

## **2.00 METHODS OF INVESTIGATION**

### **2.01 Review of Existing Data**

#### **2.01.1 Published Geologic References**

We reviewed published geologic maps and literature pertaining to geologic and seismic conditions in the project vicinity. These references included regional geologic maps, regional and localized fault maps, liquefaction-hazard maps, landslide maps, and literature documenting recorded incidents of earthquake-induced liquefaction and/or ground failures. Information obtained from the map and literature review is discussed in Section 3.00, Geologic, Seismic, and Historic Setting. A list of selected references is presented in Section 9.00, References.

#### **2.01.2 Consultant Reports**

We reviewed geotechnical and geohazard reports prepared by other consultants for nearby properties. Information obtained from the consultant-report review is discussed in Section 3.02.2, Review of Consultant Reports. A list of selected references is presented in Section 9.00, References.

#### **2.01.3 Aerial Photographs**

We reviewed eight sets of black-and-white, stereo-paired aerial photographs of the site as part of our study. These photographs were taken during the years 1939, 1947, 1949, 1953, 1957, 1959, 1971, and 1975 and ranged in scale from 1:9,600 to about 1:20,000. Observations from our review of the aerial photographs are summarized in Section 3.04, Review of Aerial Photographs. A complete listing of the photographs used in our study is included in Section 9.00, References.

## 2.02 Field Investigation

### 2.02.1 Geologic Reconnaissance

On July 15, 2020, we conducted a surface reconnaissance of the site to observe and map geologic conditions and check for signs of settlement or damage indicative of possible geotechnical concerns. The results of the geologic reconnaissance are discussed in Section 4.02, Surface Conditions.

### 2.02.2 Subsurface Exploration (Conventional Borings)

On June 22, 2020, we drilled four conventional exploratory borings (designated as B-1 through B-4) to evaluate the subsurface conditions of the site at the locations shown on the Site Plan, Figure 2. All four borings were drilled using a truck-mounted, solid-flight auger rig. The drilling was observed and recorded by our field geologist, who logged the soil cuttings, made notes of any changes in drilling conditions, and collected representative samples of the material encountered. Soil samples were obtained using 2-inch O.D. Standard Penetration Test (SPT) and 3-inch O.D. California Modified samplers. The samplers were driven by a standard automatic-trip, 140-pound hammer, with a 30-inch fall. The blows required to drive the sampler the final 12 inches of each 18-inch drive are presented on the boring logs. Blow counts presented on the boring logs are not converted to standard penetration resistance values ( $N_{60}$ ).

Following the field operations, the borings were inspected and grouted in accordance with Contra Costa County Environmental Health Department permit requirements.

The logs of materials encountered in the borings are attached in Appendix A. Soils were classified in general accordance with ASTM D2488, which is based on the Unified Soil Classification System (USCS). The USCS is described on the Key to Exploratory Boring Logs, Figure A-1, and also presented in Appendix A.

We note that the attached boring logs depict subsurface conditions only at the approximate locations shown on the Site Plan, Figure 2, on the particular date designated on the logs. The attached logs represent our current interpretation of the subsurface materials at the boring locations, and the passage of time may result in changes in the subsurface conditions. The boring locations indicated on the attached materials were determined by measuring from existing surface features and should be considered approximate.

### 2.02.3 Subsurface Exploration (CPTs)

On June 11 and 12, 2020, we advanced nine CPT soundings at the approximate locations shown on the Site Plan (Figure 2). The CPTs were advanced using a ConeTec, 30-ton, truck-mounted CPT rig and a 20-ton track-mounted CPT rig. The CPTs were located to obtain a relatively representative image of the subsurface conditions. The CPTs, designated C-1 through C-9, were advanced to depths of ranging from about 45 feet to about 52 feet below the existing site grades. ConeTec Industries, Inc. (ConeTec) of San Leandro, California performed the CPT soundings, under the observation of Mr. Jeroen van den Berg, C.E., Senior Engineer for AKA.

The CPT method involves pushing a 2-inch-diameter, conical probe into the ground using a hydraulic ram system. The CPT probe is equipped with sensors that produce a continuous record of tip resistance, sleeve friction, and pore pressure as the cone is advanced. During cone advancement, real-time data obtained from the cone sensors are displayed on a computer monitor, together with preliminary interpretive logs of soil type.

During cone advancement, Mr. van den Berg monitored the real-time cone data and determined the depths to which the probes would extend.

#### 2.02.3.1 CPT Seismic Shear-Wave Velocities

In addition to estimating interpretive soil parameters, the CPT cones were equipped with geophones and a seismic source to estimate soils and rock shear-wave velocities. During advancement of C-5, seismic shear-wave velocities were recorded at about every meter to the final depth of the cone advancement.

#### 2.02.3.2 CPT Pore Pressure Dissipation Tests

Each of the CPT soundings also recorded the approximate groundwater level using the onboard pore-pressure measurement sensors. The calculated groundwater depths are discussed below in Section 4.03.4, Groundwater, and in Appendix B, CPT Report.

ConeTec's interpretive logs of the CPT soundings are attached in Appendix B. Also included in Appendix B is information obtained from ConeTec describing their equipment and the correlative methods they used in developing their interpretive logs. The attached logs prepared by ConeTec represent their interpretations of the subsurface conditions at the approximate sounding locations indicated on the site plan on the particular date designated on the logs. The sounding location and elevation indicated on the attached materials were determined by measuring from existing fences and buildings shown on the project plans and should be considered approximate.

### 2.03 Fault Trenching

In order to more precisely establish the location of active fault traces within the subject site, we performed an exploratory trench near the southern site margin, as shown on Figure 2. The trench was excavated on July 11–12, 2020, and logging was performed on July 13–15. The trench was loosely backfilled and track-walked on July 18th. The excavation and backfilling operations were performed by Engineered Soil Repairs of Walnut Creek, California. We should note that the trench backfill will need to be re-excavated and properly compacted prior to site development.

The trench ranged in depth from about 8 to 10 feet, and the sides were stepped to provide stability. The entire south wall of the trench was cleaned to remove smeared soil and expose intact soil layers. Portions of the north side of the trench were also cleaned and examined. In general, the native soil layers were wet, as the result of previous irrigation. The trench was left open for 48 hours to allow soils to dry; however, the soils remained fairly wet throughout the three-day period when logging was performed.

### 2.04 Geotechnical Laboratory Testing

Our geotechnical laboratory-testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. The following geotechnical laboratory tests were performed:

- Water content per ASTM Test Designation D-2216;
- Dry density per ASTM Test Designation D-2937;
- Percent passing the No. 200 sieve per ASTM Test Designation D-1140; and
- Atterberg Limits per ASTM Test Designation D-4318.

The tests were conducted in general accordance with the current edition of the referenced standards at the time the tests were performed. The results of the laboratory tests are presented at corresponding depths on the boring logs in Appendix A.

### **3.00 GEOLOGIC, SEISMIC, AND HISTORIC SETTING**

#### **3.01 Regional Geology**

The site is located on a broad alluvial fan north of San Pablo Creek, near the northwest end of San Pablo Ridge (Figure 3). Within the project area, the alluvial fan slopes gently west toward San Pablo Bay. The project vicinity is densely developed.

The area lies within the northern portion of the Coast Ranges geomorphic province of California. Northwest-trending mountain ranges and valleys that generally parallel the major geologic structures, such as the San Andreas and Hayward-Rodgers Creek faults, characterize the region. The oldest widespread rocks in the region are highly deformed sedimentary and volcanic rocks of the Mesozoic-age (the period from 225 million to 65 million years before present) Franciscan Assemblage. These rocks are in fault contact with similar-age sedimentary rocks of the Mesozoic-age Great Valley Sequence. The Mesozoic rocks are, in turn, locally overlain by a diverse sequence of Tertiary-age (the period from 65 million to 1.8 million years before present) sedimentary and volcanic rocks. Since their deposition, the Mesozoic and Tertiary rocks have been extensively deformed by repeated episodes of folding and faulting.

Within the region, many valleys have been partially filled with unconsolidated sedimentary deposits of Quaternary age (the last 1.8 million years). These deposits, which include alluvium and colluvium, underlie the gently sloping valley bottoms and consist of interbedded clay, silt, sand, and gravel. The larger drainages, such as San Pablo Creek, have deposited sediments forming broad alluvial fans along the margins of San Francisco and San Pablo Bays (Figure 3).

#### **3.02 Local Geology**

##### **3.02.1 Review of Published Maps**

All of the published geologic maps reviewed for our study show that the former El Portal Elementary School site lies within a broad, gently sloping area underlain by alluvial sediments (Bishop, et al., 1973; Dibblee, 1980, 2005; Graymer, 2000; Graymer, et al., 1994; Witter, et al., 2006; Wagner et al., 1990). Graymer (2000) indicated that these deposits are of Holocene age (approximately the last 11,000 years). An excerpt of the Graymer geologic map showing the location of the site is included on the attached Figure 3. These deposits typically consist of uncemented clay, silt, and sand. These deposits are estimated by Bishop (1973) to range from less than 100 feet to more than 300 feet in thickness beneath the site, generally deepening from east to

west. Bedrock of the Franciscan Assemblage underlies the alluvium and typically consists of a mixture of sedimentary, metamorphic, and volcanic rocks of Mesozoic age.

Several published geologic maps show various possible locations for the Hayward-Rodgers Creek fault in the vicinity of the subject site. Fault locations shown on these maps are discussed below. It should be noted that these maps were primarily intended for broad geologic characterization and were not intended to determine precise fault locations.

The Alquist-Priolo Earthquake Fault Zone map (CDMG, 1982), which was based on the work of Smith (1980), shows three fault traces in the immediate site vicinity. A portion of that map is shown on Figure 4. Areas of observed creep are shown northwest and southeast of the subject site in the same areas shown by Lienkaemper (Figure 5). Fault traces are shown passing through the central portion of the subject site and near the eastern corner of this site, in general accordance with both Lienkaemper (1992) and Lettis (2003). An additional possible fault trace is shown a short distance west of the southwest corner of the subject site.

Radbruch (1974) prepared a map of the Hayward-Rodgers Creek fault zone. That map shows two traces of the Hayward-Rodgers Creek fault corresponding to the eastern and western traces mapped by CDMG (1982), as shown on Figure 4; however, the central fault trace passing through the middle of the subject site is not shown.

Dibblee (1980, 2005) shows two fault traces generally corresponding to those shown by Radbruch. The site is shown to lie within an area of alluvial sediments. Similar conditions are shown by Wagner et al., (1990).

Herd (1978) mapped a single fault trace corresponding to the documented trace that passes through the central portion of the subject site.

Lienkaemper (1992) prepared a map of active fault traces along the Hayward-Rodgers Creek fault. A portion of that map is reproduced on Figure 5. The Lienkaemper map was based on extensive remapping and aerial-photo interpretation, as well as review of available consultant reports. Lienkaemper shows two active traces of the Hayward-Rodgers Creek fault crossing the subject site (Figure 5). The traces mapped by Lienkaemper correspond with several locations where evidence of faulting or geomorphic features suggestive of faulting were observed.

### 3.02.2 Review of Consultant Reports

Several previous fault investigations have been performed in the vicinity of the site. The results of these studies are summarized below. The locations of the sites covered by these studies are shown on Figure 5.

Engeo (1978) performed a fault-hazard investigation of a site along El Portal Drive, near the I-80 freeway. The investigation concluded that no evidence of active faulting was present within the site. The site was later evaluated by Joyce Associates (2015), who reached a similar conclusion.

Terrasearch (1978) performed an investigation of a property 400 feet southwest of the subject site. The investigation found no evidence of active faulting.

Darwin Myers Associates (1987) investigated a site approximately 2000 feet north of the subject site. The investigation included excavation of a trench 235 feet in length. No indications of faulting were observed in the trench. However, the main trace of the Hayward-Rodgers Creek fault was judged to be a short distance east of

the site, and the study recommended that no habitable structures be built in the eastern portion of the site, due to proximity to the active fault.

Three past fault investigations have been performed at the El Portal School site. Herzog Associates (1990) excavated a short exploratory trench along the western fault trace. The trench was immediately adjacent to the western end of the multipurpose room, which displayed evidence of damage due to fault creep. The study concluded that an active trace of the Hayward-Rodgers Creek fault passes through the trench and the western portion of the site.

William Lettis and Associates (WLA, 2003) performed an extensive fault investigation of the El Portal School property. The investigation included two exploratory trenches, with a total length of 408 feet in length, which crossed both the eastern and western Fault traces. In general, evidence of the fault traces in the trench was fairly subtle, although indications of faulting were observed in the locations of the eastern and western fault traces. In addition, the study found that soil layers within the area between the fault traces have been bowed upward by fault movement, forming a feature referred to as a pressure ridge. This upward bowing of the soil units corresponds to the area where a low mound was observed on older aerial photographs. The locations of the trenches are shown on Figure 2 and trench logs are presented in Appendix D.

Baldwin, Givler and Lienkaemper (BGL) present the log of WLA Trench T-3. This short trench is located in the northern portion of the school property, at the location of a deflected fence and an echelon pavement cracks. Evidence of active faulting was subtle in the trench, and an “inferred fault location” was delineated.

Kleinfelder (2011) performed an evaluation of the Contra Costa College Campus located just north of El Portal School. This study addressed the Campus Center Project/New Student Activities Building, which is located east of the eastern and western fault traces shown on Figure 2. Although some evidence of past faulting was observed, no evidence of a through-going active fault was noted within the study area.

### 3.03 Seismicity

#### 3.03.1 General

Seismic activity within the northern Coast Ranges is generally associated with active faults of the San Andreas system, including major active faults both east and west of the site (Jennings and Bryant, 2010). The principal active faults in the region are the Hayward-Rodgers Creek fault, which crosses the subject site; the San Andreas, 18.5 miles to the west; and the Calaveras, 20 miles to the southeast (Figure 6). Other major active faults in the region include the San Gregorio fault, 21 miles to the west; the Greenville, 26 miles southeast; the Rodgers Creek, 15 miles north; and the Concord-Green Valley, 13.8 miles northeast. Table 1 summarizes the fault parameters of selected known, active faults closest to the site.

The active Hayward-Rodgers Creek fault crosses the subject site (Figures 2, 3, and 4). The term “active fault,” as used herein, refers to a fault that has experienced movement during Holocene time (about the last 11,000 years). The Hayward-Rodgers Creek fault is a northwest-trending zone about 51 miles long, which extends from southeastern San Jose, through the east bay communities, into San Pablo Bay. Beneath San Pablo Bay, it probably steps right (east), continuing north to Napa. To the south, near San Jose, the Hayward-Rodgers Creek fault merges with the Calaveras fault.

Table 1. Fault Parameters

| Fault                  | Distance and Direction from Site | Maximum Moment Magnitude |
|------------------------|----------------------------------|--------------------------|
| Hayward-Rodgers Creek  | Onsite                           | 7.6                      |
| Calaveras              | 20 miles southeast               | 6.8                      |
| Concord – Green Valley | 13.8 miles northeast             | 6.9                      |
| San Andreas            | 18.5 miles west                  | 7.9                      |
| San Gregorio           | 21 miles west                    | 7.3                      |
| Greenville             | 26 miles southeast               | 6.9                      |
| West Napa              | 16.5 miles north                 | 6.5                      |

During historical times, well-documented surface creep has occurred along the Hayward-Rodgers Creek fault at average rates ranging from about 0.2 to 0.4 inches per year (Lienkaemper et al., 1991).

The U.S. Geological Survey (USGS, 2016) has estimated that there is a 33-percent chance of a large earthquake (magnitude 6.7 or greater) occurring on the Hayward-Rodgers Creek fault by the year 2036. They estimate that there is a 72-percent-or-higher chance of a large earthquake occurring in the San Francisco Bay Region by the year 2043.

### 3.03.2 Historic Seismicity

The San Francisco Bay Region has experienced several large earthquakes during historical time. A summary of the more significant earthquakes in the region is given below.

1. The Hayward Earthquake of October 21, 1868  
On October 21, 1868, an earthquake of about M 6.8 occurred on the southern segment of the Hayward-Rodgers Creek fault, causing significant damage throughout the region. Surface ground rupture occurred over a length of approximately 30 miles. The northern limit of ground rupture was in the vicinity of Mills College. The epicenter of the 1868 earthquake was located in the Castro Valley area.
2. The 1858 and 1911 Earthquakes  
Two other earthquakes greater than M 6 are thought to have occurred on the Hayward-Rodgers Creek fault (Steinbrugge et al., 1987). These occurred in 1858 (M 6.1) and 1911 (M 6.6). Both of these earthquakes were centered in or near the southern portion of the Hayward-Rodgers Creek fault.
3. The San Francisco Earthquake of April 18, 1906  
The largest historical earthquake in the region was the great San Francisco earthquake of April 18, 1906, which occurred on the San Andreas fault near San Francisco. This earthquake (estimated to have been of magnitude 8.3) caused strong-to-violent ground shaking throughout much of west central California and caused widespread damage.
4. The Loma Prieta Earthquake of October 17, 1989  
On October 17, 1989, the M 7.1–Loma Prieta earthquake occurred near the San Andreas fault in the Santa Cruz Mountains. The earthquake resulted in 63 deaths and approximately

\$6 billion dollars in damage over a wide area (McNutt and Sydnor, 1990). Moderate ground shaking was felt in the San Pablo area (McNutt and Topozada, 1990).

5. The Napa Earthquake of August 24, 2014

A magnitude-6 earthquake occurred southwest of the City of Napa on the morning of August 24, 2014. The earthquake occurred on a previously unknown trace of the West Napa Fault and caused a ground rupture approximately 10 miles in length. The earthquake resulted in at least 208 injuries and 1 death and approximately \$400 million dollars in damage. Moderate to light ground shaking was felt in the San Pablo area.

The Working Group on California Earthquake Probabilities (WGCEP, 2008; 2015), in conjunction with the United States Geological Survey (USGS), has evaluated the probabilities of significant earthquakes occurring in the Bay Area over the next 30 years. The WGCEP report indicates that there is a 72-percent probability that at least one magnitude-6.7-or-greater earthquake will occur in the San Francisco Bay region before 2045. This probability is an aggregate value that considers seven principal Bay Area fault systems and unknown faults (background values). The findings of the WGCEP reports are summarized in the following table.

Table 2. WGCEP (2015) Probabilities

| Fault System          | Probability of at Least One Magnitude-6.7-or-Larger Earthquake in 2016-2044 |
|-----------------------|---|
| Hayward-Rodgers Creek | 31%   |
| West Napa             | 2.3%  |
| San Andreas           | 19%   |
| Calaveras             | 7.4%  |
| San Gregorio          | 2.6%  |
| Concord-Green Valley  | 5.3%  |
| Greenville            | 4%  |
| Background            | 14%   |

3.03.3 Liquefaction and Past Ground Effects

The subject site is shown to be in an area of “moderate” liquefaction susceptibility on two separate, published maps. These include maps published by Witter et al. (2006), and Knudsen, et al. (2000). Generalized descriptions of the liquefaction susceptibility of the local geologic unit (Holocene alluvial-fan deposits-Qhf), as discussed in Witter, et al. (2006), are presented below.

**Holocene alluvial-fan deposits (Qhf)** – Sediment deposited by streams emanating from mountain canyons onto alluvial valley floors or alluvial plains, including debris flow, hyperconcentrated mudflow, and braided stream deposits. Alluvial-fan sediment includes sand, gravel, silt, and clay, and is moderately to poorly sorted and moderately to poorly bedded. Sediment clast size and general particle size typically decrease downslope from the fan apex. Many Holocene alluvial fans exhibit levee/interlevee topography, particularly the fans associated with creeks flowing west from the East Bay hills. Alluvial-fan surfaces are steepest near their apex at the valley mouth and slope gently basinward, typically with gradually decreasing gradient. Alluvial-fan deposits are identified primarily on the basis of fan morphology and topographic expression. Holocene alluvial fans are relatively undissected when compared to older alluvial fans. In places, Holocene deposits may be only a thin veneer over Pleistocene deposits.

A set of maps of Northern California by Youd & Hoose (1978) documents areas that have experienced reported earthquake-induced ground failure during historic earthquakes. Such failures include earthquake-induced landslides, stream-bank landslides, lateral spread, ground settlement, ground cracks, and sand boils, among others. The nearest liquefaction-related failures are mapped approximately 3.5 miles southwest of the site.

The site is located in an area that has not been evaluated for liquefaction hazards by the State of California Seismic Hazards Zonation Program (CDMG, 2003).

#### 3.03.4 Landsliding

No landslides are mapped within or near the site (Bishop, et al., 1973; Nilsen, 1975; Graymer, 2000). The site is located in an area that has not been evaluated for seismically-induced landslide hazards (CDMG, 2003).

#### 3.04 Review of Aerial Photographs

Eight sets of black-and-white, stereo-paired aerial photographs were reviewed as a part of our evaluation. These photographs were taken between the years of 1939 and 1975. A complete listing of all photographs used in our study is included under Section 9.00, References.

The 1939 photographs show that the site is in agricultural use as a hay or grain field. None of the surrounding streets have been constructed. A small drainage, referred to as Rheem Creek, is present at the edge of the agricultural fields, along the east edge of the site. This drainage may have been relocated to its current position as a result of land leveling and farming operations. Very indistinct tonal lineaments are visible along both of the identified fault traces shown on Figure 2. These fault traces have likely been somewhat obscured by farming activity. A subtle, low, northwest-trending mound, which may be a pressure ridge, is visible between the two fault traces in the site. The mound appears to be more pronounced at the north end of the site.

The 1947 photographs show that several large buildings are present on the subject site. We understand that these buildings were military or government housing. The large subdivision immediately south of the site has been constructed and appears to be fully occupied. Fault features cannot be identified on these photographs. The 1949 photographs show no significant changes in the site conditions.

The 1953 photographs show that the school buildings, which are still present today, have been constructed in the area north of the subject site. The military/government buildings are also still present on the subject site.

The 1957 photographs show that the military/government buildings have been removed. The subject site consists of open grassland and does not appear to be in use for farming or other purposes. No evidence of an athletic field is visible. No significant changes in site conditions are visible on the 1959 photographs.

By 1971, some athletic fields, which appear to be baseball fields, have been constructed within the site. In addition, rows of trees have been planted along the east, south, and west sides of the site. The 1975 photographs show that a small building has been constructed in the south-central portion of the site. This building is no longer present.

## 4.00 SITE CONDITIONS

### 4.01 Topography

The site is located on a broad alluvial plain that extends west from the base of the eastern San Francisco Bay hills to the margin of San Francisco Bay. The site is about 1.8 miles inland from the Bay margin at an approximate elevation of 60 feet above sea level. The closest major drainage is San Pablo Creek, which is located about 1,700 feet south of the site.

The topographic survey of the former El Portal School campus by Kister Savio and Rei, dated September 25, 2015, shows the site as a relatively level area, with elevations ranging from approximately 57 feet at the west end to about 66 feet near the eastern property line. A slight rise exists along the eastern property boundary.

### 4.02 Surface Conditions

We performed a reconnaissance of the site and vicinity on July 15, 2020. Fault and creep features reported by Lienkaemper (1992) and additional features observed in our reconnaissance are described below and shown on Figure 5.

The project site is approximately rectangular in shape and has plan dimensions of about 500 feet by about 350 feet. The campus is bordered by a residential development to the east, Moraga Road to the west, a City of San Pablo—corporation yard to the south, and other portions of the former El Portal Elementary School to the north.

Near the intersection of Library Drive and Comet Way, left-stepping en echelon cracks are visible in the asphalt roadway, in the same area where similar cracks were reported by Lienkaemper. The reported damage to classroom buildings within the school property is also still visible. Along the southern margin of the subject site, the adjacent offsite building shows some right lateral deflection that aligns with the fault location observed in Trench T-1.

South of the subject site, we examined areas where previous evidence of fault creep was reported by Lienkaemper (1992), along Fordham Street, Arundel Way, Bowhill Lane, and Greenwood Drive. Along Arundel Way, the fault trace is marked by curbs offset in a right-lateral direction and well-developed, left-stepping, en echelon cracks in the asphalt. Near the intersection of Greenwood Drive and Bowhill Lane, several right-laterally-offset curbs were observed. The observed amount of offset in these areas is typically on the order of 2 to 3 inches. Along the north side of El Portal Drive, the fault location is indicated by a broad, right-lateral deflection of the curb.

### 4.03 Subsurface Conditions

#### 4.03.1 Exploratory Borings and CPTs

Below a thin layer of field turf, pavements, and minor fill, our borings and CPTs encountered native alluvial deposits. The alluvial deposits encountered consisted primarily of layered, lean-to-fat clays (CL and CH), with some intermittent layers of clayey and silty sands (SC) and clayey gravel (GC). The clay materials were found to be of soft-to-very-stiff consistency, with varying amounts of fine-to-coarse-grained sands, trace amounts of fine-grained gravels, and silt. The sand/gravel layers were generally loose to very dense, with varying amounts of moderately plastic fines. Atterberg Limits tests performed on samples of the alluvial material resulted in Plasticity Indices between 19 and 44.

#### 4.03.2 Fault Trenching

Throughout the trench, native soils were covered by a layer of clean, medium-grained sand, approximately one foot thick, which appeared to be a drainage blanket placed as a part of construction of the athletic field. Trench drains with perforated plastic pipes were observed at intervals along the trench. The base of the sand was a very sharp, flat contact.

Our engineering geologist and assistant geologist prepared a detailed log of the soil layers exposed in the trench, as shown on the attached log of Trench 1 (Figure 7). The trench exposed layered alluvial soils along its entire length. These soils consisted primarily of silty clay and clay silt, with some sand and rare gravels. Some lenses of sand and gravel were encountered in the central portion of the trench from Stations 177 to 205, which appear to be fluvial channel deposits. West of the fault, the soil layers are inclined gently to the west. We interpret these materials to be alluvial-fan deposits. Based on the degree of soil development, we judge that the older layers are approximately mid Holocene (about 4,000 to 7,000 years ago) in age.

In the eastern portion of the trench, a layer of black, silty clay containing abundant clamshell debris was encountered. We interpret this layer to represent a pond or creek deposit. This layer extended from the eastern edge of the trench to approximately Station 234, where it is sharply truncated by faulting. Two lenses of similar materials are present immediately west of the fault, suggesting that the pond extended across the fault trace at two different times.

An active fault trace was encountered at Station 233 in the eastern portion of Trench 1. The fault consisted of a near-vertical plane striking approximately north 30° west that was exposed in both sides of the trench. The fault sharply truncated the pond deposit and adjacent soil units. A faint clay seam was present along the fault; however, no polished parting was observed.

Groundwater seepage was observed at the base of the fluvial channel deposits, between Stations 183 and 200. Additional groundwater seepage was observed a short distance east of the fault at Station 238. The remaining portions of the excavation did not encounter groundwater seepage.

#### 4.03.3 Bedrock

Bedrock was not encountered in any of the borings or CPTs advanced at the site during our field explorations (deepest penetration extended to a depth of approximately 52 feet below existing grade). The depth to bedrock, as estimated from the Tri-Cities Alluvial Thickness Map (1973), is on the order of 100- to 200-feet-deep below grade.

#### 4.03.4 Groundwater

Groundwater was encountered in all four borings drilled during our study, at various depths ranging from approximately 5 to 23 feet. The CPT pore-pressure dissipation tests indicate that groundwater was estimated at depths from about 5 feet to 21 feet deep below grade. The borings and CPTs were grouted with lean concrete following completion to comply with Contra Costa County Environmental Health Department permit requirements. During the fault trench excavation, groundwater seepage was observed at about 9 feet below grade.

Based on our explorations, we estimate that a groundwater depth of 8 feet is appropriate for design purposes and for use in liquefaction analyses. We note that the water levels in our site borings may not have fully

stabilized at the time of our final measurements, prior to grouting. Additionally, the play field had been continuously irrigated prior to our field explorations. We anticipate that fluctuations in the groundwater levels may occur due to variations in rainfall, temperature, and other factors.

## **5.00 GEOLOGIC-HAZARDS EVALUATION**

Discussions and conclusions regarding potential geologic hazards are presented in the following sections.

### **5.01 Fault-Rupture Hazard**

Fault-rupture hazard is the hazard of ground breakage and displacement along fault traces during earthquakes and is considered the greatest geologic hazard at this site. During large earthquakes, such as the 1906 San Francisco earthquake, ground displacements of more than 10 feet have occurred. Because the Hayward-Rodgers Creek fault is a strike-slip fault, the most likely ground displacement would be a lateral movement of the ground, where the ground west of the fault moves northward with respect to the ground east of the fault. Such a displacement could be on the order of 3 to 6 feet and cause severe damage or collapse to structures placed across the fault trace. The presence of a pressure ridge within the northern part of this site suggests that during a large earthquake on this portion of the Hayward-Rodgers Creek fault, some degree of vertical upward movement could occur in the area.

A primary purpose of our subsurface investigation was to establish the location of the western fault trace at the southern end of the subject site. The northern location of the fault trace is well established by previous studies and was more precisely defined by WLA (2003). Based on the fault location reported by WLA and our own fault location at the southern end of the site, we believe that the fault can be projected through the central portion of the site with reasonable accuracy. We should point out that it would be prudent to have the fault location exposed in our Trench 1 surveyed and recorded on the project plans, along with the fault location reported by WLA (2003).

Based on the results of our geologic reconnaissance, review of available aerial photographs, published geologic maps, and consultant reports, we conclude that two active fault traces cross the subject site. In our opinion, the risk of surface fault rupture within the subject site is significant during the life of the proposed structures. The risk of damage to the proposed facilities can be reduced by positioning the proposed structures a minimum of 20 feet away from the identified active fault traces, employing relatively stiff foundations, and by ground improvement. One subgrade mitigation approach would involve constructing a thick soil mat beneath the site to span across potential fault ruptures and other geologic/geotechnical hazards. The mat would be constructed using geo-grid reinforcing with compacted on-site soil. The mat could also be created using several lifts of lime-treated on-site soil. These alternatives are discussed in detail in Section 7.0, Recommendations. Assuming that these mitigations are incorporated into the project design and are properly constructed, we conclude that the site is suitable for the proposed development from a fault-rupture-hazard standpoint. This conclusion assumes that the project developer accepts the risk of some damage occurring during a major earthquake.

We recommend that all buildings be set back a minimum of 20 feet from the identified fault locations. In addition, the proposed office building should be set back a minimum of 50 feet from the western fault trace.

Past earthquakes have shown that ground rupture is most likely to occur along pre-existing fault traces. However, there is always a risk that ground rupture could occur in an area that has not experienced prior faulting. In our opinion, the risk of ground rupture in an area where prior faulting has not occurred is very low during the life of the proposed facilities.

## 5.02 Strong Ground Shaking

The California Building Code has adopted provisions for incorporation of strong ground-shaking into the design of all structures. Our recommendations for geotechnical parameters to be used in the structural seismic design of the site structure are presented in Section 7.02, “California Building Code Seismic Design Parameters.”

## 5.03 Soil Liquefaction

According to a liquefaction-susceptibility map prepared by Witter, et. al. (2006), the site is located within a zone that may be moderately susceptible to liquefaction. Part of the scope of this study was to evaluate the site soils, estimate the potential for liquefaction, and estimate the extent of liquefaction-induced settlement at the site, based on the nine CPT soundings. Our analysis methodology and results are discussed below.

Liquefaction is the transformation of a deposit of soil from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress. Often, this transformation results from the cyclic loading of an earthquake, and the soil acquires “mobility” sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, saturated, uniformly-graded, fine-grained sands. However, recent research has shown that non-plastic to low-plasticity, fine-grained soils (silts and clays) can also experience liquefaction-type behavior in response to earthquake shaking.

Surface manifestations of liquefaction can include settlement, bearing-capacity failure, sand boils, and lateral spreading. Any of these effects, if severe enough, has the potential to cause damage to structures and other site improvements. In our judgment, surface manifestation of vertical ground settlement at the site is also a concern from a hazard perspective. Construction of a thick soil mat (as discussed above) beneath the site can significantly reduce the effects due to liquefaction settlement.

We evaluated the liquefaction potential at the site using the CPT data and methodology outlined in the summary paper by Youd and Idriss (2001) and updated in Robertson, (2009). This method involves assessing the seismic demand on a soil layer, expressed in terms of the cyclic stress ratio (CSR), and comparing this value to the capacity of the soil to resist liquefaction, expressed in terms of the cyclic resistance ratio (CRR). The factor of safety against liquefaction is determined by dividing the CRR by the CSR. Soils below the groundwater table and shallower than 50 feet having a factor of safety less than or equal to 1.0 are considered liquefiable.

To account for fluctuations in groundwater levels due to variations in rainfall, temperature, and other factors, we used a groundwater depth of 9 feet (below the ground surface) in our liquefaction analyses. Estimates of in-place density were obtained directly from the interpretive sounding data provided by ConeTec Investigations, Inc. Levels of ground shaking used in our analyses were based on an earthquake moment magnitude ( $M_w$ ) of 7.6 on the Hayward-Rodgers Creek fault, with a peak ground acceleration (corresponding to a 2-percent chance exceedance in 50 years’ level of hazard) of 1.04g.

The results of our analyses found that ground shaking at the site during a large earthquake on the Hayward-Rodgers Creek fault would be large enough to induce liquefaction within multiple layers of susceptible soils. As shown in Appendix B, the CPTs encountered several thin layers below the ground surface that are not likely laterally continuous across the site. In general, the liquefiable materials encountered consisted of medium-dense, silty sand mixtures.

We used the CPT data and the methods outlined in Zhang, Robertson, Brachman (2002) to estimate the magnitude of liquefaction-induced settlement at each of the CPT locations. We calculated between about ¼ inch and 6 inches of dynamic, cumulative, compressional deformations at depth could occur within the potentially liquefiable layers, as a result of a large seismic event. Surface manifestations (settlements) of the compression of the liquefiable layers may translate up to about 4 to 5 inches at the ground surface over the lateral extents of the site. A summary of our CPT-based liquefaction analysis is included in Appendix C.

The amount of differential settlement that may occur over a specified distance (for example, between adjacent building foundations) cannot be calculated directly using the existing data and available methods. However, it is not uncommon to assume that differential settlements between adjacent foundations could be on the order of one-half the calculated total settlement. Greater settlements would be anticipated if liquefied soil is ejected through sand boils or fissures.

## **6.00 GEOTECHNICAL CONSIDERATIONS**

From a geotechnical viewpoint, it is our opinion that the site is suitable for the proposed storage facility, provided that the conclusions and recommendations presented in this report are properly incorporated into the project design and construction. The primary geotechnical-design considerations for the proposed new building foundations are the expansive nature of the site soils and the design of foundations to account for potential static and dynamic settlements.

### **6.01 Expansive Soils**

Near-surface clayey soils exhibited moderate-to-critically-expansive potential. Expansive soils shrink and swell with changes in moisture content and have the potential to damage improvements that are supported on them. Construction of a thick soil mat (as discussed above) can significantly reduce the negative effects of expansive soils.

### **6.02 Foundation Support**

If the thickened-soil-mat system discussed above is utilized to mitigate the site, the planned building foundations may be supported on continuous, reinforced-concrete, mat-slab foundations.

## **7.00 RECOMMENDATIONS**

### **7.01 Earthwork**

Our conceptual recommendation to minimize the site hazards consists of creating a thick mat underneath the site buildings that can span across localized areas of soft/loose soils, areas of liquefaction-induced settlement and in the case that the fault trace extends to the surface during a large seismic event. The soil mats will also help mitigate differential settlements of the soils below.

As discussed earlier, the site hazards may be mitigated by constructing a thickened soil mat consisting of layers of re-compacted, on-site soil and geo-grid reinforcement beneath the developed portions of the site. The soil mat may also be constructed by chemically treating the subgrade soils with high-calcium quicklime. Either method will involve significant excavation and replacement of site soils.

#### 7.01.1 Site Preparation

The site of the proposed development should initially be cleared of all improvements, including foundations, utilities, and other obstructions, and then stripped to sufficient depth to remove surface vegetation and weeds; these materials should be removed from the site.

#### 7.01.2 Over-Excavation and Re-Compaction with Geo-Grid Reinforcement

We believe that it may be possible to mitigate the site by over-excavating the site by about 6 feet and recompaction with horizontal sheets of biaxial geo-grid reinforcing. We recommend the use of Tensar Biaxial 1100 (or equivalent) Geo-Grid. The geo-grid would extend through the entire developed portion of the site (extending at least 10 feet laterally beyond any building footprint and/or pavements) and would be placed at six feet and three feet below grade. The soil at the base of the over-excavation would first be scarified by 6 inches, then recompacted before the first geo-grid is placed. The top geo-grid layer would be placed to allow for utility trenching. The scarified base layer and subsequent layers should be placed and compacted in accordance with Section 7.01.5, Fill Placement and Compaction.

The challenge with this alternative is that the majority of soil that we observed in the top-8-to-10 feet consists of expansive clay soils, with very little coarse-grained soil component (sand/gravel). In order to use this alternative, the existing soils would need to be mixed with coarse-grained soil that would need to be imported or found elsewhere onsite.

#### 7.01.3 Lime Treatment

Another alternative would be to create a mat by chemically treating the onsite clay soils with high-calcium quicklime. A preliminary design for this alternative would include treating the top five feet of soils with 5-percent (by weight) high-calcium quicklime. Typical equipment can blade and mix the soil/lime in 18-inch maximum lifts. Similar to the geo-grid alternative, the lime treat should encompass the entire developed site and extend 10 feet laterally beyond any building footprint and/or pavement.

If the lime-treatment alternative is selected, a detailed treatment program should be implemented, using site soil samples and laboratory testing to develop optimum lime/moisture contents. We can perform this analysis as a separate scope item.

#### 7.01.4 Fill Materials

General fill material can be used to raise and/or restore site grades, except where non-expansive fill is required. General fill material should have an organic content of less than 3 percent by volume and should not contain rocks or lumps larger than 6 inches in greatest dimension. It is anticipated that existing site soils below the stripped layer will qualify for use as general engineered fill.

Non-expansive fill material should meet the requirements of general fill and should also have a Plasticity Index of 12 or less, with at least 10-percent material finer than a standard #200 sieve.

#### 7.01.5 Fill Placement and Compaction

Fill materials should be placed in a manner that minimizes lenses, pockets, and/or layers of materials differing substantially in texture or gradation from the surrounding fill materials. The soils should be spread in uniform

layers not exceeding 8 to 10 inches in loose thickness prior to compaction. Each layer should be compacted in a uniform and systematic manner. The fill should be constructed in layers such that the surface of each layer is nearly level.

General fill material should be moisture conditioned, as necessary, to between 3 and 5 points over optimum moisture content and compacted to at least 90-percent relative compaction, based on ASTM D1557-latest revision. The non-expansive fill beneath buildings, exterior flatwork, and other near-surface improvements should be moisture conditioned, as necessary, to near-optimum moisture content and compacted to at least 95-percent relative compaction, based on ASTM D1557-latest revision. We should note that many of the on-site clayey soils were found to have relatively high plasticity values and relatively high moisture contents and may require extra effort to dry out prior to placement as fill.

### 7.02 California Building Code Seismic Design Parameters

Based on our review of the site location, geology, ASCE 7-16, and the 2019 California Building Code (CBC), we recommend the following parameters be used for seismic design of the proposed site structures:

- Latitude = 37.9673°
- Longitude = -122.3377°
- Risk Category of Buildings and Other Structures = I (assumed)
- Site Class = E
- Mapped Spectral Acceleration for Short Period ( $S_s$ , Site Class B) = 2.25g
- Mapped Spectral Acceleration for 1-Second Period ( $S_1$ , Site Class B) = 0.869g
- $PGA_M = 1.04g$

Note: These seismic parameters are valid provided that site buildings are designed as "short-period structures," as defined by ASCE 7-16, where long period (>1.0s) seismic parameters are not applicable and equivalent lateral force (not MRSA) procedure is used for design.

### 7.03 Building Foundations

We recommend that the new foundations for the planned facility consist of reinforced-concrete mat slabs. The bottoms of the mats should be at least 12 inches below the lowest adjacent grade, and the mats should be at least 18 inches thick. The area for the mats should be cleared of appropriate foundation elements, slabs, and utilities from previous developments, and these materials should be removed from the site. The subgrade should be prepared as above in Section 7.01, Earthwork.

The mats can be designed assuming an allowable bearing pressure of 800 pounds per square foot for dead plus live loads, with a one-third increase for all loads including wind or seismic. This allowable bearing pressure is a net value; therefore, the weight of the mats can be neglected for design purposes. These values may be increased with chemical treatment of the site subgrade. The mats should be integrally connected to all portions of the structures, so the entire foundation system moves as a unit. The mats should be reinforced with top and bottom steel in both directions to allow the foundation to span local irregularities. As a minimum, we recommend that the mats be reinforced with sufficient top and bottom steel to support a random interior clear span of at least 20 feet. The mats can be designed using a modulus of subgrade reaction of 100 kips per cubic foot. This modulus value has been factored for the mat size and can be increased by one-third for total loads including seismic forces.

Lateral loads on the structures may be resisted by passive pressures acting against the sides of the mats and/or on shear keys extended under the mats where there is at least 10 feet of level ground in front of the shear keys

and/or mat slab edges. We recommend an allowable passive pressure equal to an equivalent fluid weighing 300 pounds per square foot per foot of depth. Alternatively, an allowable friction coefficient of 0.30 can be used between the bottom of the mats and the subgrade soils. If the perimeter of the mats is poured neat against the soils, the passive pressure and friction coefficient may be used in combination. Passive pressure should not be used within the upper one foot unless the ground surface is confined by a slab or pavement.

Interior slabs and any other areas where floor wetness would be undesirable should be directly underlain by a moisture-retarding barrier that includes a crushed rock layer and a plastic membrane. Either of the two alternative moisture-retarding barrier systems described below is considered acceptable:

- 4 inches of free-draining gravel overlain by a vapor-retardant membrane covered with 2 inches of sand.
- 6 inches of compacted Caltrans Class 2 Aggregate Base overlain by a heavy-duty impermeable membrane, installed and taped in accordance with the manufacturer's recommendations.

Should a vapor retarder be utilized below the mat slabs, an appropriate type is a Class-A vapor retarder [ASTM E 1745, latest revision]. Any tears in the retarder and all plumbing penetrations should be sealed with an appropriate taping material. If the vapor retarder is upgraded to a more substantial material (such as Stego Wrap 15-mil or approved equivalent), consideration could be given to elimination of the 2-inch sand layer. Again, any tears in the retarder and all plumbing penetrations should be sealed with an appropriate taping material.

Where the mat slabs will be surfaced with flooring material, we recommend that the specifications for slab-on-grade floors require that moisture-emission tests be performed on the slab prior to the installation of the flooring. No flooring should be installed until safe moisture-emission levels are recorded for the type of flooring to be used.

Due to the possibility of localized differential movements between the ground around the buildings and the foundations, flexible utility connections should be installed to reduce the likelihood of utility pipes developing leaks or shearing over the years or during severe movements.

#### 7.04 Exterior Flatwork

Exterior slabs-on-grade should be supported on a minimum 18-inch-thick layer of non-expansive fill soil (or chemically treated soil) meeting the specifications given in Section 7.01.4, Fill Materials, and compacted in accordance with the specifications given in Section 7.01.5, Fill Placement and Compaction. To mitigate the potential for moisture change within the expansive soils causing "lifting" of the outside edge of the slab-on-grade section, where exterior flatwork abuts lawn or landscape areas, a continuous, deepened curb or other vertical moisture cutoff should be provided at the perimeter of the exterior slabs that extend at least 24 inches below grade. Exterior slabs should be structurally separate from foundations.

#### 7.05 Surface Drainage

We recommend that rainwater collected on the roofs of the buildings be transmitted through gutters and downspouts to closed pipes, which discharge into the site storm-drain system and/or onto a paved surface that leads to a suitable storm-drain inlet. The ground surface directly adjacent to the structures should slope away from the buildings, in accordance with applicable building-code requirements.

#### 7.06 Utility Trenches

Utility trenches should be backfilled with fill placed in lifts not exceeding 8 inches in uncompacted thickness. Trenches should be filled by placing a granular shading layer beneath and around the pipe, and then 6 to 12 inches of shading should be carefully placed and tamped above the pipe. The remaining portion of the trench should be backfilled with onsite or import soil. The backfill (above shading layers) should be placed and compacted to a minimum relative degree of compaction of 90 percent, per ASTM D1557-latest revision. The compaction requirements given above should be considered minimum recommended requirements. If City and/or utility-company specifications require more stringent backfill requirements, then those specifications should be followed.

If imported granular soil is used, sufficient water should be added during the trench-backfilling operations to prevent the soil from “bulking” during compaction. All compaction operations should be performed by mechanical means only. We recommend against jetting.

If granular backfill is used for utility trenches, we recommend an impermeable plug or mastic sealant be used where utilities enter the building, to minimize the potential for free water or moisture to enter below the building.

We recommend the contractor carefully evaluate the stability of all trenches and use temporary shoring, where appropriate. The design and installation of the temporary shoring should be wholly the responsibility of the contractor. In addition, all state and local regulations governing safety around such excavations should be carefully followed.

#### 7.07 Final Geotechnical Plan Review

We recommend our firm be provided the opportunity for a general review of the geotechnical aspects of the final plans and specifications for this project, in order that the geotechnical recommendations may be properly interpreted and implemented. If our firm is not accorded the privilege of making the recommended review, we can assume no responsibility for misinterpretation of our recommendations.

#### 7.08 Construction Observation

The analyses and recommendations submitted in this report are based in part upon the data obtained from our field exploration and geotechnical laboratory testing. The nature and extent of variations across the site may not become evident until construction. If variations then become apparent, it will be necessary to re-examine the recommendations of this report.

We recommend that we be retained to provide geotechnical-engineering services during the earthwork, foundation construction, and drainage phases of the work. This is to observe compliance with the design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction. In order to effectively accomplish our observations during the project construction, we recommend that a pre-construction meeting be held to develop a mechanism for proper communications throughout the project.

## 8.00 LIMITATIONS

This report has been prepared for the exclusive use of A Recess Development Company and their consultants for specific application to the proposed Moraga Road Storage Project, in accordance with generally accepted soil- and foundation-engineering practices. No other warranty, expressed or implied, is made. In the event that any changes in the nature or design of the buildings are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the conclusions of this report are modified or verified in writing.

The findings of this report are valid as of the present date. However, the passing of time will likely change the conditions of the existing property, due to natural processes or the works of man. In addition, due to legislation or the broadening of knowledge, changes in applicable or appropriate standards may occur. Accordingly, the findings of this report may be invalidated, wholly or partly, by changes beyond our control. Therefore, this report should not be relied upon after a period of three years without being reviewed by this office.

## 9.00 REFERENCES

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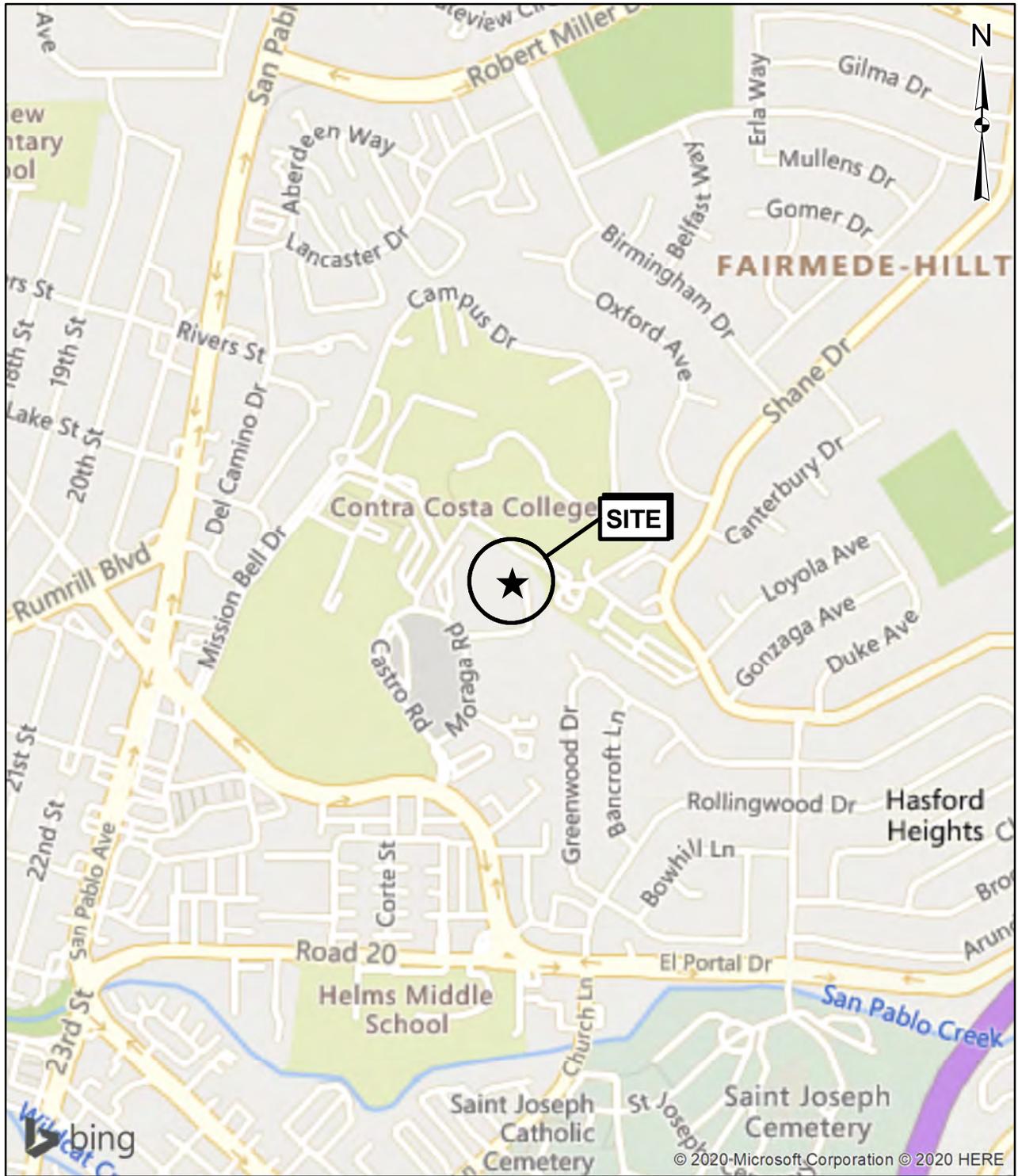
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9.03 Aerial Photographs

| <u>Type</u>                      | <u>Date</u> | <u>Scale</u> | <u>Photo No.</u> | <u>Source</u>          |
|----------------------------------|-------------|--------------|------------------|------------------------|
| Black and White,<br>Single Photo | 08/04/39    | 1:20,000     | BUU-290-24, 25   | AKA                    |
| Black and White,<br>Stereo Pair  | 03/24/47    | 1:20,000     | AV11-2-4, 5      | Pacific Aerial Surveys |
| Black and White,<br>Stereo Pair  | 09/06/49    | 1:9,600      | AV28-8-9, 10     | Pacific Aerial Surveys |
| Black and White,<br>Stereo Pair  | 08/15/53    | 1:10,000     | AV119-6-6, 7     | Pacific Aerial Surveys |
| Black and White,<br>Stereo Pair  | 05/03/57    | 1:12,000     | AV253-5-5, 6     | Pacific Aerial Surveys |
| Black and White,<br>Stereo Pair  | 08/0759     | 1:9,600      | AV337-10-9, 10   | Pacific Aerial Surveys |
| Black and White,<br>Stereo Pair  | 05/18/71    | 1:12,000     | AV995-7-5, 6     | Pacific Aerial Surveys |
| Black and White,<br>Stereo Pair  | 05/29/75    | 1:12,000     | AV1193-9-5, 6    | Pacific Aerial Surveys |

2388-5A Moraga Rd Storage – GI Report

## FIGURES



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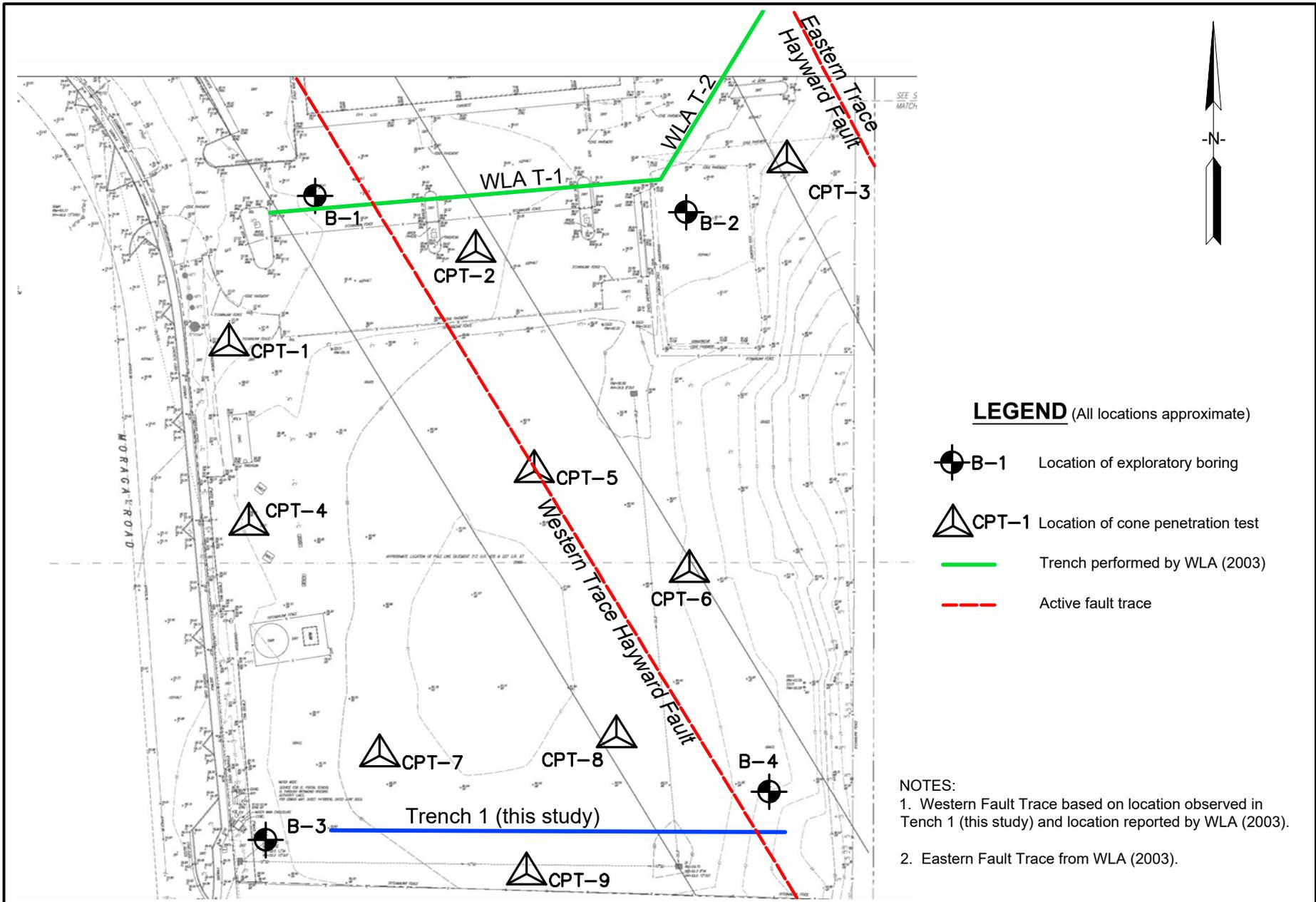
**VICINITY MAP**

MORAGA ROAD STORAGE PROJECT  
San Pablo, California

PROJECT NO.  
**2388-5A**

DATE  
**August 2020**

FIGURE **1**



**LEGEND** (All locations approximate)

-  B-1 Location of exploratory boring
-  CPT-1 Location of cone penetration test
-  Trench performed by WLA (2003)
-  Active fault trace

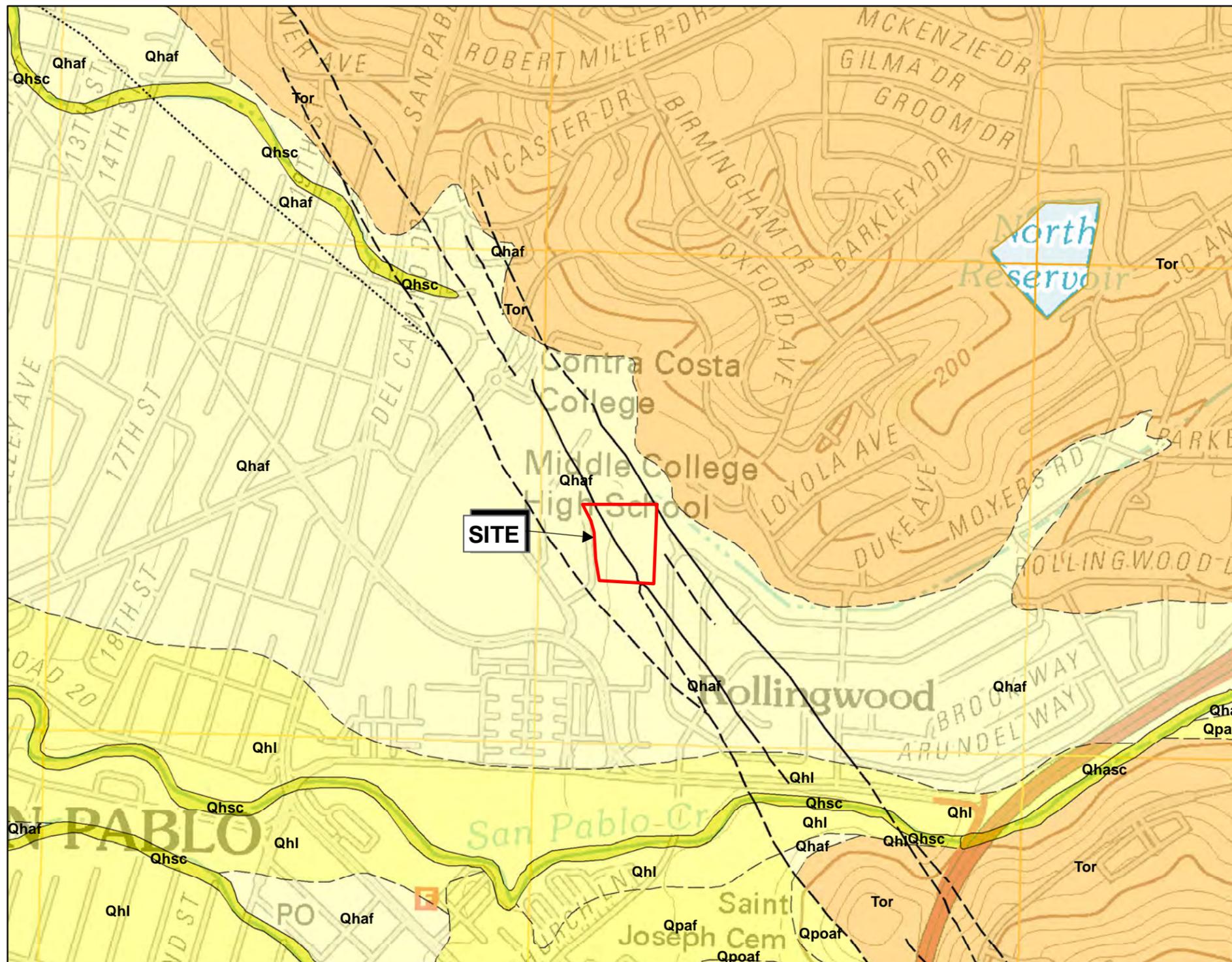
**NOTES:**

1. Western Fault Trace based on location observed in Trench 1 (this study) and location reported by WLA (2003).
2. Eastern Fault Trace from WLA (2003).




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| <b>SITE PLAN</b>                                     |                            |                 |
|--|----------------------------|-----------------|
| MORAGA ROAD STORAGE PROJECT<br>San Pablo, California |                            |                 |
| PROJECT NO.<br><b>2388-5A</b>                        | DATE<br><b>August 2020</b> | FIGURE <b>2</b> |



**LEGEND**

- Qhl** Natural levee deposits (Holocene)
- Qhaf** Alluvial fan and fluvial deposits (Holocene)
- Qpoaf** Older alluvial fan deposits (Pleistocene)
- Qhasc** Artificial stream channels (Historic)
- Qhsc** Stream channel deposits (Holocene)
- Qpaf** Alluvial fan and fluvial deposits (Pleistocene)
- Tor** Orinda formation (late Miocene)
- Traces of the Hayward Fault

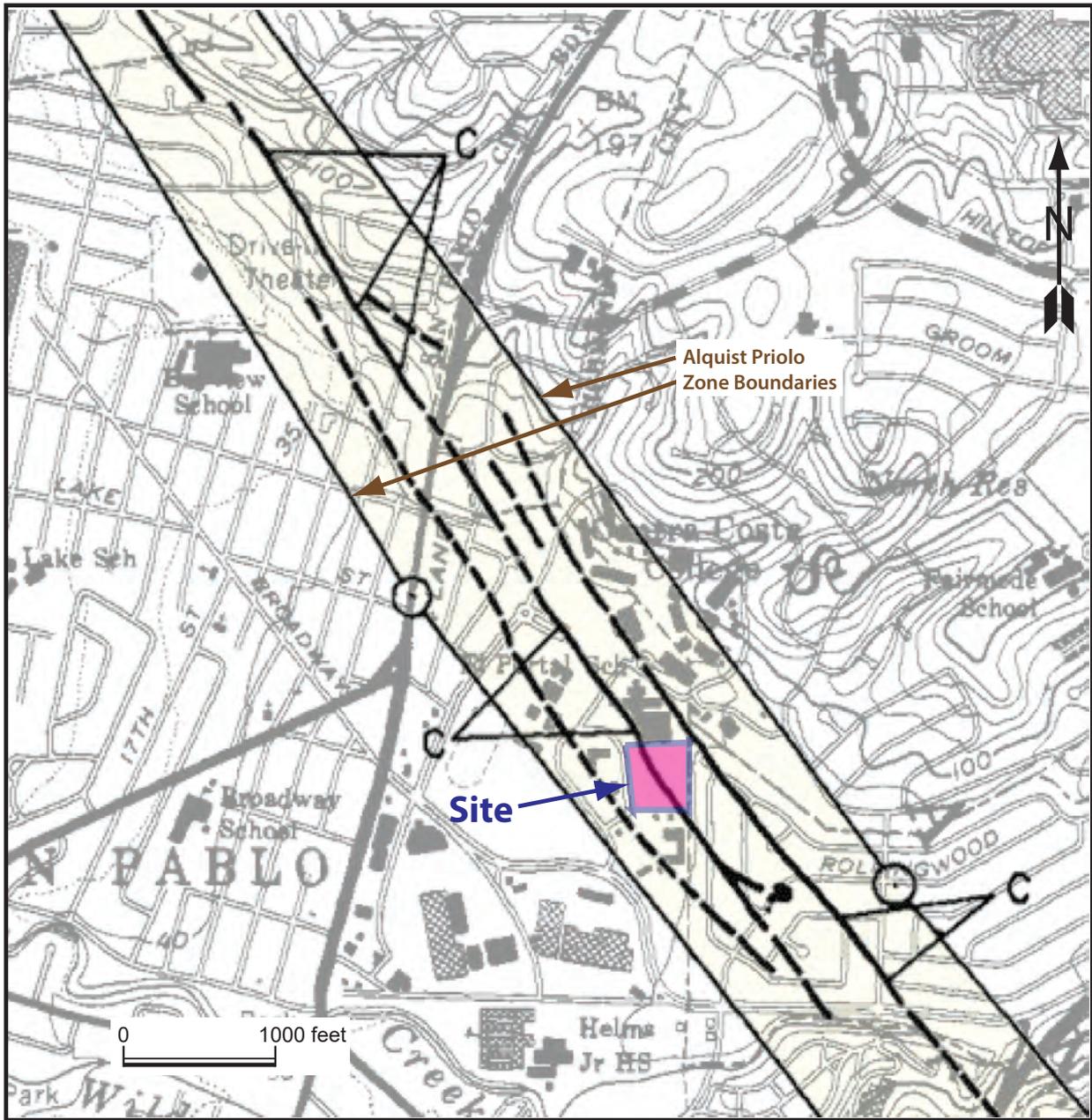
Source: "Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California," by R.W. Graymer, 2000, UGSG MF-2342.

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| REGIONAL GEOLOGY                                     |                            |                 |
|--|----------------------------|-----------------|
| MORAGA ROAD STORAGE PROJECT<br>San Pablo, California |                            |                 |
| PROJECT NO.<br><b>2388-5A</b>                        | DATE<br><b>August 2020</b> | FIGURE <b>3</b> |



**EXPLANATION**

 Fault Trace, dashed where approximately located, queried where uncertain

C Areas of observed creep, queried where uncertain

Reference: Base map taken from Alquist Priolo Earthquake Fault Zone Map, Richmond Quadrangle, California, 1982



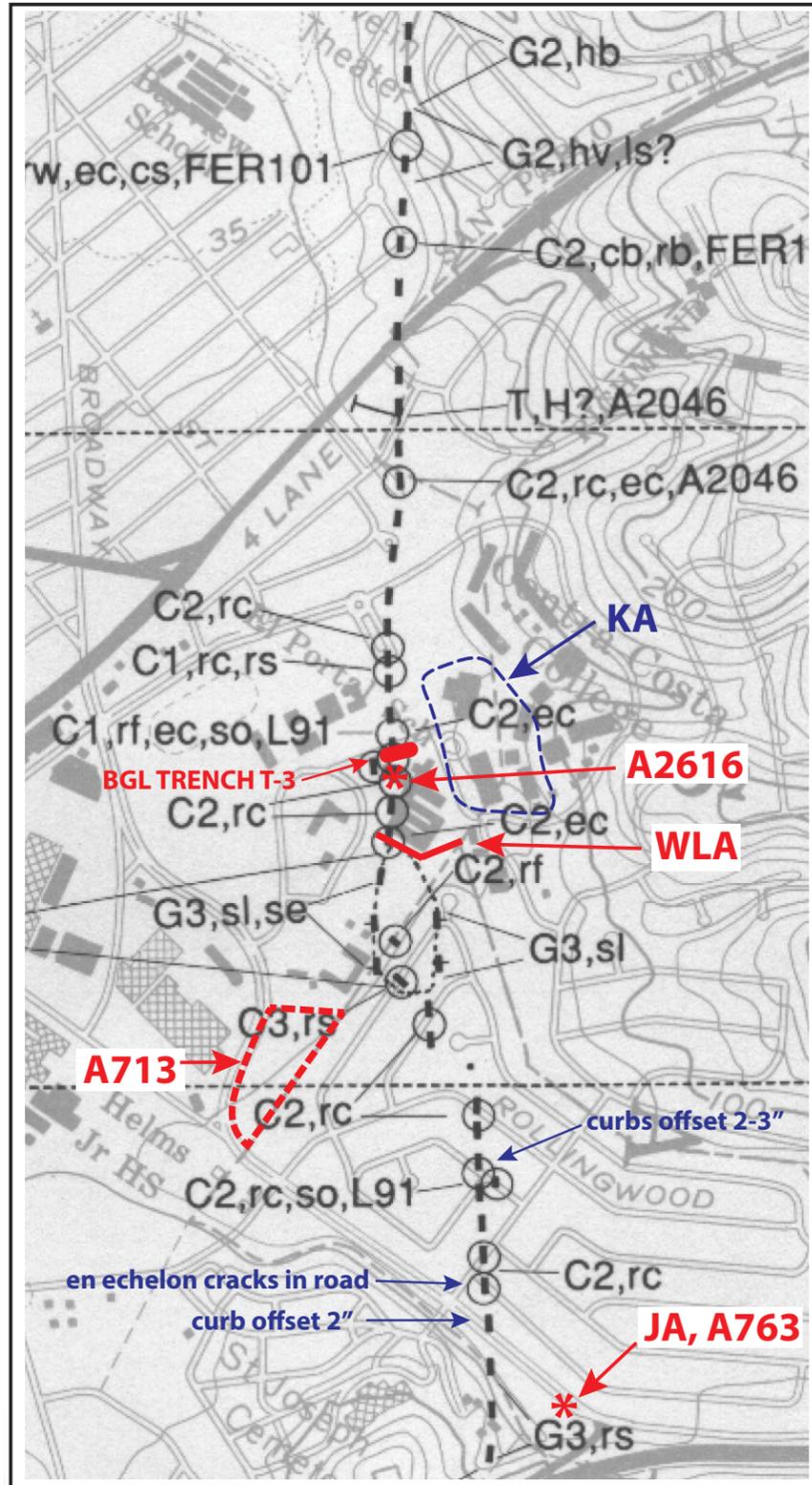
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**ALQUIST PRIOLO FAULT ZONE MAP**  
MORAGA ROAD STORAGE PROJECT  
San Pablo, California

PROJECT NO.  
**2388-5A**

DATE  
**August 2020**

FIGURE **4**



**EXPLANATION**

**Fault** - Dashed to show approximate maximum uncertainty in lateral position of fault traces. Each dash represents 30 m on map. Traces mainly derive from geomorphic data, but creep and trench data used where available to constrain the position of most intense deformation more precisely. Particular observations used to locate faults are annotated on the map in abbreviated form (see Abbreviations). Hachures indicate the lower side in apparent vertical separations of the ground surface from geomorphic evidence. Dotted traces interpolate fault locations where evidence is concealed or destroyed; queries indicate speculative connections where data is absent or suspect. Sawteeth on upper plate of fault that has significant thrust or reverse component

- Creep locality
- Trench location - Actual length
- ↘ Short trench - Not to scale
- Fault location in trench - No geomorphic trace recognized
- ∩ Small tectonic depression (df or dr) - Not to scale
- ∪ Tectonic depression (df or dr) or pressure ridge (pr) - Actual size, hand drafted
- ♀ Spring or seep (sp) - Located on active fault trace
- ⊥ Monoclinial flexure
- △ Trilateration monument - See Prescott and Lisowski (1983)
- - - ≤± 20 m uncertainty (20-m space on map between 30-m dashes)
- - - ≤± 40 m uncertainty (40-m space on map between 30-m dashes)
- - - ≤± 60 m uncertainty (60-m space on map between 30-m dashes)

Consultant Reports:

- A713** Terrasearch (1978)
- A763** Engeo (1978)
- A2046** Darwin Myers Associates (1987)
- A2616** Herzog Associates (1990)
- WLA** William Lettis & Associates (2003)
- BGL** Baldwin, Givler, and Lienkaemper (2009)
- JA** Joyce Associates (2015)
- KA** Kleinfelder (2011)

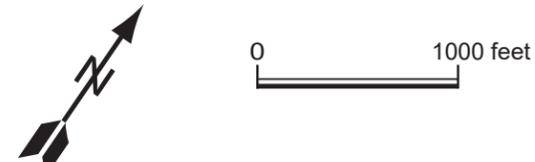
**ABBREVIATIONS**

- C - CREEP EVIDENCE**
- 1 - strongly pronounced fault creep
  - 2 - distinct and certain creep evidence
  - 3 - inconclusive evidence for creep
  - ? - additional uncertainty in tectonic origin
- G - GEOMORPHIC FEATURES**
- 1 - strongly pronounced feature
  - 2 - distinct feature
  - 3 - weakly pronounced feature
  - ? - additional uncertainty in tectonic origin
- T - TRENCH EXPOSURES**  
(and other geologic evidence)
- H1 - Holocene age of offset determined by radiocarbon (<sup>14</sup>C) dating
  - H2 - Modern soil or alluvial unit distinctly offset, or contains features conclusive of shearing, such as gouge, rotated pebbles, transported materials in shear zone, and filled fissures over distinct Pleistocene faults
  - H? - Inconclusive signs of Holocene offset, such as steps in base of soil or apparent shears in clay-rich materials. Without corroboration such evidence neither proves nor disproves either existence or age of faulting
- ABBREVIATIONS (continued)**
- aa - alignment array
  - cb - concentration of cracks in above grade structure
  - cc - concentration of cracks in concrete slab
  - cp - concentration of pavement cracks
  - cs - curb separating from sidewalk or pavement
  - ec - en echelon left-stepping cracks in pavement
  - jo - opening of joints or cracks in concrete
  - pp - multiple patches in pavement
  - pu - compressional pop-up or buckle in concrete
  - ra - right-laterally offset aqueduct, water pipe, or tunnel
  - rb - distortion or racking of above-grade structure (including separating additions and stairways)
  - rc - right-laterally offset curb or form line
  - rf - right-laterally offset fence line
  - rp - right-laterally offset painted line
  - rr - right-laterally offset railroad tracks or guardrail
  - rs - right-laterally offset sidewalk
  - rt - right-laterally offset line of trees
  - rw - right-laterally offset wall
  - so - surveyed offset feature

- mp - Youngest traces disturbed by human activities. Mapped trace bisects disturbed zone. Dash gap equals half width of disturbed zone.
- n - notch
- pr - pressure ridge in left stepover
- rr - right-laterally offset ridge line
- rs - right-laterally offset stream or gully
- sb - broad linear scarp (implies multiple traces)
- sc - scissor point, sense of vertical separation reverses
- se - subsoil exposed
- sl - linear scarp, undifferentiated
- sn - narrow linear scarp (implies dominant trace)
- sp - spring
- ss - swale in saddle
- vl - line of vegetation

- REFERENCE CODES** (see also Abbreviated Map References and text for full references.)
- A2456 - Trench log or creep evidence in Alquist-Priolo report AP-2456, filed at California Division of Mines and Geology (CDMG), San Francisco.
  - C200 - Trench log or creep evidence in non-Alquist-Priolo consultant's report filed at CDMG.
  - G70 - Non-Alquist-Priolo unpublished report referenced in abbreviated references as G70.
- ABBREVIATIONS (continued)**
- H - Active trace reported in trench, trench logs not in public file
  - HP - Distinct faulting in unconsolidated alluvium of possibly Holocene or more likely latest Pleistocene age
  - F? - Feature shown as fault in log resembles nontectonic feature such as bedrock-alluvial contact, buried terrace riser, or landslide plane
  - NF - No fault observed
  - P - Distinct evidence of significant faulting in Pliocene or Pleistocene sediments
  - RC - Roadcut log
  - WB - Ground water barrier
  - U - Age of faulting unobtainable because surficial deposits removed

Reference: Lienkaemper, 1991, Map of Recently Active Traces of the Hayward Fault, Alameda and Contra Costa Counties, California: U. S. Geological Survey Miscellaneous Field Studies Map MF-2196.



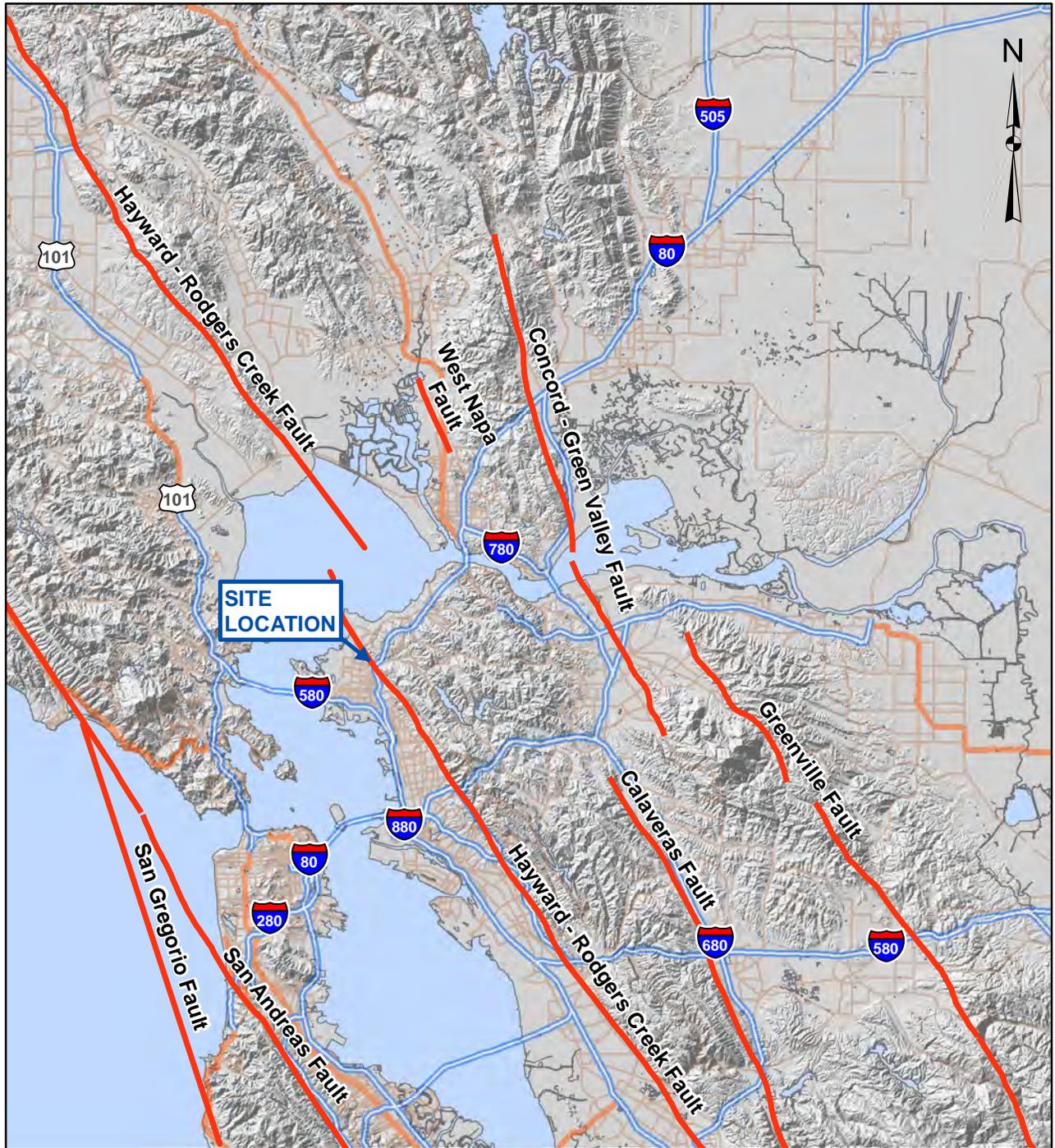
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**LIENKAEMPER FAULT MAP**

MORAGA ROAD STORAGE PROJECT  
San Pablo, California

PROJECT NO. 2388-5A  
DATE August 2020

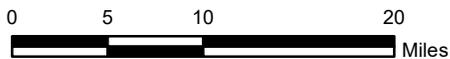
FIGURE 5



**LEGEND**

 "Active" Regional Faults (Surface Displacement within the Last 11,000 years)

Original figure produced in color.



Sources: 1) The California Geological Survey, 2001, CD's 2001-04, 2001-05, and 2001-06: GIS Files of Official Alquist-Priolo Earthquake Fault Zones ([http://www.consrv.ca.gov/CGS/geologic\\_hazards/regulatory\\_hazard\\_zones/ap\\_cd\\_htm.htm](http://www.consrv.ca.gov/CGS/geologic_hazards/regulatory_hazard_zones/ap_cd_htm.htm)).  
 2) Jennings, 1994, "Fault Activity Map of California and Adjacent Areas," CDMG, Geologic Data Map No. 6  
 3) California Division of Mines and Geology, 1997, "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada."  
 4) Background data: USGS 10m DEM.



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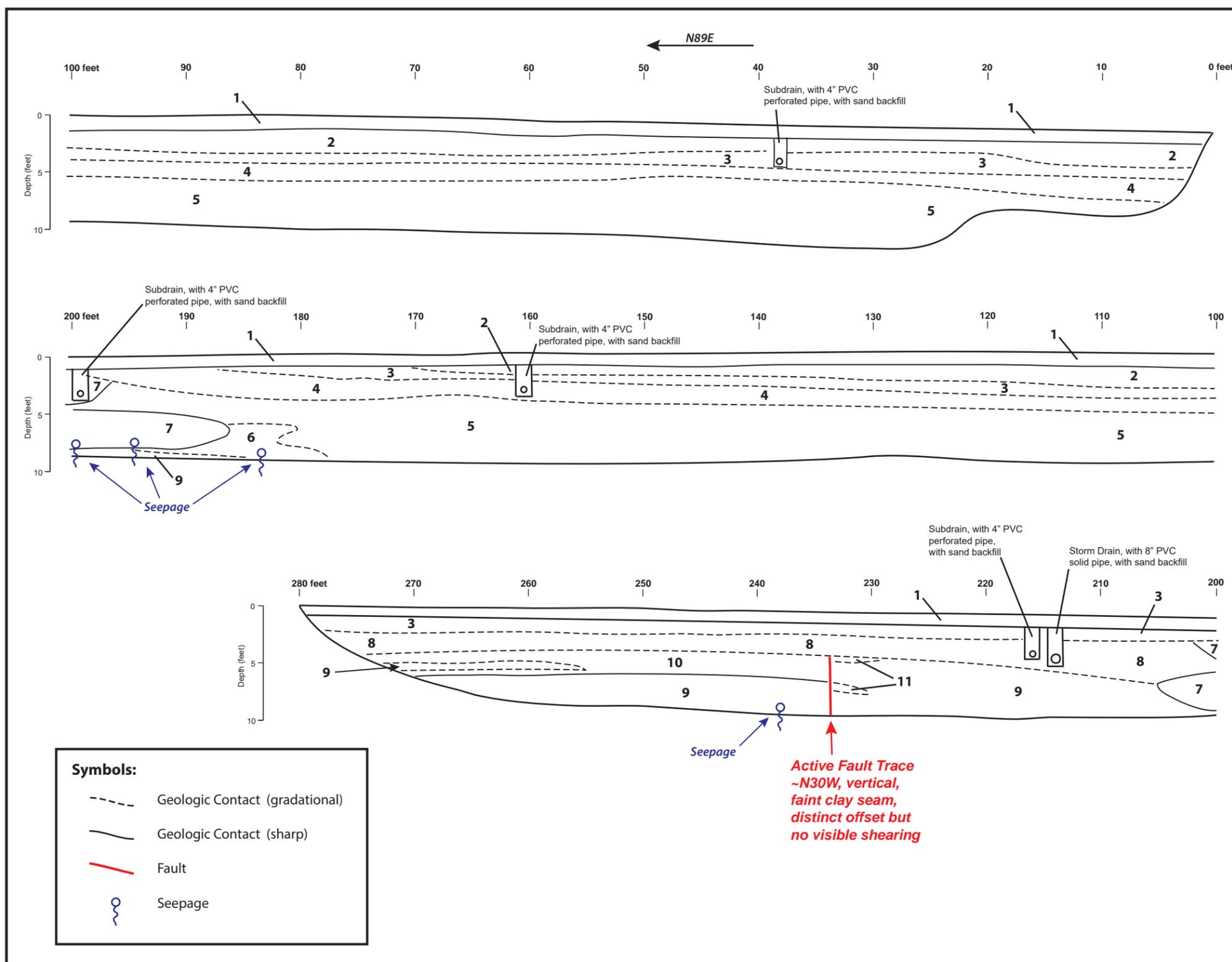
**REGIONAL FAULT MAP**

MORAGA ROAD STORAGE PROJECT  
San Pablo, California

PROJECT NO.  
**2388-5A**

DATE  
**August 2020**

FIGURE **6**



- Soil And Rock Units:**
- 1 Turf, underlain by 1 foot of medium-grained clean sand, sharp basal contact (Fill)
  - 2 Black (2.5N, wet) Clayey Silt With Sand (ML) very stiff, moist, with rootlets, with rare gravels along base, basal contact gradational. (Soil Horizon A, Holocene)
  - 3 Black (2.5N, wet) Silty Clay (CH) very stiff, wet, with rootlets, with moderately well-developed columnar structure, with rare clay films, basal contact gradational. (Soil Horizon Bt)
  - 4 Very Dark Grayish Brown (2.5Y 3/2, wet) Silty Clay (CL) very stiff, wet, with vertical black carbon streaks, basal contact gradational. (Alluvium, Soil Horizon C, Holocene)
  - 5 Dark Grayish Brown (2.5Y 4/3, wet) Silty Clay (CL) very stiff, wet, with few vertical black carbon streaks, with krotovina filled with black clay in upper 2 feet, no living roots. (Alluvium - Holocene)
  - 6 Olive Brown (2.5Y 4/4, moist) Silty Sand (SM) fine grained, dense, moist, with few gravels, with rare shell fragments, basal contact gradational. (Alluvium, Holocene)
  - 7 Olive Gray Sandy Gravel (GP) dense, moist, with well-rounded pebbles to 1" diameter, with minor shell fragments, contacts sharp, erosional (Alluvium, Holocene)
  - 8 Olive Brown Clay Silt With Sand (ML) very stiff, moist, with rootlets, with poorly-developed columnar structure, with occasional clay films, basal contact gradational. (Alluvium, Holocene)
  - 9 Very Dark Grayish Brown Silty Sand With Clay (SM) dense, wet, with rootlets (Alluvium, Holocene)
  - 10 Black Silty Clay (CL) with rounded gravels, very stiff, moist, with abundant white clam shells, fairly sharp basal contact (Pond Deposit, Alluvium, Holocene)
  - 11 Very Dark Grayish Brown (2.5Y 3/2, moist) Silty Clay (CL) hard, moist, with minor shell debris, contacts gradational. (Alluvium, Holocene)

**Symbols:**

- Geologic Contact (gradational)
- Geologic Contact (sharp)
- Fault
- Seepage



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| LOG OF TRENCH 1                                      |             |                 |
|--|-------------|-----------------|
| MORAGA ROAD STORAGE PROJECT<br>San Pablo, California |             |                 |
| PROJECT NO.  | DATE        | FIGURE <b>7</b> |
| 2388-5A  | August 2020 |                 |

APPENDIX A  
LOGS OF BORINGS

## SOIL CLASSIFICATION CHART

| PRIMARY DIVISIONS   |  |  | SECONDARY DIVISIONS   |              |                             |
|---|--|--|---|--------------|-----------------------------|
|   |  |  | CRITERIA *  | GROUP SYMBOL | GROUP NAME                  |
| COARSE-GRAINED SOILS<br>MORE THAN 50%<br>RETAINED ON NO.200 SIEVE | GRAVELS<br>MORE THAN 50% OF<br>COARSE FRACTION<br>RETAINED ON NO.4 SIEVE | CLEAN GRAVELS<br>LESS THAN<br>5% FINES         | $C_u \geq 4$ AND $1 \leq C_c \leq 3^A$  | GW           | Well-graded gravel          |
|   |  |  | $C_u < 4$ AND/OR $1 > C_c > 3$  | GP           | Poorly-graded gravel        |
|   |  | GRAVELS WITH<br>FINES - MORE<br>THAN 12% FINES | FINES CLASSIFY AS ML OR MH  | GM           | Silty gravel                |
|   |  |  | FINES CLASSIFY AS CL OR CH  | GC           | Clayey gravel               |
|   | SANDS<br>50% OR MORE OF<br>COARSE FRACTION<br>PASSES NO. 4 SIEVE         | CLEAN SANDS<br>LESS THAN<br>5% FINES           | $C_u \geq 6$ AND $1 \leq C_c \leq 3$  | SW           | Well-graded sand            |
|   |  |  | $C_u < 6$ AND/OR $1 > C_c > 3$  | SP           | Poorly-graded sand          |
|   |  | SANDS WITH<br>FINES - MORE<br>THAN 12% FINES   | FINES CLASSIFY AS ML OR MH  | SM           | Silty sand                  |
|   |  |  | FINES CLASSIFY AS CL OR CH  | SC           | Clayey sand                 |
| FINE-GRAINED SOILS<br>50% OR MORE<br>PASSES THE NO.200 SIEVE      | SILTS AND CLAYS<br>LIQUID LIMIT LESS<br>THAN 50%                         | INORGANIC                                      | PI $> 7$ AND PLOTS ON OR ABOVE "A" LINE   | CL           | Lean clay                   |
|   |  |  | PI $< 4$ OR PLOTS BELOW "A" LINE  | ML           | Silt                        |
|   |  | ORGANIC  | $\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$ | OL           | Organic Clay & Organic Silt |
|   |  |  | PI PLOTS ON OR ABOVE "A" LINE   | CH           | Fat clay                    |
|   | SILTS AND CLAYS<br>LIQUID LIMIT 50%<br>OR MORE                           | INORGANIC                                      | PI PLOTS BELOW "A" LINE   | MH           | Elastic silt                |
|   |  | ORGANIC  | $\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$ | OH           | Organic Clay & Organic Silt |
|   |  |  | PI PLOTS ON OR ABOVE "A" LINE   | PT           | Peat                        |
|   |  | HIGHLY ORGANIC SOILS                           | PRIMARILY ORGANIC MATTER, DARK<br>IN COLOR, AND ORGANIC ODOR                      |              |                             |

REFERENCE: Unified Soil Classification System (ASTM D2487-11)

\* Criteria may be done on visual basis, not necessarily based on lab testing

$$A - C_u = D_{60}/D_{100} \quad \& \quad C_c = (D_{30})^2 / (D_{10} \times D_{60})$$

### GRAIN SIZES

|                 | U. S. STANDARD SERIES SIEVE |        |        |        | CLEAR SQUARE SIEVE OPENINGS |         |          |
|-----------------|-----------------------------|--------|--------|--------|-----------------------------|---------|----------|
|                 | 200                         | 40     | 10     | 4      | 3/4"                        | 3"      | 12"      |
| SILTS AND CLAYS | SAND                        |        |        | GRAVEL |                             | COBBLES | BOULDERS |
|                 | FINE                        | MEDIUM | COARSE | FINE   | COARSE                      |         |          |

### ABBREVIATIONS

#### INDEX TESTS

- LL - Liquid Limit (%) (ASTM D4318-17)  
 PI - Plasticity Index (%) (ASTM D4318-17)  
 -200 - Passing No. 200 Sieve (%) (ASTM D1140-17)

#### STRENGTH TESTS

- PP - Field Pocket Penetrometer test of unconfined compressive strength (tsf)  
 TV - Field Torvane test of shear strength (psf)  
 UC - Laboratory unconfined compressive strength (psf) (ASTM D2166/2166M-16)  
 TXUU - Laboratory unconsolidated, undrained triaxial test of undrained shear strength (psf) (ASTM D2850-15)

#### MISCELLANEOUS

- ATOD - At time of drilling  
 psf/tsf - pounds per square foot / tons per square foot  
 psi - pounds per square inch (indicates relative force required to advance Shelby tube sampler)

### SYMBOLS

- Standard Penetration Test Split Spoon (2-inch O.D.)
- Modified California Sampler (3-inch O.D.)
- Thin-walled Sampler Tube (either Pitcher or Shelby) (3-inch O.D.)
- Rock Core
- Bag Sample
- Groundwater Level during drilling
- Groundwater Level after drilling



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### KEY TO EXPLORATORY BORING LOGS

MORAGA ROAD STORAGE PROJECT  
San Pablo, California

PROJECT NO.

DATE

FIGURE **A-1**

**2388-5A**

**August 2020**

|  |                                       |                              |
|--|---------------------------------------|------------------------------|
| <b>DRILL RIG:</b> Truck Auger B-24, Solid Flight Auger | <b>SURFACE ELEVATION:</b> 58' +/- MSL | <b>LOGGED BY:</b> PB         |
| <b>DEPTH TO GROUNDWATER:</b> (see note 1)              | <b>BORING DIAMETER:</b> 4.5 inches    | <b>DATE DRILLED:</b> 6/22/20 |

| DESCRIPTION AND REMARKS  | COLOR            | CONSISTENCY         | SOIL TYPE | DEPTH (ft) | SAMPLER TYPE | SAMPLER BLOW COUNTS | MOISTURE CONTENT (%) | DRY DENSITY (pcf)                                | OTHER TESTS                                       |
|--|------------------|---------------------|-----------|------------|--------------|---------------------|----------------------|--|---|
| <b>ASPHALT PAVEMENT - 3" THICK</b>   |                  |                     |           |            |              |                     |                      |  |   |
| <b>CLAY, Lean</b> - with fine to coarse sand, some rounded fine gravel, very moist | Dark Gray        | Stiff to Very Stiff | CL        | 1          | [X]          | [14]                | 21                   | 99   | LL = 47<br>PI = 30<br>-200 = 77%<br>PP = 2.25 tsf |
|  |                  |                     |           | 2          |              |                     |                      |  |   |
|  |                  |                     |           | 3          |              |                     |                      |  |   |
| <b>CLAY, Fat</b> - some silt and fine sand, moist                                  | Black            | Very Stiff          | CH        | 4          | [X]          | [36]                | 29                   | 86   | LL = 65<br>PI = 44<br>-200 = 98%<br>PP = 2.25 tsf |
|  |                  |                     |           | 5          |              |                     |                      |  |   |
|  |                  |                     |           | 6          |              |                     |                      |  |   |
|  |                  |                     |           | 7          |              |                     |                      |  |   |
|  |                  | Stiff to Very Stiff | 8         | [X]        | [12]         | 32                  | 82                   | LL = 49<br>PI = 30<br>-200 = 97<br>PP = 0.75 tsf |   |
|  |                  |                     | 9         |            |              |                     |                      |  |   |
|  |                  |                     | 10        |            |              |                     |                      |  |   |
|  |                  |                     | 11        |            |              |                     |                      |  |   |
|  |                  |                     | 12        |            |              |                     |                      |  |   |
|  |                  |                     | 13        |            |              |                     |                      |  |   |
| Dark Yellowish Brown   | Firm to Stiff    | CH                  | 14        | [X]        | [12]         | 32                  | 82                   | LL = 49<br>PI = 30<br>-200 = 97<br>PP = 0.75 tsf |   |
|  |                  |                     | 15        |            |              |                     |                      |  |   |
|  |                  |                     | 16        |            |              |                     |                      |  |   |
|  |                  |                     | 17        |            |              |                     |                      |  |   |
|  |                  |                     | 18        |            |              |                     |                      |  |   |
|  |                  |                     | 19        |            |              |                     |                      |  |   |
| <b>CLAY, Fat</b> - with silt and very fine sand, moist                             | Dark Olive Brown | Firm to Stiff       | CH        | 16         | [X]          | [12]                | 32                   | 82   | LL = 49<br>PI = 30<br>-200 = 97<br>PP = 0.75 tsf  |
|  |                  |                     |           | 17         |              |                     |                      |  |   |

(Continued on Next Page)

AKA BORING LOG 2388-5A BORING LOGS.GPJ AKA\_TEMPLATE.GDT 8/27/20



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**EXPLORATORY BORING LOG**  
MORAGA ROAD STORAGE  
San Pablo, California

|             |             |        |            |
|-------------|-------------|--------|------------|
| PROJECT NO. | DATE        | SHEET  | BORING NO. |
| 2388-5A     | August 2020 | 1 of 2 | 1          |

| DESCRIPTION AND REMARKS   | COLOR            | CONSISTENCY   | SOIL TYPE | DEPTH (ft) | SAMPLER TYPE | SAMPLER BLOW COUNTS | MOISTURE CONTENT (%) | DRY DENSITY (pcf) | OTHER TESTS   |
|---|------------------|---------------|-----------|------------|--------------|---------------------|----------------------|-------------------|---------------|
| <i>(Continued from Previous Page)</i>   |                  |               |           |            |              |                     |                      |                   |               |
| <b>CLAY, Fat</b> - with silt and very fine sand, moist                                  | Dark Olive Brown | Firm to Stiff | CH        |            |              |                     |                      |                   |               |
| <b>CLAY, Fat</b> - with silt and fine sand, moist                                       | Brown            | Firm          | CH        | 21         |              | [10]                | 28                   | 90                | PP = 1.2 tsf  |
|   |                  |               |           | 22         |              |                     |                      |                   |               |
|   |                  |               |           | 23         |              |                     |                      |                   |               |
|   |                  |               |           | 24         |              |                     |                      |                   |               |
|   |                  |               |           | 25         |              |                     |                      |                   |               |
|   |                  |               |           | 26         |              |                     |                      |                   |               |
|   |                  | Stiff         | SM        | 25         |              | [15]                | 28                   | 90                | PP = 1.25 tsf |
| 26  |                  |               |           |            |              |                     |                      |                   |               |
| <b>SAND, Silty</b> - uniform fine sand, trace clay, wet                                 | Olive Gray       | Loose         | SM        | 30         |              | [12]                | 28                   | 90                |               |
|   |                  |               |           | 31         |              |                     |                      |                   |               |
|   |                  |               |           | 32         |              |                     |                      |                   |               |
|   |                  |               |           | 33         |              |                     |                      |                   |               |
|   |                  |               |           | 34         |              |                     |                      |                   |               |
| <b>SAND, Poorly Graded</b> - uniform fine sand, trace silt, grades to SP/SM lenses, wet | Dark Olive Gray  | Loose         | SP        | 35         |              | [13]                | 24                   | 96                |               |
|   |                  |               |           | 36         |              |                     |                      |                   |               |

Bottom of boring at 36.5 feet.

NOTES:

- Groundwater was encountered at approximately 23 feet at the time of drilling and the boring was backfilled immediately after drilling. (See report for discussion.)
- Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
- Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.
- Elevations were estimated from plans drawn by Kister, Savio & Rei, Inc., dated 9/25/15.
- Approximate unconfined compressive strength values were recorded in the field using a pocket penetrometer. These values are shown on the logs and are preceded by the symbol "PP".

AKA BORING LOG 2388-5A BORING LOGS.GPJ AKA\_TEMPLATE.GDT 8/27/20



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**EXPLORATORY BORING LOG**  
MORAGA ROAD STORAGE  
San Pablo, California

|             |             |        |            |
|-------------|-------------|--------|------------|
| PROJECT NO. | DATE        | SHEET  | BORING NO. |
| 2388-5A     | August 2020 | 2 of 2 | 1          |

|  |                                       |                              |
|--|---------------------------------------|------------------------------|
| <b>DRILL RIG:</b> Truck Auger B-60, Solid Flight Auger | <b>SURFACE ELEVATION:</b> 60' +/- MSL | <b>LOGGED BY:</b> PB         |
| <b>DEPTH TO GROUNDWATER:</b> (see note 1)              | <b>BORING DIAMETER:</b> 4.5 inches    | <b>DATE DRILLED:</b> 6/22/20 |

| DESCRIPTION AND REMARKS   | COLOR            | CONSISTENCY | SOIL TYPE | DEPTH (ft) | SAMPLER TYPE | SAMPLER BLOW COUNTS | MOISTURE CONTENT (%) | DRY DENSITY (pcf) | OTHER TESTS  |
|---|------------------|-------------|-----------|------------|--------------|---------------------|----------------------|-------------------|--|
| <b>ASPHALT PAVEMENT - ~3" THICK</b>   |                  |             |           |            |              |                     |                      |                   |  |
| <b>CLAY, Fat</b> - with silt, some very fine sand, slightly moist             | Dark Brown       | Stiff       | CH        | 1          | [X]          | [15]                | 24                   | 90                | PP = 2.0 tsf<br><br>LL = 50<br>PI = 31<br>-200 = 86<br>PP = 2.25 tsf |
|   |                  |             |           | 2          |              |                     |                      |                   |  |
|   |                  |             |           | 3          |              |                     |                      |                   |  |
|   |                  |             |           | 4          |              |                     |                      |                   |  |
|   |                  |             |           | 5          |              |                     |                      |                   |  |
| <b>CLAY, Fat</b> - uniform very fine sand, increasing sand, very moist to wet | Olive Brown      | Firm        | CH        | 6          | [X]          | [9]                 |                      |                   | PP = 0.5 tsf   |
|   |                  |             |           | 7          |              |                     |                      |                   |  |
|   |                  |             |           | 8          |              |                     |                      |                   |  |
|   |                  |             |           | 9          |              |                     |                      |                   |  |
| <b>CLAY, Fat</b> - with silt, some very firm sand, slightly moist             | Dark Olive Brown | Stiff       | CH        | 10         | [X]          | [20]                | 19                   | 99                | LL = 56<br>PI = 38<br>-200 = 95%<br>PP = 2.2 tsf                     |
|   |                  |             |           | 11         |              |                     |                      |                   |  |
|   |                  |             |           | 12         |              |                     |                      |                   |  |
|   |                  |             |           | 13         |              |                     |                      |                   |  |
|   |                  |             |           | 14         |              |                     |                      |                   |  |
|   |                  |             |           | 15         |              |                     |                      |                   |  |
|   |                  |             |           | 16         |              |                     |                      |                   |  |
|   |                  |             |           | 17         |              |                     |                      |                   |  |
|   |                  |             |           | 18         |              |                     |                      |                   |  |
|   |                  |             |           | 19         |              |                     |                      |                   |  |

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**EXPLORATORY BORING LOG**  
MORAGA ROAD STORAGE  
San Pablo, California

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| PROJECT NO. | DATE        | SHEET  | BORING NO. |
| 2388-5A     | August 2020 | 1 of 2 | 2          |

AKA BORING LOG 2388-5A BORING LOGS.GPJ AKA\_TEMPLATE.GDT 8/27/20

| DESCRIPTION AND REMARKS  | COLOR                 | CONSISTENCY | SOIL TYPE | DEPTH (ft) | SAMPLER TYPE | SAMPLER BLOW COUNTS | MOISTURE CONTENT (%) | DRY DENSITY (pcf) | OTHER TESTS   |
|--|-----------------------|-------------|-----------|------------|--------------|---------------------|----------------------|-------------------|---------------|
| <i>(Continued from Previous Page)</i>  |                       |             |           |            |              |                     |                      |                   |               |
| <b>CLAY, Fat, Sandy</b> - with gravel, fine sand with scattered fine gravel, slightly moist            | Dark Brown            | Very Stiff  | CH        | 21         | [X]          | [24]                |                      |                   | PP = 2.7 tsf  |
|  |                       |             |           | 22         |              |                     |                      |                   |               |
|  |                       |             |           | 23         |              |                     |                      |                   |               |
|  |                       |             |           | 24         |              |                     |                      |                   |               |
|  |                       |             |           | 25         |              |                     |                      |                   |               |
| <b>CLAY, Fat, Gravelly</b> - with increasing gravel, slightly moist                                    | Olive Gray            | Stiff       | CH        | 25         | [X]          | [20]                | 19                   | 112               |               |
|  |                       |             |           | 26         |              |                     |                      |                   |               |
|  |                       |             |           | 27         |              |                     |                      |                   |               |
|  |                       |             |           | 28         |              |                     |                      |                   |               |
|  |                       |             |           | 29         |              |                     |                      |                   |               |
| <b>CLAY, Fat, Sandy</b> - very high plasticity, silty very fine sand, some iron staining, moist to wet | Olive Gray            | Stiff       | CH        | 30         | [ ]          | 13                  |                      |                   |               |
|  |                       |             |           | 31         |              |                     |                      |                   |               |
|  |                       |             |           | 32         |              |                     |                      |                   |               |
|  |                       |             |           | 33         |              |                     |                      |                   |               |
|  |                       |             |           | 34         |              |                     |                      |                   |               |
| <b>CLAY, Fat, Silty</b> - some fine sand, grades to clayey sand at bottom, moist                       | Olive Brown with Gray | Very Stiff  | CH        | 35         | [X]          | [30]                | 24                   | 95                | PP = 3.25 tsf |
|  |                       |             |           | 36         |              |                     |                      |                   |               |

Bottom of boring at 36.5 feet.

NOTES:

1. Groundwater was encountered at approximately 5 feet at the time of drilling and the boring was backfilled immediately after drilling. (See report for discussion.)
2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
3. Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.
4. Elevations were estimated from plans drawn by Kister, Savio & Rei, Inc., dated 9/25/15.
5. Approximate unconfined compressive strength values were recorded in the field using a pocket penetrometer. These values are shown on the logs and are preceded by the symbol "PP".

AKA BORING LOG 2388-5A BORING LOGS.GPJ AKA\_TEMPLATE.GDT 8/27/20



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MORAGA ROAD STORAGE  
San Pablo, California

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| PROJECT NO. | DATE        | SHEET  | BORING NO. |
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|  |                                       |                              |
|--|---------------------------------------|------------------------------|
| <b>DRILL RIG:</b> Truck Auger B-60, Solid Flight Auger | <b>SURFACE ELEVATION:</b> 58' +/- MSL | <b>LOGGED BY:</b> PB         |
| <b>DEPTH TO GROUNDWATER:</b> (see note 1)              | <b>BORING DIAMETER:</b> 4.5 inches    | <b>DATE DRILLED:</b> 6/22/20 |

| DESCRIPTION AND REMARKS  | COLOR                     | CONSISTENCY | SOIL TYPE | DEPTH (ft) | SAMPLER TYPE | SAMPLER BLOW COUNTS | MOISTURE CONTENT (%) | DRY DENSITY (pcf) | OTHER TESTS                                      |
|--|---------------------------|-------------|-----------|------------|--------------|---------------------|----------------------|-------------------|--|
| <b>PLAY FIELD SOD</b>  |                           |             |           |            |              |                     |                      |                   |  |
| <b>SAND, Poorly Graded</b> - uniform fine sand, trace fines, slightly moist (FILL) | Dark Yellowish Brown      | Loose       | SP        | 1          | ⊗            |                     |                      |                   |  |
|  |                           |             |           | 2          | ⊗            |                     |                      |                   |  |
|  |                           |             |           | 3          | ⊗            | [14]                |                      |                   | PP = 2.4 tsf                                     |
| <b>CLAY, Fat</b> - silty very fine sand, trace fine gravel, slightly moist         | Black                     | Very Stiff  | CH        | 4          | ⊗            |                     |                      |                   | LL = 58<br>PI = 35<br>-200 = 95%<br>PP = 3.7 tsf |
|  |                           |             |           | 5          | ⊗            | [34]                | 21                   | 92                |  |
|  |                           |             |           | 6          | ⊗            |                     |                      |                   | LL = 39<br>PI = 22<br>-200 = 68%                 |
| <b>CLAY, Lean, Sandy</b> - fine to medium sand, some fine gravel, moist            | Black to Dark Olive Brown | Very Stiff  | CL        | 7          | ⊗            | [32]                | 21                   | 94                |  |
|  |                           |             |           | 8          |              |                     |                      |                   |  |
|  |                           |             |           | 9          |              |                     |                      |                   |  |
|  |                           |             |           | 10         | ⊗            |                     |                      |                   |  |
| <b>CLAY, Fat</b> - with uniform fine sand, silty, slightly moist                   | Olive Brown               | Stiff       | CH        | 11         | ⊗            |                     |                      |                   |  |
|  |                           |             |           | 12         |              | [13]                | 22                   | 93                | -200 = 79%                                       |
|  |                           |             |           | 13         |              |                     |                      |                   |  |
|  |                           |             |           | 14         |              |                     |                      |                   |  |
|  |                           |             |           | 15         | ⊗            |                     |                      |                   |  |
|  |                           |             |           | 16         | ⊗            | [11]                |                      |                   |  |
|  |                           |             |           | 17         |              |                     |                      |                   |  |
|  |                           |             |           | 18         |              |                     |                      |                   | ▽  |
|  |                           |             |           | 19         |              |                     |                      |                   |  |

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AKA BORING LOG 2388-5A BORING LOGS.GPJ AKA\_TEMPLATE.GDT 8/27/20



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| PROJECT NO. | DATE        | SHEET  | BORING NO. |
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| DESCRIPTION AND REMARKS  | COLOR       | CONSISTENCY  | SOIL TYPE | DEPTH (ft) | SAMPLER TYPE | SAMPLER BLOW COUNTS | MOISTURE CONTENT (%) | DRY DENSITY (pcf) | OTHER TESTS   |
|--|-------------|--------------|-----------|------------|--------------|---------------------|----------------------|-------------------|---------------|
| <i>(Continued from Previous Page)</i>  |             |              |           |            |              |                     |                      |                   |               |
| <b>CLAY, Fat</b> - with uniform fine sand, silty, slightly moist<br>-increased sand, wet | Olive Brown | Soft to Firm | CH        | 21         | X            | [6]                 | 29                   | 88                |               |
|  |             |              |           | 22         |              |                     |                      |                   |               |
|  |             |              |           | 23         |              |                     |                      |                   |               |
|  |             |              |           | 24         |              |                     |                      |                   |               |
| <b>CLAY, Fat</b> - silty, some fine sand, very moist                                     | Dark Brown  | Stiff        | CH        | 25         | X            | [14]                |                      |                   | PP = 1.25 tsf |
|  |             |              |           | 26         |              |                     |                      |                   |               |
|  |             |              |           | 27         |              |                     |                      |                   |               |
|  |             |              |           | 28         |              |                     |                      |                   |               |
|  |             |              |           | 29         |              |                     |                      |                   |               |
|  |             |              |           | 30         |              |                     |                      |                   |               |
|  |             |              |           | 31         |              |                     |                      |                   |               |
| 32   | [21]        | 25           | 94        |            |              |                     |                      |                   |               |
| 33   |             |              |           |            |              |                     |                      |                   |               |
| <b>CLAY, Lean</b> - lean with uniform fine sand, silty, very moist                       | Olive Brown | Firm         | CL        | 34         | X            |                     |                      |                   |               |
|  |             |              |           | 35         |              |                     |                      |                   |               |
|  |             |              |           | 36         |              |                     |                      |                   |               |

Bottom of boring at 36.5 feet.

NOTES:

- Groundwater was encountered at approximately 18.3 feet at the time of drilling and the boring was backfilled immediately after drilling. (See report for discussion.)
- Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
- Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.
- Elevations were estimated from plans drawn by Kister, Savio & Rei, Inc., dated 9/25/15.
- Approximate unconfined compressive strength values were recorded in the field using a pocket penetrometer. These values are shown on the logs and are preceded by the symbol "PP".

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 MORAGA ROAD STORAGE  
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| PROJECT NO. | DATE        | SHEET  | BORING NO. |
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|  |  |                              |
|--|--|------------------------------|
| <b>DRILL RIG:</b> Truck Auger B-24, Solid Flight Auger | <b>SURFACE ELEVATION:</b> (see note 4) | <b>LOGGED BY:</b> PB         |
| <b>DEPTH TO GROUNDWATER:</b> (see note 1)              | <b>BORING DIAMETER:</b> 4.5 inches     | <b>DATE DRILLED:</b> 6/22/20 |

| DESCRIPTION AND REMARKS  | COLOR                | CONSISTENCY   | SOIL TYPE | DEPTH (ft) | SAMPLER TYPE | SAMPLER BLOW COUNTS | MOISTURE CONTENT (%) | DRY DENSITY (pcf) | OTHER TESTS                                       |              |   |
|--|----------------------|---------------|-----------|------------|--------------|---------------------|----------------------|-------------------|---|--------------|---|
| <b>PLAY FIELD SOD</b>  |                      |               |           |            |              |                     |                      |                   |   |              |   |
| <b>CLAY, Lean</b> - with silt and uniform fine sand, slightly moist  | Dark Yellowish Brown | Firm to Stiff | CL        | 1          | ⊗            |                     |                      |                   |   |              |   |
| <b>CLAY, Lean</b> - with sand, silty, very fine-grained sand, trace roots, visually dry<br><br><br><br><br><br><br><br><br><br>-with increasing fine sand            | Brown                | Stiff         | CL        | 2          | ⊗            | [12]                | 18                   | 102               | LL = 42<br>PI = 25<br>-200 = 87%<br>PP = >4.5 tsf |              |   |
|  |                      |               |           | 3          | ⊗            |                     |                      |                   |   |              |   |
|  |                      |               |           | 4          | ⊗            | [22]                |                      |                   |   | PP = 2.5 tsf |   |
|  |                      |               |           | 5          | ⊗            |                     |                      |                   |   |              |   |
|  |                      |               |           | 6          | ⊗            | [14]                | 21                   | 97                | LL = 36<br>PI = 19<br>-200 = 78%                  |              |   |
|  |                      |               |           | 7          |              |                     |                      |                   |   |              |   |
|  |                      |               |           | 8          |              |                     |                      |                   |   |              |   |
|  |                      |               |           | 9          |              |                     |                      |                   |   |              |   |
|  |                      |               |           | 10         |              |                     |                      |                   |   |              | ▽ |
|  |                      |               |           | 11         |              | Very Stiff          |                      |                   |   |              |   |
| <b>GRAVEL, Clayey</b> - with clay and sand, fine gravel, rounded to subangular coarse gravel up to 2" sampler, fine to coarse sand, wet, includes clayey sand lenses | Brown                | Dense         | GC        | 12         |              |                     |                      |                   |   |              |   |
|  |                      |               |           | 13         |              |                     |                      |                   |   |              |   |
|  |                      |               |           | 14         |              |                     |                      |                   |   |              |   |
|  |                      |               |           | 15         | ⊗            |                     |                      |                   |   |              |   |
|  |                      |               |           | 16         | ⊗            | [73]                | 15                   | 113               | -200 = 24%  |              |   |
|  |                      |               |           | 17         |              |                     |                      |                   |   |              |   |
|  |                      |               |           | 18         |              |                     |                      |                   |   |              |   |
|  |                      |               |           | 19         |              |                     |                      |                   |   |              |   |

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| DESCRIPTION AND REMARKS  | COLOR            | CONSISTENCY | SOIL TYPE | DEPTH (ft) | SAMPLER TYPE | SAMPLER BLOW COUNTS | MOISTURE CONTENT (%) | DRY DENSITY (pcf) | OTHER TESTS |
|--|------------------|-------------|-----------|------------|--------------|---------------------|----------------------|-------------------|-------------|
| <i>(Continued from Previous Page)</i>  |                  |             |           |            |              |                     |                      |                   |             |
| <b>GRAVEL, Clayey</b> - with clay and sand, fine gravel, rounded to subangular coarse gravel up to 2" sampler, fine to coarse sand, wet, includes clayey sand lenses | Brown            | Dense       | GC        | 21         | X            | [50/4"]             |                      |                   |             |
|  |                  |             |           | 22         |              |                     |                      |                   |             |
|  |                  |             |           | 23         |              |                     |                      |                   |             |
|  |                  |             |           | 24         |              |                     |                      |                   |             |
|  |                  |             |           | 25         |              |                     |                      |                   |             |
| <b>GRAVEL, Clayey</b> - some sand and clay, mostly subrounded quartz and hard rock type, wet   | Dark Olive Brown | Very Dense  | GC        | 25         |              |                     | 11                   |                   |             |
|  |                  |             |           | 26         |              |                     |                      |                   |             |
| <b>SAND, Poorly Graded</b> - uniform fine sand, trace silt, wet  | Dark Olive Brown | Very Dense  | SP        | 26         |              | 50/6"               |                      |                   |             |
|  |                  |             |           | 27         |              |                     |                      |                   |             |
|  |                  |             |           | 28         |              |                     |                      |                   |             |
|  |                  |             |           | 29         |              |                     |                      |                   |             |
|  |                  |             |           | 30         |              |                     |                      |                   |             |
|  |                  |             |           | 31         |              |                     |                      |                   |             |
|  |                  |             |           | 32         |              |                     |                      |                   |             |
|  |                  |             |           | 33         |              |                     |                      |                   |             |
|  |                  |             |           | 34         |              |                     |                      |                   |             |
|  |                  |             |           | 35         |              |                     |                      |                   |             |
| <b>GRAVEL, Clayey</b> - with sand, mostly fine gravel up to 1" rounded to subangular hard rock types, some coarse sand, very moist to wet                            | Dark Olive Gray  | Very Dense  | GC        | 34         |              |                     |                      |                   |             |
|  |                  |             |           | 35         |              |                     |                      |                   |             |
|  |                  |             |           | 36         |              |                     |                      |                   |             |

Bottom of boring at 36.5 feet.

NOTES:

1. Groundwater was encountered at approximately 9.5 feet at the time of drilling and the boring was backfilled immediately after drilling. (See report for discussion.)
2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
3. Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.
4. Elevations were estimated from plans drawn by Kister, Savio & Rei, Inc., dated 9/25/15.
5. Approximate unconfined compressive strength values were recorded in the field using a pocket penetrometer. These values are shown on the logs and are preceded by the symbol "PP".

AKA BORING LOG 2388-5A BORING LOGS.GPJ AKA\_TEMPLATE.GDT 8/27/20



**ALAN KROPP  
& ASSOCIATES**  
*Geotechnical  
Consultants*

**EXPLORATORY BORING LOG**  
MORAGA ROAD STORAGE  
San Pablo, California

|             |             |        |            |
|-------------|-------------|--------|------------|
| PROJECT NO. | DATE        | SHEET  | BORING NO. |
| 2388-5A     | August 2020 | 2 of 2 | <b>4</b>   |

APPENDIX B  
CONE PENETROMETER TEST REPORT  
(CONETEC)

# PRESENTATION OF SITE INVESTIGATION RESULTS

## El Portal

*Prepared for:*

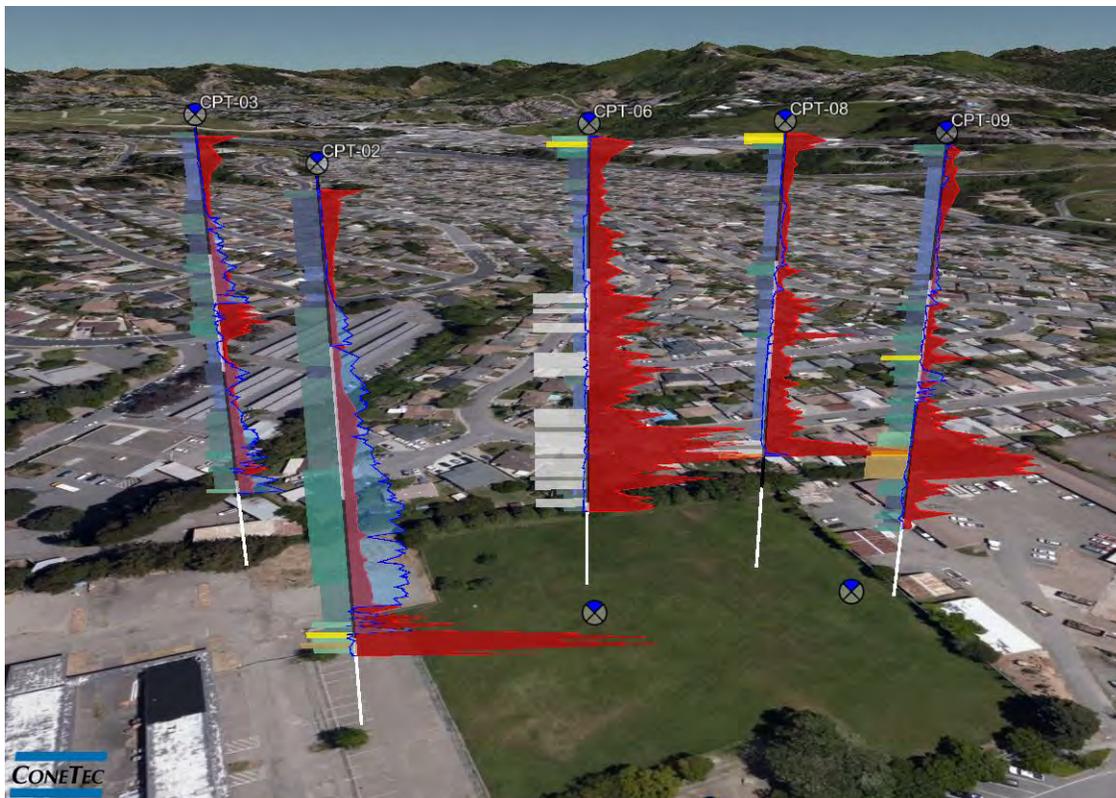
Alan Kropp & Associates

ConeTec Job No: 20-56-20953

Project Start Date: 11-Jun-2020

Project End Date: 12-Jun-2020

Report Date: 16-Jun-2020



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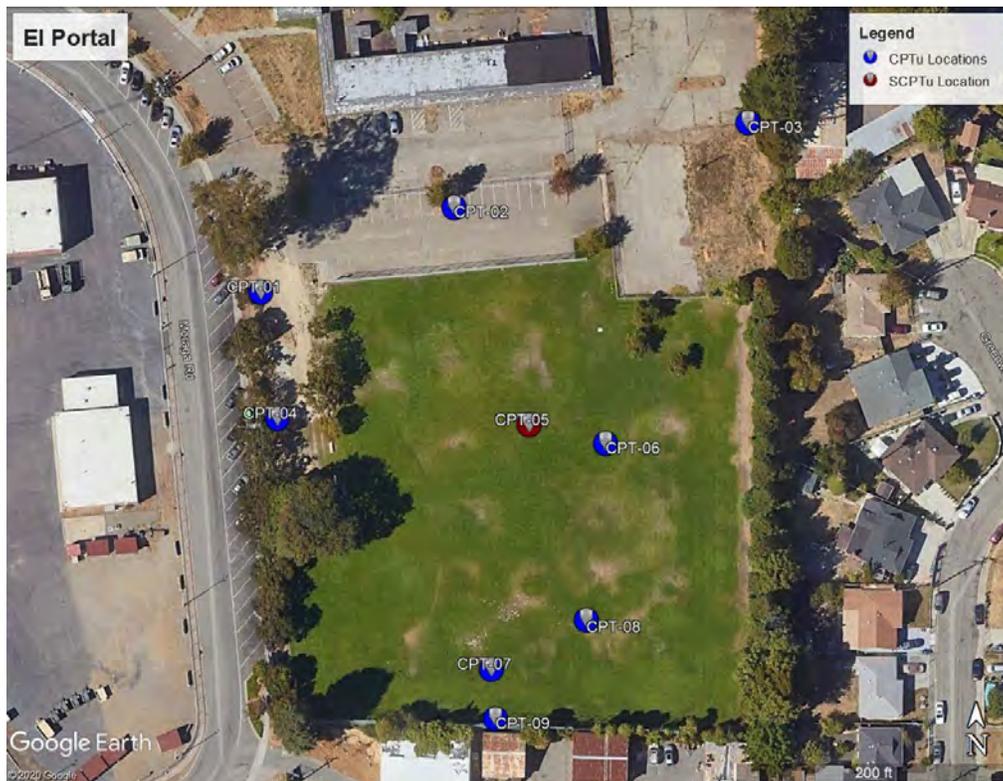
## Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for Alan Kropp & Associates of Berkeley, CA. The program consisted of cone penetration testing (CPTu) at nine (9) locations. Shear wave velocities were recorded in one (1) of the soundings.

## Project Information

|                   |                         |
|-------------------|-------------------------|
| Project           |                         |
| Client            | Alan Kropp & Associates |
| Project           | El Portal               |
| ConeTec Project # | 20-56-20953             |

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



| Rig Description      | Deployment System             | Test Type  |
|----------------------|-------------------------------|------------|
| CPT truck rig (C15)  | 30-ton truck mounted cylinder | CPTu       |
| CPT track rig (GPT2) | 20-ton track mounted cylinder | CPTu/SCPTu |

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



| Cone Penetrometers Used for this Project |             |   |                                |                    |                       |                              |
|--|-------------|---|--------------------------------|--------------------|-----------------------|------------------------------|
| Cone Description                         | Cone Number | Cross Sectional Area (cm <sup>2</sup> ) | Sleeve Area (cm <sup>2</sup> ) | Tip Capacity (bar) | Sleeve Capacity (bar) | Pore Pressure Capacity (psi) |
| 383:T1500F15U500                         | 383         | 15                                      | 225                            | 1500               | 15                    | 500                          |
| 496:T1500F15U1K                          | 496         | 15                                      | 225                            | 1500               | 15                    | 1000                         |

The CPT summary shows the cone used on each sounding.

| Cone Penetration Test      |  |
|----------------------------|--|
| Depth reference            | Depths are referenced to the existing ground surface at the time of test.  |
| Tip and sleeve data offset | 0.1 Meter<br>This has been accounted for in the CPT data files.  |
| Additional Comments        | Advanced plots with $I_c$ , $\Phi_i$ , $S_u(Nkt)$ , and $N1(60)I_c$ , Seismic plots, as well as Soil Behavior Type (SBT) Scatter plots have been included in the data release package. |

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

### Limitations

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

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



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

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

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in 5 cm<sup>2</sup>, 10 cm<sup>2</sup> and 15 cm<sup>2</sup> tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm<sup>2</sup> penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm<sup>2</sup> piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross-sectional area (typically forty-four millimeter diameter over a length of thirty-two millimeter with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

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

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

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

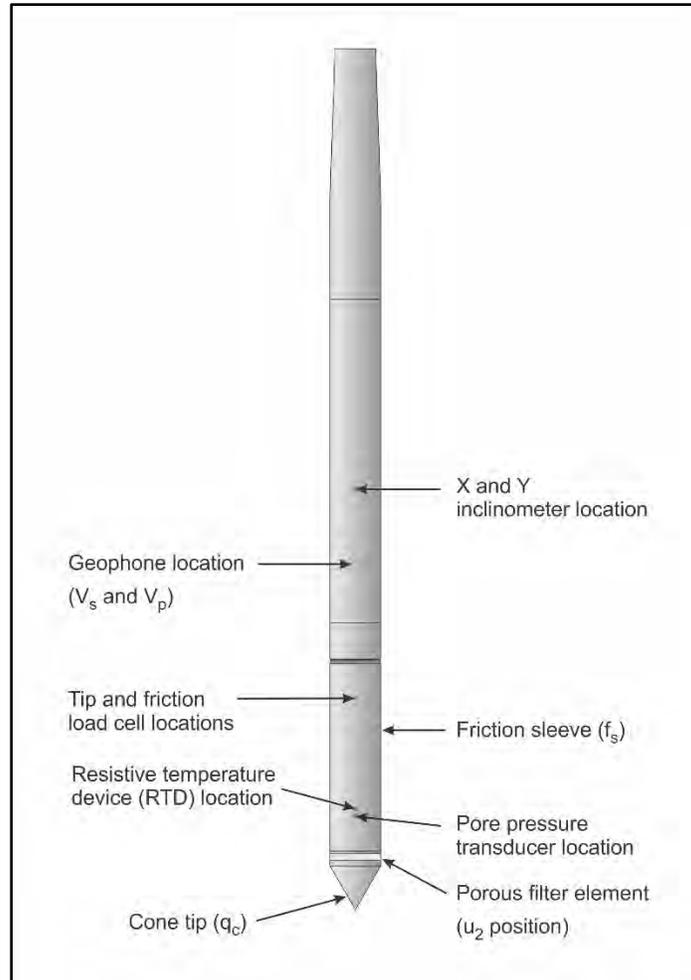


Figure CPTu. Piezocone Penetrometer (15 cm<sup>2</sup>)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a sixteen bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

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

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

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

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically, one-meter length rods with an outer diameter of 1.5 inches (38.1 millimeters) are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

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

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

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

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

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

where:  $q_t$  is the corrected tip resistance

$q_c$  is the recorded tip resistance

$u_2$  is the recorded dynamic pore pressure behind the tip ( $u_2$  position)

$a$  is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

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

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

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

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

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

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

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

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

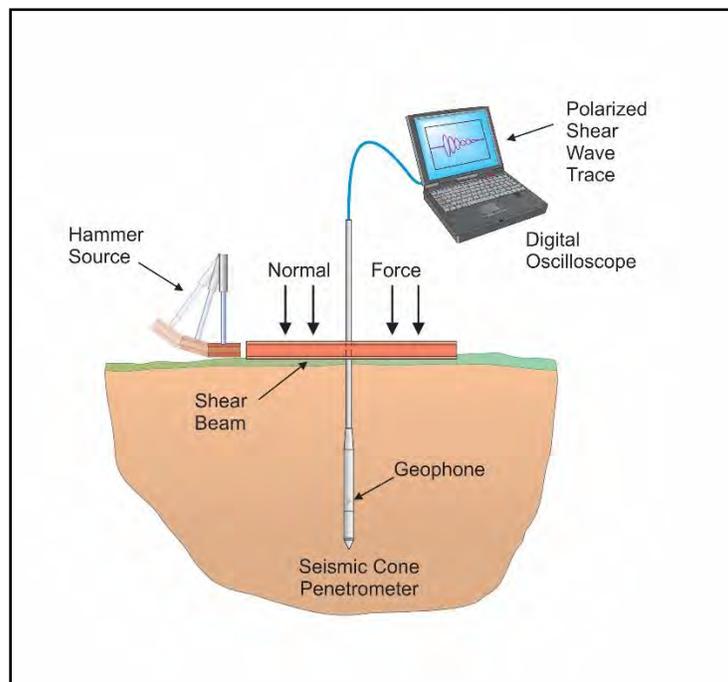


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current [ASTM D5778](#) and [ASTM D7400](#) standards.

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

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

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

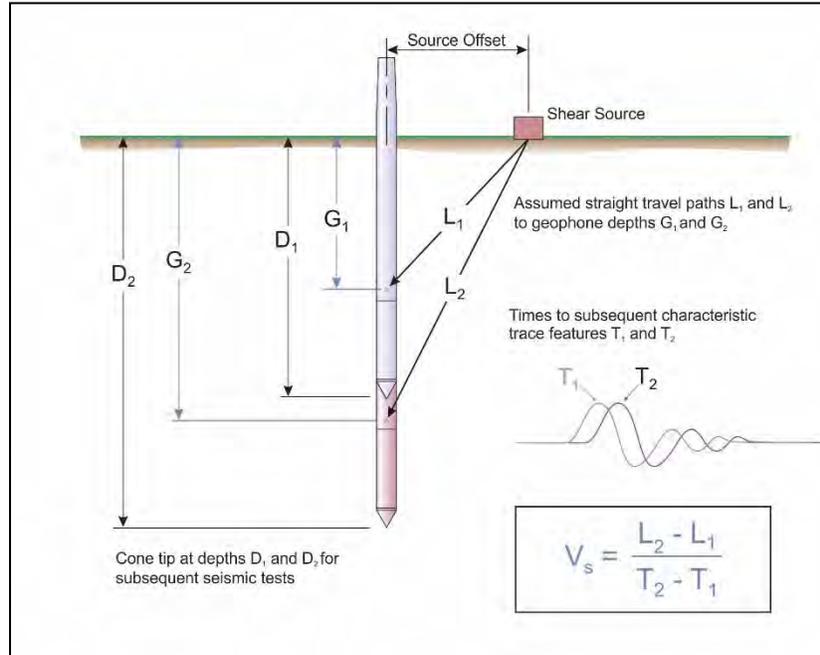


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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

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

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

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

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

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

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

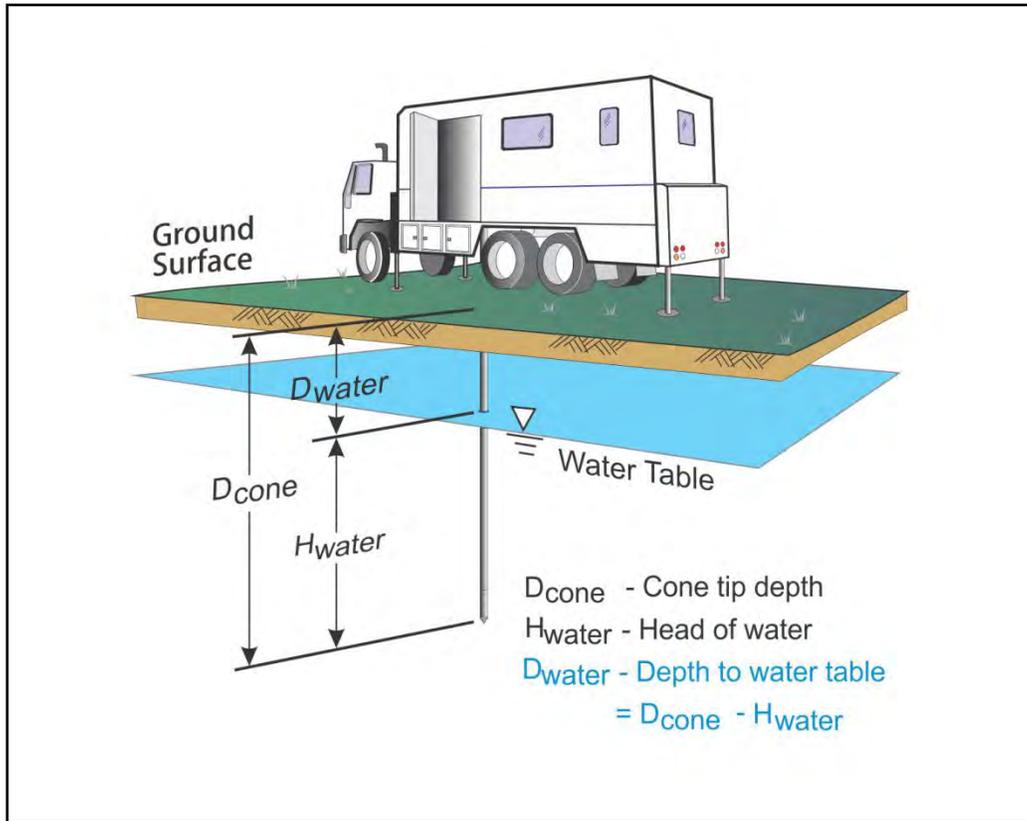


Figure PPD-1. Pore pressure dissipation test setup

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

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

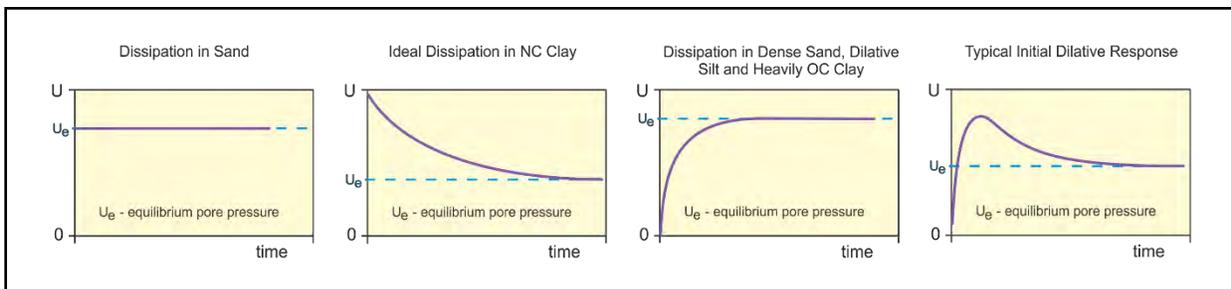


Figure PPD-2. Pore pressure dissipation curve examples

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

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

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

Where:

- $T^*$  is the dimensionless time factor ([Table Time Factor](#))
- $a$  is the radius of the cone
- $I_r$  is the rigidity index
- $t$  is the time at the degree of consolidation

Table Time Factor.  $T^*$  versus degree of dissipation ([Teh and Houlsby \(1991\)](#))

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

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

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

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

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

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

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

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The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with  $I_c$ ,  $\Phi$ ,  $S_u(Nkt)$ , and  $N1(60)I_c$
- Soil Behavior Type (SBT) Zone Scatter Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Traces
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

# Cone Penetration Test Summary and Standard Cone Penetration Test Plots



Job No: 20-56-20953  
Client: Alan Kropp & Associates  
Project: El Portal  
Start Date: 11-Jun-2020  
End Date: 12-Jun-2020

### CONE PENETRATION TEST SUMMARY

| Sounding ID | File Name        | Date        | Cone             | Assumed Phreatic Surface <sup>1</sup> (ft) | Final Depth (ft) | Northing <sup>2</sup> (m) | Easting <sup>2</sup> (m) | Elevation <sup>3</sup> (ft) | Refer to Notation Number |
|-------------|------------------|-------------|------------------|--|------------------|---------------------------|--------------------------|-----------------------------|--------------------------|
| CPT-01      | 20-56-20953_CP01 | 11-Jun-2020 | 496:T1500F15U1K  | 17.9                                       | 51.59            | 4202429                   | 558113                   | 57                          |                          |
| CPT-02      | 20-56-20953_CP02 | 11-Jun-2020 | 496:T1500F15U1K  | 5.4  | 51.43            | 4202448                   | 558156                   | 59                          |                          |
| CPT-03      | 20-56-20953_CP03 | 11-Jun-2020 | 496:T1500F15U1K  | 5.0  | 51.10            | 4202467                   | 558221                   | 62                          |                          |
| CPT-04      | 20-56-20953_CP04 | 11-Jun-2020 | 496:T1500F15U1K  | 18.9                                       | 51.92            | 4202401                   | 558117                   | 59                          |                          |
| CPT-05      | 20-56-20953_SP05 | 12-Jun-2020 | 383:T1500F15U500 | 9.4  | 44.62            | 4202400                   | 558173                   | 61                          |                          |
| CPT-06      | 20-56-20953_CP06 | 12-Jun-2020 | 383:T1500F15U500 | 9.4  | 50.52            | 4202396                   | 558190                   | 61                          | 4                        |
| CPT-07      | 20-56-20953_CP07 | 12-Jun-2020 | 383:T1500F15U500 | 20.6                                       | 50.69            | 4202346                   | 558165                   | 61                          |                          |
| CPT-08      | 20-56-20953_CP08 | 12-Jun-2020 | 383:T1500F15U500 | >44.3                                      | 44.29            | 4202357                   | 558186                   | 61                          | 5                        |
| CPT-09      | 20-56-20953_CP09 | 12-Jun-2020 | 383:T1500F15U500 | 20.0                                       | 50.77            | 4202335                   | 558166                   | 60                          |                          |

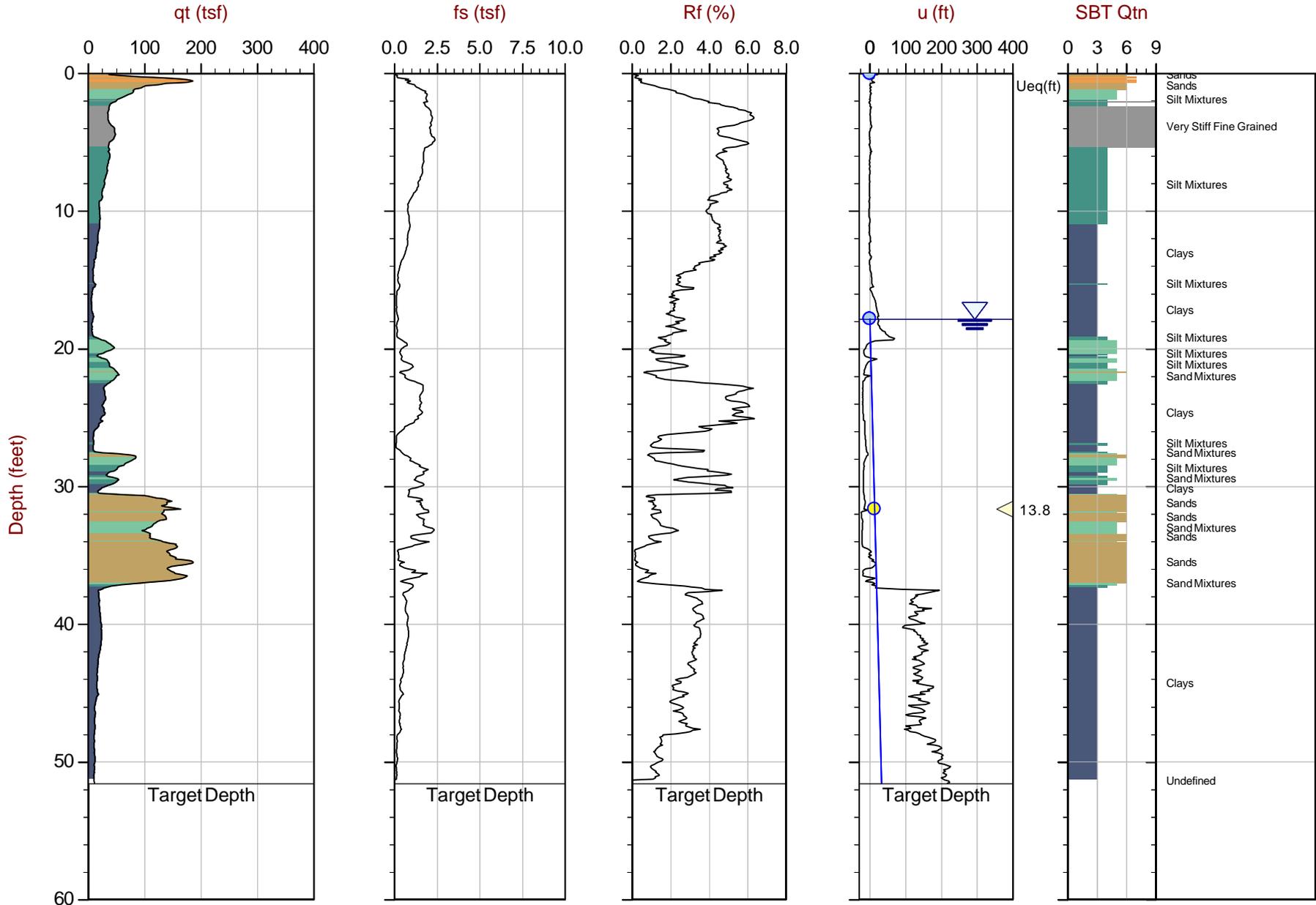
1. The assumed phreatic surface was based on the results of the shallowest pore pressure dissipation test performed within the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 10 North.
3. Elevations are referenced to the ground surface and are derived from the Google Earth Elevation for the recorded coordinates.
4. The assumed phreatic surface is based on the pore pressure dissipation test at a nearby sounding.
5. The pore pressure dissipation test in the sounding was showing the sounding was dry at the end of the test.



# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-11 10:03  
Site: El Portal

Sounding: CPT-01  
Cone: 496:T1500F15U1K



Max Depth: 15.725 m / 51.59 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP01.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202429m E: 558113m

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

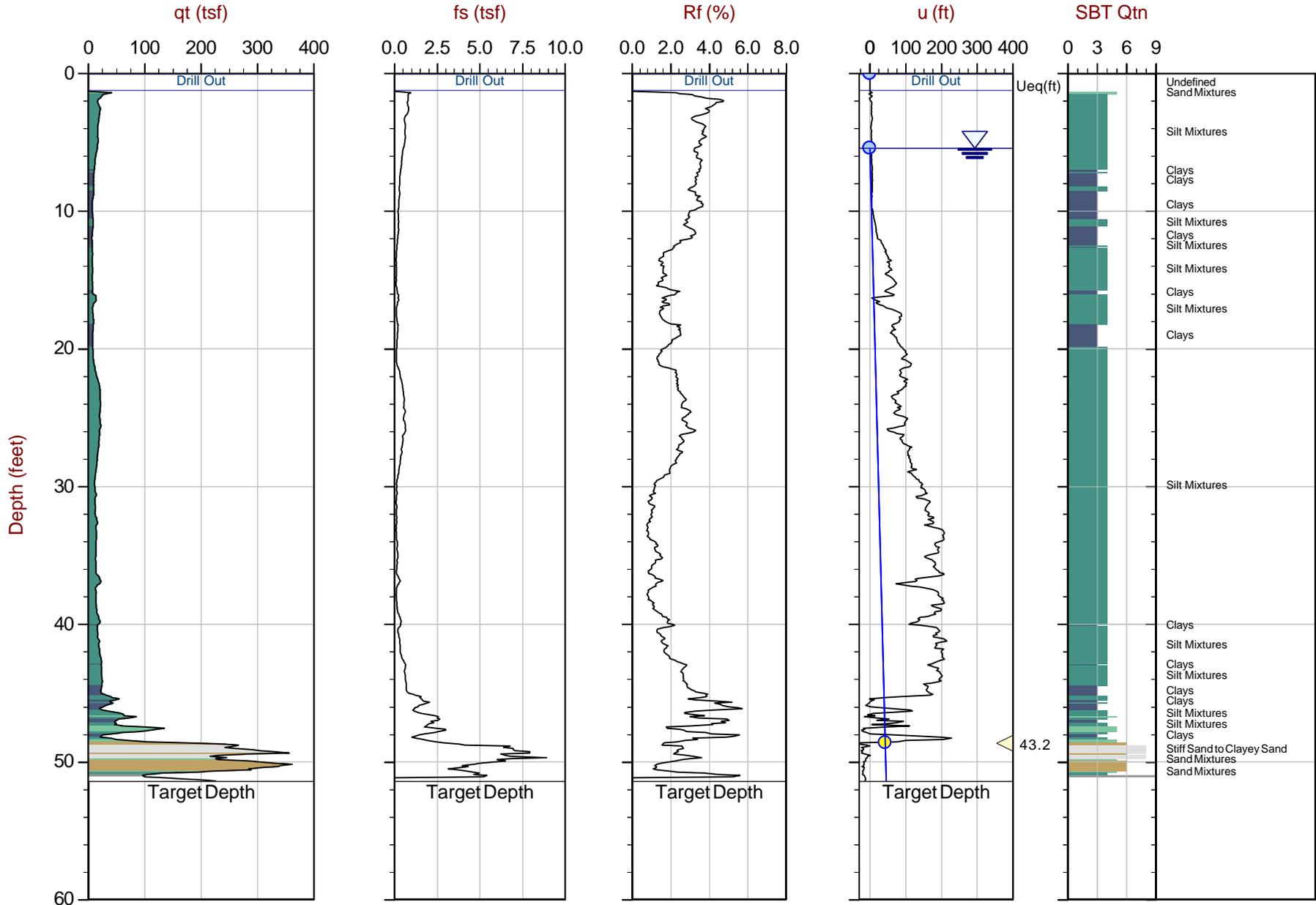
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-11 11:31  
Site: El Portal

Sounding: CPT-02  
Cone: 496:T1500F15U1K



Max Depth: 15.675 m / 51.43 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP02.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202448m E: 558156m

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▼ Dissipation, Ueq not achieved    — Hydrostatic Line

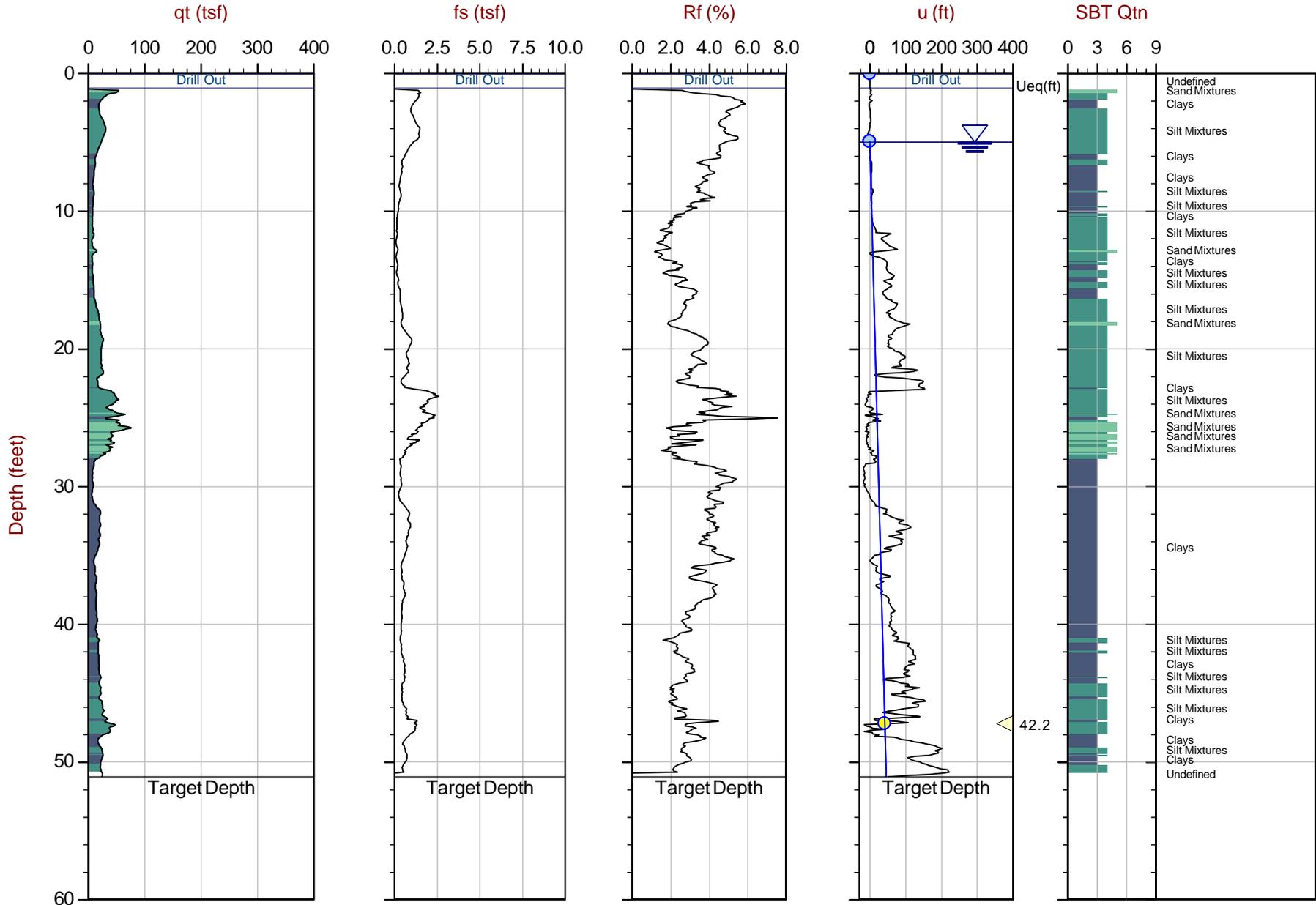
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-11 08:12  
Site: El Portal

Sounding: CPT-03  
Cone: 496:T1500F15U1K



Max Depth: 15.575 m / 51.10 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP03.COR  
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202467m E: 558221m

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

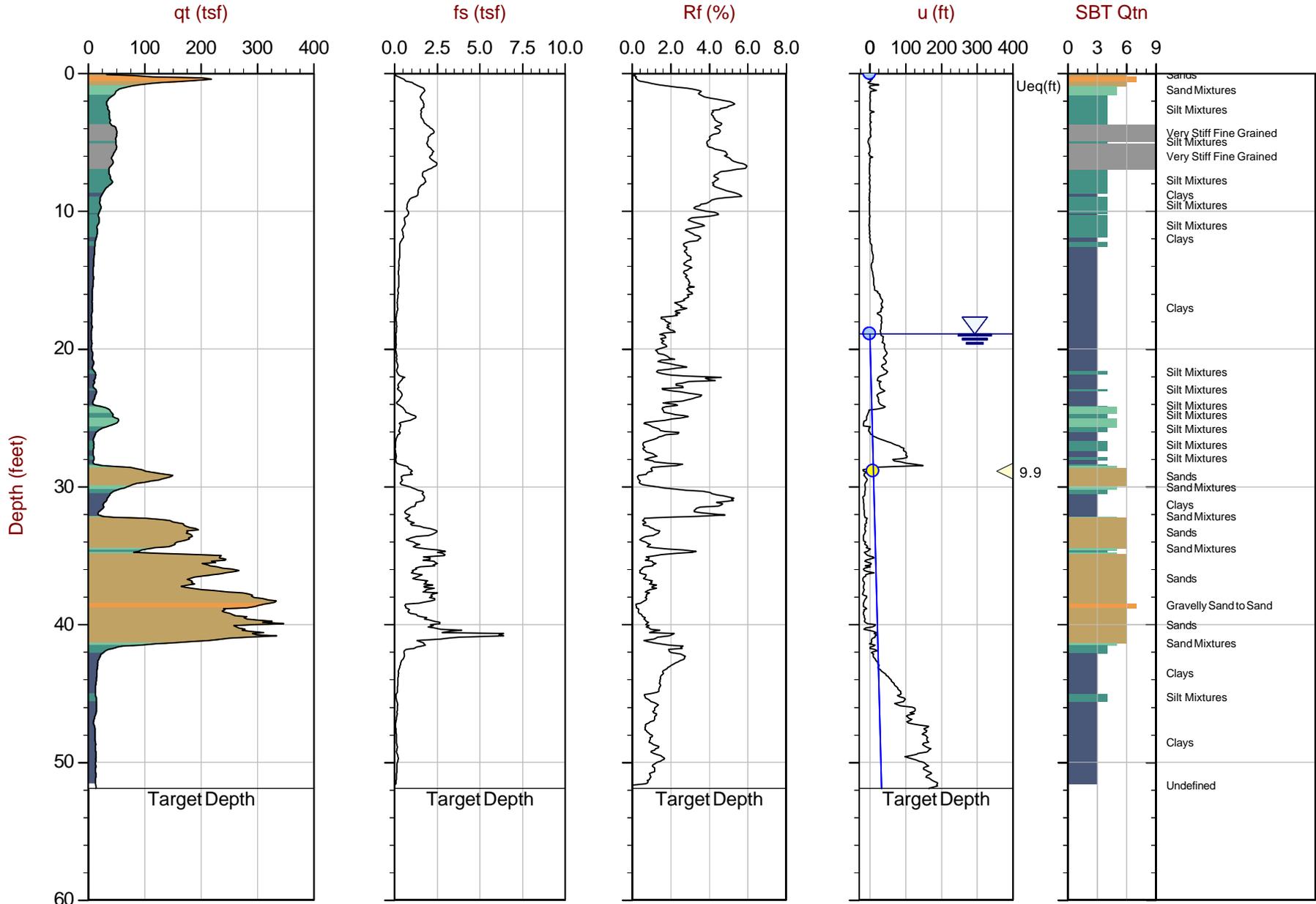
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-11 10:46  
Site: El Portal

Sounding: CPT-04  
Cone: 496:T1500F15U1K



Max Depth: 15.825 m / 51.92 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP04.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202401m E: 558117m

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

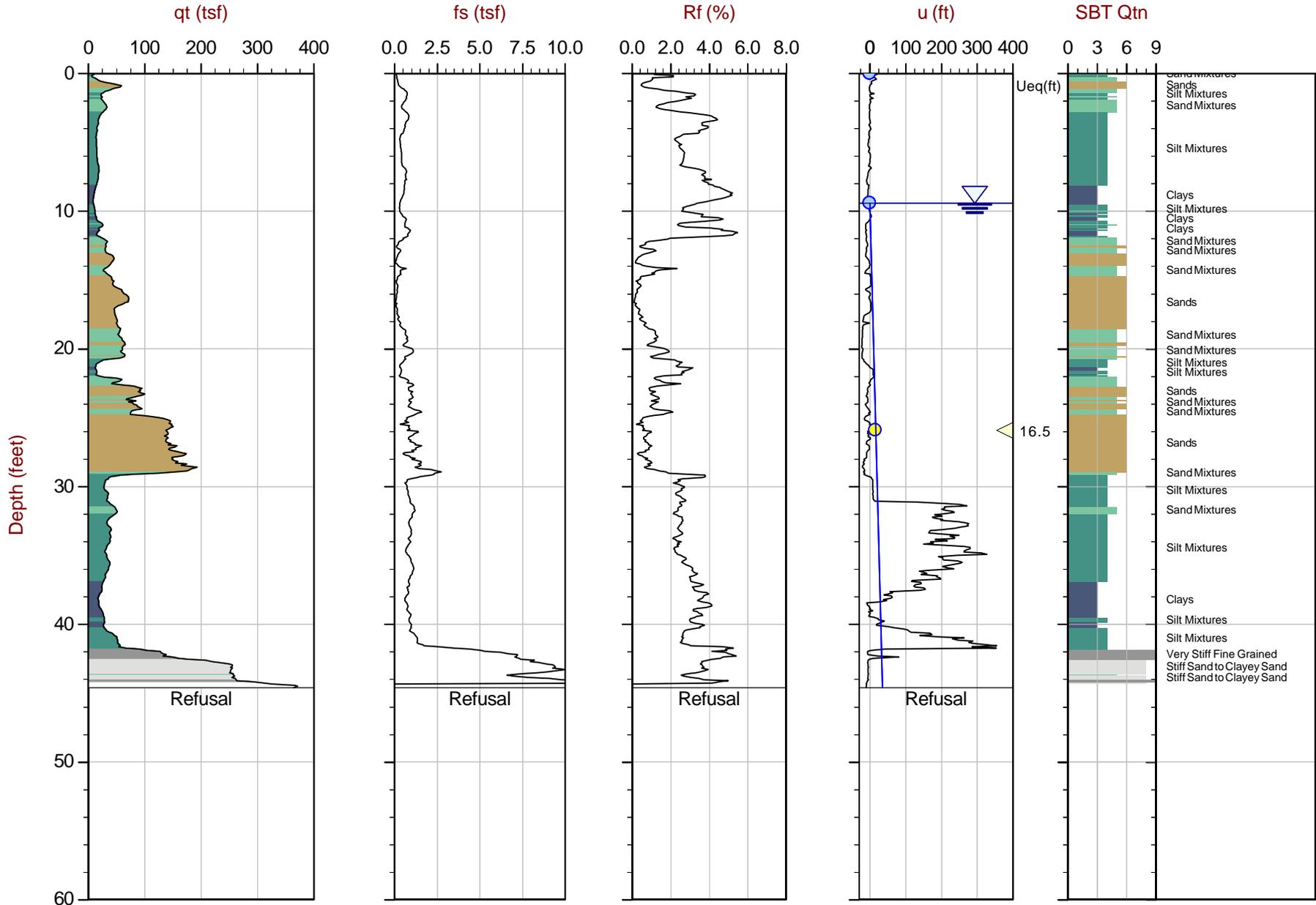
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-12 07:06  
Site: El Portal

Sounding: CPT-05  
Cone: 383:T1500F15U500



Max Depth: 13.600 m / 44.62 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_SP05.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202400m E: 558173m

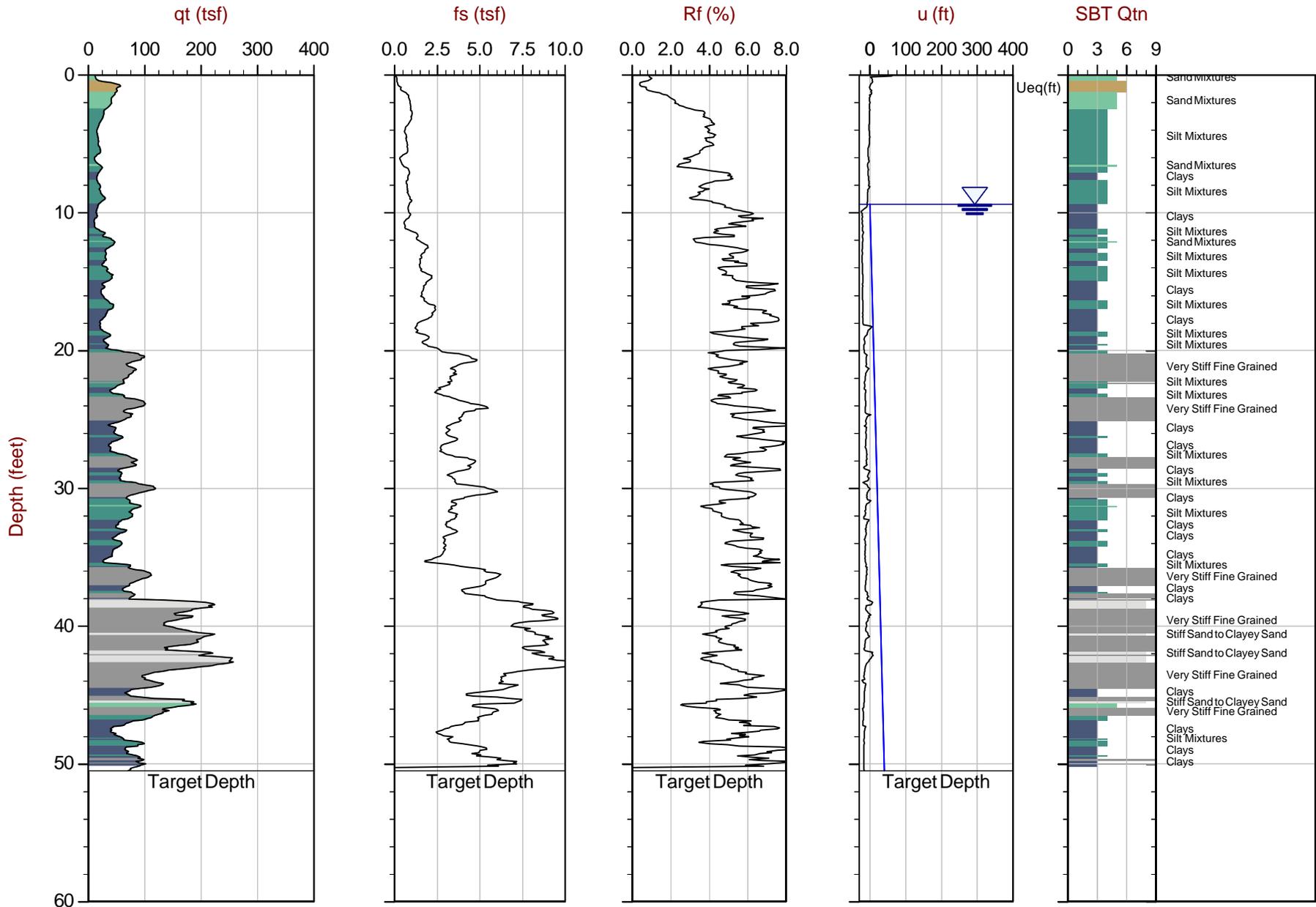
● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ◀ Dissipation, Ueq achieved    ◀ Dissipation, Ueq not achieved    — Hydrostatic Line  
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-12 08:38  
Site: El Portal

Sounding: CPT-06  
Cone: 383:T1500F15U500



Max Depth: 15.400 m / 50.52 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP06.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202396m E: 558190m

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

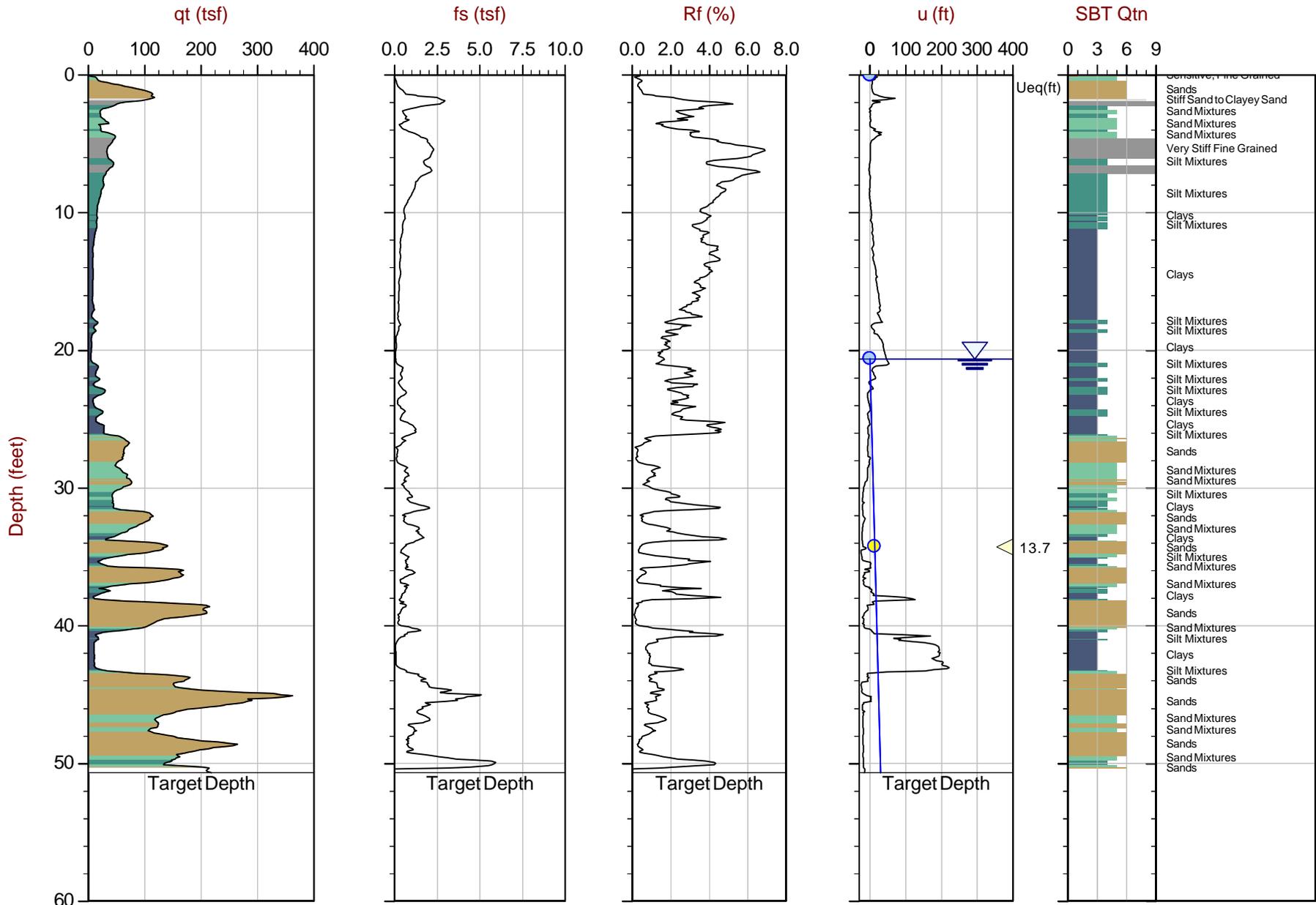
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-12 12:13  
Site: El Portal

Sounding: CPT-07  
Cone: 383:T1500F15U500



Max Depth: 15.450 m / 50.69 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP07.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202346m E: 558165m

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

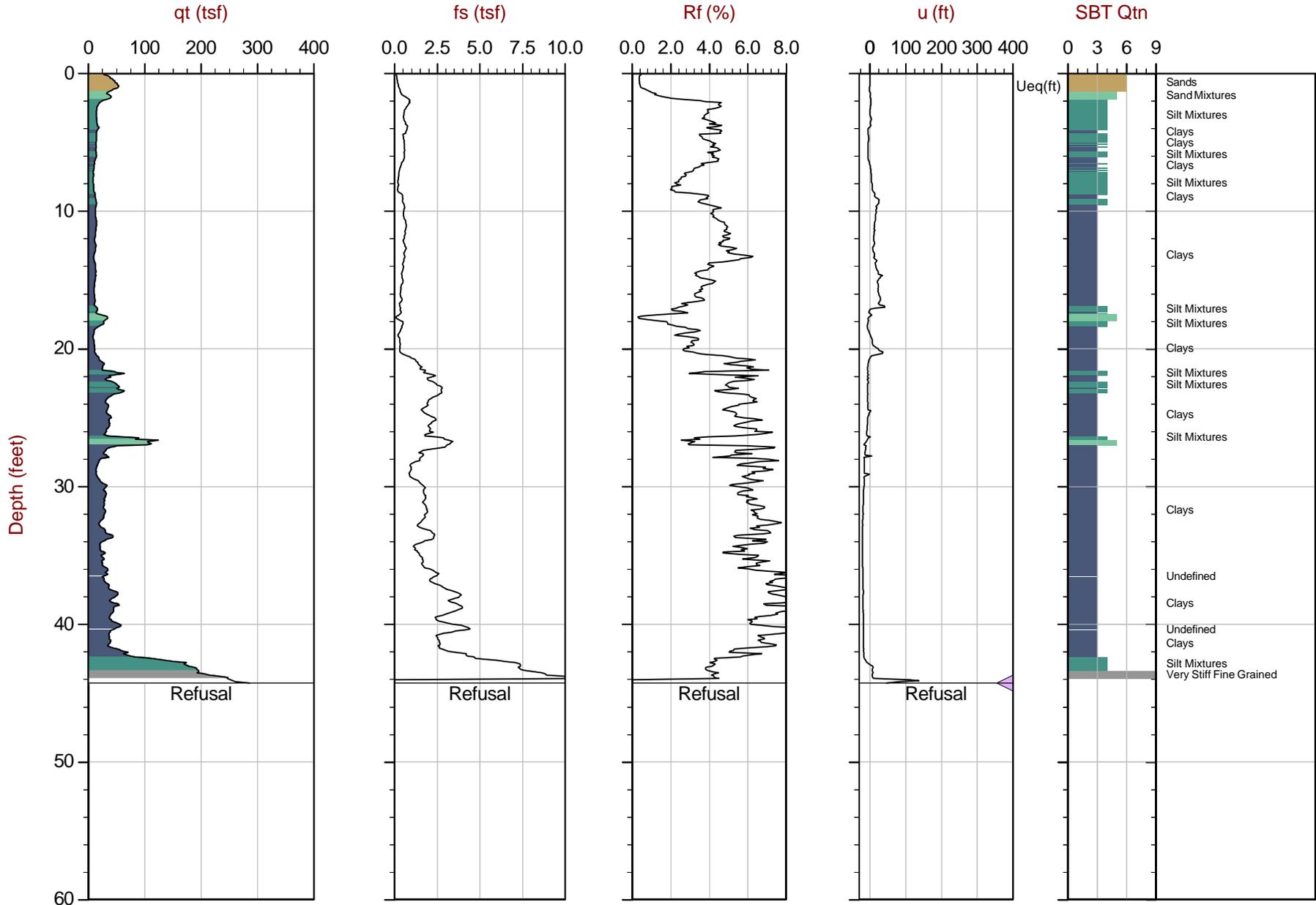
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-12 09:41  
Site: El Portal

Sounding: CPT-08  
Cone: 383:T1500F15U500



Max Depth: 13.500 m / 44.29 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP08.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202357m E: 558186m

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

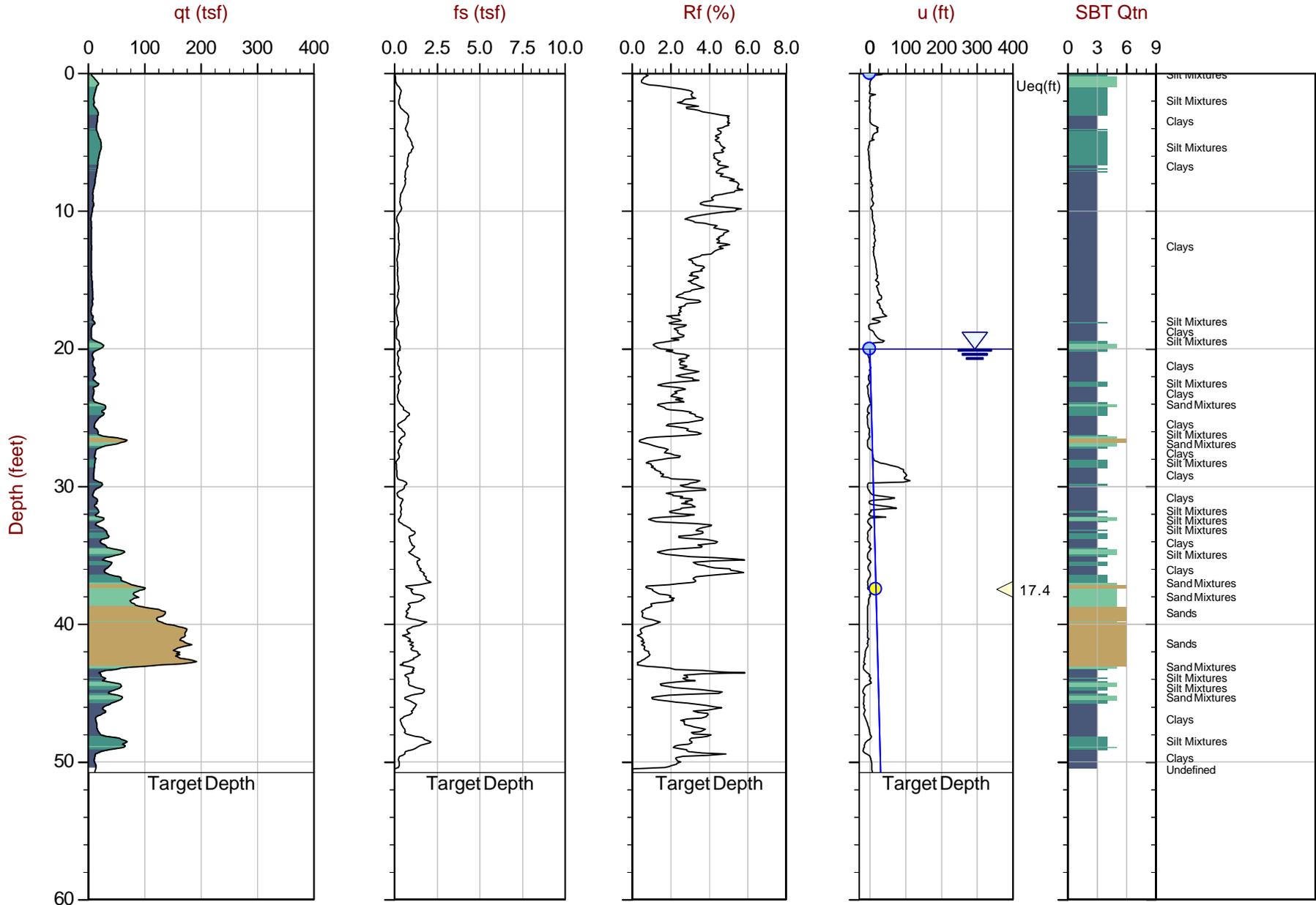
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-12 10:40  
Site: El Portal

Sounding: CPT-09  
Cone: 383:T1500F15U500



Max Depth: 15.475 m / 50.77 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP09.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202335m E: 558166m

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

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

Advanced Cone Penetration Test Plots with  $I_c$ ,  $\Phi$ ,  $S_u(N_{kt})$ , and  $N1(60)I_c$

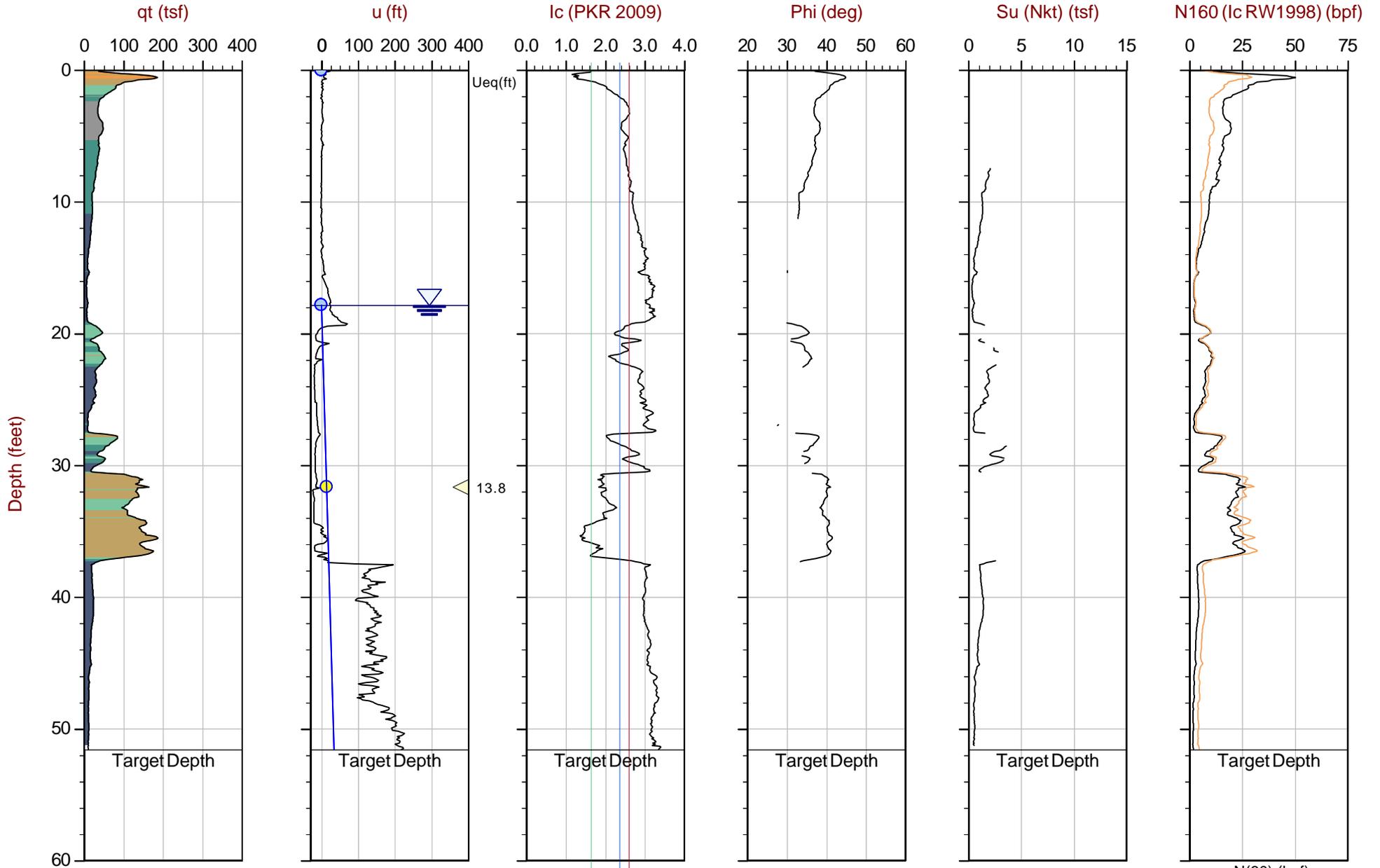




# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-11 10:03  
Site: El Portal

Sounding: CPT-01  
Cone: 496:T1500F15U1K



Max Depth: 15.725 m / 51.59 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP01.COR  
Unit Wt: SBTQtn(PKR2009)  
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202429m E: 558113m

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

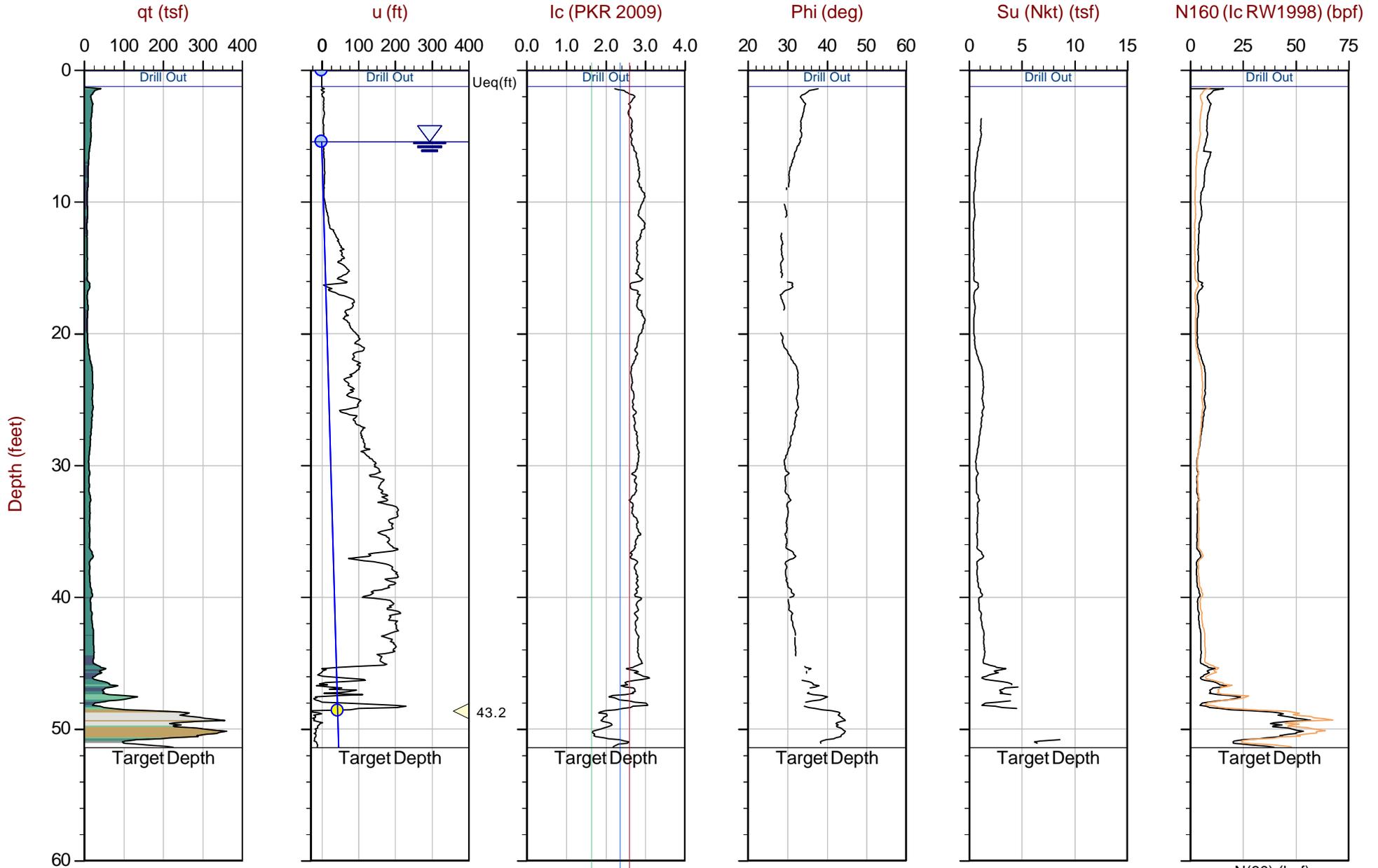
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-11 11:31  
Site: El Portal

Sounding: CPT-02  
Cone: 496:T1500F15U1K



Max Depth: 15.675 m / 51.43 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP02.COR  
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Su Nkt: 15.0

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202448m E: 558156m

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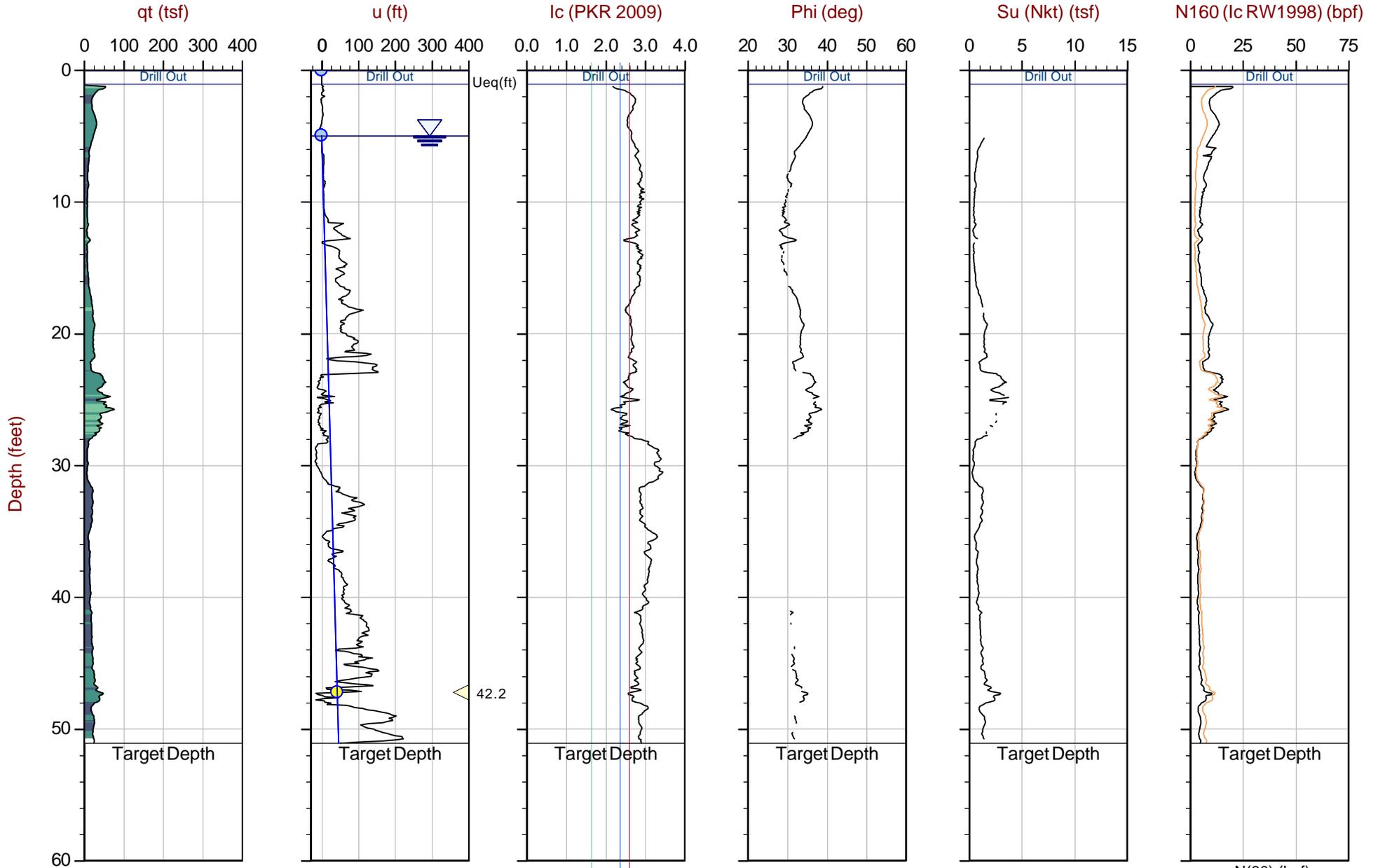
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# Alan Kropp & Associates

Job No: 20-56-20953  
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Site: El Portal

Sounding: CPT-03  
Cone: 496:T1500F15U1K



Max Depth: 15.575 m / 51.10 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP03.COR  
Unit Wt: SBTQtn(PKR2009)  
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202467m E: 558221m

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

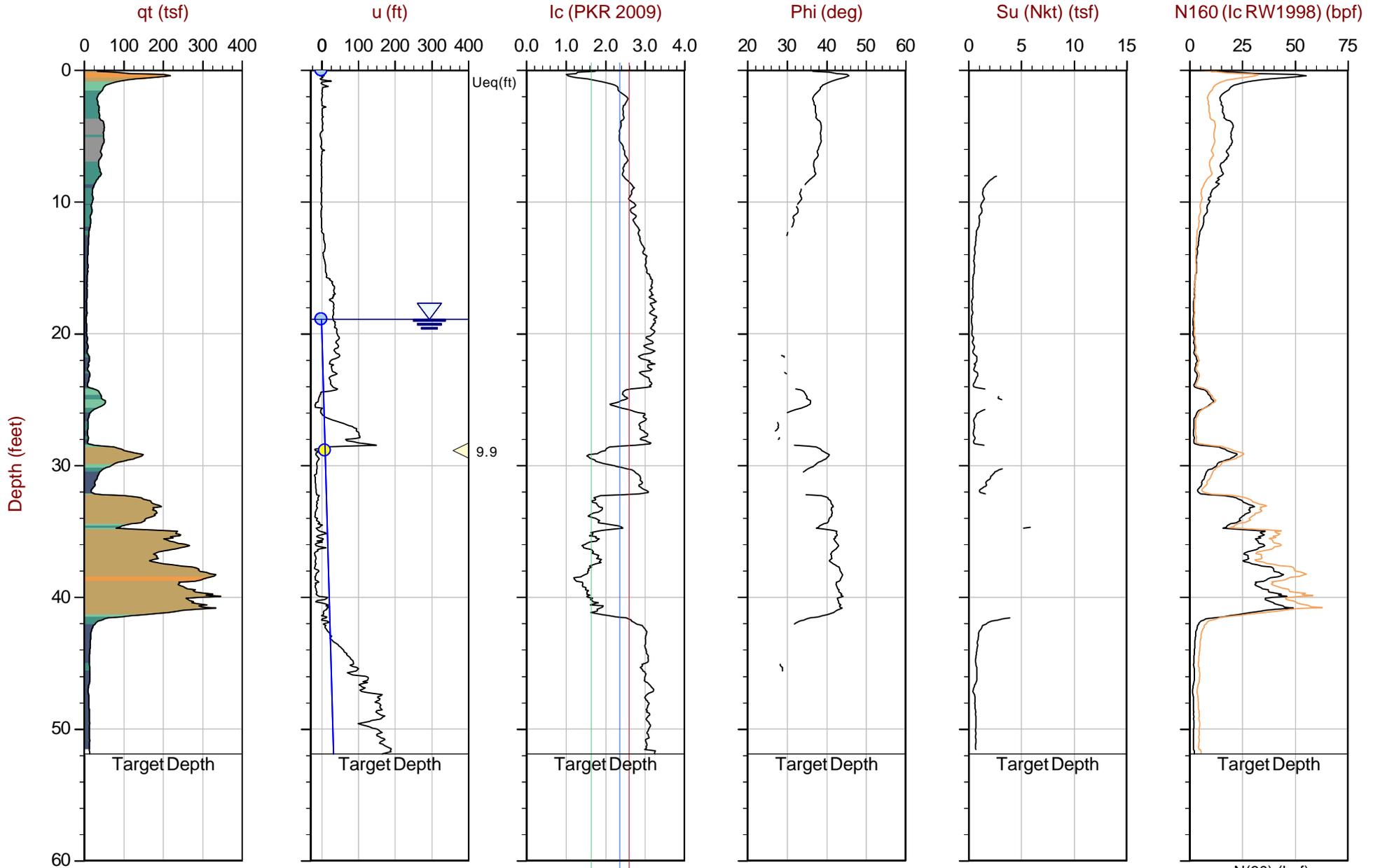
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-11 10:46  
Site: El Portal

Sounding: CPT-04  
Cone: 496:T1500F15U1K



Max Depth: 15.825 m / 51.92 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP04.COR  
Unit Wt: SBTQtn(PKR2009)  
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202401m E: 558117m

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

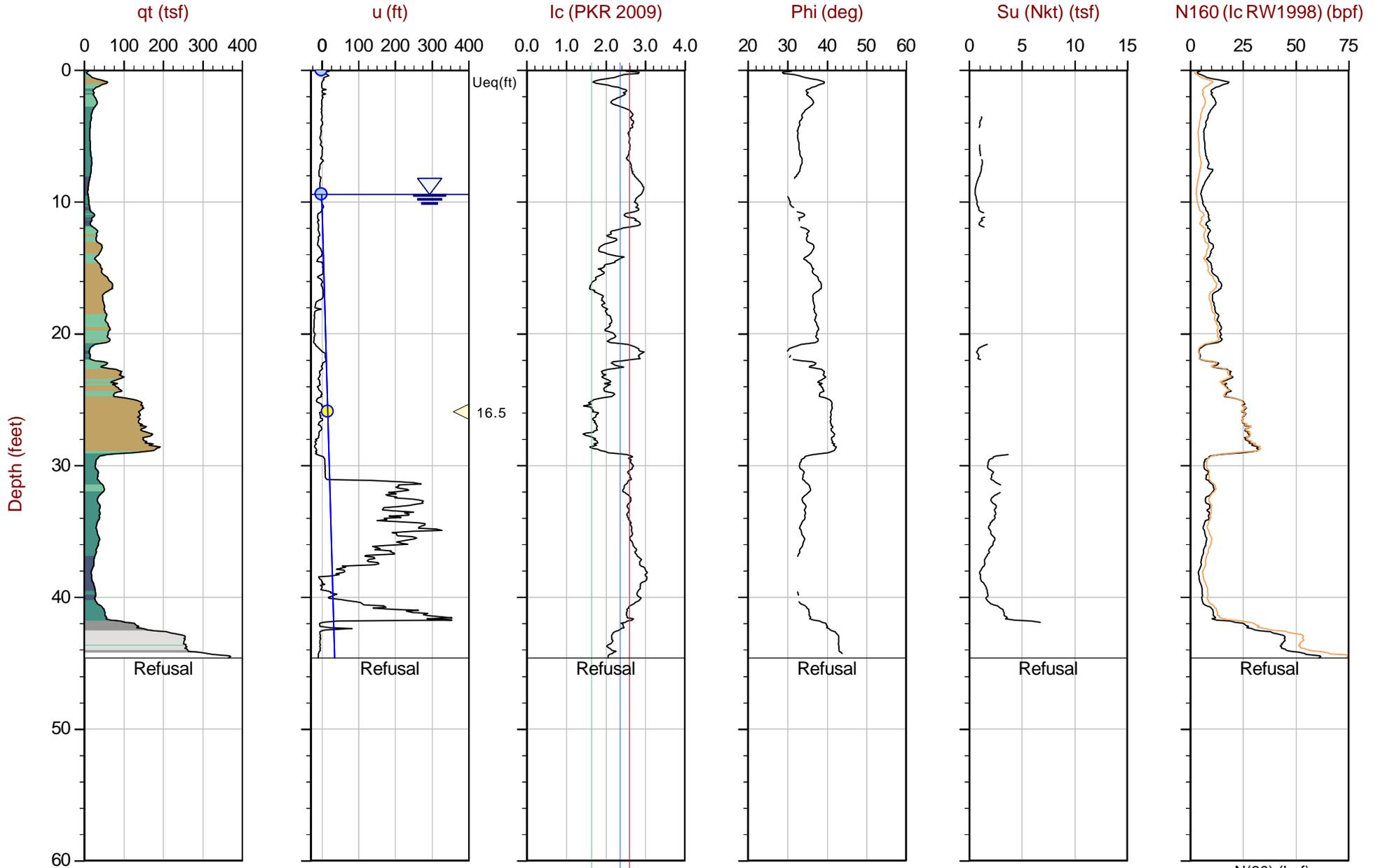
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-12 07:06  
Site: El Portal

Sounding: CPT-05  
Cone: 383:T1500F15U500



Max Depth: 13.600 m / 44.62 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_SP05.COR  
Unit Wt: SBTQn(PKR2009)  
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202400m E: 558173m

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▼ Dissipation, Ueq not achieved    — Hydrostatic Line

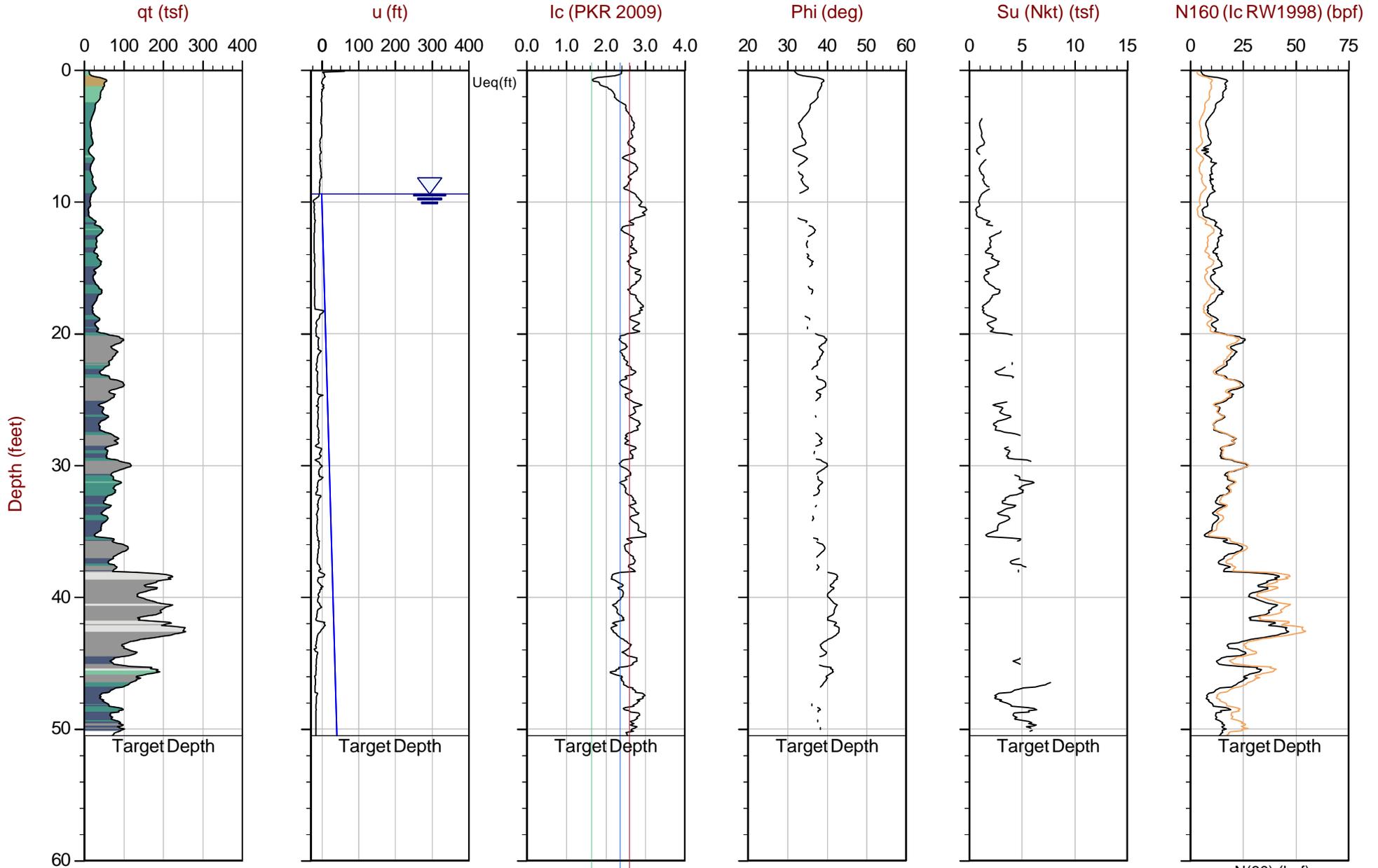
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-12 08:38  
Site: El Portal

Sounding: CPT-06  
Cone: 383:T1500F15U500



Max Depth: 15.400 m / 50.52 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP06.COR  
Unit Wt: SBTQn(PKR2009)  
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202396m E: 558190m

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▼ Dissipation, Ueq not achieved    — Hydrostatic Line

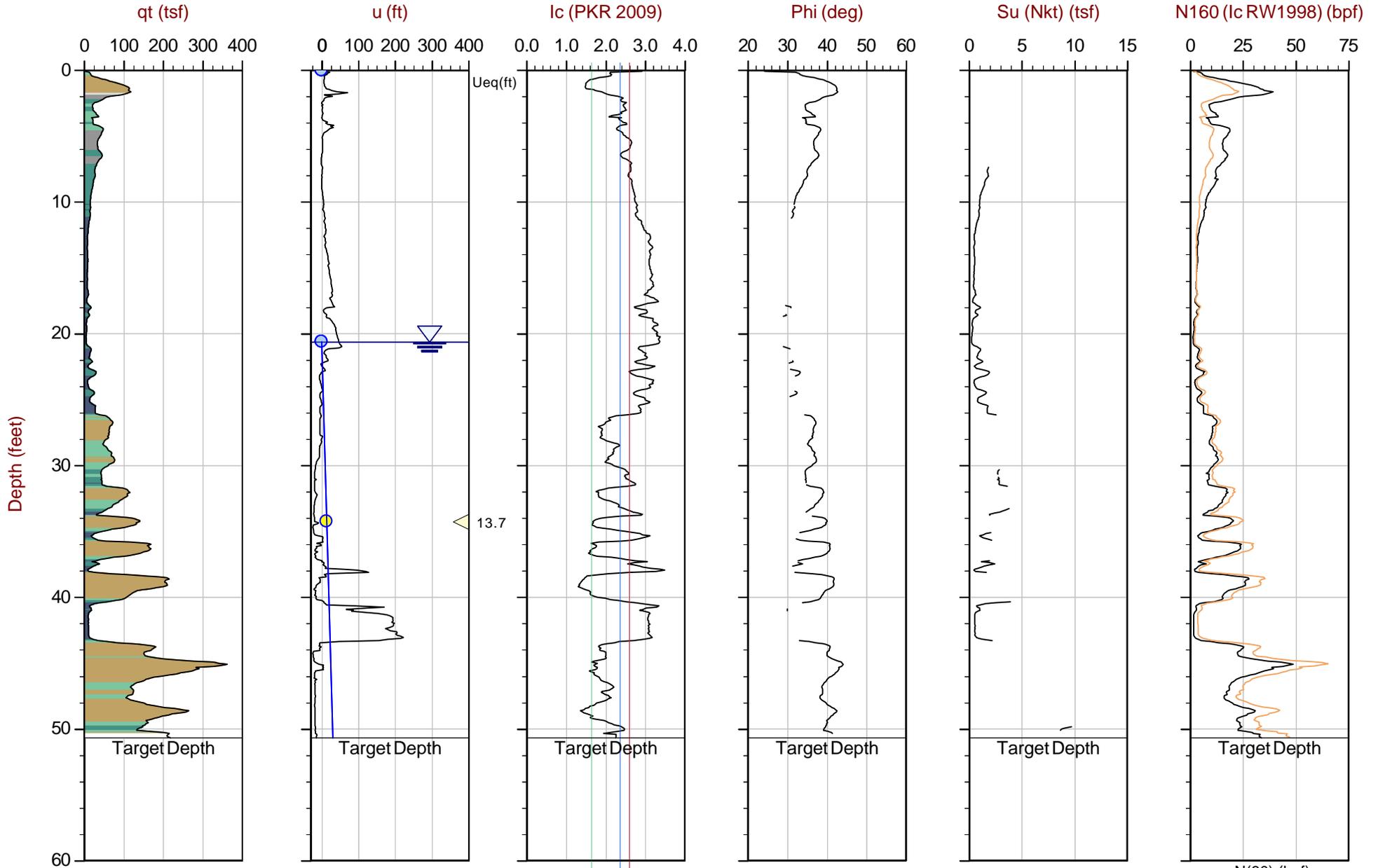
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-12 12:13  
Site: El Portal

Sounding: CPT-07  
Cone: 383:T1500F15U500



Max Depth: 15.450 m / 50.69 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP07.COR  
Unit Wt: SBTQn(PKR2009)  
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202346m E: 558165m

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 ◀ Dissipation, Ueq achieved    
 ◀ Dissipation, Ueq not achieved    
 — Hydrostatic Line

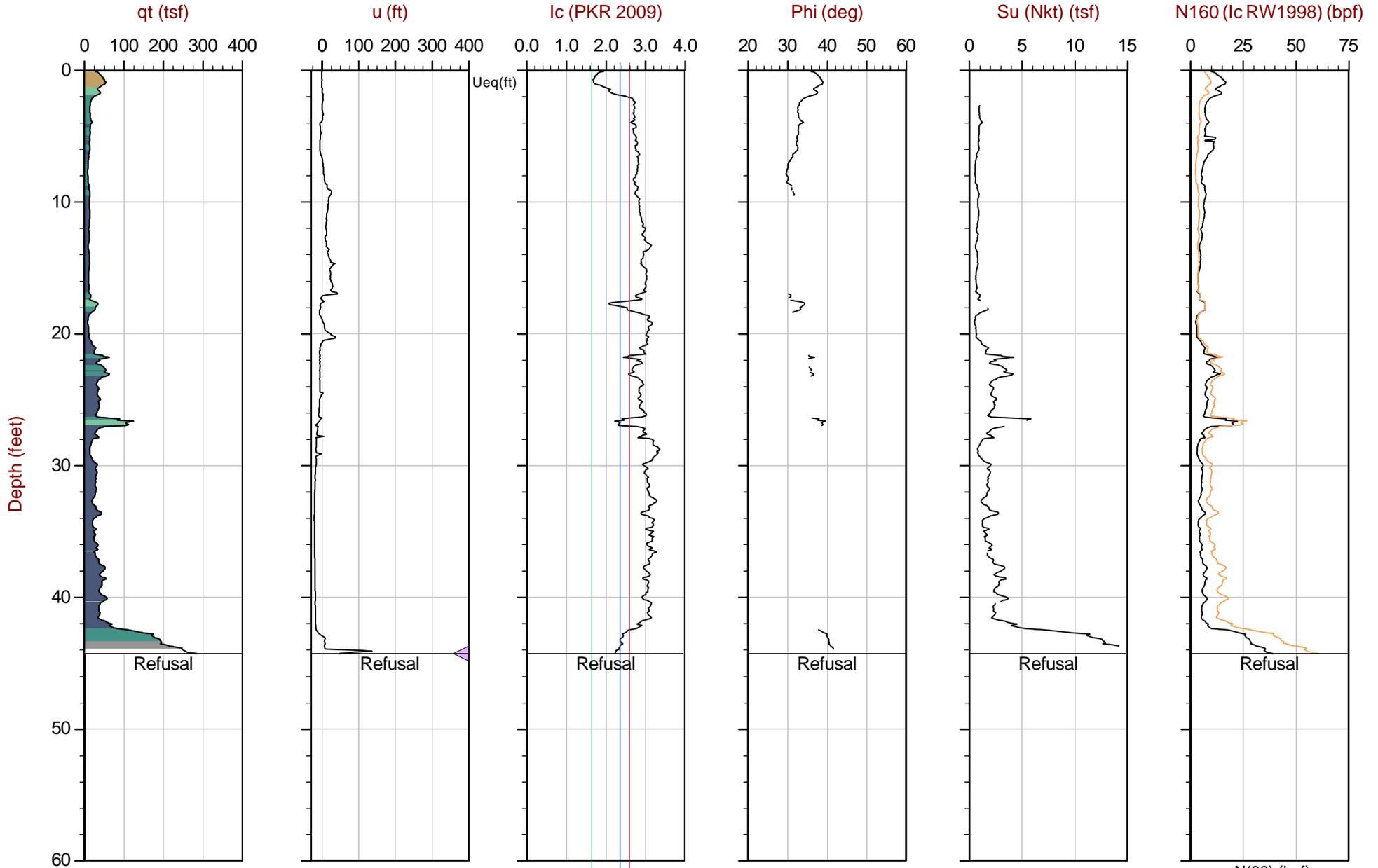
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-12 09:41  
Site: El Portal

Sounding: CPT-08  
Cone: 383:T1500F15U500



Max Depth: 13.500 m / 44.29 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP08.COR  
Unit Wt: SBTQtn (PKR2009)  
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202357m E: 558186m

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▼ Dissipation, Ueq not achieved    — Hydrostatic Line

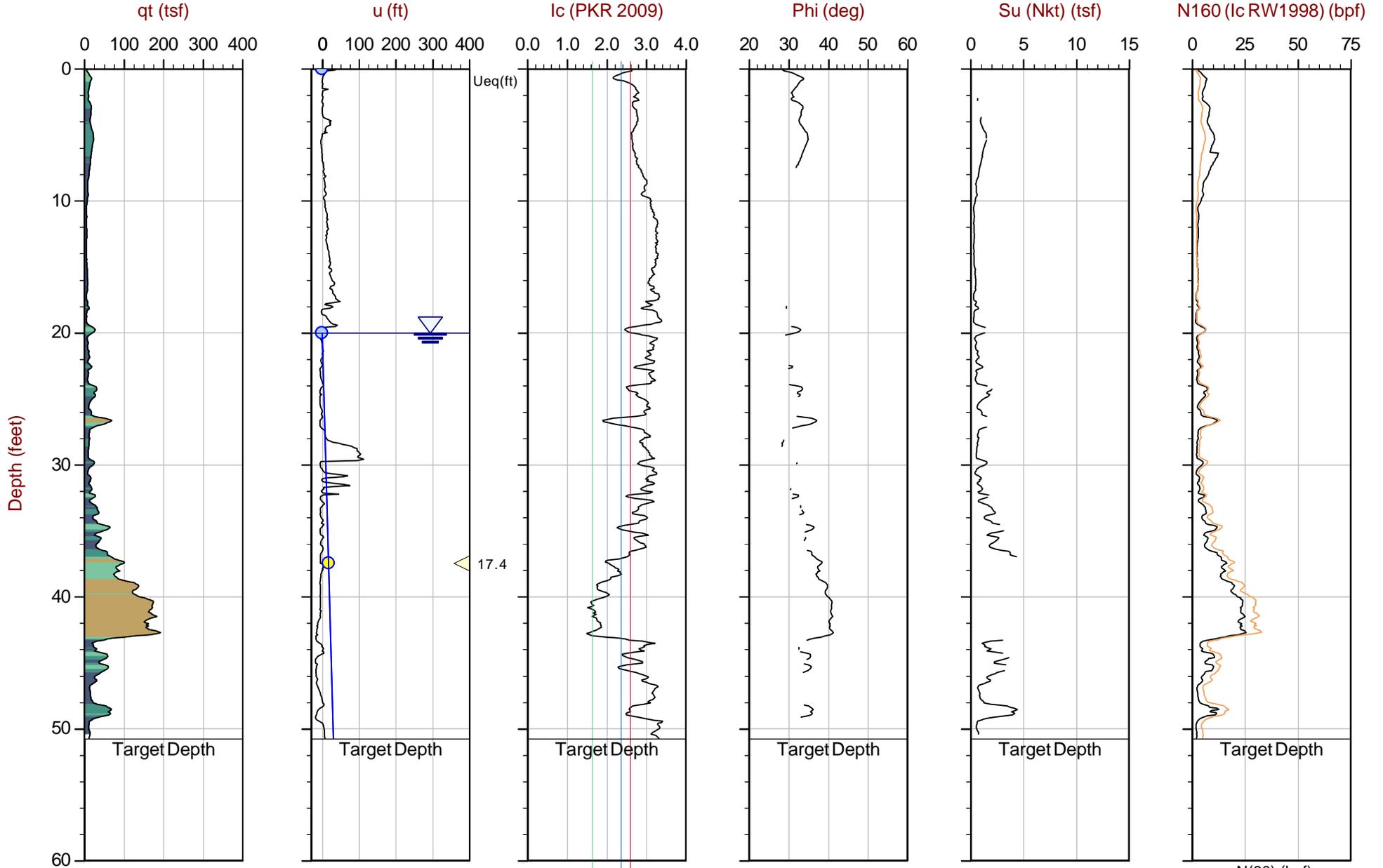
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# Alan Kropp & Associates

Job No: 20-56-20953  
Date: 2020-06-12 10:40  
Site: El Portal

Sounding: CPT-09  
Cone: 383:T1500F15U500



Max Depth: 15.475 m / 50.77 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_CP09.COR  
Unit Wt: SBTQtn(PKR2009)  
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202335m E: 558166m

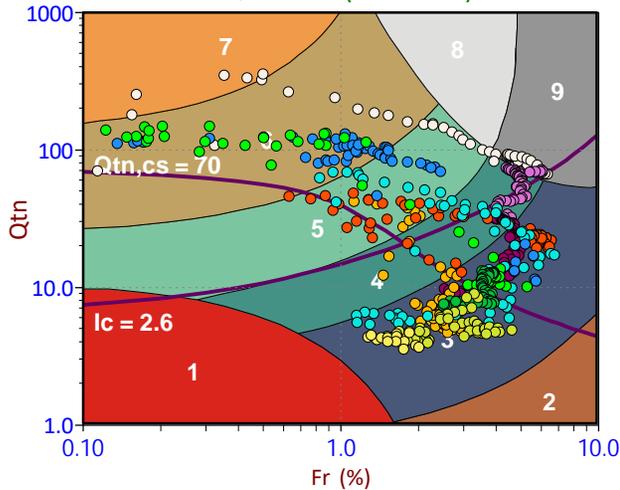
● Equilibrium Pore Pressure (Ueq)      ● Assumed Ueq      ▲ Dissipation, Ueq achieved      ▲ Dissipation, Ueq not achieved      — Hydrostatic Line

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

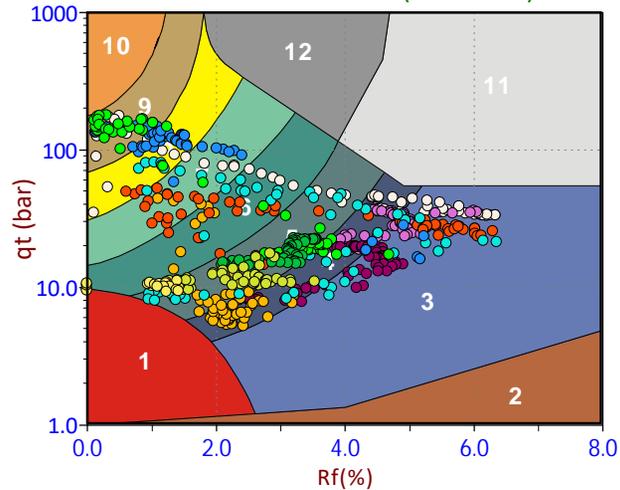
## Soil Behavior Type (SBT) Scatter Plots



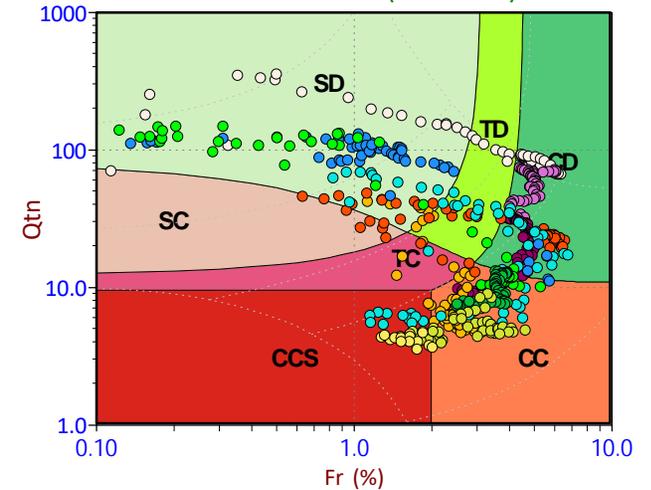
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

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

Legend

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

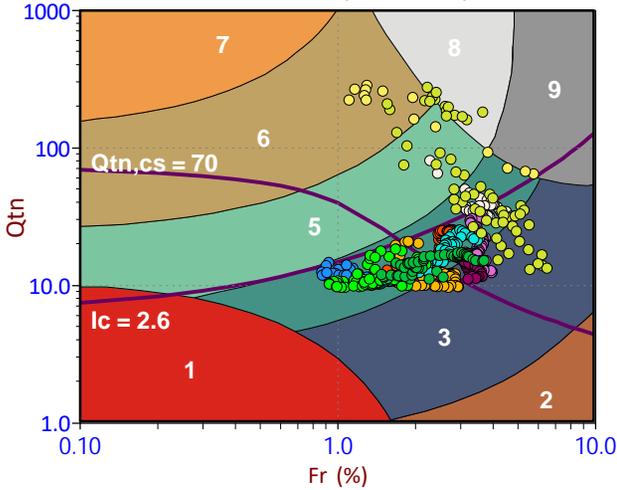
Legend

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

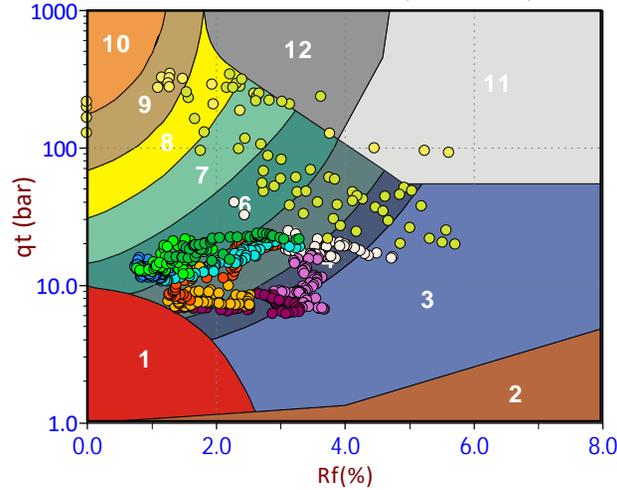
Legend

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

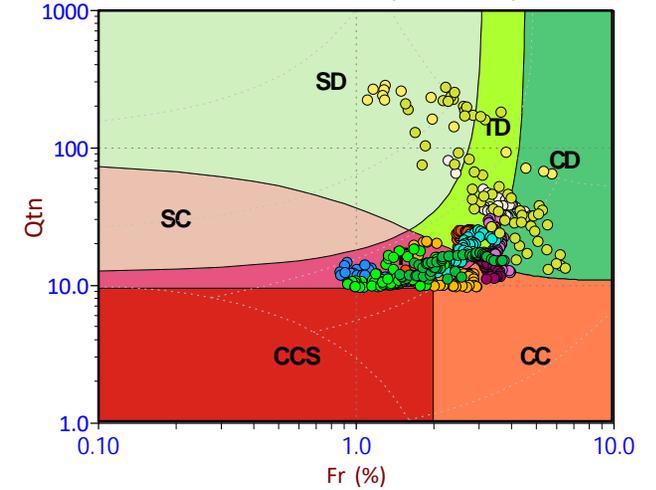
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

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

Legend

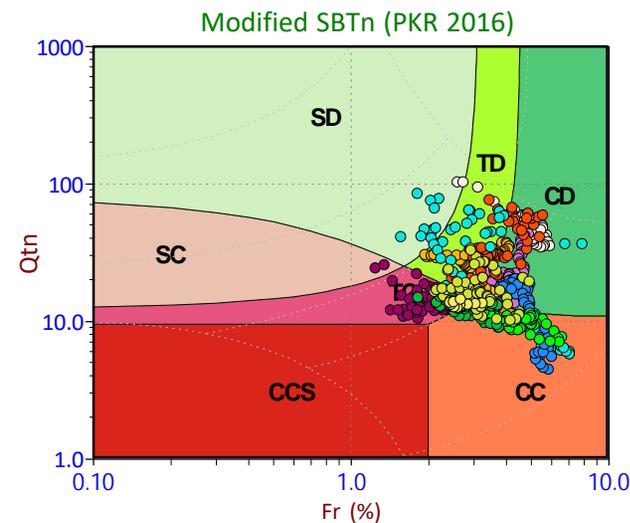
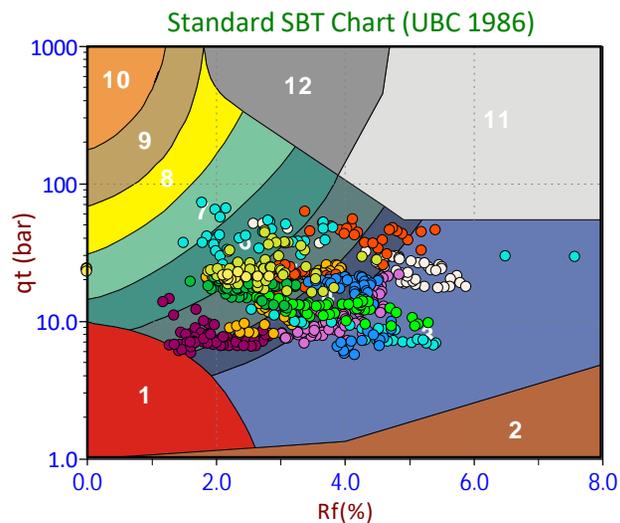
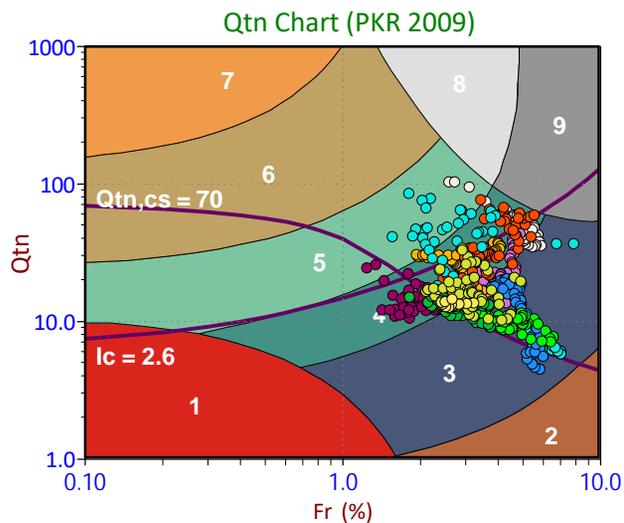
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- Sensitive Fines
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- Silt
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Legend

- CCS (Cont. sensitive clay like)
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- SC (Cont. sand like)
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- SD (Dil. sand like)



#### Depth Ranges

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

#### Legend

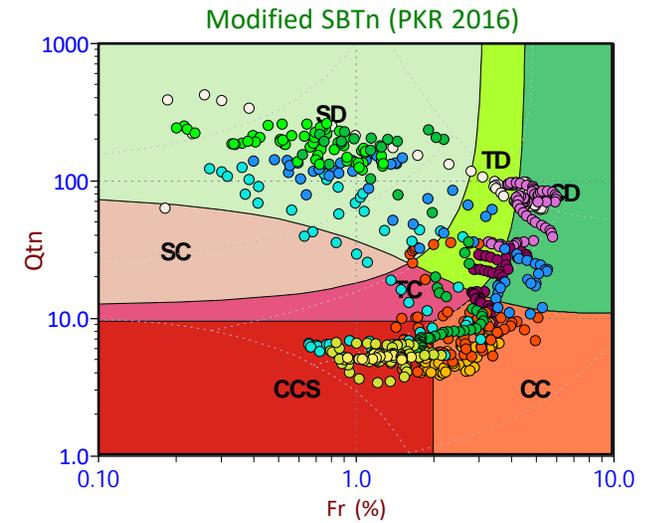
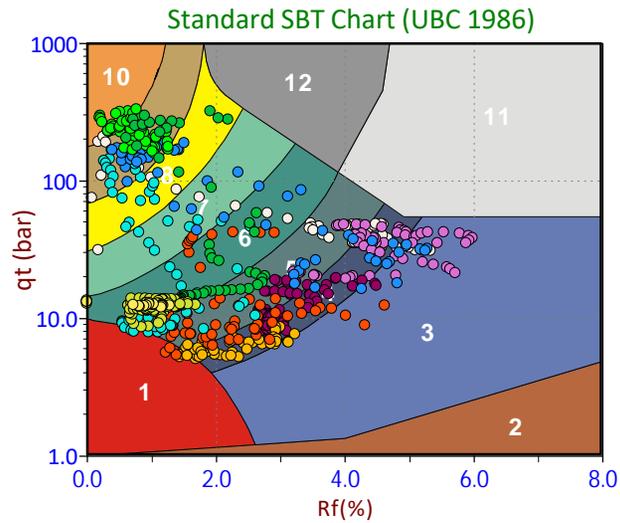
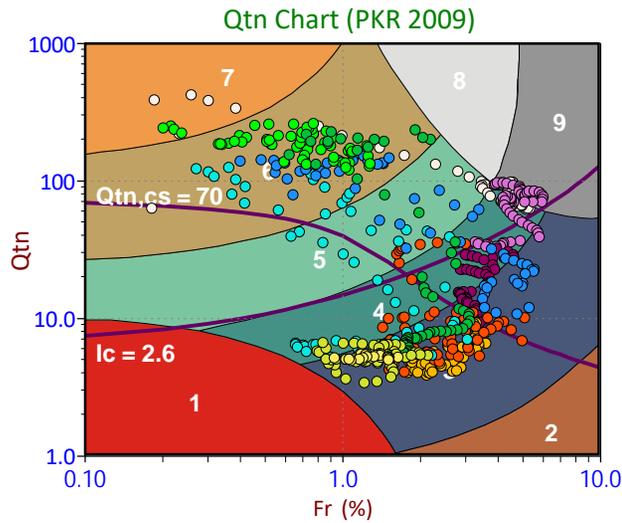
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**Legend**

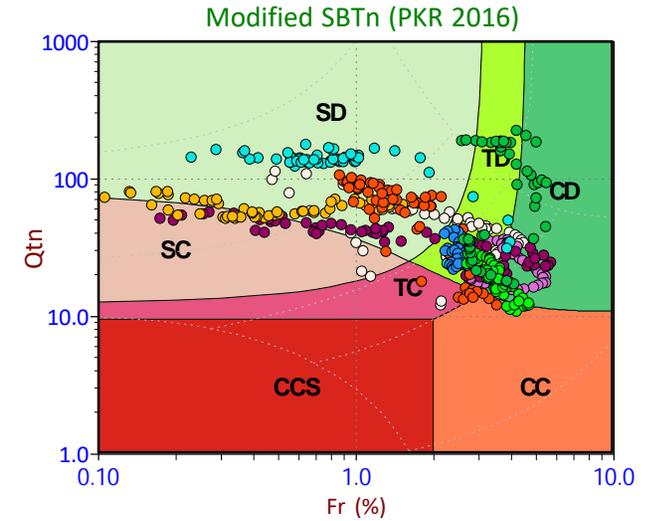
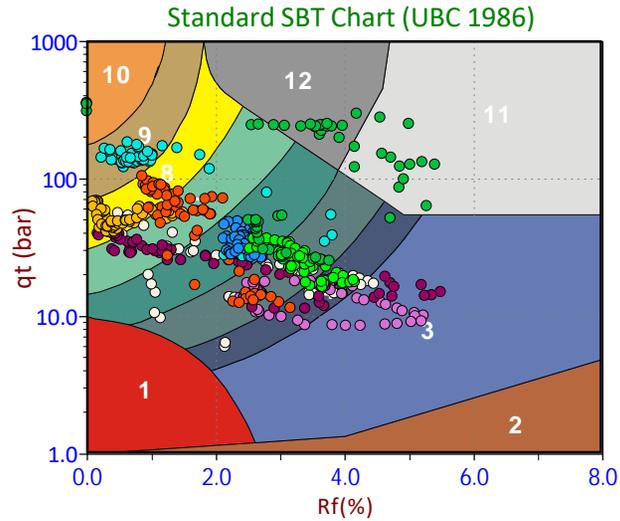
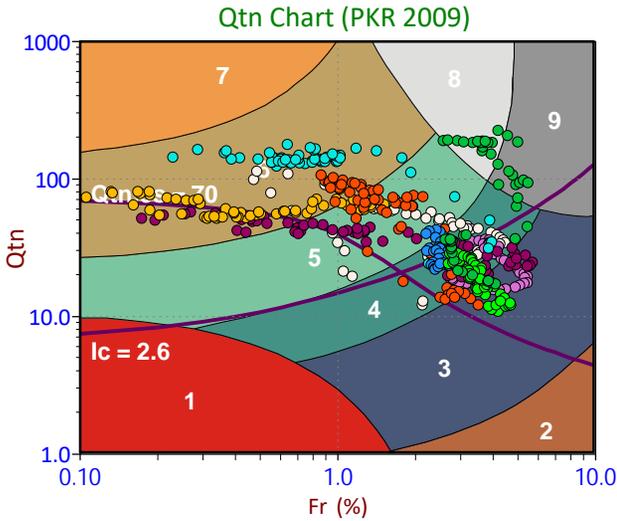
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Legend

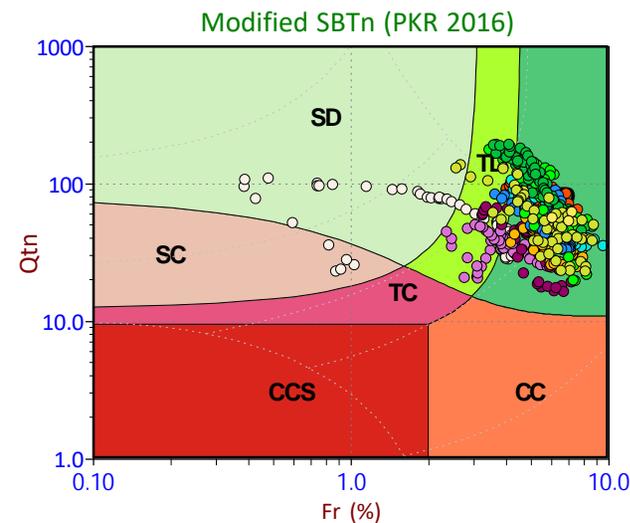
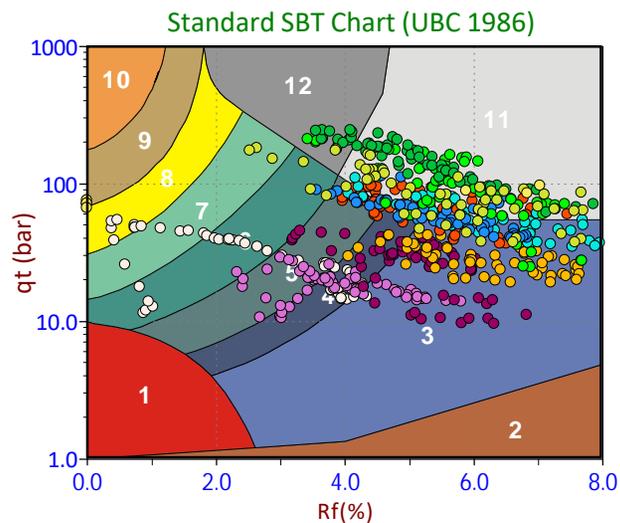
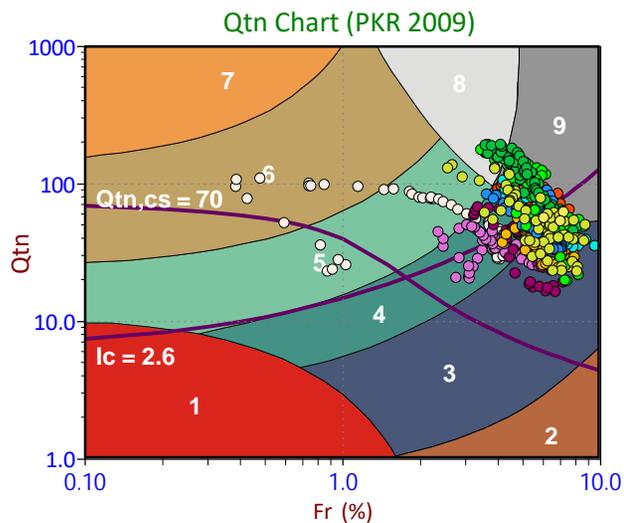
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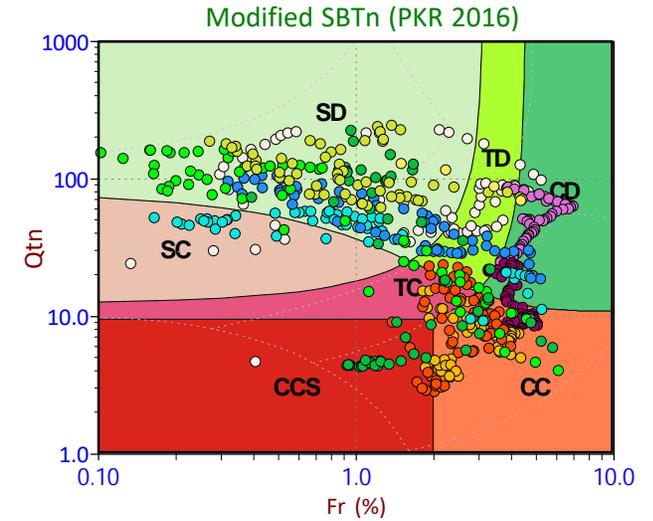
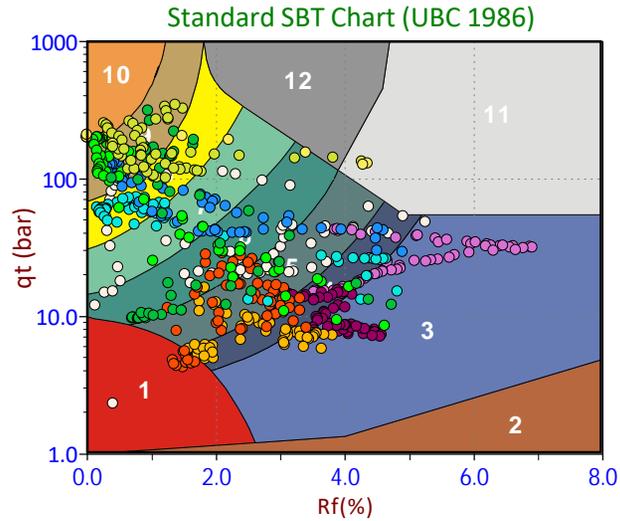
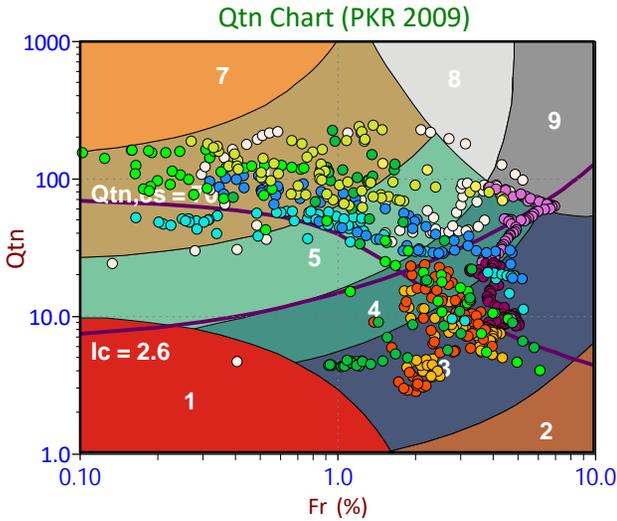
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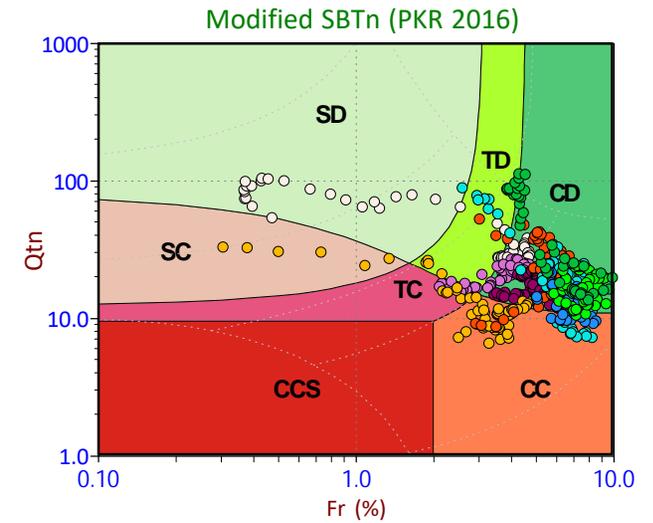
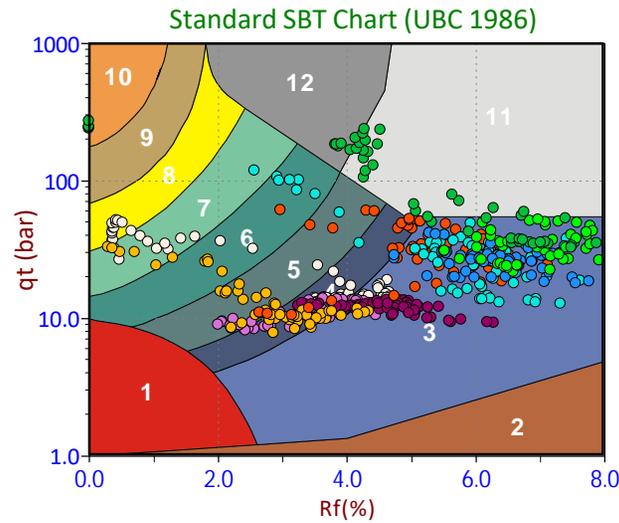
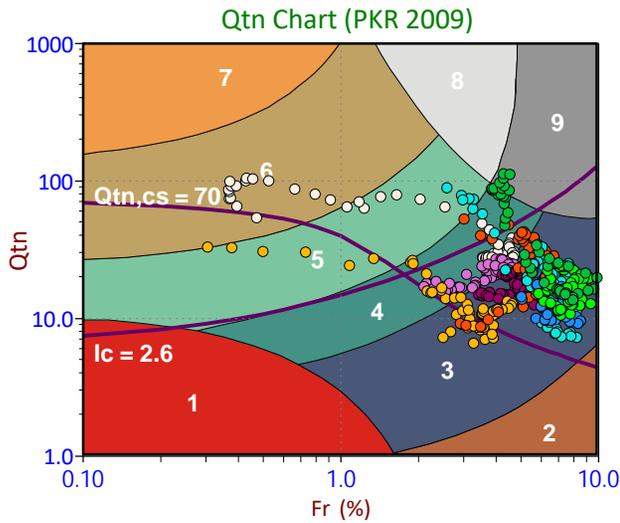
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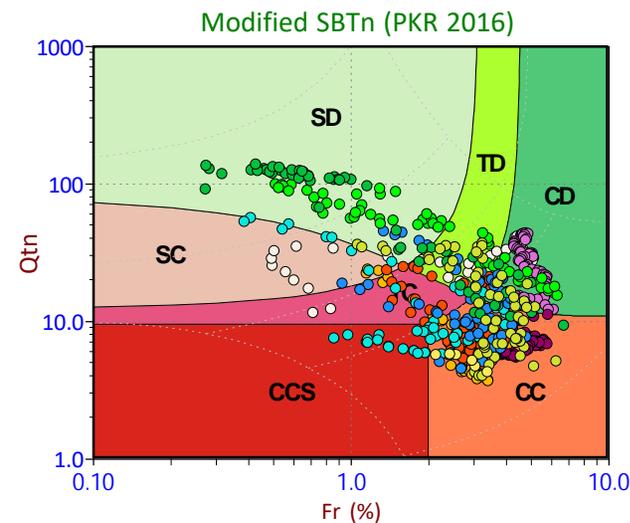
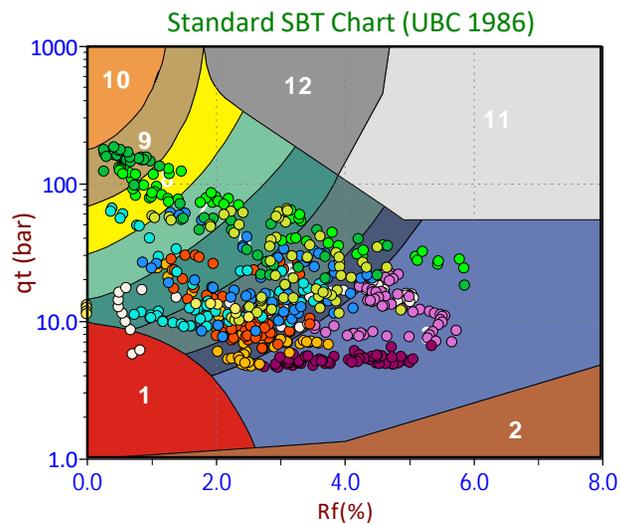
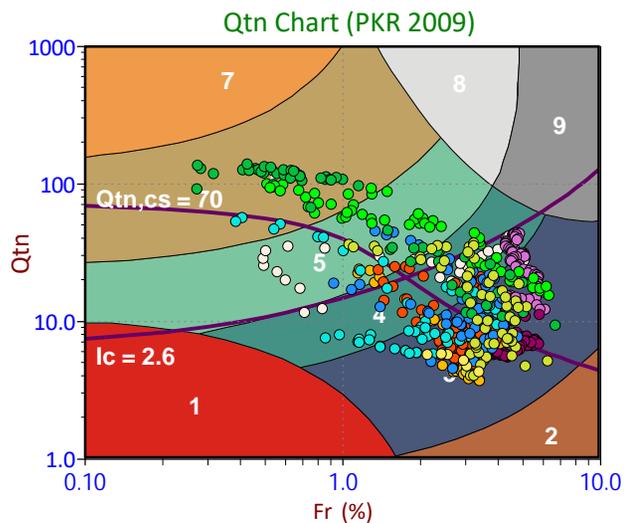
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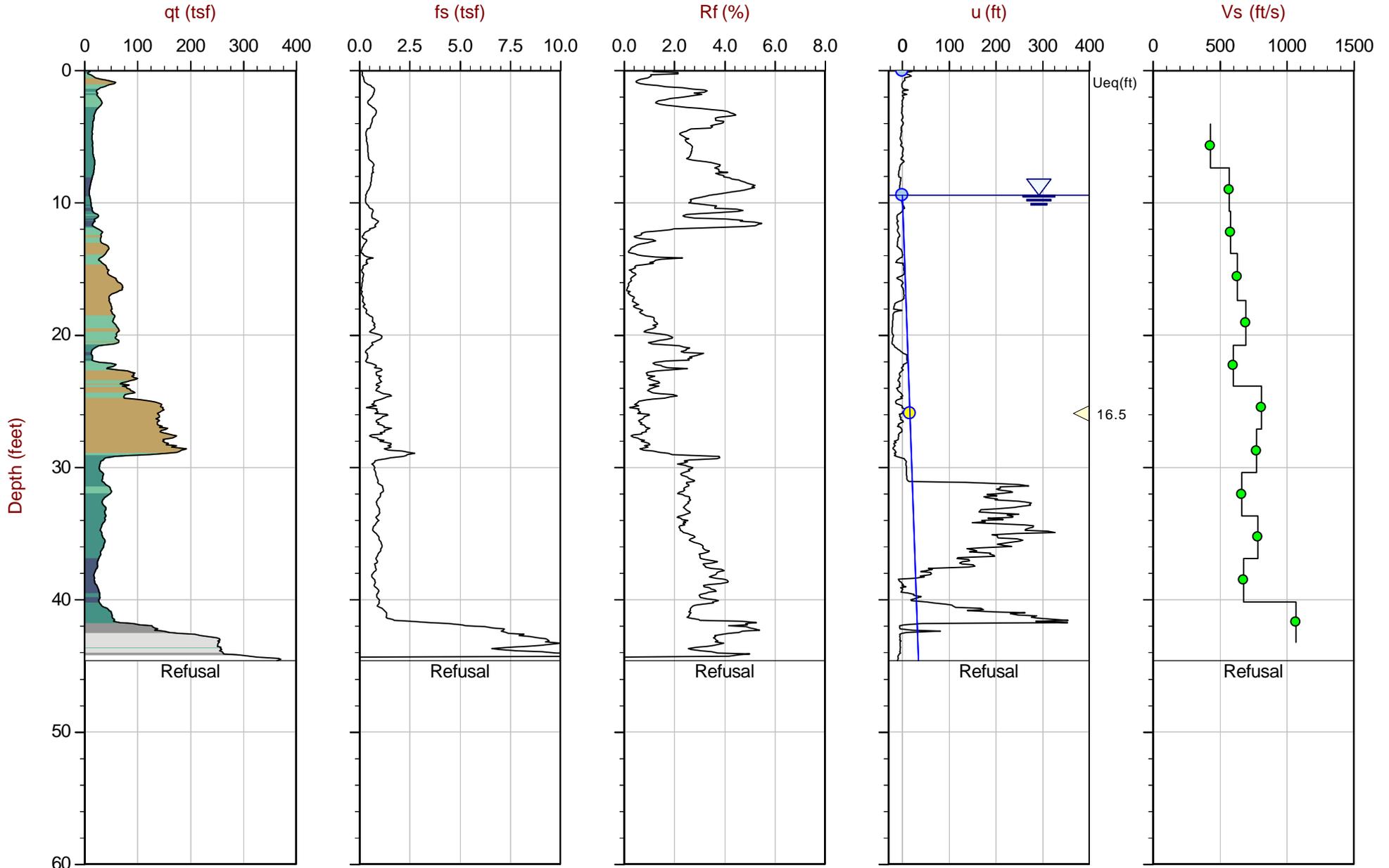
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## Seismic Cone Penetration Test Plots





Max Depth: 13.600 m / 44.62 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 20-56-20953\_SP05.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 10N N: 4202400m E: 558173m

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▼ Dissipation, Ueq not achieved    — Hydrostatic Line

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

## Seismic Cone Penetration Test Tabular Results





Job No: 20-56-20953  
Client: Alan Kropp & Associates  
Project: El Portal  
Sounding ID: CPT-05  
Date: 06:12:20 07:06

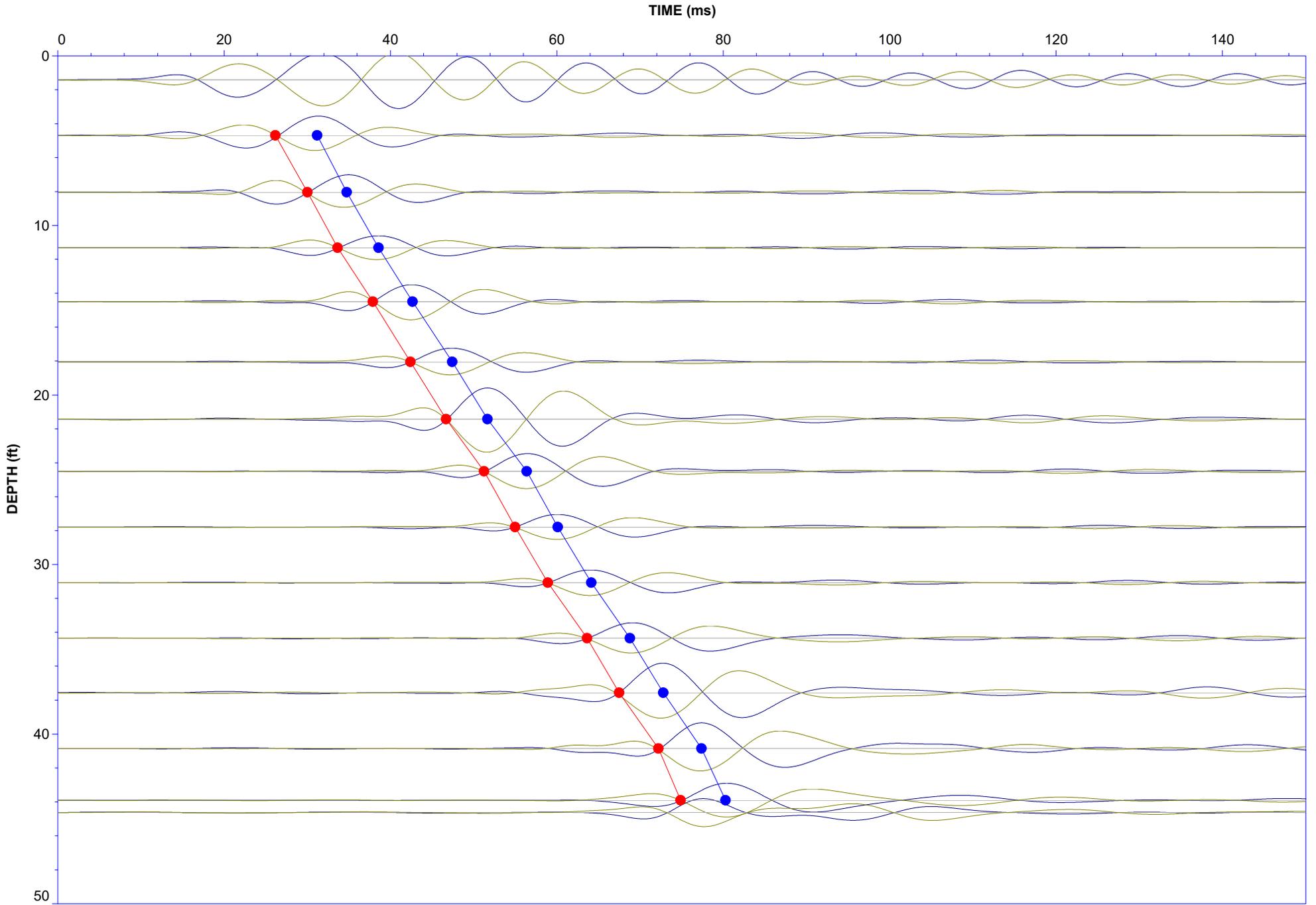
Seismic Source: Beam  
Seismic Offset (ft): 10.50  
Source Depth (ft): 0.00  
Geophone Offset (ft): 0.66

**SCPT<sub>u</sub> 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) |
|----------------|---------------------|---------------|--------------------------|---------------------------|--------------------------|
| 4.69           | 4.04                | 11.25         |                          |                           |                          |
| 8.04           | 7.38                | 12.84         | 1.59                     | 3.69                      | 430                      |
| 11.32          | 10.66               | 14.97         | 2.13                     | 3.74                      | 570                      |
| 14.50          | 13.85               | 17.38         | 2.41                     | 4.16                      | 580                      |
| 18.04          | 17.39               | 20.31         | 2.94                     | 4.65                      | 632                      |
| 21.42          | 20.77               | 23.27         | 2.96                     | 4.26                      | 695                      |
| 24.51          | 23.85               | 26.06         | 2.79                     | 4.65                      | 600                      |
| 27.79          | 27.13               | 29.09         | 3.03                     | 3.74                      | 811                      |
| 31.07          | 30.41               | 32.18         | 3.08                     | 3.98                      | 775                      |
| 34.35          | 33.69               | 35.29         | 3.12                     | 4.68                      | 666                      |
| 37.57          | 36.91               | 38.37         | 3.08                     | 3.93                      | 784                      |
| 40.85          | 40.19               | 41.54         | 3.17                     | 4.66                      | 679                      |
| 43.90          | 43.24               | 44.50         | 2.96                     | 2.77                      | 1069                     |

## Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Traces





## Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



Job No: 20-56-20953  
Client: Alan Kropp & Associates  
Project: El Portal  
Start Date: 11-Jun-2020  
End Date: 12-Jun-2020

### **CPT<sub>u</sub> PORE PRESSURE DISSIPATION SUMMARY**

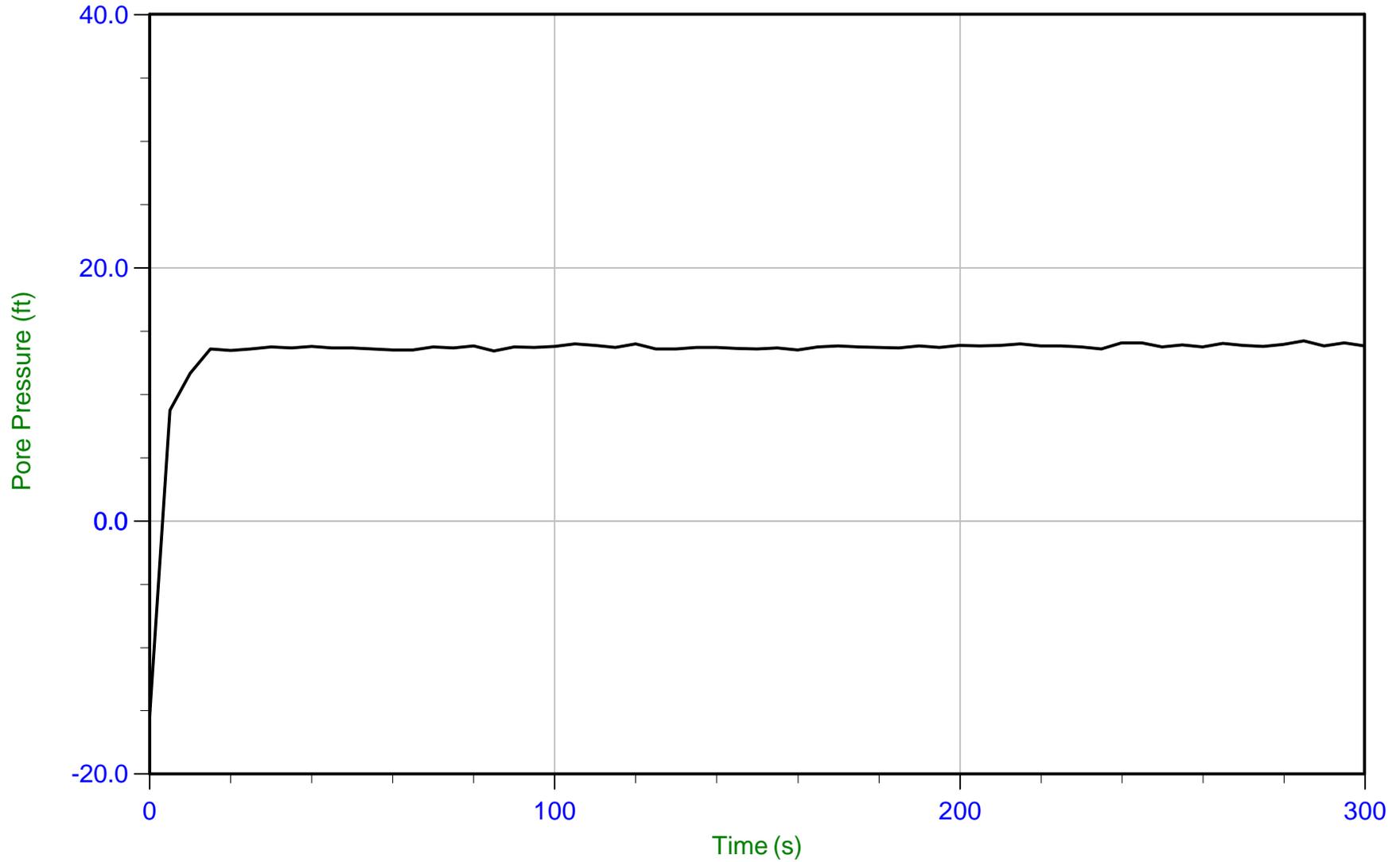
| Sounding ID | File Name        | Cone Area (cm <sup>2</sup> ) | Duration (s) | Test Depth (ft) | Estimated Equilibrium Pore Pressure U <sub>eq</sub> (ft) | Calculated Phreatic Surface (ft) |
|-------------|------------------|------------------------------|--------------|-----------------|--|----------------------------------|
| CPT-01      | 20-56-20953_CP01 | 15                           | 300          | 31.66           | 13.8   | 17.8                             |
| CPT-02      | 20-56-20953_CP02 | 15                           | 300          | 48.64           | 43.2   | 5.4                              |
| CPT-03      | 20-56-20953_CP03 | 15                           | 300          | 47.24           | 42.3   | 5.0                              |
| CPT-04      | 20-56-20953_CP04 | 15                           | 260          | 28.87           | 9.9  | 18.9                             |
| CPT-05      | 20-56-20953_SP05 | 15                           | 400          | 25.92           | 16.5   | 9.4                              |
| CPT-07      | 20-56-20953_CP07 | 15                           | 300          | 34.28           | 13.7   | 20.6                             |
| CPT-08      | 20-56-20953_CP08 | 15                           | 645          | 44.29           | Not Achieved   |                                  |
| CPT-09      | 20-56-20953_CP09 | 15                           | 600          | 37.48           | 17.5   | 20.0                             |



*Alan Kropp & Associates*

Job No: 20-56-20953  
Date: 06/11/2020 10:03  
Site: El Portal

Sounding: CPT-01  
Cone: 496:T1500F15U1K Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 20-56-20953\_CP01.PPF  
Depth: 9.650 m / 31.660 ft  
Duration: 300.0 s

u Min: -15.4 ft  
u Max: 14.2 ft  
u Final: 13.8 ft

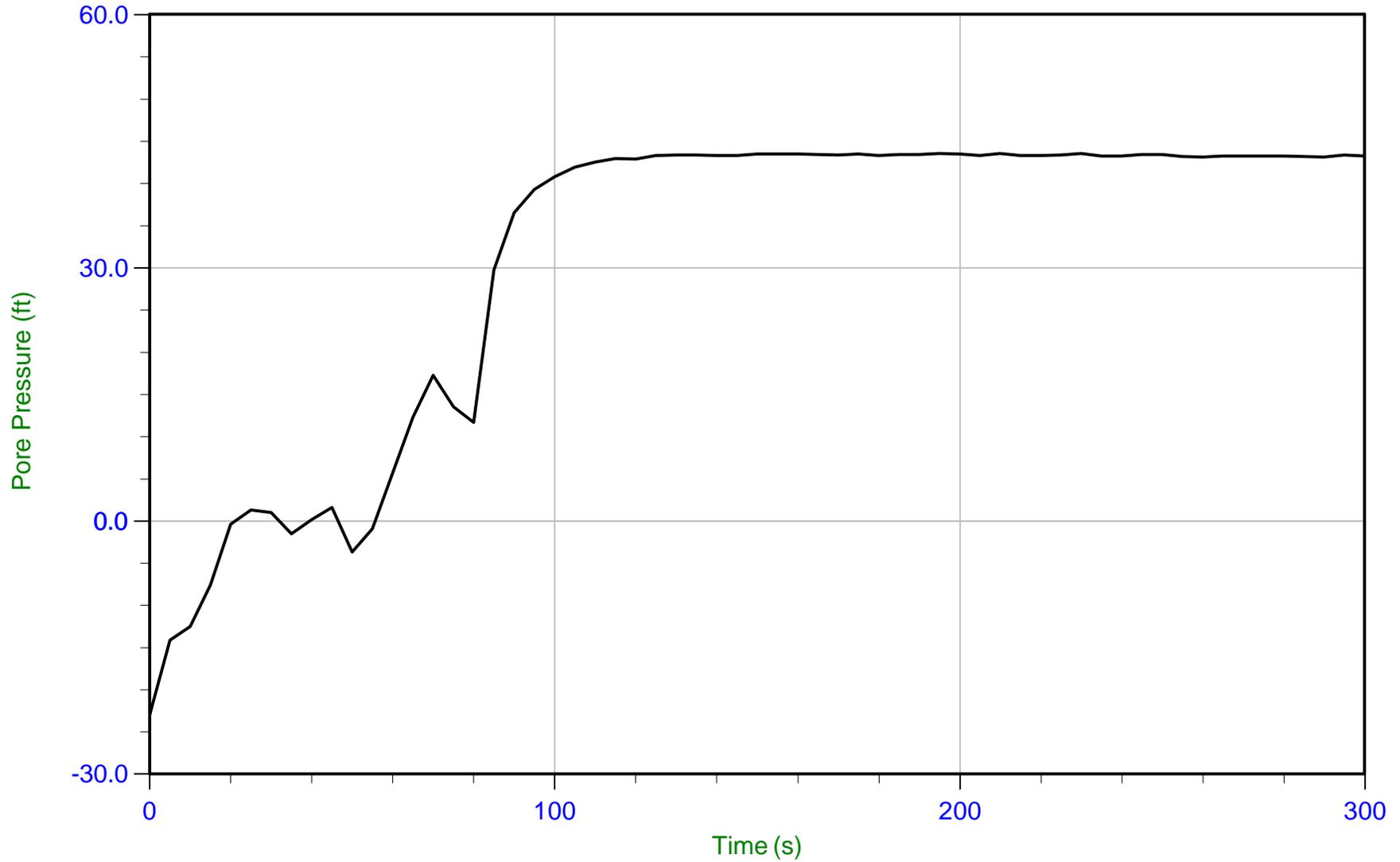
WT: 5.440 m / 17.848 ft  
Ueq: 13.8 ft



*Alan Kropp & Associates*

Job No: 20-56-20953  
Date: 06/11/2020 11:31  
Site: El Portal

Sounding: CPT-02  
Cone: 496:T1500F15U1K Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 20-56-20953\_CP02.PPF  
Depth: 14.825 m / 48.638 ft  
Duration: 300.0 s

u Min: -23.0 ft  
u Max: 43.5 ft  
u Final: 43.2 ft

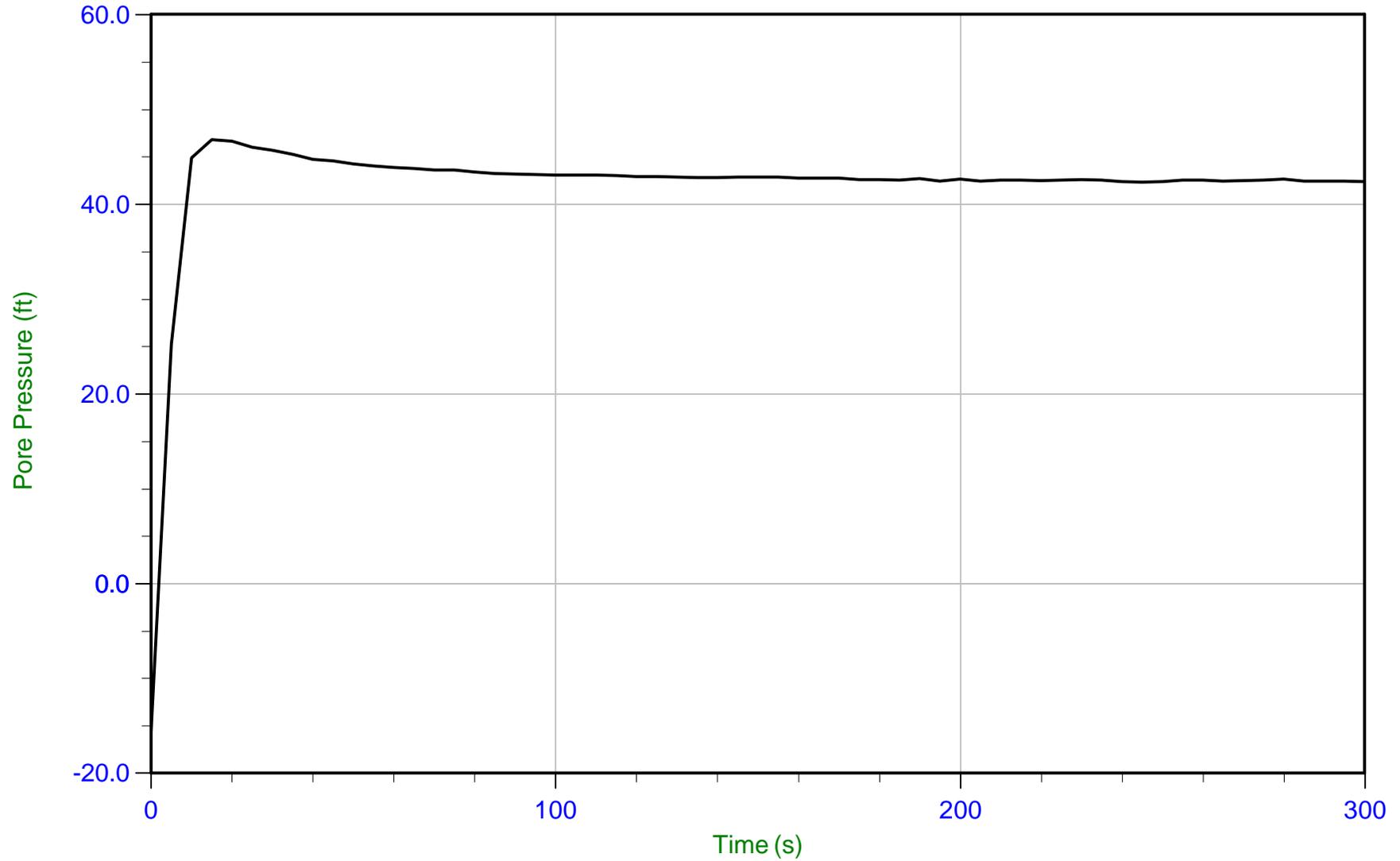
WT: 1.656 m / 5.433 ft  
Ueq: 43.2 ft



*Alan Kropp & Associates*

Job No: 20-56-20953  
Date: 06/11/2020 08:12  
Site: El Portal

Sounding: CPT-03  
Cone: 496:T1500F15U1K Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 20-56-20953\_CP03.PPF  
Depth: 14.400 m / 47.244 ft  
Duration: 300.0 s

u Min: -15.5 ft  
u Max: 46.8 ft  
u Final: 42.4 ft

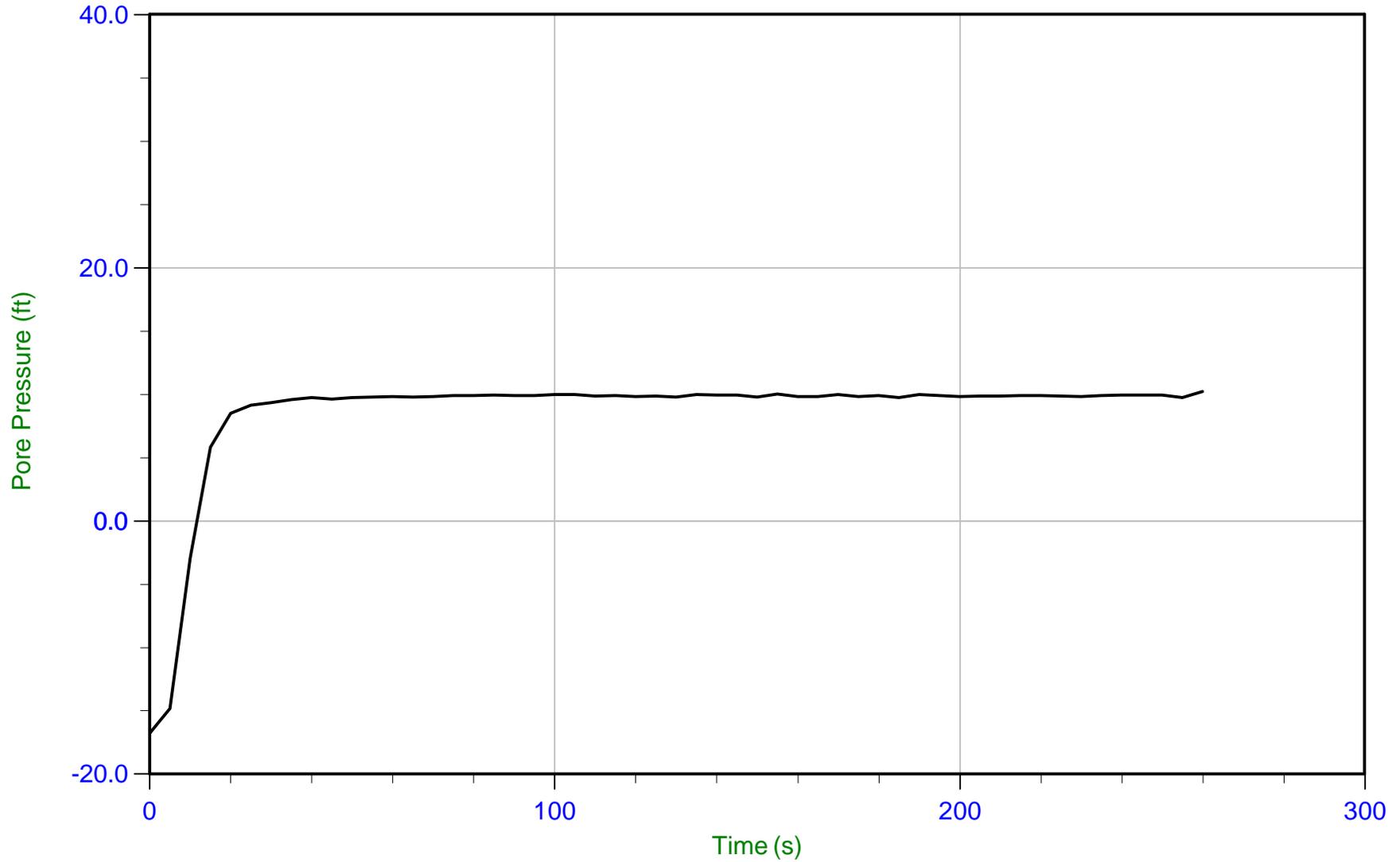
WT: 1.516 m / 4.974 ft  
Ueq: 42.3 ft



*Alan Kropp & Associates*

Job No: 20-56-20953  
Date: 06/11/2020 10:46  
Site: El Portal

Sounding: CPT-04  
Cone: 496:T1500F15U1K Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 20-56-20953\_CP04.PPF  
Depth: 8.800 m / 28.871 ft  
Duration: 260.0 s

u Min: -16.8 ft  
u Max: 10.2 ft  
u Final: 10.2 ft

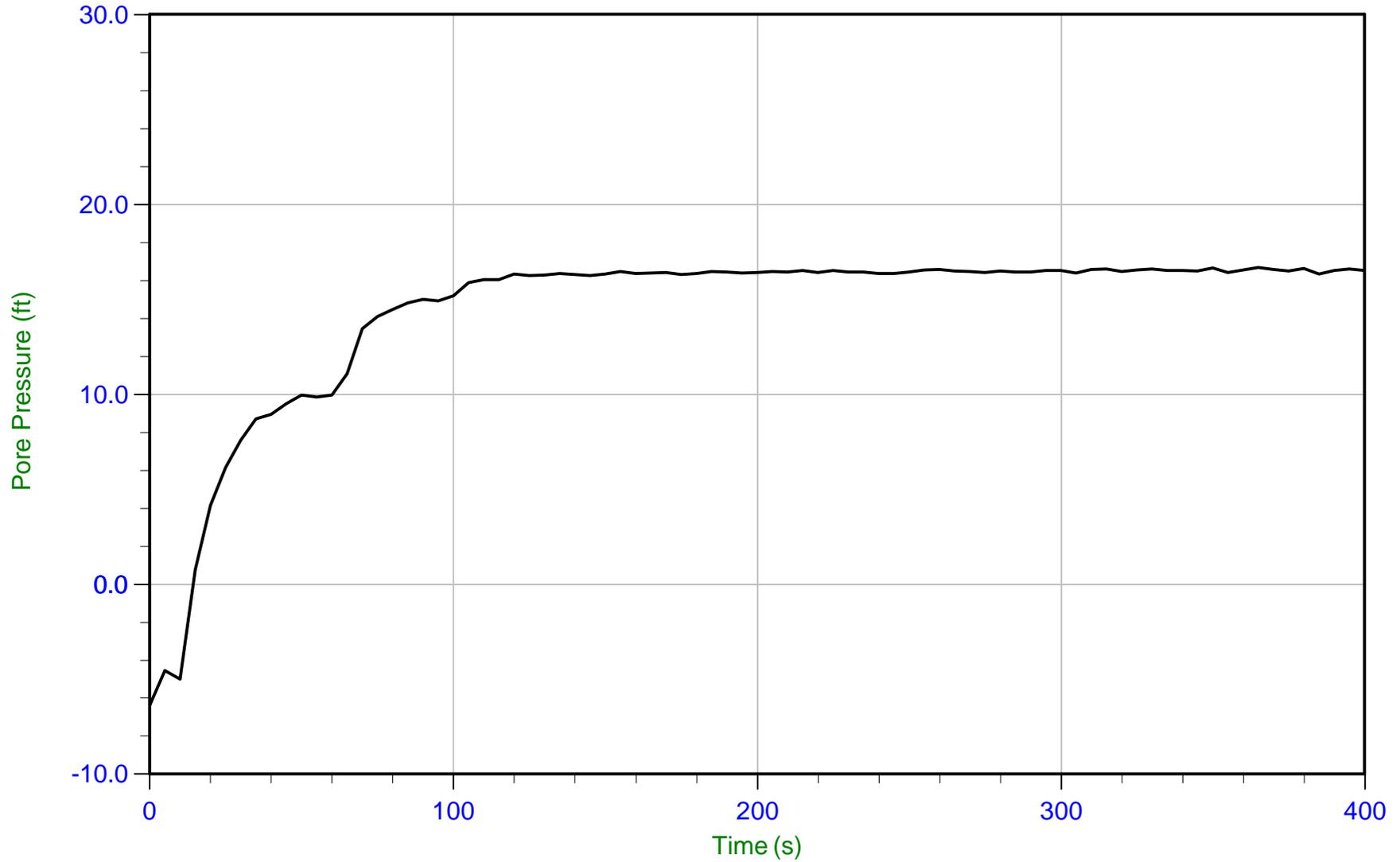
WT: 5.768 m / 18.924 ft  
Ueq: 9.9 ft



*Alan Kropp & Associates*

Job No: 20-56-20953  
Date: 06/12/2020 07:06  
Site: El Portal

Sounding: CPT-05  
Cone: 383:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 20-56-20953\_SP05.PPF  
Depth: 7.900 m / 25.918 ft  
Duration: 400.0 s

u Min: -6.4 ft  
u Max: 16.7 ft  
u Final: 16.5 ft

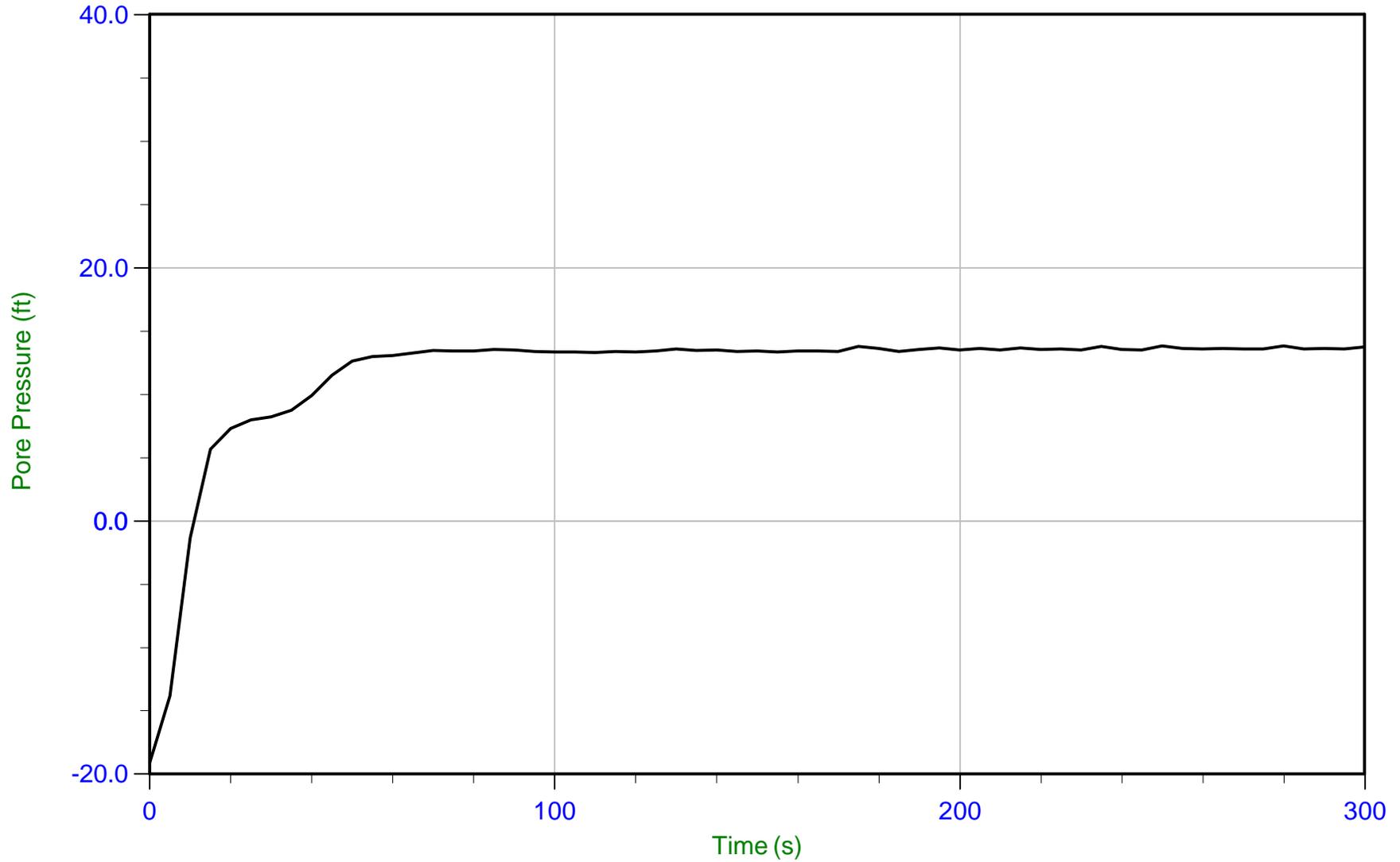
WT: 2.872 m / 9.422 ft  
Ueq: 16.5 ft



*Alan Kropp & Associates*

Job No: 20-56-20953  
Date: 06/12/2020 12:13  
Site: El Portal

Sounding: CPT-07  
Cone: 383:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 20-56-20953\_CP07.PPF  
Depth: 10.450 m / 34.284 ft  
Duration: 300.0 s

u Min: -19.1 ft  
u Max: 13.8 ft  
u Final: 13.8 ft

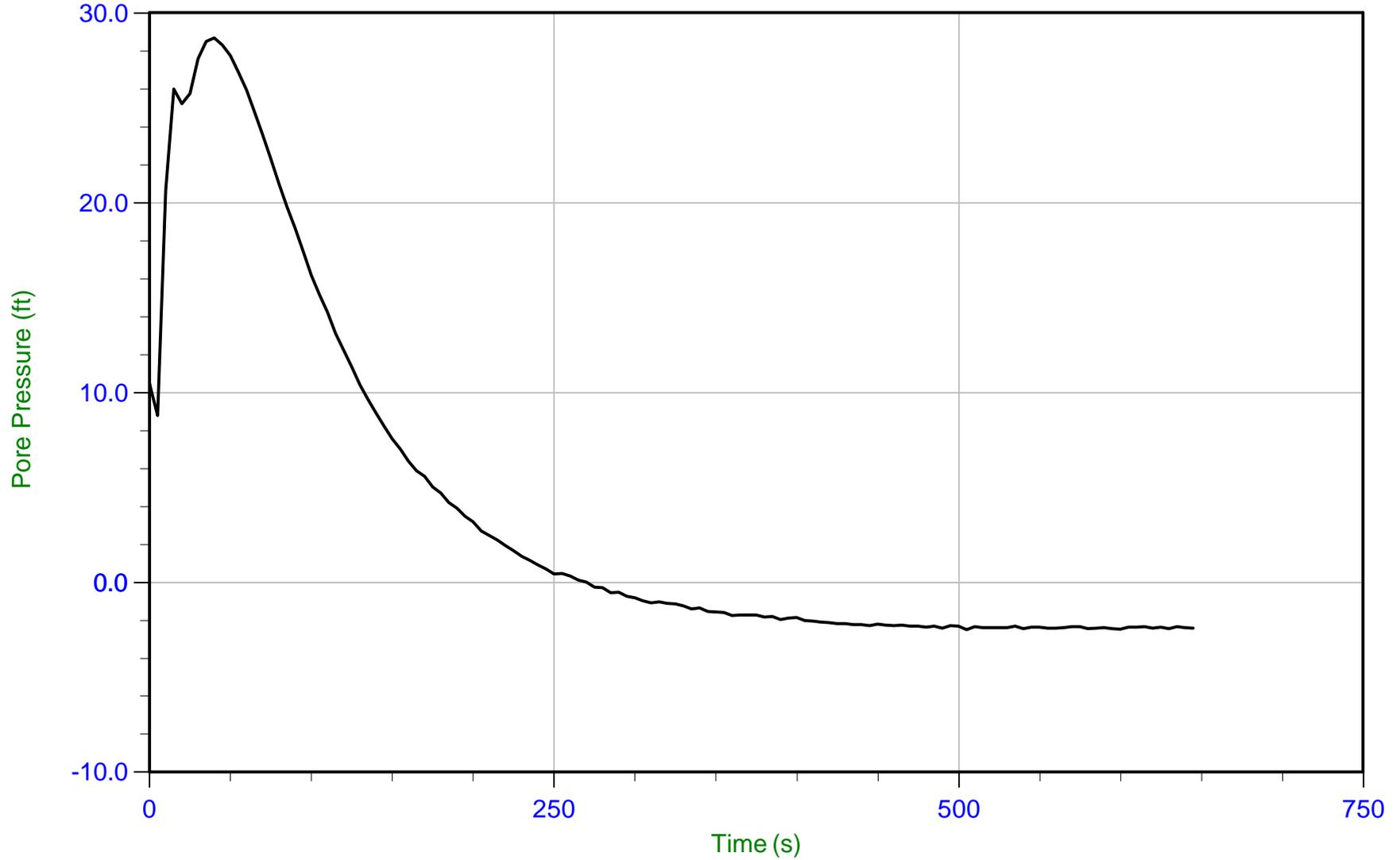
WT: 6.286 m / 20.623 ft  
Ueq: 13.7 ft



*Alan Kropp & Associates*

Job No: 20-56-20953  
Date: 06/12/2020 09:41  
Site: El Portal

Sounding: CPT-08  
Cone: 383:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 20-56-20953\_CP08.PPF  
Depth: 13.500 m / 44.291 ft  
Duration: 645.0 s

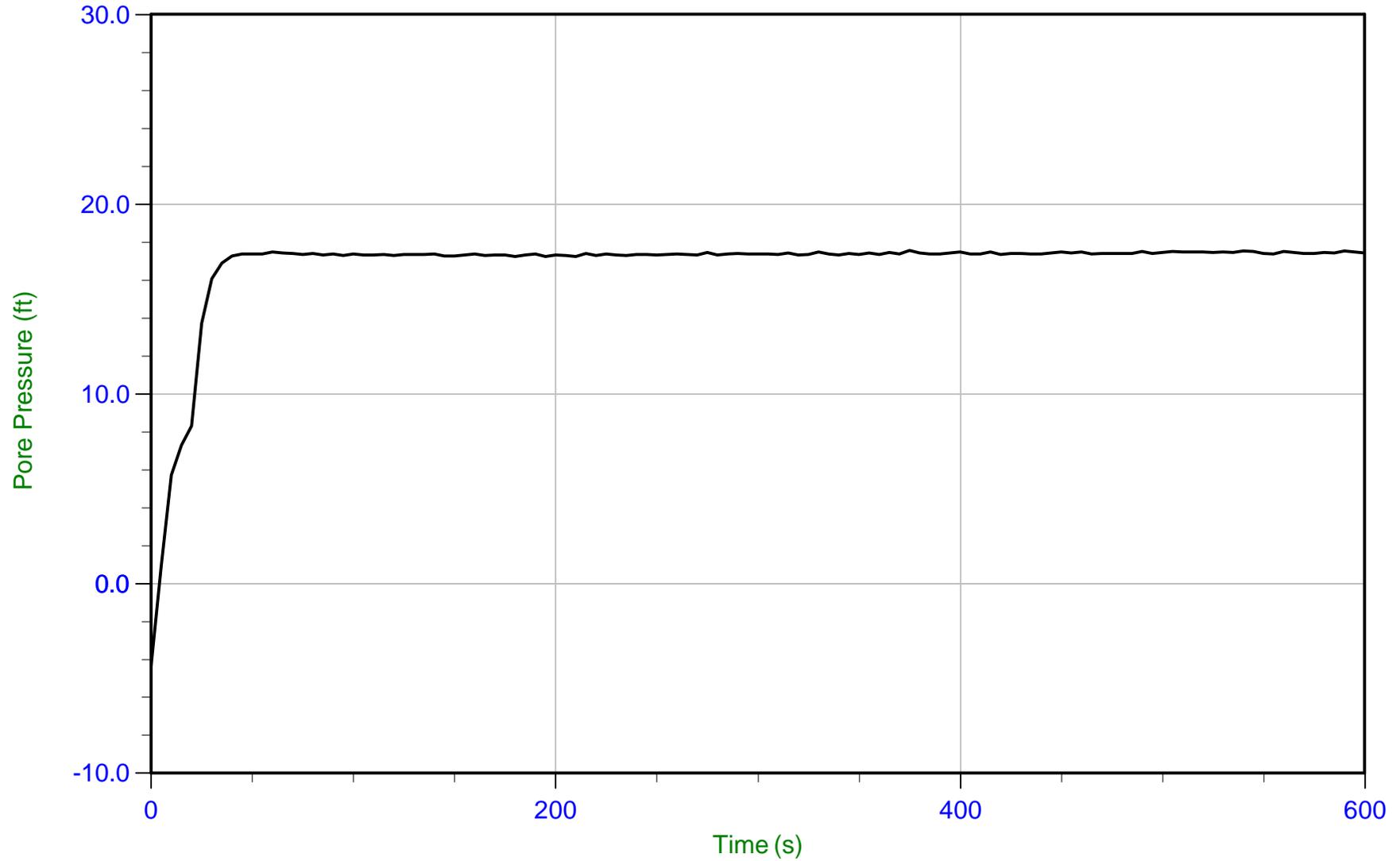
u Min: -2.5 ft  
u Max: 28.7 ft  
u Final: -2.4 ft



*Alan Kropp & Associates*

Job No: 20-56-20953  
Date: 06/12/2020 10:40  
Site: El Portal

Sounding: CPT-09  
Cone: 383:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

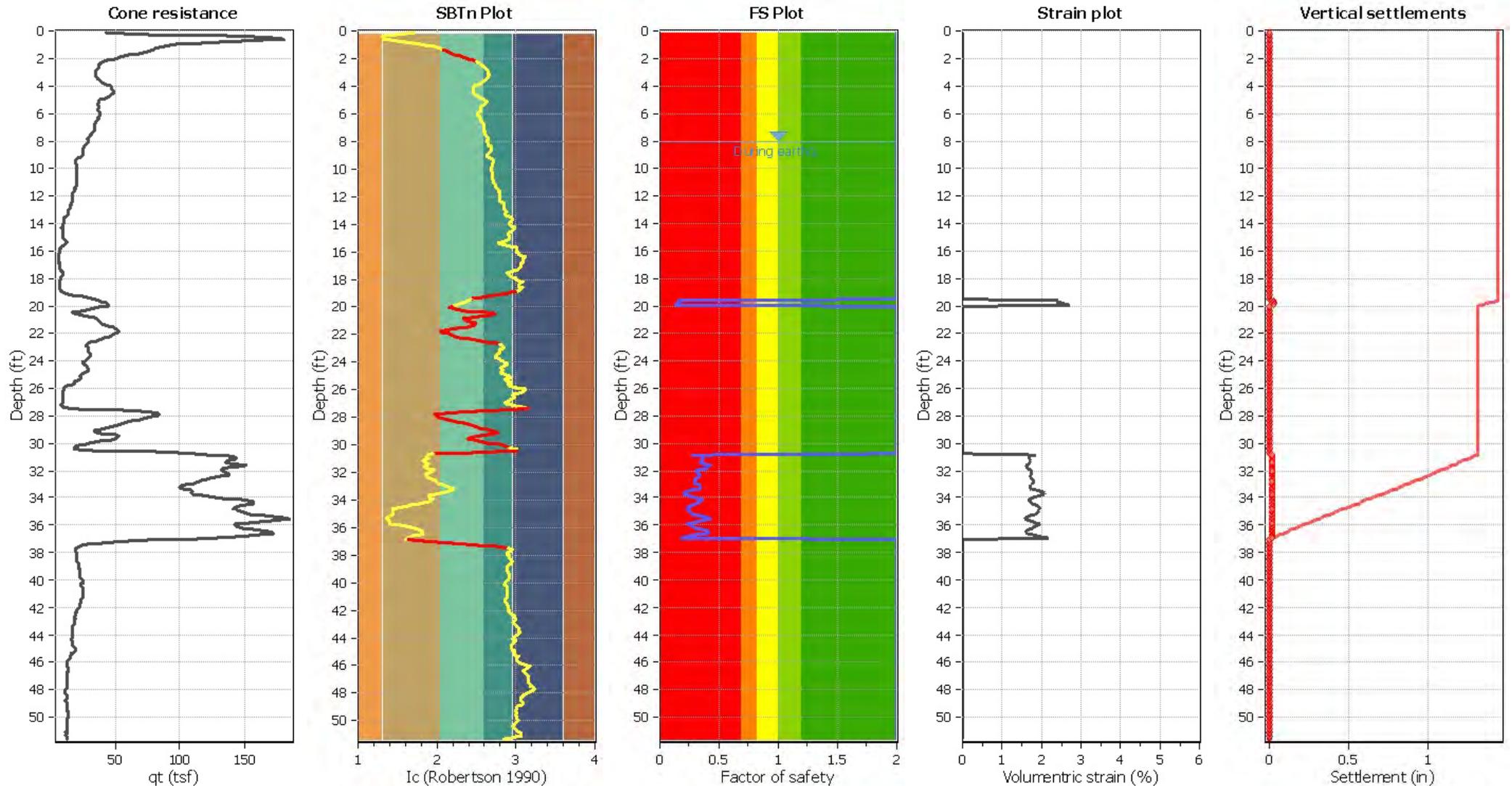
Filename: 20-56-20953\_CP09.PPF  
Depth: 11.425 m / 37.483 ft  
Duration: 600.0 s

u Min: -4.3 ft  
u Max: 17.6 ft  
u Final: 17.4 ft

WT: 6.106 m / 20.033 ft  
Ueq: 17.5 ft

APPENDIX C  
LIQUEFACTION ANALYSES

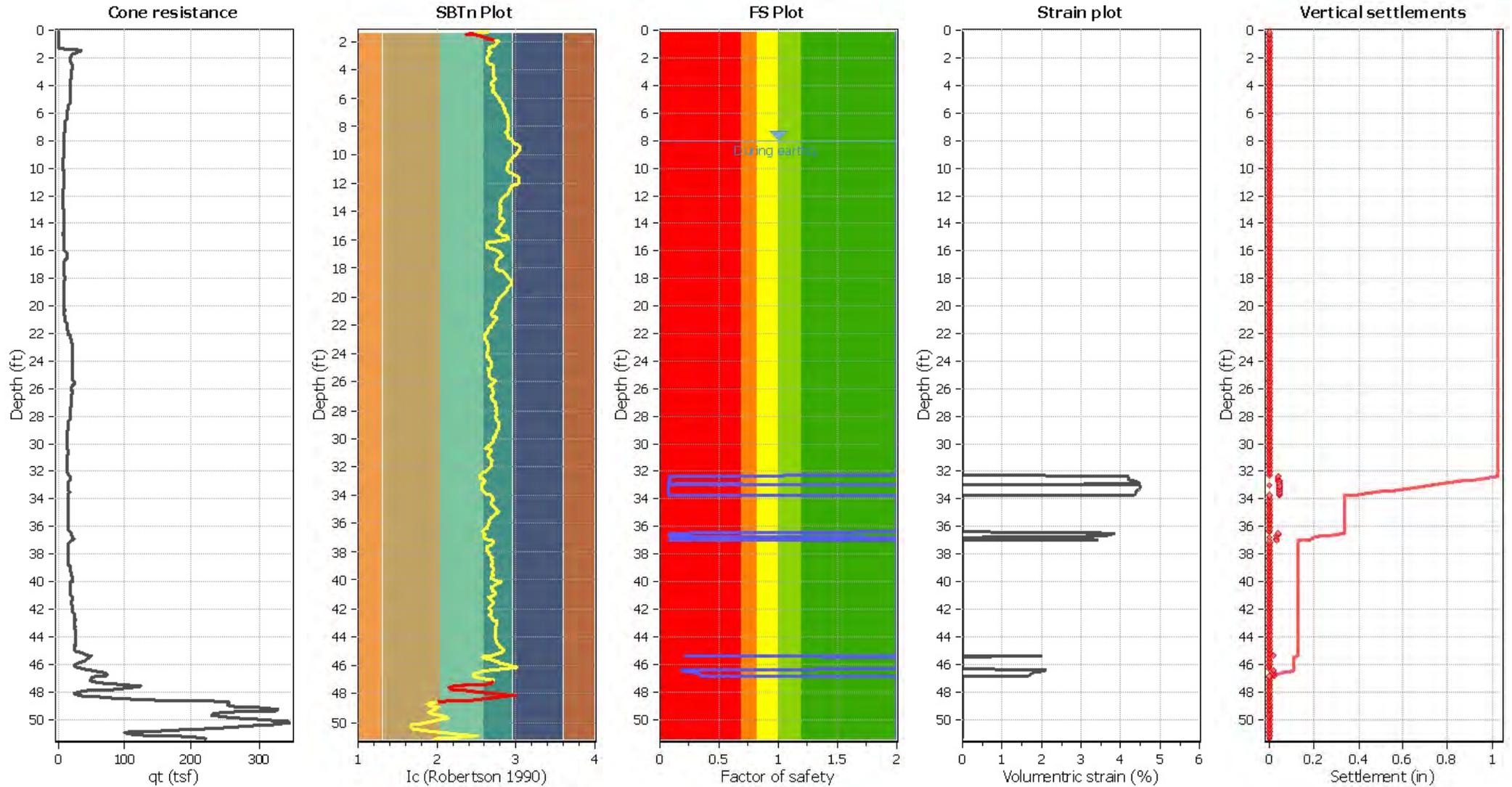
### Estimation of post-earthquake settlements



**Abbreviations**

- q<sub>c</sub>: Total cone resistance (cone resistance q<sub>c</sub> corrected for pore water effects)
- I<sub>c</sub>: Soil Behaviour Type Index
- FS: Calculated Factor of Safety against liquefaction
- Volumetric strain: Post-liquefaction volumetric strain

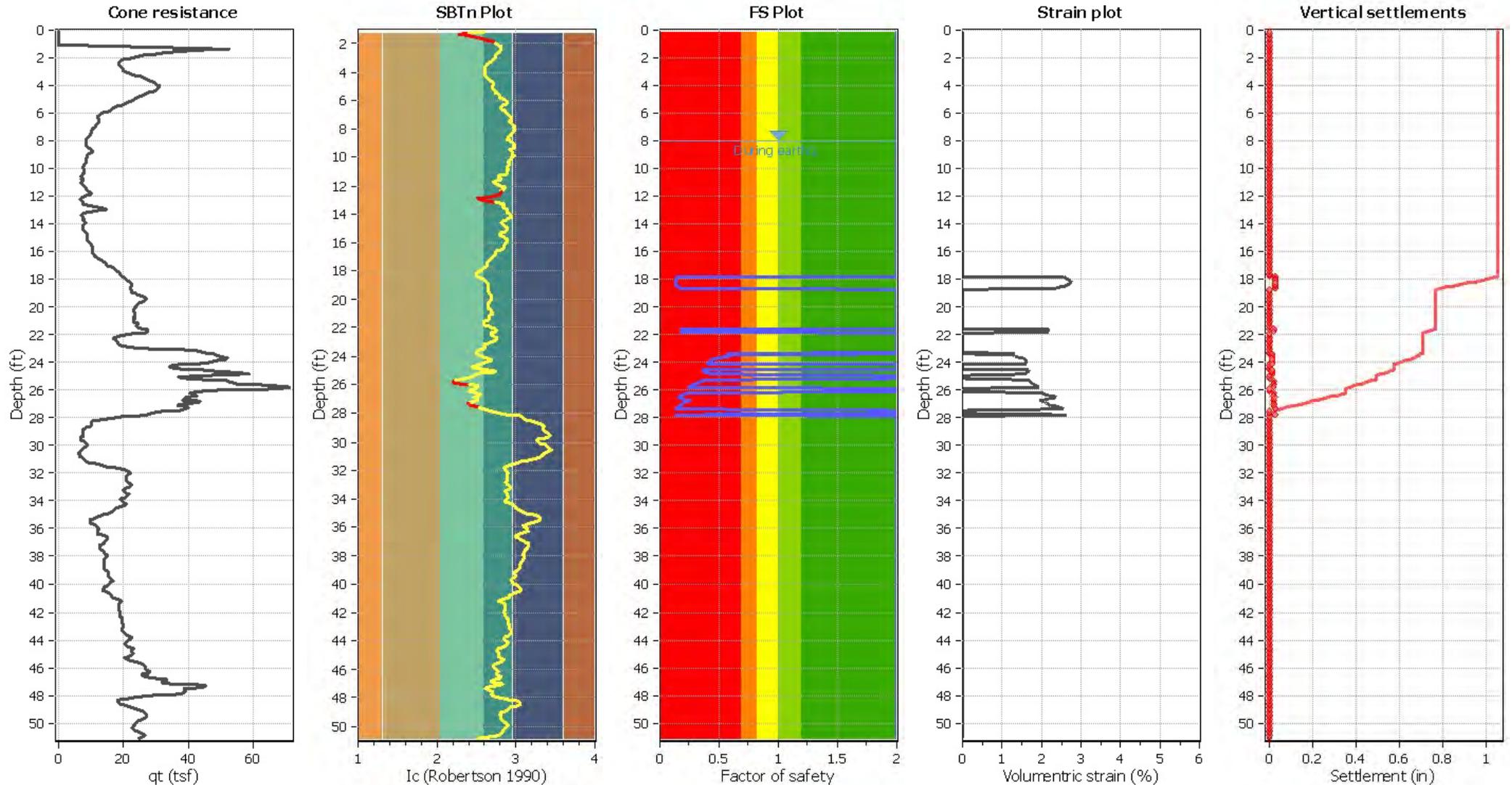
### Estimation of post-earthquake settlements



**Abbreviations**

- q<sub>c</sub>: Total cone resistance (cone resistance q<sub>c</sub> corrected for pore water effects)
- I<sub>c</sub>: Soil Behaviour Type Index
- FS: Calculated Factor of Safety against liquefaction
- Volumetric strain: Post-liquefaction volumetric strain

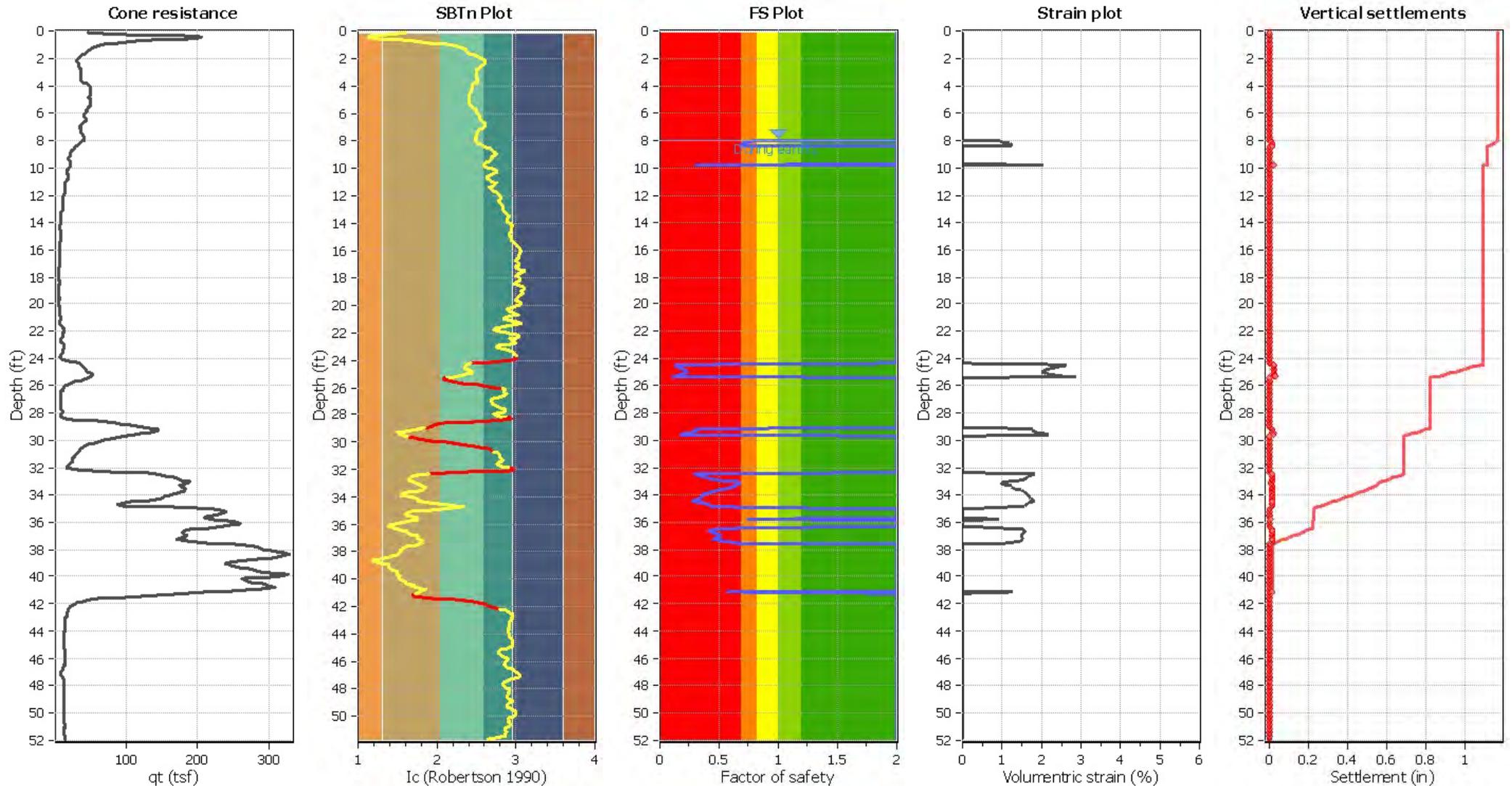
### Estimation of post-earthquake settlements



**Abbreviations**

- q<sub>c</sub>: Total cone resistance (cone resistance q<sub>c</sub> corrected for pore water effects)
- I<sub>c</sub>: Soil Behaviour Type Index
- FS: Calculated Factor of Safety against liquefaction
- Volumetric strain: Post-liquefaction volumetric strain

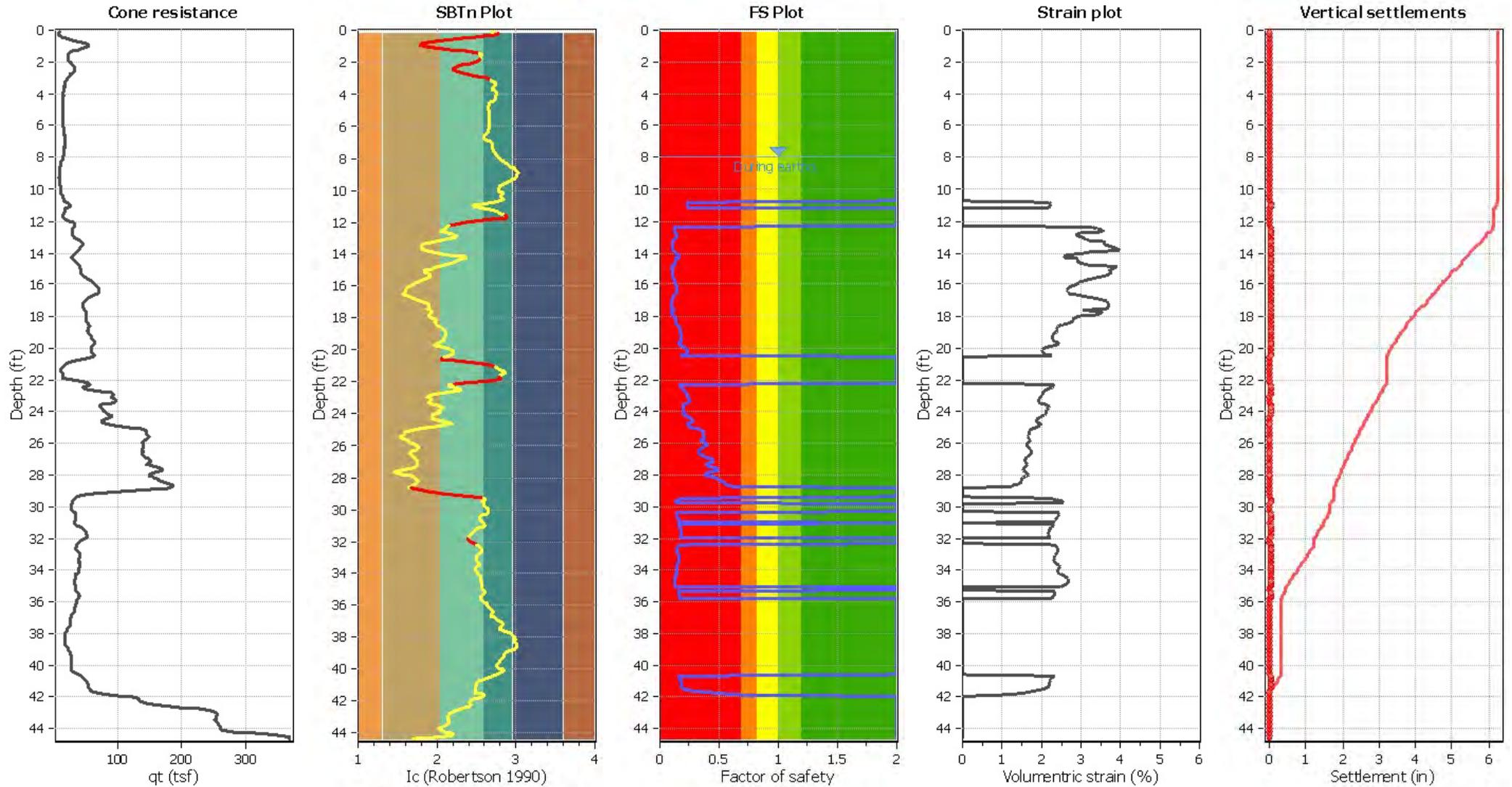
### Estimation of post-earthquake settlements



**Abbreviations**

- q<sub>c</sub>: Total cone resistance (cone resistance q<sub>c</sub> corrected for pore water effects)
- I<sub>c</sub>: Soil Behaviour Type Index
- FS: Calculated Factor of Safety against liquefaction
- Volumetric strain: Post-liquefaction volumetric strain

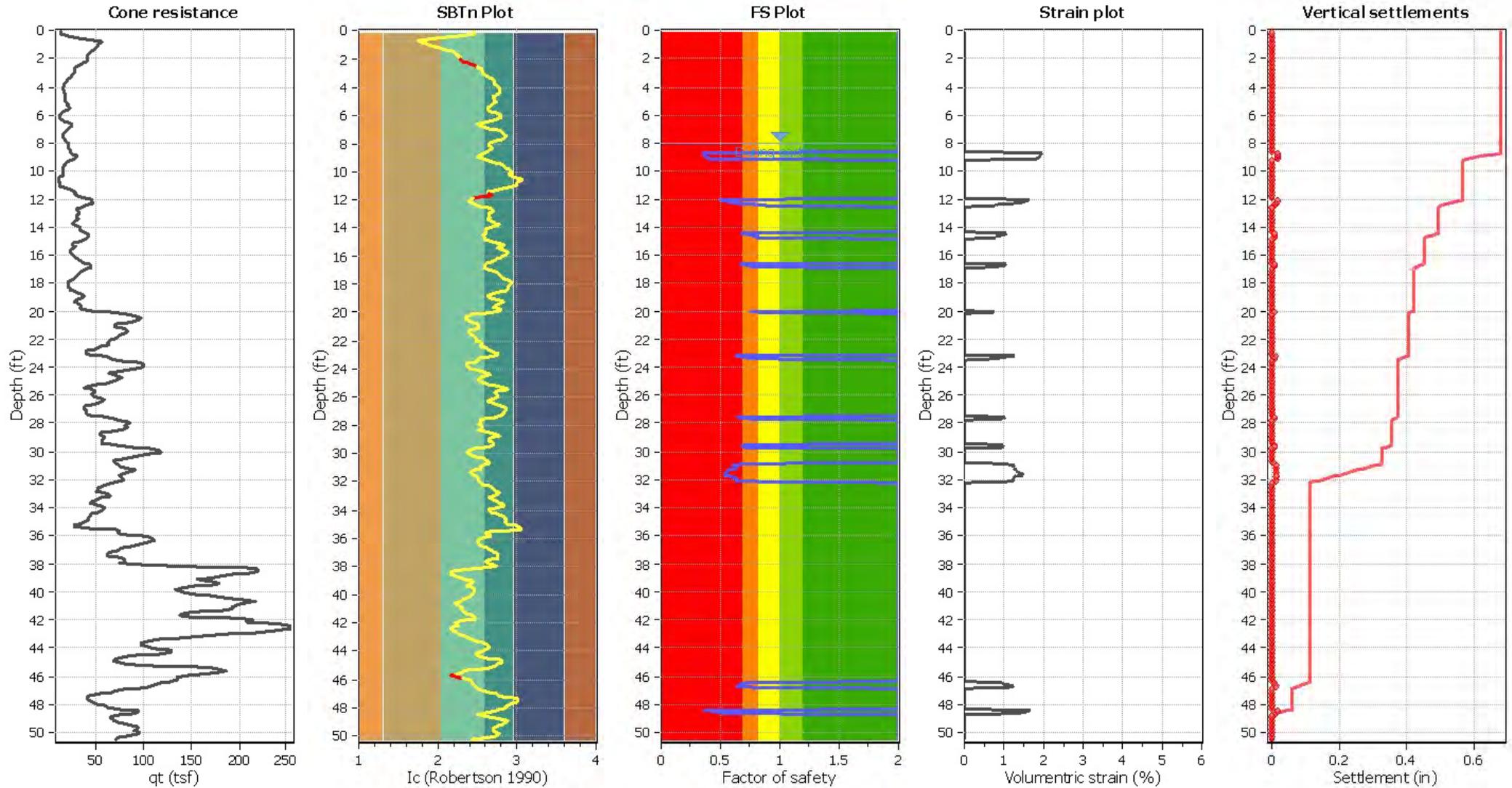
### Estimation of post-earthquake settlements



**Abbreviations**

- q<sub>c</sub>: Total cone resistance (cone resistance q<sub>c</sub> corrected for pore water effects)
- I<sub>c</sub>: Soil Behaviour Type Index
- FS: Calculated Factor of Safety against liquefaction
- Volumetric strain: Post-liquefaction volumetric strain

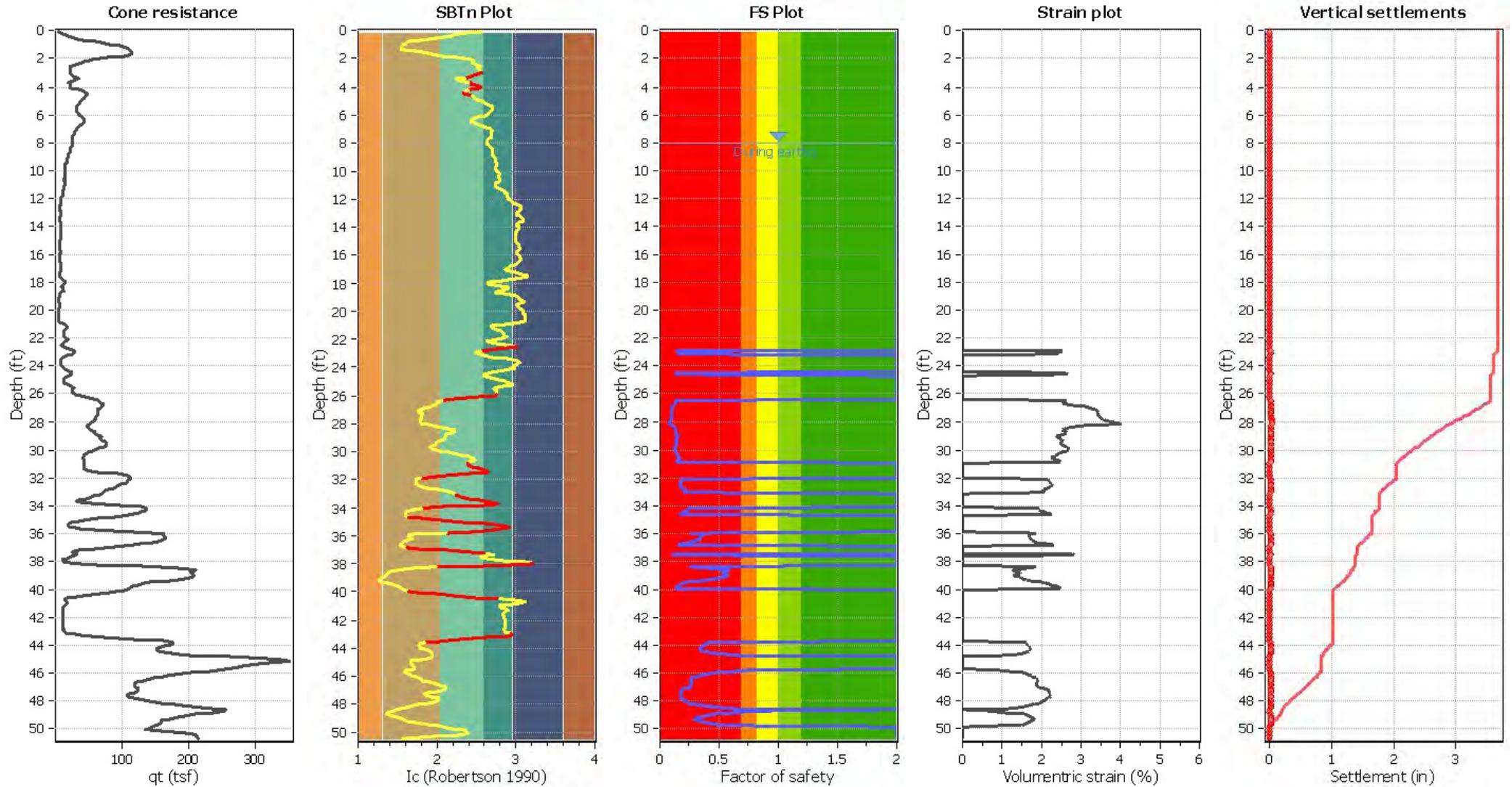
### Estimation of post-earthquake settlements



**Abbreviations**

- q<sub>c</sub>: Total cone resistance (cone resistance q<sub>c</sub> corrected for pore water effects)
- I<sub>c</sub>: Soil Behaviour Type Index
- FS: Calculated Factor of Safety against liquefaction
- Volumetric strain: Post-liquefaction volumetric strain

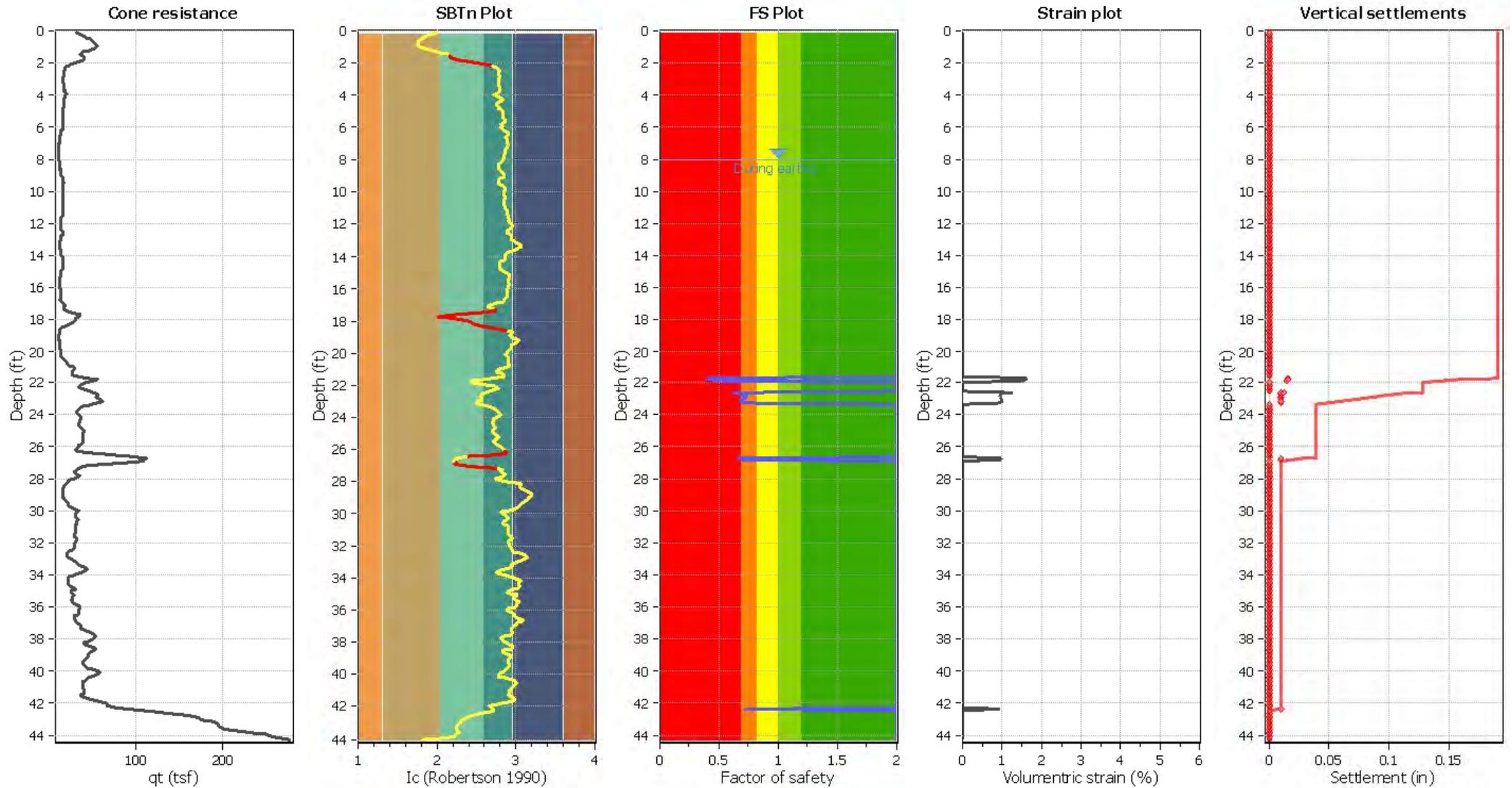
### Estimation of post-earthquake settlements



**Abbreviations**

- q<sub>c</sub>: Total cone resistance (cone resistance q<sub>c</sub> corrected for pore water effects)
- I<sub>c</sub>: Soil Behaviour Type Index
- FS: Calculated Factor of Safety against liquefaction
- Volumetric strain: Post-liquefaction volumetric strain

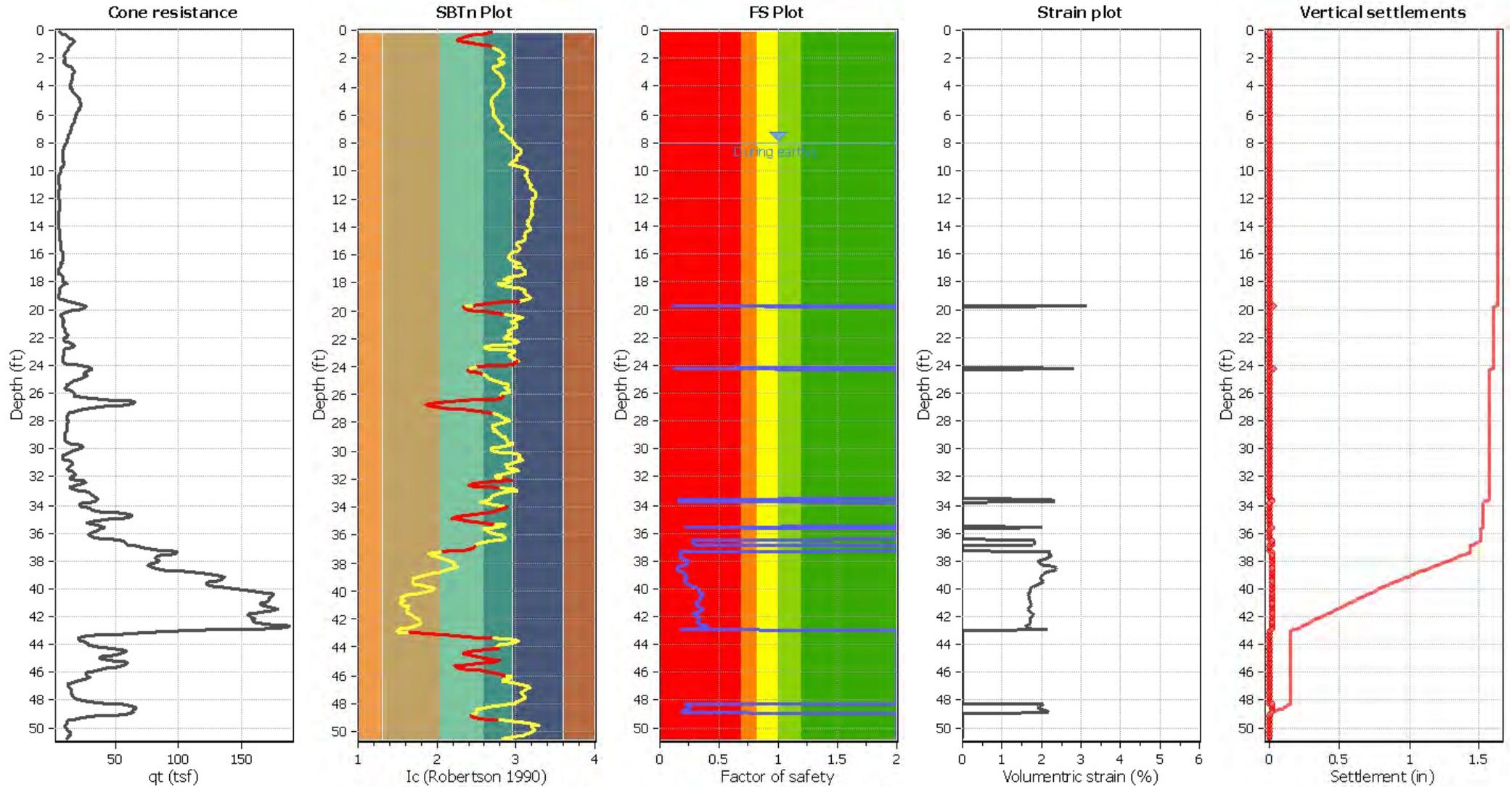
### Estimation of post-earthquake settlements



**Abbreviations**

- q<sub>c</sub>: Total cone resistance (cone resistance q<sub>c</sub> corrected for pore water effects)
- I<sub>c</sub>: Soil Behaviour Type Index
- FS: Calculated Factor of Safety against liquefaction
- Volumetric strain: Post-liquefaction volumetric strain

### Estimation of post-earthquake settlements

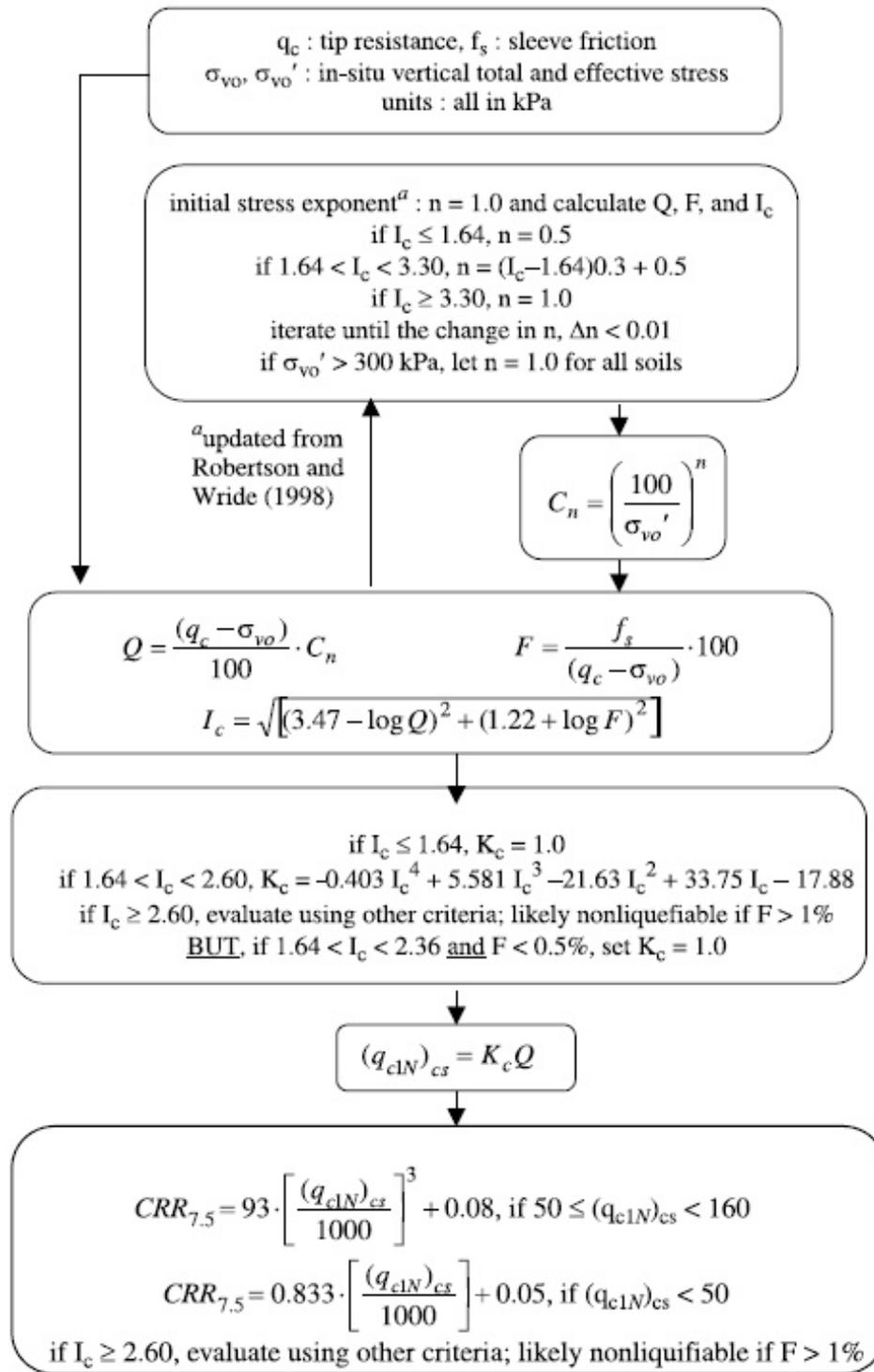


**Abbreviations**

- q<sub>c</sub>: Total cone resistance (cone resistance q<sub>c</sub> corrected for pore water effects)
- I<sub>c</sub>: Soil Behaviour Type Index
- FS: Calculated Factor of Safety against liquefaction
- Volumetric strain: Post-liquefaction volumetric strain

## Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

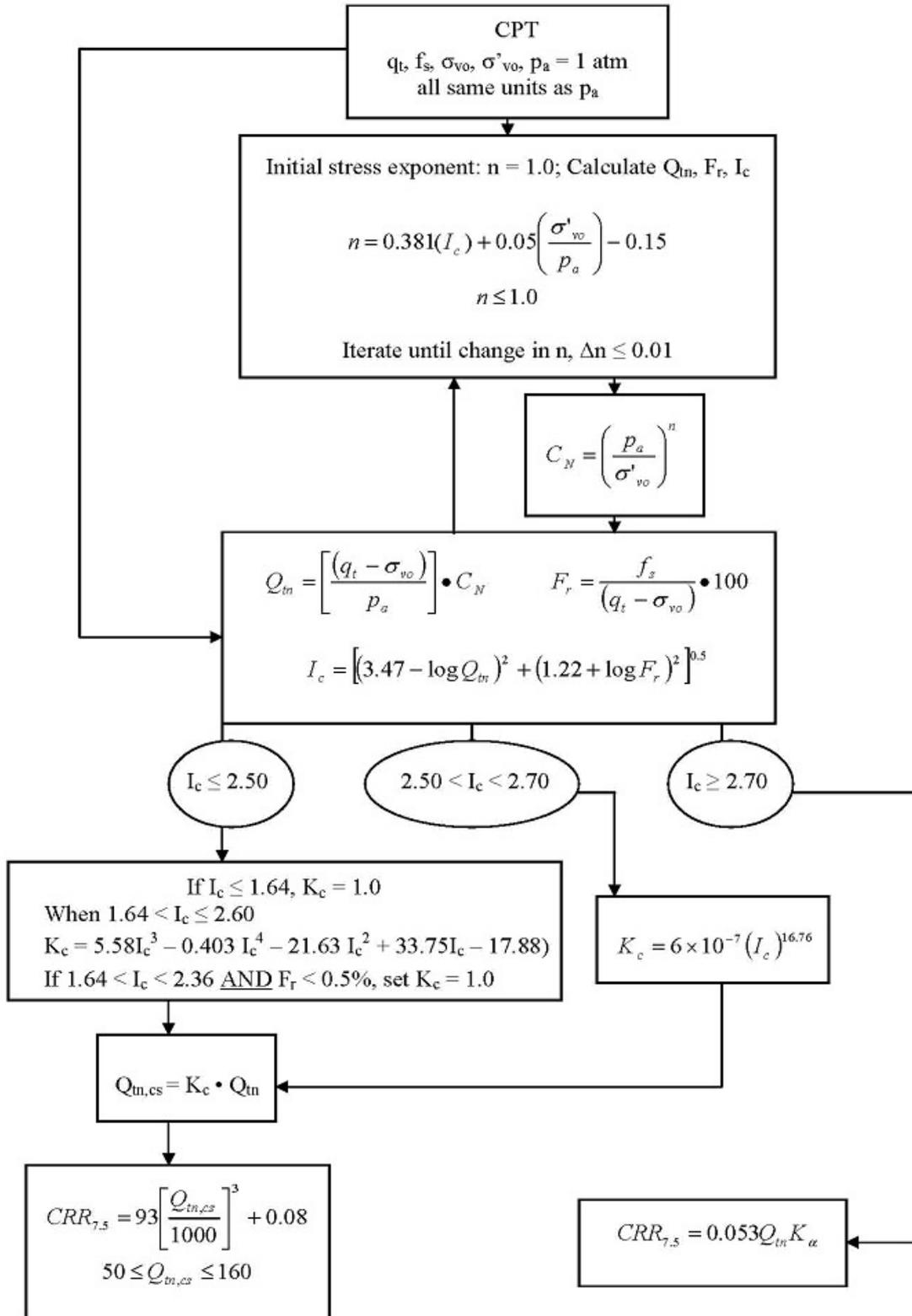
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart<sup>1</sup>:



<sup>1</sup> "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

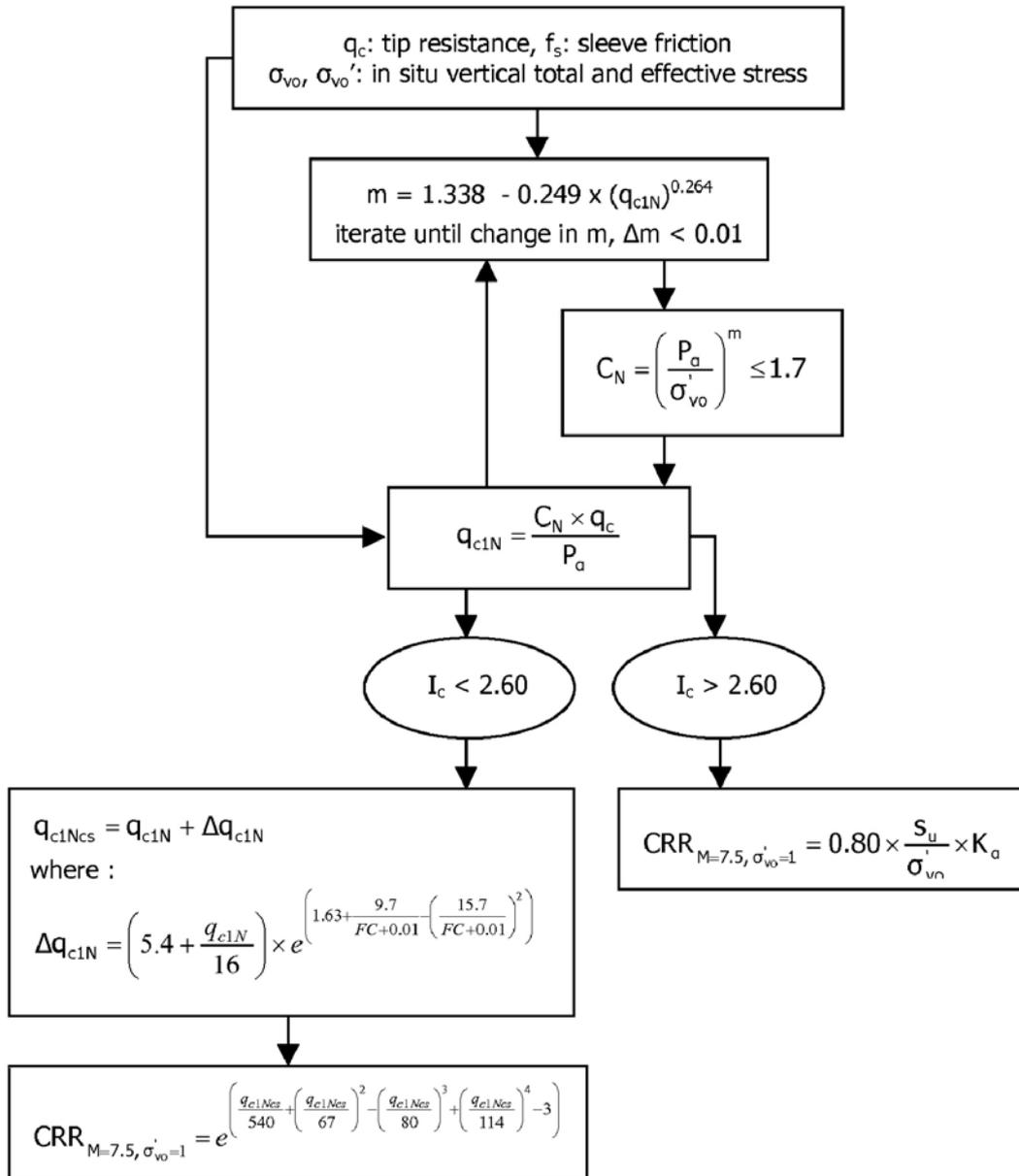
## Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart<sup>1</sup>:

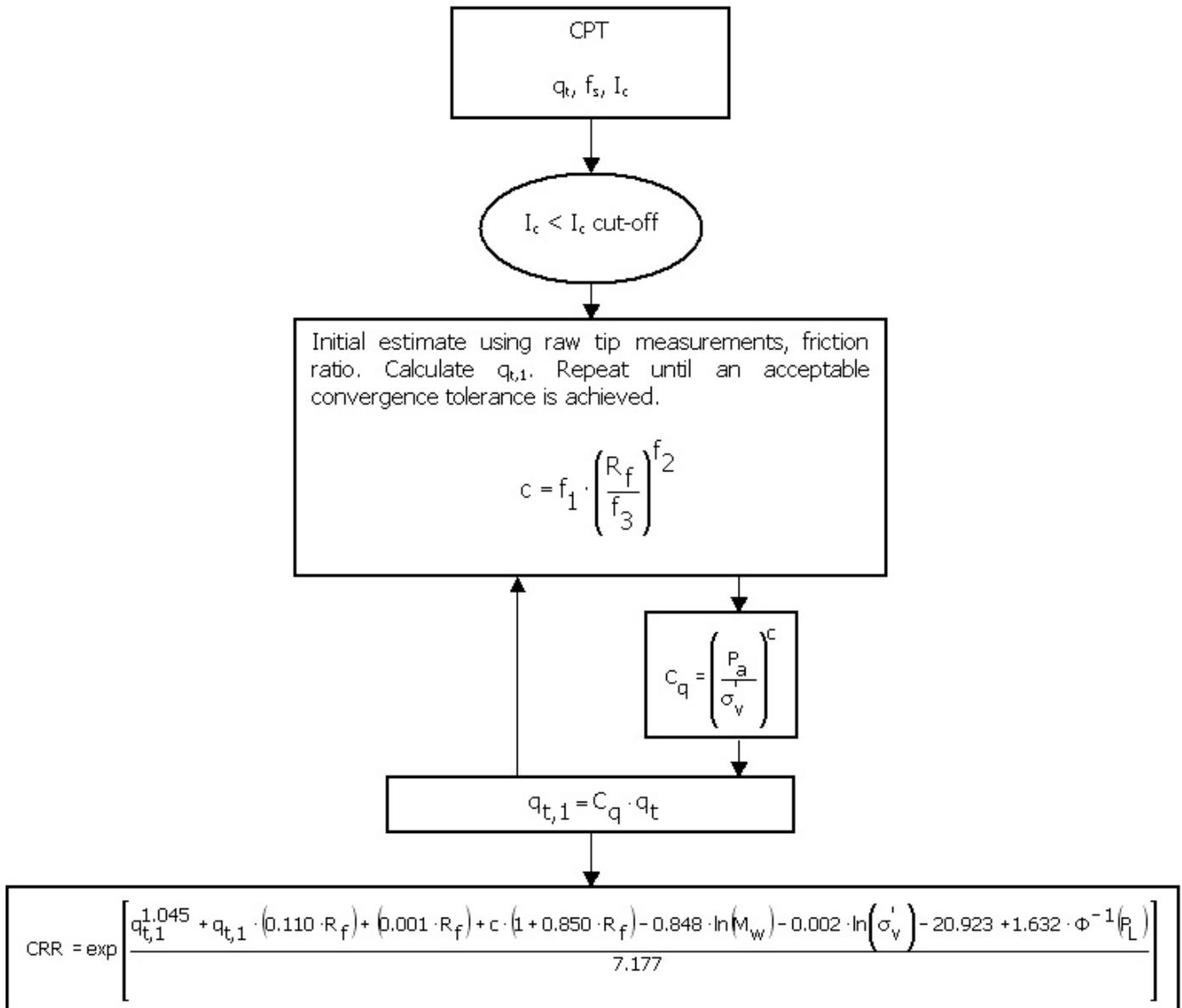


<sup>1</sup> P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

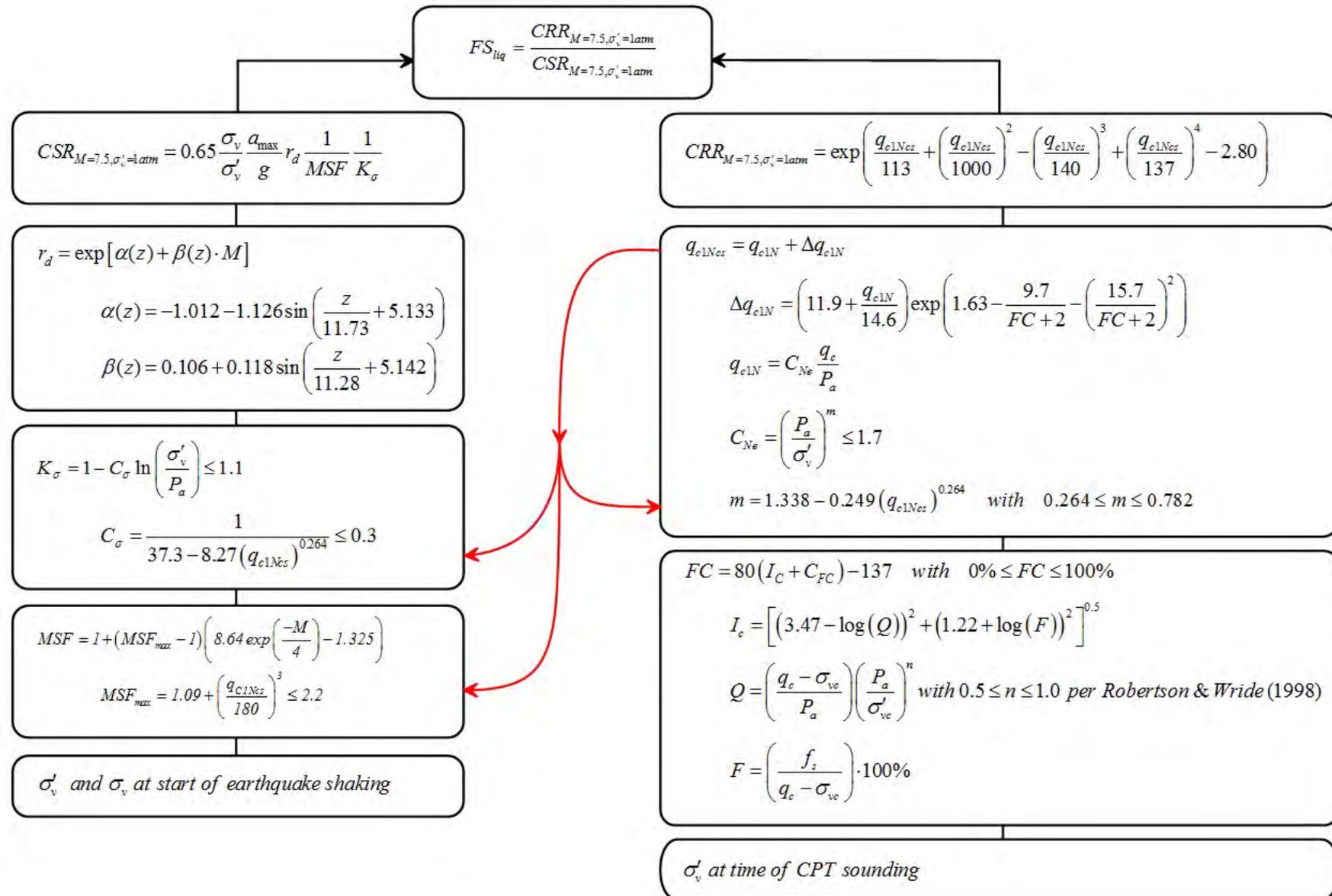
**Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)**



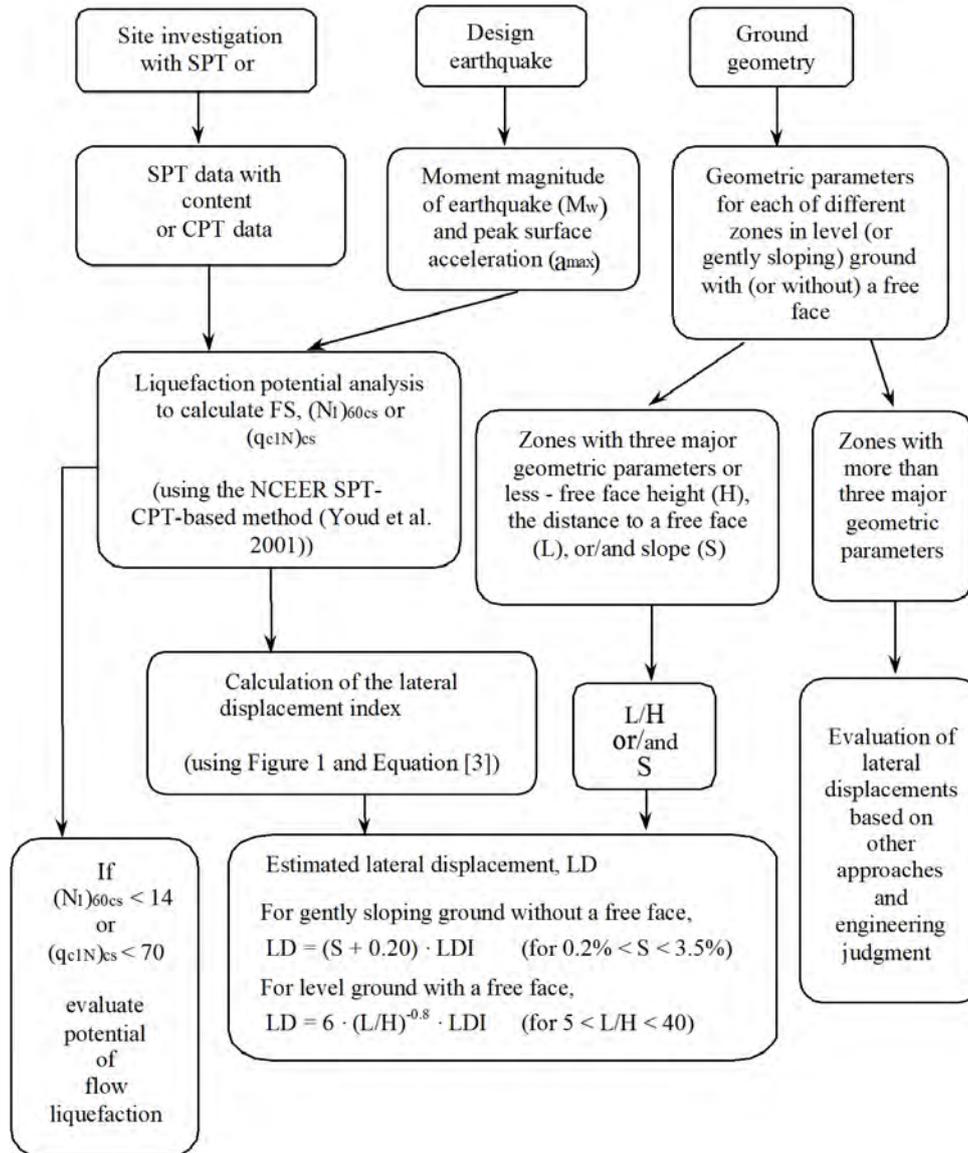
**Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)**



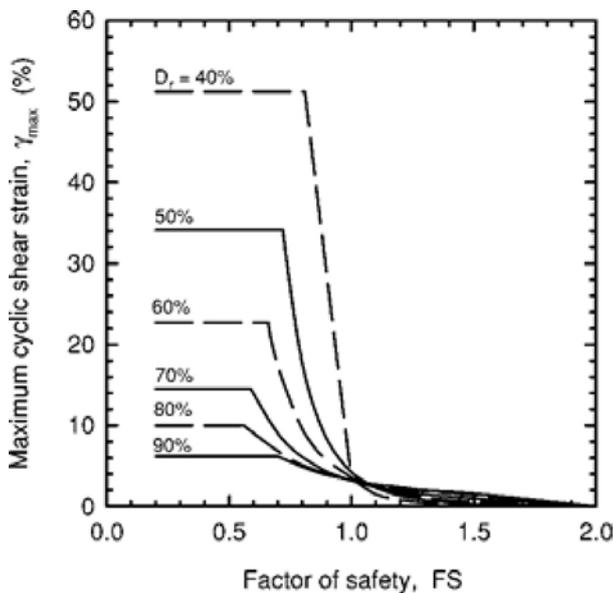
**Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)**



## Procedure for the evaluation of liquefaction-induced lateral spreading displacements



<sup>1</sup> Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



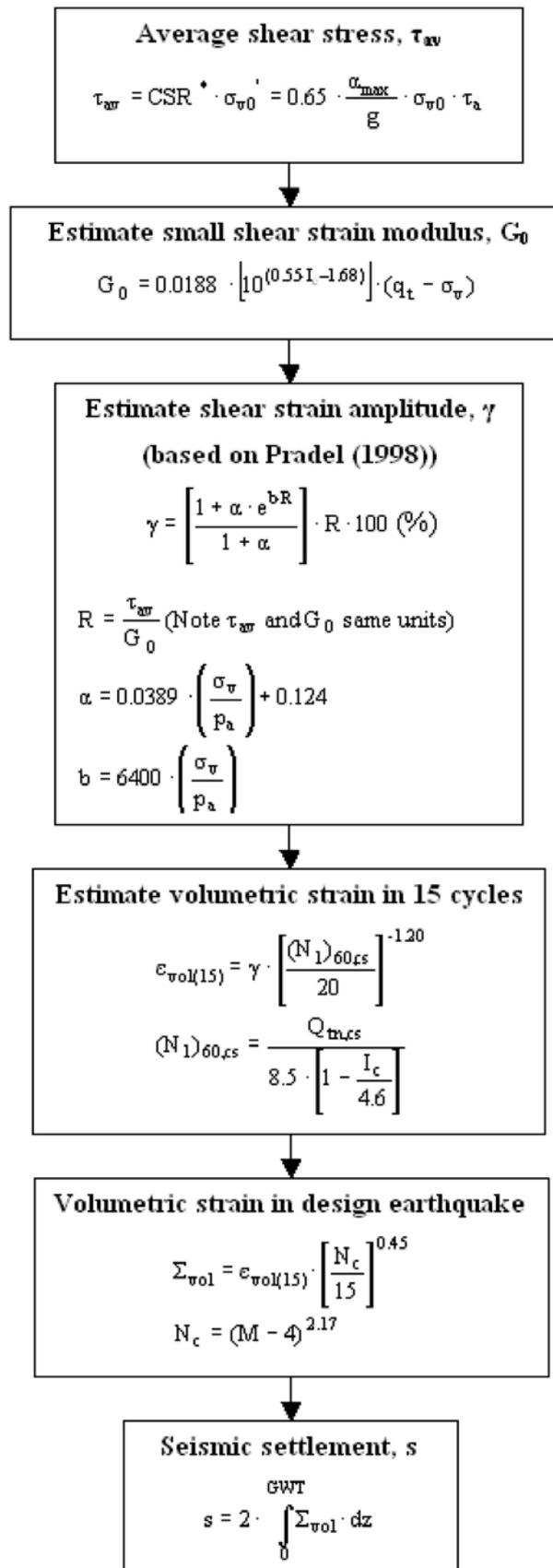
<sup>1</sup> Figure 1

$$LDI = \int_0^{Z_{max}} \gamma_{max} dz$$

<sup>1</sup> Equation [3]

<sup>1</sup> "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

## Procedure for the estimation of seismic induced settlements in dry sands



Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

## Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methodology developed by Iwasaki (1982) and is adopted by AFPS.

To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\mathbf{LPI} = \int_0^{20} (10 - 0,5z) \times F_L \times dz$$

where:

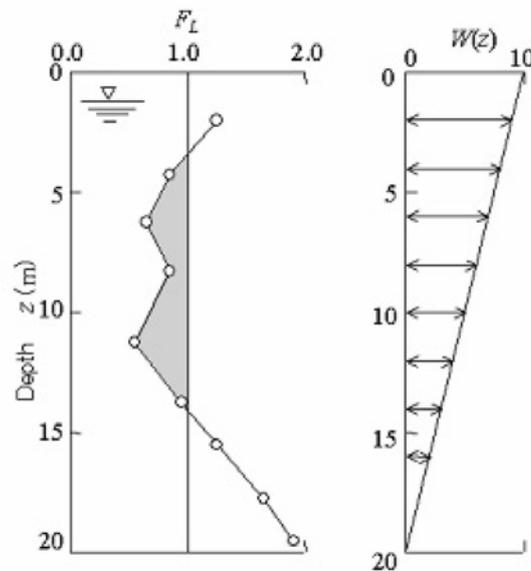
$F_L = 1 - F.S.$  when F.S. less than 1

$F_L = 0$  when F.S. greater than 1

$z$  depth of measurement in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- $0 < \text{LPI} \leq 5$  : Liquefaction risk is low
- $5 < \text{LPI} \leq 15$  : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high

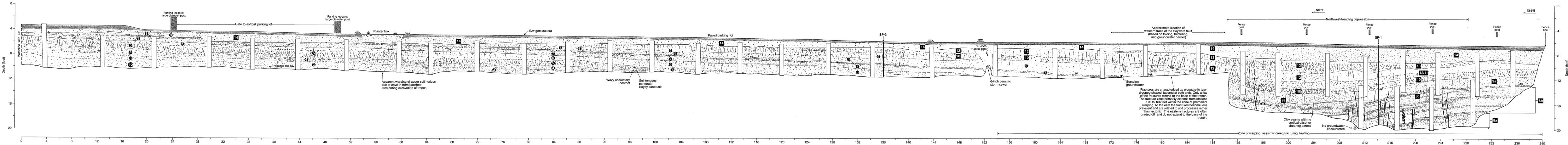


**Graphical presentation of the LPI calculation procedure**

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- R. E. S. Moss, R. B. Seed, R. E. Kayen, J. P. Stewart, A. Der Kiureghian, K. O. Cetin, CPT-Based Probabilistic and Deterministic Assessment of In Situ Seismic Soil Liquefaction Potential, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 8, August 1, 2006

APPENDIX D  
WLA TRENCH LOGS

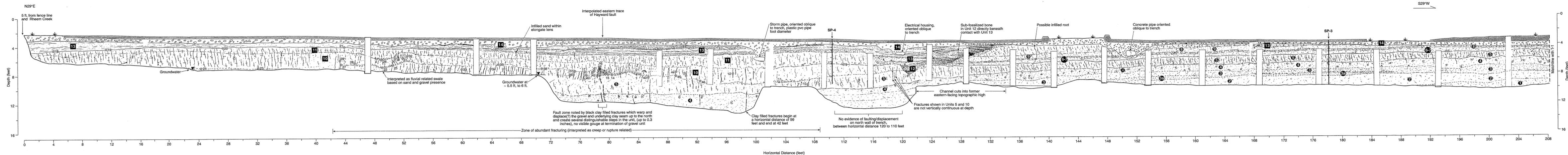


Depth (feet) 0, 4, 8, 12, 16, 20

Horizontal Distance (feet) 0, 4, 8, 12, 16, 20, 24, 28, 32, 36, 40, 44, 48, 52, 56, 60, 64, 68, 72, 76, 80, 84, 88, 92, 96, 100, 104, 108, 112, 116, 120, 124, 128, 132, 136, 140, 144, 148, 152, 156, 160, 164, 168, 172, 176, 180, 184, 188, 192, 196, 200, 204, 208, 212, 216, 220, 224, 228, 232, 236, 240

| Unit Descriptions  | Unit Descriptions (continued)   | Data Collection Stations   | Symbols   |
|--|---|--|---|
| <b>14</b> Fill   | <b>8a</b> Clayey silt (ML), 10YR 4/2 dark gray brown to silty clay (CL), 10YR 5/4 yellow brown, medium toughness, few fine pores, firm, angular pedologic structure, moderately developed clay coating, moderately plastic, some gray root casts, rare clasts, alternating layers of clayey silt and silty clay, lowest bed is a distinct incipient soil, a clay, mottled to dark gray brown (overbank deposits with soil development).   | <b>1</b> Fault: N17°W, 84°SW<br><b>2</b> Bedding: N15°E across trench<br><b>3</b> Bedding: N29°W 19°SW<br><b>4</b> Bedding: N26°W, 20°SW<br><b>5</b> Fault: N8°E, 88°E (doesn't offset overlying soil)<br><b>6</b> Fault: N10°W, 85°E<br><b>7</b> Fault: N3°E, 75°E (thin clay seam; doesn't offset overlying thin soil)<br><b>8</b> Fault: N8°E, vertical<br><b>9</b> Bedding: N38°W, 7°NE<br><b>10</b> Bedding: N30-35°W, 2-3°NE<br><b>11</b> Fault: N4°W, vertical<br><b>12</b> Bedding: N34°W, 4°E<br><b>13</b> Bedding: N27°W, 1-2°E (nearly flat)<br><b>14</b> Bedding: N27°W, 14°E<br><b>15</b> Bedding: N22°W, 6°E<br><b>16</b> Bedding: N10°W to N10°E, nearly flat (1-2' west)<br><b>17</b> Bedding: N30°E, N4°W<br><b>18</b> Bedding: N30°E, 5°NW<br><b>19</b> Bedding: N12°E, 5°NW<br><b>20</b> Bedding: N40°E, 2°NW (nearly flat)<br><b>21</b> Bedding: N46°E, 2°NW (nearly flat)<br><b>22</b> Bedding: N29°E, 1°NW (nearly flat)<br><b>23</b> Bedding: N42°E, 1°NW (nearly flat)<br><b>24</b> Bedding: N60°E, 1°NW (nearly flat)<br><b>25</b> Bedding: N29°E, 1°NW (nearly flat)<br><b>26</b> Bedding: N46°E, 1-2°NW (nearly flat)<br><b>27</b> Bedding: N14°E, 2°NW (nearly flat)<br><b>28</b> Bedding: N7°E, 1-2°NW (nearly flat)<br><b>29</b> Fracture: N15°W, 86°E<br><b>30</b> Fracture: N20°W, 86°E<br><b>31</b> Fracture: N2°E, 82°E<br><b>32</b> Fracture: N3°E, 80°E<br><b>33</b> Fracture: N17°E, 86°E | <b>2</b> Data collection point: bedding, fracture, fault<br><b>SP-1</b> Location of soil profile<br>Krotovina<br>Root<br>CaCO <sub>3</sub> caliche, nodules, and filaments<br>Soil structure<br>Fracture<br>Contact; dashed where approximate<br>Soil boundary<br>Fault; dashed where approximate |
| <b>13</b> Clayey silt (ML), 10YR 2/1 black, dry, high to medium dry strength, common pores, blocky prismatic structure, moderate to weak clay coatings, low to medium plasticity, common rootlets, basal contact is smooth to wavy and gradational to clear, upper horizon appears to be graded in places or has fill placed on top (overbank deposits). | <b>8b</b> Clayey silt (ML), 10YR 4/2 dark gray brown to silty clay (CL), 10YR 5/4 yellow brown, moist, medium toughness, few fine pores, firm, angular pedologic structure, moderately developed clay coating, moderately plastic, some gray root casts, rare clasts, alternating layers of repeating lithology, predominantly yellow brown silty clay with dark grayish brown organic overprinting, (overbank deposits with alternating beds of incipient soils).  | <b>7</b> Clayey silt (ML), 10YR 4/2 yellowish brown, massive, no bedding, soft, moderate to low plasticity, brittle, few medium pores, many small pores, weak clay coating, common root casts, occasional discontinuous organic laminae, occasional orange mottling, possibly very fine CaCO <sub>3</sub> filaments, basal contact is gradational to clear and smooth (overbank deposits).   |   |
| <b>12</b> Silt (ML), 10YR 4/3 brown, dry to clay 10YR 3/1, very dark gray, moist, common pores, high dry strength, non plastic, weak clay coating, common roots and rootlets, poor soil structure, basal contact is gradational and wavy, mottling from overlying soil (overbank deposits).  | <b>8c</b> Clayey silt (ML), 10YR 3/2 very dark grayish brown, moist, basal contact is clear to 10YR 3/2 gradational and smooth (soil developed on overbank fines).  | <b>8</b> Silty clay (CL) with minor very fine sand, 7.5YR 3/1 dark grayish brown, rare subangular pebbles, massive, weak pedologic structure, moderately developed clay films, fine subangular blocky structure, possible Bt horizon burying underlying A horizon, less than 2% very fine CaCO <sub>3</sub> filaments and very fine poorly developed CaCO <sub>3</sub> nodules, gradational basal contact, (overbank deposits with soil development).  |   |
| <b>11</b> Silt (ML), 10YR 4/2 dark gray brown, dry to clay (CL) 10YR 3/1, very dark gray, moist, common pores, medium to high dry strength, non plastic, subangular blocky structure, moderate clay coating, some very fine rootlets, basal contact is discontinuous (overbank deposits).  | <b>9</b> Silty clay with sand (CL), 10YR 3/1 very dark gray, trace sand at depth; mottled dark brown, occasional very fine rootlets, lower contact is wavy and gradational (soil boundary within overbank fine deposits).   | <b>5</b> Sandy clay (CL) to clayey sand (SC), 10YR 2/2, moist, massive, 30-40% fine sand, trace fine gravel, 50-60% fines as clay and silt, prominent soil tongues, iron-oxide nodules, CaCO <sub>3</sub> filaments, bioturbation, basal contact diffuse (fluvial deposits).   |   |
| <b>10</b> Clayey silt (ML), 10YR 2/1 black, moist, hard, high dry strength, subangular blocky structure, well developed clay coatings, low to medium plasticity, common rootlets, some bioturbation, moderate to well-developed soil, soil fingers up to 30 cm, long basal contact irregular and wavy, (overbank deposits with soil development).        | <b>8a</b> Clayey silt (ML to silt), 7.5YR 4/3 dark brown, contains interbedded very fine sand; moist; medium toughness, few fine pores, firm, angular pedologic structure, moderately well nitng (overbank silt deposits with multiple soil).   | <b>3-4</b> Silty clay (CL), 10YR 4/2 yellowish brown color, 10% sand, rare rounded to subrounded chert pebbles, massive, very fine pedologic structure, low to medium plasticity, sticky, olive to dark yellow brown mottles, bioturbation (overbank deposits).  |   |
| <b>9</b> Silty clay with sand (CL), 10YR 3/1 very dark gray, trace sand at depth; mottled dark brown, occasional very fine rootlets, lower contact is wavy and gradational (soil boundary within overbank fine deposits).  | <b>6</b> Clay with very fine sand and silt (CL), 2.5Y 4/3 to 4/4 olive brown, massive, no bedding, moist, soft to medium stiffness, common large pores, angular structure, minor clay coating, weak soil development, some discontinuous vertical fractures or root stains; lower half of the unit is very fine sandy silt, (2.5Y 3/3 dark olive brown); firm, moderately strong, moderate blocky structure, clay coating, many small pores, basal contact is clear and smooth; upper contact is clear and smooth; conformable with unit 8 (overbank deposits). | <b>1-2</b> Silty clay with sand (CL), 10YR 3/3 dark brown, 10-15% very fine sand, massive, dark yellow brown to dark brown mottles (overbank deposits).  |   |

Figure D-1



**Explanation**

**Unit Descriptions**

- 14** Fill
- 13** Sand with gravels (SW), 10YR 5/3 brown to light olive brown, occasional silt and clay interbeds, dry, typically well laminated and crossbedded sand of alternating grain size; overall poorly sorted, beds support multiple nested channels, basal contact is abrupt and clear to smooth and wavy in places (high energy channel deposits).
- 12** Gravelly clay (CL) to gravelly sand (SP), 10YR 4/3 brown to 10YR 5/4 yellowish brown, 35% clay, 25% gravel, 20% sand, 10% silt, moist, pockets of clay supported matrix alternating with clast supported gravels, (clay matrix is moderately plastic, firm and moderately tough), (gravels are poorly sorted, locally moderately bedded), basal contact is clear and smooth (channel deposits).
- 11** Silty clay (CL), 2.5Y 3/0 dark grayish black to 10YR 2/2 very dark brown, occasional subrounded fine clasts (3-5% clasts), massive, no bedding, poor to weak structure, angular surfaces, weak to little clay coatings, subrounded clasts define basal contact in places, basal contact is gradational and wavy (overbank deposits).
- 10** Silty clay (CL) with fine sand lenses, 10YR 3/1 dark gray to dark brownish gray, massive, up to 5% > 7 inch diameter subrounded clasts, firm, medium toughness, moderate clay coatings, moderate blocky structure, medium to high plasticity; sand lenses are up to about 2 inches thick, contains < 10% silt; basal contact is clear to gradational and smooth (fluvial deposit with moderate soil development). East of station 100, Unit 10 is a silty clay (CL), 2.5Y 3/0 gray to 2.5Y 3/3 black, moist, < 5% subrounded clasts, moist, some weak soil development, angular weak clay on surfaces, moderately plastic, medium toughness, firm to hard, locally basal contact is defined by gravels, basal contact is clear to gradational and wavy, locally fractures visible (overbank deposits).
- 9** Silty clay (CL), 10YR 3/1 dark brown to dark gray, moist, firm, very few pores, occasional rootlets, moderate to high plasticity, medium structure, weak prismatic structure, good blocky structure, medium to well developed clay coatings, basal contact is gradational and wavy (overbank fines).
- 8** Silty clay (CL) with some fine sand, 2.5Y 4/3 to 4/4 olive brown, moist, few pores, many root casts, moderate angular blocky structure, moderate to low plasticity, medium developed clay coatings, some soil fingers (dark brown in color from overlying soil), lower half of the unit is a dark grayish brown silty clay, moist, firm to hard, moderate blocky structure, few pores, occasional root casts, discontinuous moderately developed clay coatings, moderate to high plasticity, basal contact is clear and smooth (overbank deposits).

**Unit Descriptions (continued)**

- 6** Silty clay (CL), 10YR 3/1 dark gray brown, rare fine subrounded clasts; massive, moist, medium to hard toughness, some small pores, firm, moderately angular blocky structure, weak clay coatings, slightly plastic, rare CaCO<sub>3</sub>, basal contact is gradational and wavy (overbank deposits).
- 4** Clayey silt (ML) with very fine sand, 10YR 4/2 olive brown, massive, moist, medium firmness, weak structure, weak clay coatings, common pores, common root casts, abundant krotovinas to south, basal contact is clear and smooth becoming more gradational to north (overbank fines).
- 3** Sandy silt (ML) with clay, 10YR 5/2 yellowish brown to 10YR 4/2 olive brown, moist to dry, hard dry strength, medium firmness, moist, weak to no structure, no plasticity, mottles, pores common, some rootlets, grades to sandier toward north, basal contact is clear and smooth, (overbank fines).
- 2** Silty clay (CL), 10YR 4/2 yellow brown to 2.5Y 4/3 olive brown, no structure, no clay coatings, slightly more organic subunit within unit 3, basal contact is clear to gradational and smooth (overbank fines).
- 1** Very fine sandy silt (ML), 10YR 4/2 gray brown, minor clay, rare clasts, moist, slightly firm, large common pores, no structure, no clay coating, mottles, a few large (> 3cm diameter) gravels at base (overbank fines).
- 1** Very fine sandy silt (ML), 10YR 4/2 gray brown, some clay (more than unit 2), moist, slightly firm, large pores, no structure, weak clay coatings, no clasts (overbank fines).

**Data Collection Stations**

- 1 Bedding: N82°W, 1°N
- 2 Bedding: horizontal
- 3 Bedding: N70°E, 1°N (approx. horizontal)
- 4 Bedding: N65°W, 4°N
- 5 Bedding: N76°W, 5°N
- 6 Thalweg: N65°W, 6°N
- 7 Thalweg: N45°W, 0°
- 8 Thalweg: N45°W, 0°
- 9 Bedding: N89°E, 0.5°N (approx. horizontal)
- 10 Fracture: N10°W, 86°N
- 11 Bedding: N51°W, 3°N
- 12 Fracture: N60°W, vertical
- 13 Fracture: N57°W, vertical
- 14 Bedding: N80°W
- 15 Bedding: N89°E, 1°N (approx. horizontal)
- 16 Fracture: N45°W (fault)
- 17 Fracture: N60°E
- 18 Bedding: N35°W, 6°NE
- 19 Bedding: N55°W, 5°NE
- 20 Bedding: N80°E, 1°N (approx. horizontal)
- 21 Bedding: N30°W, 1°NE (approx. horizontal)
- 22 Swale: N88°E, horizontal (?)
- 23 Bedding: N80°E, 1°N (approx. horizontal)

**Data Collection Stations (continued)**

- 24 Bedding: N42°W, 1°N (approx. horizontal)
- 25 Bedding: horizontal
- 26 Bedding: N85°W, 12°N
- 27 Bedding: N85°W, 8°N
- 28 Bedding: N85°W, 1°N (approx. horizontal)
- 29 Fracture: N78°E, 85°N
- 30 Fracture: N70°E, 81°N
- 31 Fracture: N89°W, 81°S
- 32 Fracture: N60°W, vertical
- 33 Fracture: N78°W, 83°S
- 34 Fracture: N10°W, 86°N
- 35 Fracture: N85°W, 81°W
- 36 Fracture: N70°W, vertical
- 37 Bedding: N82°W, 1 to 2°S (approx. horizontal)
- 38 Bedding: N89°E, 1°N (approx. horizontal)
- 39 Fracture: N75°W, 84°S
- 40 Fracture: N65°W, 89°N
- 41 Fracture: N45°W, 86°N
- 42 Fracture: N86°W, 86S
- 43 Fracture: N43°W, 89N
- 44 Fracture: N69°W, 86N
- 45 Bedding: N65°W, 6°S
- 46 Bedding: N80°E, 1°N (approx. horizontal)

**Symbols**

- 28 Data collection point: bedding, fracture
- SP-1 Location of soil profile
- Krotovina
- Root
- CaCO<sub>3</sub> caliche, nodules, and filaments
- Soil structure
- Fracture
- Contact; dashed where approximate
- Soil boundary
- Fault; dashed where approximate

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**Figure D-2**