GEOTECHNICAL INVESTIGATION REPORT

Project Zeus

Mare Island, Vallejo, California

Prepared by:

Amec Foster Wheeler Environment & Infrastructure, Inc.

May 1, 2017

Project No. 6166150082

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Project Zeus
Confidential Client

Subject: Geotechnical Investigation Report
Project Zeus
Mare Island Site
Vallejo, California
Project No. 6166150082.14

Dear Project Zeus Client Team:

Amec Foster Wheeler Environment & Infrastructure, Inc. (Amec Foster Wheeler) is pleased to present this Geotechnical Investigation Report for the Project Zeus Mare Island site, located in Vallejo, California. We appreciate the opportunity to work with you on this project. If you have any questions or require additional information, please feel free to contact Chris Coutu at chris.coutu@amecfw.com or (510) 663-4156, or any of the undersigned team members.

Sincerely,

Amec Foster Wheeler Environment & Infrastructure, Inc.

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GEOTECHNICAL INVESTIGATION REPORT
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1.0 INTRODUCTION

Amec Foster Wheeler Environment & Infrastructure, Inc. (Amec Foster Wheeler) has prepared this geotechnical investigation report in accordance with our understanding of the scope of work as discussed with the Project Zeus team and described in our proposal dated August, 2015. Within this report, we present the results of our project specific field and laboratory investigation and analyses regarding the geologic and seismic hazards at the site located on Mare Island in Vallejo, California. Following the discussion of our investigation, we present recommendations to the Project Zeus project team regarding geotechnical considerations for the design and construction of proposed new structures and facilities at the Mare Island site.

1.1 PURPOSE

The purpose of this investigation and report is to perform field investigations and laboratory testing, and to develop and provide geologic, seismic, and geotechnical information and recommendations to facilitate the analysis, design, and construction of the proposed structures (warehouse and manufacturing facilities) proposed for the site on Mare Island. We understand that the project team has requested that Amec Foster Wheeler prepare this and other reports to assist the project team in planning, permitting, design, and development of the site. The study is further intended to fulfill the requirements for an Engineering Geologic and Geotechnical Report as defined in the 2013 California Building Code (CBC; California Building Standards Commission [CBSC], 2013.

1.2 PROJECT DESCRIPTION

The proposed Project Zeus Mare Island development site is a 144-acre parcel located on Mare Island along the east side of San Pablo Bay at the mouth of the Carquinez Strait, in Solano County, California. Mare Island is rectangular in shape, and is separated from the main part of the City of Vallejo by the Napa River and Mare Island Strait (Figure 1). The island is the former location of the Mare Island Naval base, which existed at this site for over 150 years. The project site on Mare Island is bounded on the north by Highway 37, on the west by Azuar Drive, on the south by G Street, and on the east by the Mare Island Straight (Figure 2). The majority of the site was previously developed and maintained as a part of the Mare Island Naval Base (Figure 3). However, we note that there is an approximately 400-foot wide area of wetlands along the east part of the project site, immediately adjacent to Mare Island Strait, and additional wetlands along the north part of the site adjacent to Highway 37.

Amec Foster Wheeler understands that ownership of the majority of the proposed project development site has been transferred to the City of Vallejo. The remaining portion of the site is still owned by the United States (U.S.) Department of the Navy (Navy), including the IR-17 site on the west side of the project. The Navy owned portions are currently undergoing remediation work to mitigate environmental contamination at IR-17. It is our understanding that the Navy parcels will be transferred to the city of Vallejo sometime in 2017.

Based on the preliminary site layout provided to Amec Foster Wheeler by Client, and shown on Figure 2, we understand that the development will include the following key components:

- New office, warehouse, and manufacturing buildings (about 2 million (M) square feet);
- New yard and road space (about 4M square feet);
- Various utility, infrastructure, and landscaping improvements.

We understand that column loads for the proposed facility are up to 500 kips per square foot (ksf), wall loads are up to 6 kips per lineal foot, and floor loads in some material storage areas could be as high as 3.5 ksf, although most floor loads will be lower.

We understand that the proposed manufacturing facility will include adjoining buildings for storage, assembly, and finishing processes, as well as an adjacent multistory office building. Other supplemental and support facilities include parking areas, receiving and storage areas, roadways, an electrical substation, storm water handling features, and product load out areas. In addition, levee or other flood control features may be constructed at the site.

1.3 SCOPE OF WORK

In support of the proposed Project Zeus development of the Mare Island site, Amec Foster Wheeler evaluated geologic and seismic hazards and developed recommendations for site development and foundation support for the proposed facilities at the site. Our evaluations included the following:

- Exploration program including review of previous studies, field reconnaissance, geotechnical field program, and laboratory and in-situ testing.
- Evaluation of the geology (including subsurface conditions), seismic and geotechnical hazards, and other hazards at the site.
- Evaluation of suitability of various soils and rock at the site for foundations, slabs, and other features
- Recommendations for foundations, excavations, grading, and other geotechnical considerations for design and construction of the project.

Services related to environmental assessment, characterization, and/or remediation were not part of the requested scope of work, and investigations and analysis of potential environmental contaminants have not been conducted or presented as part of this investigation.

1.4 PROJECT TEAM

Amec Foster Wheeler personnel led the geotechnical investigation and engineering services, coordinated subcontractors, prepared project deliverables, and communicated with the project team.

Underground utility clearance services were provided by SubDynamic Locating Services Inc. San Jose, California. Conventional and seismic CPTs were performed by Gregg Drilling of Martinez, California. Soil borings were drilled by Pitcher Drilling of East Palo Alto, California. Geotechnical laboratory testing was performed by Cooper Testing Labs of Palo Alto, California. Environmental testing of the geotechnical drilling spoils at the site was completed by Test America of Pleasanton, California.

1.5 ORGANIZATION OF REPORT

The body of this report is organized into seven sections. This section (section 1) provides a brief overview of the project description and scope of work. Section 2 covers the results of our review of the regional and local site setting, seismicity, and geology. Section 3 presents the results of our geotechnical data review and site reconnaissance. Section 4 and 5 present the results of the exploration and testing program and our analysis of this and other data regarding the subsurface conditions likely to exist at the site. Section 6 presents the results of our evaluation of the seismic and geologic hazards at the site. Section 7 presents geotechnical recommendations to support the project development. Section 8 presents general limitations and the basis for recommendations, and references are presented in Section 9.

The results of the geotechnical field explorations and geotechnical laboratory testing are presented in Appendixes A and B, respectively. Photographs from site reconnaissance activities are provided in Appendix C. Details of the methodology and results of the seismic hazard evaluation are provided in Appendix D.

2.0 REGIONAL AND LOCAL SITE SETTING

The Project Zeus site is located on Mare Island in Vallejo, California, on the site of a former U.S. Navy base. In this section, we present the results of our analysis of the regional geologic and tectonic setting, and the local seismic and site settings.

2.1 REGIONAL GEOLOGIC AND TECTONIC SETTING

West of the site and the Napa River estuary that separates the island from the mainland, lies a belt of low hills that flank the western edge of the Central Valley. Bedrock materials in the region are Cretaceous to Jurassic (approximately 65 to 206 million years before present) marine sedimentary rocks of the Great Valley Group (Graymer, 2002). Regionally the Great Valley Group occupies the boundary between the Central Valley and the California Coast Ranges. The marine sedimentary rocks have been folded and faulted as a result of subduction (compression) and strike slip motion occurring along the Pacific – North American plate boundary, forming northwest-southeast trending anticlines and synclines characterized by moderate to steeply dipping fold limbs (Dibblee, 1981; Graymer, 2002). These northwest-southeast striking sediments are exposed in the high ground across the southern part of Mare Island and in Vallejo across the Mare Island Strait. The regional geology of the site is shown on Figure 4.

Of note due to its proximity to the site, is the northwest-trending Franklin fault, which includes two strands as mapped by Graymer et al. (2002). The eastern trace is mapped along the Mare Island Strait east of the project site, and the western trace extends along the southwest margin of the island, and crossing northward through the northern half of the island to connect with the eastern trace north of State Highway 37 (Figure 4). The Franklin fault is not considered to be active, as discussed in Section 6 of this report. Other active faults near the site include the Hayward-Rodgers Creek, West Napa, and Concord Green Valley faults, as discussed the following section.

Quaternary erosion and deposition in the region has been controlled by vertical and lateral displacement on active faults and climatic effects. Specifically, sea level was more than 200 feet lower during the Last Glacial Maximum (LGM), extending from about 40 to 18 thousand years (ka), and rose steadily during the transition to the present interglacial epoch. Most of the sea level rise following the LGM occurred prior to about 7 ka, when sea level reached an elevation of about -33 feet (NAV88), followed by steady slow rise to the present (Meyer, 2014). When sea level was lower during the LGM, the shoreline was located well to the west of the present shoreline, and San Francisco Bay was characterized by through-going rivers and drainages extending offshore through the Golden Gate towards the Farallon Islands. The Napa River may have been entrenched along its present course along the Mare Island Strait merging with the Sacramento River at the intersection of the Carquinez Strait and San Pablo Bay, or may entered directly into San Pablo Bay northwest of Mare Island. During the LGM and late Pleistocene-Holocene transition period of sea level rise, alluvial deposition apparently occurred along the margins of the bedrock upland on the southern part of Mare Island, followed by deposition of bay and marsh deposits as sea level approached the present level in the mid to late Holocene.

2.2 SEISMIC SETTING

The San Francisco Bay region is considered to be one of the more seismically active regions of the world. During the past 200 years, faults within this plate boundary zone have produced numerous small-magnitude and at least fifteen moderate to large (i.e., M > 6) earthquakes affecting the Bay Area (Toppozada et al., 1981; Ellsworth, 1990; Bakun, 1999). The U.S. Geological Survey (USGS) Working Group on California Earthquake Probabilities (WGCEP) recently completed an assessment of the probability of occurrence of large magnitude earthquakes for the San Francisco Bay Area and all of California (Field et al., 2013; U.S. Geological Survey, 2015). The results of this study, titled the Uniform California Earthquake Rupture Forecast (UCERF) 3, indicate that the average repeat time for moment magnitude (**M**) 6 and 7 earthquakes in the San Francisco Bay Area is about 9 years and 48 years, respectively. In addition, the 30-year probability for occurrence of an earthquake of **M** 6 or larger is 98 percent, and for an earthquake of **M** 7 or larger is 51 percent.

Many active faults within the Bay Area contribute to the aggregate probability described above, and several may have significance with regard to potential earthquake ground shaking at Mare Island. Major active faults near the project site include the Concord-Green Valley, Calaveras, and Hayward-Rodgers Creek faults as shown on Figure 5. Other faults that may be the source of large earthquakes that could cause strong ground shaking at the site include the West Napa fault, Mt. Diablo blind thrust fault, the Clayton-Marsh Creek-Greenville fault system, and other discontinuous faults of the Contra Costa Shear Zone located south of the site along the hills between Walnut Creek and the Carquinez Strait.

Several moderate-magnitude nineteenth- and early twentieth-century events on or near the Hayward, Rodgers Creek, and San Andreas Faults generated notable ground shaking in the site vicinity (Toppozada et al., 1981; Toppozada and Parke, 1982a, 1982b), including the estimated magnitude 6.9 event on the southern Hayward Fault in October 1868, the estimated magnitude 6.4 Mare Island earthquake near the southern end of the Rodgers Creek fault in March, 1898, and the magnitude 8 (moment magnitude [**M]** 7.8) San Francisco earthquake on the San Andreas fault in April 1906 (Bakun, 1999). The epicenter of the 1898 magnitude 6.4 Mare Island earthquake is uncertain; however, violent shaking was reported (Modified Mercalli Intensity [MMI] IX) and numerous buildings on the island either collapsed or were damaged (Toppozada et al., 1981). Newspaper reports indicate that no chimneys were toppled on Mare Island, but about 10 percent of the chimneys in Vallejo suffered damage as a result of ground shaking from the 1906 earthquake; this damage is interpreted to indicate the ground shaking was MMI VI to VII (Boatwright and Bundock, 2005). The 1989 **M** 6.9 Loma Prieta earthquake was centered in the Santa Cruz Mountains near the San Andreas fault and resulted in moderate ground shaking in the general site vicinity (Modified Mercalli Intensity [MMI] V), with a peak ground acceleration

(PGA) of about 0.06 g at a station (epicentral distant 122 km) located at Dry Docks #3 (Boore et al., 1989) and 0.14 g at a station (MZD, epicentral distance 111 km) located along Highway 4 about 13 km southeast of Mare Island (U.S. Geological Survey Shakemap for October 17, 1989 Loma Prieta earthquake).[1]

The recent (August, 2014) M 6.0 South Napa earthquake was felt widely across the region and apparently fulfils the UCERF3 probability of occurrence for a **M** 6 earthquake in the Bay Area within the 30-year prediction time frame of UCERF3 (U.S. Geological Survey, 2015). Ground shaking in the vicinity of Mare Island is reported as very strong, (MMI VII). The closest recorded ground motion acceleration from the Center for Engineering Strong Motion Data (CESMD) database was at the strong motion station at the eastern end of the State Highway 37 bridge (CE.68310), located at the north end of Mare island, which reported the PGA as 0.19g at an epicentral distance of 10.8 km.[2]The next closest recorded ground motion acceleration was at the strong motion station at the south end of Mare Island (NC NMI Mare Island), which reported a PGA of 0.38g at an epicentral distance of 16.3 km. Similar PGA's for the South Napa earthquake was recorded at other stations in central Vallejo and Crockett. Damage reported from Mare Island included toppled chimneys, some damage to brick facades, parapets, and corrugated metal siding on buildings, and minor ground deformation. The observed ground deformation included ground cracking slight ground settlement around some foundations, and some cracking of pavement (Bray et al., 2014). These effects are consistent with MMI VI to VII shaking across the island.[3]

We note that there was limited evidence for occurrence of liquefaction and ground settlement anywhere in the area affected by the 2014 Napa earthquake, even in areas where the PGA exceeds the generally accepted lower limit of 0.15 to 0.2g necessary for occurrence of liquefaction. However, the absence of observed liquefaction and settlement in areas where young saturated deposits are present likely is due to the short duration of ground shaking resulting from this **M** 6.0 earthquake, and does not provide any evidence to indicate that such deposits are not susceptible to settlement and liquefaction effects that may result from larger magnitude earthquakes on nearby faults.

In summary, except for the 1898, 1906, and 2014 earthquakes, for which there are a few damage reports, none of the other historical earthquakes appear to have resulted in any significant damage to structures or ground failure effects in the Mare Island and Vallejo area.

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[1] Shakemap obtained from U.S. Geological Survey at: http://earthquake.usgs.gov/earthquakes/shakemap/nc/shake/Loma_Prieta/#Peak_Ground_Acceleration.

[2] CESMD Internet Data Report for the South Napa Earthquake of 24 August, 2014: http://www.strongmotioncenter.org/cgi-bin/CESMD/iqr_dist_DM2.pl?IQRID=SouthNapa_24Aug2014_72282711&SFlag=0&Flag=2

[3] Shakemap obtained from U.S. Geological Survey at http://earthquake.usgs.gov/earthquakes/eventpage/nc72282711#general_map.

Stronger ground shaking is expected to occur if large earthquakes occur on the Hayward, Concord-Green Valley, or other faults close to the site.

2.3 LOCAL SITE SETTING

Mare Island was originally developed starting in 1853 as the first Navy base on the Pacific Coast. As part of the development, roads, rail systems, and numerous buildings were constructed and many low lying marshy areas around the island were filled to create additional buildable land and dock facilities. This is particularly the case for the project site on the northern part of Mare Island, where the Fill was used extensively to raise the land and increase the usable surface area in the marshy regions along both sides of the narrow spit that connected the Strait. The original extent of the northern half of Mare Island as mapped in 1851, was considerably narrower than the current island footprint, about 700 to 1000 feet in 1851 compared to about 6, 500 feet in the present day (Figure 6). A major portion of site and most of the footprint for the proposed buildings lie in an area that was originally submerged mud flats along the Napa River/Mare Island Strait and San Pablo Bay (Figure 6). As San Pablo Bay and the Mare Island Strait lie within the zone of tidal influence in San Francisco Bay and the Sacramento River, mid Holocene to modern deposits along the margins of San Pablo Bay and on the west side of the Mare Island Strait are generally fine-grained, representing the deposition from slack water in the intertidal zone. In contrast, deposition of coarser grained (sandier) materials likely occurred along the banks of the through-going river present when sea level was lower during the latest Pleistocene (the LGM).

The current topography of the site is relatively flat lying, with surface elevations ranging from about 10 to 15 feet (above mean sea level) across most of the site, and local areas up to 20 feet elevation at some buildings. The easternmost portion of the site, within about 500 feet of the current shoreline along the Mare Island Strait, grades from about 10 feet elevation down to mean sea level. One other low lying area at an elevation of about 5 to 8 feet occurs at the northwest corner of the site near Q Street. In addition, there are a few man-made berms and roadways which are elevated several feet from the surrounding ground.

3.0 DATA REVIEW AND SITE RECONNAISSANCE

Amec Foster Wheeler reviewed geologic and geotechnical reports for previous developments at the project site, including results of field and laboratory investigations, and performed field reconnaissance to observe the existing surface and site conditions. The following sections discuss the results of our review and site reconnaissance.

3.1 DATA REVIEWED

Data from previous geotechnical investigations was used to supplement data obtained from field investigations performed for the present study. The existing data was particularly helpful to reduce the exploration work for this study, as well as to provide data for portions of the site where access was limited, such as in and around the Navy's IR-17 site. Existing studies reviewed included:

- California Department of Transportation (CALTRANS; 1964) Foundation Report Napa River Bridge Superstructure;
- California Department of Transportation (1993) Site Specific [Seismic Hazard] Analysis;
- California Department of Transportation (1995) Foundation Investigation for Seismic Retrofit Design of Napa River Bridge;
- Engeo (2006) Preliminary Geotechnical Cost Summary;
- Harding and Lawson (1978) Soils Report for building 513;
- Harding and Lawson (1982) Soil Investigation for Electrical Substation and Duct Bank;
- Harding and Lawson (1984) Foundation Investigation for Unaccompanied Enlisted Personnel Housing;
- Hoover and Associates (1988) Satellite Communications Technical Center Concept Study;
- Lavine Fricke (2001) Preliminary Geotechnical Engineering Study and Consolidation Evaluation;
- Naval Facilities Engineering Command (1980) Master Plan for [the] Mare Island Naval Complex;
- Parikh (2003) Geotechnical Design and Materials Report [for the] Mare Island Route 37 Project; and
- Various geotechnical explorations and laboratory testing completed by Engeo (2003, 2011) and others.

The data from these sources included logs and testing results of over 50 explorations from previous studies that were incorporated into our planning for new investigations and into our evaluations for this project. Locations of explorations performed by others on the Project Zeus site are shown on Figure 2. In addition, previous analyses by others included results of seismic hazard studies at the site, and the development of geologic cross sections at various locations within and around the site. This data was used to enhance our understanding of the site geology.

3.2 Site Reconnaissance and Surface Conditions

Amec Foster Wheeler conducted site visits and performed field reconnaissance at various times from May through December of 2015. Field visits were made to observe on-site conditions including exposed soils and existing foundations, and to prepare for and perform geotechnical explorations. Photos documenting specific observations made during field visits are included in Appendix C.

While on site, Amec Foster Wheeler observed a number of abandoned, damaged, and un-occupied buildings and foundations. These buildings include old residential buildings, an abandoned gas station, abandoned warehouse structures, docks, and other supporting facilities and structures. It is our understanding that the majority of the existing structures were supported on pile foundations. We understand that many of the duct banks and utility structures at the site were also supported on piles. Differential settlements between the free field ground surface and the pile supported utility lines and building foundations was observed to be as much as one to two feet or more.

The settlements were observed to have caused distress to the various structures and utility lines on site. Settlement adjacent to utility lines has caused distress to the overlying roads; in some cases causing over six inches of differential settlement in the existing roads. Distress included large upward mounding of the road in areas supported by the utility structures, potholes and caving of the subgrade and overlying asphalt, and general distress of the surrounding surface. Although some of these utility lines reportedly are abandoned, other active utility lines cross through the site, including water, sewer, gas, and electric utilities. We understand that the majority of the large utility lines through the site follow the roads, specifically Railroad Avenue.

In addition to the foundations and utilities, other on-site-features include abandoned rail lines, old wells/vaults, and other buried or abandoned structures. Most of the buried structures are located on the western and southern portions of the site in the areas of the densest historical development (see Figure 3). Other features of note include a shallow natural gas deposit at a depth of about 20 to 30 feet on the northwest corner of the site, possibly formed from decay of buried marsh deposits (at locations of CPT-1 and CPT-5 shown on Figure 2).

4.0 EXPLORATION AND TESTING PROGRAM

Amec Foster Wheeler performed an extensive exploration and testing program to facilitate development of data and recommendations as input to design of new facilities at the Mare Island site. The exploration and testing program was designed to develop a comprehensive understanding of the relatively complex geologic setting and challenging geotechnical conditions across the site, to provide data to limit the uncertainty in geotechnical parameters, and for development of cost effective recommendations for supporting new structures and improvements. The exploration program included geotechnical explorations (CPTs and soil borings), in-situ vane shear testing, geophysical testing, and geotechnical laboratory testing. The results of the exploration and testing program are discussed below.

4.1 GEOTECHNICAL FIELD EXPLORATION

As a part of the field work completed for this study, Amec Foster Wheeler conducted a series of geotechnical explorations, including cone penetrometer tests (CPTs), soil borings, and in-situ vane shear tests.

Prior to performing any ground disturbance activities at the site, Amec Foster Wheeler obtained access permission from the Navy, obtained the required permits from City of Vallejo and Solano County, and performed utility clearance for each of the proposed geotechnical exploration locations. Utility clearance consisted of notifying public and private utility companies of our intended activities and locations through Underground Services Alert, and reviewing locations with a private utility locating firm, Subdynamic of San Jose, California. As an extra precaution, all CPT and soil boring locations were pre-cleared using hand equipment to a depth of 5 feet prior to the start of drilling/pushing.

CPTs were performed by Gregg Drilling and Testing of Martinez California on October 9, 12, 13, 14, 15, 17, and December 4, 2015. CPTs provide a nearly continuous profile of subsurface materials and properties by pushing an instrumented probe into the ground. The CPT records probe tip resistance, probe sleeve resistance, and pore pressure at 2 cm intervals. The CPT also can measure shear wave velocity (i.e. seismic CPT or SCPT). These data are then processed and correlated to determine material types and engineering characteristics of the soils penetrated. At two of the CPT locations, shear wave velocity measurements were recorded (SCPT).

Because samples are not typically collected from the CPTs, Amec Foster Wheeler performed soil borings to collect samples of various subsurface soils identified from interpretation of the CPT data and to aid in characterization of the subsurface stratigraphy. Soil borings were performed by Pitcher Drilling of Palo Alto, California on October 19 through 23, 2015. Soil borings were advanced using rotary-wash drilling methods to optimize the stability of the drilled hole as well as the quality of soil sampling and testing. Several flight auger borings were performed as a matter of expediency when rotary wash procedures could not be used due to equipment issues. In addition to collecting samples, in-situ vane shear testing (VST) was performed to obtain a strength profile in two borings. The vane shear testing was performed by Robert Y. Chew Geotechnical of Hayward, California.

Subsurface explorations conducted for this study extended from about 30 to 120 feet below the ground surface (bgs), with the majority extending to depths greater than 60 feet bgs to characterize the thickness of soft Young Bay Mud deposits and portions of the underlying Old Bay Clay and/or Older Alluvial deposits. In total, over 1,700 feet of CPT/SCPT explorations, and 431 feet of soil boring explorations were completed during two phases of work. The initial phase consisted of 6 borings (2 with VST), 28 CPT's, and 2 SCPT's (seismic CPTs with shear wave measurements), and the second phase consisted of 3 CPT's in areas where access approval was delayed by the Navy. The exploration locations and sampling/testing were selected in consideration of the location of the proposed new facilities and the existing data from previous exploration programs by Engeo, Harding and Lawson, Caltrans, and others (Figure 2).

The original testing program presented in our proposal was modified to account for the additional useable data identified during the data review. The modified field and laboratory testing program focused on filling in any gaps in the existing data and supplementing existing information with additional testing. A combination of drive samples taken using Modified California Drive Samplers (with an inside diameter (I.D.) of 2.375 inches and an outside diameter (O.D.) of 2.5 inches), and thin wall Shelby Tube Samplers (with a 2.87 inch I.D. and a 3.0 inch O.D.) were obtained from the soil borings. The Modified California Drive samples are considered to be moderately disturbed, while the Shelby Tube samples are considered to be undisturbed. Bulk samples of the upper five feet of soil were also obtained from all borings and some of the CPT explorations.

The results of the field program including the logs of the CPTs, SCPTs, and soil borings are presented in Appendix A. Sample locations and types are indicated on the boring logs along with sample number, an indication of the laboratory testing completed on the sample, and in many cases, the laboratory result. The complete laboratory test results are discussed below and presented in Appendix B.

4.2 LABORATORY TEST RESULTS

The laboratory testing program was planned specifically to target areas of uncertainty or to fill gaps in the data obtained during previous studies. Undisturbed and moderately disturbed samples were collected at specific areas of interest.

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to evaluate the pertinent engineering properties of the subsurface soils. The following tests were performed in general accordance with standards of the American Society of Testing and Materials (ASTM), the California Department of Transportation (CAL), or the Uniform Building Code (UBC):

- Moisture Density (ASTM D 7263b)
- Sieve Analysis / 200 Sieve Wash (ASTM D 422 / 1140)
- Atterberg Limit Test (ASTM D 4318)
- Modified Proctor (ASTM D 1557)
- Consolidation (ASTM D 2435)
- Minimum Resistivity, pH, Chloride, and Sulfate Tests (CAL 643, 417, 422m)
- Expansion Index Testing
- R-Value (CAL 301)
- Direct Shear (Modified ASTM D 3080)
- Triaxial UU Testing (ASTM D 2850)

Details of the laboratory testing program are presented in Appendix B.

4.3 VANE SHEAR TEST RESULTS

Vane shear testing was completed at soil borings B-2 and B-6 on October 22, 2015 by Gregg Drilling and Robert Y Chew Geotechnical. Vane shear testing was used to determine peak and remolded shear strengths of the Young Bay Mud at depths ranging from about 18 to 35 feet. Results of the vane shear testing are indicated on the logs of soil borings B-2 and B-6 in Appendix A.

4.4 GEOPHYSICAL TEST RESULTS

Geophysical testing was completed to characterize the shear wave velocity of the site for use in analysis and interpretation of site class, soil stratigraphy, and site response analyses. Geophysical testing was completed by Gregg Drilling and Testing on October 9, and October 14, 2015 at the locations of SCPT-26 and SCPT-13, using the seismic cone penetrometer test equipment (SCPT). For this study, shear wave velocity readings were generally obtained by striking a source plate at the surface and measuring the response time at depth (in the tip of the SCPT probe). Interval speeds are obtained by taking readings at multiple depths and/or locations. Testing intervals were typically 5 to 10 feet.

Shear wave velocity (V_S) from downhole (DH), suspension log (PS), and SCPT testing were also measured as a part of previous exploration programs completed by others. Engeo had previously performed SCPT testing near the south east corner of the site (E11-C-01, Figure 2). Caltrans provided geophysical testing data in the vicinity of highway 37, including SCPT testing under the Walnut Avenue overcrossing (CAL-C1, Figure 2), as well as suspension logging (CAL-PS1, Figure 2), downhole (CAL-D1, Figure 2), and SCPT (CAL-C5, CAL-C9, Figure 2) testing in locations just to the north and east of the site under the Napa River Bridge.

Generally the shear wave velocity variations are consistent across the site and increase with depth. Shear wave velocity varied from about 160 feet per second (fps) near the surface to about 1800 fps at depth. The lowest shear wave velocities were observed in the Young Bay Mud. A plot of shear wave velocity versus depth from the available tests is shown on Figure 6.

5.0 SUBSURFACE CONDITIONS

Subsurface materials encountered during the exploration program consisted of fill (FILL), Young Bay Mud (YBM), Old Bay Clay (OBC), Older Alluvium (OA), and Bedrock (BDR). Classification and physical properties of these materials were determined based on the results of the field and laboratory testing, reported soil types and corresponding material properties from previous studies at the site, and our experience with similar soils and bedrock in the Bay Area.

5.1 GEOLOGIC MATERIALS

As noted above, the primary geologic materials beneath the surface at the project site include Fill, Young Bay Mud, Old Bay Clay, Older Alluvium, and Bedrock. Interpreted geologic contacts along idealized cross section A-A' (extending from Highway 37 to G Street) are presented in Figure 8. Interpreted geologic contacts along idealized cross sections B-B' and C-C' (approximately perpendicular to cross section A-A') are presented in Figure 9. The locations of cross sections A-A', B-B', and C-C' are shown on Figure 2. The location and depth of subsurface explorations from this investigation and from previous investigations used to develop the cross sections are shown as stick logs (showing geologic contacts) on the figures. Our interpretation of the extent of contacts between different materials is shown by dashed lines between the exploration locations. The cross sections represent our best estimate of the subsurface layering, based on the available information and on our interpretation of the geology at the site. Some variation within soil layers is expected. Actual conditions may deviate from those shown. Material properties, material types, and layer boundaries are subject to uncertainty with increasing uncertainty further from the geotechnical explorations.

5.1.1 Fill

Fill at the Project Zeus, Mare Island site was reportedly derived from local sources. Compaction and placement of the Fill, along with the sources from which the Fill was derived, are not well documented. Based on the geotechnical explorations, the thickness of Fill varies from about 3 to 12 feet across the site. A contour map of the elevation of the bottom of Fill is presented in Figure 10. The contours on Figure 10 were developed primarily from the thickness of Fill identified in the subsurface explorations shown on Figure 2. Actual conditions may deviate from those shown.

The Fill consists primarily of clayey gravel (GC) or clayey gravel with sand, but varies significantly from location to location and includes zones of poorly graded gravel, poorly graded sand, and fat clay. Some zones of the Fill are compressible, weak, and/or susceptible to liquefaction. The Fill also contains many abandoned utilities, abandoned foundations, some cobble-sized debris material, and organics. In some locations, Fill may contain chemical constituents from former activities that took place at the site. We are aware that the Navy is planning a remediation at the IR-17 site. Stained soil was observed in soil cuttings recovered from the Fill during drilling of soil boring B-3.

Results of geotechnical laboratory testing indicate that the Fill has an average unit weight of about 120 pcf with an average moisture content of about 23%. The shear wave velocity of the Fill is about 390 fps. The Fill is estimated to have a strength of 35 degrees with a cohesion of 100 psf.

We note that the Fill is quite variable in composition, and the average properties above may not adequately characterize the Fill in all locations. The thickness and composition of Fill should be observed during construction as required to verify that material properties meet specifications and design requirements.

5.1.2 Young Bay Mud

A continuous layer of Young Bay Mud underlies the Fill across the site. The Young Bay Mud is composed of very soft to soft fat clay. The Young Bay Mud varies in thickness from less than one foot to 53 feet with the shallower section generally to the west and south and thickening (deepening) to the north and east. We prepared an interpretation of the elevation of the bottom of Young Bay Mud across the site based on the depth of the contact from subsurface explorations and in consideration of the expected geomorphic setting of the site as sea level rose following the LGM. This interpretation is presented as elevation contours of the bottom of Young Bay Mud on Figure 11. We note that the contours are highly interpretive and should be used as a generalized indication of the elevation of the bottom of the Young Bay Mud layer. We expect there is significant variation of the actual base of Young Bay Mud from our generalized interpretation, particularly in areas where the thickness changes rapidly or in areas more distant from subsurface exploration locations. The contours on Figure 11 were developed primarily from the thickness of Young Bay Mud identified in the subsurface explorations shown on Figure 2. Actual conditions may deviate from those shown.

At exploration locations CPT-1 and CPT-5 in the northwest corner of the site, the Young Bay Mud deposit was found to contain natural gas under pressure. It is likely that the natural gas is from a confined peat layer within the Young Bay Mud. The extent of the peat is not well defined.

Testing of the Young Bay Mud was completed on relatively undisturbed Shelby tube samples recovered from soil borings B-1 through B-6. Results of geotechnical laboratory testing indicate that the Young Bay Mud has a total unit weight of about 100 pcf with an average moisture content of about 70%. The shear wave velocity of the Young Bay Mud is about 270 fps. The Young Bay Mud has an average plasticity index (PI) of about 45, and is estimated to have an undrained shear strength of about 500 pounds per square foot (psf).

5.1.3 Old Bay Clay

In most locations the Young Bay Mud layer is underlain by a layer of Old Bay Clay (OBC). The Old Bay Clay consists of soft to stiff fat clay with some interbedded sand and silt. On the west side of the site a relatively continuous layer of loose sand and sandy silt occurs at a depth of approximately 25 to 30 feet (below the Young Bay Mud and above the Old Bay Clay).

Based on the geotechnical explorations, the bottom of the Old Bay Clay layer was encountered at depths of about 90 to 95 feet bgs on the south end of the site to over 120 feet bgs on the north end of the site. The thickness of the Old Bay Clay layer is significantly reduced near the south west corner of the site (near the intersection of G Street and Azuar Drive).

Results of geotechnical laboratory testing indicate that the Old Bay Clay has a total unit weight of about 115 to 120 pcf, with a moisture content of about 30 to 35%. The shear wave velocity of the Old Bay Clay is about 585 to 725 fps. The Old Bay Clay has an average plasticity index (PI) of about 40, and is estimated to have an undrained shear strength of about 1250 to 2250 psf.

5.1.4 Older Alluvium

Older Alluvium beneath the Fill, Young Bay Mud, and Old Bay Clay generally consists of very stiff lean clay with some fine sand and silt.

Results of geotechnical laboratory testing indicate that the Older Alluvium has a total unit weight of about 120 pcf, with a moisture content of about 30%. The shear wave velocity of the Older Alluvium is about 950 fps, and the Older Alluvium is estimated to have an undrained shear strength of about 3,500 psf.

5.1.5 Bedrock

Regionally bedrock is composed of marine sedimentary rocks of the Great Valley Group. Bedrock logged in borings B-04, E11-B-01 (Figure 2), and in several CALTRANS borings to the north of the site is consistent with the geology mapped at the south end of the island (decomposed sandstone and claystone; Dibblee, 1981). Bedrock varies in depth from about 95 feet bgs on the south end of the site to over 240 feet just north of the site at State Highway 37 (Figure 8). The variability in the bedrock depth across the site is not known but is thought to generally increase towards the north and east. Based on data from Caltrans (1995), the bedrock is assumed to have a unit weight of about 140 pcf. The shear wave velocity of the bedrock at the site was taken at around 1,800 fps (550 m/s), consistent with the maximum recorded shear wave velocity at the site as recorded in CAL-D1 (Figure 2).

5.2 SITE CONDITIONS

Results of geophysical testing by Amec Foster Wheeler, Caltrans, and Engeo show that the shear wave velocity of Bay Mud, Old Bay Clay, and old alluvium in the upper 100 to 120 feet bgs is fairly consistent across the site, although the thickness of the deposits varies across the site (as shown on Figures 8 and 9). For the purposes of characterizing the site class, the three SCPT closest to the proposed building (SCPT-13, SCPT-26, and E11-SCPT-01) were used. The average shear wave velocity values over the top 100 feet (30 m) (V_{S30}) obtained from these three SCPT locations ranged from 443 to 449 fps (132 to 137 m/s) which falls within the range for Site Class E (defined as less than 180 m/s in ASCE 7-10 and the 2013 CBC). Review of the boring logs, and other data show that this site condition (Site Class E) is likely is present over the majority of the site, and particularly under the building. We note that this shear wave velocity is low enough that site response analysis is necessary to appropriately characterize ground motions for the soft soils at the site. This is appropriate as the site velocity falls below the applicable range of ground motion prediction equations (GMPEs) as described in Section 6.

5.3 GROUNDWATER CONDITIONS

Groundwater at the site is fairly shallow because the site elevation is low (typically 10 to 15 feet above mean sea level), and because the site is adjacent to the Mare Island Strait and San Pablo Bay (both at sea level). Thus groundwater is typically expected to occur at about sea level. Standing groundwater was observed in several excavations at the locations of former structures at the site. The depth to groundwater observed in the excavations or depressions varied from about 2 to 4 feet below the ground surface. Groundwater was also observed in several of the latest borings to extend from about 2.5 to 8 feet below the ground surface. The depth to groundwater recorded on logs of previous explorations at the site varied between 3 and 10 feet below the ground surface. Based on this data, it should be expected that groundwater may be as shallow as 2 feet below the ground surface or shallower in some low-lying areas, but more typically will be about 5 feet to 8 feet below the ground surface.

6.0 GEOLOGIC AND SEISMIC HAZARDS EVALUATION

Hazard evaluations were completed based on the characterization of the regional and local geology determined from the lab and field test data and based on our judgement and experience in the area. Amec Foster Wheeler performed these analyses in accordance with the applicable portions of the 2013 CBC and ASCE 7-10, and following the standards of engineering geology and geotechnical engineering practice.

Seismic hazards considered for the proposed project developments include strong ground shaking, surface fault rupture, ground deformation (soil liquefaction and ground settlement), slope instability, and tsunami inundation. Geotechnical and geological hazards that are considered include soil shrink and swell potential, soil corrosion, settlement and consolidation, and flooding. These hazards may detrimentally impact foundations and structures and are evaluated to provide adequate design consideration for the potential hazards at the site.

The following sections present the results of our evaluation of the seismic and geotechnical hazards at the Mare Island project site. Section 6.1 presents our assessment of site specific and code-based ground motions at the site for the top of rock below the site. Section 6.2 describes modification of the site specific ground motions for soil amplification effects. Sections 6.3 and 6.4 present our assessment of geological and geotechnical hazards, and seismic-geologic hazards, respectively.

6.1 SEISMIC HAZARD ANALYSIS

This section provides an assessment of earthquake-induced ground shaking potential for the Mare Island site. As part of this assessment, both a probabilistic seismic hazard analysis (PSHA) and a deterministic seismic hazard analysis (DSHA) were performed to characterize earthquake ground shaking that may occur at the site during future seismic events in the region. The PSHA was conducted to estimate the probability of exceedance of peak ground acceleration (PGA) and response spectral accelerations (S_a) at the site during selected exposure times. The DSHA was conducted to estimate PGA and S_a that may be experienced at the site due to large magnitude earthquakes on active faults in the region. Specifically, the objective of the assessment was to develop site-specific horizontal response spectra suitable for use in the evaluation and design of the planned facilities for Mare Island site and as required by ASCE 7-10 and the 2013 CBC. Further detailed information on the methodology and equations used for the calculation of hazard are presented in Appendix D. A brief summary of the PSHA and DSHA results and the development of the site specific design spectra are provided in this section.

6.1.1 Approach for Probabilistic Ground Motion Analysis

The probabilistic analysis, commonly termed a "probabilistic seismic hazard analysis" (PSHA) is based on an assessment of the recurrence of earthquakes on potential seismic sources in the region and on ground motion prediction equations appropriate for the types of seismic sources in the region and the subsurface conditions interpreted for the project site. Results of the hazard analysis are expressed as relationships between amplitudes of peak ground acceleration and response spectral acceleration, and the annual frequencies or return periods (return period being the reciprocal of annual frequency) for exceeding those ground motion amplitudes.

The PSHA analysis procedure requires the specification of probability functions to describe the uncertainty in both the time and location of future earthquakes and the uncertainty in the ground motion level that will be produced at the project site. The basic elements of the analysis are:

1. Identification of potential (active) seismic sources that could significantly contribute to seismic hazard at the project site;
2. Specification of an earthquake recurrence relationship for each seismic source, defining the frequency of occurrence of various magnitude earthquakes up to the maximum magnitude possible on the source;
3. Specification of attenuation relationships defining ground motion levels as a function of earthquake magnitude and distance from an earthquake rupture; and
4. Calculation of the probability of exceedance of peak ground acceleration and response spectral accelerations (i.e., seismic hazard) using inputs from the elements above, and development of equal-hazard (i.e., equal-probability-of-exceedance) response spectra from the results.

The probabilistic seismic hazard analysis conducted for this study is based on a seismic source model for the San Francisco Bay Area and surrounding region developed by Amec Foster Wheeler. This seismic hazard model is based on information presented in published and unpublished source, primarily fault parameters and activity rates developed by the U.S. Geological Survey (USGS) and California Geological Survey (CGS) Working Group on California Earthquake Probabilities (WGCEP) in models published in 2003, 2008, and 2013 (WCGEP, 2003, 2008, and Field et al., 2013). The fault sources include results prepared using both time dependent and time independent models (Tables 1 and 2, respectively), and models for multiple segment and linked fault ruptures. The fault traces used in the seismic hazard analysis are shown on Figure 5.

The ground motion prediction equations (GMPEs) selected for this analysis are the Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation (NGA) West 2 equations. These models provide estimates of spectral accelerations in the period range of 0.01 seconds to 10 seconds (spectral frequencies of 0.1 to 100 Hz), representing the median horizontal component of ground motions. The GMPEs are defined in terms of **M** (moment magnitude). Four of the NGA West 2 models (Abrahamson et al., 2014 [ASK14]; Boore et al., 2014 [BSSA14]; Campbell and Bozorgnia, 2014 [CB14]; Chiou and Youngs, 2014 [CY14]; and Idriss, 2014 [ID14]) were used in the analysis. The initial ground motions were developed for the top of weathered rock, with average shear wave velocity (V_{S30}) of 550 m/s.

6.1.2 Results of the Deterministic Seismic Hazard Analysis

Deterministic response spectra represented by the median and 84th percentile ordinate values are developed for maximum earthquakes occurring on the active and potentially active faults that are capable of producing the strongest ground shaking at the site. We considered maximum earthquake scenarios for the Hayward-Rodgers Creek, Franklin, West Napa, Concord-Green Valley and San Andreas faults (Figures 4 and 7). We note that the Franklin fault is included in the 2013 Uniform California Earthquake Rupture Forecast (UCERF) 3 probability by the WGCEP (Field et al., 2013). This fault is not included in the seismic source model for the PSHA prepared for this analysis because several studies have shown that it does not appear to be active (see Section 6.4) . However, because it is considered in UCERF 3, we have provisionally considered it as a source in the DSHA. The final earthquake scenarios used the DSHA were developed based on consideration of combined ruptures from Table 1 (WGCEP, 2008) and scenarios developed for this study.

The deterministic spectra were developed using the same weighted GMPE's used for the probabilistic analysis. The specific parameters for slip type, distance, and magnitude for the faults used in the deterministic analysis are listed in the table below. As noted above, the V_{S30} is taken as 550 m/s. Additional parameters required for implementation of the NGA West 2 GMPEs include the fault dip (90 for the sources listed below), location on hanging wall or footwall (if appropriate), and depth to a shear wave velocity of 1.0 km/s and 2.5 km/s. In the absence of site specific information, the depths to a shear wave velocity of 1.0 km/s and 2.5 km/s are taken as default values calculated in the GMPEs.

Fault[4]	Slip Type	Distance - R_{jb} (km)	Distance - R_{rup} (km)	M_W	Slip Rate (mm/yr)
Franklin	SS	0.7	0.7	6.8	< 1
West Napa	SS	6.7	6.7	6.9	1-2
Hayward-Rodgers Creek	SS	7.3	12.7	7.3	8-10
Concord-Green Valley	SS	14.0	14.0	7.1	3-4
San Andreas	SS	41.9	41.9	8.0	17-24

The median spectra for these fault rupture scenarios are presented in Table 3. The highest median (and 84th percentile) deterministic spectra result from the **M** 6.8 rupture on the Franklin fault at a closest distance of 0.7 km for all periods (columns 3 and 4 of Table 4). The ASCE 7-10 requires use of deterministic spectra at the 84th percentile level, and it also requires that response spectra be adjusted from mean demand (termed GMROTD50 for NGA West 2 GMPEs) to maximum demand. Period dependent factors to adjust median NGA West spectra to

[4] Distance measures are calculated as appropriate for use with individual NGA GMPEs, where R_{jb} represents the closest distance to the surface projection of the fault, and R_{rup} represents the closest distance to the fault.

84th percentile spectra developed by (Huang et al., 2008) are listed in the right-most column of Table 3.

6.1.3 Results of the Probabilistic Seismic Hazard Analysis

The basic results of the PSHA are presented below in terms of annual frequency of exceedance versus spectral acceleration (commonly referred to as hazard curves). Detailed seismic hazard results were developed for the Mare Island site to show the total mean hazard and relative contributions from each individual seismic source to peak ground acceleration (PGA) and spectral acceleration at 1.0 second period. We also prepared deaggregations of the PGA and 1.0-second period results for a 2% P_E in 50 years (Figure 12).

The results indicate that the largest contributions to hazard are from the Hayward and West Napa faults (at 10 km distance bin) and from the Concord Green Valley faults (at the 15 km distance bin), with a smaller contribution from the Contra Costa Shear Zone.

For this study, we provided five-percent damped horizontal equal-hazard response spectra for 2% P_E in 50 year time periods (corresponding to an equivalent return period of 2,475 years) (Figure 13; column no. 1 of Table 4), and other return periods as described in Appendix D. This spectrum represents the average or GMRotD50 result of the NGA West 2 GMPEs.

Two adjustments for horizontal ground motions are specified in ASCE/SEI 7-10, including 1) increasing spectra from geomean (GMRotD50) to maximum demand (GMRotD100), and 2) modifying probabilistic spectra to be risk-targeted. The USGS National Seismic Hazard Mapping Project prepared region-specific factors for short period (0.2 second) and long period (1.0 second) spectra. The risk-targeted factors are taken as the same at shorter and longer periods, respectively, and are interpolated for periods between 0.2 and 1.0 seconds (column 2 of Table 4). Similar to the deterministic response spectrum, the probabilistic response spectrum also is adjusted to maximum demand; the factors to adjust mean to maximum demand are taken from the Federal Emergency Management Agency (FEMA) P-750 (Building Seismic Safety Committee [BSSC], 2009) and are interpolated for additional periods. The probabilistic equal hazard response spectrum is compared to code-based general procedure spectrum and to the deterministic response spectrum to develop the design response spectrum as described below.

6.1.4 Development of General Procedure Response Spectrum

The general procedure spectrum (GPS) is constructed following the procedures of Section 1613.3 of the 2013 CBC and Section 11.4 of ASCE/SEI 7-10. The values of S_S, S_1, F_a, and F_v used in development of the site-adjusted maximum considered earthquake (MCE) spectra are listed below. The values of S_S and S_1 were obtained from the USGS National Seismic Hazard Mapping Program (NSHMP) website (http://earthquake.usgs.gov/hazards/designmaps/) for ASCE/SEI 7-10 MCE maps using the same site location used for the PSHA. The values of F_a

and F_v are for Site Class C, which is identified as the appropriate Site Class designation for the top of weathered rock below the Mare Island site (based on V_{s30} = 550 m/sec described in Section 5.1.5).

Parameter	MCE (2010 CBC Maps)
S_S (0.2 sec S_A)	1.50 g
S_1 (1.0 sec S_A)	0.60 g
Site Class	C
F_a	1.0
F_v	1.3
$S_{MS} = S_S * F_a$	1.50 g
$S_{M1} = S_1 * F_v$	0.78 g

Based on the value for S_1 identified above, the seismic design category is identified as C as specified in Section 11.4.5 of the ASCE7-10. The long period transition, T_L, is identified at 8 seconds from the NSHMP.

The GPS for the MCE_R is calculated from the site-modified spectral parameters following the ASCE/SEI 7-10 (2010) Section 11.4.5 and 11.4.6, and is shown in the column no. 5 of Table 4. The Design GPS is equal to two-thirds of the MCE GPS (column no. 6 of Table 4). The GPS are risk-targeted and are at maximum demand as described in Petersen et al. (2014).

6.1.5 Development of Site-Specific Horizontal Response Spectra

Two levels of ground shaking are specified in Chapter 21 of the ASCE/SEI 7-10 for use in design of new buildings; these are defined as follows:

1. Risk-Targeted Maximum Considered Earthquake (MCE_R). The MCE_R spectral response acceleration at any period, S_{aM}, shall be taken as the lesser of the spectral response accelerations from the risk-targeted probabilistic maximum considered earthquake and the deterministic maximum considered earthquake. The risk-targeted probabilistic MCE is taken as the mean spectra at the 2 percent in 50 years probability of exceedance level multiplied by the risk-target factor. The deterministic MCE is taken as the 84th percentile deterministic spectra spectral response acceleration computed at that period, for characteristic earthquakes on the active and potentially active faults within the region, but not less than the deterministic limit spectrum. The deterministic limit is taken as the response spectrum developed from the site coefficients F_a and F_v, with the value of the mapped short period spectral response acceleration (S_S) taken as 1.5g and the value of the mapped spectral response acceleration at 1.0-second period (S_1) taken as 0.6g. ASCE/SEI 7-10 requires that the site-specific probabilistic and deterministic spectra be adjusted to maximum demand.

2. Design. The design level ground motion is defined as equal to two-thirds times the MCE_R spectrum. The site-specific design spectrum may not be taken as less than 80-percent of the design response spectrum constructed following the General Procedures Approach.

The MCE_R and Design spectra are evaluated following the approach specified in Chapter 21 of ASCE/SEI 7-10.

A comparison of the site-specific risk-targeted horizontal probabilistic spectrum for 2 percent P_E in 50 years, the 84th percentile deterministic spectrum (both adjusted to maximum demand), and the deterministic limit spectrum is illustrated on Figure 14 and in Table 5 (column nos. 11, 8, and 9, respectively). The deterministic MCE spectrum is taken as the higher of the deterministic limit spectrum and 84th percentile deterministic spectrum (is taken as the 84th percentile spectra at all periods less than 7.5 seconds as shown in column no. 10 of Table 4). The resulting deterministic MCE spectrum is higher than the risk-targeted 2% P_E in 50 years spectrum at all periods between 0.03 seconds and 0.75 seconds, and the probabilistic spectra are higher at shorter and longer periods (Figure 14, and columns 10 and 11 of Table 4). The MCE_R spectrum is taken as the lower of these two as shown on Figure 14 and in column no. 12 in Table 4).

The site-specific design level spectrum of ASCE/SEI 7-10 is equal to ⅔ * MCE_R spectrum, but must not be less than 80 percent of the design GPS at any period (ASCE/SEI 7-10 Section 21.3). A comparison of 80 percent of the design level GPS (equal to ⅔ * MCE GPS * 0.8) and ⅔ * MCE_R (site-specific) is shown in column nos. 14 and 13 of Table 6. The ⅔ * MCE_R spectra are all higher than the corresponding 80 percent of the design level GPS. Therefore, the site-specific Design spectra are taken as ⅔ * MCE_R spectra at all periods (column 15 of Table 6).

In summary, site-specific five-percent damped horizontal MCE_R and Design response spectra presented in columns no. 12 of Table 5 and no. 15 of Table 6 are appropriate for use in site response analysis to develop time histories of ground motion and response spectra at the foundation level of the planned new buildings and other structures. The specific target spectra to be used in the site response analysis should be selected by the client based on the selected building occupancy category from ASCE/SEI 7-10.

6.2 SITE SPECIFIC GROUND MOTIONS AND SITE RESPONSE

Determination of the peak ground acceleration (PGA) at the ground surface was made using the site-specific procedures outlined in ASCE 7-10. The site specific ground motions were modeled using an equivalent linear random vibration theory analysis (RVT) as implemented in STRATA ([alpha, revision: 399]; Kottke and Rathje, 2009).

6.2.1 Site-Specific Site Response Analysis

The site response analysis was performed to modify ground motions developed for the top of weathered rock to account for the effects of the soft soil profile at the project site, in particular the expected de-amplification effect of the soft soil profile on ground motions propagating to the ground surface. The RVT methodology was developed to relate the Fourier amplitude spectrum (FAS) to the acceleration response spectrum in the absence of time domain input motions. The RVT method allows the estimation of the expected peak values for each frequency using the statistical characteristics (first four moments) of the FAS and the expected ground motion duration. Because the results of RVT analysis are based on the distribution function of expected peak values, the results are stochastic in nature, where the calculations predict the median value of the peak spectral acceleration at each frequency.

Ground motions input is approximated from the FAS developed from a target response spectrum. For the site at Mare Island the target response spectrum was developed from the results of the site specific seismic hazard analysis, specifically the geomean MCE (MCE_G); this spectrum is calculated in the same manner as the MCE_R, but is not adjusted to maximum demand, and is not risk-targeted. The FAS was fit to this spectrum using a ground motion duration of 10 seconds. Following guidance provided by Kamai et al. (2015), and consistent with the development of the site-specific response spectrum, the motion was input to the profile at the depth of the top of bedrock.

Variations in the ground motions due to variations in the site parameters can be large. In order to capture both the body and the range of the results, a randomization of the soil parameters can be performed. In particular, we note that Kottke and Rathje (2013) suggest that results of the RVT model yield more consistently equivalent results to more traditional time domain analyses when applied in this manner.

The potential variation in the material profile was accounted for using 25 randomly generated soil profiles. The profiles were randomly generated following the methodology of Toro (1995) to vary the both the shear wave velocity and the depth to bedrock. The shear wave velocity within each layer was varied between a minimum and maximum value about the mean, and the bedrock depth was allowed to vary between 95 feet and 240 feet below the ground surface. The bedrock depth range was selected based on the observed depth to the top of bedrock from north to south across the site as shown on Figure 8. The other layer boundaries were fixed at depths representing the middle of the range of the depth of each contact beneath the building footprint. The resulting randomized shear wave velocity profiles are shown on Figure 17.

Material curves were assigned to the soils based on previous experience with similar material in the Bay Area. The Fill was modeled using the relationships of Rollins et al. (1998) for gravels. This material curve was judged to be representative of a majority of the Fill at the site, which consists of gravels and gravelly clays. Young Bay Mud and Old Bay Clays were modeled using project specific material curves derived from previous testing in Young Bay Mud and Old Bay Clays from a number of sites in the Bay Area. The older alluvium was modeled using the relationship of Vucetic and Dobry (1991) for a PI = 50. The rock was modeled as a semi-infinite half space with a unit weight of 140 pcf and a damping of 5%. The various material properties, ranges, and nonlinear material curves are shown on Table 7 and Figure 18.

Using the above material properties, the geomean (median) amplification of the peak ground velocity (PGA) was determined from the results of the RVT site response analysis. The peak ground acceleration as a function of depth is presented on Figure 20. The site specific median PGA at the ground surface was determined to about 0.29 g $\pm\sigma_{ln}$=0.24. The de-amplification of the PGA (taken at a period = 0.01 seconds) at this site is consistent with the expected results at a soft soil site where high strains and large damping occur. The peak shear strains, peak ground displacements, and peak shear stresses by depth are presented on Figure 19. Although the peak shear strain is relatively high for this type of analysis (median value of more than 10% strain) the results are likely conservative for values of PGA at the ground surface. Analyses performed previously by Amec Foster Wheeler at a similar site have shown that PGA values from the equivalent linear site response were generally conservative when compared to non-linear analyses for the PGA.

6.2.2 Site Specific Design Peak Ground Acceleration

For geotechnical analyses the geomean of the peak ground acceleration or PGA_M is used. The ASCE/SEI 7-10 standard specifies that this value cannot be less than 80% of the value determined using the general procedures approach of Section 11 in ASCE/SEI 7-10. For this comparison, we obtain the mapped PGA at the ground surface from the USGS National Seismic Hazard Mapping Program (NSHMP) website (http://earthquake.usgs.gov/hazards/designmaps/) as described in Section 6.1.4, and considering the Site Class of E at the ground surface, consistent with the V_{S30} of the upper 100 feet of soil (Sections 4.4 and 5.2). The mapped PGA for Site Class B/C obtained using the online tool provided by the USGS is 0.52 g, and the site amplification factor (F_{PGA}) is 0.9 for Site Class E where the mapped PGA greater than 0.50 g. Thus, the general procedure PGA_M for the site is equal to the product of these two values (F_{PGA} x PGA_{map}) or 0.47g, and the minimum allowed PGA_M is 80% of this value, or 0.38 g.

A comparison of the results of the general procedure PGA_M and the site-specific PGA_M determined following the procedures in section 21 of ASCE 7-10 shows that the general procedure minimum value of PGA_M falls just outside the range of the site specific site response analysis (median value of 0.29 and 84th percentile value of 0.36 g) calculated from the site response results. Based this comparison, the general procedure PGA_M value governs for the site and was used in the geotechnical analysis completed in this investigation.

In addition to the design peak ground acceleration of 0.38 g, a representative magnitude for deterministic analysis of geotechnical hazards was also determined from the results of the site specific hazard analysis. After reviewing the results of the DSHA and PSHA (i.e. deaggregation of the 2% in 50 year uniform hazard results at the PGA) we selected a magnitude of **M** 7.0 to represent the hazard for deterministic analyses at the site. We note that this magnitude is only representative of the hazard and that both larger and smaller magnitude events may occur. The PGA and magnitude are used in the assessment of liquefaction and seismically induced settlement and consolidation described in Section 6.3 and Section 7.

6.3 GEOLOGIC HAZARDS

This section provides an evaluation of the site geologic conditions for slope stability, flooding, settlement, shrink-swell, and corrosion, and identifies geologic conditions that may require mitigation.

6.3.1 Slope Instability and Landsliding

The ground surface at the project site is relatively level across the majority of the site except in the vicinity of the Mare Island Channel along the east side of the site and northwest of Walnut Ave at the north side of the site. The area within about 300 to 500 feet of the proposed development slopes gently from about 10 to 15 feet down to the strait at sea level. The bottom depth of the channel is at about -17 to -27 feet within about 100 to 300 feet from the shoreline according nautical charts by the National Oceanic and Atmospheric Administration (NOAA, 2015). The area northwest of Walnut Ave slopes down to the northwest from about 10 to 12 feet to 3 to 6 feet at the property limit.

The stability of the Mare Island Channel under seismic and static loading was not evaluated because project developments are restricted to a 200 foot minimum setback from the San Francisco Bay Conservation and Development Commission (SF-BCDC) tidal marsh boundary (Figure 16, SF-BCDC, 2008), and any deformation along the channel margin is not expected to extend close to the area of planned developments. However, we note that the relatively low strengths of the Young Bay Mud in the top 40 to 50 feet of the site may lead to some instability immediately along the channel in areas not improved previously by the Navy, especially during seismic loading. If permanent or heavy temporary facilities are installed near (within 400 feet) of the channel, these facilities should be evaluated for stability under both static and seismic loading.

Stability analysis were not performed for the gently sloping area northwest of Walnut Avenue. It is our understanding that this entire area will be re-graded as a part of the project. We expect that the majority of the surficial soils in this area will be removed or improved, and the remaining area filled or otherwise stabilized during the development.

We preformed limited static short term stability analyses at the site for generalized semi-infinite embankments for a range of embankment (fill) heights. The stability analyses were performed using Spencer’s method. The results of the analysis show that any embankments over 15 feet of height are likely to be unstable under expected potential preloading embankment loads. As such the practical upper bound for preloading at the site is a 15 foot high embankment.

6.3.2 Flooding and Sea Level Rise

Flood risks are based on local hydrology, topology, precipitation, flood protection measures such as levees, and other scientific data. Concentrated flows of water from flooding may cause acute erosion of stream banks, slopes, and swales, possibly resulting in land sliding. Acute erosion around shallow building foundations may undermine the foundations and damage buildings or other structures.

The site will be located on relatively level ground, and there are no lakes, reservoirs, water storage tanks, or water retention facilities upslope and near to the site that could fail and result in flooding at the site.

The main portion of the site lies at an elevation of about 8 to 15 feet above sea level, between San Pablo Bay and the Mare Island Strait. Review of Flood Insurance Rate Maps prepared by the Federal Emergency Management Agency (FEMA) shows that the base flood elevation level (for the 100 year flood, equivalent to a 1% P_E in one year) for the project area on Mare Island is 10 feet. This mapping shows that the northwest part of the site is at risk from flooding for the current site elevation (Figure 16).

Review of information and maps assessing potential for sea level rise in San Francisco Bay indicate that sea level may rise by 39 to 55 inches by the year 2100 (Heberger et al., 2012). Although this would not inundate the site, except possibly at the northwest corner, this would increase the portion of the site that is exposed to a significant flood hazard to include areas up to about 15 feet elevation. We note, however, that while there is significant uncertainty in the expected sea level rise over the next 100 years or so, the estimate of 39 to 55 inches is based on scenarios in which the rate of global warming is reduced from the present rate. If the rate of global warming is not reduced, the sea level rise could be much higher.

Based on the current FEMA flood inundation mapping, and the potential sea level rise, we judge that there is a moderate flooding hazard for facilities at the site. Although it is possible to mitigate the flood hazard for the proposed buildings by raising the base elevation of the structures, access to and operations of the building could be significantly affected by inundation of the surrounding portions of the island.

6.3.3 Settlement and Consolidation

Subsidence occurs from the consolidation of fine-grained compressible beds and interbeds due to fill placement or to a reduction of pore-pressure caused by lowering of the groundwater.

The site is underlain by Fill and thick Young Bay Mud deposits, and consolidation is expected to occur in areas where loads from new structures or fill will be placed as part of the proposed site development. The location and amount of consolidation that may occur due to planned development is discussed in Section 7.1.

6.3.4 Shrink Swell Potential

As described in Section 5, the Fill is mostly composed of clayey gravel (GC) or clayey gravel with sand with significant variation from location to location. Expansion testing of this material gave results of 59 and 23 for borings B-3 and B-4 respectively. Atterberg Limits tests on soils within the Fill had Plasticity Indexes (PI) ranging between about 7 and 30. Based on these test results, the Fill materials may be prone to moderate volume changes (shrinkage and swelling) with seasonal or other fluctuations in soil moisture, especially portions of the Fill composed of fat clays.

In addition, in some areas of the site Young Bay Mud is relatively shallow (less than 5 feet below the ground surface). Young Bay Mud typically has higher PI, with values commonly greater than 40. Based on the PI of the Young Bay Mud and our experience with it in other areas, there is a large potential for volume changes if subject to wetting and drying, which could cause distress to overlying foundations. However, because groundwater appears to be very shallow, it may be unlikely that these materials will be subject to much drying.

Soils susceptible to moderate or severe shrink/swell behavior may cause damage to building slabs that are supported by these soils. Recommendations to mitigate the shrink/swell potential of the near-surface soils for slab support are presented in Section 7.4.2.

6.3.5 Corrosion of Buried Concrete and Metals

Laboratory tests were performed on four representative samples of near surface soils to evaluate pH, sulfate and chloride contents, resistivity characteristics. The results of laboratory tests are presented in Appendix B and summarized below.

Chemical Analysis	Results	Corrosion Classification
Chlorides	9 to 41 mg/kg	Non-corrosive with respect to bare steel or ductile iron
Sulfates	57 to 298 mg/kg	Mildly corrosive to non-corrosive with respect to mortar coated steel
pH	8.1 to 8.5	Non-corrosive with respect to bare steel or ductile iron
Minimum Resistivity	893 to 2822	Corrosive to moderately corrosive with respect to bare steel or ductile iron

Recommendations for protection of buried pipelines are discussed in Section 7.5.

6.4 SEISMIC GEOLOGIC HAZARDS

This section provides an assessment of the earthquake-related geologic/geotechnical hazards for the site, including the potential for surface fault rupture; liquefaction; seismically-induced settlements; seismically-induced land sliding; and inundation due to tsunami, seiche, or seismically-induced failure of water-retention facilities.

6.4.1 Surface Fault Rupture

Earthquakes generally are caused by a sudden slip or displacement along a zone of weakness, termed a fault, in the Earth's crust. Surface fault rupture, which is a manifestation of the fault displacement at the ground surface, usually is associated with moderate- to large-magnitude earthquakes (magnitudes of about 6 or larger) occurring on active faults having mapped traces or zones at the ground surface. The amount of surface fault displacement can be as much as 10 feet (3 meters) or more, depending on the earthquake magnitude and other factors.

Review of published maps showing Quaternary faults in the vicinity indicates that buried traces of the Franklin fault extend generally northwest-southwest along Mare Island (Jennings and Bryant, 2010; Graymer et al., 2006). These maps show that the Franklin fault has two traces, one passing east of the site along the Mare Island Strait, and the other passing along the west side of the bedrock hill at the southern end of Mare Island, and extending northward across the project site to join the eastern trace of the fault north of Mare Island (Figure 4). Although the fault has been identified as having Quaternary displacement, the age the youngest Quaternary activity on the fault had not been constrained (Jennings and Bryant, 2010; Holzer et al., 2002), and because there was no definitive evidence for Holocene slip on the Franklin fault, the California Geological Survey did not established an Earthquake Fault Zone along any part of the Franklin fault on the State of California Official Maps (Hart and Bryant, 2007; http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps).

The most comprehensive studies to assess the activity of the Franklin fault in the area of Mare Island (Kelson et al, 2004, 2005; Brossy et al, 2010) showed that marine terraces around Mare Island and along the Carquinez Strait are not offset vertically across the Franklin fault, but these marine terraces are offset vertically where they extend across the projection of the Southhampton fault, across the Carquinez Strait to the east of Vallejo (Figure 4). This assessment supports an interpretation that late Pleistocene or Holocene vertical offset has occurred on the Southhampton fault. The potential slip type for rupture of the Franklin fault is uncertain. The past movement has been predominantly reverse slip, but the expected slip on the Southhampton and other small faults included in the active Contra Costa Shear Zone is strike-slip (Kelson et al., 2004, 2005; Brossey et al., 2010; Field et al., 2013). Although the absence of vertical displacement on marine terraces crossing the Franklin fault does not preclude the possibility of late Pleistocene or Holocene strike slip movement, Kelson et al. (2004, 2005) note that the shore-line angles of the marine terraces do not appear to be offset, and they suggest that the modeled right lateral shear on traces of the Contra Costa Shear Zone cross-cuts the “inactive Franklin fault”. In summary, no specific evidence for late Pleistocene or Holocene offset has been identified for the mapped traces of the Franklin fault in the vicinity of the Carquinez Strait and Mare Island.

The nearest known active fault is the northwest-southeast trending West Napa fault, which is lies at a closest distance of about 3.8 miles (6.2 km) north of the site (Figure 5). Based on this information, the potential for surface fault rupture at the site is judged to be low.

6.4.2 Liquefaction and Lateral Spreading

Liquefaction is a soil behavior phenomenon in which a soil loses a substantial amount of strength due to high excess pore-water pressure generated by strong earthquake ground shaking. Soil liquefaction can lead to a variety of undesirable effects including settlement, lateral spreading of the ground, downdrag on piles, and loss of strength. Typically, recently-deposited (i.e., Holocene, within about the past 11,000 years) and relatively unconsolidated soils and artificial fills located below the groundwater surface are considered susceptible to liquefaction (Youd and Perkins, 1978). The soils that are most susceptible to liquefaction include relatively clean, loose, uniformly graded sand, silty sand, and non-plastic or low plastic fines.

We analyzed liquefaction susceptibility and triggering at the site based on the methodology of Boulanger and Idriss (2014) and using the data obtained from CPT analyses at the site. For CPT's, susceptibility is typically evaluated using a soil behavior type index (Ic) cutoff. This cutoff can vary from site to site but is typically taken as 2.6 for most sites (Boulanger and Idriss 2014). A magnitude (M_W 7.0) and acceleration of 0.38 g and were used as discussed in Section 6.2. For the purposes of this analysis, the occurrence of liquefaction was limited to depths shallower than 60 feet. A cutoff of 60 feet was used as this is the approximate depth below which liquefaction effects (e.g. settlement and lateral spread) typically have little impact on structures at the ground surface (Cetin et al. 2009).

Using the procedure outlined above, we analyzed a number of CPT's at the site and determined that for the majority of the site, the soils are not susceptible to liquefaction. There are a few limited zones of liquefiable soils, specifically at some locations in the Fill at depths between about 5 and 12 feet, and in a deeper layer which approximately follows the old shoreline at a depth of about 25 to 35 feet.

Because of the limited nature and lateral extent of the liquefiable soils at the site both vertically and laterally, lateral spreading is unlikely to occur at the site. The site is relatively flat, and there are no free faces which could fail, except for the area along Mare Island Strait directly east of the project site. Based on the data from the CPT's, and because of the relatively flat topography, we judge that lateral spreading is unlikely to occur at the Mare Island site and lateral spreading was not further evaluated.

Strength loss due to liquefaction was likewise determined to pose little risk to the site, due to the likely limited extent of the liquefied zones, and thus was not evaluated.

6.4.3 Seismically Induced Compression and Settlement

Seismic compression is a soil behavior phenomenon in which contractive volumetric strains accumulate in unsaturated soils during strong earthquake ground shaking. Typically, the soils that are most susceptible to seismic compression include unsaturated and relatively clean loose sand, silty sand, and non-plastic silt deposits. Compacted fills whose voids are not fully filled with water are also susceptible to seismic compression due to densification of the soil particles resulting from earthquake-induced shear deformations.

Liquefaction induced settlement is also possible for the Project Zeus Mare Island site. Liquefaction induced settlement occurs when liquefaction triggers causing a void redistribution and densification of the soil. Soil ejecta can also increase the risks form this phenomena as the soil exits the ground and piles on the ground surface.

Values of seismically induced compression and settlement were calculated using the program CLiq version 1.7.6.49 created by Peter Robertson. For the purposes of this analysis, the groundwater table depth was fixed at a depth of 5 feet below the ground surface. Since the upper five feet were hand augered no data was available for these depths and seismic compression was thus not evaluated for the majority of the site.

Mean values of seismically induced settlement and compression were generally small (less than 0.5 inches) for a majority of the locations at the site. However, as noted previously there were smaller zones where the liquefaction induced settlement was greater with values in these areas generally ranging from about 0.5 to 2 inches (see Figure 21). We note that a single exploration (CPT-12) indicates a large estimated liquefaction settlement of over 6 inches. This higher value is generally not supported by other, nearby explorations.

Note that the values of liquefaction settlement are computed using empirically derived equations are generally representative of the mean response of soils. Reporting the results to 0.01 inch is misleading as the uncertainty associated with these methods can be as high as a factor of two (Cetin et al., 2009). Thus for example, a mean value of 0.5 inches might produce liquefaction induced settlements of as low as 0.25 inches or as high as 1 inch.

Based on the results of our analysis we expect that mean seismically induced total settlements under the manufacturing facility structure(s) will vary from 0 to 2 inches, with the majority of the larger settlements following near the historical shoreline along the west edge of the proposed building(s). Mean seismically induced total settlements under the office structure on the east of the site are likely to vary between 0 and 0.5 inches. Differential settlements are difficult to predict due to variations in topography, and other factors but we expect that mean differential settlements from liquefaction will not be less than half of the total settlements from liquefaction, thus under the manufacturing facilities, differential settlements on the order of 0 to 1 inch are possible. Because of the large liquefaction settlement value estimated at CPT-12, differential settlement from liquefaction along the northern building edge could be 3 inches or more. Under the Administration building, differential settlements from liquefaction on the order of 0 to 0.5 inches are possible.

Potential mitigations for liquefaction are discussed in Section 7.1.

Dashti et al. (2010) noted that estimates of settlement using free field approaches (such as the approach used in this study) do not adequately account for the effects of buildings on total settlement. They noted that actual building settlements due to liquefaction observed during centrifuge testing "…quickly surpass[ed] free-field ground settlements...." The differences between free field predictions and actual observed settlements were attributed to several mechanisms tied to soil structure interaction. The effect of soil structure interaction is not accounted for in the current analysis.

6.4.4 Seismically Induced Transient and Permanent Displacements

Seismically induced transient and permanent displacements may occur at the site due to ground shaking. The peak transient horizontal displacement of the ground due to an earthquake can be estimated by integrating the maximum shear strains obtained from the site response analyses with respect to depth such that the total transient displacement is the cumulative displacement from bedrock to the depth of interest. This conservatively assumes that the maximum shear strains occur at the same instant in time and provides a maximum upper bound on the likely displacements at the site.

Transient displacements were evaluated using the results of the site response analyses at the site. The geomean of the site response results were used in the analysis of transient displacements. Note that the current evaluation of transient displacements does not account for soil structure interaction effects.

For the Project Zeus site at Mare Island, the seismically induced transient displacements are likely to be quite large, with as much as 2 or more feet of horizontal movement likely to occur at the ground surface. Some amount of un-estimated vertical translation is also likely. Differential transient displacements (such as between the ground and the foundations) are likely to be small for most cases where foundations are at shallow depths. Structures founded at deeper depths (such as pile supported sections of the building) will likely experience much larger differential displacements with respect to the ground surface. Evaluations of soil structure interaction or estimations of seismically induced displacements and stresses are recommended for these and other critical cases.

6.4.5 Seismically Induced Landsliding

Earthquake ground shaking can reduce the stability of a slope and cause sliding or falling of the soil or rock materials composing the slope. During ground shaking, seismic inertia forces are induced within the slope, increasing the loads that the slope materials must sustain to resist landsliding (or rockfalls). If the forces tending to cause landsliding exceed the strength of the materials resisting landsliding, a temporary instability is created that is manifested by lateral or downslope displacement of the slope materials. In some cases, strong ground shaking can also reduce the strength of the soil or rock materials, reducing their ability to resist the forces that cause landsliding.

The site is located on relatively level ground. There are no slopes in the vicinity of the site that could fail and then progressively expand to eventually impact the proposed buildings. There also are no slopes in the site vicinity that could fail in a "flowing" manner and impact any structures on the site. Based on this information, and excepting the potential for ground deformation at the eastern margin of the site due to slope instability and lateral spreading along the Mare Island Strait described in Sections 6.3.1 and 6.4.2, the potential for seismically induced landsliding at the site is judged to be negligible.

6.4.6 Seismically Induced Inundation

Seismically induced inundation can be caused by a variety of phenomena, including tsunami waves, seiche waves, or flooding resulting from seismically induced failure of water-retention facilities. Tsunamis are either ocean waves generated by vertical seafloor displacements associated with large offshore earthquakes or waves in any body of water that result from rapid landsliding into the water or rapid landsliding of slopes covered by the body of water. Seiches are waves associated with the oscillating surface of an enclosed or partly enclosed body of water caused by interaction of the water body with arriving seismic waves. Flooding and acute erosion may result from seismically induced failure of levees or water-retention facilities such as dams, reservoirs, or tanks upstream or upslope of a site.

The site is located at an elevation of approximately 8 to 15 feet above mean sea level and abuts the Mare Island Straight on the east side. San Pablo Bay lies about 5,000 feet west of the site, across low-lying brownfields at elevations of about 5 to 10 feet above mean sea level. Tsunami inundation maps have been prepared for San Francisco Bay, San Pablo Bay, and the Mare Island Strait by the California Emergency Management Agency, California Geological Survey, and University of Southern California (2009). The mapping by these groups shows that expected maximum inundation area does not extend into the project area (Figure 16). We note, however, that this inundation area does not account for potential sea level rise as described in Section 6.3. Based on this information, we judge that the potential for tsunami inundation at the project site is low.

As noted in Section 6.3, there are no dams that could fail and result in flooding at the site. Therefore, the potential for inundation or acute erosion from seismically induced failure of water-retention facilities is judged to be negligible.

7.0 GEOTECHNICAL RECOMMENDATIONS

The Mare Island site is suitable for development, but has some challenging geotechnical conditions. The artificial fill (uppermost 10 feet) is of variable consistency, generally loose, and may experience vertical deformations up to 2 inches (potentially 6 inches in the vicinity of CPT-12) as a result of strong earthquake shaking at the site. A large earthquake anticipated within the lifetime of the proposed facility will likely cause settlement, which may impact the integrity and performance of overlying structures and improvements, including utilities, unless addressed during design.

The Young Bay Mud immediately below the artificial fill is highly compressible. New loads (from proposed building foundations, building slabs, or from new fill) will induce settlement at the ground surface. The magnitude of the settlement will depend on several factors (discussed below), and may be significant.

During our investigation, groundwater was encountered at relatively shallow depths, approximately 2 to 5 feet below current ground surface elevations.

Geotechnical recommendations are presented in the following sections.

7.1 SETTLEMENT FROM LIQUEFACTION

As discussed in Section 6.4.3, total vertical settlements on the order of 2 inches, and differential vertical settlements on the order of 1 inch are possible during and immediately after strong earthquake shaking at the site.

Where the estimated vertical settlement from liquefaction values exceed structural tolerances, structures and/or floor slab areas could be founded on deep elements that derive support from stiffer soils below the Fill and Young Bay Mud. Driven precast concrete piles are an example of a deep foundation element system that have been used locally to support structures on sites with similar subsurface conditions. It is our understanding that many of the existing Navy structures (buildings, and slabs) on the site, including some utilities, are supported on driven timber piles and/or driven precast concrete piles. The estimated cost for driven pile foundations are discussed in Section 7.3 below.

Alternatively, the loose, potentially liquefiable Fill soils at the site can be mitigated through ground improvement. The purpose of the ground improvement would be to densify the Fill to reduce the potential for liquefaction under anticipated earthquake loads. The following ground improvement methods are generally appropriate to mitigate liquefaction at this site:

- Soil-cement mixing;
- Stone columns; and
- Deep Dynamic Compaction.

Some general information about these improvement methods are discussed below.

Soil-cement mixing can be an effective way to reduce liquefaction settlement, but is generally very expensive. Soil-cement mixing involves using a large (crane mounted) piece of equipment to physically blend in-situ Fill and Young Bay Mud soils with cement grout. The product of the improvement would be a stiff (much denser, less compressible) soil-cement with a compressive strength on the order of 100 psi (14,400 psf). The percentage of mixing/replacement required to mitigate liquefaction would need to be assessed through design but probably would be on the order of 50% under building columns, but less under floor slabs. Soil-cement mixing can also be an effective way to reduce settlement from soft compressible Young Bay Mud at the site. The general cost of soil-cement mixing is discussed in Section 7.2.4.3.

There are a few methods for stone column type ground improvements. Generally a truck or crane mounted rig is used to ram or vibrate a column of gravel-size stones into a loose, or soft compressible, layer. After installation, the stones in each column are densely packed, the drainage of any compressible layer is improved, and the loose, soft soils surrounding each column have been densified, compressed and strengthened. The product of the improvement at the Project Zeus site would be a denser (not liquefiable) composite stone - fill layer. The columns can be detailed to be individual load supporting elements. Stone columns may also help to potentially reduce settlement from soft compressible Young Bay Mud at the site. The general cost of one variation of stone column improvement is discussed in Section 7.2.4.4.

Deep dynamic compaction is generally a simple and cost effective way to densify loose granular soils near the surface. The method involves using a crane to drop a heavy weight repeatedly at the surface to densify the ground. The frequency and spacing of drops is optimized during design. This type of improvement is relatively inexpensive when compared to stone columns or soil-cement mixing. Unfortunately however shallow groundwater in the Fill and soft Young Bay Mud at the bottom of the Fill will reduce the effectiveness of this improvement method, which will require some additional consideration. Deep dynamic compaction does not help to reduce settlement from consolidation of soft compressible Young Bay Mud at the site.

Driven piles, soil-cement mixing, and stone columns are ground improvement methods that are also effective for mitigating settlement from compressibility of Young Bay Mud, which is discussed in Section 7.2.

7.2 SETTLEMENT FROM COMPRESSIBLE YOUNG BAY MUD

Below the Fill, the Project Zeus site subsurface includes a 10 to 55 feet-thick deposit of very soft, compressible Young Bay Mud soil.

Settlement of proposed buildings and floor slabs will be a major consideration when planning development of the Project Zeus site. Wherever possible, designers should minimize the magnitude of new loads. Anything more than minor grading (plus or minus a few inches) will likely induce settlement at the ground surface. The degree of settlement anticipated will depend on several factors, including actual loads applied, and the thickness and specific characteristics of the Young Bay Mud immediately under the applied new loads. Settlements on the order of 12 to 18 inches are possible under an areal load of 500 psf applied near the ground surface and on the order of about 5 or more feet are possible under an areal load of 3,500 psf. Because of the variable nature of the Young Bay Mud, there is potential for differential settlement over relatively short distances. It is our understanding that many, potentially all, of the existing Navy structures, including floor slabs, at the site were/are supported on driven timber pile and/or driven precast concrete pile foundations, because of the potential for large settlements. Observations of existing buildings at the project site suggest that in some locations the ground surface has experienced settlement on the order of several inches to a few feet under the weight of the existing site fill, which generally ranges from 5 to 10 feet-thick. Photos from Amec Foster Wheeler's site reconnaissance (included in Appendix C) illustrate the magnitude of settlement, and in some cases corresponding distress, currently visible at discrete locations at the site.

The estimated thickness of the YBM is shown on the idealized profile in Figures 8 and 9. As described in Section 4, the thickness of Young Bay Mud at the site varies from about 1 foot in the southern corner (near the intersection of G Street and Azuar Drive) to about 53 feet along the north western edge (in the vicinity of Highway 37). Within the approximate boundaries for the proposed new structures, the Young Bay Mud is generally 30 feet thick, however there is an area, up to about 50 feet-thick near the center of the main manufacturing structure(s), adjacent to Amec Foster Wheeler recent explorations B-5 and CPT-8.

7.2.1 Loads

Based on the current layout, the manufacturing facility includes about 2 million square feet of new building area. All of the manufacturing buildings will be single story except for the paint building, which will be 100 feet high. The heaviest anticipated long-term column loads for the manufacturing buildings may be up to 500 kips in the paint building, wall loads may be up to 6 kips per lineal foot. Floor loads in some material storage areas may be 3.5 kips per square foot, but most floor loads will be lower. There will also be an Administration Building, which could be several stories high. Immediately surrounding the facility will be paved yard areas, for deliveries, staging, and storage.

7.2.2 Consolidation Parameters

Consolidation parameters for the Young Bay Mud at the project site have been investigated and evaluated in multiple previous reports. In October 2015, Amec Foster Wheeler collected additional undisturbed samples of Young Bay Mud within the footprint of the proposed manufacturing buildings and performed additional laboratory consolidation testing. Detailed laboratory consolidation test results are included in Appendix B. Field and laboratory test data indicate that the upper 15 feet of Young Bay Mud may be slightly over-consolidated (slightly less compressible) and that the lower zones of Young Bay Mud are normally consolidated.

7.2.3 Consolidation Settlement Analysis

Settlement analyses were performed using the software SETTLE3D (Rocscience, 2014), a 3-dimensional program for the analysis of vertical consolidation and settlement under foundations, embankments and surface loads.

7.2.3.1 Anticipated Settlement without Mitigation

A generalized profile was developed based on the contours of Fill and Young Bay Mud at the site. The ground surface was modeled at a constant elevation of +10 feet (NAVD 88). Soil layer thicknesses were then modeled based on the contours developed in Section 4 and presented on Figures 10 and 11. The resulting unit thicknesses vary with location across the site. A profile consisting of 10 feet fill over about 50 feet of Young Bay Mud over stiffer soils was evaluated for various applied foundation loads. Table 8 shows the approximate material properties and depth ranges for each of the layers and sublayers in the consolidation analysis. The groundwater level was assumed to be at elevation +5 feet, 5 feet below the elevation of the current ground surface.

A range of applied foundation loads were selected to capture results corresponding to lightly loaded and heavy loaded structures.

Results of the consolidation settlement analysis presented in the table below are based on the following additional assumptions:

- Site grades remain at existing site ground surface elevations (i.e. only minor site grading, +/- a few inches was assumed).
- Consolidation parameters for Young Bay Mud used in the analyses were best estimates from laboratory consolidation tests (both from this study and from previous studies) performed on samples of Young Bay Mud recovered from the site.
- The proposed building foundations and/or loaded floor slab areas are large enough that pressure reductions at depth for footing/loaded-area size do not apply.

Applied Foundation (Building or Slab) Load (psf)	Estimated Building Settlement at 30 years after Construction (inches)	Estimated Ultimate Building Settlement at approximately 100 years after Construction (inches)
0	0 to 1*	3 to 5*
200	2 to 5	4 to 7
1,000	16 to 21	31 to 36
1,800	27 to 33	50 to 56
3,500	49 to 54	87 to 93

* Settlement analyses indicate that the Young Bay Mud at the site may still be compressing under the weight of the existing fill placed during original development (and subsequent maintenance) of the site.

A range of estimated settlement values are presented above for each of the applied foundation loads to reflect the potential variability of the Young Bay Mud materials and the uncertainty associated with predicting future settlement. The settlement values above correspond to an area within the proposed manufacturing building(s) where the Young Bay Mud was observed to be up to about 50 feet thick (adjacent to Amec Foster Wheeler explorations CPT-8 and boring B-5). For the majority of the proposed building areas, Young Bay Mud was observed to be thinner than 50 feet (typically about 30 feet). In these areas, the anticipated settlement would be on the order of 10% to 20% less at 30 years than the values shown above, and on the order of 20% to 30% less at 100 years than the values shown above. For areas where the thickness of the Young Bay Mud is 30 feet-thick, the ultimate consolidation will occur quicker, say over about 60 years, rather than 100 years.

7.2.4 Mitigation for Settlement

Where the applied foundation (building or slab) loads and corresponding settlement values exceed structural tolerances, structures and/or floor slab areas could be founded on deep elements that derive support from stiffer soils below the Young Bay Mud. Driven precast concrete piles are an example of a deep foundation element system that has been used locally to support structures on sites with similar subsurface conditions. It is our understanding that the existing Navy structures (buildings, and slabs) on the site, including some utilities, are supported on driven timber piles and/or driven precast concrete piles. The estimated cost for driven pile foundations are discussed in Section 7.3 below.

Alternatively, the compressible Young Bay Mud soils at the site can be mitigated through ground improvement. The purpose of the ground improvement would be to make the Young Bay Mud layer stiffer, and less compressible, to reduce the potential for settlement under applied loads to within tolerable limits. The following ground improvement methods are generally appropriate for the subsurface conditions at this site:

- Preloading the site with surcharge fill;
- Reducing new loads, by using lightweight fill materials;
- Soil-cement mixing; and
- Stone columns.

Some general information about these improvement methods are discussed below.

Preloading the site with surcharge fill is generally considered a cost effective technique, particularly when wick drains, about 3 to 7 feet on center, are inserted through the Young Bay Mud to increase the rate of consolidation and settlement. In other developments on Mare Island (and around San Francisco Bay) the compressibility of Young Bay Mud was mitigated by preloading the site with a surcharge fill. The general feasibility and cost of preloading is discussed in Section 7.2.4.1. The potential benefits of preloading can be combined with the potential benefits of using light weight fill, discussed below. Preloading will not effectively address the liquefaction settlement described in Section 7.1.

Light weight fill materials (lightweight soil, or lightweight concrete) are becoming increasingly popular and could help to reduce the magnitude of new loads and thereby reduce consolidation and settlement of the Young Bay Mud. For example, if the proposed manufacturing facility requires a 3-foot-high loading dock, the weight of the soil for the dock-high fill could be reduced from about 360 psf to about 180 psf by using lightweight soil. It would also be possible to further reduce areal loads by about 180 psf by replacing the upper 3 feet of the site subgrade with lightweight fill soil. Mass site excavation deeper than 3 feet, should generally be avoided as the groundwater at the site is shallow. The general cost of using light weight soil is discussed in Section 7.2.4.2. The potential benefits of using light weight soil can be combined with the potential benefits of preloading, discussed above. Lightweight fill will not effectively address the liquefaction settlement described in Section 7.1.

Soil-cement mixing can be an effective way to reduce settlement in both loose Fill (liquefaction) and compressible Young Bay Mud, but is generally very expensive. Soil-cement mixing involves using a large (crane mounted) piece of equipment to physically blend in-situ Fill and Young Bay Mud soils with cement grout. The product of the improvement would be a stiff (much less compressible) soil-cement with a compressive strength on the order of 100 psi (14,400 psf). The percentage of mixing/replacement required would need to be assessed through design but probably would be on the order of 50% under building columns, but less under floor slabs. The general cost of soil-cement mixing is discussed in Section 7.2.4.3. There is no increased benefit of combining soil-cement mixing with preloading or using light weight materials.

There are also methods for stone column type ground improvements that can reduce settlement in loose Fill (liquefaction) and possibly also in compressible Young Bay Mud. Generally a truck or crane mounted rig is used to ram or vibrate a column of gravel-size stones into a loose, or soft compressible, layer. After installation, the stones in each column are densely packed, the drainage of the compressible layer is improved, and the soft soils surrounding each column have been compressed and strengthened. The product of the improvement at the Project Zeus site would be a stiff (less compressible) composite stone - clay layer. The columns can be detailed to be individual load supporting elements. The general cost of one variation of stone column improvement is discussed in Section 7.2.4.4. The potential benefits of stone column improvement can be combined with the potential benefits of preloading, discussed above.

Driven piles, cement soil mixing, and stone columns are ground improvement methods that are also effective for mitigating liquefaction as discussed in Section 7.1 above. Preloading and lightweight fill are not effective for mitigating liquefaction.

7.2.4.1 Preloading

To demonstrate the level of improvement that can be expected through preloading, Amec Foster Wheeler performed additional settlement analyses for the same generalized profile (10 feet fill over 50 feet of Young Bay Mud over stiffer soils), for various applied foundation loads, and for various surcharge fill conditions. Surcharges fill heights of 5, 10, and 15 feet, and surcharge fill durations of 6, and 12 months, were considered. In the evaluation, surcharge fills were applied over the course of one month and removed over the course of one month. For all surcharge cases, wick drains, installed through the full thickness of the Young Bay Mud at 5 feet on-center, were incorporated into the analysis.

Results of the additional consolidation settlement analysis are presented in the table below.

Duration	Applied Foundation (Building or Slab) Load (psf)	Estimated Building Settlement at 30 years after	
		Surcharge Duration 6 months	Surcharge Duration 12 months
Settlement from Surcharge Fill Height = 5 ft ~ 600 psf (inches)			
10 (6 months) to 14 (12 months)	200	1 to 4	1 to 3
Settlement from Surcharge Fill Height = 10 ft ~ 1,200 psf (inches)			
22 (6 months) to 25 (12 months)	200	1 to 2	not eval.
	1,000	9 to 18	9 to 18
Settlement from Surcharge Fill Height = 15 ft ~ 1,800 psf (inches)			
31 (6 months) to 36 (12 months)	200	1 to 2	not eval.
	1,000	3 to 12	not eval.
	1,800	18 to 24	18 to 24

The results presented in the table above generally indicate that:

- Preloading the site with a 5 feet-high surcharge fill (~600 psf) may be effective for mitigating settlement of lightly loaded, say less than 300 psf, structures and floor slabs depending on tolerances.
- Preloading the site with a 15 feet-high surcharge fill (~1,800 psf) may be effective for mitigating settlement of moderately to heavily loaded, say less than 1,200 psf, structures and floor slabs, assuming 3 to 12 inches of total settlement can be tolerated.
- Preloading the site with surcharge fill less than 15 feet high (~1,800 psf) is probably not effective for mitigating settlement of heavily to very heavily loaded, greater than 1,200 psf, structures and floor slabs.
- There is a relatively small benefit to keeping surcharge fill in place for durations exceeding 6 months when wick drains are used, because of their dramatic ability to reduce times for consolidation and settlement of the Young Bay Mud.

To maximize the effectiveness of preloading, the surcharge load should be greater than the proposed building or slab load(s). To develop an effective preloading program for Project Zeus, Amec Foster Wheeler will need to work with the designers to identify the following:

1. What are the applied loads?
2. What amount of settlement is tolerable?
3. What is the life of the project?

4. For a given load and wick spacing, what is the calculated settlement that would occur, with no surcharging, over the life of the project (i.e. 30 years)?
5. What is the surcharge that will produce the settlement calculated in step 4 above, less the tolerable settlement from step 2 above, within the time available for surcharging?

The settlement values above are generally maximum settlement corresponding to an area within the proposed building where the thickness of Young Bay Mud was observed to be up to about 50 feet thick (adjacent to Amec Foster Wheeler explorations CPT-8 and boring B-5). For the majority of the proposed building areas, Young Bay Mud was observed to be thinner than 50 feet (typically about 30 feet). In these areas, the anticipated settlement would be on the order of 20 to 30% less at 30 years than the values shown above.

Closer (less than 5 feet) spacing of wick drains may help to further reduce future building settlements.

The degree of settlement under the weight of the various surcharge loads are indicated in the table above. Any existing utilities or structures to remain within, or immediately adjacent to, the surcharge area(s) will be potentially be impacted.

Engeo prepared a Preliminary Geotechnical Cost Summary in 2006 for a different development scheme at this site. In that report, Engeo estimated a cost of $9 per square foot of development for 10 foot-high surcharge fill (duration 6 months), with wick drains spaced at 7 feet on-center. Using this number, and assuming about 2,000,000 square foot building footprint, the estimated cost of surcharging for Project Zeus would be on the order $18M if the entire footprint required surcharging. Amec Foster Wheeler’s settlement analysis, discussed above, assumed wick drains at 5 feet on-center, which could add another $1M to $2M to the cost of this mitigation. A ten feet-high surcharge over a 2,000,000 square foot footprint will require about 750,000 cubic yards of soil for preloading, though this volume could be reduced by a staged preloading sequence and reuse of surcharge fill from one building area to another. After preloading is complete, the surcharge fill would need to be either removed from the site or placed in an area where corresponding settlement under the weight of the Fill would not impact existing or proposed improvements, including underground utilities.

7.2.4.2 Lightweight Fill

As discussed in Section 6.3.1, lightweight fill can be used on its own or in combination with a preloading program to mitigate consolidation settlement by potentially reducing the magnitude of new loads.

From Engeo’s Preliminary Geotechnical Cost Summary (2006), the estimated cost of excavating 5 feet of the site subgrade and replacing it with lightweight fill (for an applied load reduction of about 300 psf) is $20 per square foot of development. Using this number, and assuming about 2,000,000 square foot building footprint, the estimated cost of using lightweight fill for Project

Zeus would be on the order $40M, would require about 370,000 cubic yards of imported lightweight fill, if the entire footprint required treatment. This mitigation would generate the same volume of existing fill that would need to be removed from the site or placed in an area where corresponding settlement under the weight of the Fill would not impact existing or proposed improvements, including underground utilities.

Because of the shallow groundwater encountered at the site, it may be smart to limit the depth of excavation to 3 feet for constructability. In this case, the applied load reduction would be 180 psf. Assuming the cost is $15 per square foot of development, and a 2,000,000 square foot building footprint, the estimated cost of using lightweight fill for Project Zeus for a 3 feet deep replacement could be on the order $30M and require 220,000 cubic yards of imported lightweight fill.

The duration of the excavation and fill activities will be highly dependent on volume as well as the Contractor's resources and the location and availability of lightweight fill. A duration on the order of 3 to 6 months seems realistic.

Generally proposed floor elevations should be as close to the elevation of the current ground surface as possible to limit potential for future settlement. If a dock-high fill is required for any part of the manufacturing facility, the use of light weight fill materials may be an effective means for reducing corresponding degree of settlement.

7.2.4.3 Soil-cement mixing

Soil-cement mixing is an effective alternative for mitigating settlement from liquefaction (discussed in Section 7.1) and settlement from compressibility of soft Young Bay Mud (discussed in Section 7.2). Amec Foster Wheeler has been involved in several soil-cement mixing projects at sites with similar soil conditions as those encountered at the Project Zeus site. Several contractors now have the equipment and experience to execute this work. Details of a soil-cement mixing mitigation will need to be evaluated during design. For the purposes of this discussion, we can assume approximately 40% replacement (say 50% under building columns, less under slabs). Assuming a 2,000,000 square foot building footprint, a 40 feet-deep zone of improvement, and an estimated cost of $90 per cubic yard of treated soil, the estimated cost of soil-cement mixing for Project Zeus is on the order of $100M.

As a result of the improved soil stiffness, settlement of the structure(s) over soil-cement improved areas will be very small.

It is important to note that the soil-cement process generates spoil materials. The volume of spoil generated is typically on the order of 25% of the volume of the treated soil. In this example (2,000,000 square feet, 40 feet deep, 40% replacement), the volume of spoil could be about 300,000 cubic yards. The soil-cement spoil could be used for fill in other areas of the site, but would require some special handling considerations as it hardens quickly.

The duration of the soil-cement mixing activities will be highly dependent on volume as well as the Contractor's resources. A duration on the order of 12 to 18 months seems realistic for 2 rigs each working 2 shifts per day.

7.2.4.4 Stone Columns

Stone Columns are an effective alternative for mitigating settlement from liquefaction (discussed in Section 7.1) and to a lesser extent for mitigating settlement from compressibility of soft Young Bay Mud (discussed in Section 7.2). Amec Foster Wheeler has been involved in stone column improvement projects, but mostly on sites where the columns are used to improve loose granular deposits. We have not been involved in a project where stone columns were used to improve Young Bay Mud. Before fully recommending this approach for the project Zeus site, we recommend further vetting the method and site conditions with local Contractors.

From Engeo's Preliminary Geotechnical Cost Summary (2006), the estimated cost of installing GeoPiers, a variation of stone column improvement, is about $6 per square foot of development. This number seems low and likely does not include installing columns through the entire thickness of Young Bay Mud. Assuming a cost of about $15 per square foot of development and about a 2,000,000 square foot building footprint, the estimated cost of using stone column improvement for Project Zeus would be on the order $30M if the entire footprint required treatment.

7.2.4.5 Deep Dynamic Compaction

Deep dynamic compaction is an effective alternative for mitigating settlement from liquefaction in shallow layers (discussed in Section 7.1) but not for mitigating settlement from compressibility of soft Young Bay Mud (discussed in Section 7.2). Before fully recommending this approach for the project Zeus site, we recommend further vetting the method and site conditions with local Contractors.

Assuming a cost of about $5 per square foot of development and about a 2,000,000 square foot building footprint, the estimated cost of using stone column improvement for Project Zeus would be on the order $10M if the entire footprint required treatment.

The duration of the deep dynamic compaction activities will be highly dependent on actual treatment area and design details. A duration on the order of 6 to 12 months seems realistic.

7.2.5 Differential Settlement

Total estimated settlement values are large and will likely require mitigation.

Differential settlement can be estimated once there is more detailed information available regarding specific building usage, loads, and the mitigation method(s) preferred to reduce compressibility of Young Bay Mud. That said, accurately estimating differential settlements may be quite difficult for preloading or lightweight fill options because of the potential variability of the

Young Bay Mud. This implies that these improvement methods should only be used if the building structure(s) and/or floor slab can be designed to accommodate some risk of unexpected settlement.

7.2.6 Next Steps

Given the large settlements anticipated under the weight of new loads at the site, and the cost and schedule implications to various ground improvement methods, we recommend that Amec Foster Wheeler meet with the Owner and Designer to better define specific building location, usage, reasonable estimates of loading, and project budget and schedule constraints, to help further evaluate the suitability of various improvement methods and/or foundation alternatives.

7.3 FOUNDATIONS

7.3.1 Shallow Foundation Support

The feasibility of shallow foundation support at the Project Zeus will be mainly governed by the degree of corresponding settlement as discussed in Section 7.2. For existing site conditions (no improvement for fill or Young Bay Mud), the recommended ultimate (FS=1.0) bearing capacity for shallow foundations is 2,700 psf. Recommended allowable bearing pressures are presented below.

Loading Case	Recommended Factor of Safety	Recommended Allowable Bearing Capacity (psf)
Dead Loads	3.0	900
Dead plus Live Loads	2.0	1,300
All Loads, incl Seismic	1.5	1,800

For existing site conditions (no improvements for Fill or Young Bay Mud), the settlement corresponding to this level of loading will be large. If however the existing Fill and Young Bay Mud are improved, the degree of settlement corresponding to this level of loading may be within tolerable limits. If the existing Fill and Young Bay Mud are improved, the recommended ultimate and allowable bearing capacity values may also potentially increase as result of the improvements. Bearing capacity and corresponding settlement of specific shallow foundation schemes can be re-evaluated after potential mitigations and corresponding implementation details are selected.

7.3.2 Driven Pile Foundations

Where anticipated settlement exceeds structural tolerances, new structural loads (for buildings and slabs) can be supported on driven displacement piles that derive support from competent soils below the Young Bay Mud. Driven piles are typically long (can be up to 150 feet long), slender (1 to 5 feet diameter) elements that are hammered into the ground using a pile driving hammer. Driven piles can develop resistance to new structural loads from a combination of side friction (at the interface between the pile and the surrounding soil) and end bearing at the bottom of the pile. Driven piles are structurally connected to building columns, and if necessary to building slabs, and would be designed such that settlement under the proposed structural loads are within acceptable tolerances. It is our understanding that building and slab loads for many, possibly all, of the previous Navy structures are/were supported on driven timber and/or driven precast concrete piles. Timber piles are no longer typically used for new structures. Amec Foster Wheeler is familiar however with several commercial, industrial, and municipal sites around San Francisco Bay, that have subsurface conditions similar to those encountered at the Project Zeus site, which have their structural loads supported on driven precast concrete piles.

7.3.2.1 Vertical Load Capacity

The piles will derive their vertical load carrying capacity mainly from skin friction in the competent soils below the existing fill materials and Young Bay Mud deposits anticipated at the site. Estimated ultimate axial pile capacity in compression for new 16 inch-wide, and 24 inch-wide, square, precast concrete piles are presented on Figures 22 and 23 respectively. In these charts, the estimated ultimate pile capacities are plotted as a function of the pile tip elevations. We recommend that a factor of safety of 2.0 be used for combined dead and live loads and a factor of safety of 1.5 for all loads including wind or seismic. Piles designed using these charts are expected to settle less than one inch under design loads.

The ultimate pile capacity charts in Figures 22 and 23 are based on average soil layer thickness and soil properties encountered in the exploratory borings. Variations in both the layer thickness and soil properties do occur along the Project Zeus site and could affect the ultimate pile capacities. Bedrock was encountered as shallow as 95 feet (approximate elevation -85 feet) along the southern portion of the site. We recommend that an indicator pile driving program be used to evaluate the pile drivability and as-installed axial capacity at select locations before production piles are cast.

7.3.2.2 Other Driven Pile Types and Sizes

Precast concrete piles can be cast in various sizes and shapes. Amec Foster Wheeler has been involved in projects that included 14 inch-wide Square, 24 inch-diameter Octagonal, and 48 inch (outside diameter), by 6 inch-thick wall Cylindrical precast concrete piles as well as other sizes. The vertical load capacity will be different for each size and shape. Upon request, Amec Foster Wheeler can provide vertical load capacity estimates for shapes other than square, and for sizes other than 16 inch- and 24 inch-wide.

Occasionally there may be a cost benefit to using steel pipe piles, which also come in a broad range of sizes. Like with precast concrete piles, the vertical load capacity will be different for each size (diameter) of pipe pile. Upon request, Amec Foster Wheeler can provide vertical load capacity estimates for steel pipe piles. If steel pipe piles are used, we recommend that a steel plate be added to close the bottom of the pile before installation.

Low displacement pile types (e.g. steel H sections or steel W sections) are not effective for the conditions at the Project Zeus site.

7.3.2.3 Lateral Load Capacity

Driven piles are capable of resisting lateral loads. The load resisting characteristics of driven piles are heavily dependent on pile type, shape, and size. Amec Foster Wheeler can provide estimates of lateral load resistance, deflection, and bending moment of specific pile sections to proposed loading or deflection criteria once details regarding pile type, shape and size have been selected.

7.3.2.4 Pile Installation

The Contractor should select a hammer and driving system that is capable of driving the piles to the desired tip elevation and capacity without overstressing the piles in either tension or compression. Prior to the start of pile installation at the site, the Contractor should submit the following information regarding the hammer and driving system to the Engineer:

- hammer type and rated energy;
- helmet weight, including striker plate;
- hammer cushion material, cross-section area, and thickness; and
- pile cushion material and thickness.

7.3.2.5 Indicator Pile Driving

We recommend that an indicator pile driving program be incorporated into the project to evaluate pile drivability before the production piles are cast. An indicator pile program will help to confirm drivability for the hammer-pile system, confirm axial pile capacity, and will help to minimize installation costs. The indicator piles should be driven with the same equipment and hammer proposed for the production driving. The indicator piles will be driven at production pile locations and they can be used for subsequent structure support provided that the piles are not damaged. We recommend that approximately 5 to 10 percent of the proposed production piles be driven and tested during the indicator pile program.

We also recommend that a program of dynamic pile monitoring be undertaken during installation of the indicator piles and production piles to obtain information regarding pile capacity and stresses in the piles during driving. Dynamic pile monitoring consists of measuring force and acceleration near the top of the pile during driving and analyzing the data with a pile driving analyzer (pda). By analyzing piles during installation of the indicator piles and, in some cases, during redriving several days later, pile capacity and pile lengths can be more completely assessed. Stresses in the piles during driving and appropriate pile-driving criteria can also be evaluated from results of the dynamic pile monitoring.

It is recommended that at least 50% of the indicator piles be monitored with a pile driving analyzer during initial driving and restrike driving (a minimum of 14 days after initial driving). For purposes of scheduling and coordination, the dynamic pile monitoring work should be the responsibility of the Contractor.

7.3.2.6 Approximate Cost of Driven Pile Foundations

Assuming 80 feet-long, 16 inch-wide precast concrete piles, we estimate a cost of about $40 per square foot of development. For a 2,000,000 square foot building footprint, the estimated cost to support buildings and slabs on driven precast concrete piles would be on the order $80M.

7.3.3 Alternatives to Driven Pile Foundations

There are several variations of deep foundations that are installed by drilling as opposed to hammering. Many of these systems aren't recommended for thick deposits of Young Bay Mud as encountered at the Project Zeus site. Auger pressure grouted (APG) piles are however a drilled system that has been used around San Francisco Bay in similar soil conditions as those encountered on the Project Zeus site. During construction of an APG pile, a hole is drilled to a planned depth using a hollow stem continuous flight auger system. Though the total depth that can be achieved depends on site subsurface conditions, 90 to 100 feet is typically a practical maximum depth for non-displacement APG piles. During auger withdrawal, the hole is filled with cement grout under pressure from the auger tip. Some of the cuttings are displaced from the hole during the grouting process, but generally fewer spoils are generated during construction of APG piles than during construction of other drilled piles (e.g. CIDH piles, micropiles). Reinforcing steel is then placed in the hole to increase the stiffness and strength of the pile.

Displacement APG piles are similar to the non-displacement APG piles described above except that little or none of the cuttings are removed from the hole; rather, the materials are displaced to the side as the auger is twisted into the ground. When a hard or dense zone is encountered by the auger, this method requires an increase in downward pressure (down-crowd) and torque to advance the auger. Displacement APG piles have an advantage compared to conventional augercast piles in that they produce a relatively small volume of cuttings for disposal (this may be particularly attractive for sites with environmental considerations), and end-bearing zones can be detected by monitoring the down-crowd and torque. Conversely, they have the disadvantage of having less ability to penetrate through resistant zones to provide sufficient embedment for developing required uplift or lateral capacities. Each APG pile Contractor will have slightly different equipment and installation procedures that affect the ability to achieve penetration as well as the actual pile capacities. Proposed APG equipment, materials, and installation procedures should be vetted prior to selection of a Contractor.

It is our opinion that driven precast pile foundations are better suited than APG piles at the Project Zeus site because they generally have a higher percentage of reinforcing steel to resist damage during lateral loading, which could be a significant issue in the thick, soft Young Bay Mud soils.

7.3.4 Lateral Earth Pressures

Lateral loads may be resisted by soil friction and passive resistance of the soils.

A coefficient of friction of 0.4 may be used between foundations that are bearing on underlying soils (i.e. not pile supported). The passive resistance of properly compacted fill may be assumed to be 300 pounds per cubic foot.

A one third increase in the passive values may be used when considering wind or seismic loads. The passive resistance and the frictional resistance of the soils may be combined without reduction in determining the total lateral resistance.

7.4 Geotechnical Observation

The reworking of the upper soils, installation of foundations, and the compaction of all required fill should be observed and tested during placement by a qualified representative of the Client. This representative should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The representative should also observe proof-rolling and delineation of areas requiring overexcavation.
- Evaluate the suitability of onsite and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the Fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.
- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.
- Observe implementation of any ground improvement work.
- Observe installation of any deep foundation elements.

Any agencies having jurisdiction over the project should be notified prior to commencement of site preparation so that the necessary permits can be obtained and arrangements can be made for required inspection(s). The Contractor should be familiar with the inspection requirements of the reviewing agencies.

8.0 GENERAL LIMITATIONS AND BASIS FOR RECOMMENDATIONS

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report. This report has been prepared for Project Zeus and their design consultants to be used solely in the design of the proposed Mare Island Site. This report has not been prepared for use by other parties, and may not contain sufficient information for purpose of other parties or other uses.

The recommendations provided in this report are based upon our understanding of the described project information and on our interpretation of the data collected during our subsurface explorations. We have made our recommendations based upon experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the structure configuration, loads, location, or the site grades should be provided to us so that we can review our conclusions and recommendations and make any necessary modifications.

The recommendations provided in this report are also based upon the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical investigation and essential to verify that the actual soil conditions are as expected. This also provides for the procedure whereby the Client can be advised of unexpected or changed conditions that would require modifications of our original recommendations. If another firm is retained for the geotechnical observation services, our professional responsibility and liability would be limited to the extent that we would not be the Geotechnical Engineer of record.

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TABLES

TABLE 1

SEISMIC SOURCE PARAMETERS FOR FAULT SOURCES USING TIME DEPENDENT RECURRENCE MODELS
Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

Fault	Segment	Length, km [1]			WG03 Endpoint Uncertainty, km [2]	WG03 Dip	Rupture Width, km [3]		Seismic Scaling Factor		Slip Rate, mm/yr [4]	
		Preferred	Minimum	Maximum			Preferred	90% bounds	Values	Weights [5]	Preferred	95% bounds
San Andreas	Santa Cruz Mountains (SAS)	62	47	77	N±10, S±5	90°	15	13-17	0.84/0.94/1.0	(a)	18.6	12.9-22.7
	Peninsula (SAP)	85	60	110	N±15, S±10	90°	13	11-15	0.9/1.0	(b)	15.1	10.8-19.1
	North Coast (SAN)	190	170	210	N±5, S±15	90°	11	9-13	0.9/1.0	(b)	18	13.2-22.8
	Offshore (SAO)	136	116	156	N±15, S±5	90°	11	9-13	0.8/0.9/1.0	(b)	17	7.2-24.4
Hayward – Rodgers Creek	South (HS)	52	37	67	N±10, S±7.5	90°	13.4	11.4-15.4	0.72/0.82/0.92	(a)	9.8	7.8-11.8
	North (HN)	35	20	50	N±5, S±10	90°	11.1	9.1-13.1	0.74/0.84/0.94	(a)	8.3	6.6-9.7
	Rodgers Creek (RC)	64	54	74	N±5, S±5	90°	12	10-14	0.71/0.81/0.91	(a)	5.7	3.1-7.5
Calaveras	South (CS)	19	9	29	N±5, S±5	90°	9.5	8.5-11.5	0.42/0.52/0.62	(a)	11.5	7.7-14.6
	Central (CC)	59	49	69	N±5, S±5	90°	11	9-13	0.51/0.61/0.71	(a)	10.1	5.5-14.9
	North (CN)	45	35	55	N±5, S±5	90°	14	12-16	0.5/0.6/0.7	(a)	4.6	3.1-6.1
Concord – Green Valley	Concord (CON)	20	12	28	N±3, S±5	90°	15	13-17	0.45/0.55/0.65	(a)	3.1	2.1-3.9
	Green Valley South (GVS)	22	16	28	N±3, S±3	90°	14	12-16	0.19/0.29/0.39	(a)	3.5	1.7-5.0
	Green Valley North (GVN)	14	6	22	N±5, S±3	90°	14	12-16	0.19/0.29/0.39	(a)	3.5	1.7-5.0
San Gregorio	South (SGS)	66	46	86	N±5, S±3	90°	11.6	9.6-13.6	0.8/0.9/1.0	(a)	2.1	1.1-3.2
	North (SGN)	110	85	134	N±5, S±5	90°	12	10-14	0.89/0.99/1.0	(a)	4.6	1.8-7.05
Greenville	South (GS)	24	16	32	N±5, S±3	90°	10.6	8.6-12.6	0.8/0.9/1.0	(a)	1.5	0.1-2.5
	North (GN)	27	17	37	N±5, S±5	90°	15	12-18	0.46/0.56/0.66	(a)	2.3	1.3-3.0
Mt Diablo Thrust	(MDT – FM3.2)	25	15	35	N±5, S±5	30°SE	14.2	12.2-16.2	0.9/1.0	(b)	1.25	0.65-1.8

Notes:

1. The ruptures include each individual and all combinations of adjoining segments (e.g., rupture segments for the Hayward-Rodgers Creek are RC, HN, HS, HN+HS, RC+HN, and RC+HN+HS). Rupture models and weights for models taken from WG03 (WGCEP, 2003).
2. Represents uncertainty in the location of the northern (N) and southern (S) endpoints of each fault segment shown in WG03 (WGCEP, 2003). The uncertainty in the endpoint is modeled independently for each rupture segment, resulting in nine alternative lengths and locations for each rupture segment. The endpoints weights are: preferred - 0.63; N, S - 0.185. The fault trace locations are taken from WG03 and WG08 (WGCEP, 2008).
3. Down-dip width of rupture calculated based on upper and lower seismogenic depth from UCERF3 (Field et al., 2013) .Top of rupture is at ground surface (0), except for Mt. Diablo Thrust, where the top of rupture is located at 4 km depth following WG03 and WG08.
4. Slip rates are solution mean, and solution minimum and maximum from Fault Section Data excel file (UCERF3). Slip rates for Calaveras reduced by slip assigned to Calaveras West Napa extended source. Slip rates for Concord-Green Valley, Mt. Diablo Thrust, and Greenville faults reduced by slip assigned to Bartlett Springs - Concord extended fault source. Weights for slip rates are 0.13/0.74/0.13, except for San Andreas at 0.185/0.63/0.185.
5. Scaling factors based on data from Fault Section Data excel file (UCERF3) and WG03. Weights: (a) Mean and 90% bounds: 0.185/0.63/0.185. (b) Only lower 90% bound set: 0.185/0.815.

TABLE 2

SEISMIC SOURCE PARAMETERS FOR FAULT SOURCES USING POISSON RECURRENCE MODEL

Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

Fault (Slip Type) [Model Weight] [1]	Total Fault Length (km)	Fault Dip and Direction	Rupture Length (km) [2]	Depth to Top of Rupture (km) [3]	Downdip Width of Rupture (km) [4]	Maximum Magnitude (M) [5]	Slip Rate (mm/yr) [6]
Cull Canyon-Lafayette-Reliz Valley (SS) [1.0]	25	90°	25 (1.0)	0	9 (0.185) 12 (0.63) 15 (0.185)	6.5	0.012 (0.185) 0.97 (0.63) 3.07 (0.185)
Briones Zone (SS) [1.0]	23	90°	23	0	11 (0.185) 13 (0.63) 15 (0.185)	6.5	0.012 (0.185) 1.83 (0.63) 5.70 (0.185)
Las Trampas (R-SS) [0.5]	10.7	45°SW (0.33) 60°SW (0.34) 75°SW (0.33)	10.7 (1.0)	0	11 (0.185) 14 (0.63) 17 (0.185)	6.1	0.43 (0.2) 0.85 (0.6) 2.55 (0.2)
West Napa and St. Helena/Dry Creek (SS) [1.0]	57 (SH/DC+WN) 38 (WN) 26 (SH/DC)	90°	1) 52 (0.15) 2) Floating M 6.5 WN-SH/DC (0.35) 3) 38 WN + 24 SH/DC (0.15) 4) 20 SH/DC + floating M 6.4 on WN (0.35)	0	11 (0.185) 13 (0.63) 15 (0.185)	1) 6.9 2) 6.5 3) 6.8 4) 6.4	0.72 (0.185) 1.16 (0.63) 3.7 (0.185)
Clayton (SS) [1.0]	16.4	90°	16.4 (1.0)	0	14 (0.185) 16 (0.63) 18 (0.185)	6.3	0.33 (0.185) 0.68 (0.63) 1.53 (0.185)
Potrero Hills (R) [0.7]	8	30°SW (0.25) 40°SW (0.5) 50°SW (0.25)	8 (1.0)	0	7 (0.185) 9 (0.63) 11 (0.185)	5.8	0.1 (0.2) 0.3 (0.6) 0.6 (0.2)

TABLE 2

SEISMIC SOURCE PARAMETERS FOR FAULT SOURCES USING POISSON RECURRENCE MODEL

Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

Fault (Slip Type) [Model Weight] [1]	Total Fault Length (km)	Fault Dip and Direction	Rupture Length (km) [2]	Depth to Top of Rupture (km) [3]	Downdip Width of Rupture (km) [4]	Maximum Magnitude (M) [5]	Slip Rate (mm/yr) [6]
Wragg Canyon (SS) [0.7]	22	90°	17 (1.0)	0	12 (0.185) 15 (0.63) 18 (0.185)	6.3	0.1 (0.2) 0.3 (0.6) 0.5 (0.2)
Cordelia (SS) [1.0]	19	90°	19 (1.0)	0	12 (0.185) 15 (0.63) 18 (0.185)	6.3	0.05 (0.4) 0.6 (0.5) 1.0 (0.1)
Extended Fault Sources							
West Napa-Briones-Cull Canyon-Las Trampas-Calaveras (WN-Cal.) (SS) [1.0]	269	90°	269 (1.0)	0	10.8 (0.185) 12.8 (0.63) 14.8 (0.185)	7.8	0. 15 (1.0)
Bartlett Springs-Hunting Creek-Berryessa-Green Valley-Concord-Clayton-Greenville (BS-GVN) (SS) [1.0]	384	90°	384 (1.0)	0	11.6 (0.185) 13.6 (0.63) 15.6 (0.185)	8.0	0. 30 (1.0)
Faults at eastern margin of Coast Range/western margin of Central Valley (Great Valley blind thrusts)							
Los Medanos/Roe Island Fold and Thrust belt [1.0]	1) 19 - LM + RI un-segmented	30° NE (0.2) 45° NE (0.6) 60° NE (0.2)	15 (0.2)	0	17 ± 2	6.3	0.01 (0.185) 0.18 (0.63) 0.57 (0.185)
	2) 5 - Roe Island, 10 - Los Medanos		5 on RI and 10 on LM (0.8)	0	RI - 5 ± 2 LM - 10 ± 2	5.4 5.9	
Mysterious Ridge (GV-3) (blind thrust) [1.0]	35	20° W (0.25) 25° W (0.5) 30° W (0.25)	35 (1.0)	Top 9, Bottom 14	11 (0.185) 13 (0.63) 15 (0.185)	6.7	0.26 (0.185) 1.32 (0.63) 1.97 (0.185)

TABLE 2

SEISMIC SOURCE PARAMETERS FOR FAULT SOURCES USING POISSON RECURRENCE MODEL

Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

Fault (Slip Type) [Model Weight] [1]	Total Fault Length (km)	Fault Dip and Direction	Rupture Length (km) [2]	Depth to Top of Rupture (km) [3]	Downdip Width of Rupture (km) [4]	Maximum Magnitude (M) [5]	Slip Rate (mm/yr) [6]
Trout Creek-Gordon Valley (GV-4a and b) (blind thrust) [1.0]	1) 38 TC+GV un-segmented (0.2)	15° W (0.25) 25° W (0.5) 35° W (0.25)	38 (1.0)	Top 9, Bottom 14	11 (0.185) 13 (0.63) 15 (0.185)	6.7	0.26 (0.185) 1.20 (0.63) 1.65 (0.185)
	2) 20 - TC and	15° W (0.25) 25° W (0.5) 35° W (0.25)	20 - TC	Top 9, Bottom 14	11 (0.185) 13 (0.63) 15 (0.185)	6.4 - TC	0.26 (0.185) 1.20 (0.63) 1.65 (0.185)
	18 - GV (0.8)	25° W (0.25) 30° W (0.5) 35° W (0.25)	18 - GV			6.3 - GV	
Pittsburgh-Kirby Hill (GV-5) (RO) [1.0]	1) 28 un-segmented PB+KH (0.5)	45°E (0.25) 55°E (0.5) 65°E (0.25)	24 (1.0)	0	11 (0.185) 13 (0.63) 15 (0.185)	6.6	0.23 (0.185) 1.04 (0.63) 1.72 (0.185)
	2) 28 un-segmented PB+KH (0.5)	90° (1.0)	24 (1.0)	10	8 (0.185) 10 (0.63) 12 (0.185)	6.6	0.45 (0.185) 1.01 (0.63) 1.44 (0.185)
Montezuma Hills Zone (RO) [0.5]	18	70°W	N/A	0	10 (0.185) 15 (0.63) 20 (0.185)	6.4	0.05 (0.3) 0.25 (0.4) 0.5 (0.3)

Notes:
1. The model weights shown in brackets represent the probability that the fault is active/capable of generating large earthquakes, and are taken from the Delta Risk Management Study (DRMS) Seismology Report (URS Corporation/J.R. Benjamin & Associates, 2007).
2. Rupture lengths for the Briones and Montezuma Hills fault source zones represent the maximum length of faults within the zone. Fault locations are from DRMS.
3. Depth to top of rupture from Fault Section Data excel spreadsheet published in UCERF3 (Field et al., 2013) or from DRMS.
4. Rupture widths calculated based on fault dip and depth of seismogenic zone. Thickness based on seismicity data from WG02 and Simpson et al (2004), and the lower seismogenic depth presented in UCERF3 (ofr2013-1165_FaultSectionData.xls, available at http://pubs.usgs.gov/of/2013/1165/).
5. Maximum magnitude estimated based on tectonic setting, historical earthquakes, fault geometry, and previous published assessments of maximum magnitudes, and is consistent for maximum magnitudes from DRMS model.
6. Slip rates from UCERF3 (ofr2013-1165_FaultSectionData.xls), DRMS or WG08 (WGCEP, 2007).

TABLE 3

HORIZONTAL MEDIAN DETERMINISTIC RESPONSE SPECTRA FOR SIGNIFICANT FAULT SOURCES

Geotechnical Investigation
Project Zeus – Mare Island
Vallejo, California

Period (seconds) [1]	Median Spectral Acceleration (g) [2]					
	Hayward-Rodgers Creek M 7.3 at 7.3 km	Concord-Green Valley M 7.1 at 14 km	Franklin M 6.8 at 0.7 km	West Napa M 6.9 at 6.7 km	San Andreas M 8.0 at 42 km	Scale Factor for 84^{th} Percentile Maximum Demand [3]
0.01	0.275	0.241	0.549	0.357	0.157	2.00
0.02	0.278	0.244	0.561	0.363	0.158	2.00
0.03	0.298	0.262	0.603	0.390	0.167	2.00
0.05	0.361	0.318	0.728	0.473	0.199	2.00
0.075	0.449	0.398	0.894	0.588	0.241	2.00
0.1	0.518	0.460	1.020	0.678	0.273	2.00
0.15	0.613	0.544	1.206	0.805	0.313	2.00
0.2	0.643	0.568	1.278	0.846	0.329	2.00
0.25	0.626	0.550	1.267	0.825	0.327	2.00
0.3	0.591	0.517	1.206	0.778	0.317	2.00
0.4	0.515	0.447	1.041	0.670	0.283	2.05
0.5	0.450	0.388	0.900	0.579	0.253	2.10
0.75	0.327	0.278	0.663	0.417	0.190	2.20
1	0.258	0.217	0.517	0.325	0.151	2.30
1.5	0.174	0.144	0.326	0.210	0.109	2.40
2	0.130	0.106	0.227	0.151	0.085	2.50
3	0.086	0.069	0.140	0.095	0.061	2.60
4	0.063	0.049	0.093	0.065	0.047	2.70
5	0.047	0.036	0.065	0.047	0.038	2.70
7.5	0.025	0.019	0.030	0.023	0.023	2.70
10	0.015	0.011	0.017	0.013	0.015	2.70

Notes:

1. Response spectra are five-percent damped, except for PGA (0.01 seconds), which is not damped.
2. Distances represent closest distance of rupture to site (R_{Rup}).
3. Factors to adjust median (GMROTI50) NGA West spectra to 84^{th} percentile maximum demand spectra from Huang et al., 2008)

TABLE 4

HORIZONTAL SITE-SPECIFIC PROBABILISTIC AND DETERMINISTIC RESPONSE SPECTRA, TARGETED RISK FACTORS, AND GENERAL PROCEDURE SPECTRA

Geotechnical Investigation
Project Zeus – Mare Island
Vallejo, California

Period (seconds)[1]	Spectral Acceleration (g)					
	1. 2% P_E in 50 Years – GMRotD50[2, 3]	2. Targeted Risk Factors (ASCE 7-10)	3. Median Deterministic[3]	4. 84th Percentile Deterministic[3]	5. General Procedure Spectra – MCE[4]	6. General Procedure Spectra – Design
0.01	0.896	1.089	0.549	0.980	0.687	0.458
0.02	0.916	1.089	0.561	1.003	0.773	0.515
0.03	1.009	1.089	0.603	1.084	0.860	0.573
0.05	1.302	1.089	0.728	1.328	1.033	0.688
0.075	1.703	1.089	0.894	1.654	1.249	0.833
0.1	1.969	1.089	1.020	1.895	1.465	0.977
0.15	2.231	1.089	1.206	2.232	1.500	1.000
0.2	2.299	1.089	1.278	2.380	1.500	1.000
0.25	2.217	1.088	1.267	2.376	1.500	1.000
0.3	2.074	1.087	1.206	2.289	1.500	1.000
0.4	1.788	1.085	1.041	2.002	1.500	1.000
0.5	1.554	1.082	0.900	1.756	1.500	1.000
0.75	1.127	1.077	0.663	1.330	1.500	1.000
1	0.852	1.071	0.517	1.049	1.500	1.000
1.5	0.525	1.071	0.326	0.665	1.040	0.693
2	0.366	1.071	0.227	0.464	0.780	0.520
3	0.223	1.071	0.140	0.286	0.520	0.347
4	0.153	1.071	0.093	0.187	0.390	0.260
5	0.113	1.071	0.065	0.131	0.260	0.173
7.5	0.062	1.071	0.030	0.060	0.195	0.130
10	0.798	1.071	0.017	0.035	0.156	0.104

Notes:

1. Response spectra are five-percent damped, except for PGA (0.01 seconds), which is not damped.
2. The site-specific spectra in columns 1, 3, and 4 are mean spectra, referred to as GMRotD50. These spectra have not been adjusted for maximum demand. P_E – Probability of Exceedance.
3. Median and 84th percentile deterministic spectra are for controlling deterministic source (largest spectral acceleration) at each period (from Table 3). These spectra have not been adjusted to maximum demand.
4. The General Procedure Spectra (GPS) are for Site Class C (F_a = 1.0, F_v = 1.3). The GPS spectra are calculated following Chapter 11.4.5 of ASCE 7-10. The corner periods, T_o and T_s, are calculated to be 0.104 and 0.520 seconds, respectively, for Site Class C, and the long period transition, T_L, is 8 seconds, from ASCE 7-10.

TABLE 5

SCALE FACTORS FOR MAXIMUM DEMAND AND COMPARISON OF HORIZONTAL SITE-SPECIFIC PROBABILISTIC AND DETERMINISTIC RESPONSE SPECTRA FOR SELECTION OF MCE_R RESPONSE SPECTRA

Geotechnical Investigation
Project Zeus – Mare Island
Vallejo, California

Period (seconds)[1]		Spectral Acceleration (g)				
	7. Scale Factor for Median Maximum Demand[1]	8. 84^{th} Percentile Deterministic (Maximum Demand)[2]	9. Deterministic Limit	10. Deterministic MCE	11. 2% P_E in 50 Years (RT, Maximum Demand)[2, 3]	12. Site-Specific MCE_R (Maximum Demand)
0.01	1.10	1.098	0.687	1.098	1.073	1.073
0.02	1.10	1.123	0.773	1.123	1.098	1.098
0.03	1.10	1.207	0.860	1.207	1.209	1.207
0.05	1.10	1.457	1.033	1.457	1.560	1.457
0.075	1.10	1.788	1.249	1.788	2.040	1.788
0.1	1.10	2.040	1.465	2.040	2.359	2.040
0.15	1.10	2.412	1.500	2.412	2.673	2.412
0.2	1.10	2.556	1.500	2.556	2.754	2.556
0.25	1.10	2.535	1.500	2.535	2.653	2.535
0.3	1.10	2.412	1.500	2.412	2.480	2.412
0.4	1.15	2.134	1.500	2.134	2.229	2.134
0.5	1.20	1.890	1.500	1.890	2.018	1.890
0.75	1.25	1.458	1.040	1.458	1.517	1.458
1	1.30	1.189	0.780	1.189	1.187	1.187
1.5	1.30	0.782	0.520	0.782	0.730	0.730
2	1.30	0.568	0.390	0.568	0.510	0.510
3	1.35	0.365	0.260	0.365	0.322	0.322
4	1.40	0.250	0.195	0.250	0.229	0.229
5	1.40	0.175	0.156	0.175	0.170	0.170
7.5	1.40	0.080	0.104	0.104	0.093	0.093
10	1.40	0.048	0.062	0.062		0.062

Notes:

1. ASCE/SEI 7-10 requires that spectra be developed using Next Generation Attenuation (NGA) relationships and to be adjusted to maximum demand. The scale factors for maximum demand in column 7 are based on factors listed in Federal Emergency Management Agency (FEMA) P-750 (BSSC, 2009) and are interpolated for additional periods.
2. The site-specific spectra in columns 8 and 11 are adjusted from GMRotD50 (for probabilistic spectra) or 84^{th} percentile (for deterministic spectra) to maximum demand spectra using these scale factors.
3. P_E – Probability of Exceedance; RT – Risk Targeted.

TABLE 6

COMPARISON OF SITE-SPECIFIC MCE_R SPECTRA WITH GENERAL PROCEDURE SPECTRA FOR SELECTION OF DESIGN RESPONSE SPECTRA

Geotechnical Investigation
Project Zeus – Mare Island
Vallejo, California

Periods (seconds)	Spectral Acceleration (g)		
	13. ⅔ of MCE_R (Maximum Demand)	14. 80% of General Procedure Spectra - Design	15. Site-Specific Design (Maximum Demand)
0.01	0.716	0.366	0.716
0.02	0.732	0.412	0.732
0.03	0.804	0.458	0.804
0.05	0.971	0.551	0.971
0.075	1.192	0.666	1.192
0.1	1.360	0.782	1.360
0.15	1.608	0.800	1.608
0.2	1.704	0.800	1.704
0.25	1.690	0.800	1.690
0.3	1.608	0.800	1.608
0.4	1.423	0.800	1.423
0.5	1.260	0.800	1.260
0.75	0.972	0.555	0.972
1	0.791	0.416	0.791
1.5	0.487	0.277	0.487
2	0.340	0.208	0.340
3	0.215	0.139	0.215
4	0.152	0.104	0.152
5	0.113	0.083	0.113
7.5	0.062	0.055	0.062
10	0.042	0.033	0.042

Note:

1. The spectra in columns 13 and 15 have been adjusted from GMRotI50 or Median spectra to maximum demand spectra as shown on Table 2 and are risk-targeted consistent with ASCE/SEI 7-10.

TABLE 7

MATERIAL PROPERTIES USED IN SITE RESPONSE ANALYSIS

Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

Material Description	Symbol	Depth Range (ft)	Unit Weight (pcf)	Shear Wave Velocity (ft/sec)			Nonlinear Property Curves
				General Profile	Min	Max	
Fill	FILL	0 to 10	120	390	300	510	Rollins et al. (1998)
Young Bay Mud	BM	10 to 40	100	270	240	300	Bay Mud (Doyle)
Old Bay Clay (1)	OBS	40 to 60	115	585	495	690	Old Bay Clay (Doyle)
Old Bay Clay (2)	OBS2	60 to 90	120	725	670	780	Old Bay Clay (Doyle)
Old Alluvium	OA	90 to 170	120	950	815	1115	Vucetic and Dobry (1991), PI =50
Bedrock[1]	BDR	170+	140	1800	1450	2150	Linear 5% Damped

Notes:

1. Bedrock depth randomized about 170 feet from a minimum of 95 feet to a maximum of 240 feet.
2. Randomization generally follows Toro (1992) Site Variation Model

TABLE 8

MATERIAL PROPERTIES USED IN CONSOLIDATION ANALYSIS

Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

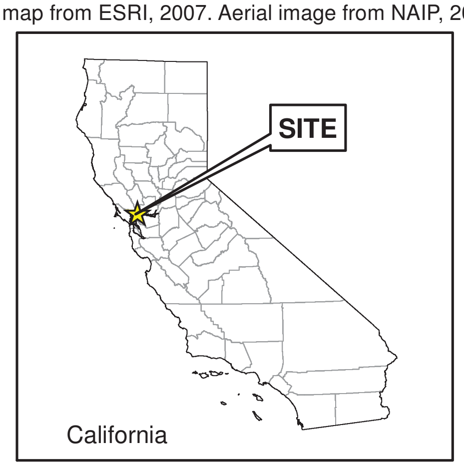
Material Description	Symbol	Approximate Depth Range (ft)	Unit Weight (pcf)	C_{ce}	C_{re}	OCR	C_v (ft^2/yr)	C_{vr} (ft^2/yr)
Pre-Load Material	PL	5, 10, and 15 feet height	115	-	-	-	-	-
Fill	FILL	0 to 10	120	-	-	-	-	-
Young Bay Mud	BM	10 to 12	100	0.3	0.04	1 to 1.5	15	100
Young Bay Mud 2	BM2	12 to 30	100	0.3	0.04	1 to 1.2	15	150
Young Bay Mud 3	BM3	30 to 40	100	0.3	0.04	1	10	200
Old Bay Clay (1)	OBC	40 to 60	115	0.23	0.04	3	10	100
Old Bay Clay (2)	OBC2	60 to 85	120	0.23	0.04	3	10	100

Notes:

1. Consolidation properties represent best estimate of representative value for soil layers.
2. Fill not evaluated as a part of consolidation analysis.

FIGURES

Street map from ESRI, 2007. Aerial image from NAIP, 2012.



N

0 1 2 3 4 Kilometers

SITE LOCATION MAP
Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

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Figure
1

Date: 12/11/2015

Project No. 6166150082.14.04

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Explanation

- Site boundary
- Proposed Building Footprint
- Approximate location of borings by performed by AMECFW for this study (2015)
- Approximate location of CPT performed by AMECFW for this study (2015)
- Approximate location of borings performed by others
- Approximate location of CPT performed by others

P03: Parikh (2003)
E02 and E11: ENGEO (2006)
H78: Harding Lawson (1978)
H82: Harding Lawson (1982)
H84: Harding Lawson (1984)
L01: Levine Fricke (2001)
N01: Ninyo & Moore (2001)
CAL: Caltrans (1996)

0 500 Feet

SITE EXPLORATION MAP
Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

Date: 12/16/2015 | Project No. 6166150082.14.04

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Figure 2

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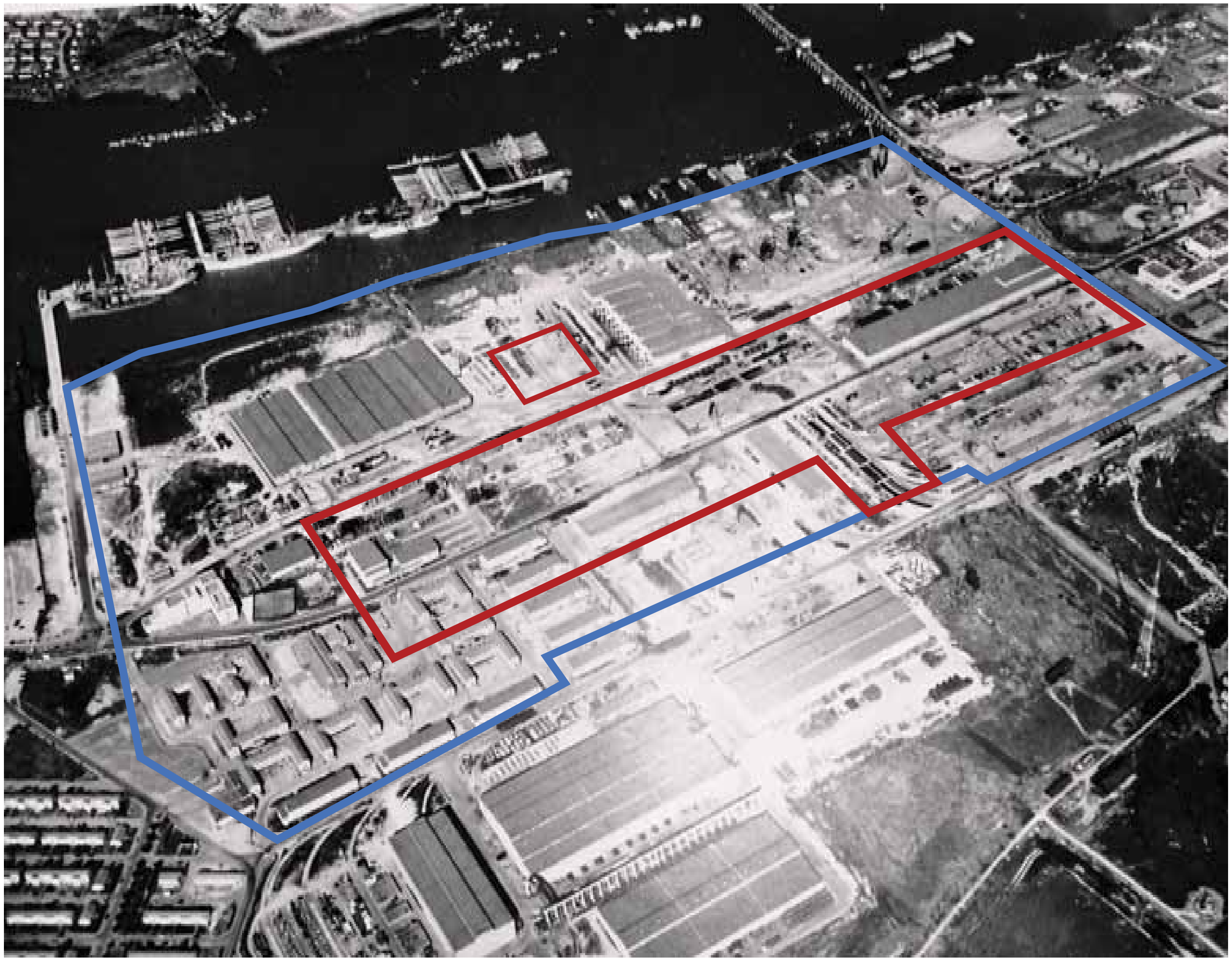
I:\Project\6166150082\5000 investigation\Mare Island\site photos\historical photos\iphone



Approximate Site Boundary

Approximate Proposed Building Footprint

Photograph circa 1946 from Mare Island Museum view to southeast

OBLIQUE AERIAL PHOTOGRAH
SHOWING HISTORICAL USE
Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

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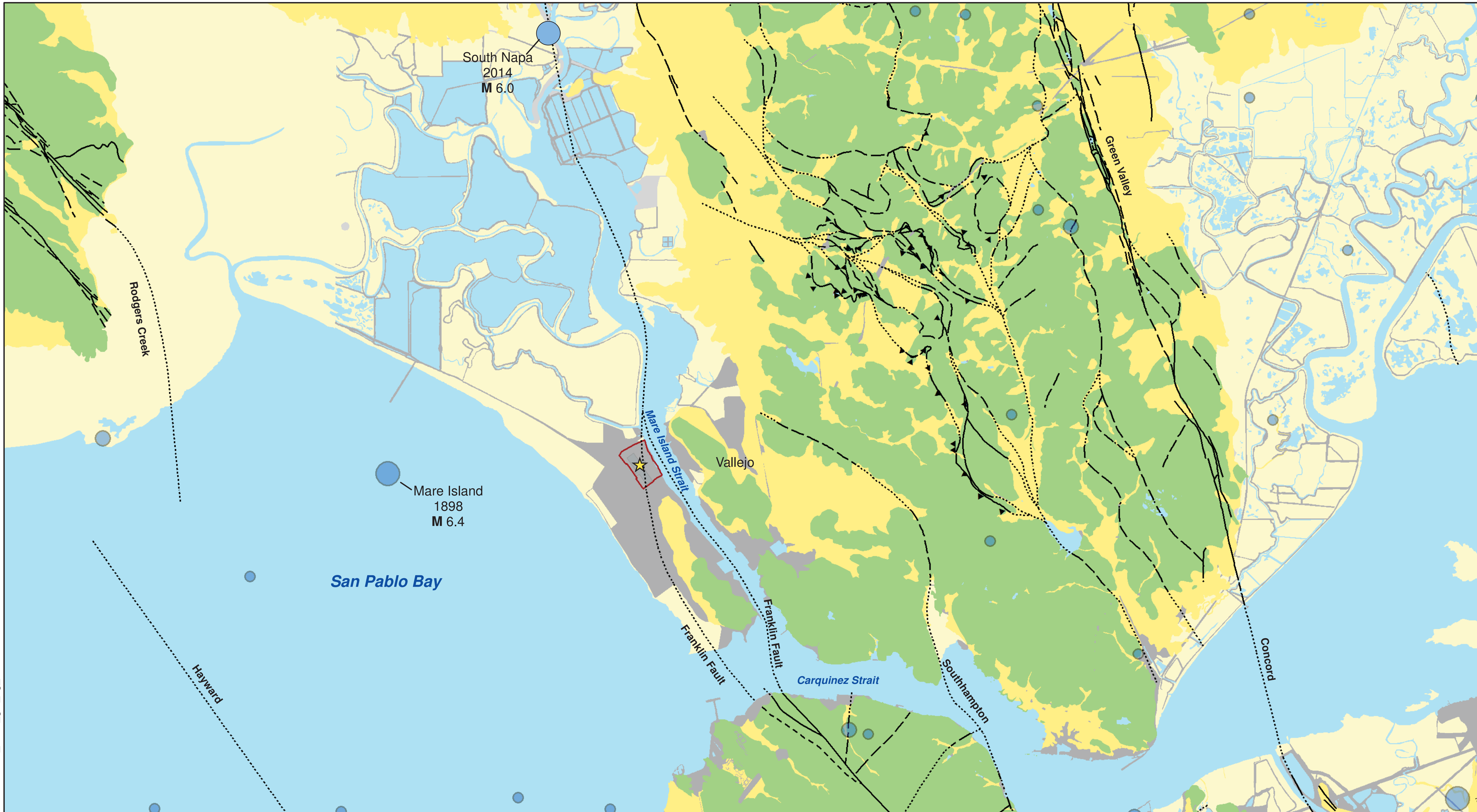
Date: 12/14/2015	Project No. 6166150082.14.04	Figure 3



Explanation

☆ Site

Site Boundary

Independent Seismicity
M (1800 - 2003)
2.5
3.0
4.0
5.0
6.5

Geologic units (Graymer, 2002)
Bedrock older than 1 million years
Surficial deposits less than 1 million years old
Bay mud less than 10 thousand years old
Artificial Fill

Fault - Dashed where approximately located, dotted where concealed, queried where location is uncertain

Reverse Fault - Dashed where approximately located, dotted where concealed, queried where location is uncertain

0 2 Miles

N

REGIONAL GEOLOGIC MAP
Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

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Date: 12/14/2015 | Project No.6166150082.14.04

Figure 4

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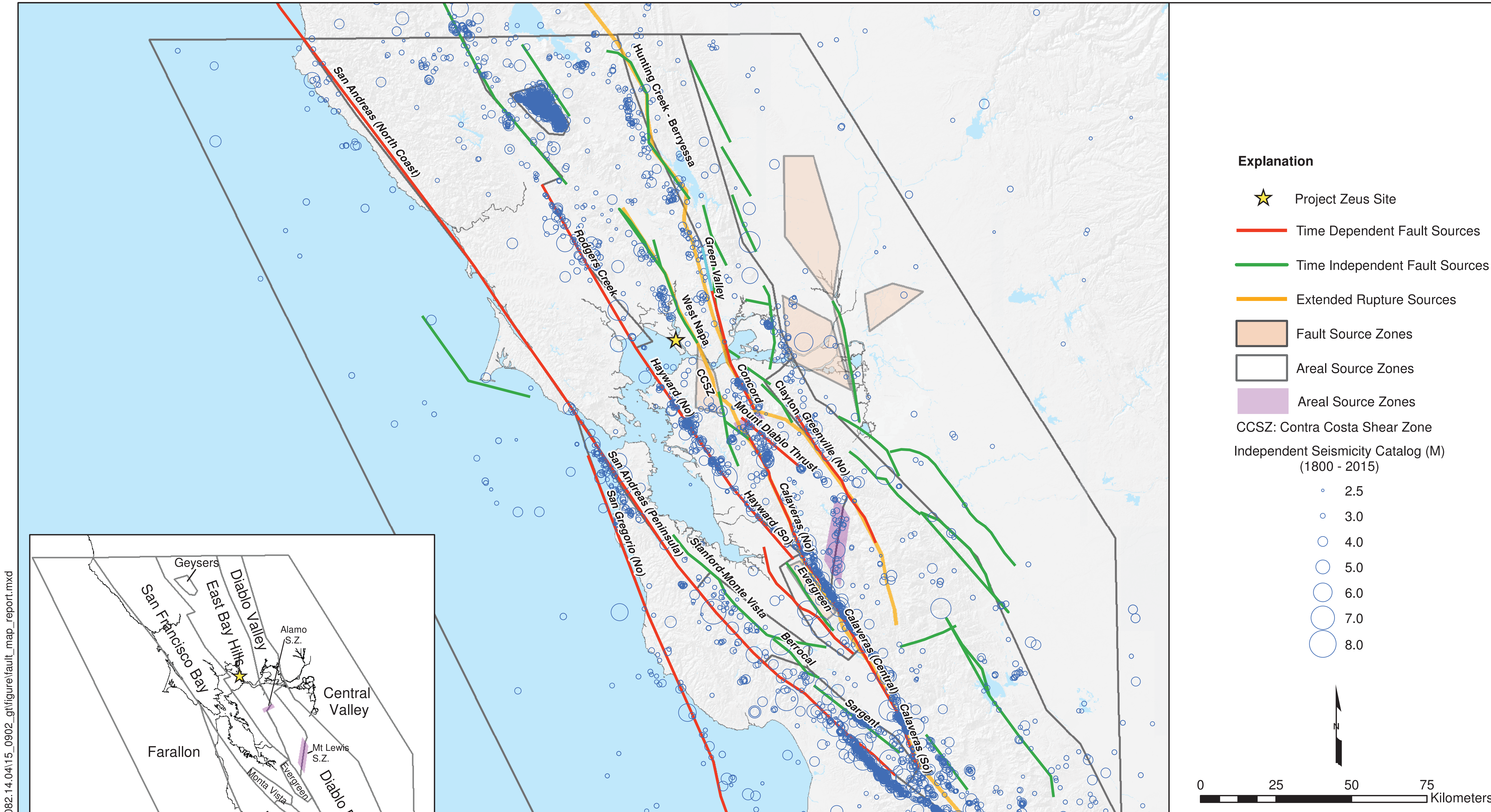

FAULT AND AREAL ZONE SOURCES
WITH INDEPENDENT SEISMICITY
Geotechnical Investigation
Project Zeus - Mare Island
Vallejo, CA

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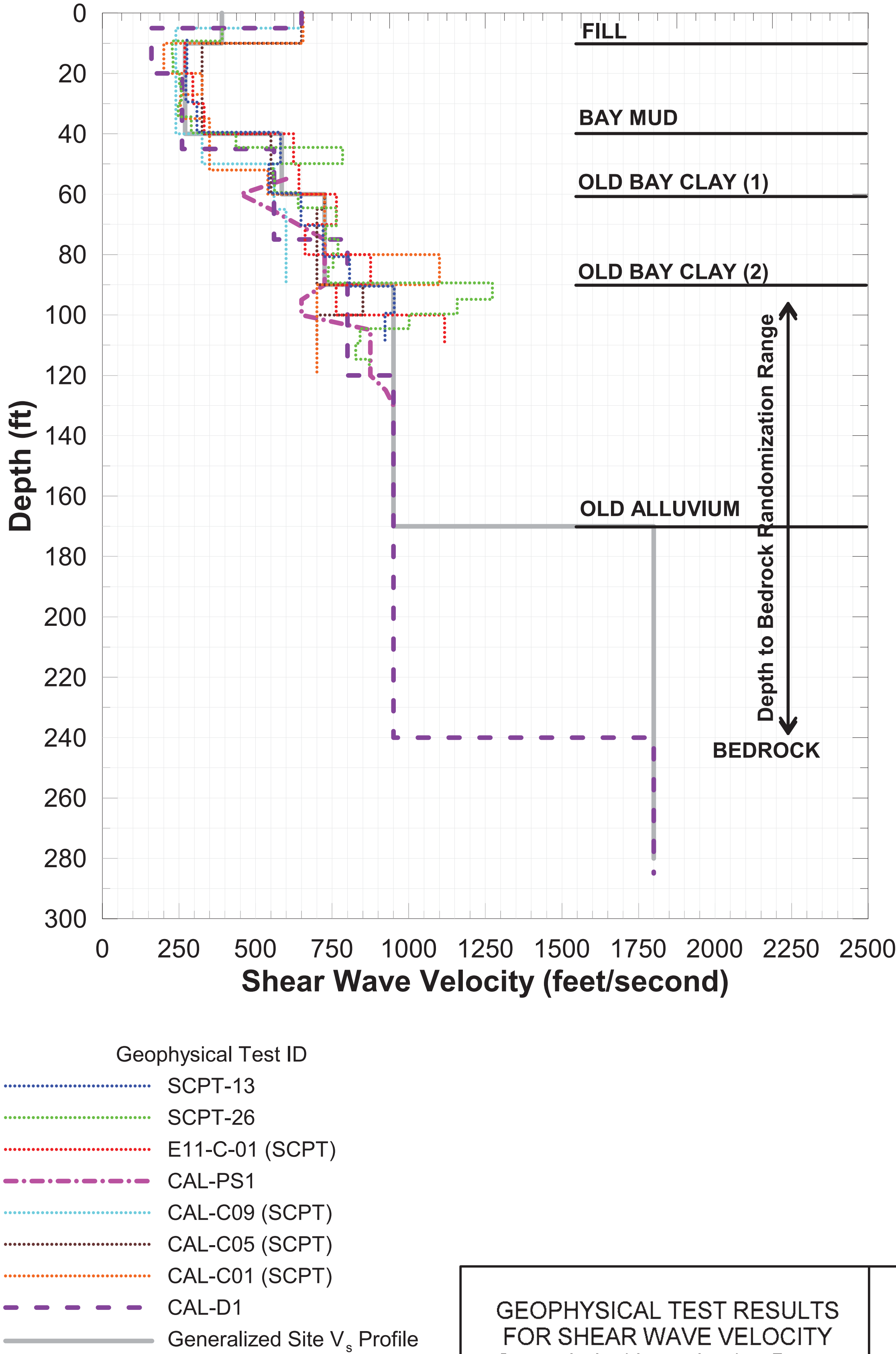
Date: 12/16/2015 | Project No.6166150082.14.04

Figure 5



GEOPHYSICAL TEST RESULTS
FOR SHEAR WAVE VELOCITY
Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

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Date: Nov 2015 | Project No. 6166150082

Figure 6

I:\Project\6166150082\12000 CALC and EVAL\Mare Island\Site Response

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Explanation

Site boundary

Proposed Building Footprint

Historic Shoreline (NFEC, 1980)

0 1,000 2,000 Feet

HISTORICAL SHORELINE
Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

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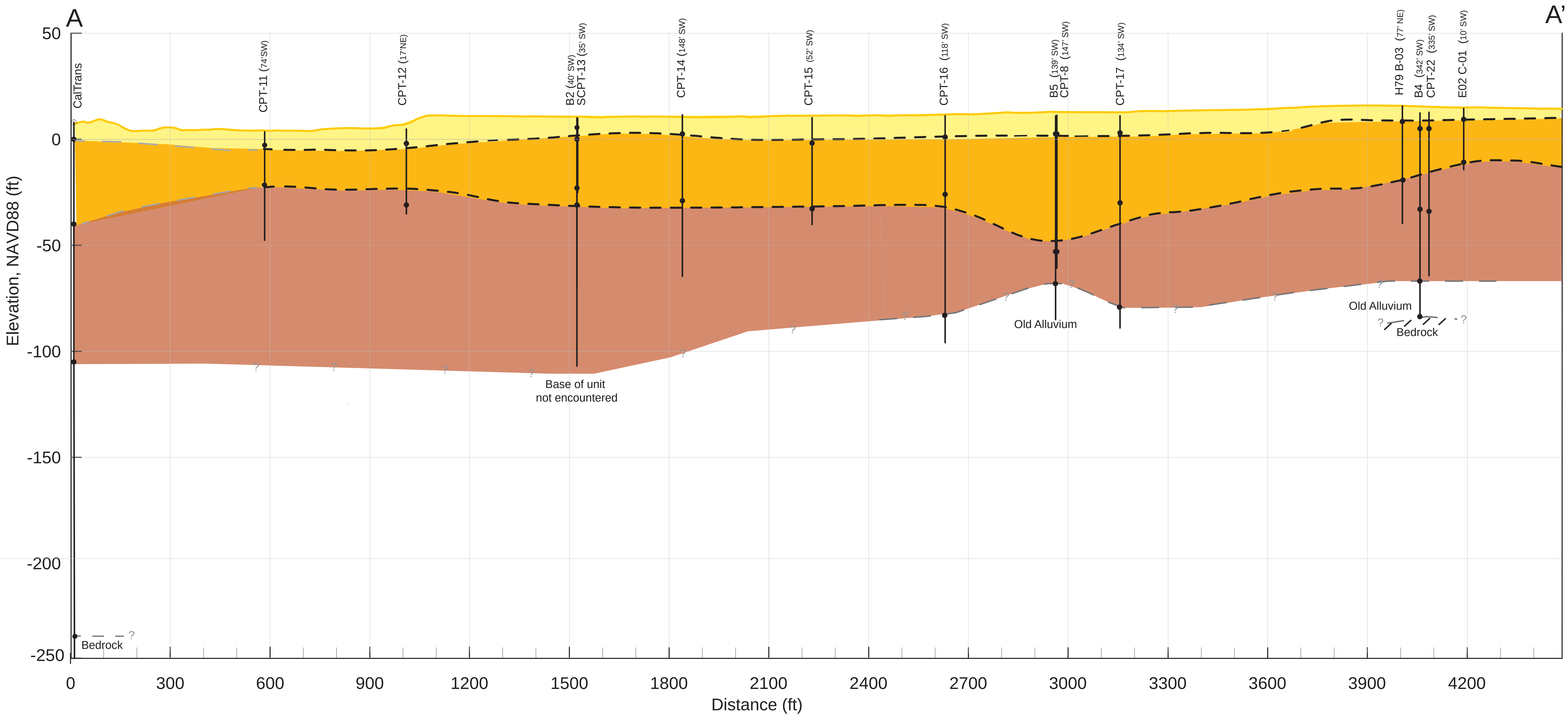
Date: 12/14/2015

Project No. 6166150082.14.04

Figure 7



CPT-12 (17'NE)

Exploration ID, projection distance and direction from exploration to cross section

Geologic contact at exploration location

Fill

Young Bay Mud

Old Bay Sediments

Existing ground surface

Approximate geologic contact

Inferred geologic contact

Top of weathered bedrock

Notes-

1. Ground surface elevation from NED (USGS, 2014); See Figure 2 for map of exploration locations; See Appendix B for logs of explorations.
2. The geologic contacts mapped in the section line are defined by the contact depths identified in exploration sites close to the section line as well as the dip of the contact inferred from contact depths identified in additional explorations away from the section line. Therefore, the mapped contacts do not intersect all of the contacts at exploration locations projected into thes section line.
3. Depth to geologic contacts at Caltrans exploration location are taken from Walnut Street Overcrossing section and Downhole boring at station 403+40 (Caltrans, 1993)

CROSS SECTION A - A'
Geotechnical Investigation
Project Zeus - Mare Island
Vallejo, California

Date: 12/09/2015 | Project No. 616615082.14.04

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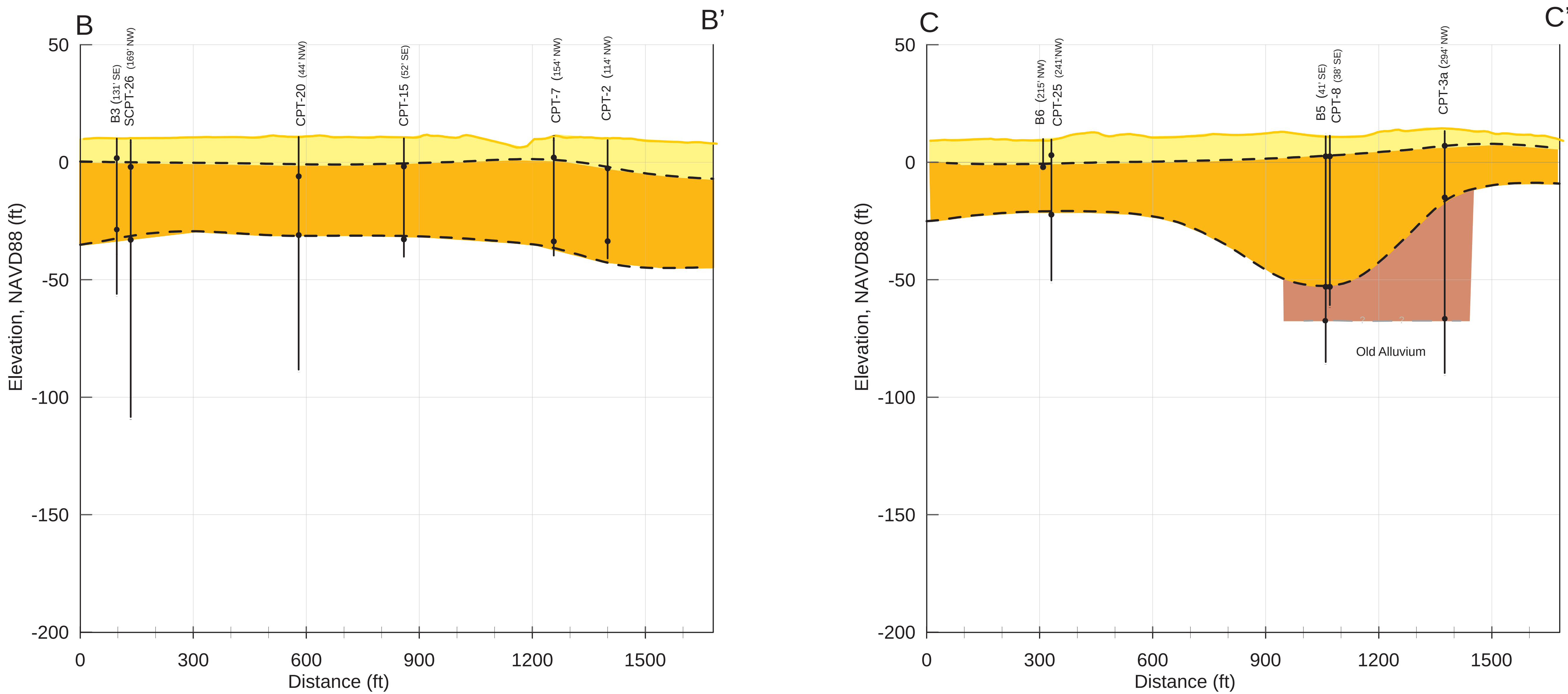
Figure
8

I:\Project\616615082\12000 CALC and EVAL\Mare Island\Geology



CPT-12 (17'NE)

Exploration ID, projection distance and direction from exploration to cross section

Geologic contact at exploration location

Fill

Young Bay Mud

Old Bay Sediments

Existing ground surface

Approximate geologic contact

Inferred geologic contact

Top of weathered bedrock

Notes-

1. Ground surface elevation from NED (USGS, 2014); See Figure 2 for map of exploration locations; See Appendix B for logs of explorations.
2. The geologic contacts mapped in the section line are defined by the contact depths identified in exploration sites close to the section line as well as the dip of the contact inferred from contact depths identified in additional explorations away from the section line. Therefore, the mapped contacts do not intersect all of the contacts at exploration locations projected into the section line.

CROSS SECTION B - B'
and C - C'
Geotechnical Investigation
Project Zeus - Mare Island
Vallejo, California

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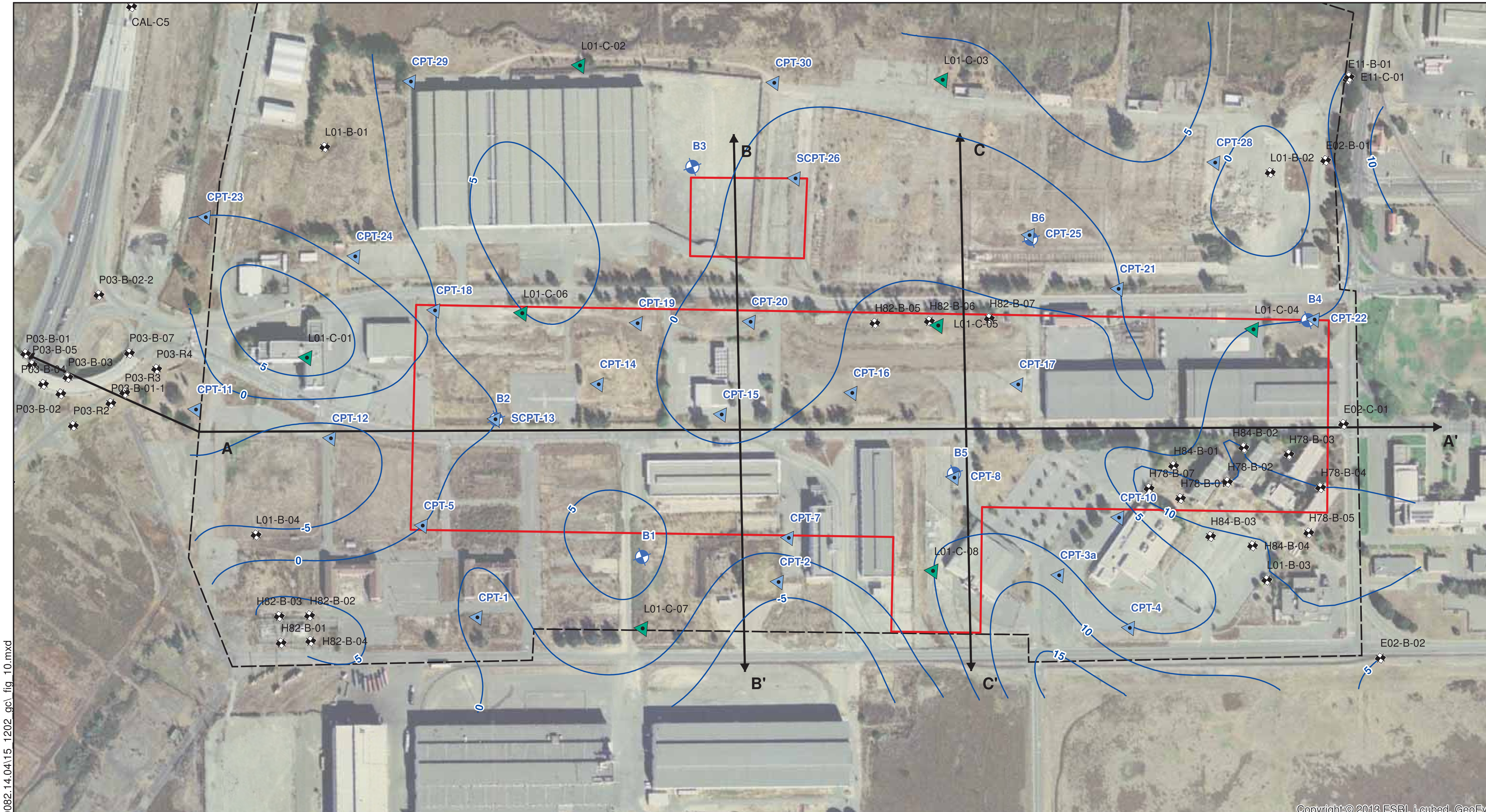
Figure
9

Date: 12/09/2015 | Project No. 616615082.14.04

I:\Project\616615082\12000 CALC and EVAL\Mare Island\Geology



Explanation

Proposed Building Footprint

Site boundary

Bottom of FILL

Elevation Datum: NAVD 88

Approximate location of borings by performed by AMECFW for this study (2015)

Approximate location of CPT performed by AMECFW for this study (2015)

Approximate location of borings performed by others

Approximate location of CPTperformed by others

Note: Some borings not shown

0 500 Feet

ELEVATION CONTOURS OF BOTTOM OF FILL
Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

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Date: 12/14/2015 | Project No. 6166150082.14.04

Figure 10

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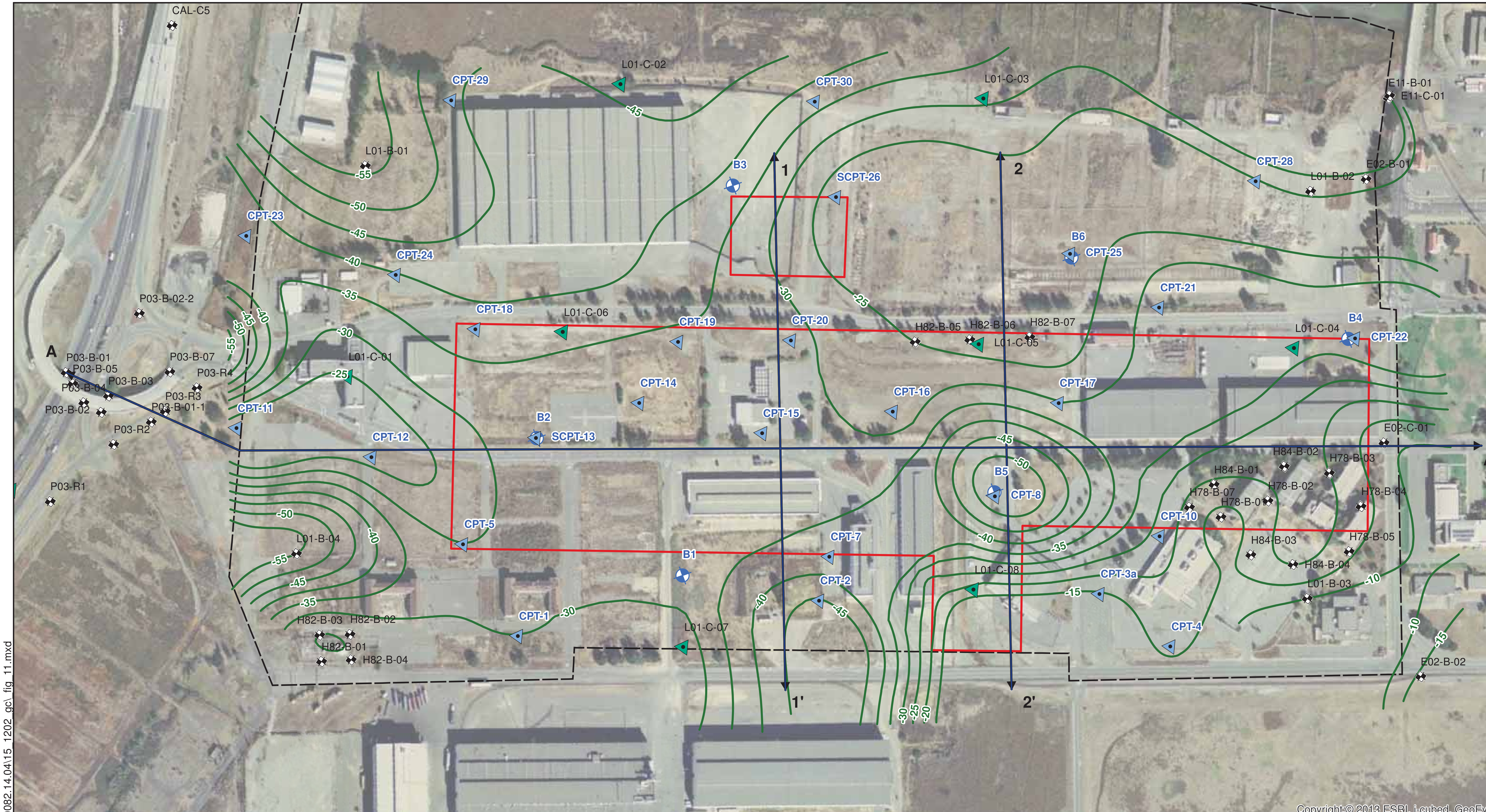

Explanation

Proposed Building Footprint

Site boundary

Bottom of BAY MUD

Elevation Datum: NAVD 88

Approximate location of borings by performed by AMECFW for this study (2015)

Approximate location of CPT performed by AMECFW for this study (2015);

Approximate location of borings performed by others

Approximate location of CPT performed by others

Note: Some borings not shown

CONTOURS OF BOTTOM OF BAY MUD
Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

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Figure 11

Date: 12/14/2015

Project No. 6166150082.14.04

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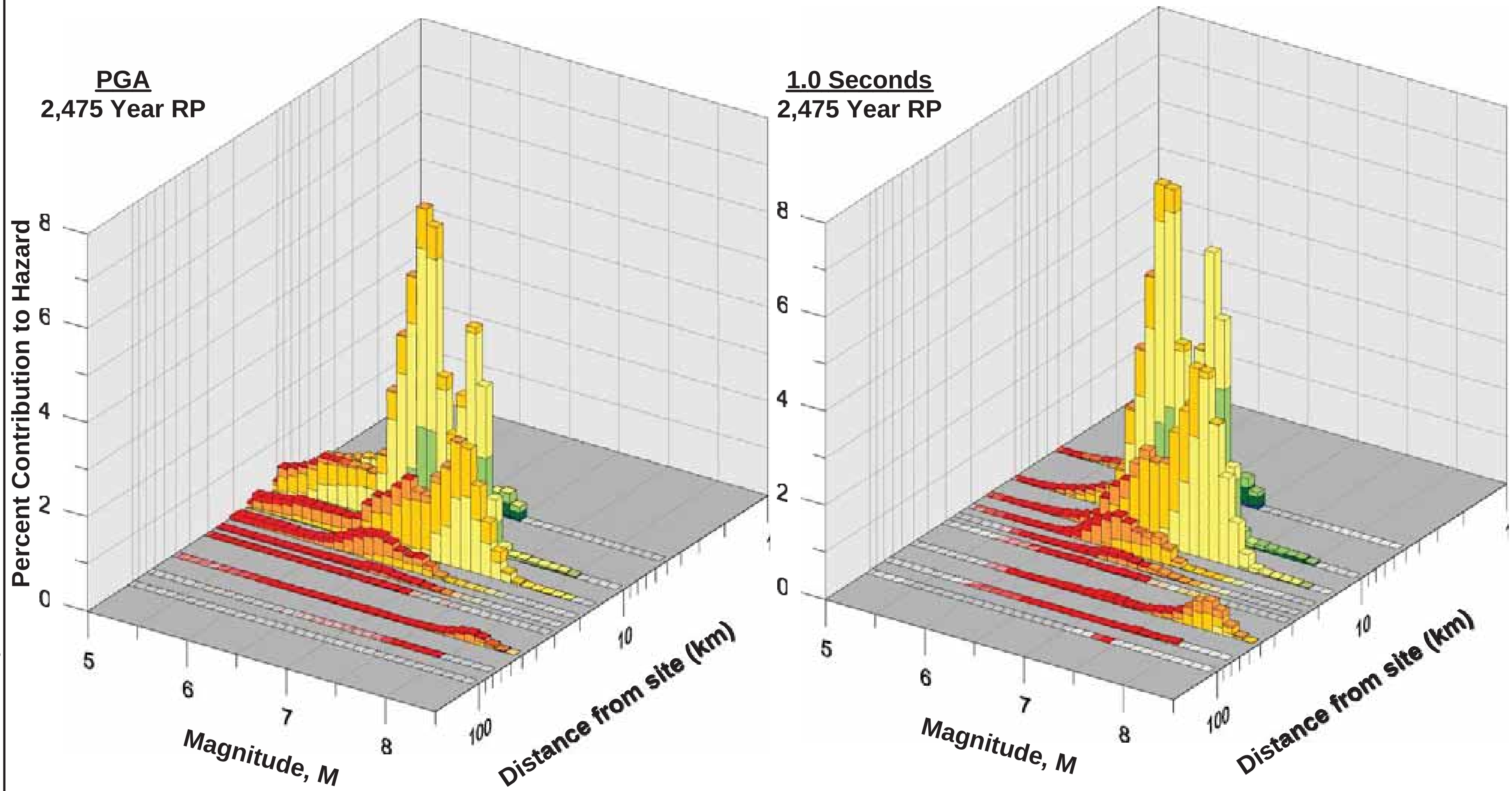

	Epsilon: 3 to 9		Epsilon: -0.5 to 0
	Epsilon: 2.5 to 3		Epsilon: -1 to -0.5
	Epsilon: 2 to 2.5		Epsilon: -1.5 to -1
	Epsilon: 1.5 to 2		Epsilon: -2 to -1.5
	Epsilon: 1 to 1.5		Epsilon: -2.5 to -2
	Epsilon: 0.5 to 1		Epsilon: -3 to -2.5
	Epsilon: 0 to 0.5		Epsilon: -9 to -3

DEAGGREGATION OF MEAN HAZARD FOR PEAK GROUND ACCELERATION AND 1.0-SEC SPECTRAL ACCELERATION FOR 2,475 YEAR RETURN PERIOD
Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

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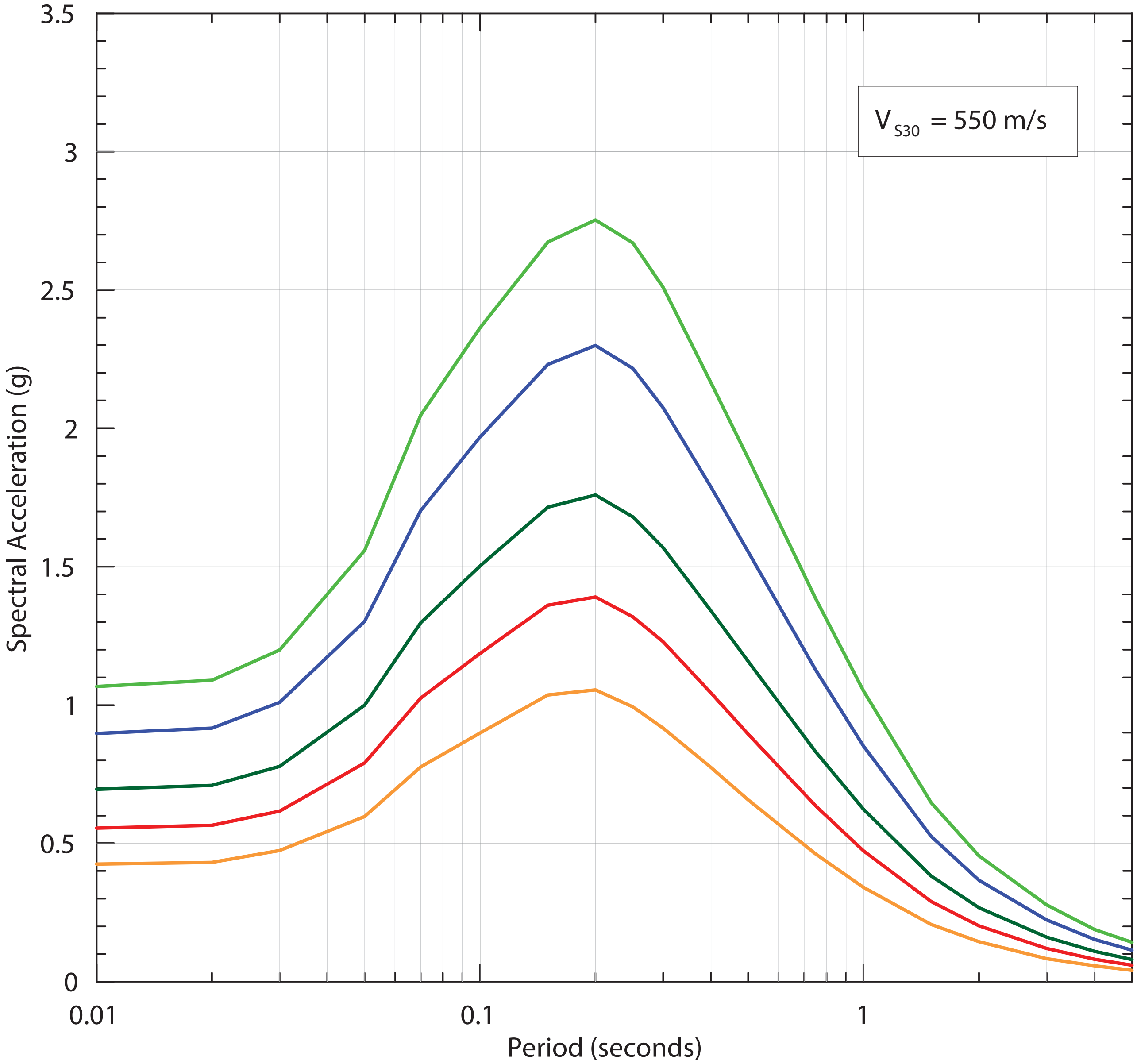
Date: Dec 2015 | Project No. 6166150082.14.04

Figure 12

I:\Project\6166150082\12000 CALC and EVAL\Mare Island\Seismic Hazard\figs



Explanation

- 225 Year RP - 20% PE in 50 Years
- 475 Year RP - 10% PE in 50 Years
- 975 Year RP - 5% PE in 50 Years
- 2475 Year RP - 2% PE in 50 Years
- 4975 Year RP - 1% PE in 50 Years

Notes:
1. Spectra are five-percent damped.
2. RP - Return period;
   PE - Probability of exceedance

MEAN UNIFORM HAZARD
RESPONSE SPECTRA
Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

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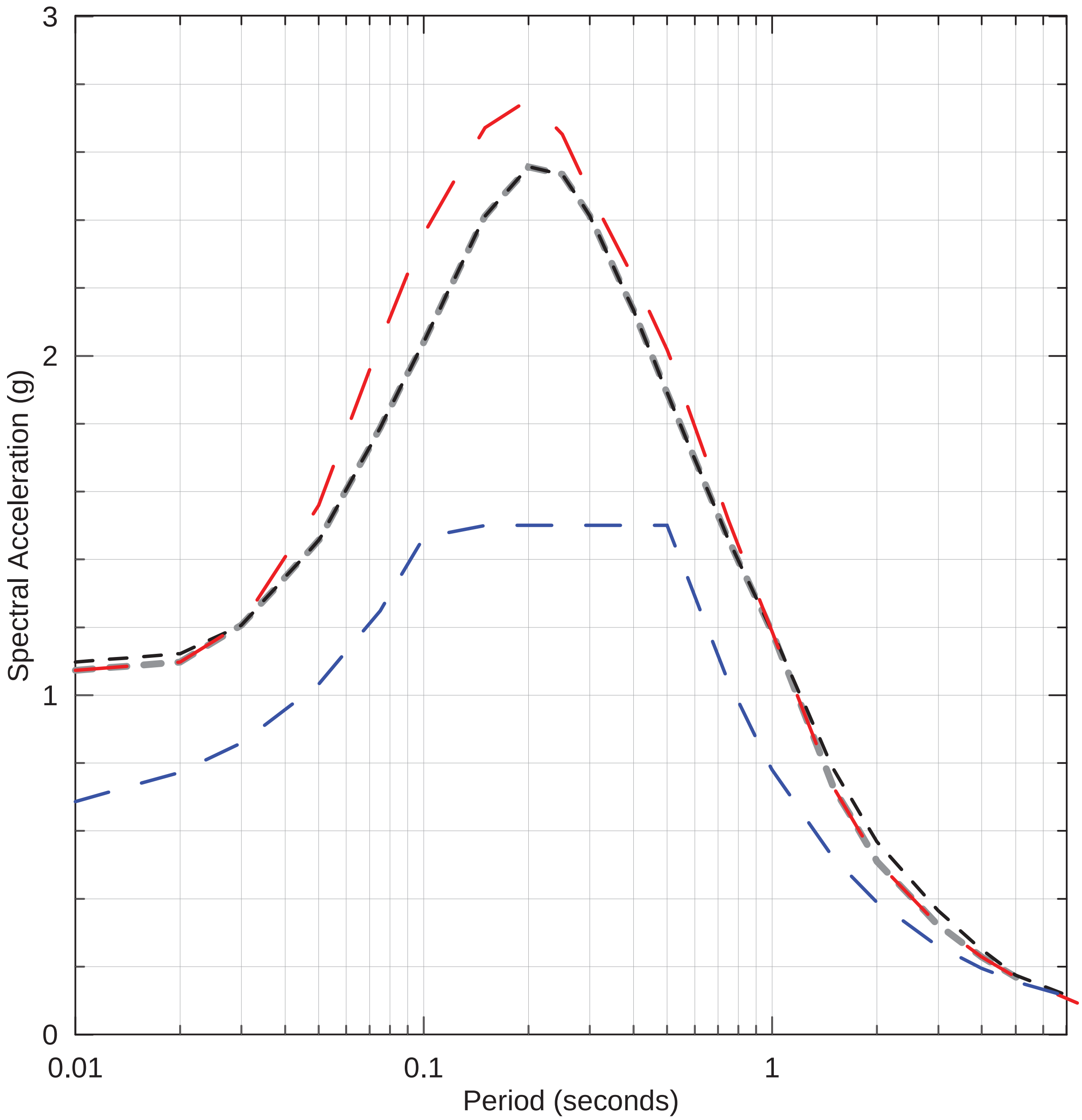
Date: Nov 2015	Project No. 6166150082	Figure 13

I:\Project\6166150082\12000 CALC and EVAL\Mare Island\Seismic Hazard\figs



Explanation

Risk Targeted 2% PE in 50 Years (Max Demand)
Deterministic Limit Spectra
84th Percentile Deterministic (Max Demand)
MCE_R

Notes
1. Spectra for Site Class C
V_{S30} = 550 m/s

COMPARISON OF PROBABILISTIC AND DETERMINISTIC RESPONSE SPECTRUMS FOR SELECTION OF MCE_R RESPONSE SPECTRUM
Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

Date: 12/11/2015	Project No. 6166150082.14.04	Figure 14

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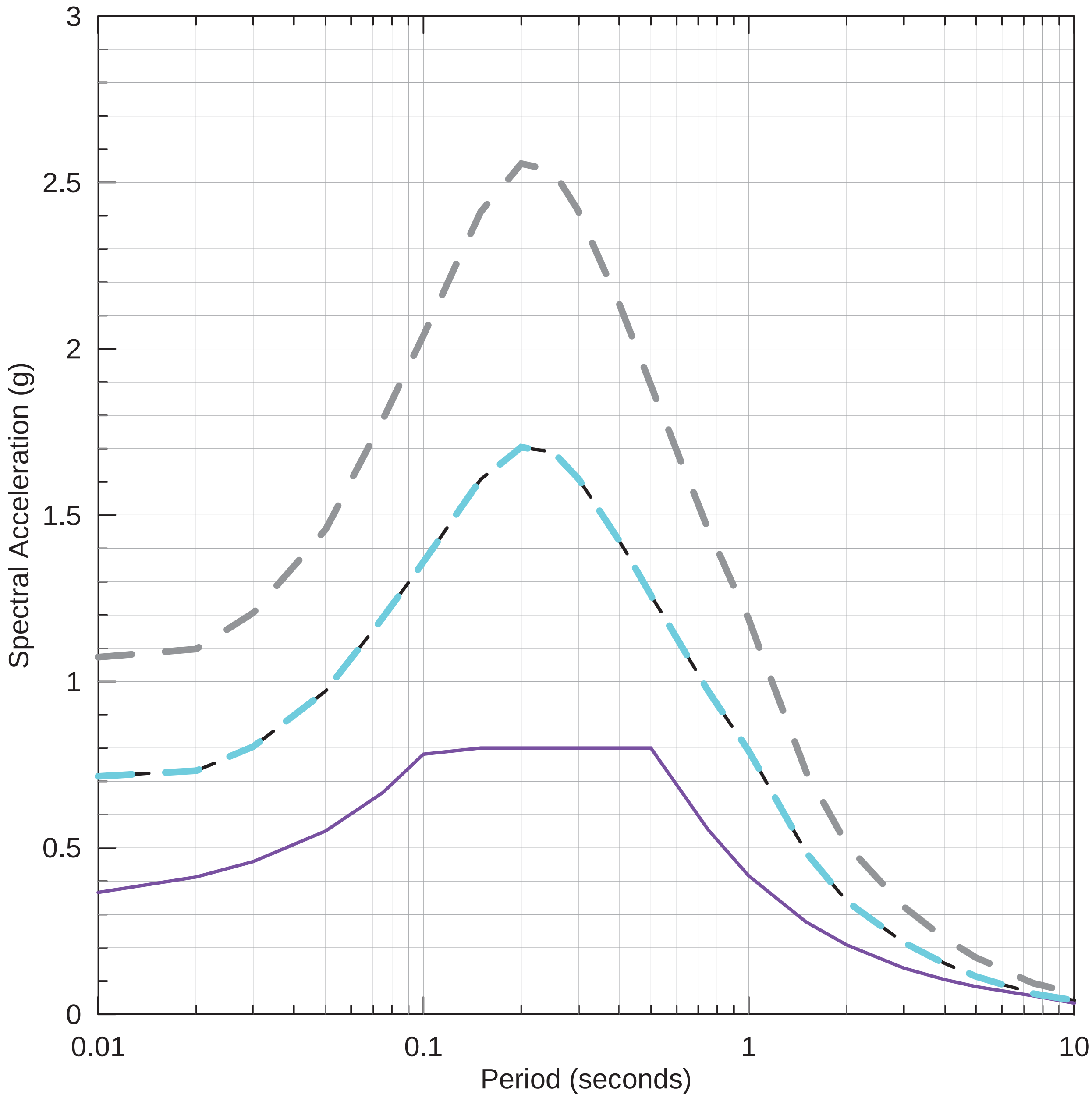

Explanation

- MCE$_R$ - Site Specific (Max Demand)
- 2/3 * MCE$_R$ - Site Specific (Max Demand)
- 0.8 * Design GPS
- Design - Site Specific (Max Demand)

Notes

1. Spectra for Site Class C
   V_{S30} = 550 m/s.

COMPARISON OF MCE AND DESIGN RESPONSE SPECTRUMS WITH LOWER LIMIT GENERAL PROCEDURE SPECTRUM
Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

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Date: 12/11/2015	Project No. 6166150082.14.04	Figure 15

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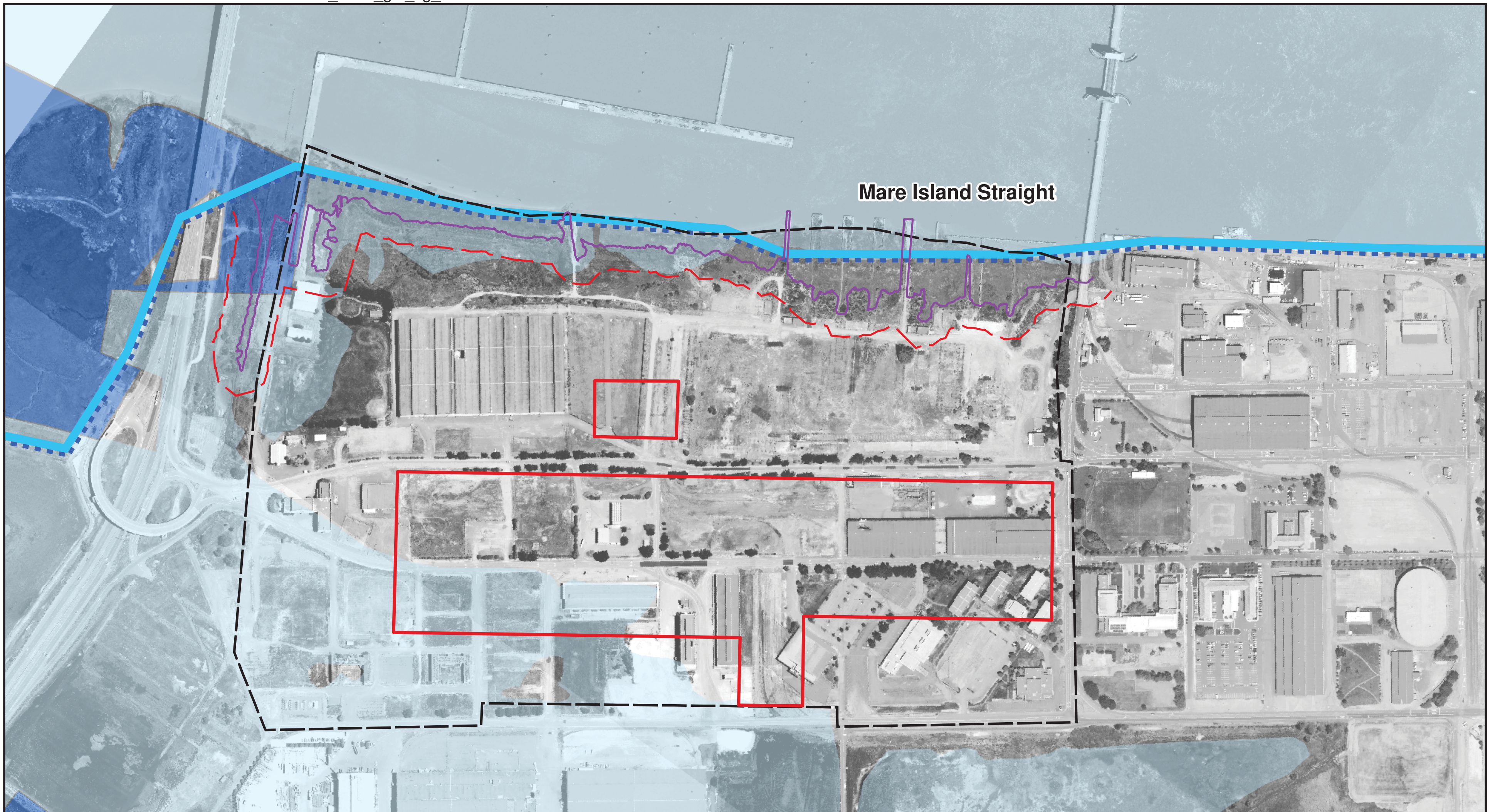

Explanation

Site boundary

Proposed Building Footprint

BCDC 100' SETBACK

BCDC LIMIT

Tsunami inundation boundary (CEMA, 2009)

FEMA Flood Zone (FEMA, 2010)

High Risk Zones

A 100 year flood zone

AE 100 year flood zone (Coastal)

0 500 Feet

SITE PLAN WITH FLOOD AND
INUNDATION/TSUNAMI ZONES
Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

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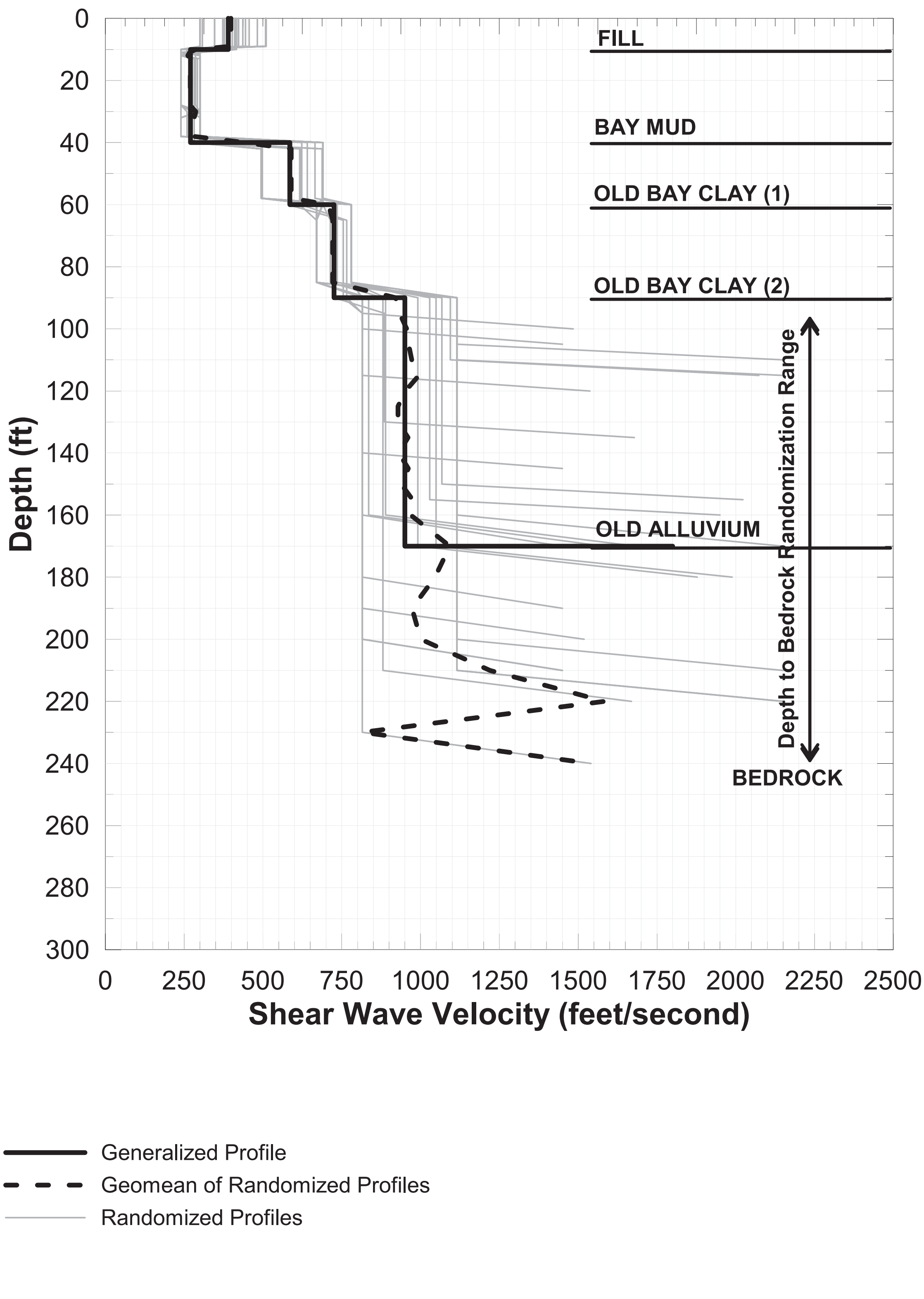
Date: 12/14/2015

Project No. 6166150082.14.04

Figure
16



RANDOMIZED V_S PROFILES
SITE RESPONSE MCE_G
Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

Date: Nov 2015 | Project No. 6166150082

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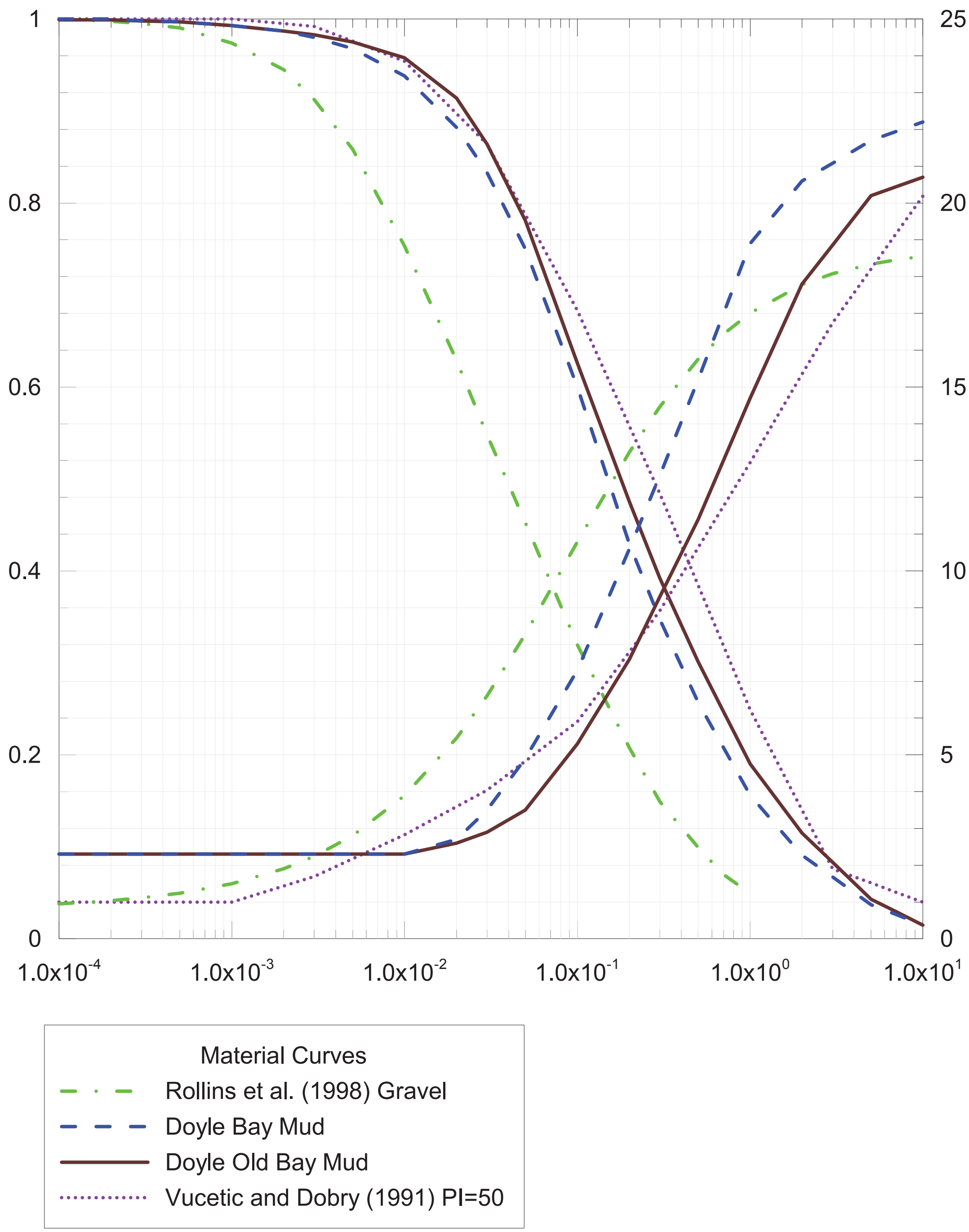
Figure 17

I:\Project\6166150082\12000 CALC and EVAL\Mare Island\Site Response



NOTE:
1. Half space modeled with linear 5% damping.
2. Large strain damping (>10%) modeled using damping at 10%.

NONLINEAR MATERIAL CURVES
SITE RESPONSE MCE_G
Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

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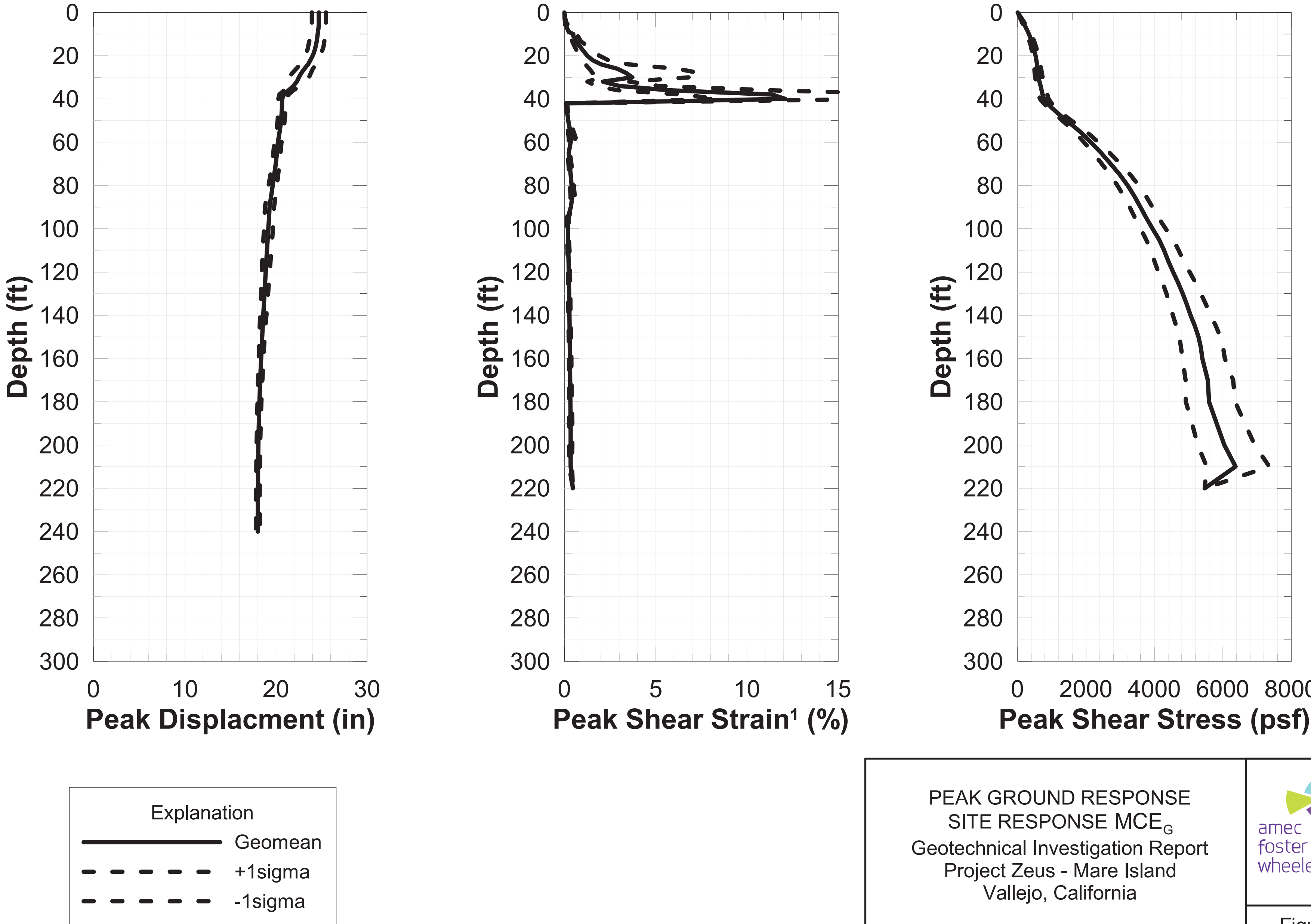
Date: Nov 2015 | Project No. 6166150082

Figure
18

I:\Project\6166150082\12000 CALC and EVAL\Mare Island\Site Response



PEAK GROUND RESPONSE
SITE RESPONSE MCE_G
Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

Date: 08/05/2015 | Project: 180630

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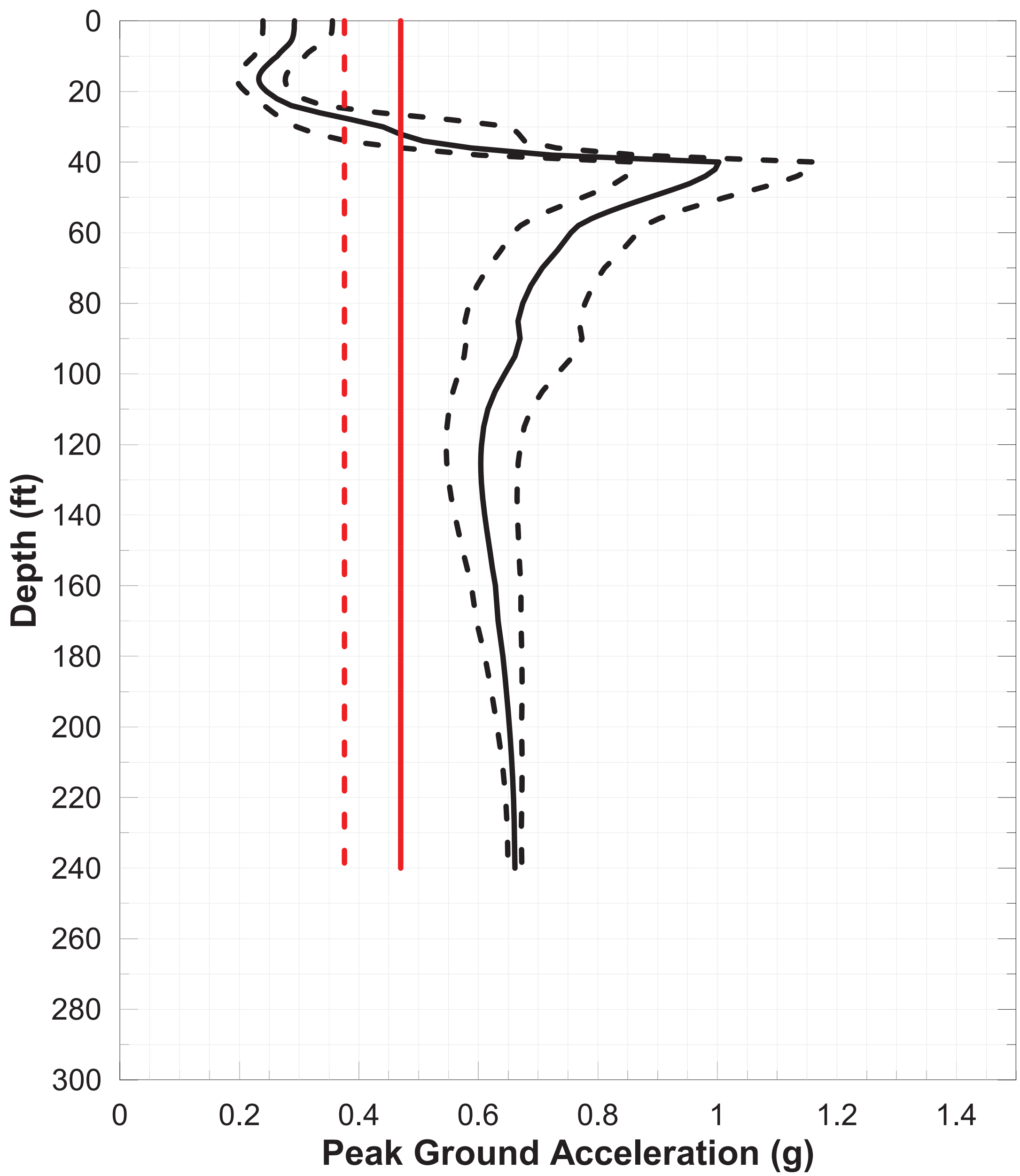
Figure
19



PGA_M Code (F_{PGA} * PGA_{map})
(ASCE 7-10 Section 11.8)

80% of PGA_M (Minimum)
(ASCE 7-10 Section 21.5)

Site Specific PGA_M - Geomean
(ASCE 7-10 Section 21)

Geamean+1sigma

Geomean-1sigma

PEAK GROUND ACCELERATION
SITE RESPONSE MCE_G
Geotechnical Investigation Report
Project Zeus - Mare Island
Vallejo, California

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Date: Nov 2015 | Project No. 6166150082

Figure
20

I:\Project\6166150082\12000 CALC and EVAL\Mare Island\Site Response

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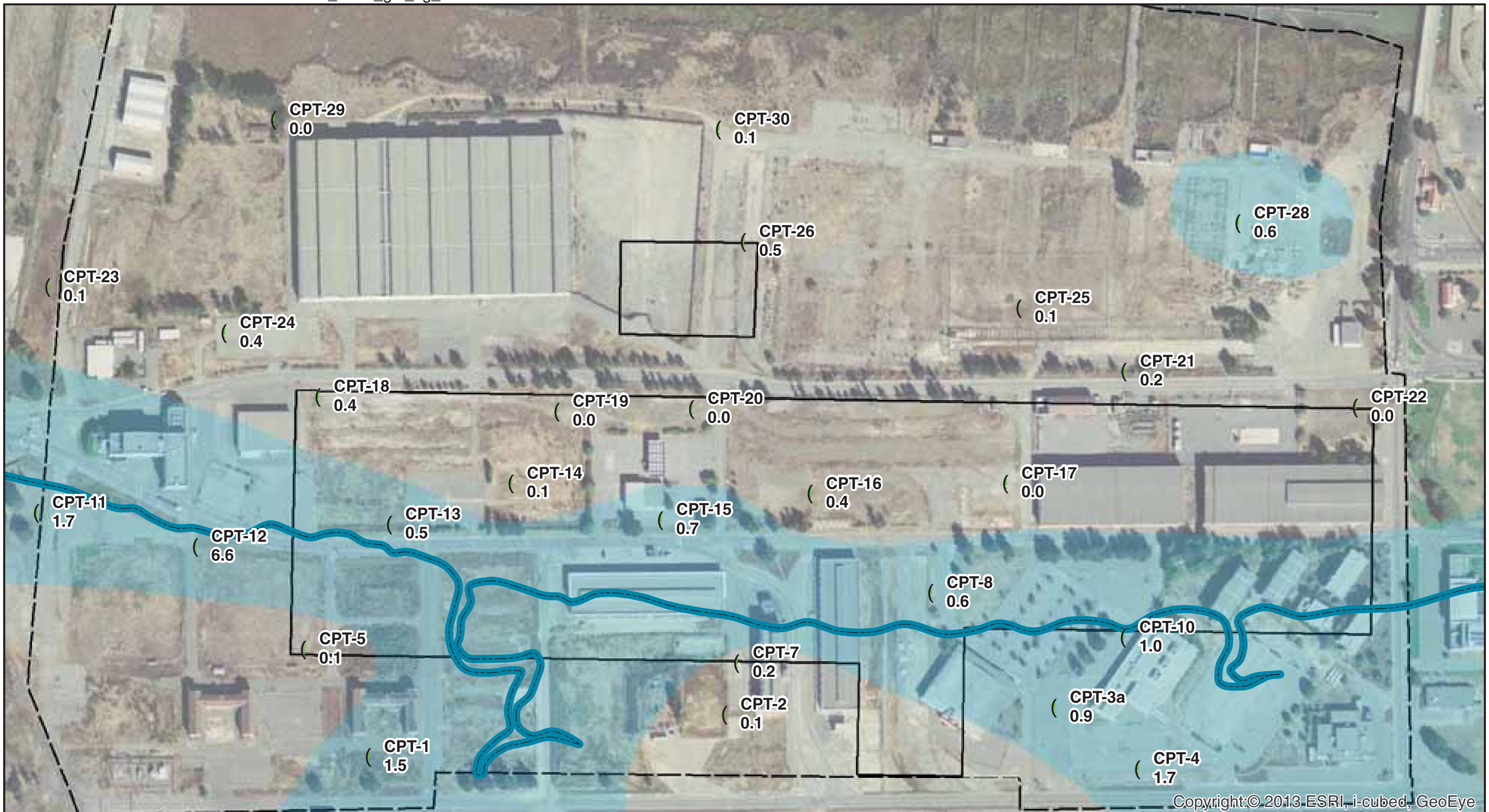

Explanation

Site boundary

Propsed Building Footprint

Liquefaction Analysis Results (in)

Historic Shoreline of 1875 (NFEC, 1980)

0 500 Feet

LIQUEFACTION RESULTS
Geotechnical Investigation Report
Project Zeus – Mare Island
Vallejo, California

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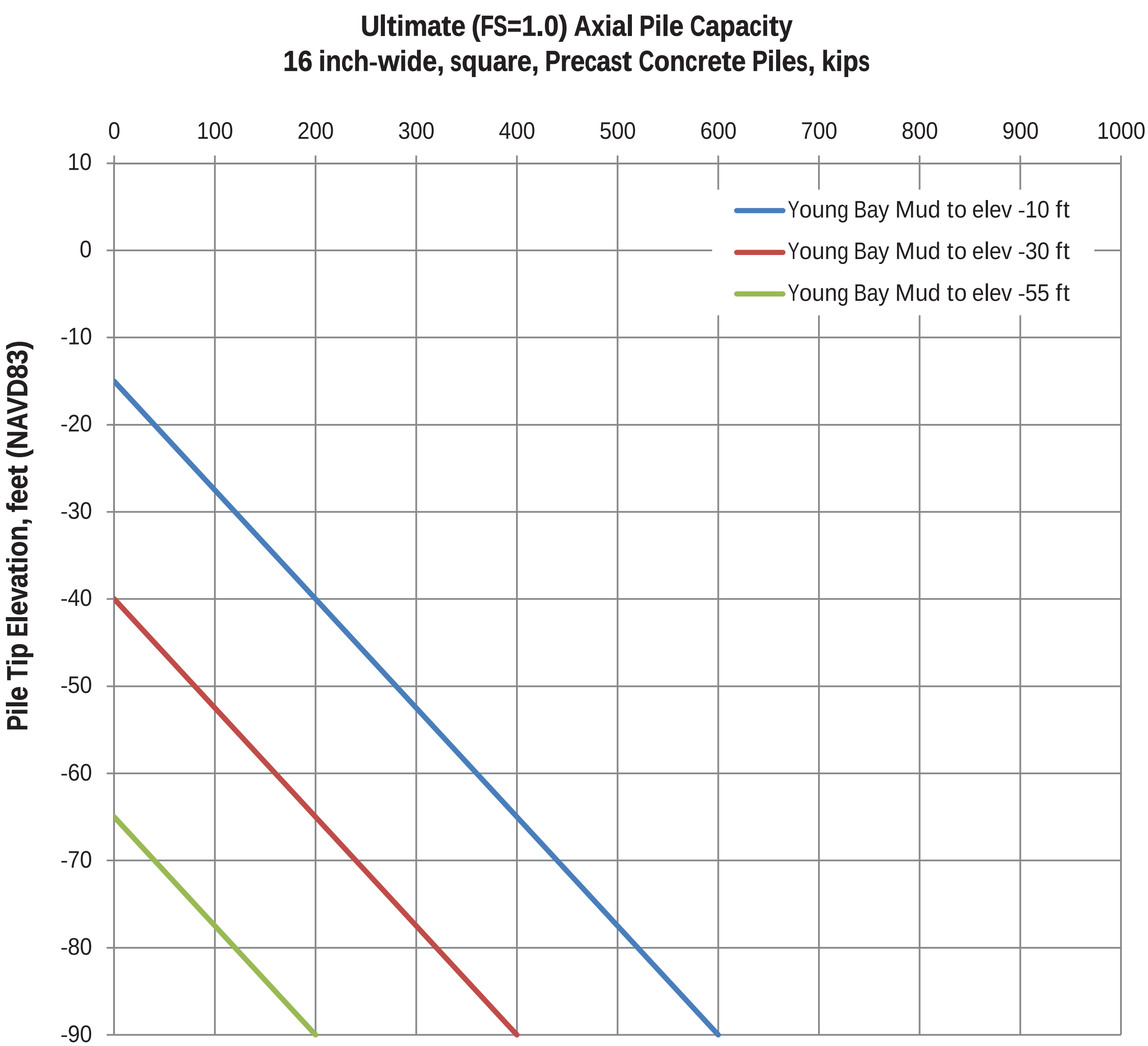
Figure
21

Date: 12/16/2015

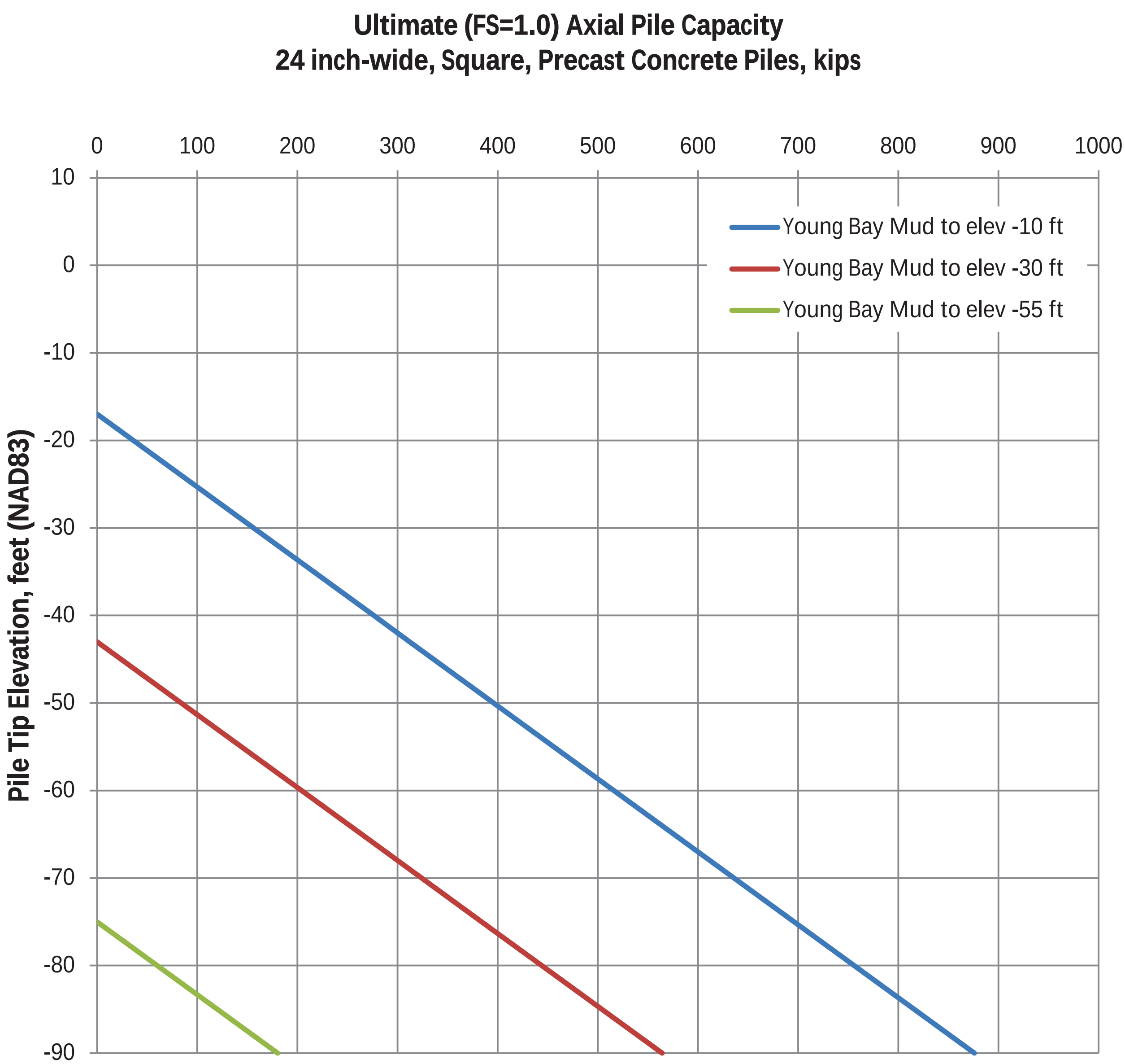
Project No. 6166150082.14.04



Notes:

1. The compressive capacities indicated are for sustained compressive loads. For transient loads, an increase of 33% is recommended.
2. The compressive capacities indicated assume a pile spacing of at least three diameters (center-to-center).
3. The compressive capacities above assume the thickness of Young Bay Mud as indicated. The actual thickness of Young Bay Mud varies across the site.
4. The compressive capacities indicated incorporate a downdrag component corresponding with potentially large settlements in Young Bay Mud. If the settlement is limited to a small value, these curves can be adjusted (increased) accordingly.

RECOMMENDED AXIAL PILE CAPACITY IN COMPRESSION 16 inch-wide Square Driven Precast Concrete Piles Project Zeus - Mare Island Vallejo, California		amec foster wheeler
Date: 12/11/2015	Project No. 6166150082.14.04	Figure **22**



Notes:

1. The compressive capacities indicated are for sustained compressive loads. For transient loads, an increase of 33% is recommended.
2. The compressive capacities indicated assume a pile spacing of at least three diameters (center-to-center).
3. The compressive capacities above assume the thickness of Young Bay Mud as indicated. The actual thickness of Young Bay Mud varies across the site.
4. The compressive capacities indicated incorporate a downdrag component corresponding with potentially large settlements in Young Bay Mud. If the settlement is limited to a small value, these curves can be adjusted (increased) accordingly.

RECOMMENDED AXIAL PILE CAPACITY IN COMPRESSION 24 inch-wide Square Driven Precast Concrete Piles Project Zeus - Mare Island Vallejo, California		amec foster wheeler
Date: 12/11/2015	Project No. 6166150082.14.04	Figure **23**

APPENDIX A

Field Explorations

APPENDIX A

FIELD EXPLORATIONS

Geotechnical Investigation Report
Project Zeus
Mare Island, Vallejo, California

Between October 5 and December 4, 2015, Amec Foster Wheeler performed a series of geotechnical explorations including Cone Penetration Test (CPT) probes, Seismic Cone Penetration Test (SCPT) probes, and soil borings to investigate and sample subsurface materials under the supervision of Mr. Chris Coutu and Mr. Alexander Wright. All subsurface sampling, field logging, and exploration work was completed in general accordance with ASTM D5434.

Soil borings were completed by Pitcher Drilling of East Palo Alto, California, using a Failing 1500 truck mounted drill rig using mud rotary and flight auger drilling methods. Soil sampling equipment included modified California samplers and the Shelby tube samplers. Sampling was performed in general accordance with ASTM D1586 and ASTM D1587. Field torvane and pocket penetrometer tests were performed on all recovered cohesive samples. Preliminary soil classifications were made visually in the field in general accordance with ASTM D2488. Soil colors were described using the Munsell Soil Color Chart. Final boring logs were developed from conditions recorded on the field logs and laboratory testing results.

Vane shear testing was subcontracted by Gregg Drilling of Martinez, California to Robert Y. Chew Geotechnical of Hayward, California. Vane shear testing was completed.at soil boring locations B-2 and B-6 in general accordance with ASTM D2573.

CPTs and SCPTs were completed by Gregg Drilling of Martinez, California, using an electrical resistivity cone penetrometer. CPT’s were performed from a 30-ton truck mounted rig. Specific information about Gregg Drilling’s electrical resistivity cone penetrometer and track mounted rig are included in this appendix.

Upon completion, all soil borings, CPTs and SCPTs were backfilled with lean cement grout in accordance with Solano County requirements. Soil cuttings and drilling mud generated during drilling were collected in drums and stored temporarily on-site

BORING LOGS

These boring logs depict subsurface information only at the locations and at the times the borings were performed. Soil and groundwater conditions at other locations may differ from those observed at these locations. The passage of time may result in changes in soil and groundwater conditions at these locations.

Table A-1 presents key terms used to describe the physical properties of the soils on the boring logs. The legend for the boring logs is presented as Figure A-0. Subsequently borings are presented by boring number and page number (i.e. boring 3 page 2 as A-3-2).

TABLE A-1

KEY TERMS USED TO DESCRIBE PHYSICAL PROPERTIES OF SOILS

SPT	Modified California Samplers		Density[1]
	51 mm (2 inch) ID	66 mm (2.5 inch) ID	
<4	<5	<7	very loose
4-10	5-13	7-17	Loose
10-30	13-40	17-50	medium dense
30-50	40-67	50-83	Dense
>50	>67	>83	very dense

Consistency	Identification Procedure	Approximate SPT N-value (blows/30 cm)	Approximate Shear Strength (psf)
very soft	squeezes between finger when hand is closed	0-2	less than 250
Soft	easily molded by fingers	2-4	250-500
medium stiff	molded by strong finger pressure	4-8	500-1000
Stiff	dented by strong finger pressure	8-15	1000-2000
very stiff	dented only slightly by finger pressure	15-30	2000-4000
Hard	dented only slightly by pencil point	30+	4000+

Moisture	Criteria
Dry	Absence of moisture, dusty, dry to touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

[1] Refers to in situ natural density of coarse-grained soils

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

AT	Liquid Limit(LL), Plastic Limit(PL), Plasticity Index(PI) (ASTM D 4318)
CL	Collapse Potential (ASTM D 5333)
CN	Consolidation (ASTM D 2435)
CR	Corrosion, Sulfates, Chlorides (CTM 643; CTM 417; CTM 422m)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (Modified ASTM D 3080)
EI	Expansion (ASTM D 4829)/UBC Expansion Index
LP	Point Load Index (ASTM D 5731)
MC	Moisture Content (ASTM D 2216)
MD	Moisture Density (ASTM D 7263b)
MP	Modified Procter (ASTM D 1557)
OC	Organic Content (ASTM D 2974)
PM	Permeability (CTM 220)
PP	Pocket Penetrometer
PR	Pressure Meter
RV	R-Value (CTM 301)
SG	Specific Gravity (AASHTO T 100)
SI	Particle Size Analysis (ASTM D 422)
SL	Shrinkage Limit (ASTM D 427)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166) Unconfined Compression - Rock (ASTM D 2938)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
VS	Vane Shear
WA	Fines Wash (ASTM D 1140) (Fines Content - FC)

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler
- Shelby Tube
- Piston Sampler
- Bag Sample
- Vane Shear Test
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Direct Push or Dynamic Cone
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (short-term)
- Static Water Level Reading (long-term)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

Project No.: 6166150082	Figure: A-0

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
A. Wright	**10-19-15**	**10-20-15**	**38° 6' 44.3235" , -122° 17' 12.0723"**	**B-1**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Pitcher Drilling	**Mare Island, Vallejo, CA**	**9.1 ft NAVD88**

DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Hand Auger/Rotary Wash	**Fraste Multidrill XL**	**3 7/8**

SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Bulk, Bag, Mod Cal (2.4" I.D.), Shelby (2.87" I.D.)	**Autohammer / 140 lbs / 30 in / 5 ft drillrod stickup**	**92%**

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Neat Cement Grout		**4.0 ft**	**Not Observed**	**111.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Remarks
	0	Gravelly Fat CLAY with Sand (CH) brown (10YR 4/3); dry; coarse to fine Gravel; coarse to fine Sand [FILL].	1-A							PI=28 LL=51 FC=60.2	AT, CR, SI, MP, RV	-Hand auger upper 5 ft using 4 in dia. hand auger and 8 in dia. core barrel
7.10	2											
	3	Gravelly Fat CLAY with Sand (CH) dark grayish brown (10YR 4/2); moist; coarse to fine Gravel; coarse to fine Sand [FILL].	1-B									
5.10	4		2									-Sudden loss of drilling fluid at 6 ft.
3.10	6	Gravelly Lean CLAY (CL) soft; very dark grayish brown (10YR 3/2); wet; coarse to fine Gravel; coarse Sand; some gravels look like asphalt concrete fragments [FILL].	3-4	33	0 0 0	0						
1.10	8		4	70	24 in - 50 psi 6 in - 450 psi	push	117.3	32.0		PI=23 LL=47 FC=60.7	AT, CN, WA	-Driller indicates gravelly at 7.5 ft.
-0.90	10											-Large gravel wedged in tip of sampler at about 10 ft (approx. same dia. as tube)
-2.90	12											
	13	Fat CLAY (CH) soft; dark greenish gray (10BG 4/1); wet [BAY MUD].										
-4.90	14		5	100	30 in - 0 psi	push			PP=NA TV=0.15			
-6.90	16											
-8.90	18											
-10.90	20											
-12.90	22											
-14.90	24											
	25											

(continued)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID **B-1**

Date: 12/14/15 Project No.: 6166150082

This log is part of the report prepared by Amec Foster Wheeler Environment & Infrastructure, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Figure: A-1-1

I:\PROJECT\6166150082\10000 BORING LOGS\BORING LOG DRAFT.GPJ 12/14/15

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Remarks
	25	Fat CLAY (CH) soft; dark greenish gray (10BG 4/1); wet [BAY MUD]. *(layer description continued from previous page)*	6	97	30 in - 0 psi	push	93.0	87.2	PP=NA TV=0.13	PI=35 LL=68	AT, CN	
-16.90	26											
	27											
-18.90	28											
	29											
-20.90	30		7	100	30 in - 0 psi	push			PP=0.05 TV=0.25			
	31											
-22.90	32											
	33											
-24.90	34											
	35											
-26.90	36											
	37											
-28.90	38											
	39											
-30.90	40		8	100	24 in - 0 psi 6 in - 150 psi	push			PP=0.20 TV=0.15 to 0.20			
	41											
-32.90	42	Fat CLAY (CH) stiff to very stiff; dark greenish gray (10BG 4/1) with slight dark yellowish brown (10YR 3/4) mottling; moist; silty, with shell fragments [OLD BAY CLAY].	9-3 9-4	100	5 9 11	20			PP=1.5			
	43											
-34.90	44											
	45	Fat CLAY (CH) medium stiff to stiff; dark yellowish brown (10YR 3/4) mottled with dark greenish gray (10BG 4/1); moist; fine Sand; sand in pockets and reddish brown in color [ALLUVIUM].										-Driller notes color change to yellowish brown at 45 ft.
-36.90	46											
	47											
-38.90	48											
	49											
-40.90	50		10		12 in - 400 psi 12 in - 725 psi 6 in - 800 psi	push	111.4	43.5	PP=0.75 to 1.00	PI=30 LL=55	AT, CN	
	51											
-42.90	52											
	53	Lean CLAY (CL) medium stiff to stiff; grayish brown (10YR 5/2); moist; fine Sand; sand in pockets and reddish brown in color [ALLUVIUM].	11-3 11-4	100	3 5 6	11	112.7, 115.7	43.3, 38.9	PP=0.50 to 0.75 TV=0.55	PI=21, 25 LL=43, 50 FC=94.0, 95.0	AT, MD, WA	
-44.90	54											
	55											

(continued)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID **B-1**

Date: 12/14/15 | Project No.: 6166150082

Figure: A-1-2

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method Casing Depth	Remarks
	55		Lean CLAY (CL) medium stiff to stiff; grayish brown (10YR 5/2); moist; fine Sand; sand in pockets and reddish brown in color [ALLUVIUM]. *(layer description continued from previous page)*												
-46.90	56														
	57														
-48.90	58														
	59		Lean CLAY (CL) very stiff; grayish brown (10YR 5/2); moist; fine Sand; oxide stained veins/streaking [ALLUVIUM].												-Driller indicates stiffer drilling
-50.90	60														
	61														
-52.90	62				12-3 12-4		7 11 16	27			PP=1.00 to 1.50 TV=0.6				
	63														
-54.90	64														
	65														
-56.90	66														
	67														
-58.90	68														
	69														
-60.90	70		-At 70 ft, abundant silt.		13-3 13-4		6 6 8	14			PP=1.25 to 1.50 TV=0.20				
	71														
-62.90	72														
	73														
-64.90	74														
	75		Silty Clayey SAND (SC-SM) medium dense; dark grayish brown (10YR 4/2); wet; fine Sand; low plasticity Fines [ALLUVIUM].												
-66.90	76														
	77														
-68.90	78														-Reddish brown medium to coarse sand returned in drill mud
	79														
-70.90	80				14-3 14-4		7 8 13	21							
	81														
-72.90	82		Fat CLAY (CH) very stiff to hard; grayish brown (10YR 5/2); moist; fine Sand; black staining and intermittent pockets of fine sand [ALLUVIUM].												
	83														
-74.90	84														
	85														

(continued)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID

B-1

Date: 12/14/15

Project No.: 6166150082

Figure:

A-1-3

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method Casing Depth	Remarks
	85		Fat CLAY (CH) very stiff to hard; grayish brown (10YR 5/2); moist; fine Sand; black staining and intermittent pockets of fine sand [ALLUVIUM]. *(layer description continued from previous page)*												
-76.90	86														
	87														
-78.90	88														
	89														
-80.90	90														
	91														
-82.90	92														
	93														
-84.90	94														
	95				15-3 15-4	100	8 9 12	21	117.1	36.6	PP=2.25+	FC=99.6	MD, WA		
-86.90	96														
	97														
-88.90	98														
	99														
-90.90	100														
	101														
-92.90	102														
	103		Sandy Fat CLAY (CH) stiff to hard; grayish brown (10YR 5/2); some fine sand and silt; oxide streaking [ALLUVIUM].												
-94.90	104														
	105														
-96.90	106														
	107														
-98.90	108														
	109														
-100.90	110				16-3 16-4	100	7 10 16	26	132.4	19.6	PP=1.75 to 2.25		MD		
	111														
-102.90	112		Bottom of borehole at 111.5 ft bgs. Backfilled with neat cement grout.												
	113		NOTES: 1. Shear strengths shown are provided in terms of undrained shear strength in TSF. 2. Sampling and soil descriptions in general accordacne with ASTM 5434.												
-104.90	114														
	115														

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID **B-1**

Date: 12/14/15

Project No.: 6166150082

LOGGED BY **A. Wright**	BEGIN DATE **10-22-15**	COMPLETION DATE **10-22-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **38° 6' 50.4541" , -122° 17' 10.9719"**	HOLE ID **B-2**
DRILLING CONTRACTOR **Pitcher Drilling**			BOREHOLE LOCATION (Offset, Station, Line) **Mare Island, Vallejo, CA**	SURFACE ELEVATION **9.9 ft NAVD88**
DRILLING METHOD **Hand Auger/Flight Auger/Vane Shear**			DRILL RIG **Fraste Multidrill XL**	BOREHOLE DIAMETER **5 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **Bulk, Vane Shear, Shelby (2.87" I.D.)**			SPT HAMMER TYPE **Autohammer / 140 lbs / 30 in / 5 ft drillrod stickup**	HAMMER EFFICIENCY, ERi **92%**
BOREHOLE BACKFILL AND COMPLETION **Neat Cement Grout and Bentonite**			GROUNDWATER READINGS — DURING DRILLING **5.0 ft** — AFTER DRILLING (DATE) **Not Observed**	TOTAL DEPTH OF BORING **35.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Remarks
	0	Lean CLAY with Sand (CL) yellowish brown (10YR 5/4); dry; coarse to fine, angular Gravel; coarse to fine Sand; low to medium plasticity Fines [FILL].	1-A							FC=78.3	SI, CR	-Hand auger upper 5 ft using 4 in dia. hand auger and 8 in dia. core barrel
7.90	2											
	2.5	Clayey SAND with Gravel (SC) yellowish brown (10YR 5/4); moist; coarse to fine Gravel; coarse to fine Sand; low to medium plasticity Fines; cobble sized concrete piece observed [FILL].										
5.90	4		1-B									
	5	Clayey GRAVEL with Sand (GC) very loose; yellowish brown (10YR 5/4); moist; coarse to fine, angular Gravel; coarse to fine Sand; low to medium plasticity Fines [FILL].	2-4	67	2 2 3	5				PI=17 LL=42 FC=40.7	AT, SI	-Switch to flight auger at 5 ft.
3.90	6											
1.90	8											
-0.10	10	Gravelly Fat CLAY (CH) soft; dark greenish gray (10BG 4/1); wet; coarse to fine, angular Gravel; coarse to fine Sand [BAY MUD].										-Change in color and consistency of cuttings
-2.10	12	Fat CLAY (CH) very soft to soft; dark greenish gray (10BG 4/1); wet; fine Sand; trace rootlets [BAY MUD].	3	40	30 in - 50 psi	push	93.8	80.7	TV=0.19		CN	
-4.10	14											-Switch to vane shear at 14.5 ft.
-6.10	16											
-8.10	18		4						VS=0.27 (0.10)			
-10.10	20											
-12.10	22											
	23		5						VS=0.35			
-14.10	24											
	25											

(continued)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID **B-2**

Date: 12/14/15 Project No.: 6166150082

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method	Casing Depth	Remarks
	25		Fat CLAY (CH) very soft to soft; dark greenish gray (10BG 4/1); wet; fine Sand; trace rootlets [BAY MUD]. *(layer description continued from previous page)*													
-16.10	26															
	27															
-18.10	28				6						VS=0.45					-Vane shear caught on gravel piece; reading may be invalid or artifically high.
	29															
-20.10	30				7						VS=0.38 (0.16)					
	31															
-22.10	32															
	33															
-24.10	34															
	35				8						VS=0.43 (0.10)					

Bottom of borehole at 35.5 ft bgs.
Backfilled with neat cement grout.

NOTES:
1. Shear strengths shown are provided in terms of undrained shear strength in TSF (remolded strengths shown in parenthesis).
2. Sampling and soil descriptions in general accordacne with ASTM 5434.
3. Unable to push tremmie pipe past 20 ft due to collapse of hole after removal of vane shear device.

ELEVATION (ft)	DEPTH (ft)
-26.10	36
	37
-28.10	38
	39
-30.10	40
	41
-32.10	42
	43
-34.10	44
	45
-36.10	46
	47
-38.10	48
	49
-40.10	50
	51
-42.10	52
	53
-44.10	54
	55

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID

B-2

Date: 12/14/15

Project No.: 6166150082

Figure: A-2-2

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
A. Wright	**10-19-15**	**10-19-15**	**38° 6' 49.9857" , -122° 16' 58.4842"**	**B-3**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Pitcher Drilling	**Mare Island, Vallejo, CA**	**10.1 ft NAVD88**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Hand Auger/Rotary Wash	**Fraste Multidrill XL**	**3 7/8**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Bulk, Bag, Mod Cal (2.4" I.D.), Shelby (2.87" I.D.)	**Autohammer / 140 lbs / 30 in / 5 ft drillrod stickup**	**92%**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING — AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Neat Cement Grout	**Not Observed due to drilling method**	**66.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
	0	Sandy Lean CLAY with Gravel (CL) brown (10YR 4/3); dry; coarse to fine, subangular Gravel; coarse to fine Sand [FILL].
	1	
8.10	2	
	3	-large fragment of asphalt at 2.5 ft (~6 in diameter, 2 to 3 inches thick).
6.10	4	Lean CLAY with Sand (CL) dark grayish brown (10YR 4/2); moist; trace Gravel; fine Sand [FILL].
	5	
4.10	6	
	7	Poorly Graded SAND (SP) loose to medium dense; strong brown (7.5YR 4/6); dry; medium to fine Sand; moderate cementation [FILL].
2.10	8	Clayey SAND with Gravel (SC) loose; yellowish brown (10YR 5/4) mottled with dark grayish brown (10YR 4/2); moist; fine Gravel; trace coarse to fine Sand; medium to high plasticity Fines [FILL].
	9	
0.10	10	Fat CLAY (CH) very soft; very dark greenish gray (10BG 3/1); moist to wet [BAY MUD].
	11	
-1.90	12	
	13	
-3.90	14	
	15	
-5.90	16	
	17	
-7.90	18	
	19	
-9.90	20	
	21	
-11.90	22	
	23	
-13.90	24	
	25	

Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests
0-A								CR, EI, MP
0-B								
1-3	44	3 24 10	34	123.0	25.3	DS=0.65	PI=17 LL=40 FC=71.4	AT, DS, WA
2-3 2-4	67	2 4 4	8			PP=1.00	PI=30 LL=59 FC=48.5	AT, WA
3	53	12 in - 0 psi 18 in - 50 psi	push	99.8	63.1	PP=0.50		CN
4	93	30 in - 0 psi	push	95.0	79.9	UU=0.31	PI=39 LL=73 FC=99.9	AT, UU, SI

Remarks

-Hand auger upper 5 ft using 4 in dia. hand auger and 8 in dia. core barrel.

-Stained soil observed; bagged sample PID = 14.5 ppm. Advanced casing to cut off stained layer prior to starting mud.
-Switched to rotary wash at 5 ft.
-Driller indicates change in stiffness at 7.25 ft.

(continued)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

Date: 12/14/15 | Project No.: 6166150082

HOLE ID **B-3**

Figure: A-3-1

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method	Casing Depth	Remarks
	25		Fat CLAY (CH) very soft; very dark greenish gray (10BG 3/1); moist to wet [BAY MUD]. *(layer description continued from previous page)*		5	20	28 in - 0 psi 2 in - 25 psi	push	91.6	84.6		PI=38 LL=75	AT, CN			
-15.90	26															
	27															
-17.90	28		-organic fibers observed at 27.5 ft.		6	100	30 in - 0 psi	push			PP=0.25					
	29															
-19.90	30															
	31															
-21.90	32															
	33															
-23.90	34															
	35															
-25.90	36															
	37															
-27.90	38															
	39															
-29.90	40															
	41															
-31.90	42															
	43															
-33.90	44															
	45															
-35.90	46															
	47															
-37.90	48		Fat CLAY (CH) very stiff; greenish gray (10BG 5/1); wet; trace fine Sand; oxide staining on sand [OLD BAY CLAY].													-Driller indicates stiffer drilling at 48 ft
	49															
-39.90	50															
	51															
-41.90	52															
	53				7	100	18 in - 150 psi 12 in - 725 psi	push	109.3	44.7	PP=1.00 UU=1.49		UU			
-43.90	54															
	55															

(continued)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID

B-3

Date: 12/14/15

Project No.: 6166150082

This log is part of the report prepared by Amec Foster Wheeler Environment & Infrastructure, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Figure:

A-3-2

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method Casing Depth	Remarks
	55		Fat CLAY (CH) medium stiff; greenish gray (10BG 5/1) to very dark greenish gray (10BG 3/1); moist; yellowish brown (10YR 5/4) oxide staining [OLD BAY CLAY]. *(layer description continued from previous page)* -at 55.5 ft, grades to no sand, mottled with yellowish brown (10YR 5/4) oxide staining.												
-45.90	56				8-3 8-4	83	3 5 8	13			PP=0.88				
	57														
-47.90	58														
	59														
-49.90	60														
	61														
-51.90	62														
	63														
-53.90	64														
	65				9-3 9-4	100	0 4 4	8	108.9	50.7	PP=0.50	PI=30 LL=55 FC=99.5	AT, MD, WA		
-55.90	66														
	67		Bottom of borehole at 66.5 ft bgs. Backfilled with neat cement grout.												
-57.90	68														
	69														
-59.90	70														
	71														
-61.90	72														
	73														
-63.90	74														
	75														
-65.90	76														
	77														
-67.90	78														
	79														
-69.90	80														
	81														
-71.90	82														
	83														
-73.90	84														
	85														

NOTES:
1. Shear strengths shown are provided in terms of undrained shear strength in TSF.
2. Sampling and soil descriptions in general accordacne with ASTM 5434.

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID

B-3

Date: 12/14/15

Project No.: 6166150082

Figure:

A-3-3

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
A. Wright	**10-21-15**	**10-21-15**	**38° 6' 31.6986" , -122° 16' 49.4949"**	**B-4**
DRILLING CONTRACTOR **Pitcher Drilling**			BOREHOLE LOCATION (Offset, Station, Line) **Mare Island, Vallejo, CA**	SURFACE ELEVATION **12.4 ft NAVD88**
DRILLING METHOD **Hand Auger/Rotary Wash**			DRILL RIG **Fraste Multidrill XL**	BOREHOLE DIAMETER **3 7/8**
SAMPLER TYPE(S) AND SIZE(S) (ID) **Bulk, Bag, Mod Cal (2.4" I.D.), Shelby (2.87" I.D.)**			SPT HAMMER TYPE **Autohammer / 140 lbs / 30 in / 5 ft drillrod stickup**	HAMMER EFFICIENCY, ERi **92%**
BOREHOLE BACKFILL AND COMPLETION **Neat Cement Grout**			GROUNDWATER READINGS — DURING DRILLING — AFTER DRILLING (DATE) **Not Observed due to drilling method**	TOTAL DEPTH OF BORING **96.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Remarks
	0	Clayey GRAVEL with Sand (GC) brown (10YR 4/3); dry; coarse to fine Gravel; coarse to fine Sand; low to medium plasticity Fines [FILL].	1-A							PI=14 LL=36 FC=43.6	AT, EI, MP, SI	-Hand auger upper 5 ft using 4 in dia. hand auger and 8 in dia. core barrel. -Encountered obstruction that smelled of sulfer when struck in initial hand cleared location at a depth of 1.5 ft. Moved boring approximately 5 ft to the north. -Switched to rotary wash.
10.40	2	Poorly Graded SAND (SP) grayish brown (10YR 5/2); moist; coarse to fine Sand [FILL].										
	3	Lean CLAY with Sand (CL) dark olive gray (5Y 3/2); moist; trace Gravel; fine Sand [FILL]. -grades to Sandy Lean Clay.	1-B									
8.40	4											
	5		2									
6.40	6	Fat CLAY (CH) very soft to medium stiff; dark gray (10YR 4/1); moist to wet [BAY MUD].	3-3 3-4		0 0 1	1	95.9	82.2	PP=1.00 to 1.25	FC=99.5	MD, WA	
	7											
4.40	8											
	9											
2.40	10		4	100	30 in - 0 psi	push	104.0	53.7	PP=0.75 TV=0.88	PI=41 LL=73	AT, CN	
	11											
0.40	12											
	13											
-1.60	14											
	15											
-3.60	16											
	17											
-5.60	18											
	19											
-7.60	20											
	21											
-9.60	22											
	23											
-11.60	24											
	25											

(continued)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID **B-4**

Date: 12/14/15 Project No.: 6166150082

Figure: A-4-1

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method Casing Depth	Remarks
	25		Fat CLAY (CH) very soft to medium stiff; dark gray (10YR 4/1); moist to wet [BAY MUD]. *(layer description continued from previous page)*		5	93	30 in - 0 psi	push			PP=0.25				
-13.60	26														
	27														
-15.60	28														
	29														
-17.60	30														
	31														
-19.60	32														
	33														
-21.60	34														
	35														
-23.60	36														
	37														
-25.60	38														
	39														
-27.60	40														
	41														
-29.60	42														
	43														-At 43 ft, shells in mud return.
-31.60	44														
	45														
-33.60	46		Fat CLAY (CH) stiff to hard; brown (10YR 4/3) with yellow (10YR 7/6) and dark red (10R 3/6) streaks/patches; moist; trace coarse to fine Sand [OLD BAY CLAY].		6-3 6-4	89	8 10 10	20	123.5	29.2	PP=2.00 to 2.25 TV=1.15	PI=42 LL=60 FC=98.8	AT, MD, WA		-Driller notes color change and stiffer at 45.5 ft.
	47														
-35.60	48				7	93	12 in - 250 psi 12 in - 750 psi refusal	push	113.9, 115.0	34.4, 34.8	PP=1.25 UU=1.79		CN, UU		
	49														
-37.60	50														
	51														
-39.60	52														
	53														
-41.60	54														
	55														

(continued)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID

B-4

Date: 12/14/15

Project No.: 6166150082

Figure:

A-4-2

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method Casing Depth	Remarks
	55		Fat CLAY (CH) stiff to hard; brown (10YR 4/3) with yellow (10YR 7/6) and dark red (10R 3/6) streaks/patches; moist; trace coarse to fine Sand [OLD BAY CLAY]. *(layer description continued from previous page)*		8-3 8-4	78	3 5 7	12			PP=0.88 to 1.25				
-43.60	56														
	57														
-45.60	58														
	59														
-47.60	60														
	61				9-4	89	5 7 11	18	121.3	30.7	PP=1.75	PI=32 LL=52 FC=96.7	AT, MD, WA		-At 61 ft, driller notes very stiff.
-49.60	62														
	63														
-51.60	64														
	65														
-53.60	66														
	67														
-55.60	68														
	69														
-57.60	70														
	71														
-59.60	72														
	73														
-61.60	74														-Driller notes more red/dark red soil in mud return and stiffer at 75 ft.
	75		Sandy Lean CLAY (CL) medium stiff; grayish brown (10YR 5/2) with strong brown (7.5YR 4/6) mottling; moist; fine Sand [ALLUVIUM].												
-63.60	76				10-3 10-4	100	7 7 10	17	117.8	36.7	PP=1.00 to 2.00	PI=27 LL=47 FC=97.5	AT, MD, SI		
	77		Lean CLAY (CL) stiff; grayish brown (10YR 5/2) with strong brown (7.5YR 4/6) mottling; moist; trace fine Sand; low to medium plasticity Fines [ALLUVIUM].												
-65.60	78														
	79														
-67.60	80														
	81														
-69.60	82														
	83														
-71.60	84														
	85														

(continued)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID

B-4

Date: 12/14/15

Project No.: 6166150082

This log is part of the report prepared by Amec Foster Wheeler Environment & Infrastructure, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Figure:

A-4-3

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method	Casing Depth	Remarks
	85		Lean CLAY (CL) stiff; grayish brown (10YR 5/2) with strong brown (7.5YR 4/6) mottling; moist; trace fine Sand; low to medium plasticity Fines [ALLUVIUM]. *(layer description continued from previous page)*													
-73.60	86															
	87															
-75.60	88				11-3 11-4	78	8 12 14	26	130.4	22.0	PP=1.50 to 1.75	FC=86.5	MD, WA			
	89															
-77.60	90															
	91															
-79.60	92															
	93															
-81.60	94															
	95				12-3 12-4	67	18 27 50/4"	77/10"	127.5	25.9	PP=1.75 to 2.00	PI=35 LL=61 FC=87.7	AT, MD, WA			
-83.60	96		SEDIMENTARY ROCK (SANDSTONE), coarse-grained, bluish gray with white, intensely to moderately weathered, dipping 25° [DECOMPOSED BEDROCK].													
	97															
-85.60	98		Bottom of borehole at 96.5 ft bgs. Backfilled with neat cement grout.													
	99		NOTES: 1. Shear strengths shown are provided in terms of undrained shear strength in TSF. 2. Sampling and soil descriptions in general accordacne with ASTM 5434.													
-87.60	100															
	101															
-89.60	102															
	103															
-91.60	104															
	105															
-93.60	106															
	107															
-95.60	108															
	109															
-97.60	110															
	111															
-99.60	112															
	113															
-101.60	114															
	115															

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID

B-4

Date: 12/14/15

Project No.: 6166150082

Figure:

A-4-4

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
A. Wright	**10-23-15**	**10-23-15**	**38° 6' 37.9182" , -122° 17' 2.3826"**	**B-5**
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Pitcher Drilling			**Mare Island, Vallejo, CA**	**11.0 ft NAVD88**
DRILLING METHOD			DRILL RIG	BOREHOLE DIAMETER
Hand Auger/Rotary Wash			**Fraste Multidrill XL**	**3 7/8**
SAMPLER TYPE(S) AND SIZE(S) (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Bulk, Bag, Mod Cal (2.4" I.D.), Shelby (2.87" I.D.)			**Autohammer / 140 lbs / 30 in / 5 ft drillrod stickup**	**92%**
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: **2.5 ft**; AFTER DRILLING (DATE): **Not Observed**	TOTAL DEPTH OF BORING
Neat Cement Grout				**96.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Remarks
	0	Silty Clayey GRAVEL with Sand (GC-GM) loose; dark yellowish brown (10YR 4/4); dry; coarse, angular Gravel; coarse to fine Sand; some fine gravel and cobbles [FILL].	1-A							PI=7 LL=28 FC=37.3	AT, SI	-Hand auger upper 5 ft using 4 in dia. hand auger and 8 in dia. core barrel -Caving in top 4 feet.
9.00	2											
	3	-At 3 ft, grades to a Silty Clayey SAND with Gravel.										
7.00	4	Lean CLAY with Sand (CL) soft; light olive brown (2.5Y 5/3); wet; fine Sand [FILL].	1-B									
5.00	5	Clayey GRAVEL with Sand (GC) soft; light olive brown (2.5Y 5/3); wet; coarse Gravel; coarse to fine Sand; low to medium plasticity Fines [FILL].										
3.00	8											-At 8 ft, switch to rotary wash. -At 8.5 ft, drill grinding.
	9	Fat CLAY (CH) very soft to medium stiff; very dark gray (N 3/); moist [BAY MUD].										
1.00	10		2	100	30 in - 0 psi	push	99.3, 97.4	64.5, 59.2	PP=0.13 to 0.38 UU=0.54		CN, UU	
	12.5	-At 12.5 ft, trace fine sand.										
	15		3	100	30 in - 0 psi	push	97.2	72.8	PP=0.13 to 0.25 TV=0.24		CN	
-13.00	24											
	25											

(continued)

amec foster wheeler	PROJECT ZEUS - MARE ISLAND Vallejo, California	HOLE ID **B-5**
	Date: 12/14/15 — Project No.: 6166150082	

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ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method Casing Depth	Remarks
	25		Fat CLAY (CH) very soft to medium stiff; very dark gray (N 3/); moist [BAY MUD]. *(layer description continued from previous page)*												
-15.00	26														
	27														
-17.00	28														
	29														
-19.00	30														
	31														
-21.00	32														
	33														
-23.00	34														
	35														
-25.00	36														
	37														
-27.00	38														
	39														
-29.00	40				4	100	30 in - 0 psi	push			PP=0.25 TV=0.24				
	41														
-31.00	42														
	43														
-33.00	44														
	45														
-35.00	46														
	47														
-37.00	48														
	49														
-39.00	50														
	51														
-41.00	52														
	53														
-43.00	54														
	55														

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method Casing Depth	Remarks
	55		Fat CLAY (CH) very soft to medium stiff; very dark gray (N 3/); moist [BAY MUD]. *(layer description continued from previous page)*												
-45.00	56														
	57														
-47.00	58														
	59														
-49.00	60														
	61														
-51.00	62														
	63				5	100	12 in - 250 psi 12 in - 750 psi refusal	push			PP=0.75				
-53.00	64		Sandy Lean CLAY (CL) medium stiff; brown (10YR 5/3); moist; coarse to fine Sand [OLD BAY CLAY].												
	65														
-55.00	66														
	67														
-57.00	68														
	69														
-59.00	70														
	71														
-61.00	72														
	73														-At 72.5 ft, driller notes whiter material in mud return.
-63.00	74														
	75		-At 75 ft, becomes very stiff to hard with dark red veins.		6-3 6-4	78	8 13 18	31	134.3	17.8	PP=1.75 to 2.25+	PI=13 LL=27	AT, MD		
-65.00	76														
	77														
-67.00	78														
	79														
-69.00	80		Clayey SAND (SC) medium dense to dense; dark yellowish brown (10YR 3/4); moist; fine Sand; low to medium plasticity Fines; some dark red banding [ALLUVIUM].												-Boundary assumed from CPT 3a
	81														
-71.00	82														
	83														
-73.00	84														
	85														

(continued)

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

Date: 12/14/15

Project No.: 6166150082

HOLE ID

B-5

Figure: A-5-3

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method / Casing Depth	Remarks
	85		Clayey SAND (SC) medium dense to dense; dark yellowish brown (10YR 3/4); moist; fine Sand; low to medium plasticity Fines; some dark red banding [ALLUVIUM]. *(layer description continued from previous page)*												
-75.00	86														
	87														
-77.00	88														
	89														
-79.00	90														
	91														
-81.00	92														
	93														
-83.00	94														
	95				7-3 7-4	72	7 16 29	45	118.6	35.4		FC=43.2	MD, WA		
-85.00	96														
	97		Bottom of borehole at 96.5 ft bgs. Backfilled with neat cement grout.												
-87.00	98		NOTES:												
	99		1. Shear strengths shown are provided in terms of undrained shear strength in TSF.												
-89.00	100		2. Sampling and soil descriptions in general accordacne with ASTM 5434.												
	101														
-91.00	102														
	103														
-93.00	104														
	105														
-95.00	106														
	107														
-97.00	108														
	109														
-99.00	110														
	111														
-101.00	112														
	113														
-103.00	114														
	115														

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PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID

B-5

Date: 12/14/15

Project No.: 6166150082

Figure:

A-5-4

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
A. Wright	**10-22-15**	**10-22-15**	**38° 6' 40.1452" , -122° 16' 53.1442"**	**B-6**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Pitcher Drilling	**Mare Island, Vallejo, CA**	**9.9 ft NAVD88**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Hand Auger/Flight Auger/Vane Shear	**Fraste Multidrill XL**	**3 7/8**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Bulk, Vane Shear	**Autohammer / 140 lbs / 30 in / 5 ft drillrod stickup**	**92%**

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Neat Cement Grout and Bentonite		**8.0 ft**	**Not Observed**	**26.0 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Remarks
	0	Clayey GRAVEL with Sand (GC) dark brown (10YR 3/3); dry; coarse to fine, angular Gravel; coarse to fine Sand; low plasticity Fines [FILL].	1-A						PI=12 LL=33 FC=19.6	AT, CR, SI, RV	-Hand auger upper 5 ft using 4 in dia. hand auger and 8 in dia. core barrel
5.90	4	Sandy Lean CLAY (CL) medium stiff; dark greenish gray (10BG 4/1); moist; fine Sand [FILL].	1-B								
3.90	5–6		2-4	6 6 7	13	126.0	21.8	DS=1.604		DS	-At 5 ft, switched to flight auger.
	7	Poorly Graded SAND (SP) dense; dark greenish gray (10BG 4/1); moist; fine Sand; weak to moderate cementation [FILL].									
	9	Sandy Fat CLAY (CH) soft; dark greenish gray (10BG 4/1); moist; trace fine Gravel; coarse to fine Sand [FILL]. -At 10 ft, encountered wood fragment..									
-2.10	12	Fat CLAY (CH) very soft to soft; dark greenish gray (10BG 4/1); moist [BAY MUD].									-Driller notes gravelly drilling to about 12 ft.
	14.5										-At 14.5 ft, switched to vane shear.
-8.10	18		3					VS=0.27 (0.11)			
	23		4					VS=0.24			

Elevation scale: 7.90 (2), 5.90 (4), 3.90 (6), 1.90 (8), -0.10 (10), -2.10 (12), -4.10 (14), -6.10 (16), -8.10 (18), -10.10 (20), -12.10 (22), -14.10 (24); depth 0–25 ft.

(continued)

PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID **B-6**

Date: 12/14/15 | Project No.: 6166150082

Figure: A-6-1

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Recovery (%)	Blows per 6 in.	Blows per foot	Total Unit Weight (pcf)	Moisture Content (%)	Shear Strength (tsf)	PI, LL, and FC	Lab Tests	Drilling Method	Casing Depth	Remarks
	25															
-16.10	26				5						VS=0.38 (0.16)					

Bottom of borehole at 26.0 ft bgs.
Backfilled with neat cement grout.

NOTES:
1. Shear strengths shown are provided in terms of undraiend shear strength in TSF (remolded strengths shown in parenthesis).
2. Sampling and soil descriptions in general accordacne with ASTM 5434.
3. Unable to push tremmie pipe past 16 ft due to collapse of hole after removal of vane shear device.

ELEVATION (ft)	DEPTH (ft)
	27
-18.10	28
	29
-20.10	30
	31
-22.10	32
	33
-24.10	34
	35
-26.10	36
	37
-28.10	38
	39
-30.10	40
	41
-32.10	42
	43
-34.10	44
	45
-36.10	46
	47
-38.10	48
	49
-40.10	50
	51
-42.10	52
	53
-44.10	54
	55

amec foster wheeler

PROJECT ZEUS - MARE ISLAND

Vallejo, California

HOLE ID

B-6

Date: 12/14/15

Project No.: 6166150082

Figure:

A-6-2

VANE SHEAR TESTING

Vane shear testing was subcontracted by Gregg Drilling of Martinez, California to Robert Y. Chew Geotechnical of Hayward, California. Vane shear testing was completed.at soil boring locations B-2 and B-6 in general accordance with ASTM D2573 at depths as shown on the boring logs. Results of the vane shear testing including both peak and remolded shear strengths.

October 30, 2015

AMEC

Subject: CPT Site Investigation
Project Zeus
Vallejo, California

Dear Project Manager:

The following report presents the results of GREGG Drilling & Testing's Field Vane Shear Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☐
2	Pore Pressure Dissipation Tests	(PPD)	☐
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	Resistivity Cone Penetration Tests	(RCPTU)	☐
5	UVOST Laser Induced Fluorescence	(UVOST)	☐
6	Groundwater Sampling	(GWS)	☐
7	Soil Sampling	(SS)	☐
8	Vapor Sampling	(VS)	☐
9	Vane Shear Testing	(VST)	☒
10	SPT Energy Calibration	(SPTE)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (562) 427-6899.

Sincerely,
GREGG Drilling & Testing, Inc. / Pitcher Drilling Company

Peter Robertson
Technical Operations

Field Vane Shear Test Summary

-Table 1-

FVST Identification	Date	Test Depths (Feet)	Comments
B-2	10/22/15	18.0, 23.0, 28.0, 30.0, 35.0	
B-6	10/22/15	17.5, 22.5, 25.5	

Bibliography

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Mayne, P.W., "NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization", available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp.107=112.

Richards, Adrian F. (Editor), "Vane Shear Testing in Soils", The International Symposium on Laboratory and Field Vane Shear Strength Testing, January 1987.

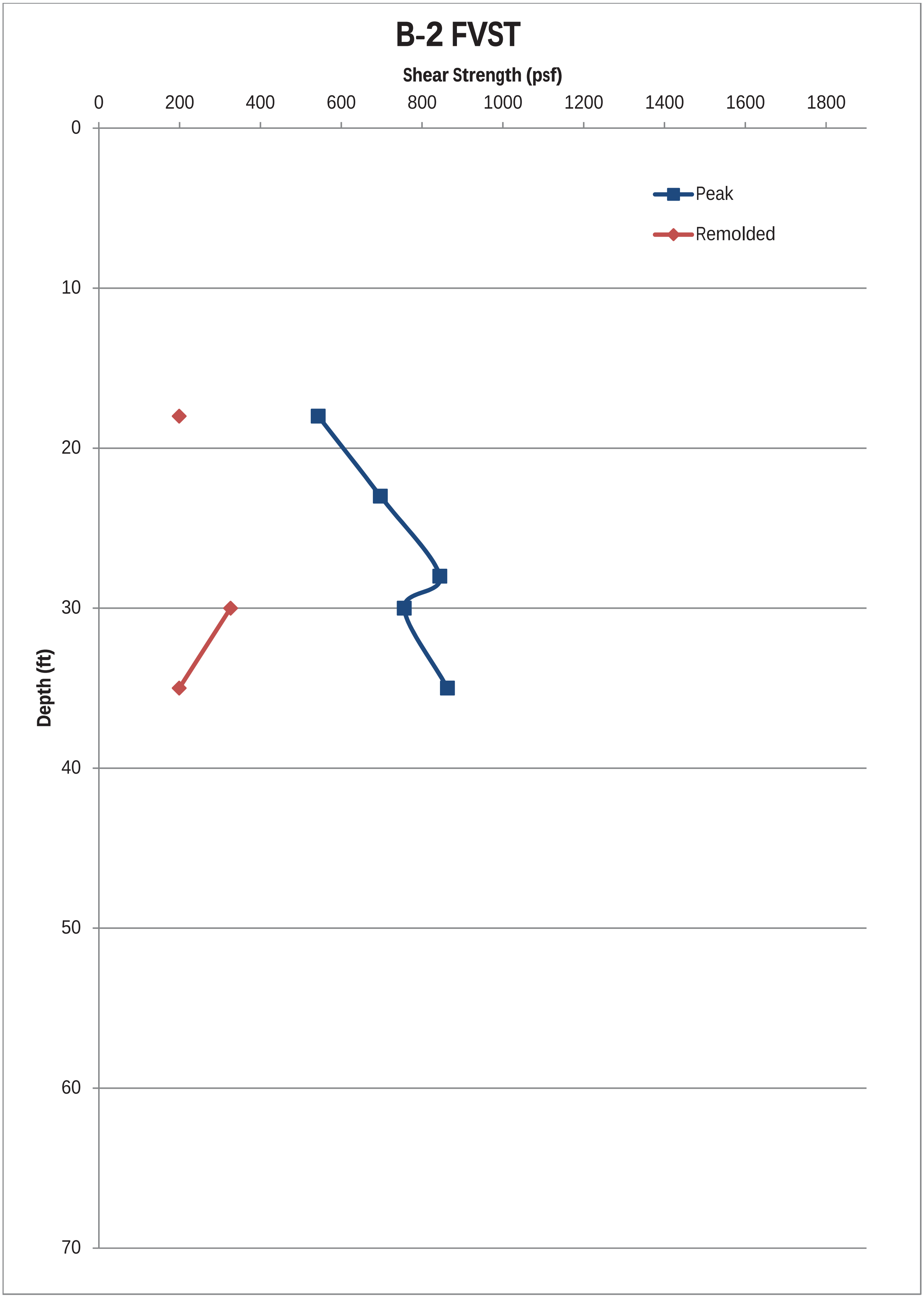
Chandler, R.J., "The In-Situ Measurement of the Undrained Shear Strength of Clays Using the Field Vane," Vane Shear Testing in Soils: Field and Laboratory Studies, ASTM STP 1014, A.F. Richards, Ed., American Society for Testing and Materials, Philadelphia, 1988, pp. 13-44.

Copies of ASTM Standards are available through www.astm.org

CLIENT	AMEC (Foster Wheeler)
SITE	Vallejo, CA
LOCATION	B-2
VANE TYPE	Geonor H-10
VANE DIAMETER, d (mm)	55
VANE LENGTH, l (mm)	110

GREGG

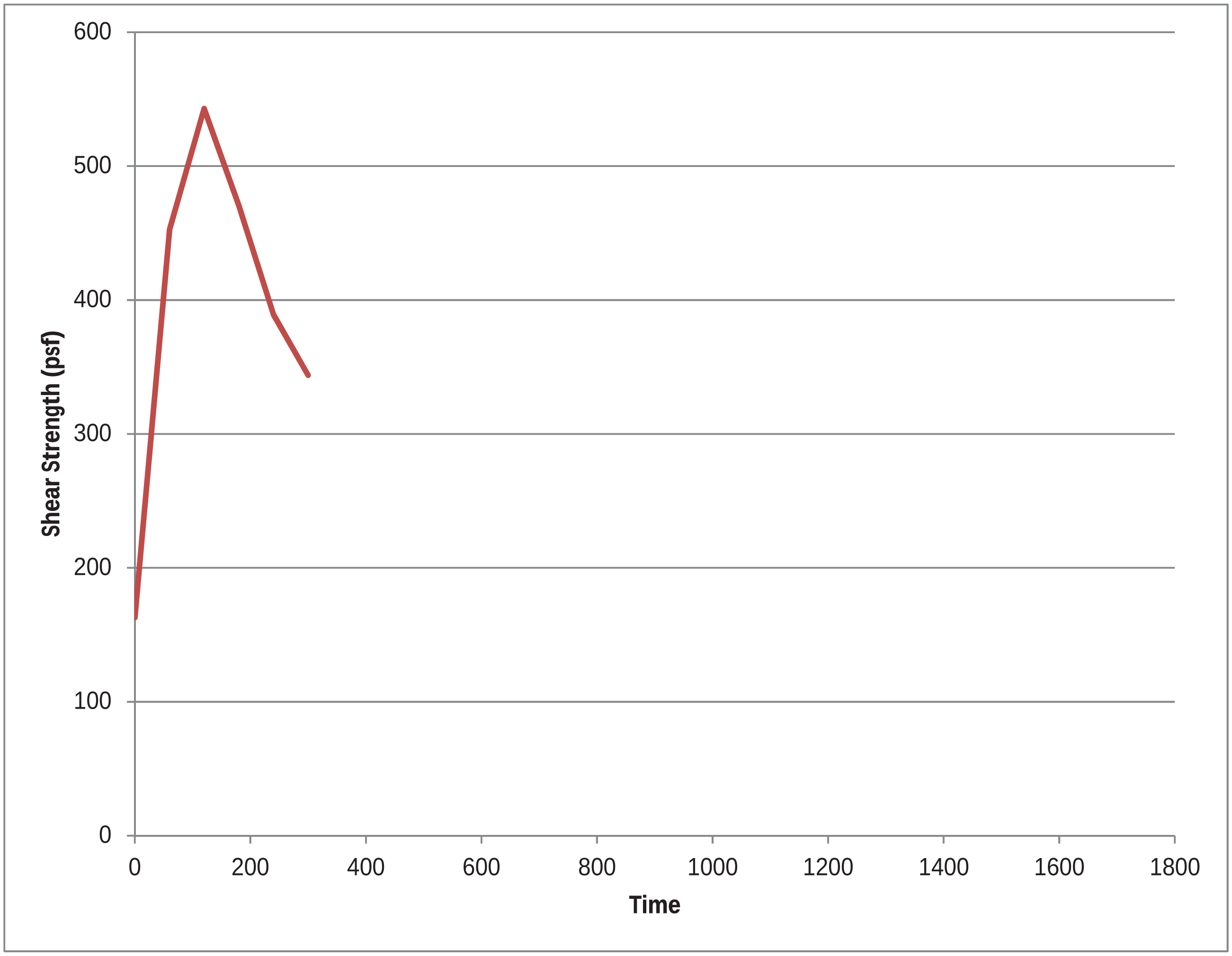
				REMOLDED		
DEPTH (m)	DEPTH (ft)	SHEAR STRENGTH (KPa)	SHEAR STRENGTH (psf)	SHEAR STRENGTH (KPa)	SHEAR STRENGTH (psf)	SENSITIVITY
5.49	18.00	25.99	543.00	9.50	199.00	2.70
7.01	23.00	33.37	697.00			
8.54	28.00	40.40	844.00			
9.15	30.00	36.19	756.00	15.60	326.00	2.30
10.67	35.00	41.31	863.00	9.50	199.00	4.30



B-2 FVST
Shear Strength (psf)
0
200
400
600
800
1000
1200
1400
1600
1800
Depth (ft)
0
10
20
30
40
50
60
70
Peak
Remolded

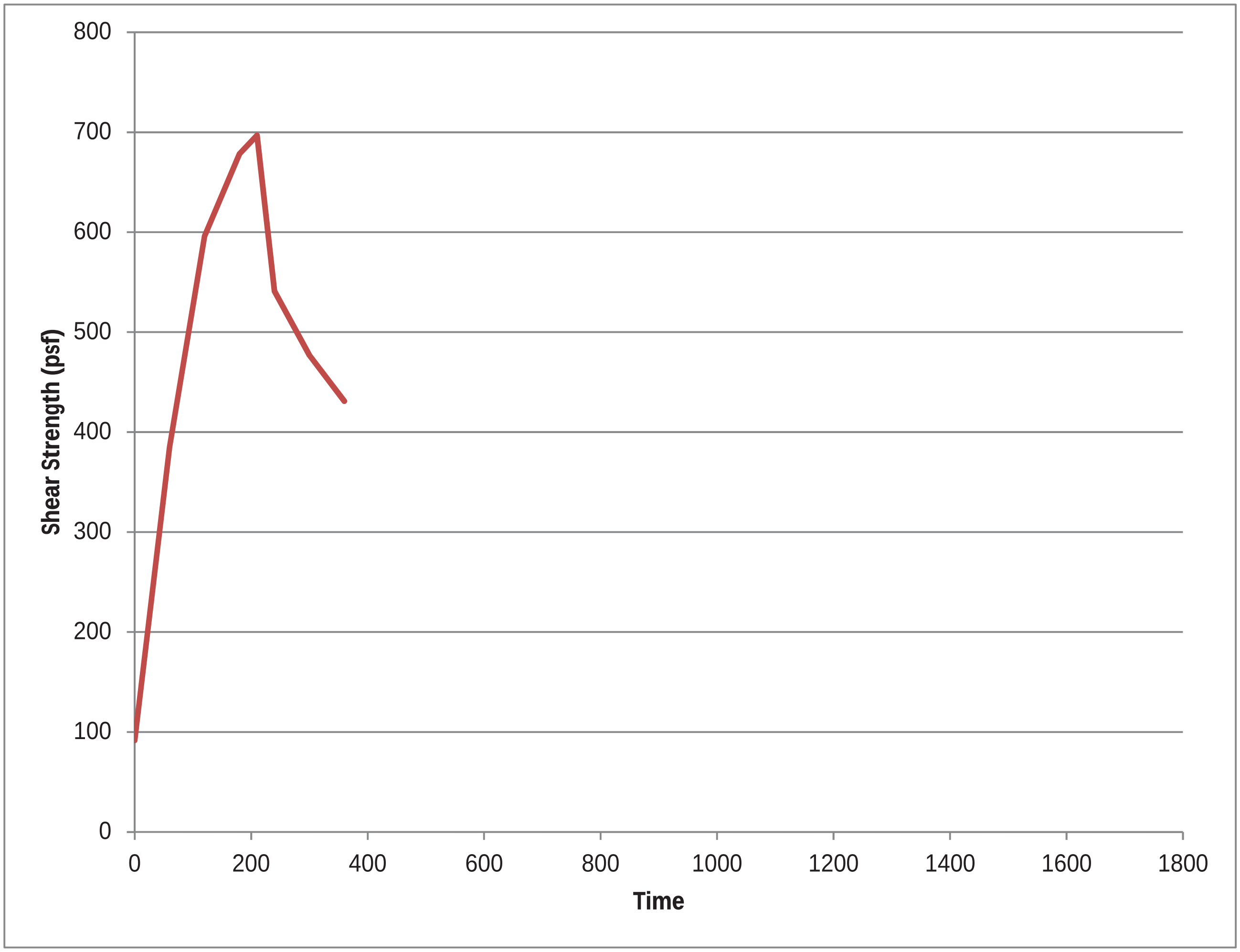
Location: B-2
Depth: 18

Max Shear 25.99 KPa
543.00 psf



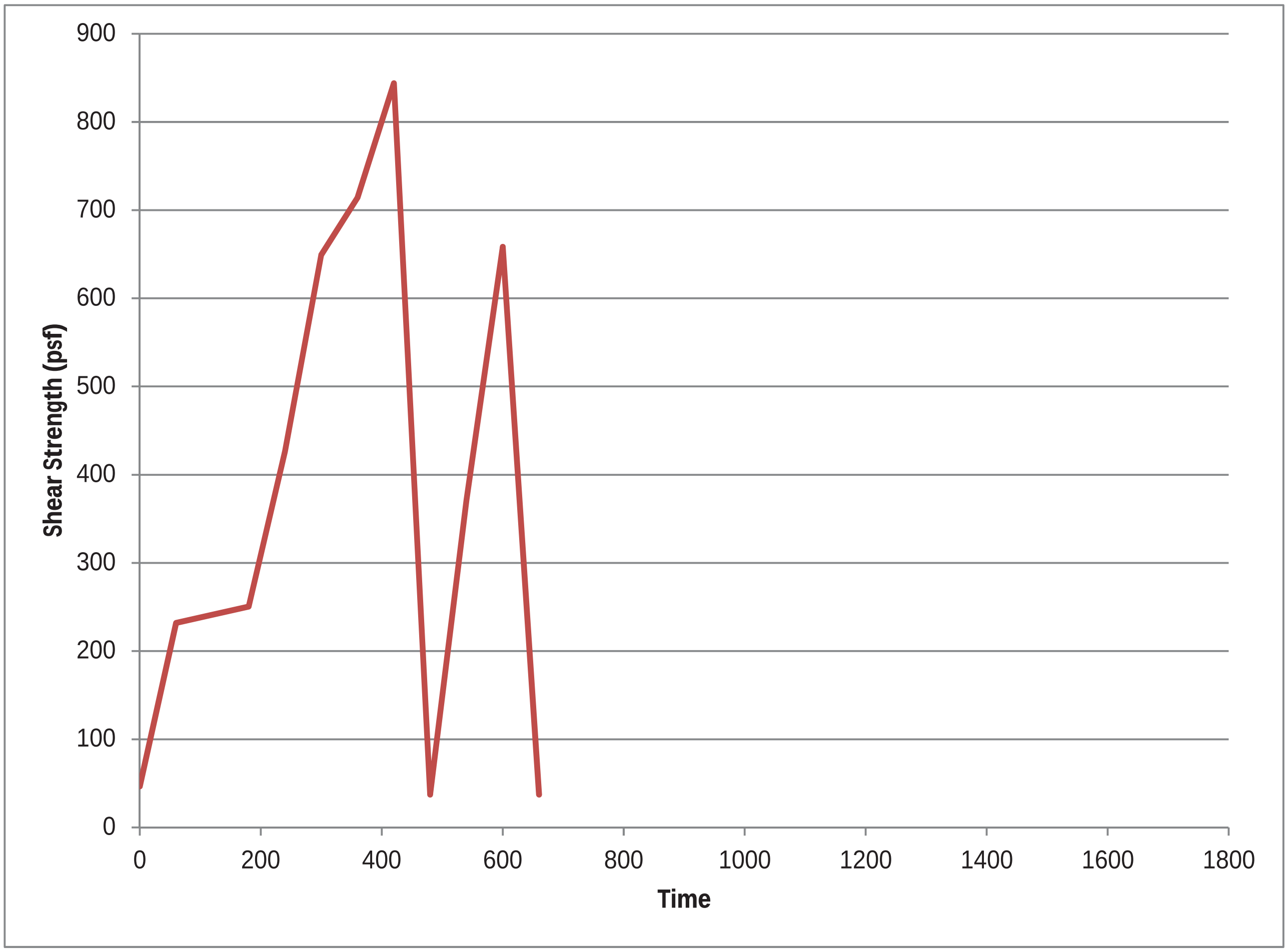
Location: B-2
Depth: 23

Max Shear Strength: 33.36 KPa
696.92 psf



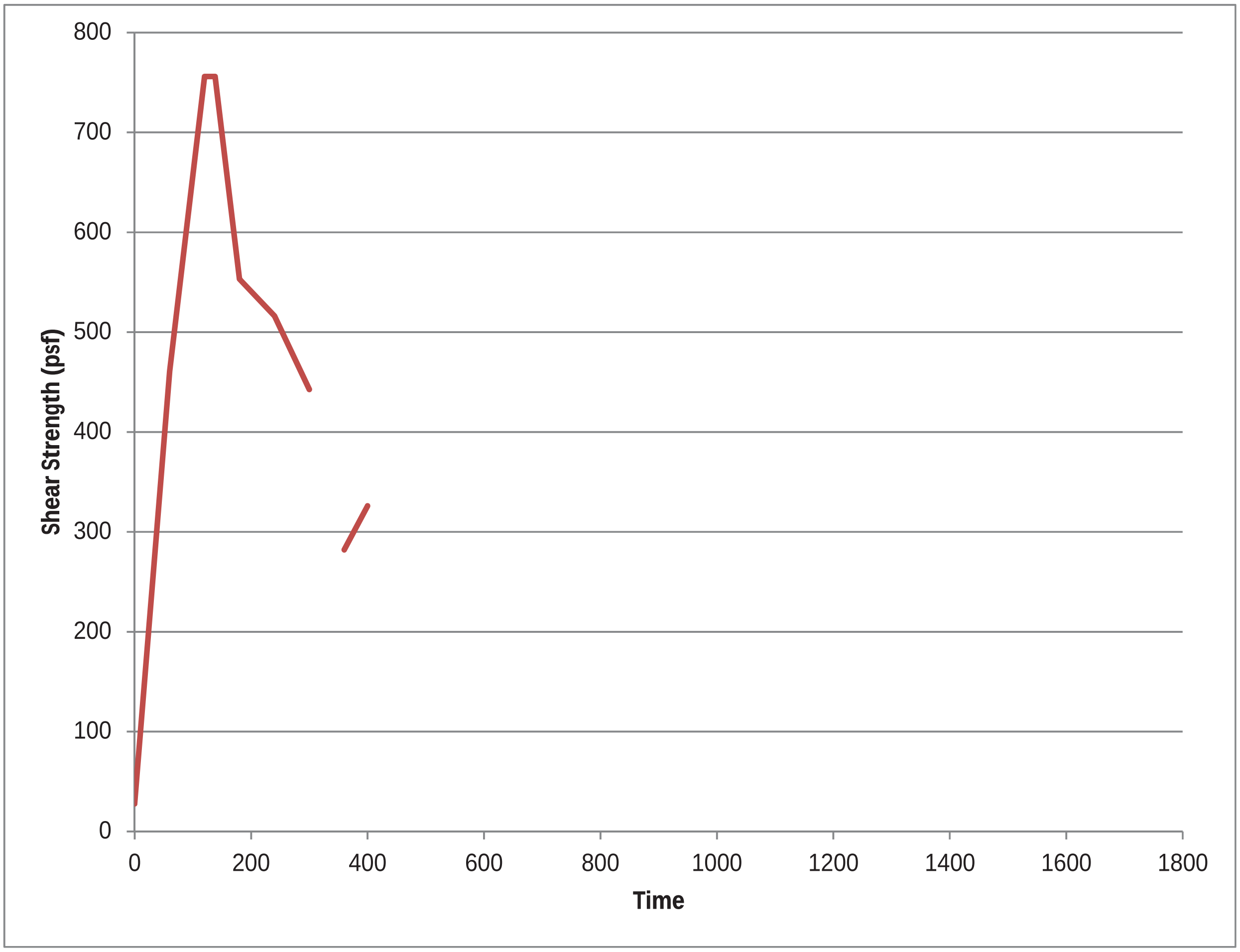
Location: B-2
Depth: 28

Max Shear Strength: 40.40 KPa
844.03 psf

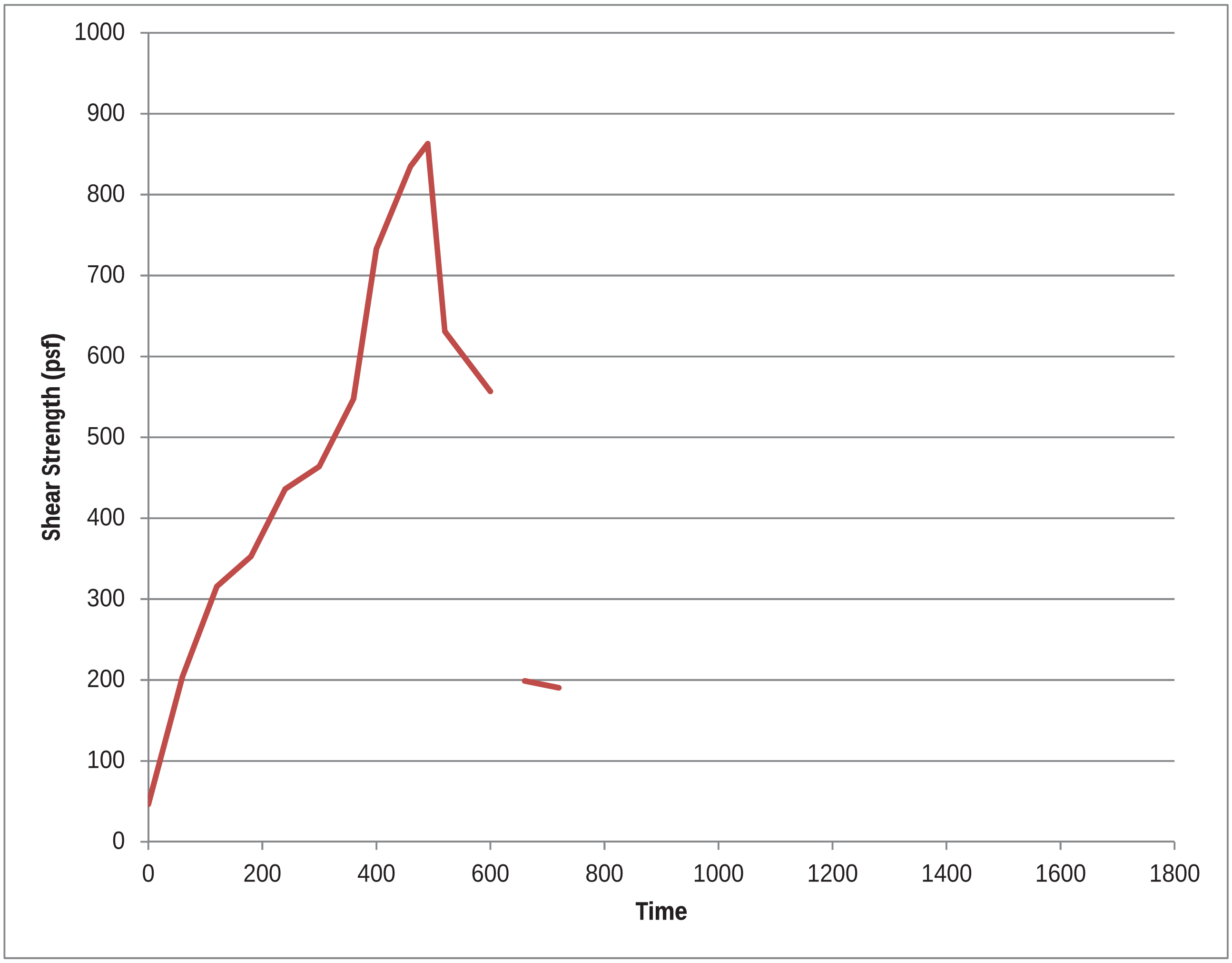


Location: B-2
Depth: 30

Max Shear Strength: 36.19 KPa
756.04 psf



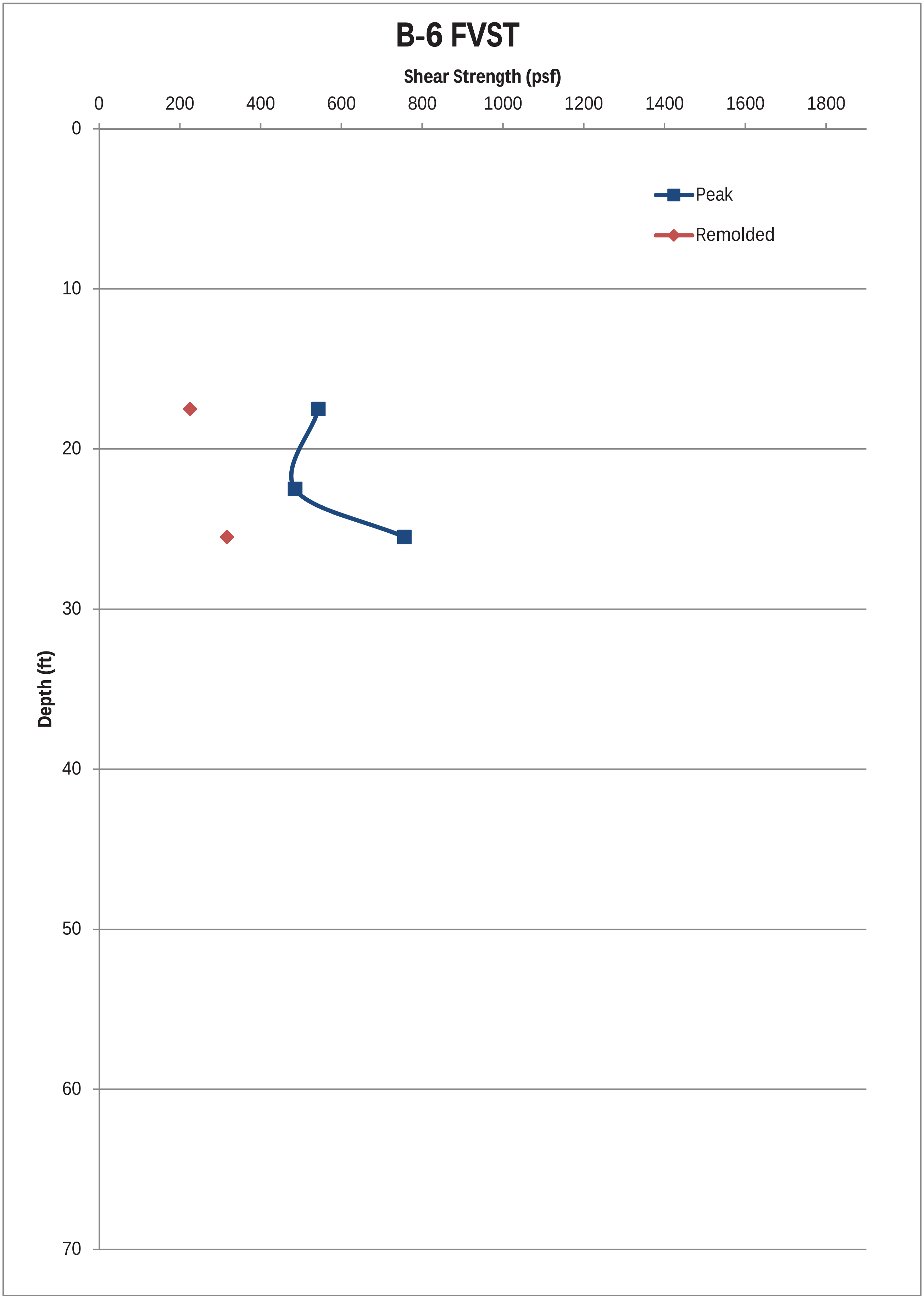
Location:	B-2	**Max Shear**	41.31	KPa
Depth:	35	**Strength:**	863.04	psf



CLIENT AMEC (Foster Wheeler)
SITE Vallejo, CA
LOCATION B-6
VANE TYPE Geonor H-10

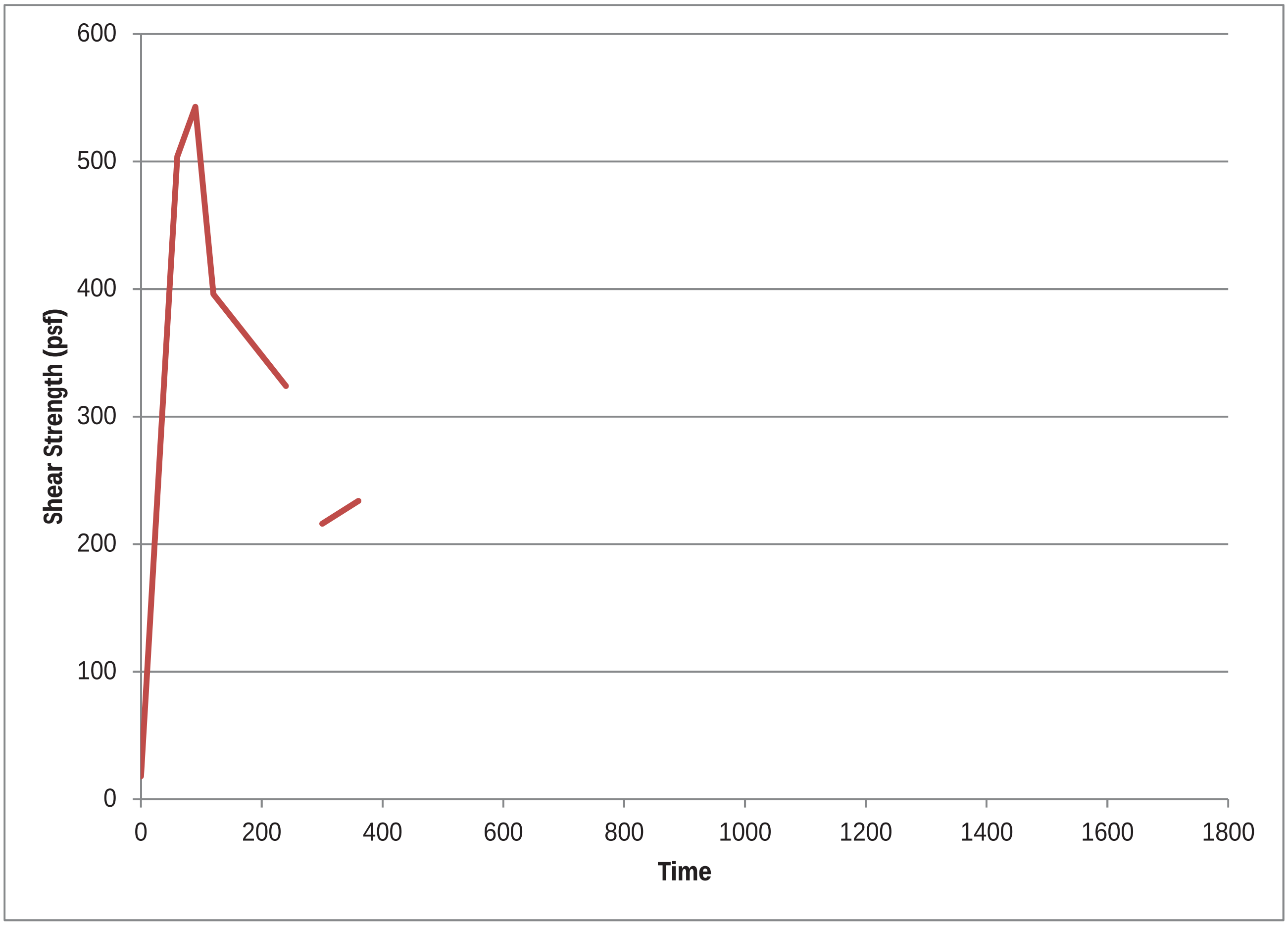
VANE DIAMETER, d (mm)	55
VANE LENGTH, l (mm)	110

DEPTH (m)	DEPTH (ft)	SHEAR STRENGTH (KPa)	SHEAR STRENGTH (psf)	REMOLDED SHEAR STRENGTH (KPa)	REMOLDED SHEAR STRENGTH (psf)	REMOLDED SENSITIVITY
5.34	17.50	26.00	543.00	10.77	225.00	2.40
6.86	22.50	23.21	485.00			
7.77	25.50	36.19	756.00	15.13	316.00	2.40

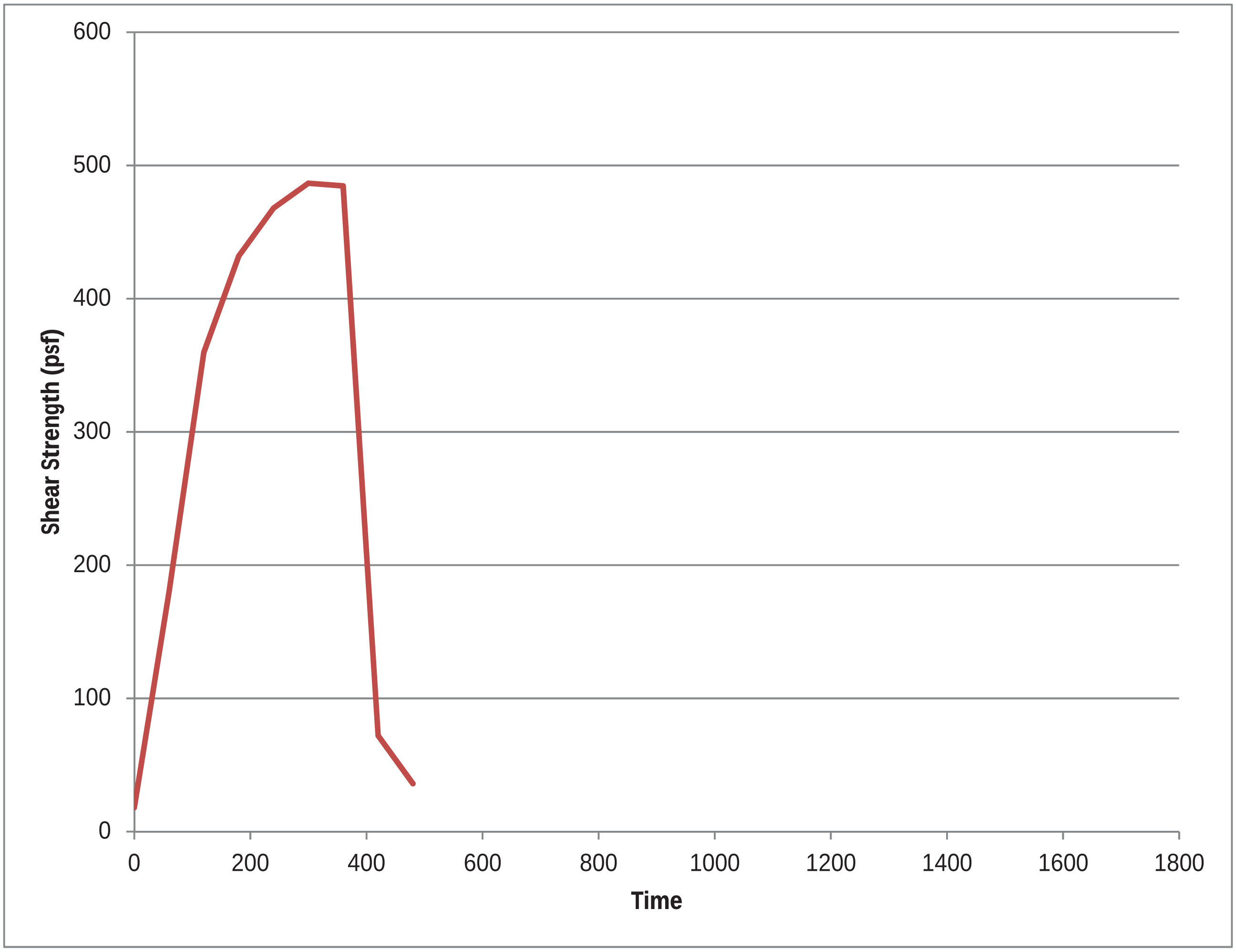

B-6 FVST
Shear Strength (psf)
0
200
400
600
800
1000
1200
1400
1600
1800
Depth (ft)
0
10
20
30
40
50
60
70
Peak
Remolded

Location: B-6
Depth: 17.5

Max Shear Strength: 26.00 KPa
543.00 psf

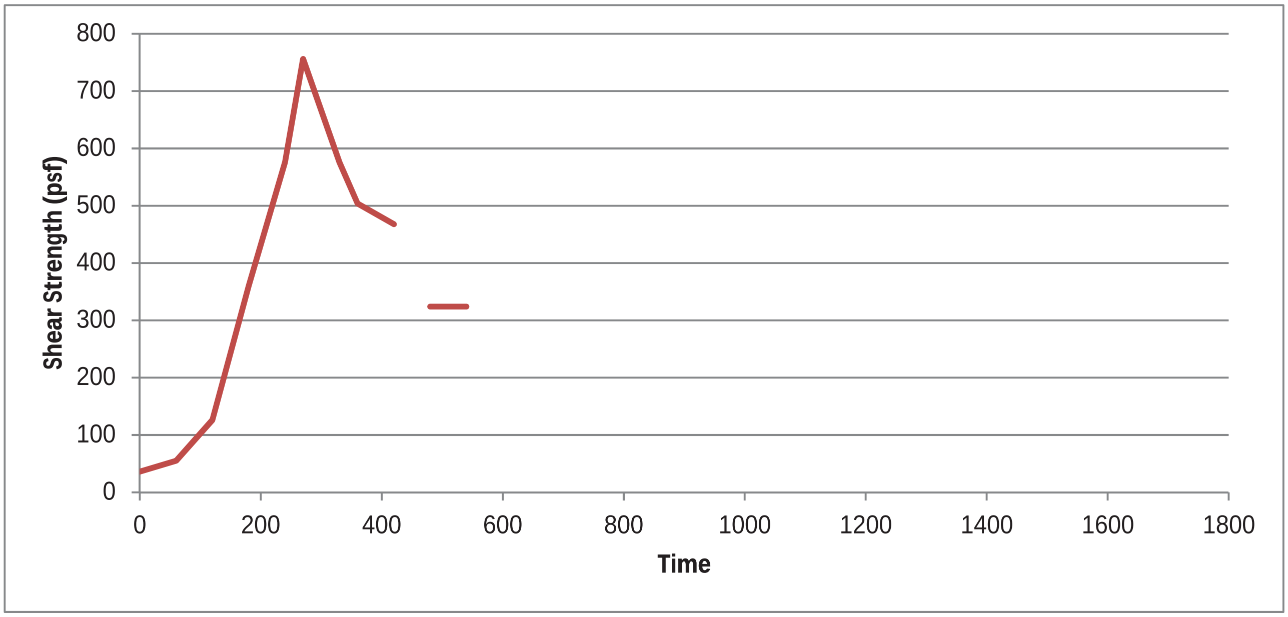


Location:	B-6	**Max Shear**	23.20 KPa
Depth:	22.5	**Strength:**	486.65 psf



Location: B-6
Depth: 25.5

Max Shear Strength: 36.91 KPa
756.20 psf



GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

December 10, 2015

AMEC Foster Wheeler
Attn: Alexander Wright

Subject: CPT Site Investigation
Project Zeus
Vallejo, California
GREGG Project Number: 15-187MA

Dear Mr. Wright:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☐
3	Seismic Cone Penetration Tests	(SCPTU)	☒
4	UVOST Laser Induced Fluorescence	(UVOST)	☐
5	Groundwater Sampling	(GWS)	☐
6	Soil Sampling	(SS)	☐
7	Vapor Sampling	(VS)	☐
8	Pressuremeter Testing	(PMT)	☐
9	Vane Shear Testing	(VST)	☐
10	Dilatometer Testing	(DMT)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden

Mary Walden
Operations Manager

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
CPT-01	10/12/15	55	-	-	-
CPT-3A	10/17/15	104	-	-	-
CPT-04	10/13/15	55	-	-	-
CPT-05	10/12/15	75	-	-	-
CPT-08	10/17/15	72	-	-	-
CPT-10	10/13/15	65	-	-	-
CPT-11	10/17/15	50	-	-	-
CPT-12	10/12/15	75	-	-	-
CPT-13	10/14/15	118	-	-	-
CPT-14	10/15/15	75	-	-	-
CPT-16	10/15/15	108	-	-	-
CPT-17	10/15/15	100	-	-	-
CPT-18	10/09/15	70	-	-	-
CPT-19	10/17/15	50	-	-	-
CPT-20	10/15/15	75	-	-	-
CPT-21	10/17/15	50	-	-	-
CPT-22	10/13/15	100	-	-	-
CPT-23	10/17/15	60	-	-	-
CPT-24	10/14/15	75	-	-	-
CPT-25	10/13/15	60	-	-	-
CPT-26	10/09/15	118	-	-	-
CPT-28	10/17/15	50	-	-	-
CPT-29	10/14/15	75	-	-	-
CPT-30	10/09/15	60	-	-	-
CPT-02	12/04/15	65	-	-	-
CPT-07	12/04/15	50	-	-	-
CPT-15	12/04/15	50	-	-	-

Bibliography

Lunne, T., Robertson, P.K. and Powell, J.J.M., "Cone Penetration Testing in Geotechnical Practice" E & FN Spon. ISBN 0 419 23750, 1997

Roberston, P.K., "Soil Classification using the Cone Penetration Test", Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

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

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Campanella, R.G. and I. Weemees, "Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies", Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

DeGroot, D.J. and A.J. Lutenegger, "Reliability of Soil Gas Sampling and Characterization Techniques", International Site Characterization Conference - Atlanta, 1998.

Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, "Detection of Polyaromatic Hydrocarbon Contaminants Using the UVIF-CPT", 53rd Canadian Geotechnical Conference Montreal, QC October pp. 733-739, 2000.

Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, "Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers" BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

Copies of ASTM Standards are available through www.astm.org

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5 cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (*PPDT*). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

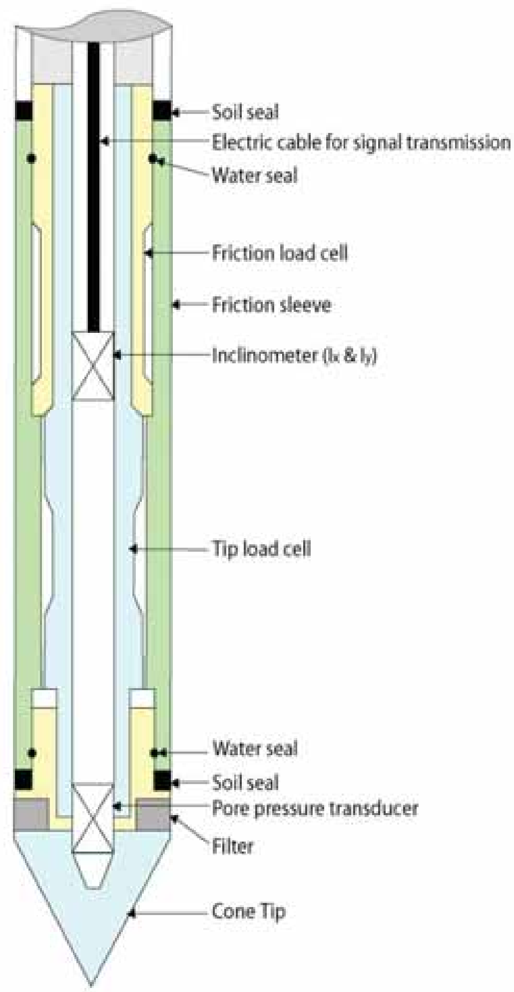


Figure CPT

Gregg 15cm^2 Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.80
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

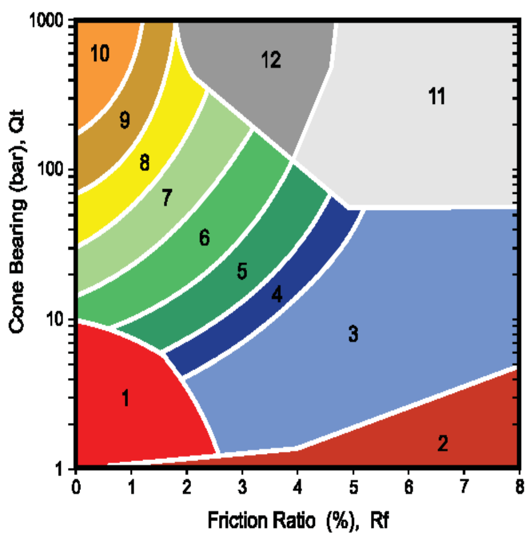
Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty clay to clay
5		Clayey silt to silty clay
6		Sandy silt to clayey silt
7		Silty sand to sandy silt
8		Sand to silty sand
9		Sand
10		Gravely sand to sand
11		Very stiff fine grained*
12		Sand to clayey sand*

*over consolidated or cemented

Figure SBT (After Robertson et al., 1986) – Note: Colors may vary slightly compared to plots

Cone Penetration Test (CPT) Interpretation

Gregg uses a proprietary CPT interpretation and plotting software. The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

1 Units for display (Imperial or metric) (atm. pressure, p_a = 0.96 tsf or 0.1 MPa)
2 Depth interval to average results (ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3 Elevation of ground surface (ft or m)
4 Depth to water table, z_w (ft or m) – input required
5 Net area ratio for cone, a (default to 0.80)
6 Relative Density constant, C_{Dr} (default to 350)
7 Young's modulus number for sands, α (default to 5)
8 Small strain shear modulus number
   a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
   b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
9 Undrained shear strength cone factor for clays, N_{kt} (default to 15)
10 Over Consolidation ratio number, k_{ocr} (default to 0.3)
11 Unit weight of water, (default to γ_w = 62.4 lb/ft^3 or 9.81 kN/m^3)

Column

1 Depth, z, (m) – CPT data is collected in meters
2 Depth (ft)
3 Cone resistance, q_c (tsf or MPa)
4 Sleeve resistance, f_s (tsf or MPa)
5 Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)
6 Other – any additional data
7 Total cone resistance, q_t (tsf or MPa) $q_t = q_c + u\,(1-a)$

8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t) \times 100\%$
9	Soil Behavior Type (non-normalized), SBT	see note
10	Unit weight, γ (pcf or kN/m^3)	based on SBT, see note
11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \sigma$ z
12	In-situ pore pressure, u_o (tsf)	$u_o = \gamma_w (z - z_w)$
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{t1}	$Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
16	Normalized Pore Pressure ratio, B_q	$B_q = u – u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with I_c)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60}, blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r, (%)	see note
24	Estimated Friction Angle, ϕ', (degrees)	see note
25	Estimated Young's modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, Go (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u/σ_v'
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

1 Soil Behavior Type (non-normalized), SBT (Lunne et al., 1997 and table below)

2 Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)

3 Soil Behavior Type (Normalized), SBT_n Lunne et al. (1997)

4 SBT_n Index, I_c $I_c = ((3.47 – \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$

5 Normalized Cone resistance, Q_{tn} (n varies with Ic)

$Q_{tn} = ((q_t - \sigma_{vo})/pa) (pa/(\sigma'_{vo})^n$ and recalculate I_c, then iterate:

When $I_c < 1.64$, n = 0.5 (clean sand)
When $I_c > 3.30$, n = 1.0 (clays)
When $1.64 < I_c < 3.30$, $n = (I_c – 1.64)0.3 + 0.5$
Iterate until the change in n, $\Delta n < 0.01$

6 Estimated permeability, k_{SBT} based on Normalized SBT_n (Lunne et al., 1997 and table below)

7 Equivalent SPT N_{60}, blows/ft Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$$

8 Equivalent SPT $(N_1)_{60}$ blows/ft $(N_1)_{60} = N_{60}\,C_N,$
where $C_N = (pa/\sigma'_{vo})^{0.5}$

9 Relative Density, D_r, (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

10 Friction Angle, ϕ', (degrees) $\tan\phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$

Only SBT_n 5, 6, 7 & 8 *Show'N/A' in zones 1, 2, 3, 4 & 9*

11 Young's modulus, E_s $E_s = \alpha\, q_t$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

12 Small strain shear modulus, Go
a. $G_o = S_G\,(q_t\ \ \sigma'_{vo}\ pa)^{1/3}$ *For SBT_n 5, 6, 7*
b. $G_o = C_G\,q_t$ *For SBT_n 1, 2, 3& 4*
Show 'N/A' in zones 8 & 9

13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

14 Over Consolidation ratio, OCR $OCR = k_{ocr}\,Q_{t1}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

The following updated and simplified SBT descriptions have been used in the software:

SBT Zones

1 sensitive fine grained
2 organic soil
3 clay
4 clay & silty clay
5 clay & silty clay
6 sandy silt & clayey silt

SBT_n Zones

1 sensitive fine grained
2 organic soil
3 clay
4 clay & silty clay

7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand
11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

*heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print 'clays & silty clays')

Estimated Permeability (see Lunne et al., 1997)

SBT_n	Permeability (ft/sec)	(m/sec)
1	$3x\ 10^{-8}$	$1x\ 10^{-8}$
2	$3x\ 10^{-7}$	$1x\ 10^{-7}$
3	$1x\ 10^{-9}$	$3x\ 10^{-10}$
4	$3x\ 10^{-8}$	$1x\ 10^{-8}$
5	$3x\ 10^{-6}$	$1x\ 10^{-6}$
6	$3x\ 10^{-4}$	$1x\ 10^{-4}$
7	$3x\ 10^{-2}$	$1x\ 10^{-2}$
8	$3x\ 10^{-6}$	$1x\ 10^{-6}$
9	$1x\ 10^{-8}$	$3x\ 10^{-9}$

Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft^3)	(kN/m^3)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0

Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (*u*) with time is measured behind the tip of the cone and recorded.

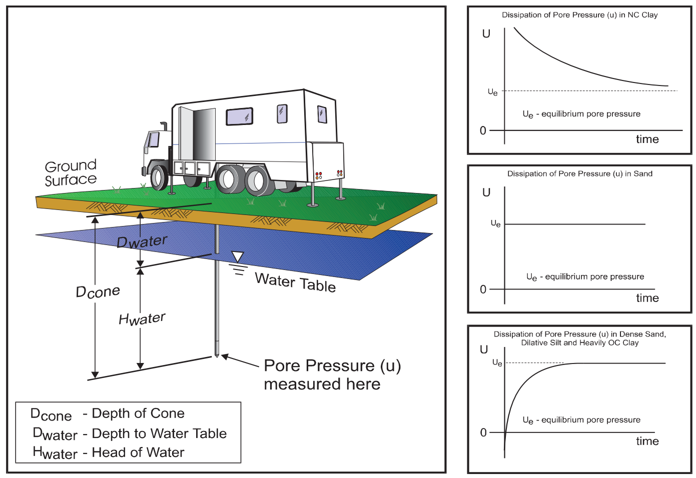
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests are summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors: 1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

Seismic Cone Penetration Testing (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (Vs) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg's cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: $v = \Delta d/\Delta t$

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary the shear wave velocities, arrival times and wave traces are provided with the report.

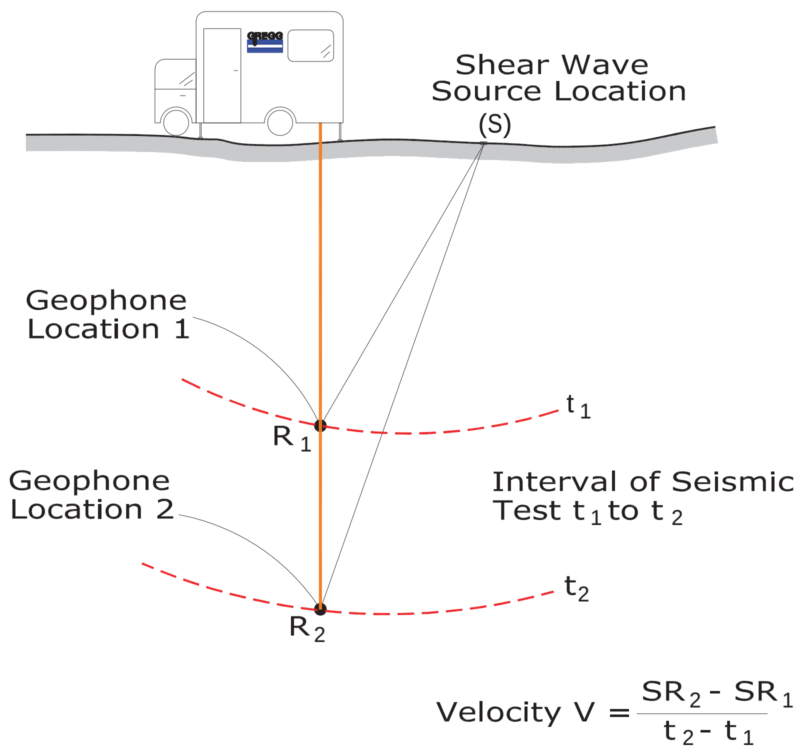


Figure SCPT

Groundwater Sampling

Gregg Drilling & Testing, Inc. conducts groundwater sampling using a sampler as shown in *Figure GWS*. The groundwater sampler has a retrievable stainless steel or disposable PVC screen with steel drop off tip. This allows for samples to be taken at multiple depth intervals within the same sounding location. In areas of slower water recharge, provisions may be made to set temporary PVC well screens during sampling to allow the pushing equipment to advance to the next sample location while the groundwater is allowed to infiltrate.

The groundwater sampler operates by advancing 44.5mm (1¾ inch) hollow push rods with the filter tip in a closed configuration to the base of the desired sampling interval. Once at the desired sample depth, the push rods are retracted; exposing the encased filter screen and allowing groundwater to infiltrate hydrostatically from the formation into the inlet screen. A small diameter bailer (approximately ½ or ¾ inch) is lowered through the push rods into the screen section for sample collection. The number of downhole trips with the bailer and time necessary to complete the sample collection at each depth interval is a function of sampling protocols, volume requirements, and the yield characteristics and storage capacity of the formation. Upon completion of sample collection, the push rods and sampler, with the exception of the PVC screen and steel drop off tip are retrieved to the ground surface, decontaminated and prepared for the next sampling event.

For a detailed reference on direct push groundwater sampling, refer to Zemo et. al., 1992.

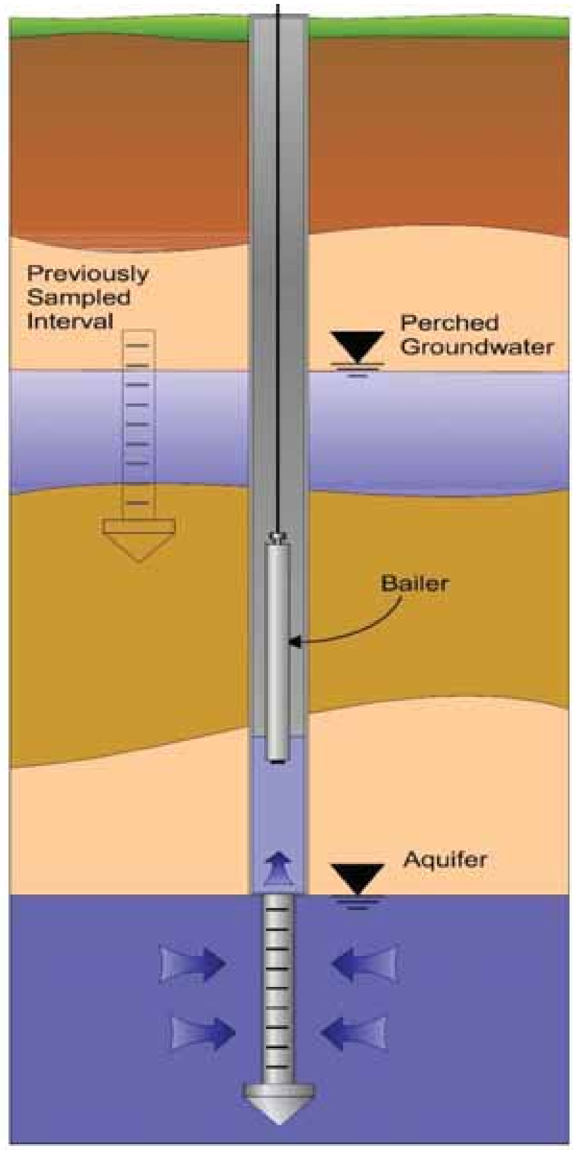


Figure GWS

Soil Sampling

Gregg Drilling & Testing, Inc. uses a piston-type push-in sampler to obtain small soil samples without generating any soil cuttings, *Figure SS*. Two different types of samplers (12 and 18 inch) are used depending on the soil type and density. The soil sampler is initially pushed in a "closed" position to the desired sampling interval using the CPT pushing equipment. Keeping the sampler closed minimizes the potential of cross contamination. The inner tip of the sampler is then retracted leaving a hollow soil sampler with inner 1¼" diameter sample tubes. The hollow sampler is then pushed in a locked "open" position to collect a soil sample. The filled sampler and push rods are then retrieved to the ground surface. Because the soil enters the sampler at a constant rate, the opportunity for 100% recovery is increased. For environmental analysis, the soil sample tube ends are sealed with Teflon and plastic caps. Often, a longer "split tube" can be used for geotechnical sampling.

For a detailed reference on direct push soil sampling, refer to Robertson et al, 1998.

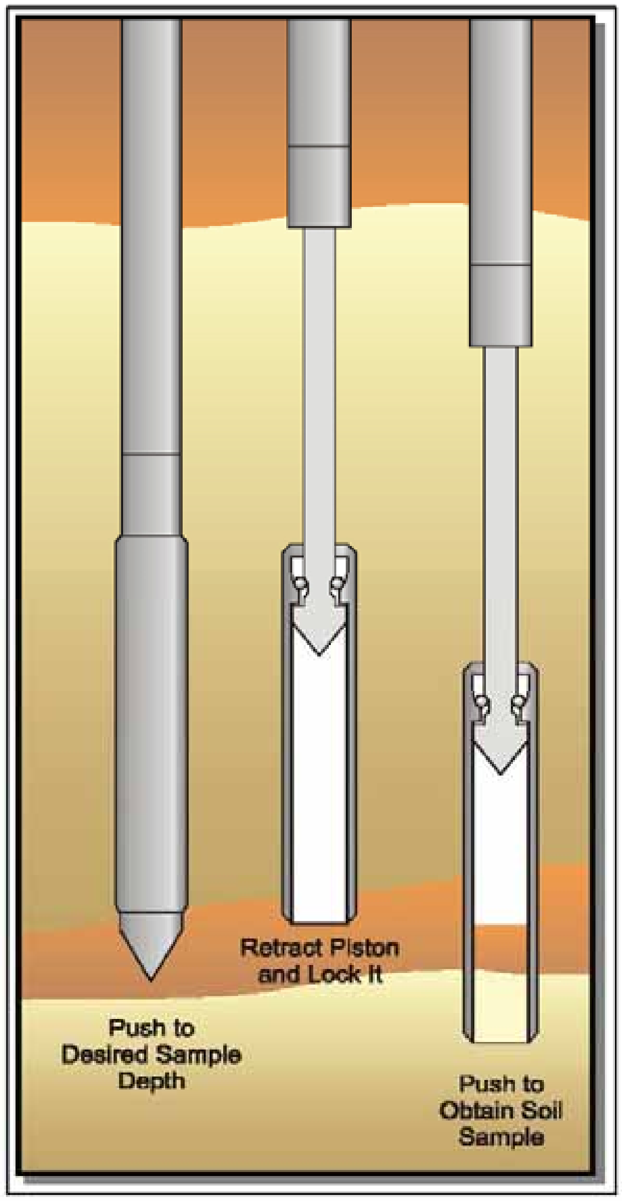


Figure SS

Ultra-Violet Induced Fluorescence (UVOST)

Gregg Drilling conducts Laser Induced Fluorescence (LIF) Cone Penetration Tests using a UVOST module that is located behind the standard piezocone, *Figure UVOST*. The laser induced fluorescence cone works on the principle that polycyclic aromatic hydrocarbons (PAH's), mixed with soil and/or groundwater, fluoresce when irradiated by ultra violet light. Therefore, by measuring the intensity of fluorescence, the lateral and vertical extent of hydrocarbon contamination in the ground can be estimated.

The UVOST module uses principles of fluorescence spectrometry by irradiating the soil with ultra violet light produced by a laser and transmitted to the cone through fiber optic cables. The UV light passes through a small window in the side of the cone into the soil. Any hydrocarbon molecules present in the soil absorb the light energy during radiation and immediately re-emit the light at a longer wavelength. This re-emission is termed fluorescence. The UVOST system also measures the emission decay with time at four different wavelengths (350nm, 400nm, 450nm, and 500nm). This allows the software to determine a product "signature" at each data point. This process provides a method to evaluate the type of contaminant. A sample output from the UVOST system is shown in *Figure Output*. In general, the typical detection limit for the UVOST system is <100 ppm and it will operate effectively above and below the saturated zone.

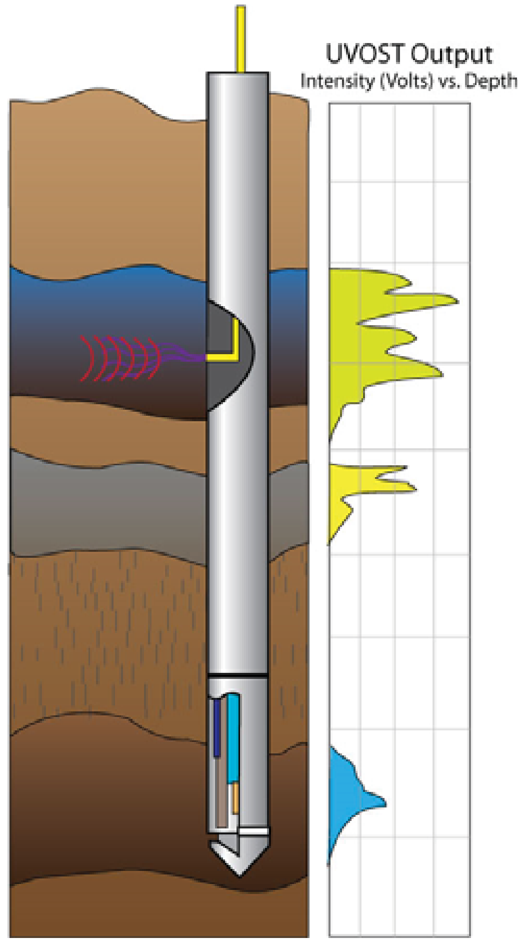


Figure UVOST

With the capability to push up to 200m (600ft) per day, laser induced fluorescence offers a fast and efficient means for delineating PAH contaminant plumes. Color coded logs offer qualitative information in a quick glance and can be produced in the field for real-time decision making. Coupled with the data provided by the CPT, a complete site assessment can be completed with no samples or cuttings, saving laboratory costs as well as site and environmental impact.

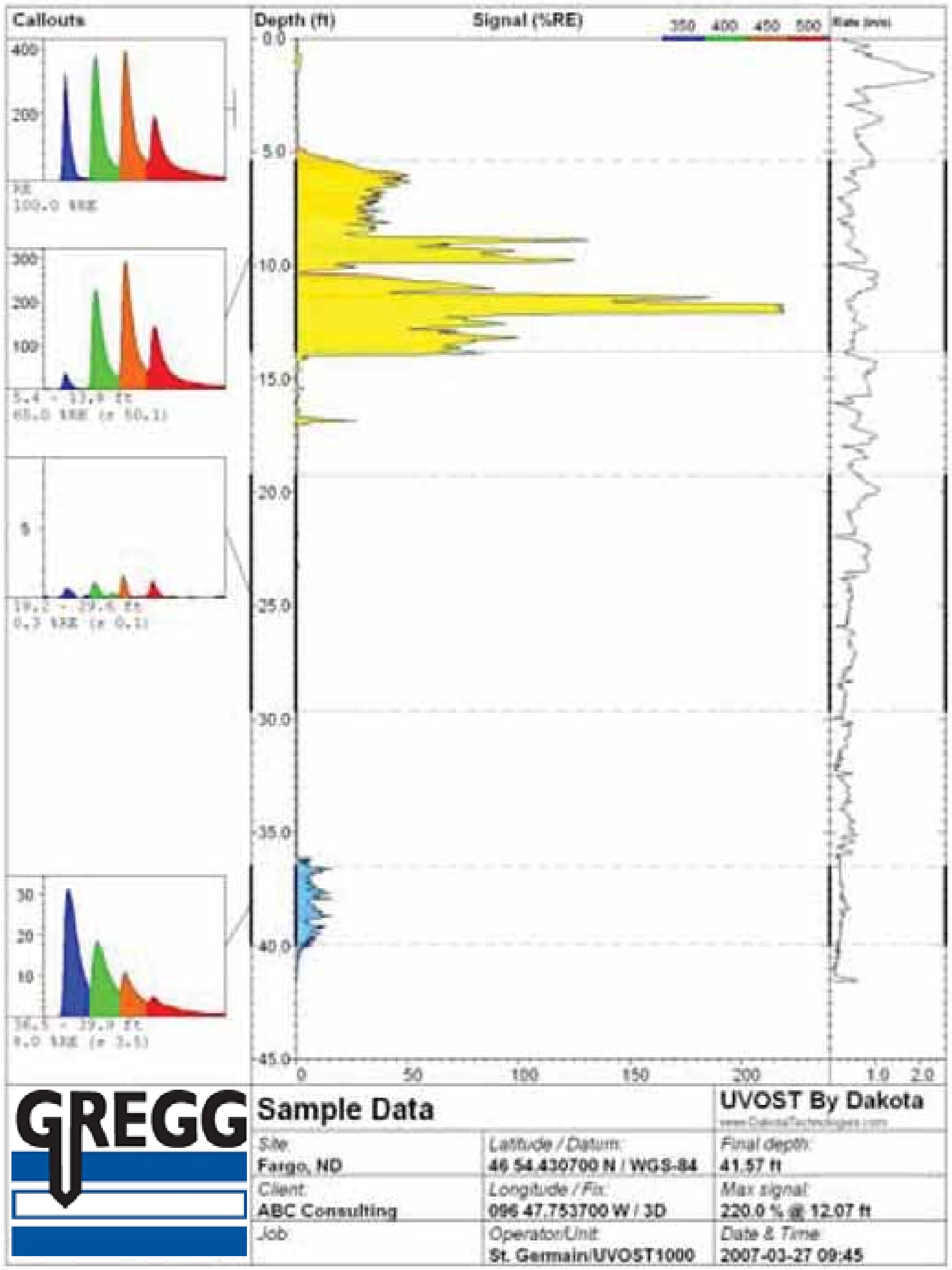


Figure Output

Hydrocarbons detected with UVOST

- Gasoline
- Diesel
- Jet (Kerasene)
- Motor Oil
- Cutting fluids
- Hydraulic fluids
- Crude Oil

Hydrocarbons rarely detected using UVOST

- Extremely weathered gasoline
- Coal tar
- Creosote
- Bunker Oil
- Polychlorinated bi-phenols (PCB's)
- Chlorinated solvent DNAPL
- Dissolved phase (aqueous) PAH's

Potential False Positives (fluorescence observed)

- Sea-shells (weak-medium)
- Paper (medium-strong depending on color)
- Peat/meadow mat (weak)
- Calcite/calcareous sands (weak)
- Tree roots (weak-medium)
- Sewer lines (medium-strong)

Potential False Negatives (do not fluoresce)

- Extremely weathered fuels (especially gasoline)
- Aviation gasoline (weak)
- "Dry" PAHs such as aqueous phase, lamp black, purifier chips
- Creosotes (most)
- Coal tars (most) gasoline (weak)
- Most chlorinated solvents
- Benzene, toluene, zylenes (relatively pure)

Dakota Technologies

UVOST Log Reference

2008-12-12

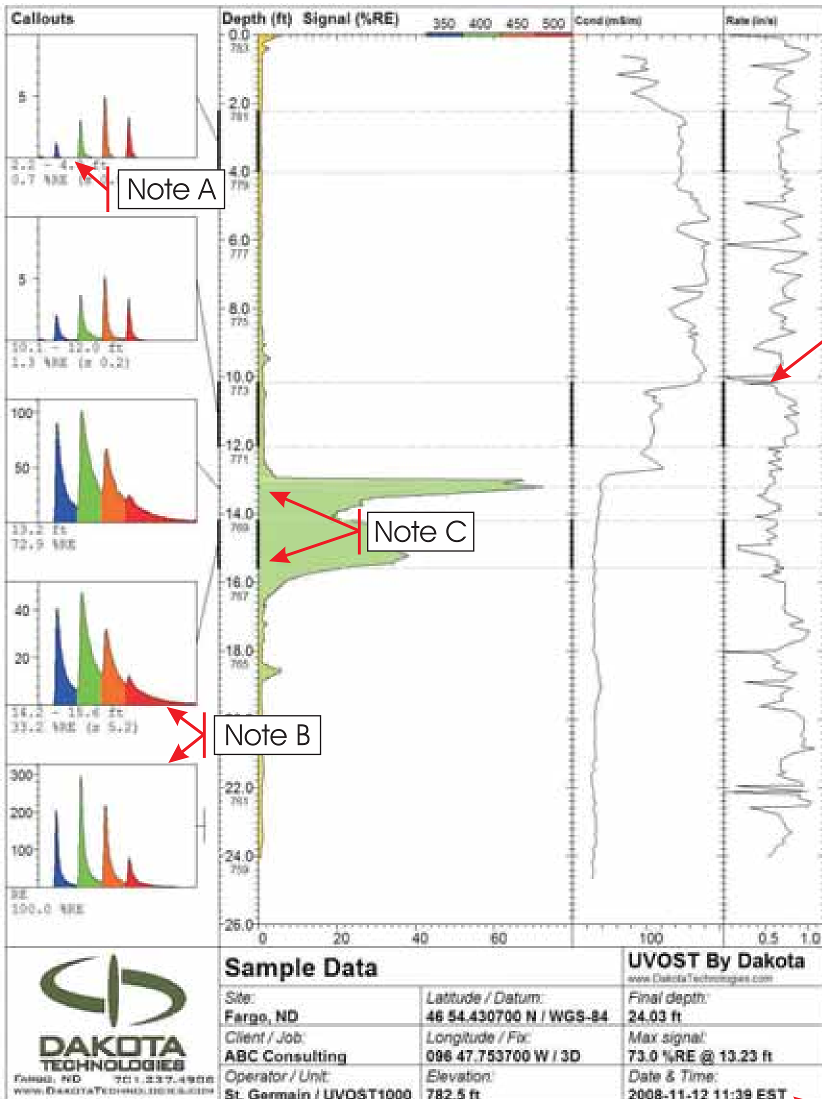
Main Plot :
Signal (total fluorescence) versus depth where signal is relative to the Reference Emitter (RE). The total area of the waveform is divided by the total area of the Reference Emitter yielding the %RE. This %RE scales with the NAPL fluorescence. The fill color is based on relative contribution of each channel's area to the total waveform area (see callout waveform). The channel-to-color relationship and corresponding wavelengths are given in the upper right corner of the main plot.

Callouts :
Waveforms from selected depths or depth ranges showing the multi-wavelength waveform for that depth.

The four peaks are due to fluorescence at four wavelengths and referred to as “channels”. Each channel is assigned a color.

Various NAPLs will have a unique waveform "fingerprint" due to the relative amplitude of the four channels and/or broadening of one or more channels.

Basic waveform statistics and any operator notes are given below the callout.



Conductivity Plot :
The Electrical Conductivity (EC) of the soil can be logged simultaneously with the UVOST data. EC often provides insight into the stratigraphy.
Note the drop in EC from 10 - 13 ft, indicating a shift from consolidated to unconsolidated stratigraphy. This correlates with the observed NAPL distribution.

Rate Plot :
The rate of probe advancement. ~ 0.8in (2cm) per second is preferred.

A noticeable decrease in the rate of advancement may be indicative of difficult probing conditions (gravel, angular sands, etc.) such as that seen here at ~5 ft.

Notice that this log was terminated arbitrarily, not due to "refusal", which would have been indicated by a sudden rate drop at final depth.

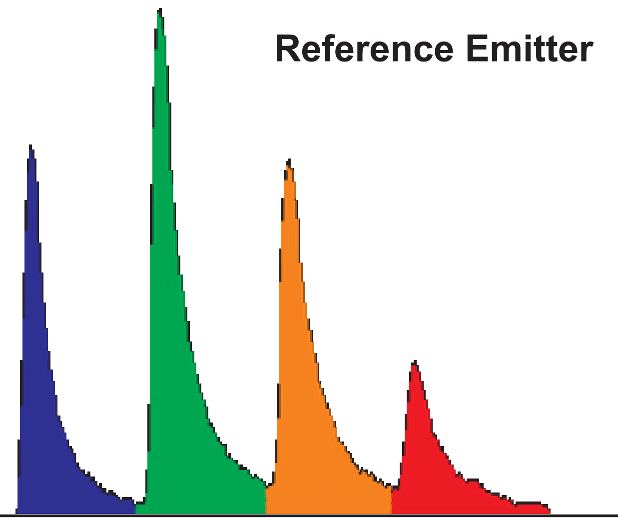
Info Box :
Contains pertinent log info including name and location.

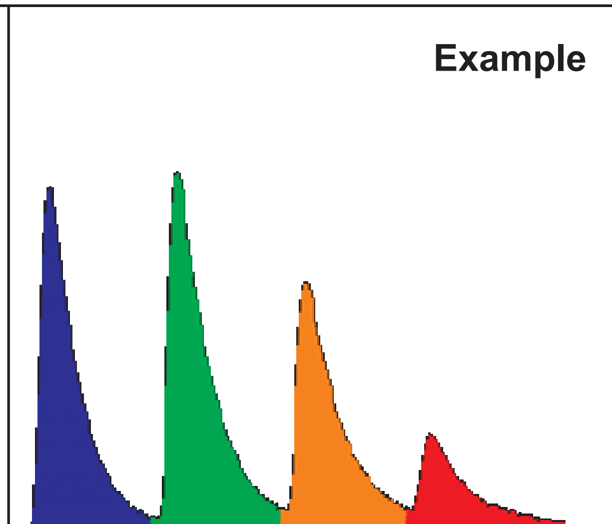
Note A :
Time is along the x axis. No scale is given, but it is a consistent 320ns wide.
The y axis is in mV and directly corresponds to the amount of light striking the photodetector.

Note B :
These two waveforms are clearly different. The first is weathered diesel from the log itself while the second is the Reference Emitter (a blend of NAPLs) always taken before each log for calibration.

Note C :
Callouts can be a single depth (see 3rd callout) or a range (see 4th callout). The range is noted on the depth axis by a bold line. When the callout is a range, the average and standard deviation in %RE is given below the callout.

Waveform Signal Calculation



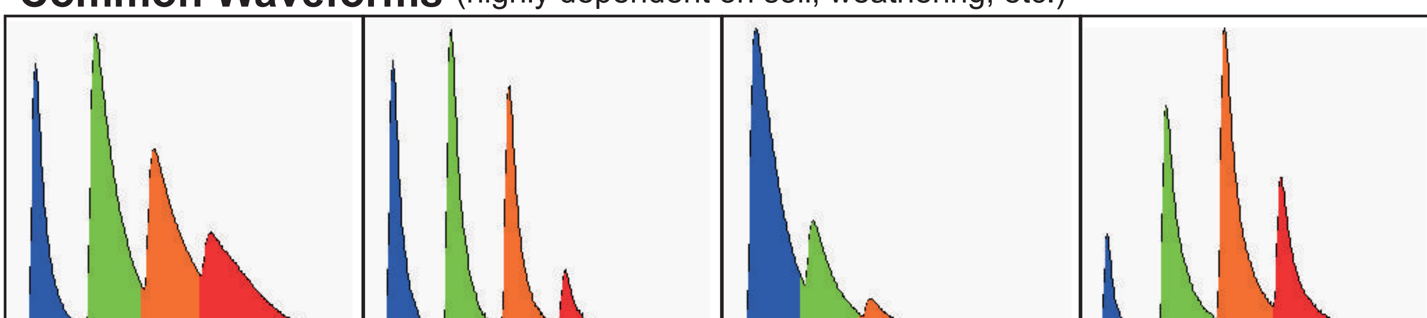


CH1	CH2	CH3	CH4	Total	*Channel*	CH1	CH2	CH3	CH4	Total
4820 +	8108 +	6249 +	2984 =	**22161**	***Area (pVs)***	4923 +	5743 +	4166 +	1735 =	**16587**
21.7	36.6	28.2	13.5	**100%**	***Percent RE***	22.3	25.9	18.8	7.8	**75%**

Data Files

***.lif.raw.bin**	Raw data file. Header is ASCII format and contains information stored when the file was initially written (e.g. date, total depth, max signal, gps, etc., and any information entered by the operator). All raw waveforms are appended to the bottom of the file in a binary format.
***.lif.plt**	Stores the plot scheme history (e.g. callout depths) for associated Raw file. Transfer along with the Raw file in order to recall previous plots.
***.lif.jpg**	A jpg image of the OST log including the main signal vs. depth plot, callouts, information, etc.
***.lif.dat.txt**	Data export of a single Raw file. ASCII tab delimited format. No string header is provided for the columns (to make importing into other programs easier). Each row is a unique depth reading. The columns are: Depth, Total Signal (%RE), Ch1%, Ch2%, Ch3%, Ch4%, Rate, Conductivity Depth, Conductivity Signal, Hammer Rate. Summing channels 1 to 4 yields the Total Signal.
***.lif.sum.txt**	A summary file for a number of Raw files. ASCII tab delimited format. The file contains a string header. The summary includes one row for each Raw file and contains information for each file including: the file name, gps coordinates, max depth, max signal, and depth at which the max signal occured.
***.lif.log.txt**	An activity log generated automatically located in the OST application directory in the 'log' subfolder. Each OST unit the computer operates will generate a separate log file per month. A log file contains much of the header information contained within each separate Raw file, including: date, total depth, max signal, etc.

Common Waveforms (highly dependent on soil, weathering, etc.)



Diesel | **Gas** | **Kerosene** | **Motor Oil**

GREGG

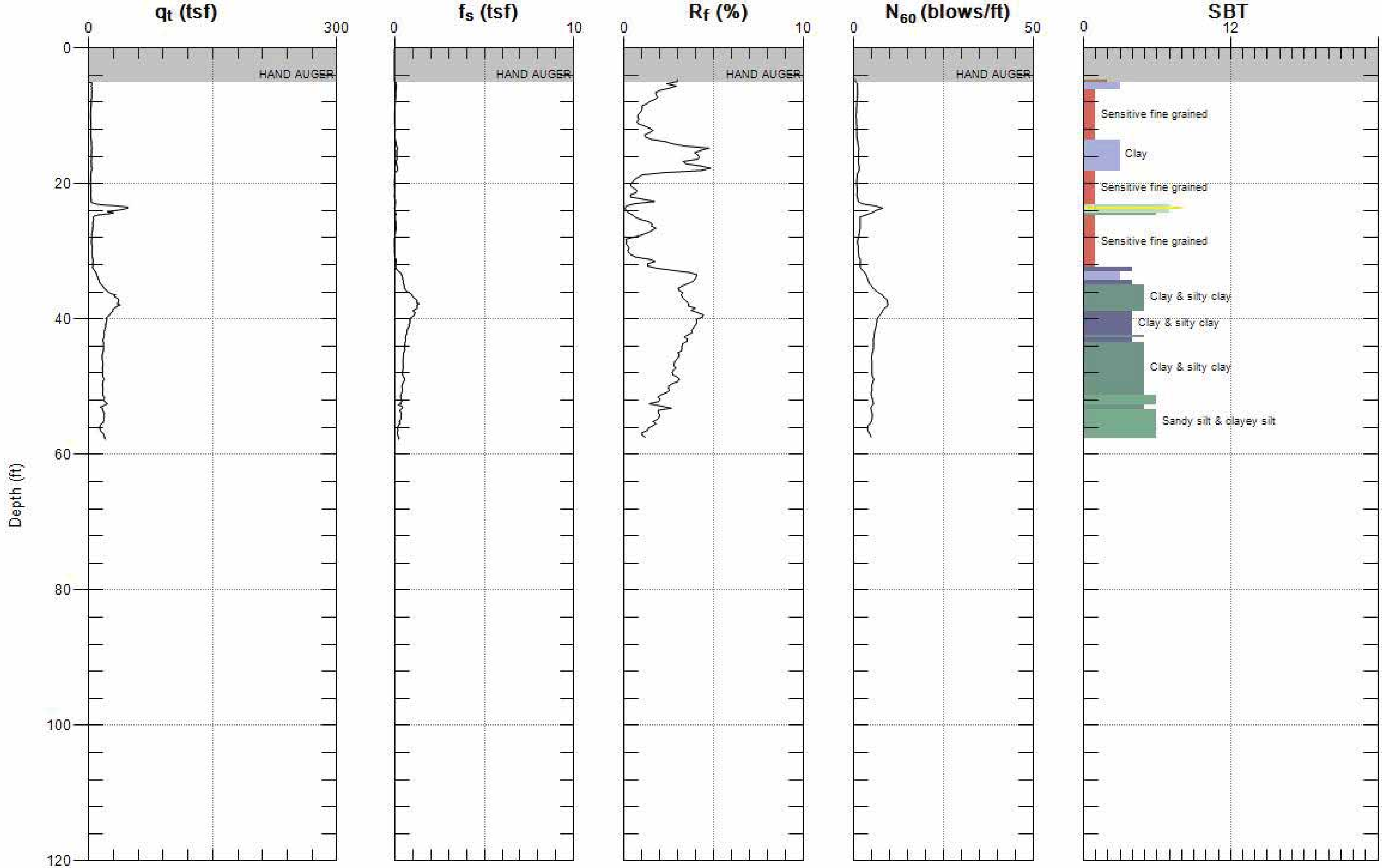
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-01

Engineer: C.COUTU

Date: 10/12/2015 12:11



Max. Depth: 57.743 (ft)
Avg. Interval: 0.328 (ft)

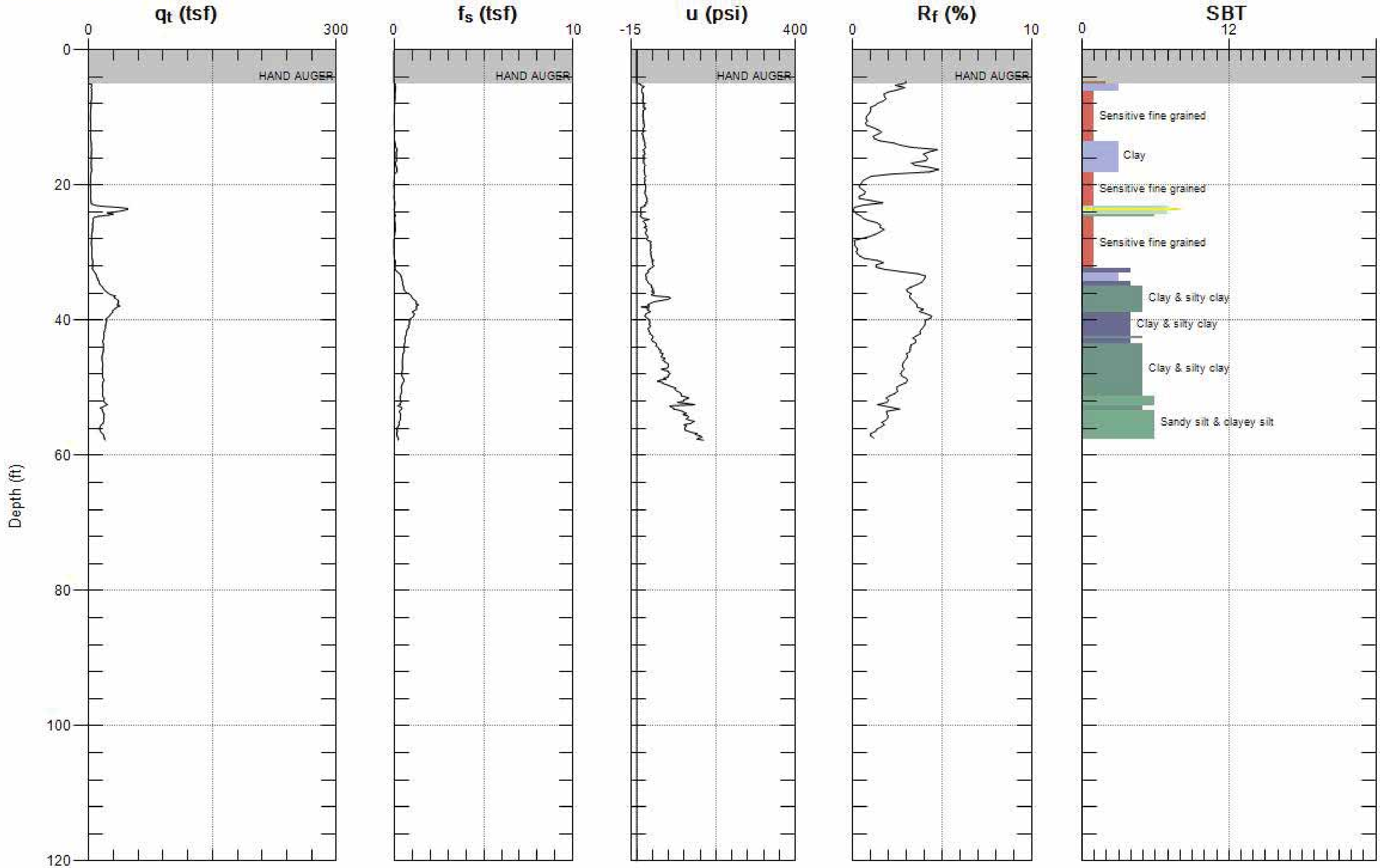
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-01

Engineer: C.COUTU
Date: 10/12/2015 12:11



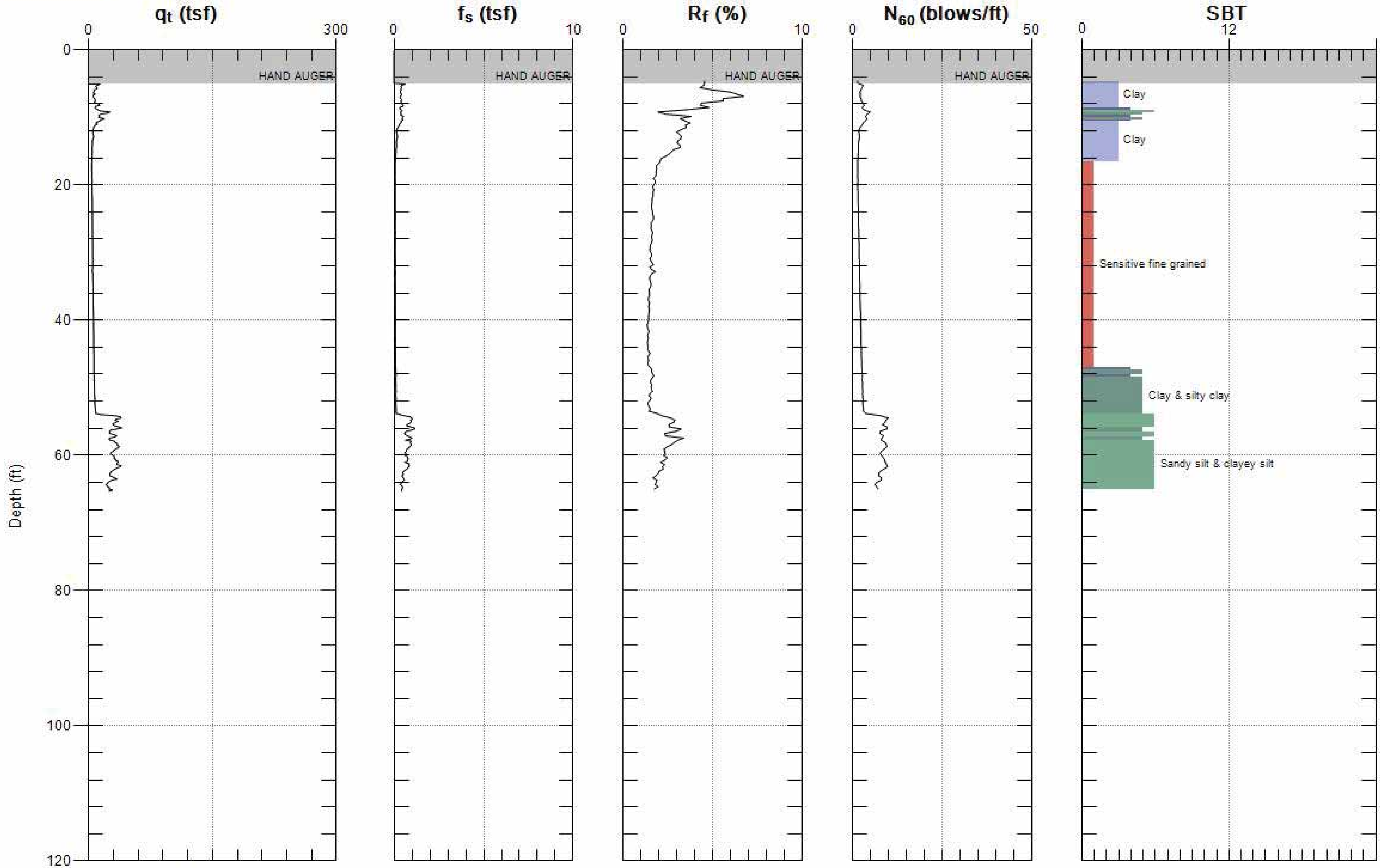
Max. Depth: 57.743 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **AMEC FOSTER WHEELER**

Site: PROJECT ZEUS
Sounding: CPT-2

Engineer: C.COUTU
Date: 12/4/2015 12:38



Max. Depth: 65.289 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

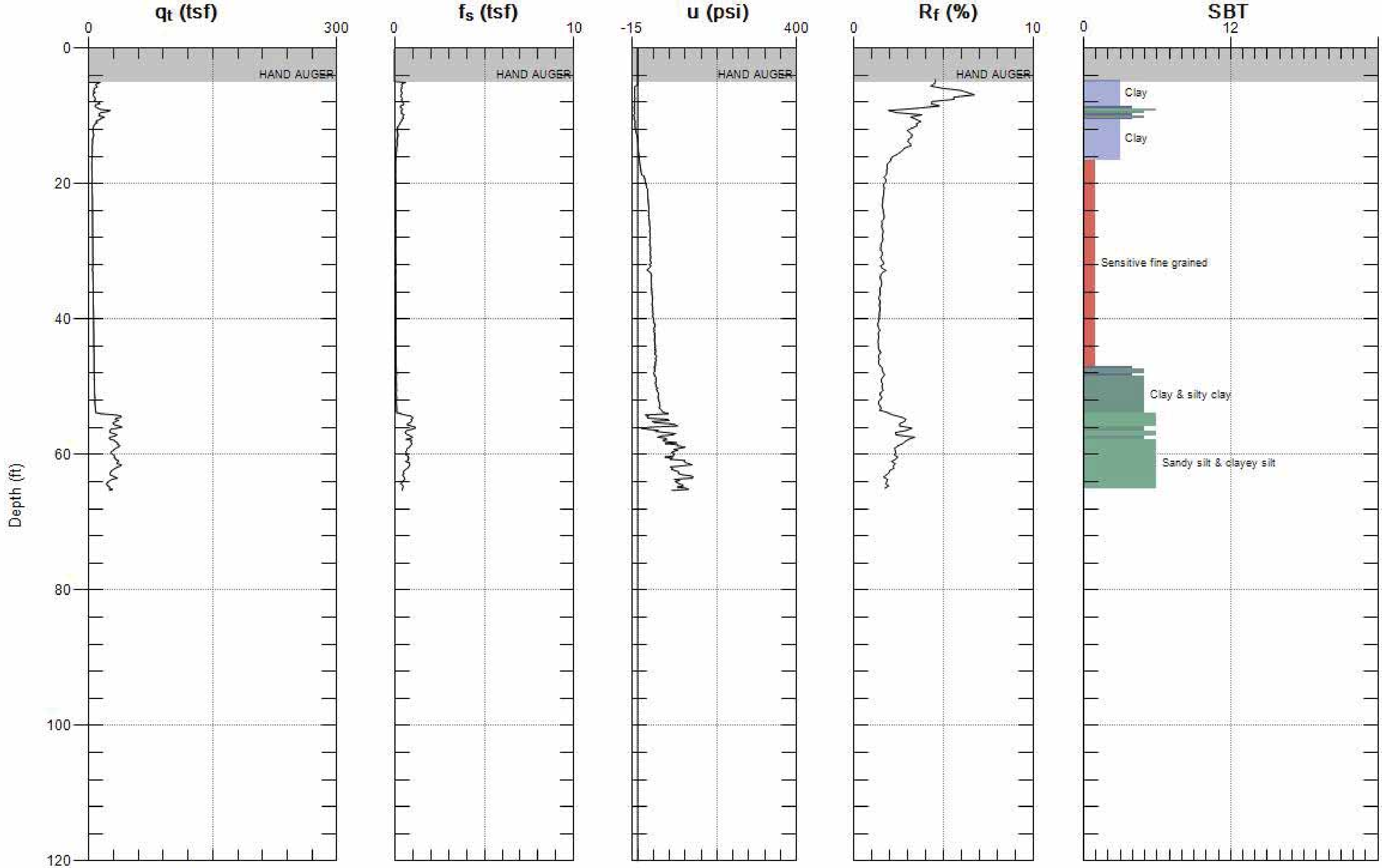
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-2

Engineer: C.COUTU

Date: 12/4/2015 12:38



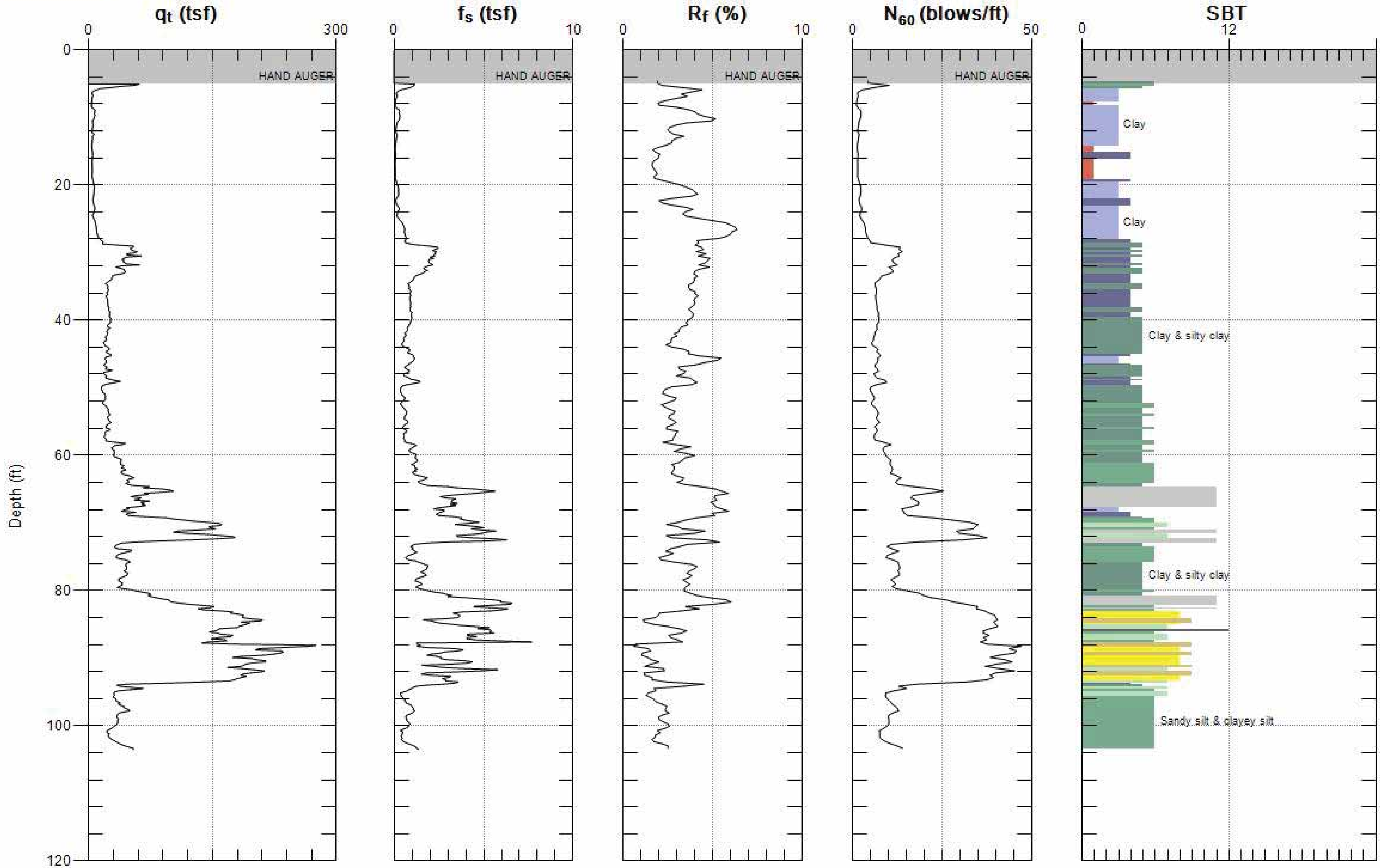
Max. Depth: 65.289 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **AMEC FOSTER WHEELER**

Site: PROJECT ZEUS
Sounding: CPT-3A

Engineer: C.COUTU
Date: 10/17/2015 11:43



Max. Depth: 103.511 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

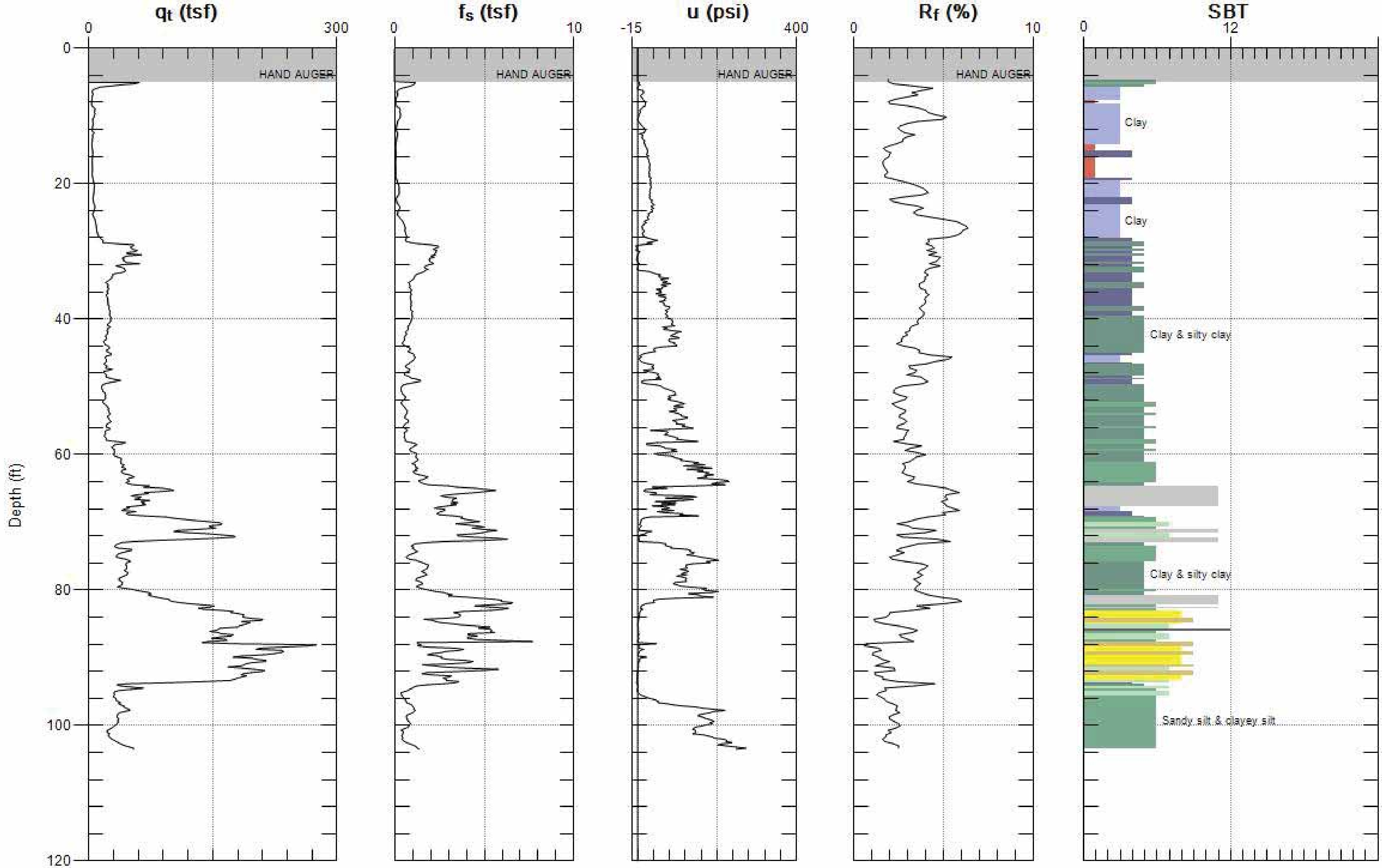
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-3A

Engineer: C.COUTU

Date: 10/17/2015 11:43



Max. Depth: 103.511 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

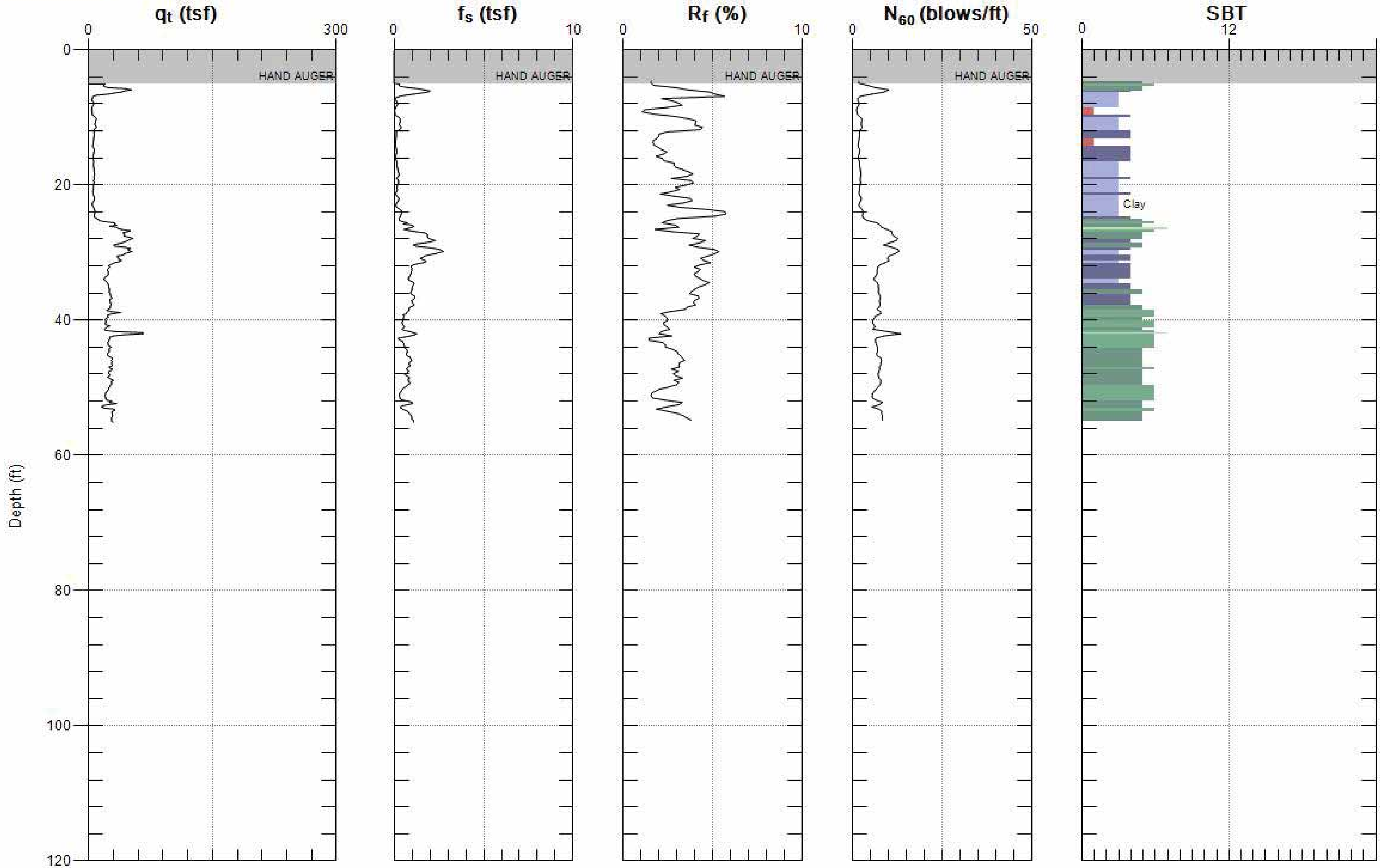
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-04

Engineer: C.COUTU

Date: 10/13/2015 08:05



Max. Depth: 55.118 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

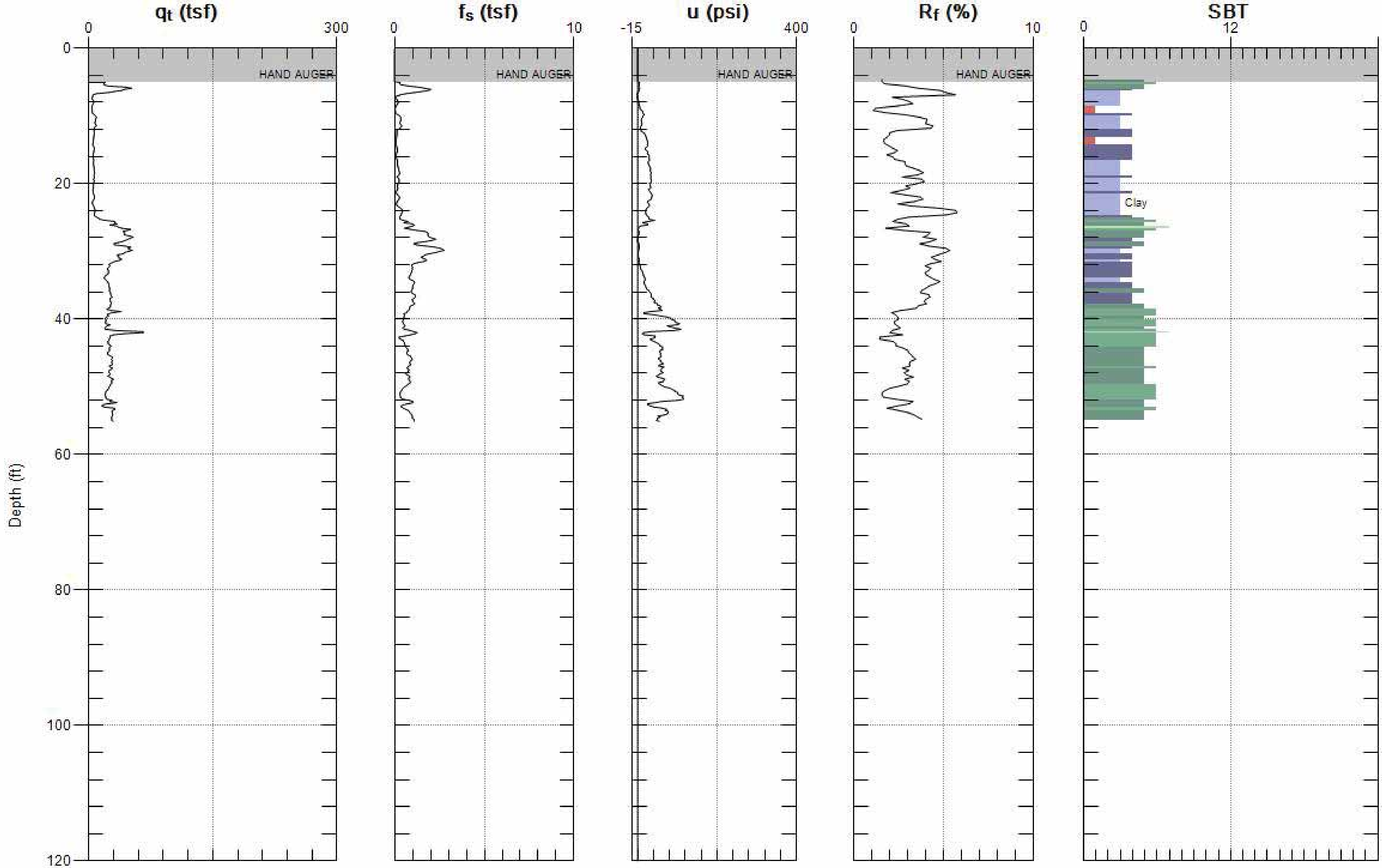
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-04

Engineer: C.COUTU

Date: 10/13/2015 08:05



Max. Depth: 55.118 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

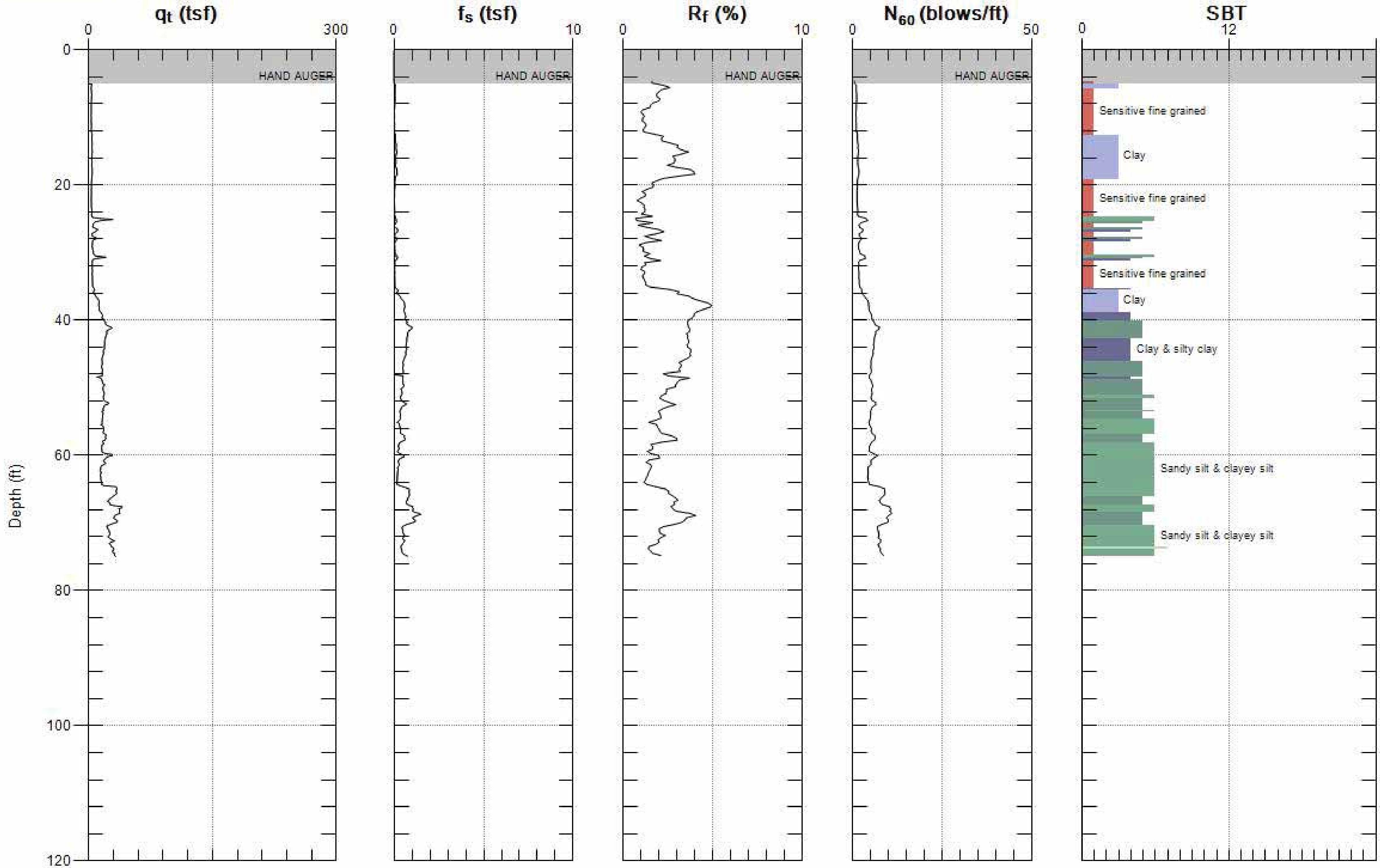
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-05

Engineer: C.COUTU

Date: 10/12/2015 09:39

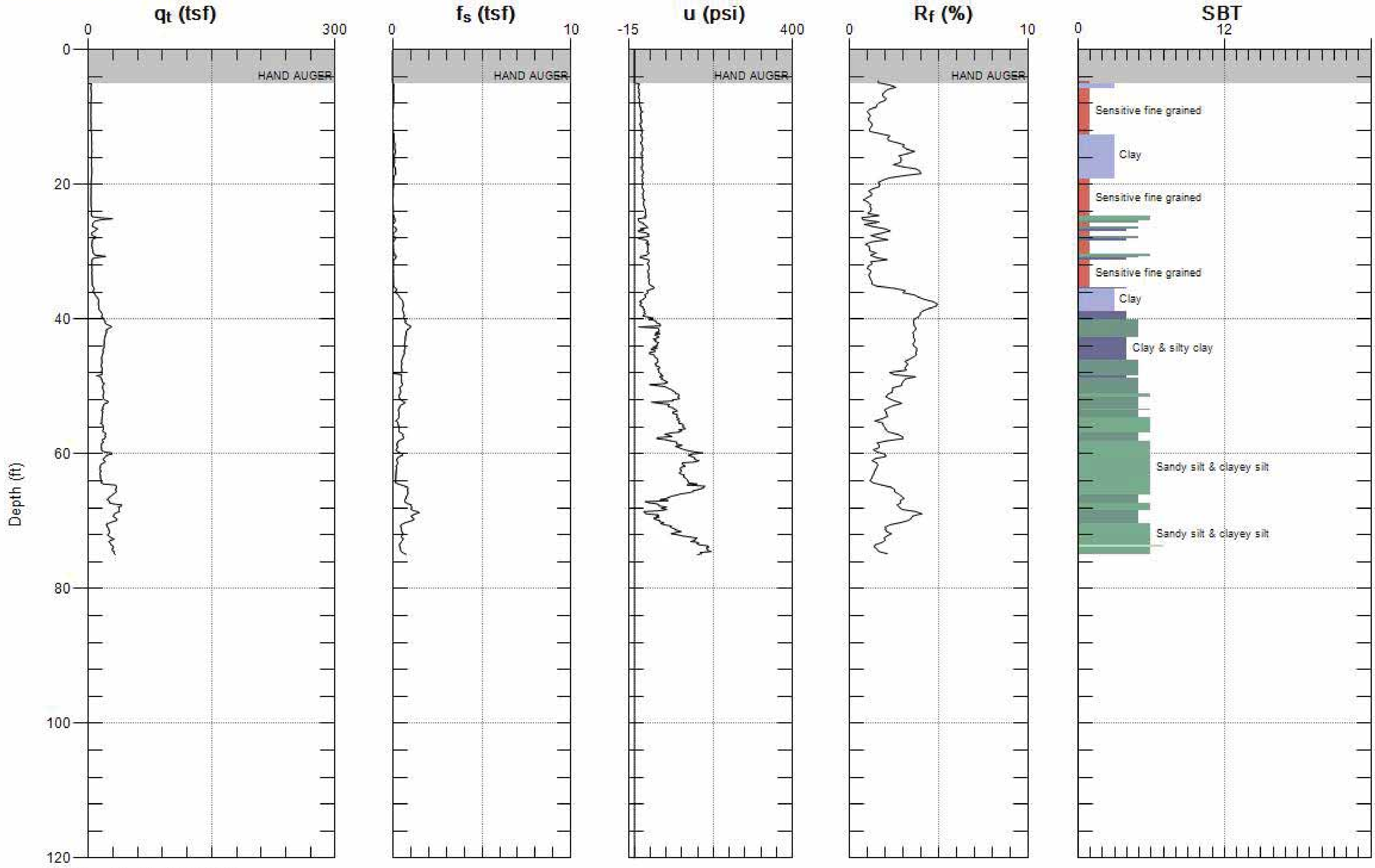


Max. Depth: 74.967 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

AMEC FOSTER WHEELER

Site: PROJECT ZEUS | Engineer: C.COUTU
Sounding: CPT-05 | Date: 10/12/2015 09:39



Max. Depth: 74.967 (ft)
Avg. Interval: 0.328 (ft)

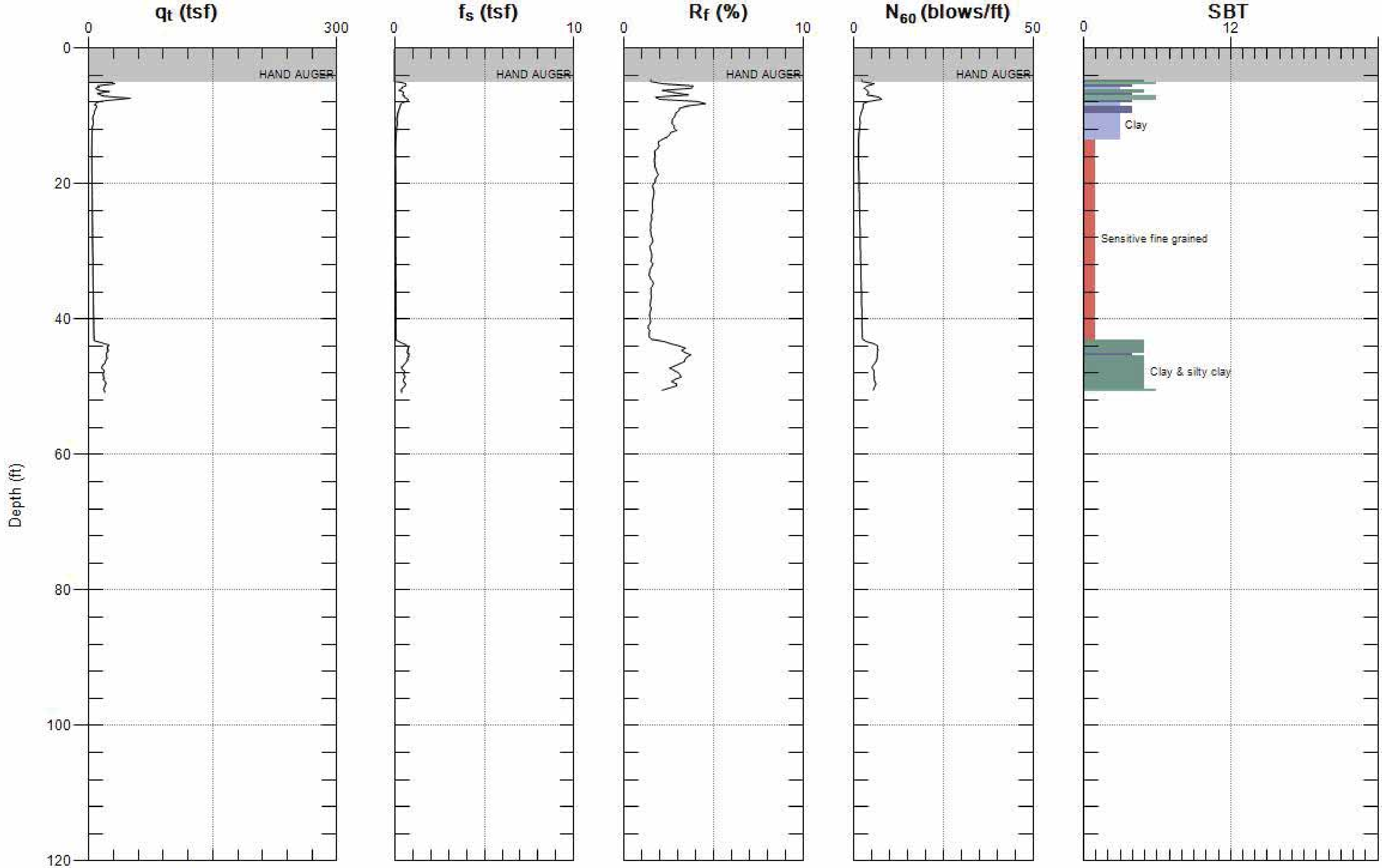
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-7

Engineer: C.COUTU
Date: 12/4/2015 10:15



Max. Depth: 50.853 (ft)
Avg. Interval: 0.328 (ft)

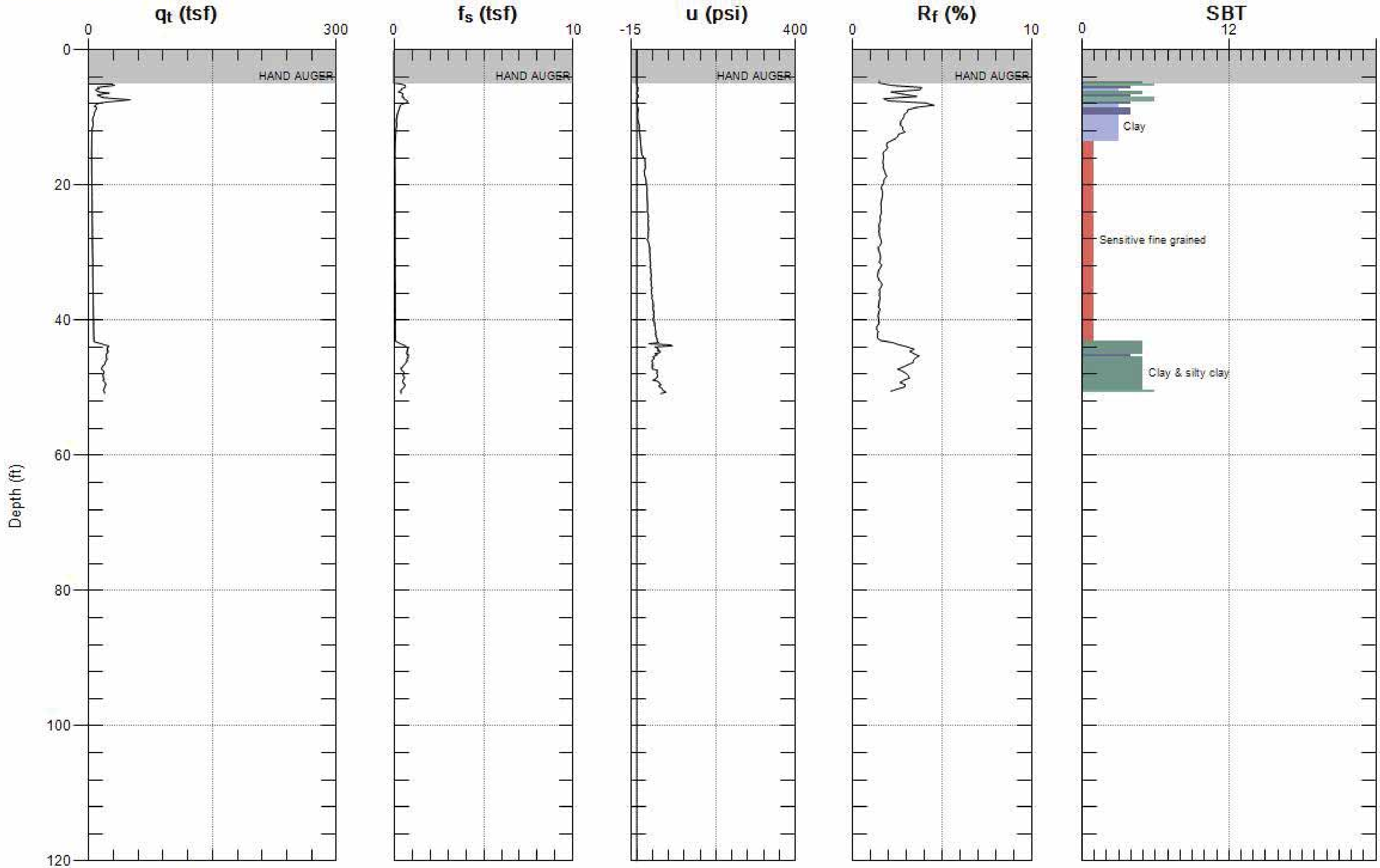
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-7

Engineer: C.COUTU
Date: 12/4/2015 10:15



Max. Depth: 50.853 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

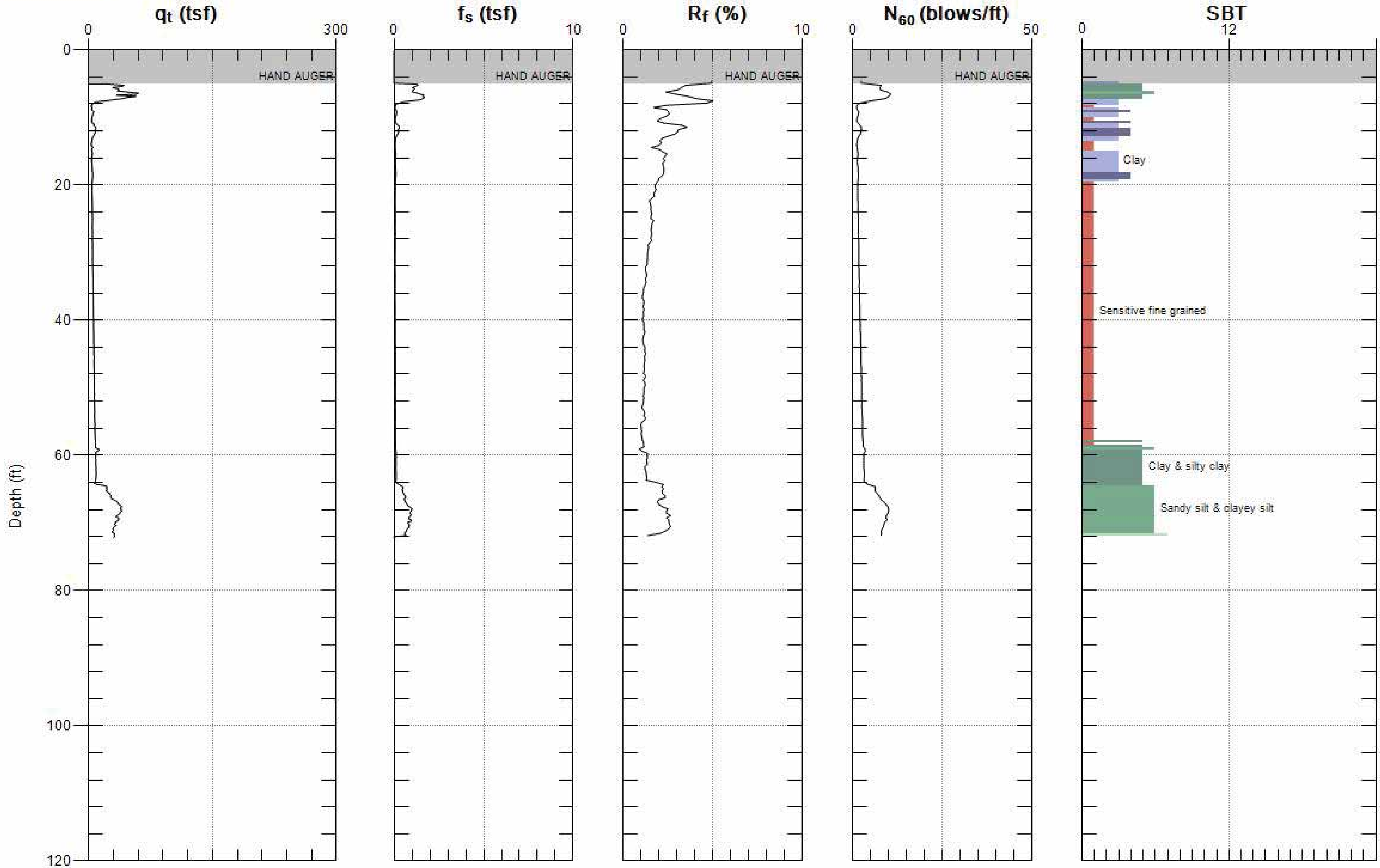
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-08

Engineer: C.COUTU

Date: 10/17/2015 11:59



Max. Depth: 72.178 (ft)

Avg. Interval: 0.328 (ft)

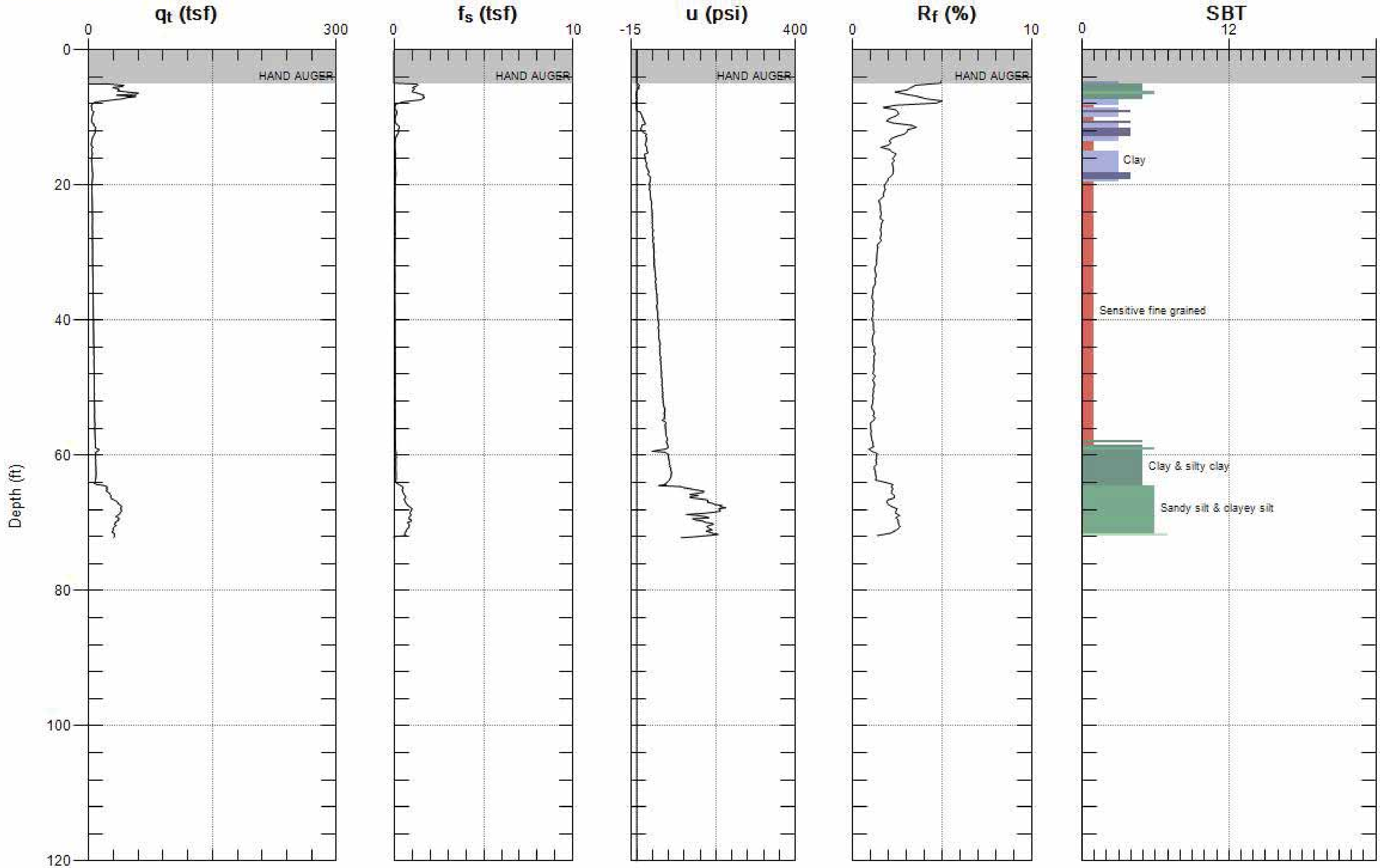
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-08

Engineer: C.COUTU
Date: 10/17/2015 11:59



Max. Depth: 72.178 (ft)
Avg. Interval: 0.328 (ft)

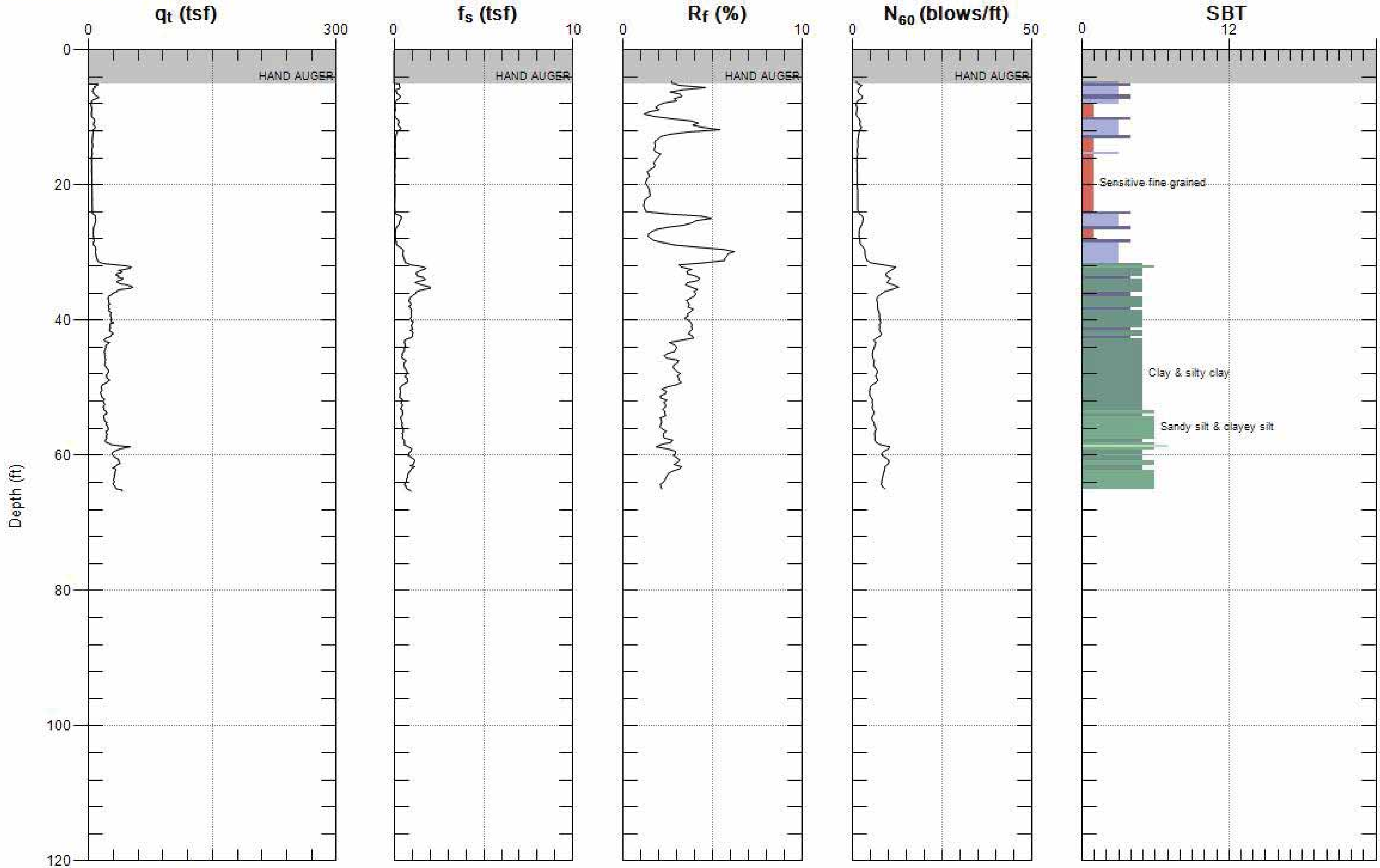
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-10

Engineer: C.COUTU
Date: 10/13/2015 11:22



Max. Depth: 65.289 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

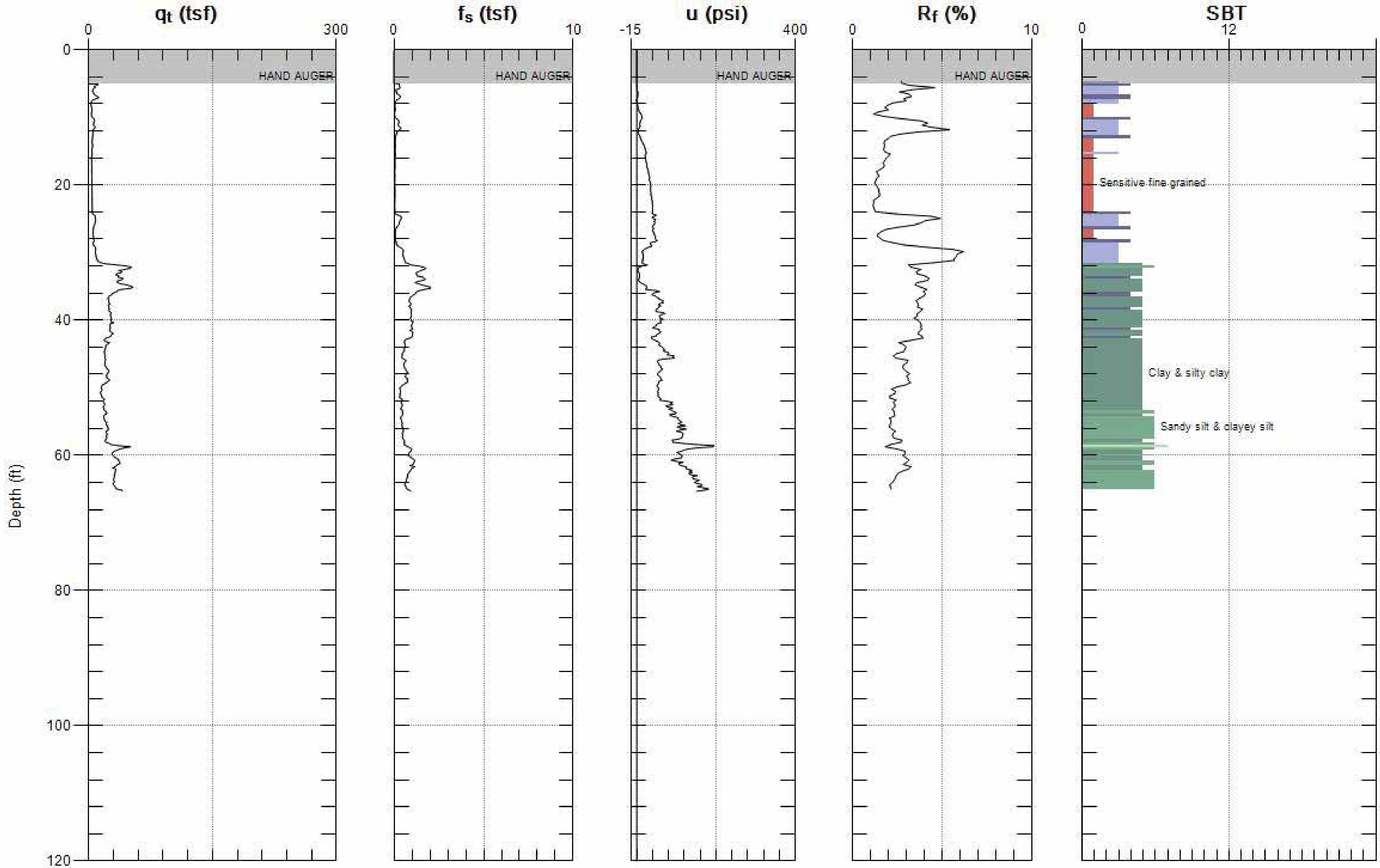
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-10

Engineer: C.COUTU

Date: 10/13/2015 11:22



Max. Depth: 65.289 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

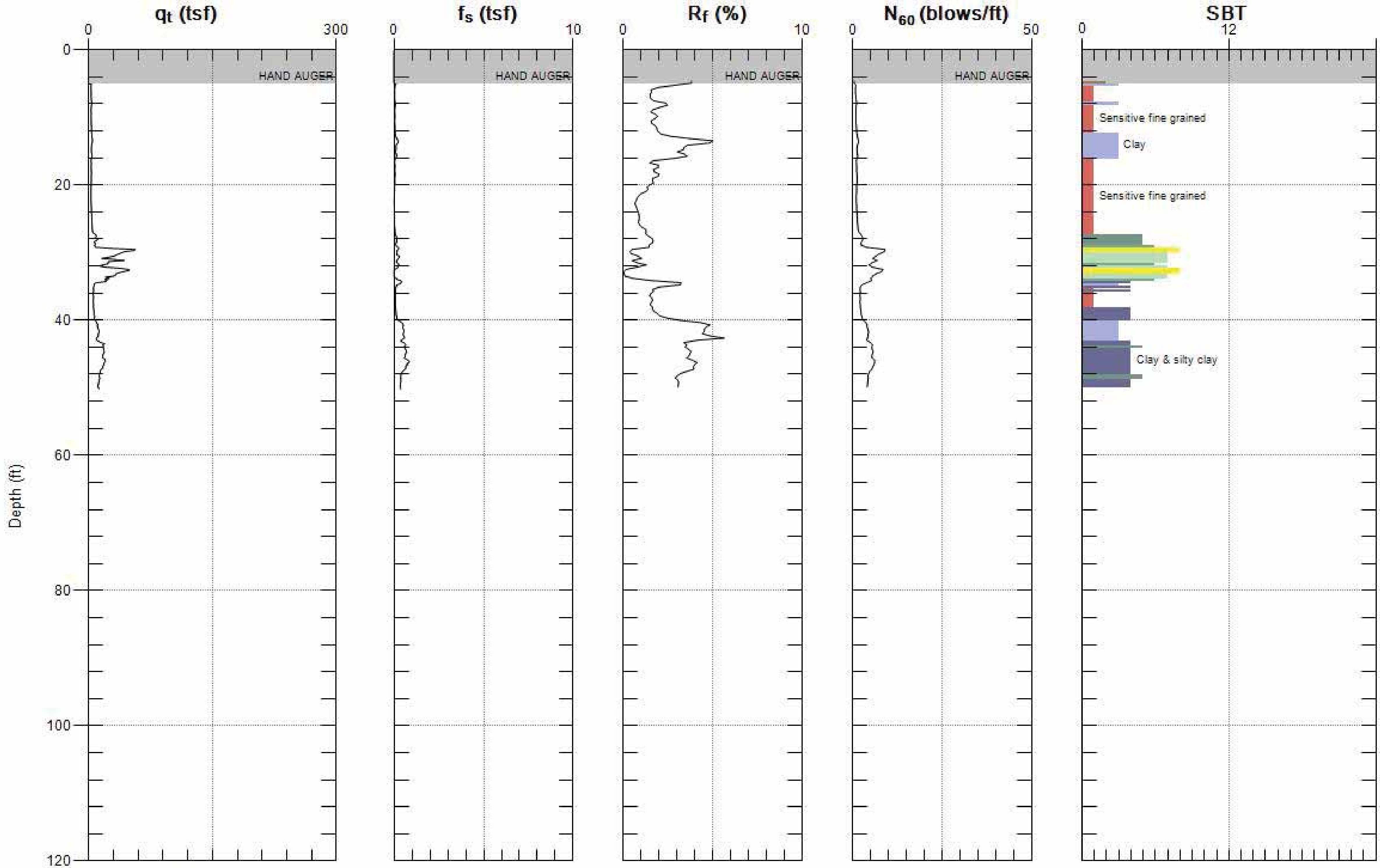
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-11

Engineer: C.COUTU

Date: 10/17/2015 09:40



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

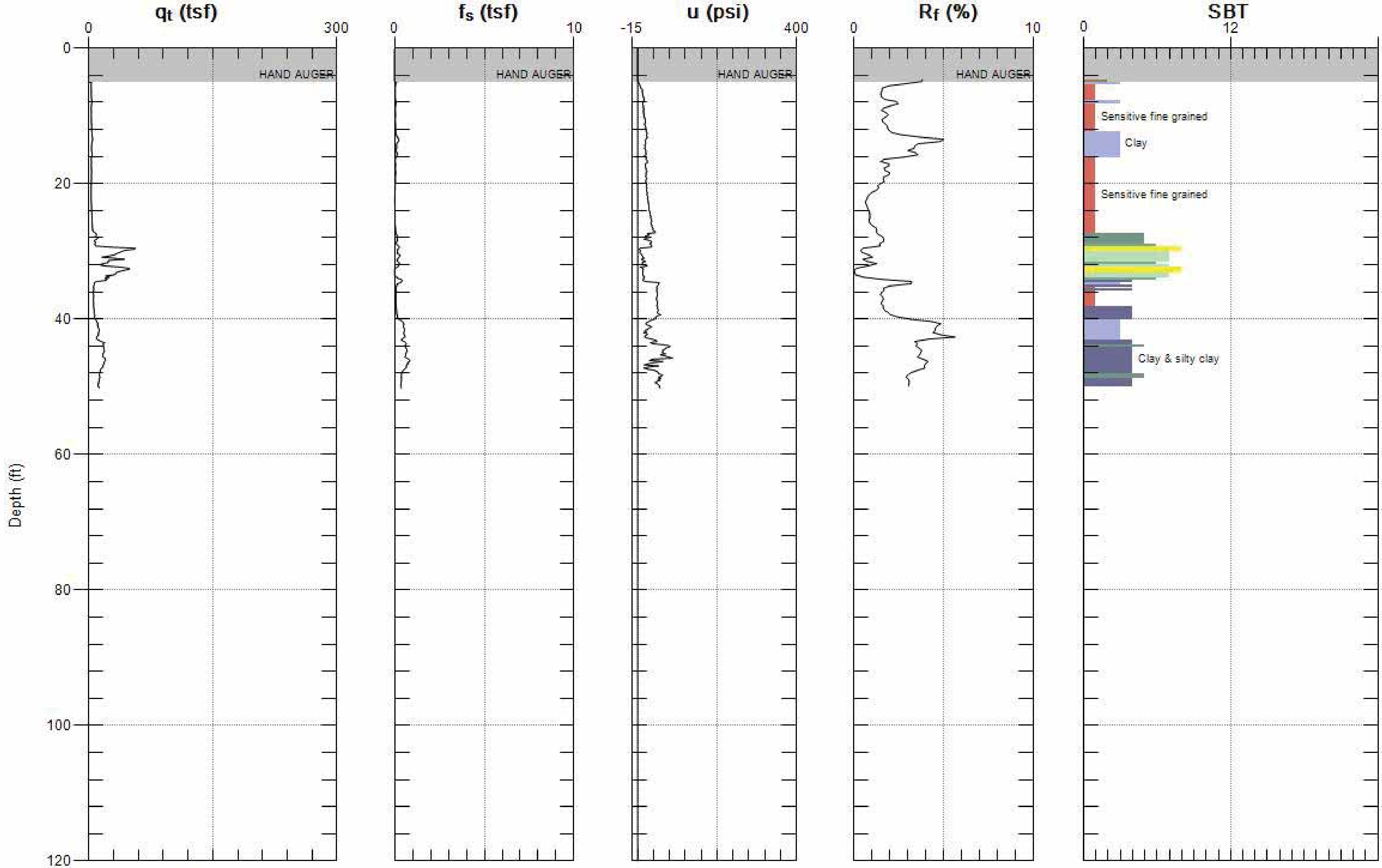
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-11

Engineer: C.COUTU

Date: 10/17/2015 09:40



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

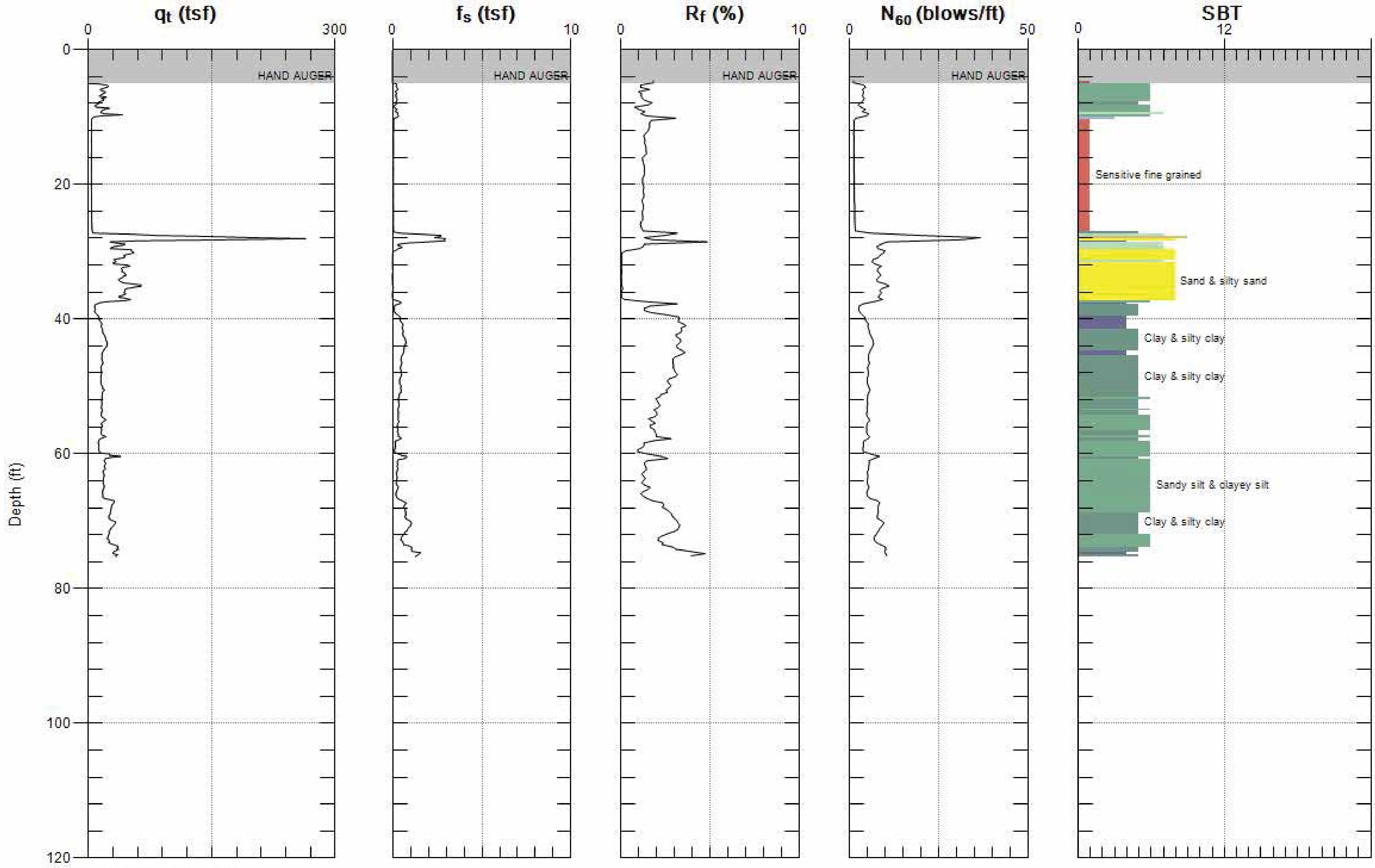
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-12

Engineer: C.COUTU

Date: 10/12/2015 07:40



Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

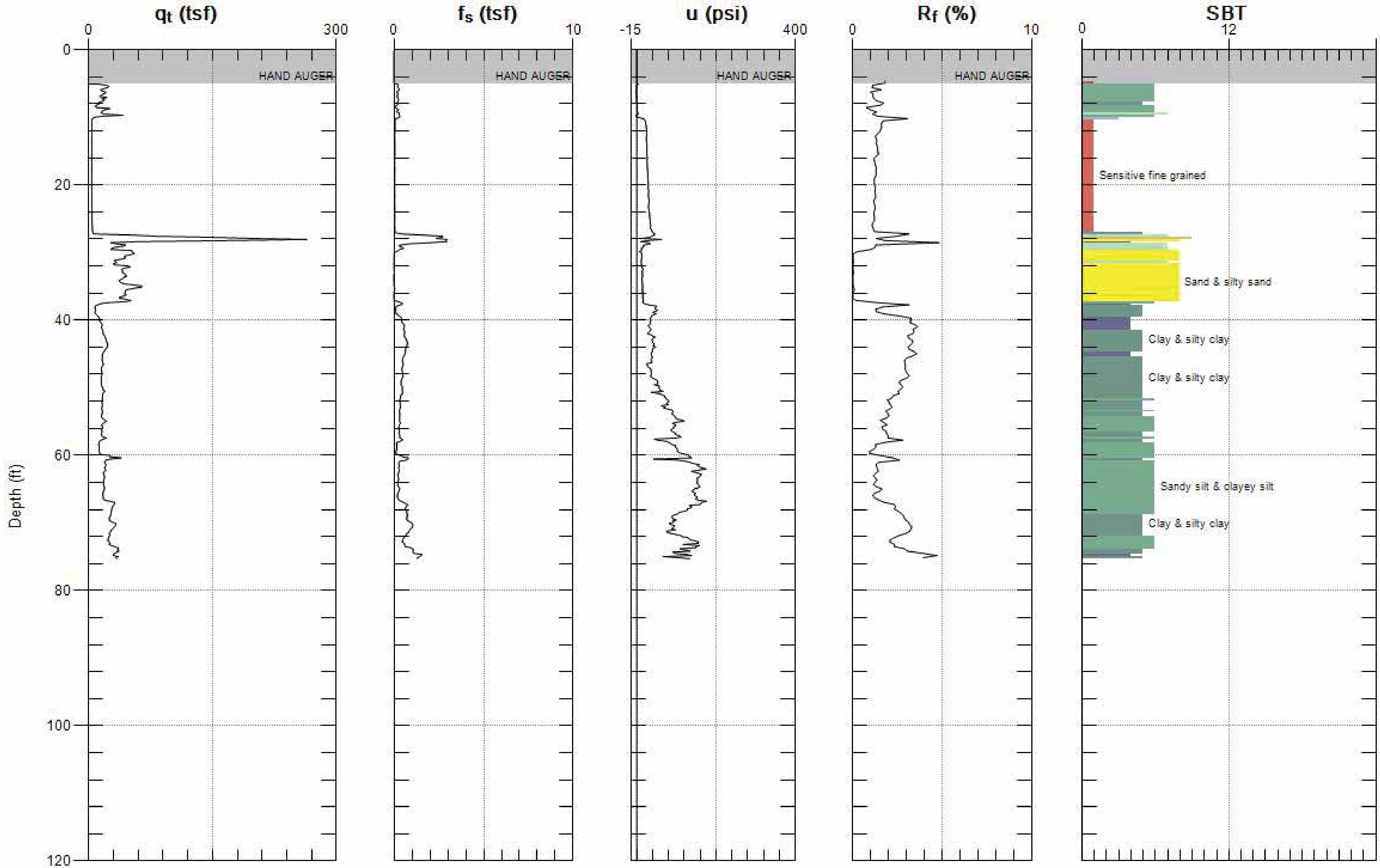
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-12

Engineer: C.COUTU

Date: 10/12/2015 07:40



Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

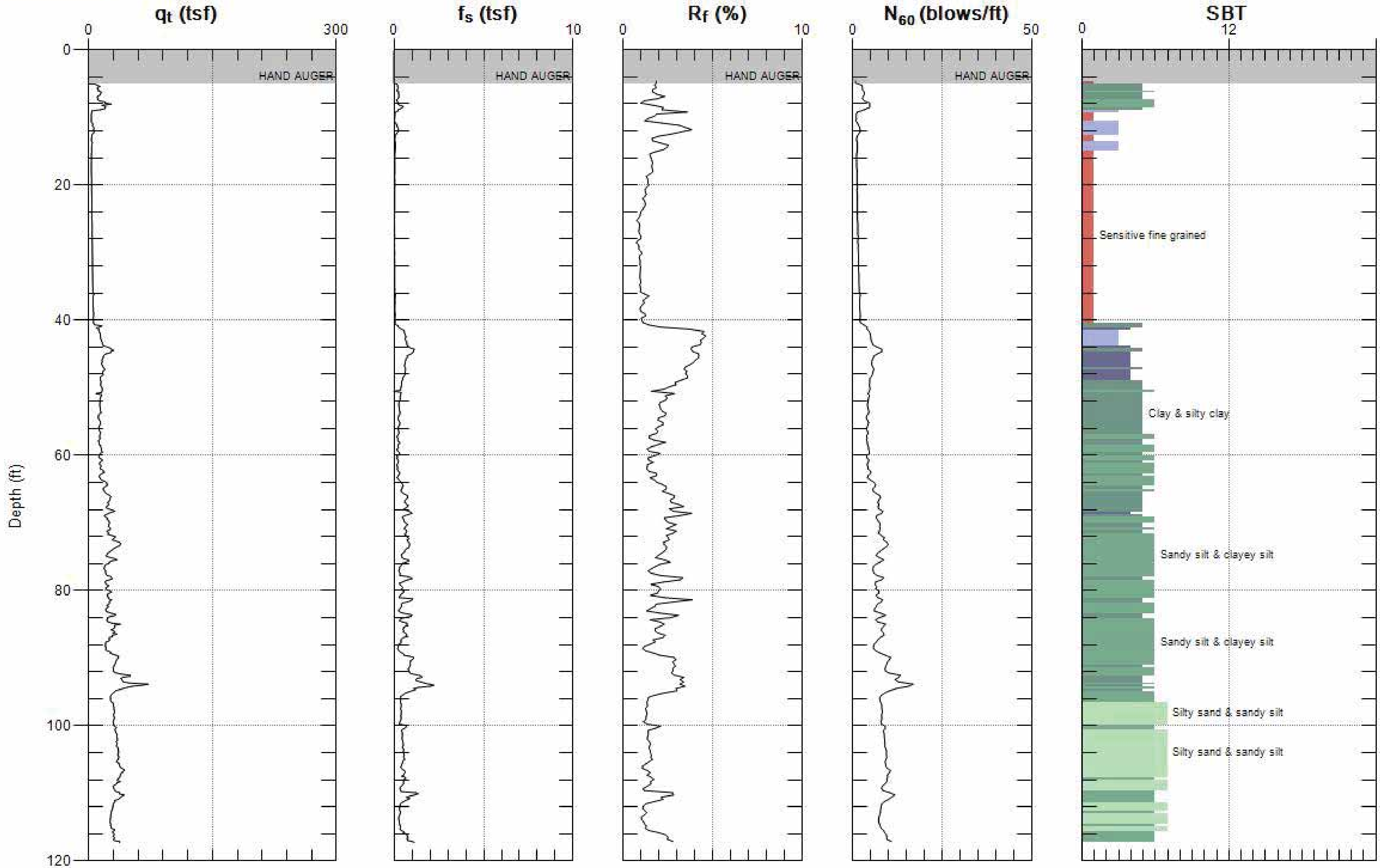
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-13

Engineer: C.COUTU
Date: 10/14/2015 01:41



Max. Depth: 117.290 (ft)
Avg. Interval: 0.328 (ft)

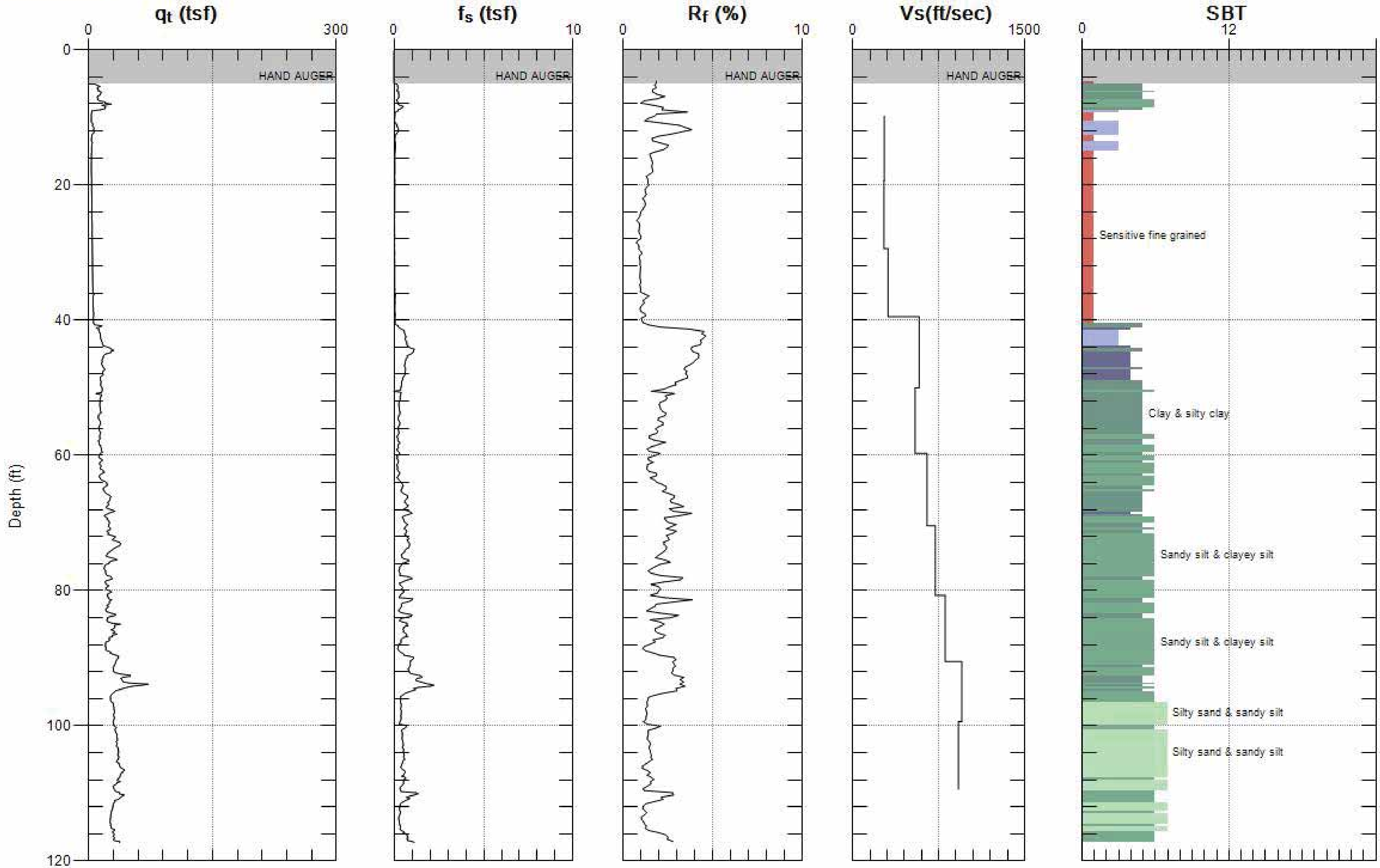
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-13

Engineer: C.COUTU
Date: 10/14/2015 01:41



Max. Depth: 117.290 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

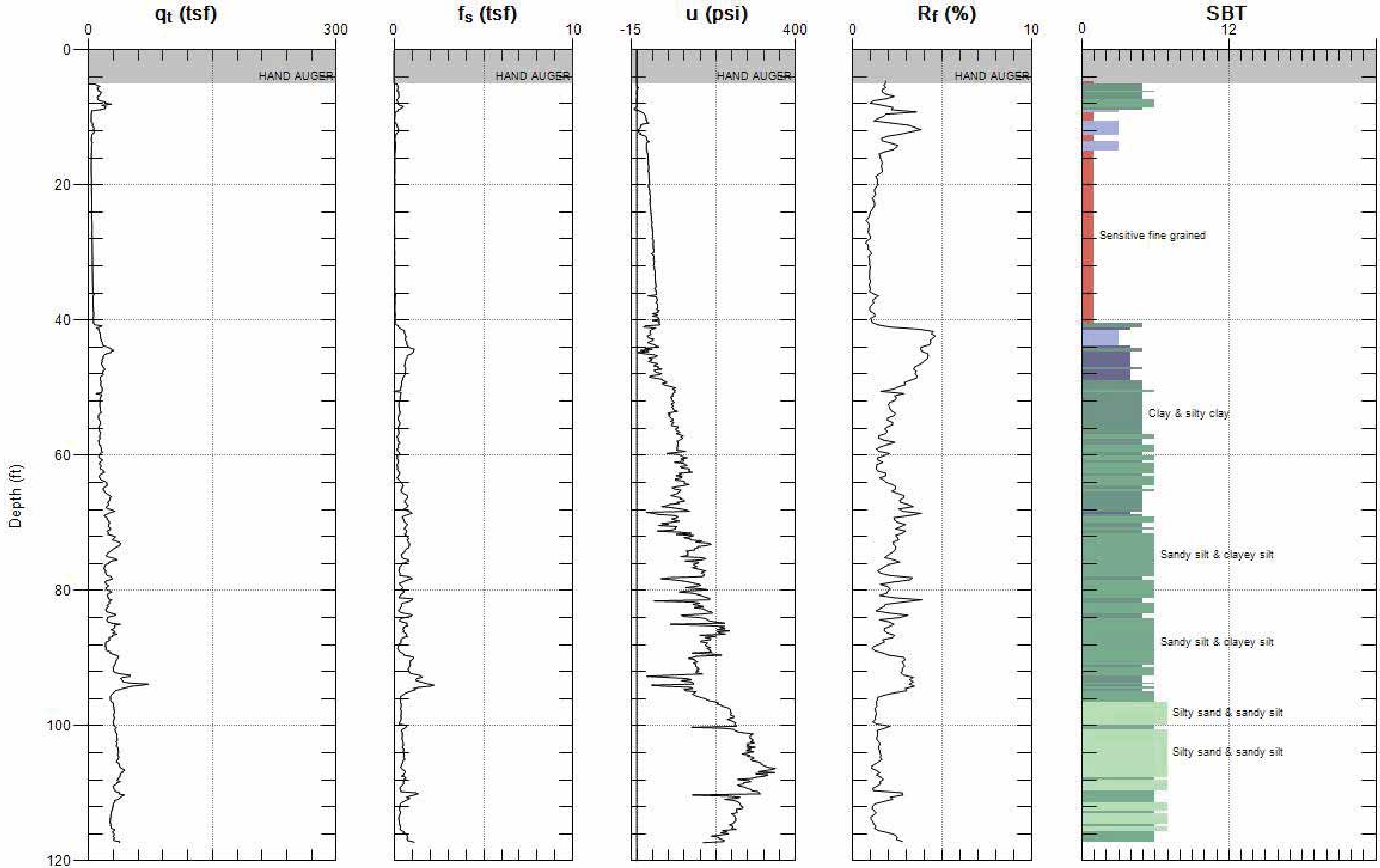
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-13

Engineer: C.COUTU

Date: 10/14/2015 01:41



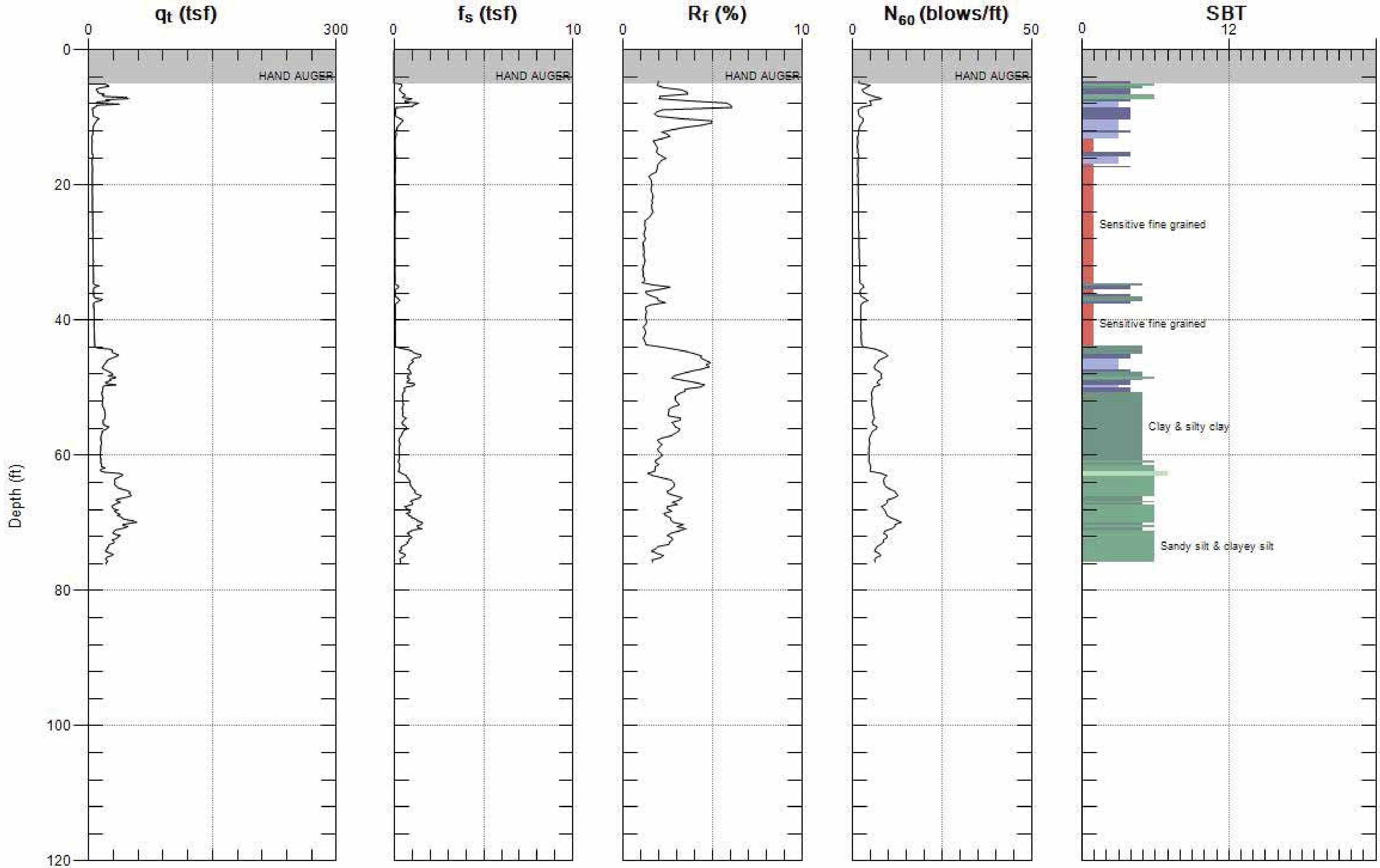
Max. Depth: 117.290 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **AMEC FOSTER WHEELER**

Site: PROJECT ZEUS | Engineer: C.COUTU

Sounding: CPT-14 | Date: 10/15/2015 07:51



Max. Depth: 76.115 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

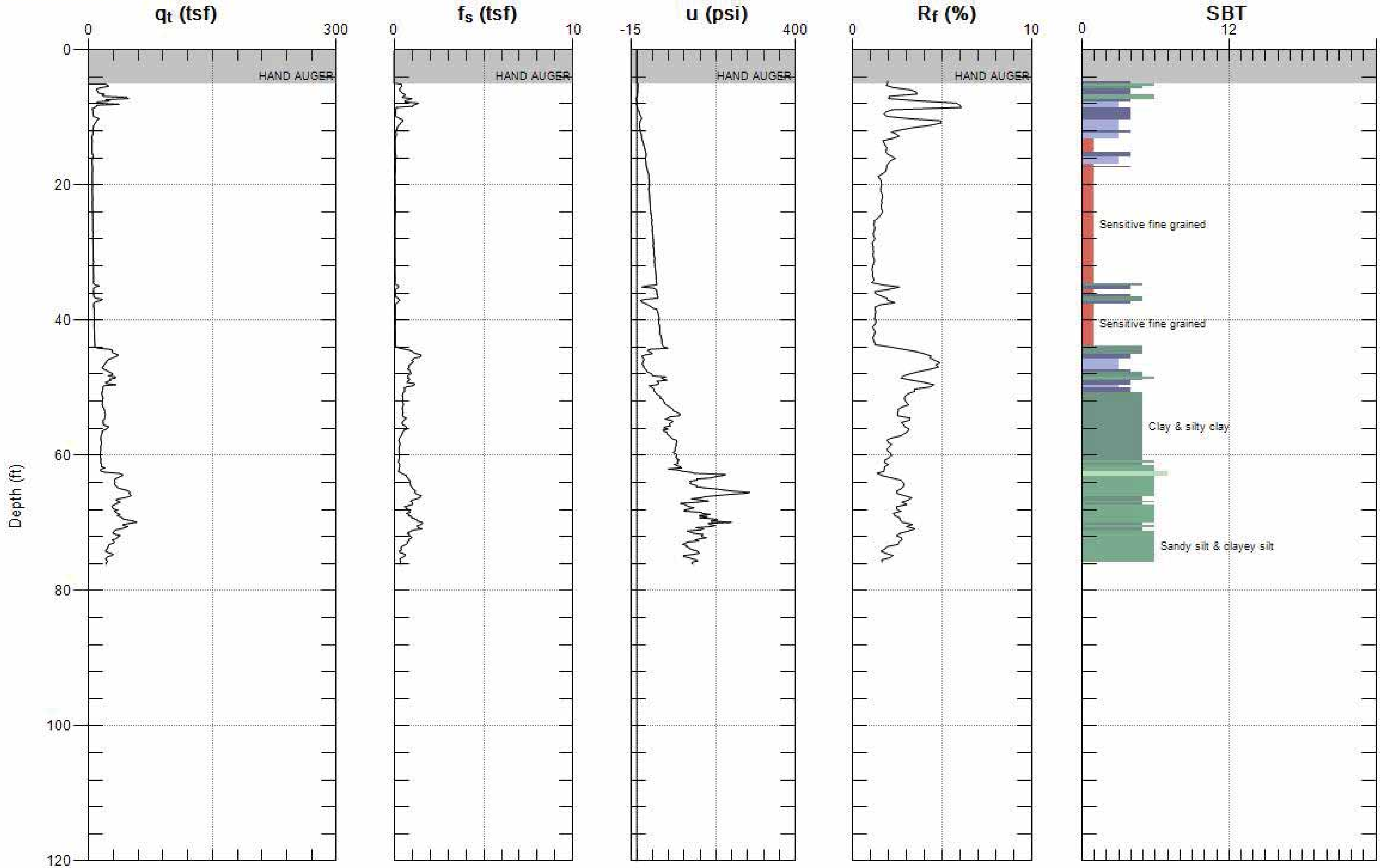
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-14

Engineer: C.COUTU

Date: 10/15/2015 07:51



Max. Depth: 76.115 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

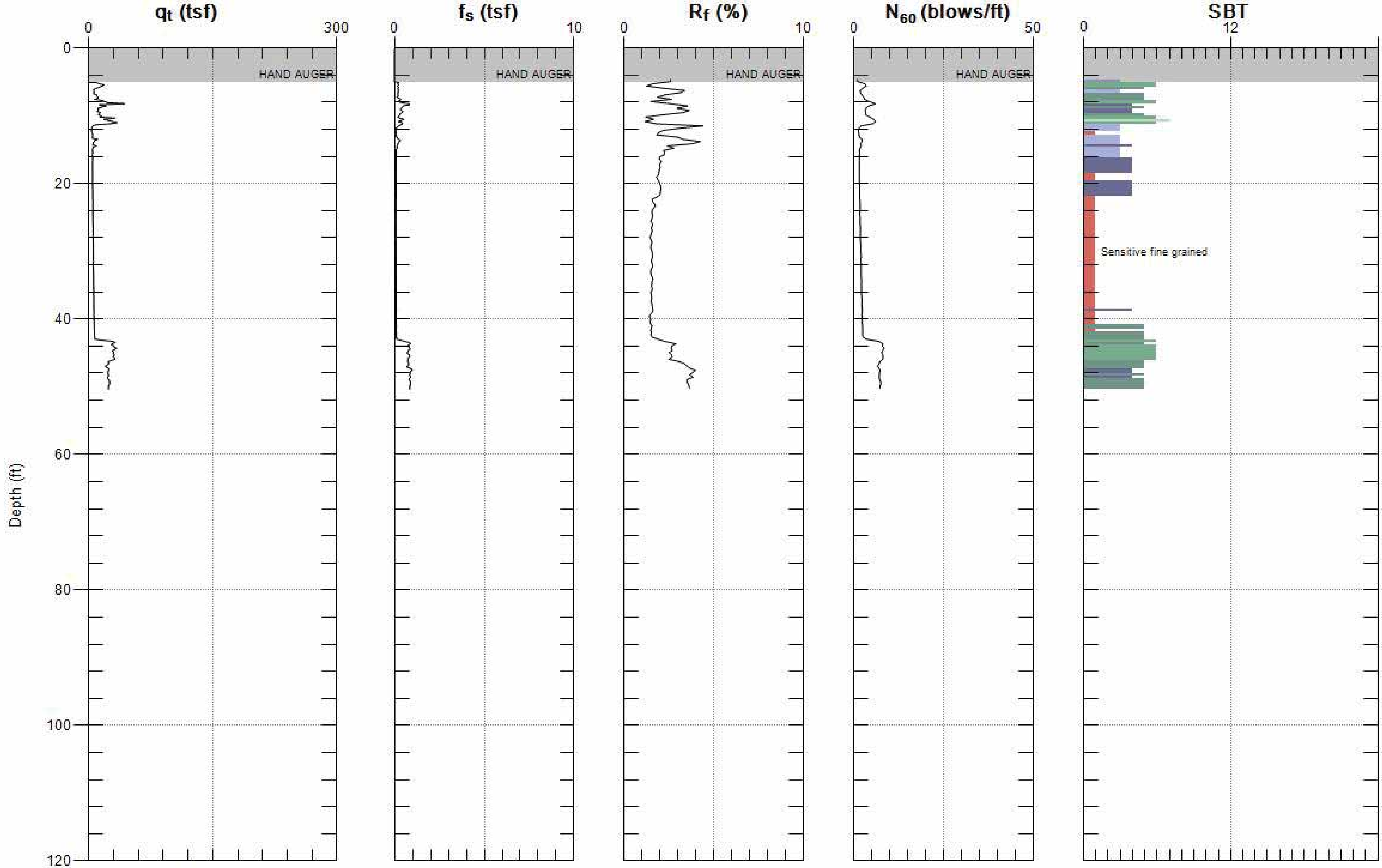
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-15

Engineer: C.COUTU

Date: 12/4/2015 07:33



Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

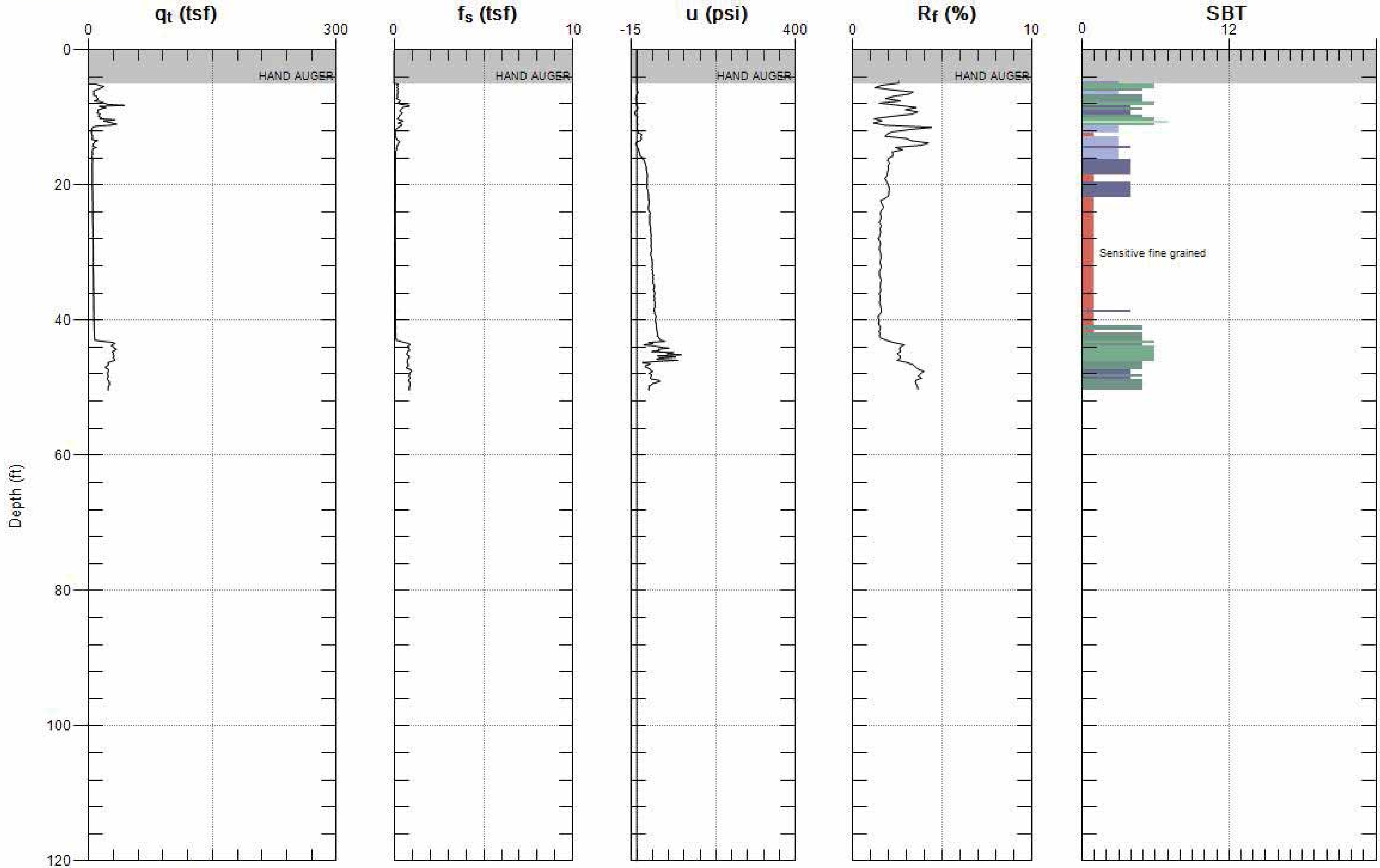
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-15

Engineer: C.COUTU

Date: 12/4/2015 07:33



Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

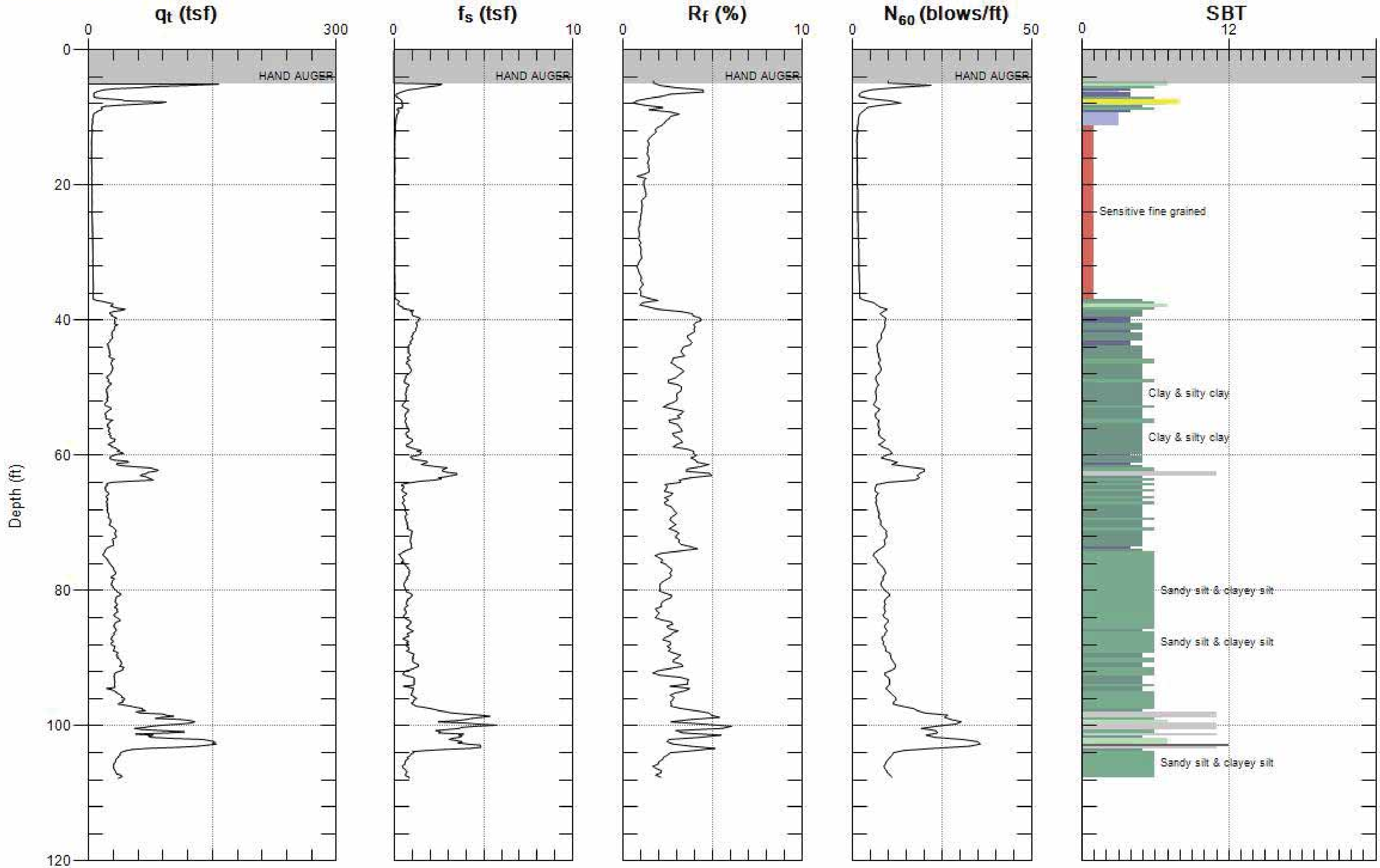
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-16

Engineer: C.COUTU

Date: 10/15/2015 02:01



Max. Depth: 107.776 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

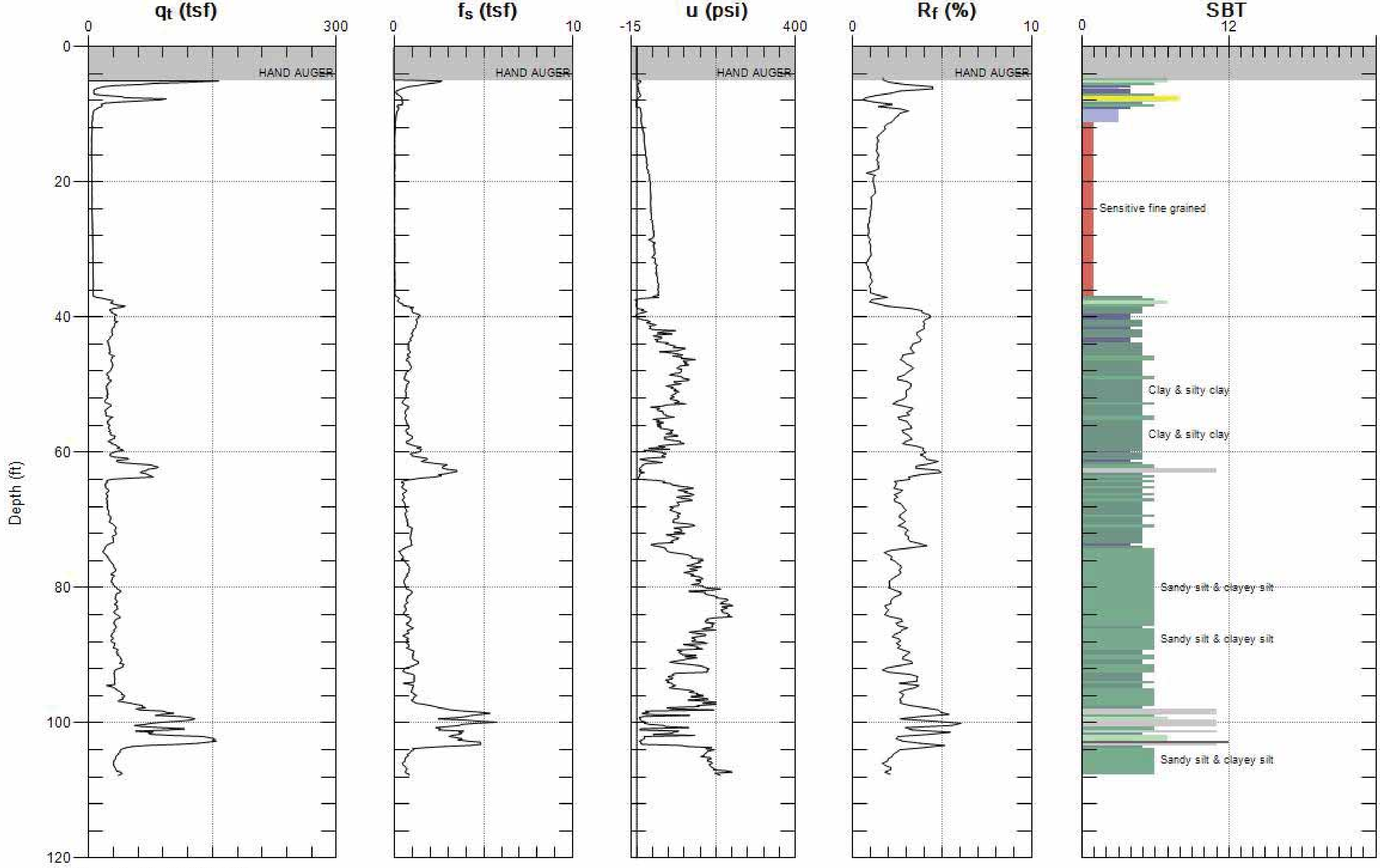
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-16

Engineer: C.COUTU

Date: 10/15/2015 02:01



Max. Depth: 107.776 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

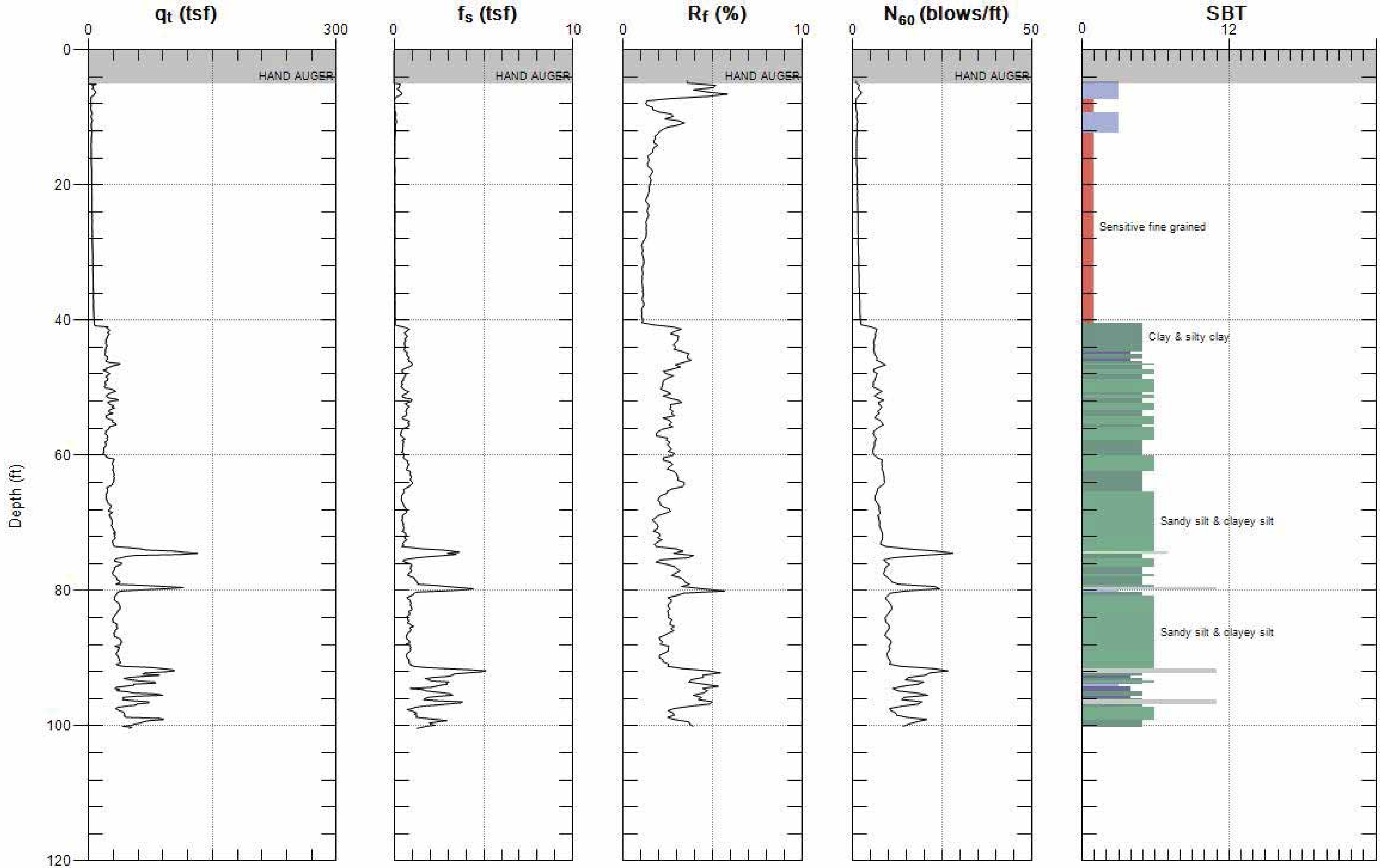
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-17

Engineer: C.COUTU

Date: 10/15/2015 11:35

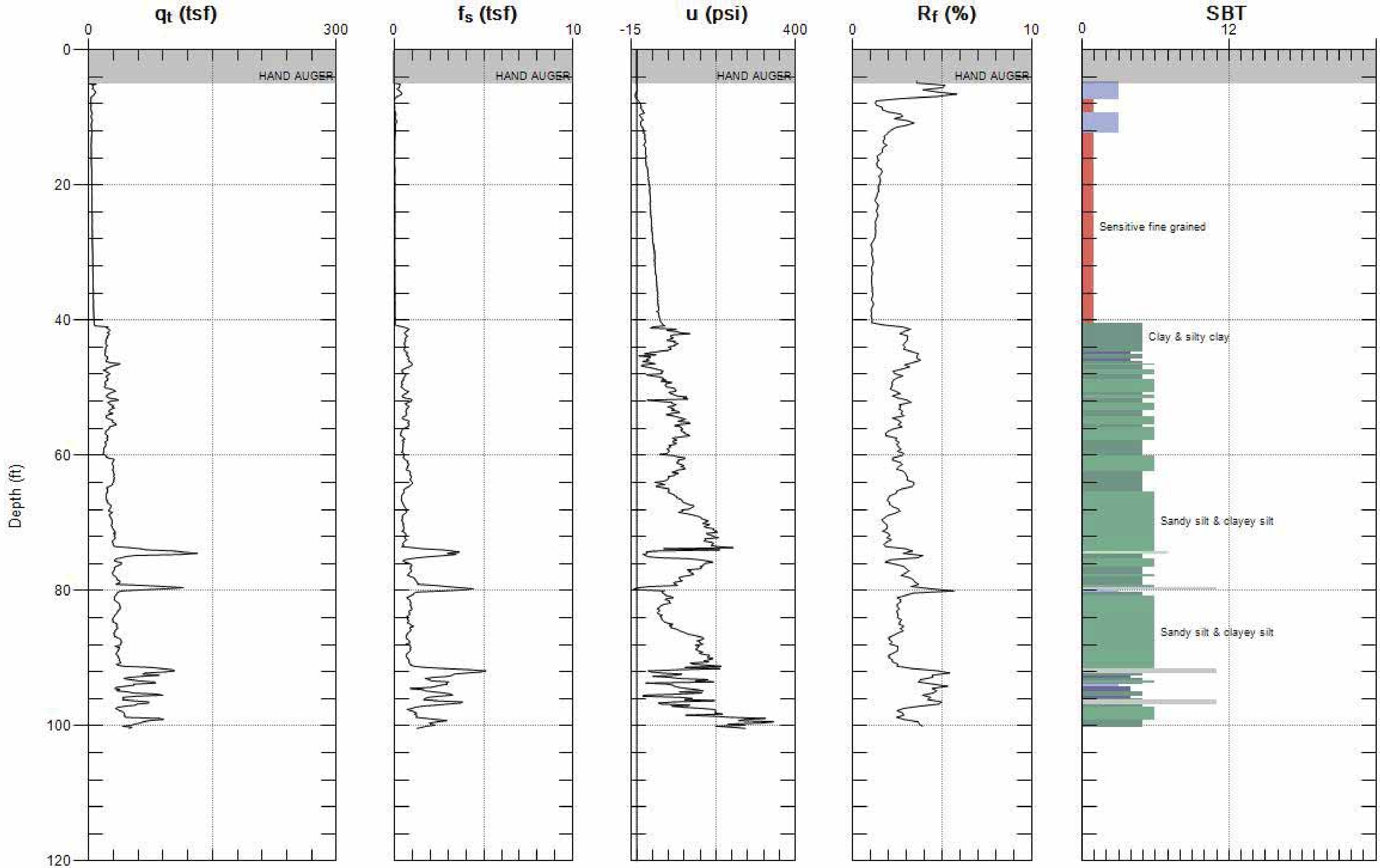


Max. Depth: 100.394 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

AMEC FOSTER WHEELER

Site: PROJECT ZEUS | Engineer: C.COUTU
Sounding: CPT-17 | Date: 10/15/2015 11:35



Max. Depth: 100.394 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

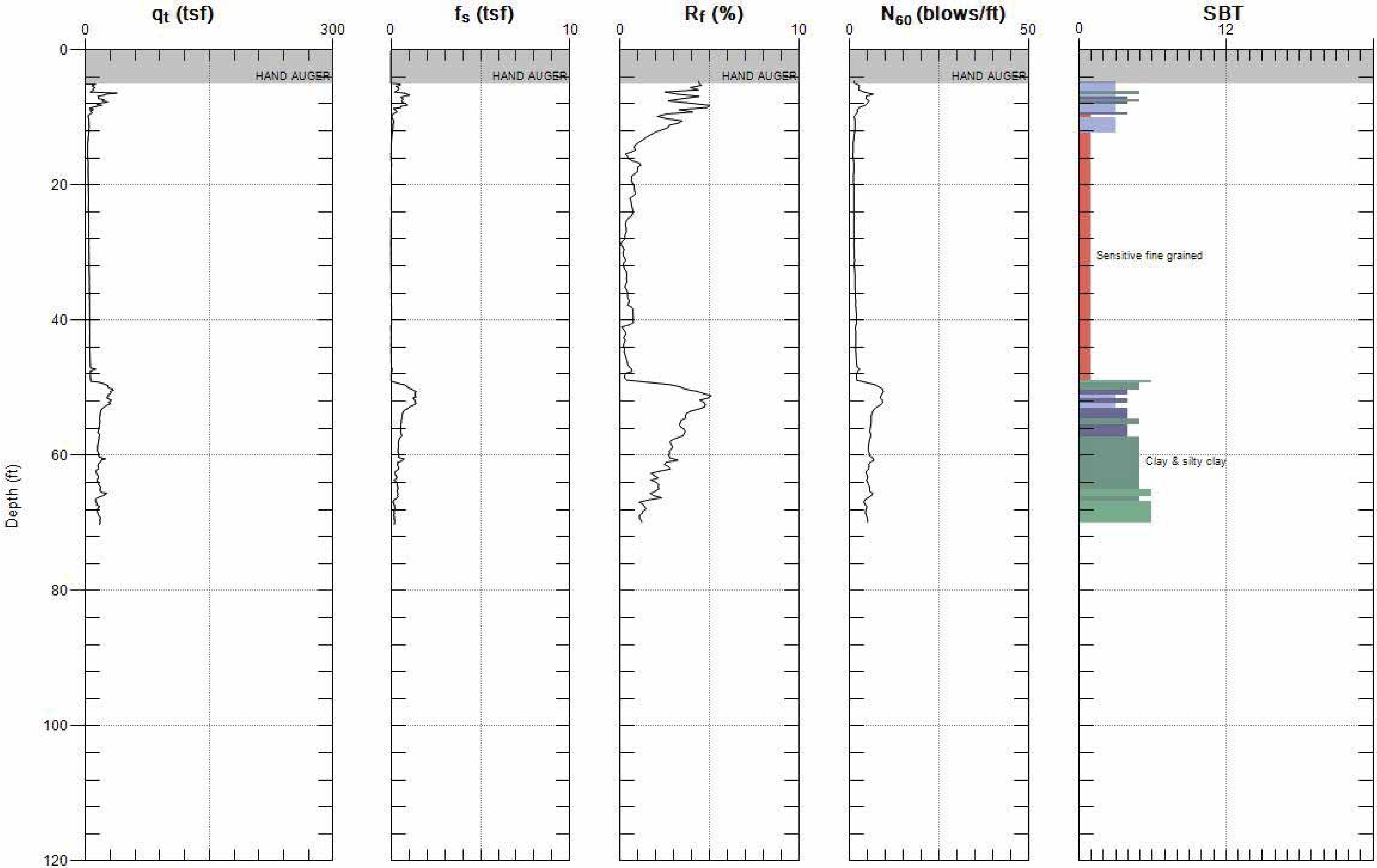
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-18

Engineer: C.COUTU

Date: 10/9/2015 03:25



Max. Depth: 70.210 (ft)
Avg. Interval: 0.328 (ft)

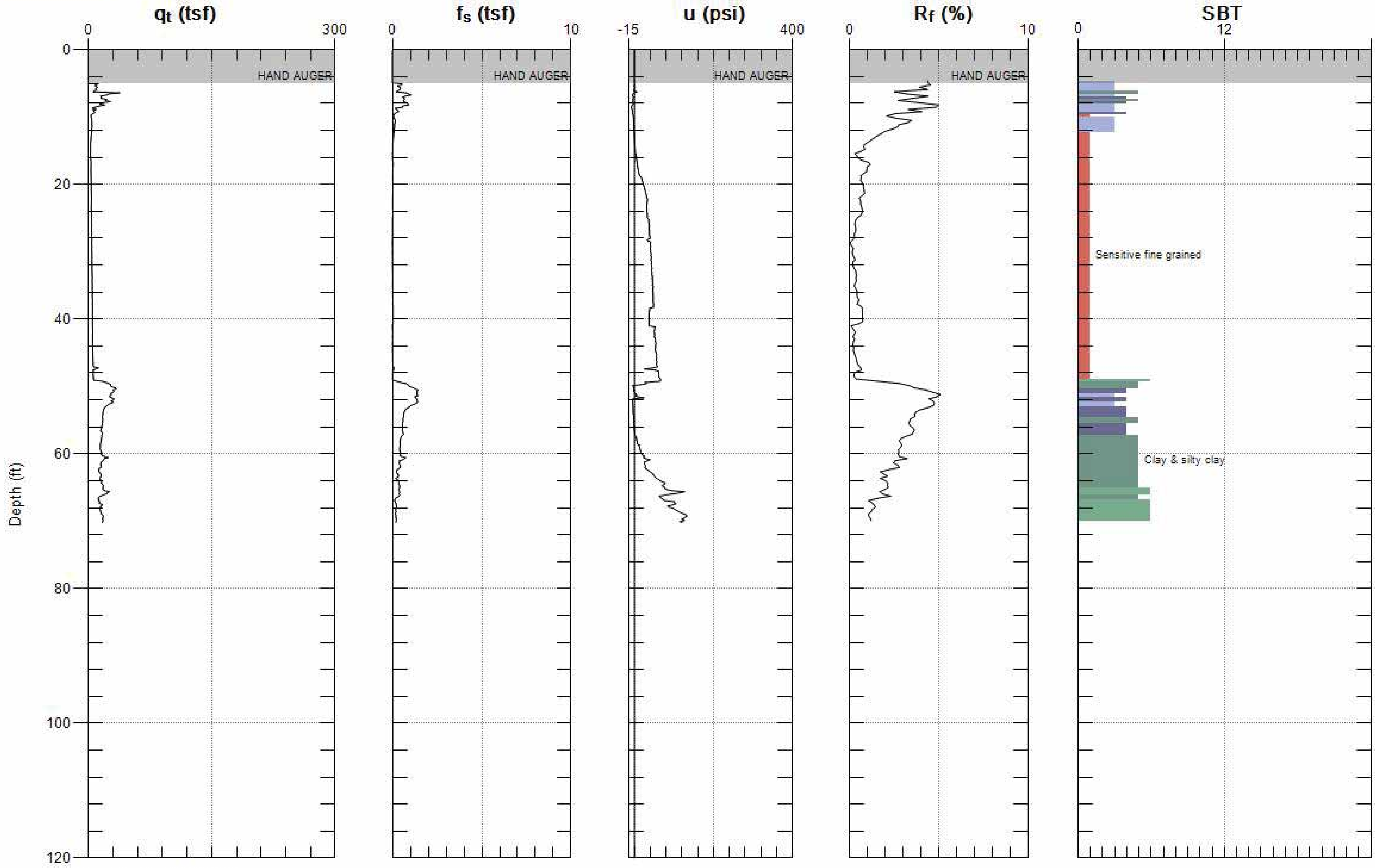
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-18

Engineer: C.COUTU
Date: 10/9/2015 03:25



Max. Depth: 70.210 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

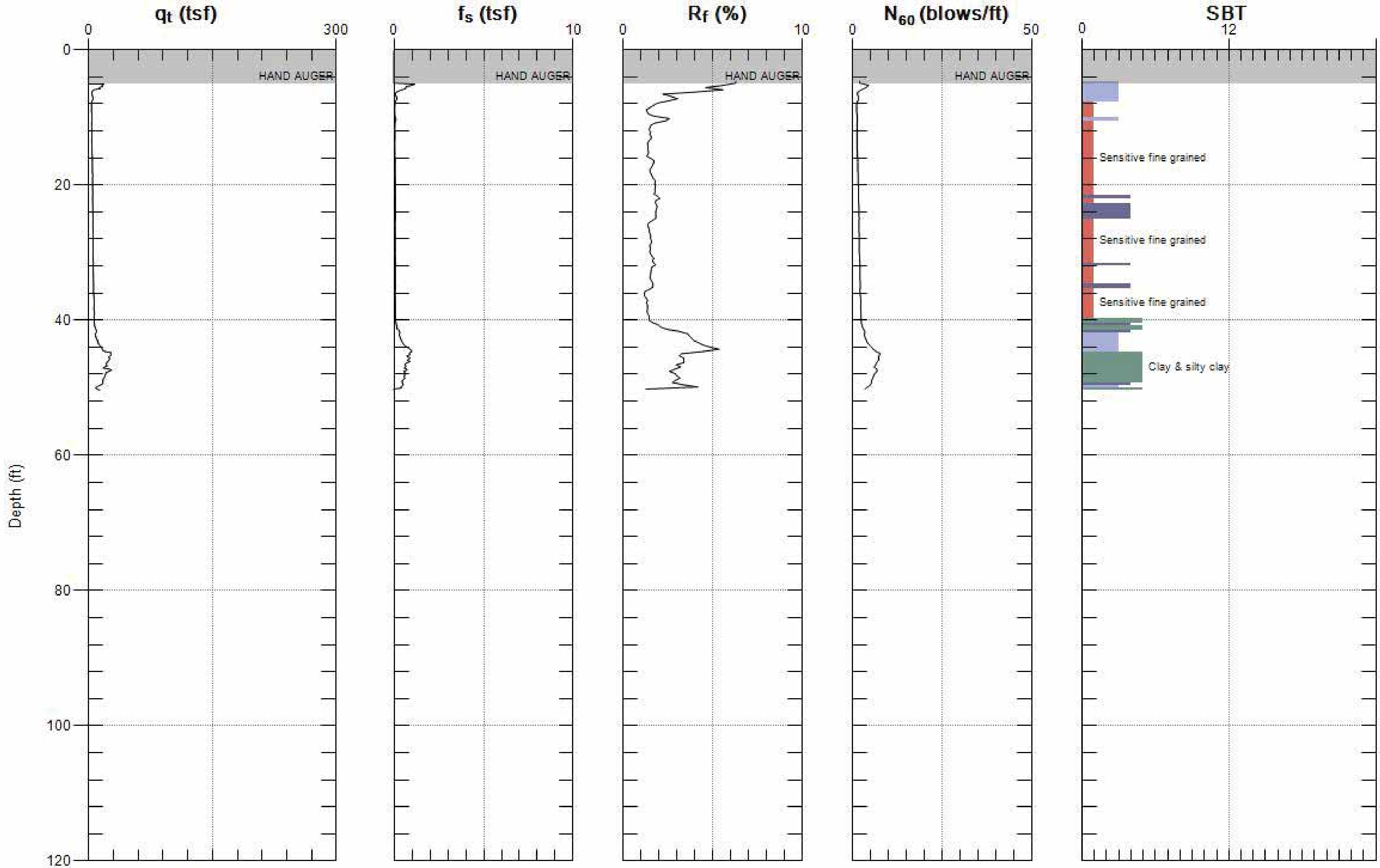
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-19

Engineer: C.COUTU

Date: 10/17/2015 08:18



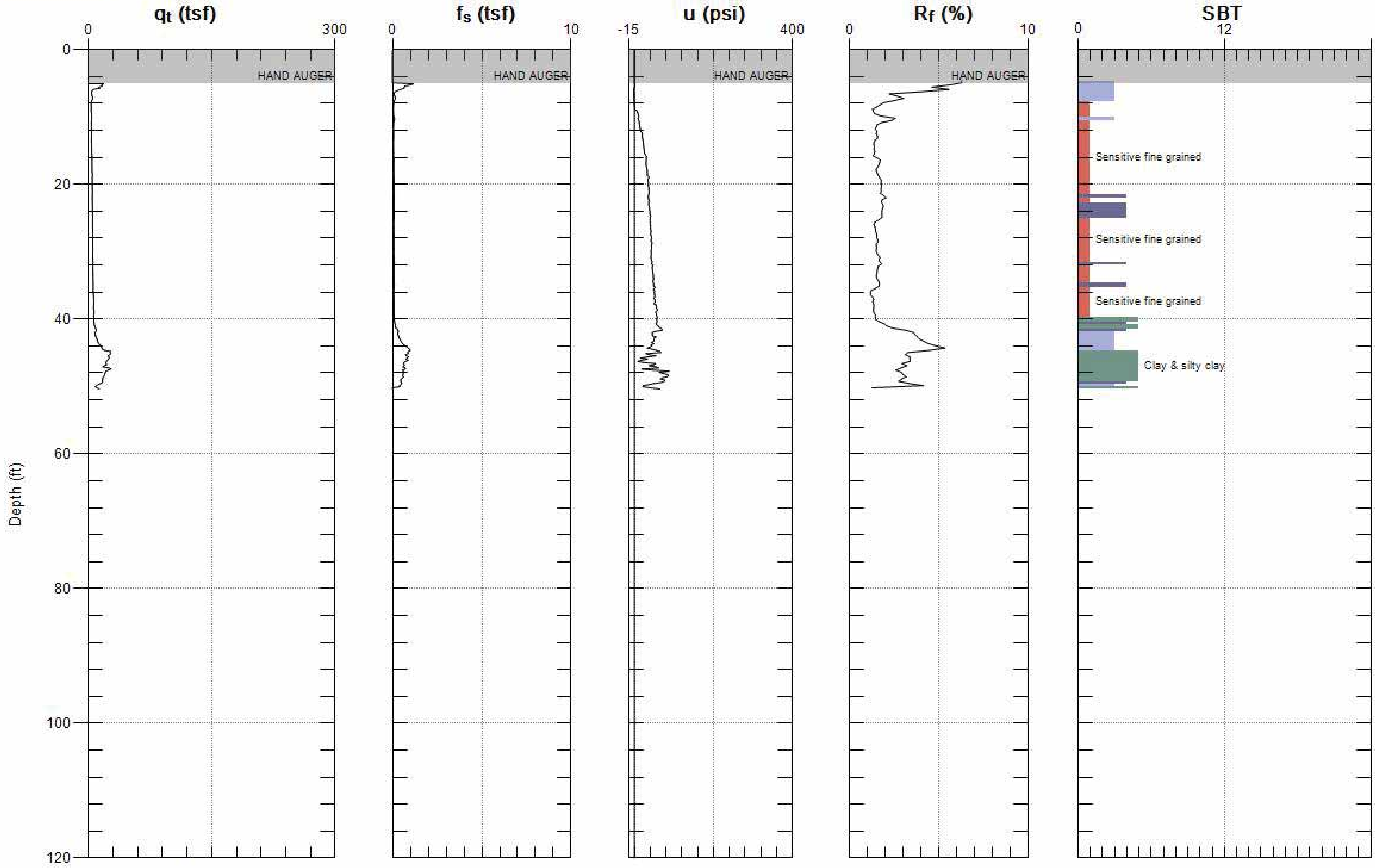
Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-19

Engineer: C.COUTU
Date: 10/17/2015 08:18



Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

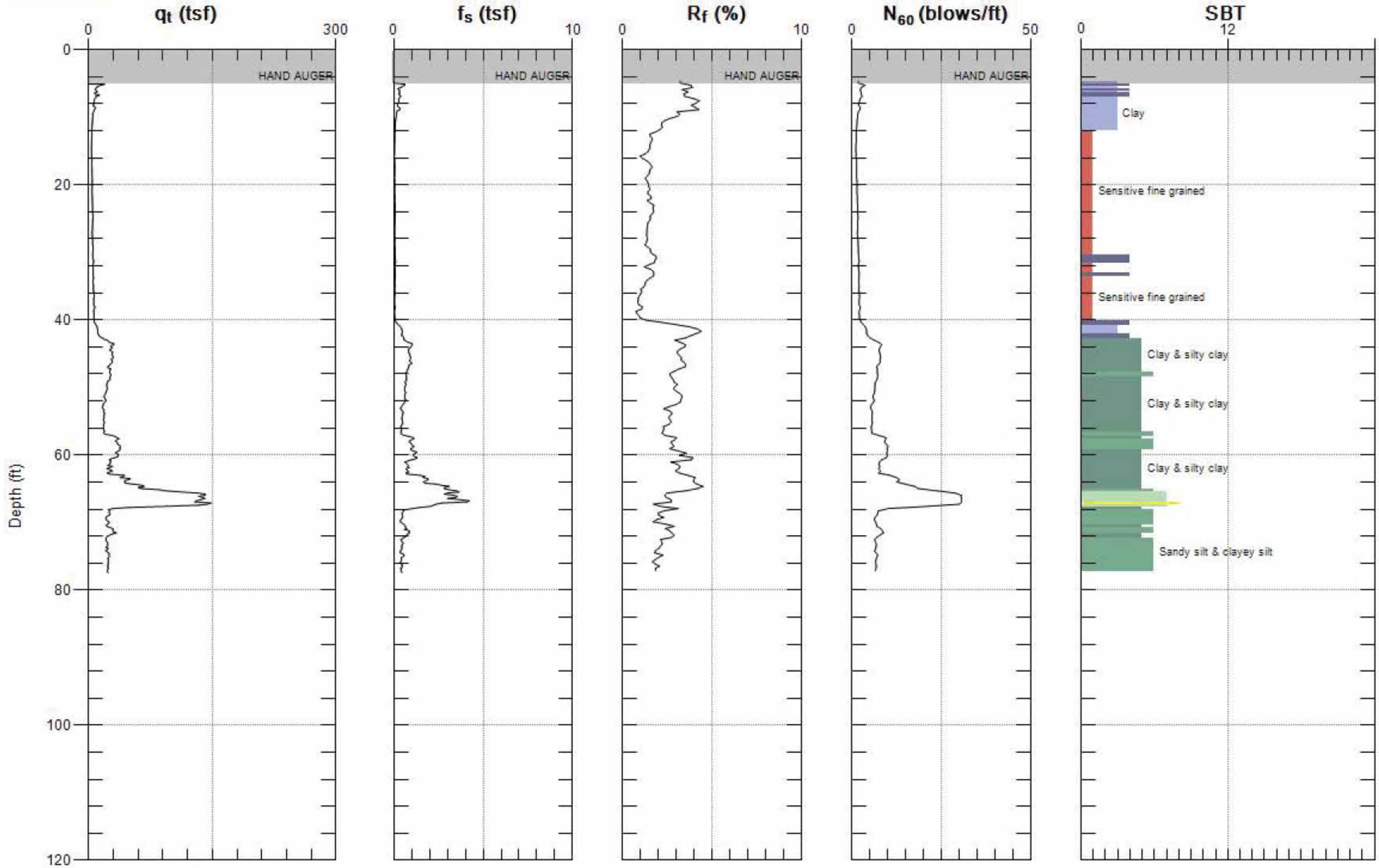
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-20

Engineer: C.COUTU
Date: 10/15/2015 09:42



Max. Depth: 77.428 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

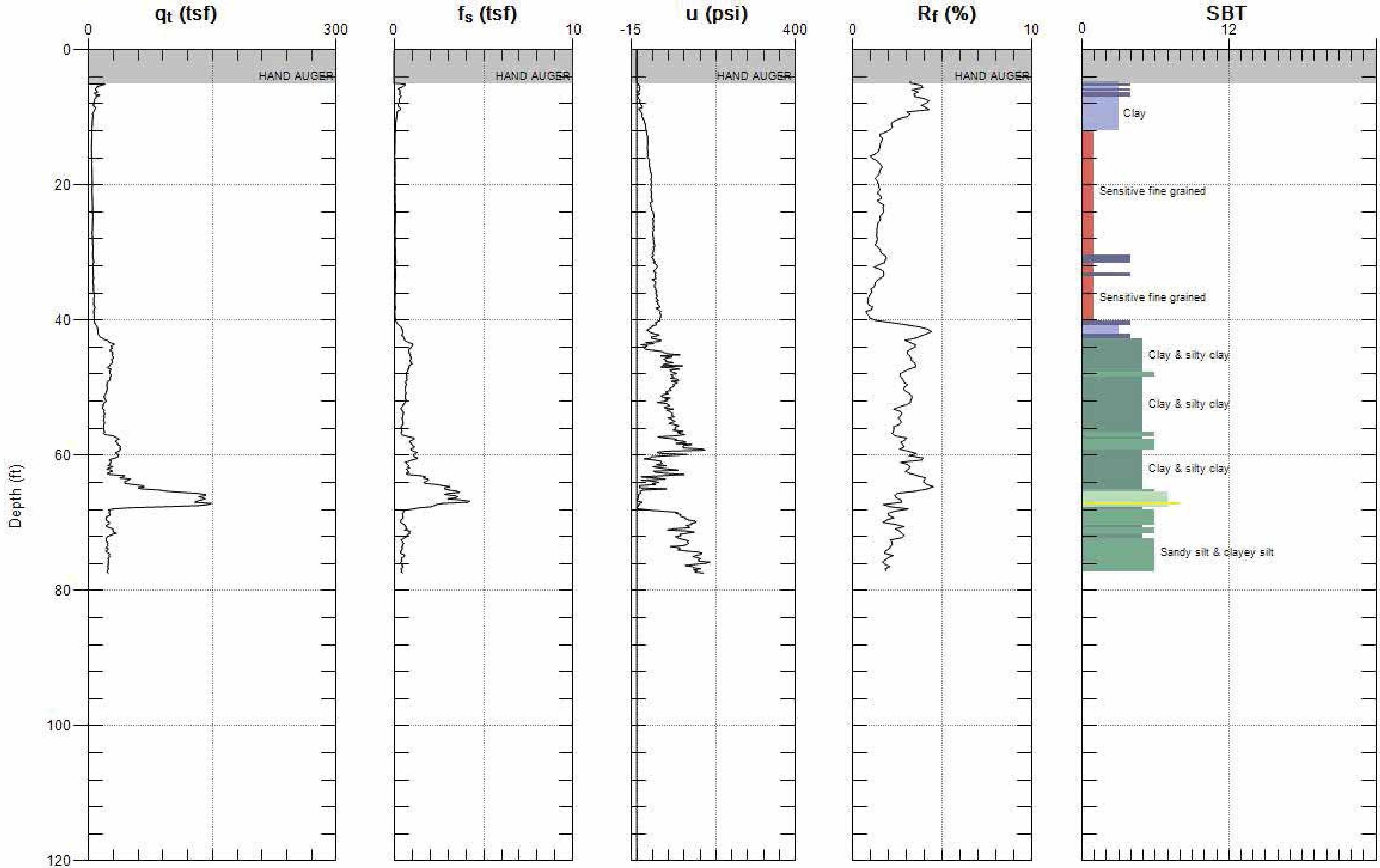
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-20

Engineer: C.COUTU

Date: 10/15/2015 09:42



Max. Depth: 77.428 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

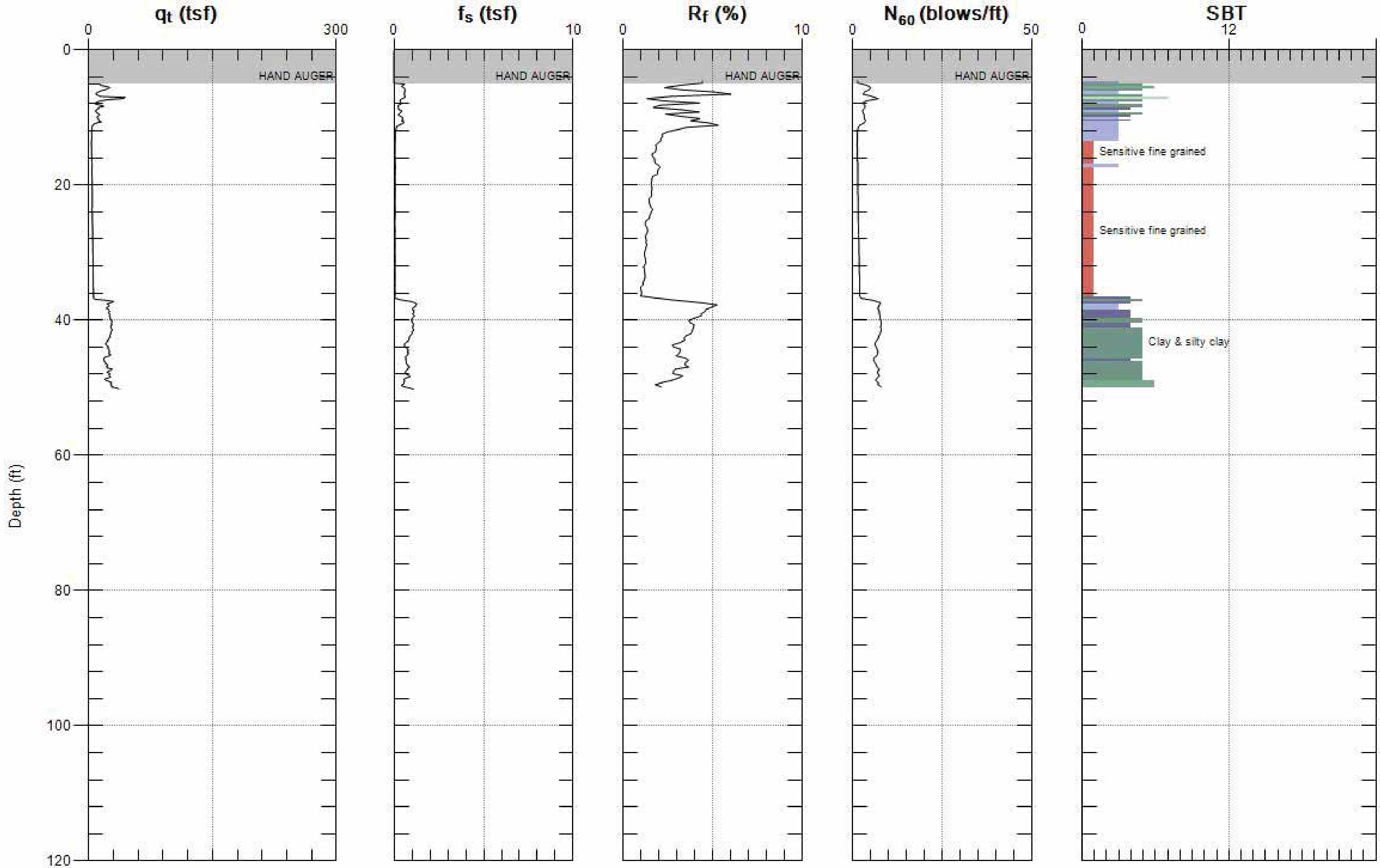
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-21

Engineer: C.COUTU

Date: 10/17/2015 08:17



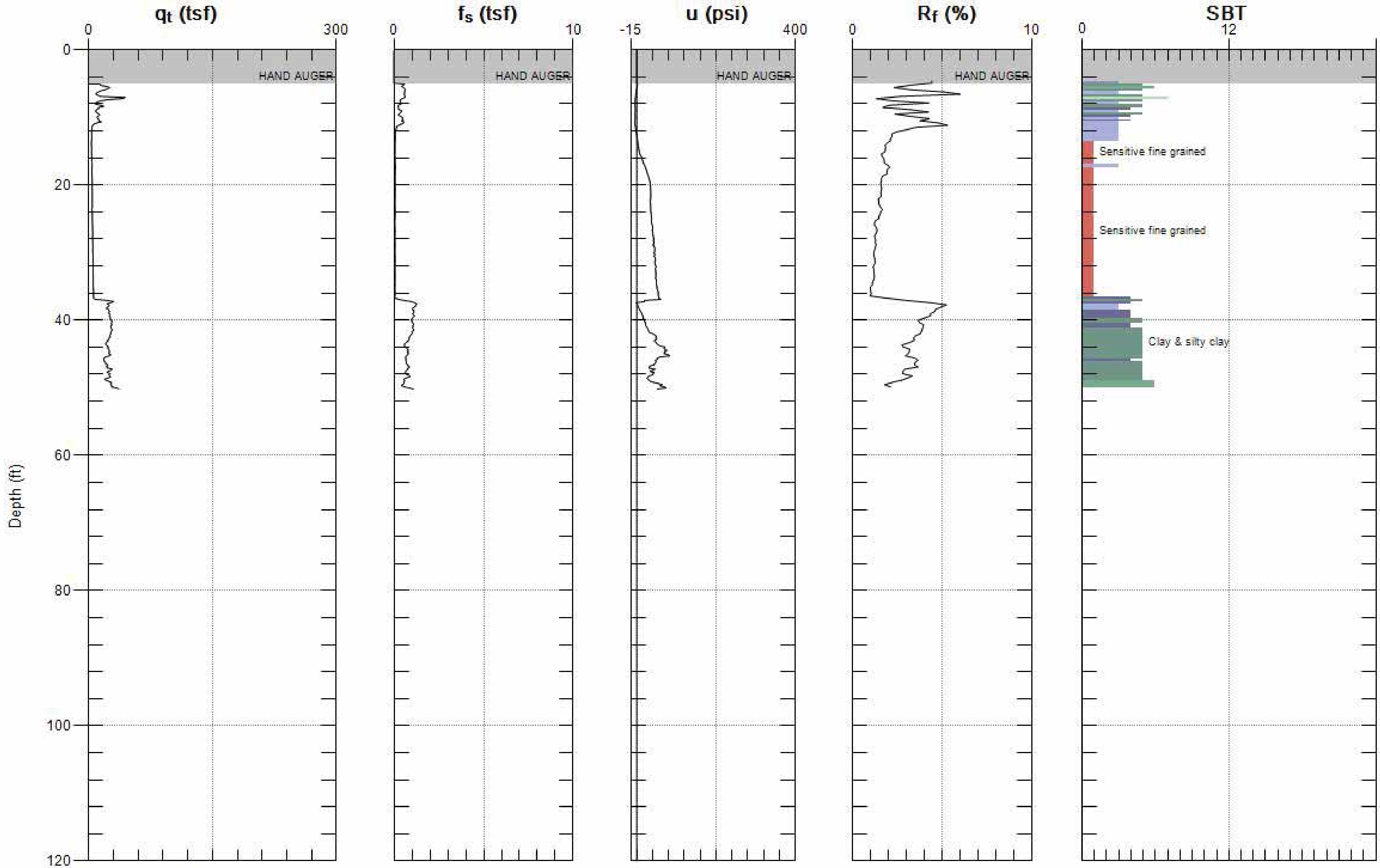
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-21

Engineer: C.COUTU
Date: 10/17/2015 08:17



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

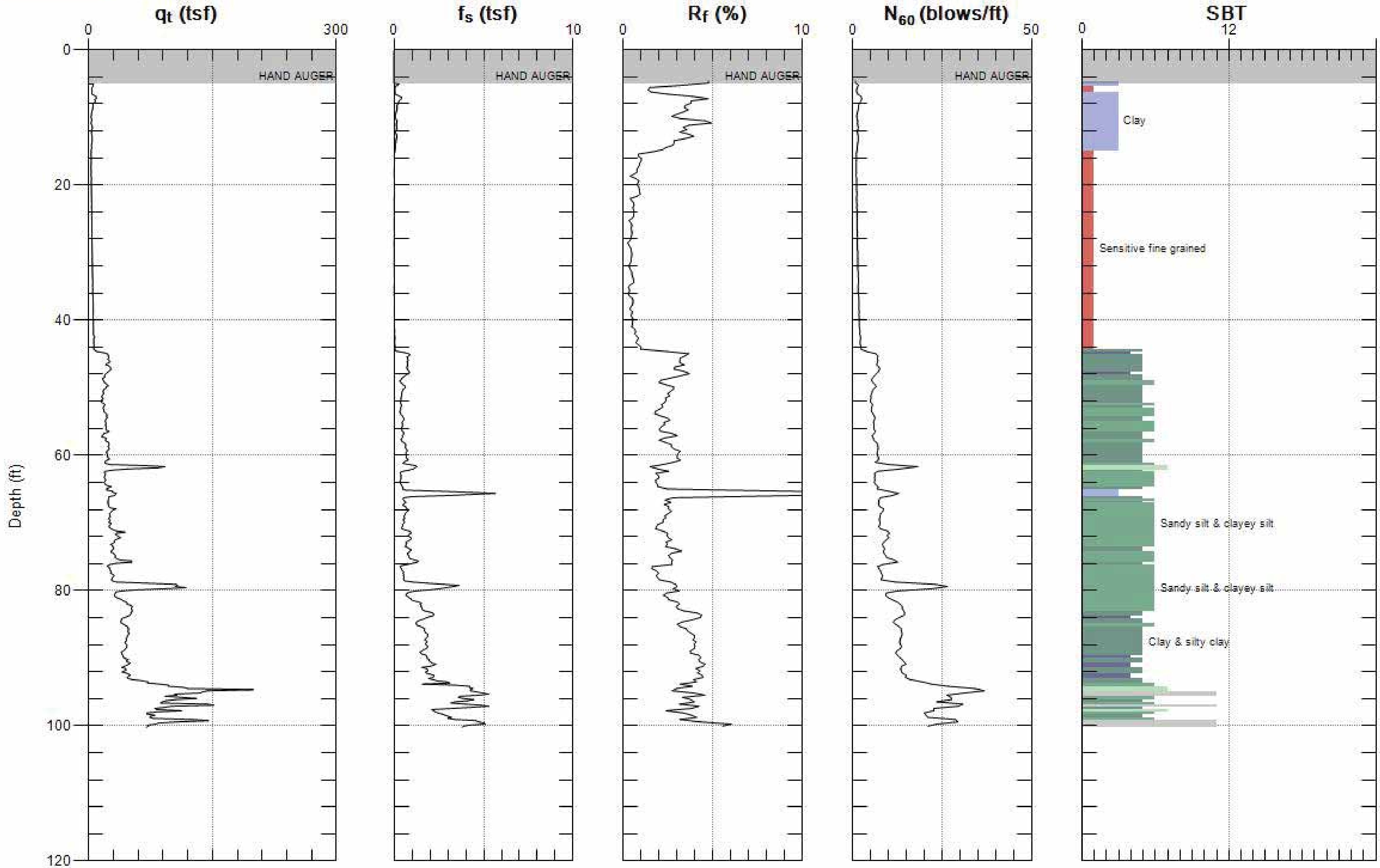
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-22

Engineer: C.COUTU

Date: 10/13/2015 01:05



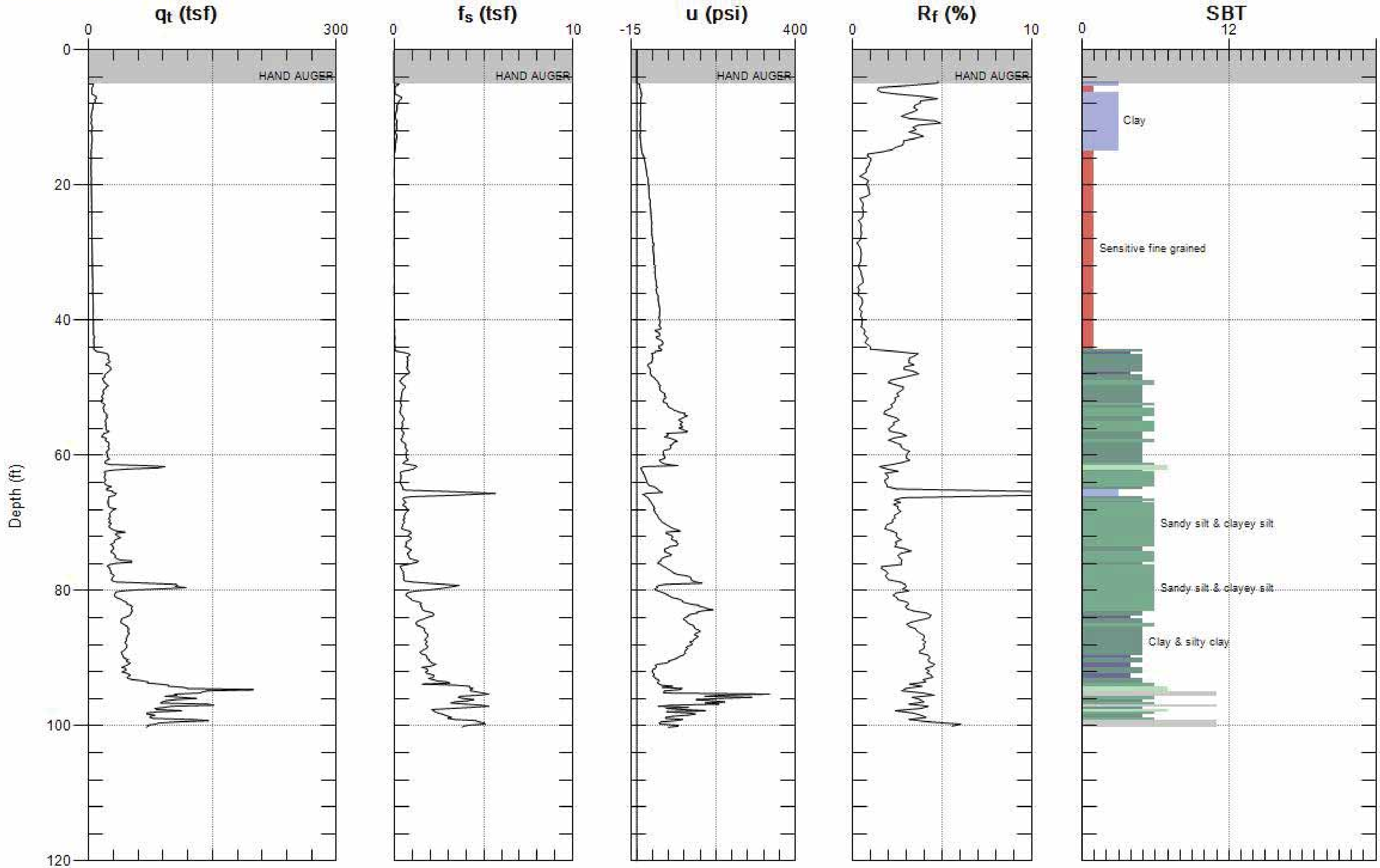
Max. Depth: 100.230 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **AMEC FOSTER WHEELER**

Site: PROJECT ZEUS
Sounding: CPT-22

Engineer: C.COUTU
Date: 10/13/2015 01:05



Max. Depth: 100.230 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

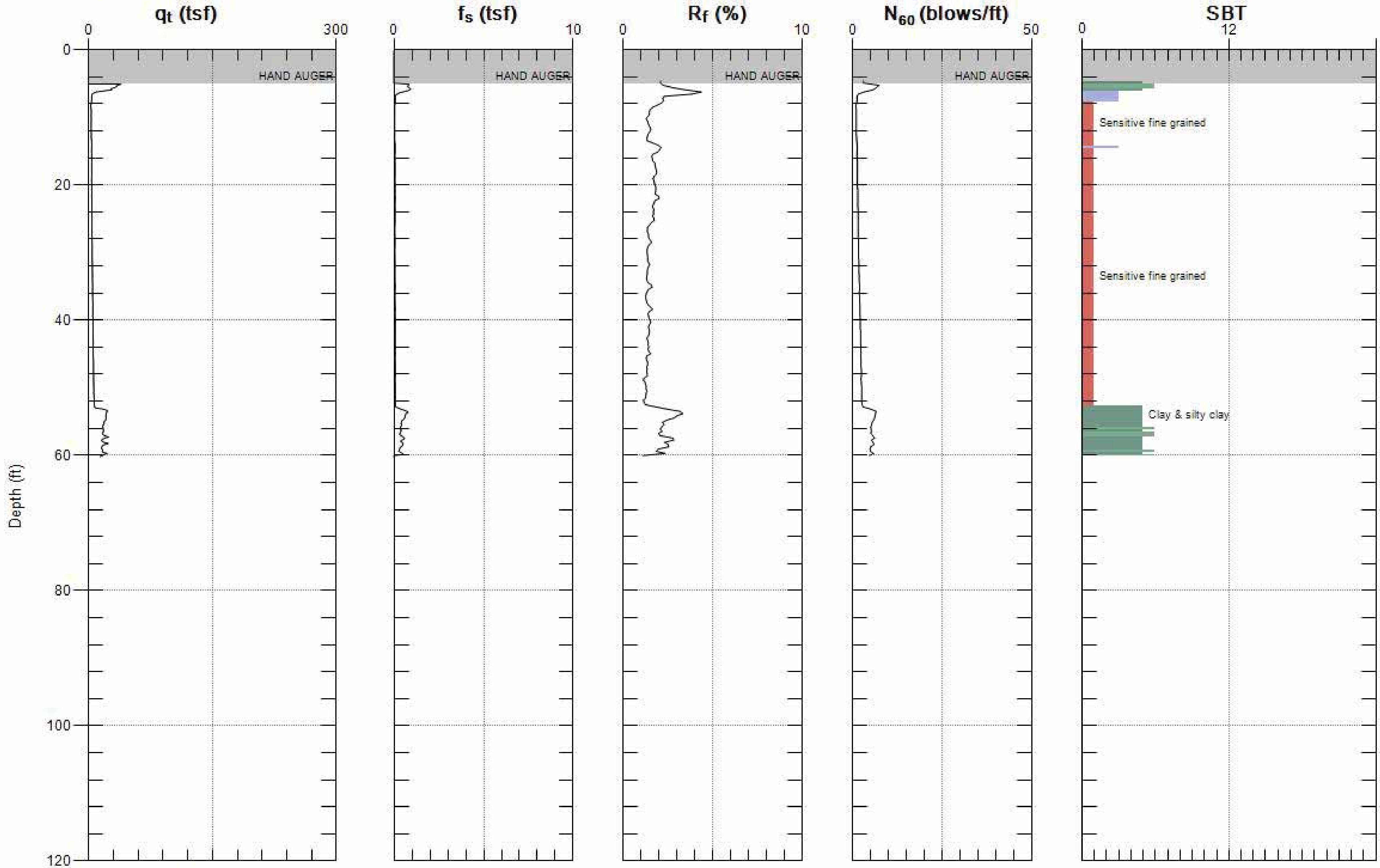
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-23

Engineer: C.COUTU

Date: 10/17/2015 09:36



Max. Depth: 60.203 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

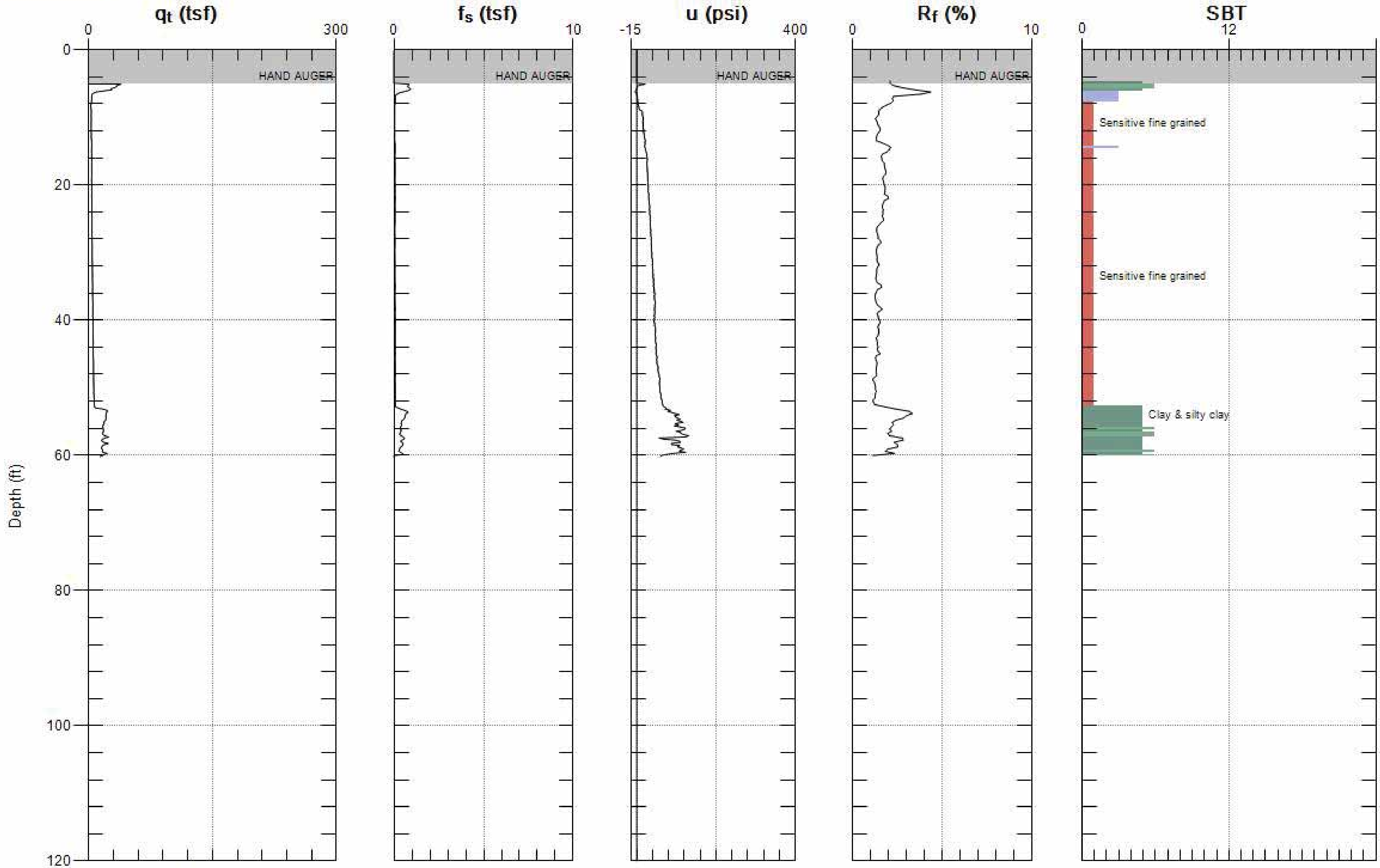
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-23

Engineer: C.COUTU

Date: 10/17/2015 09:36



Max. Depth: 60.203 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

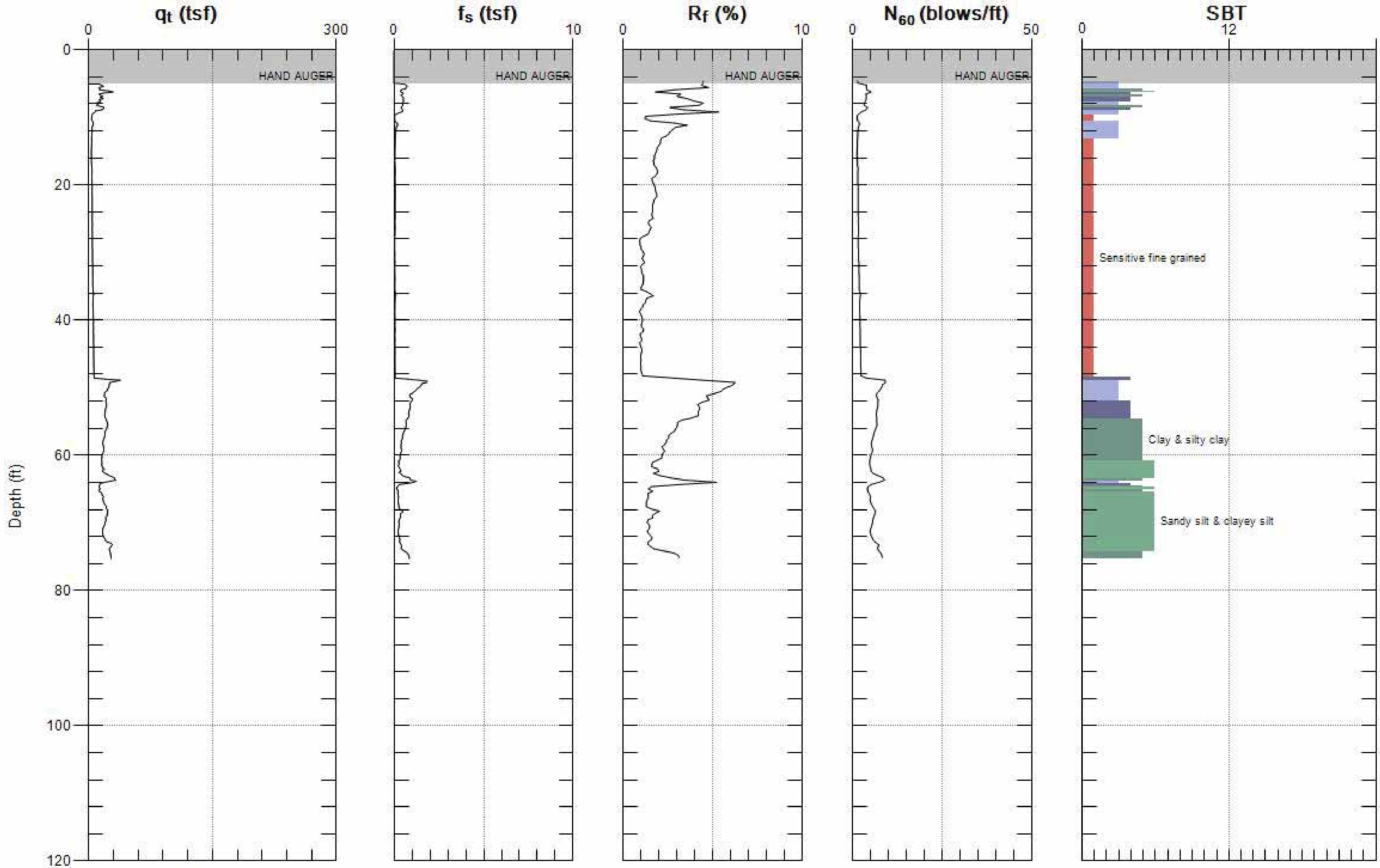
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-24

Engineer: C.COUTU

Date: 10/14/2015 09:56



Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

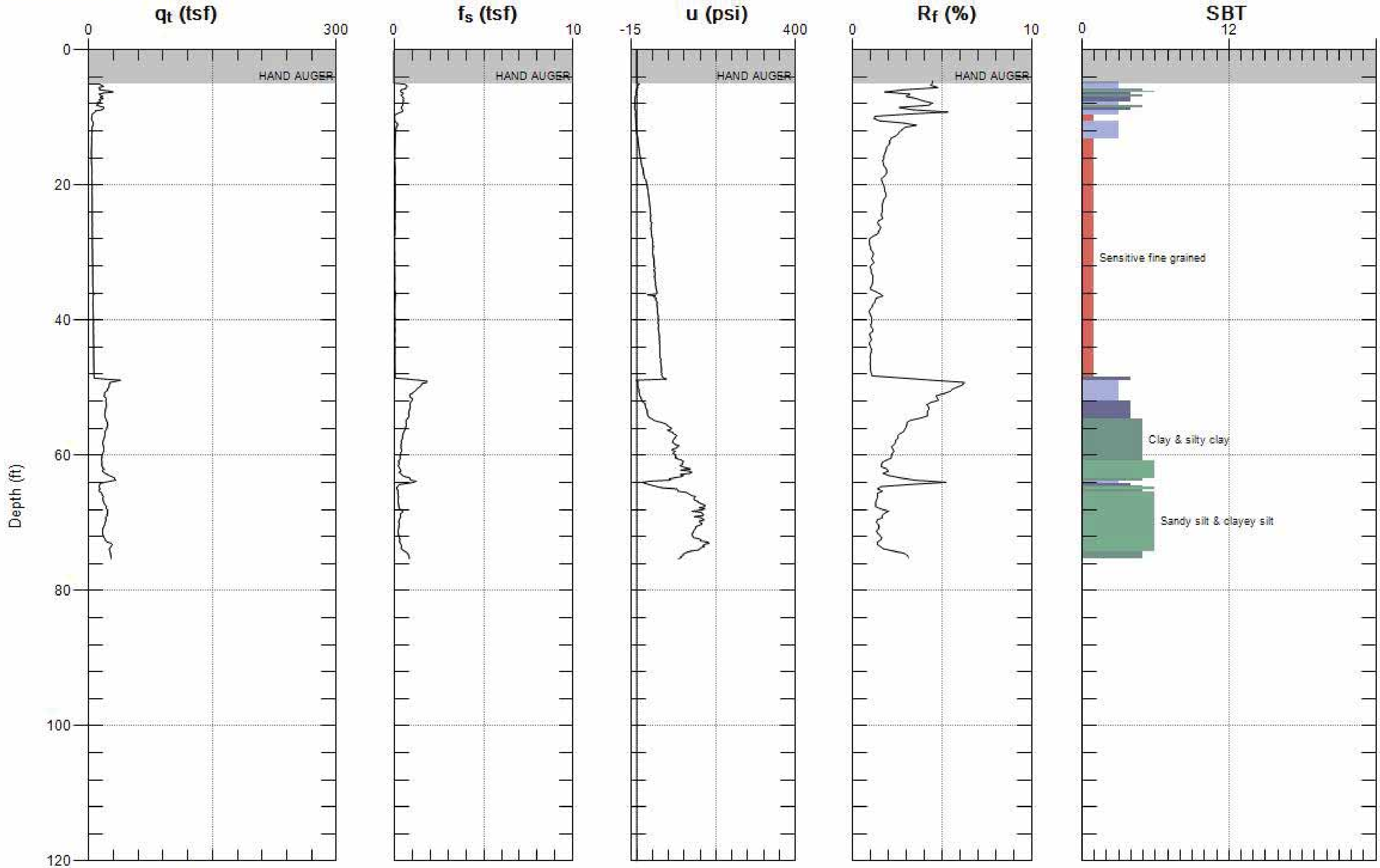
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-24

Engineer: C.COUTU

Date: 10/14/2015 09:56



Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

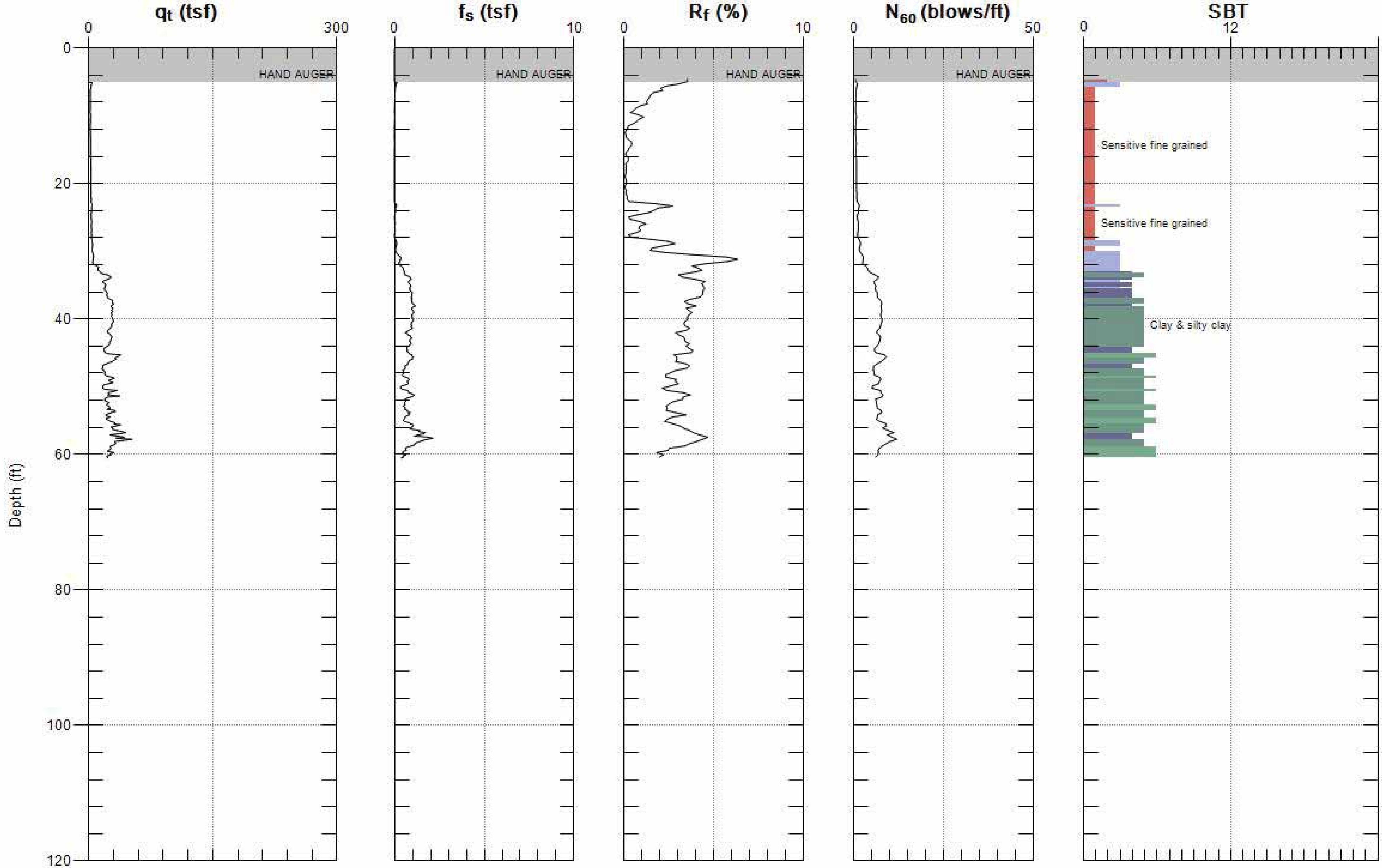
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-25

Engineer: C.COUTU

Date: 10/13/2015 03:07



Max. Depth: 60.531 (ft)
Avg. Interval: 0.328 (ft)

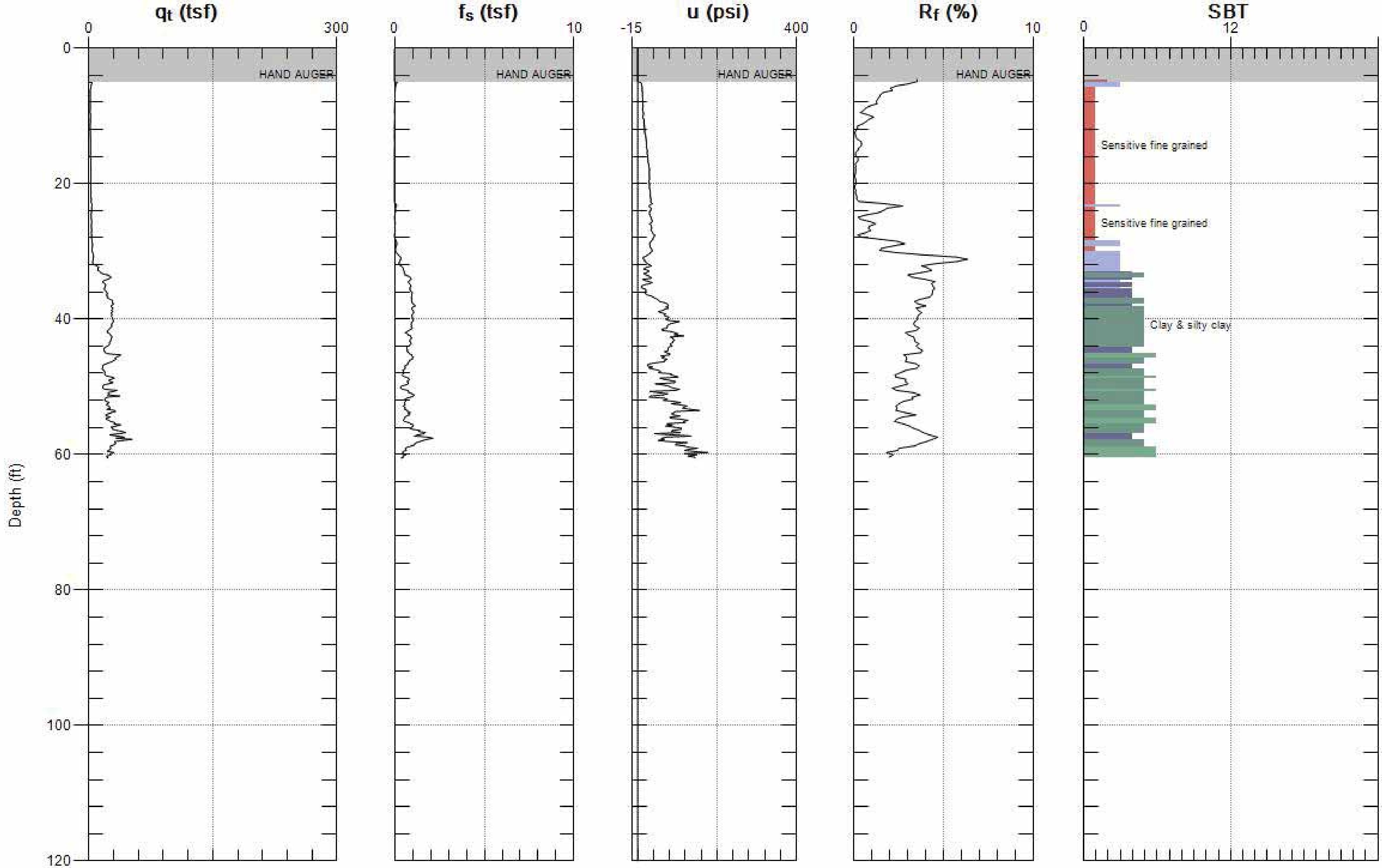
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-25

Engineer: C.COUTU
Date: 10/13/2015 03:07



Max. Depth: 60.531 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

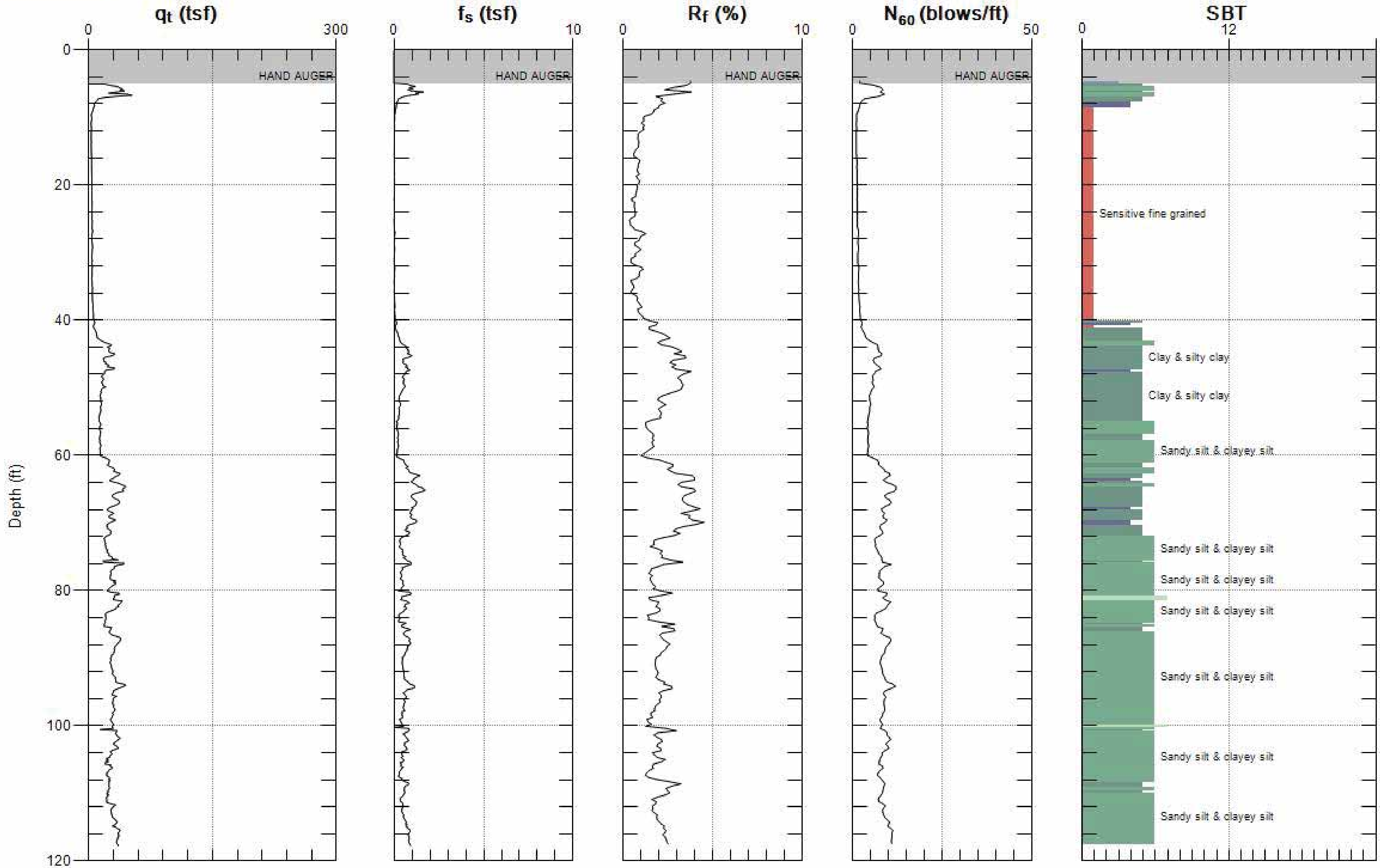
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-26

Engineer: C.COUTU

Date: 10/9/2015 11:45



Max. Depth: 117.782 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

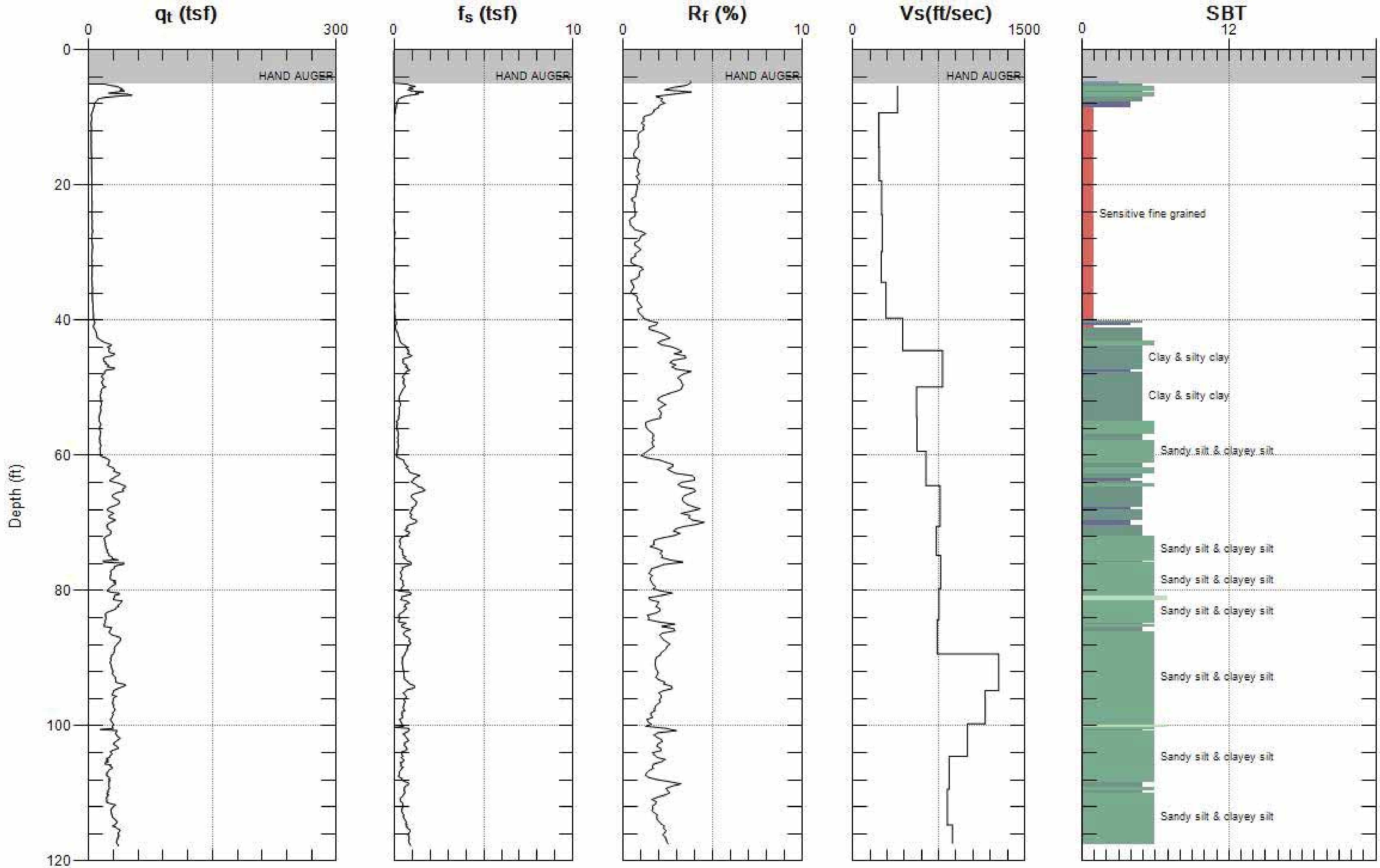
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-26

Engineer: C.COUTU

Date: 10/9/2015 11:45



Max. Depth: 117.782 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

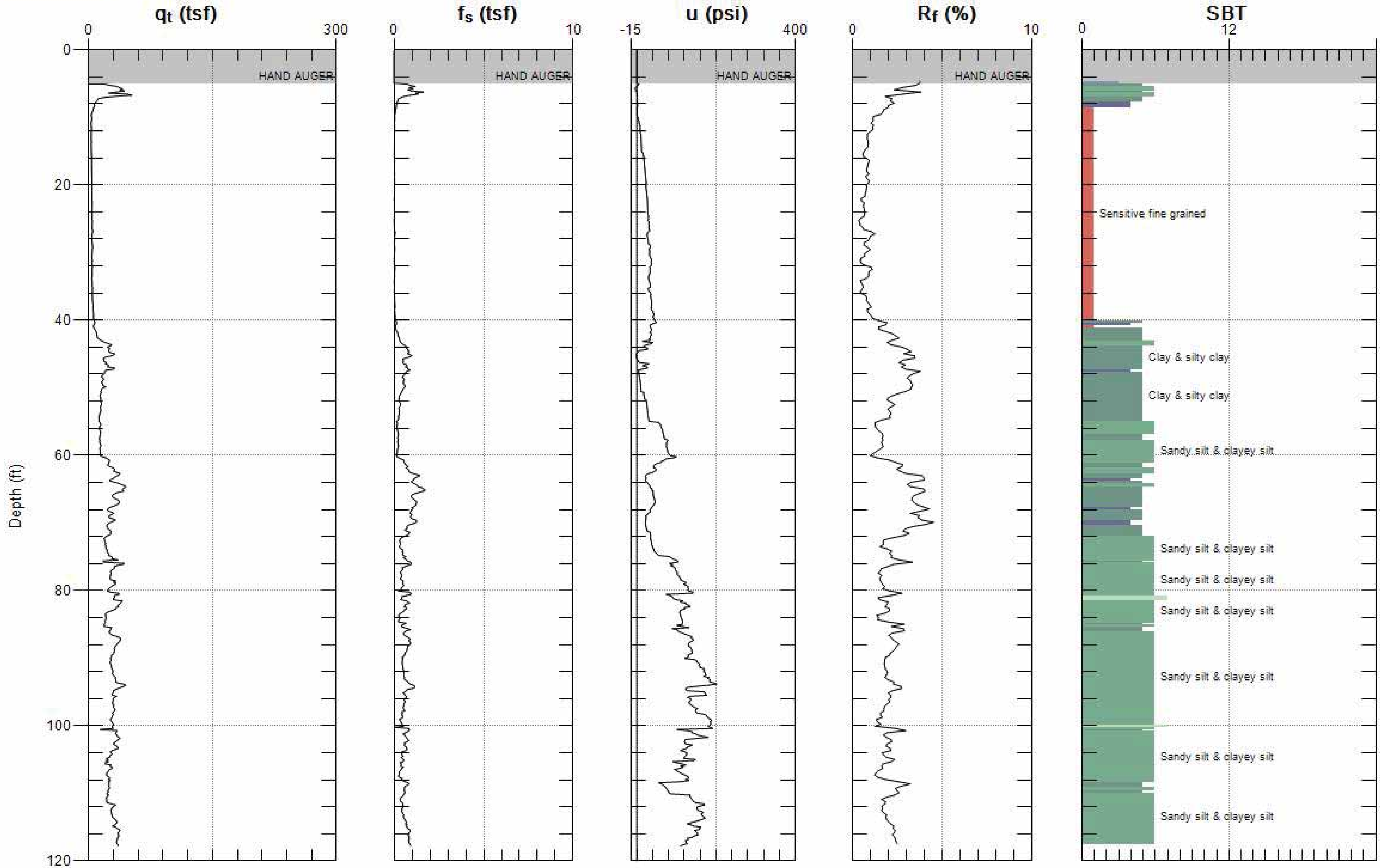
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-26

Engineer: C.COUTU

Date: 10/9/2015 11:45



Max. Depth: 117.782 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

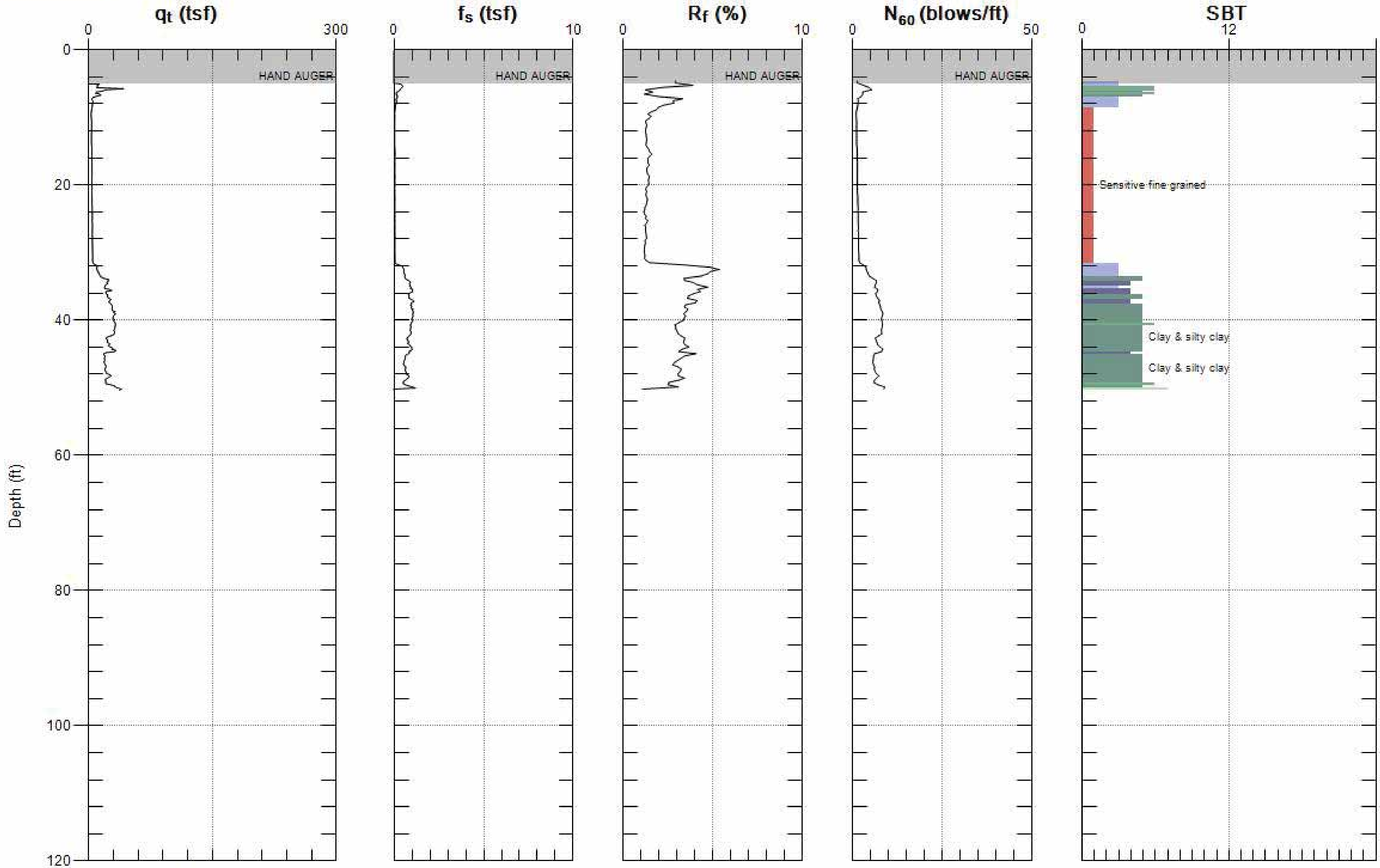
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-28

Engineer: C.COUTU

Date: 10/17/2015 01:58



Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

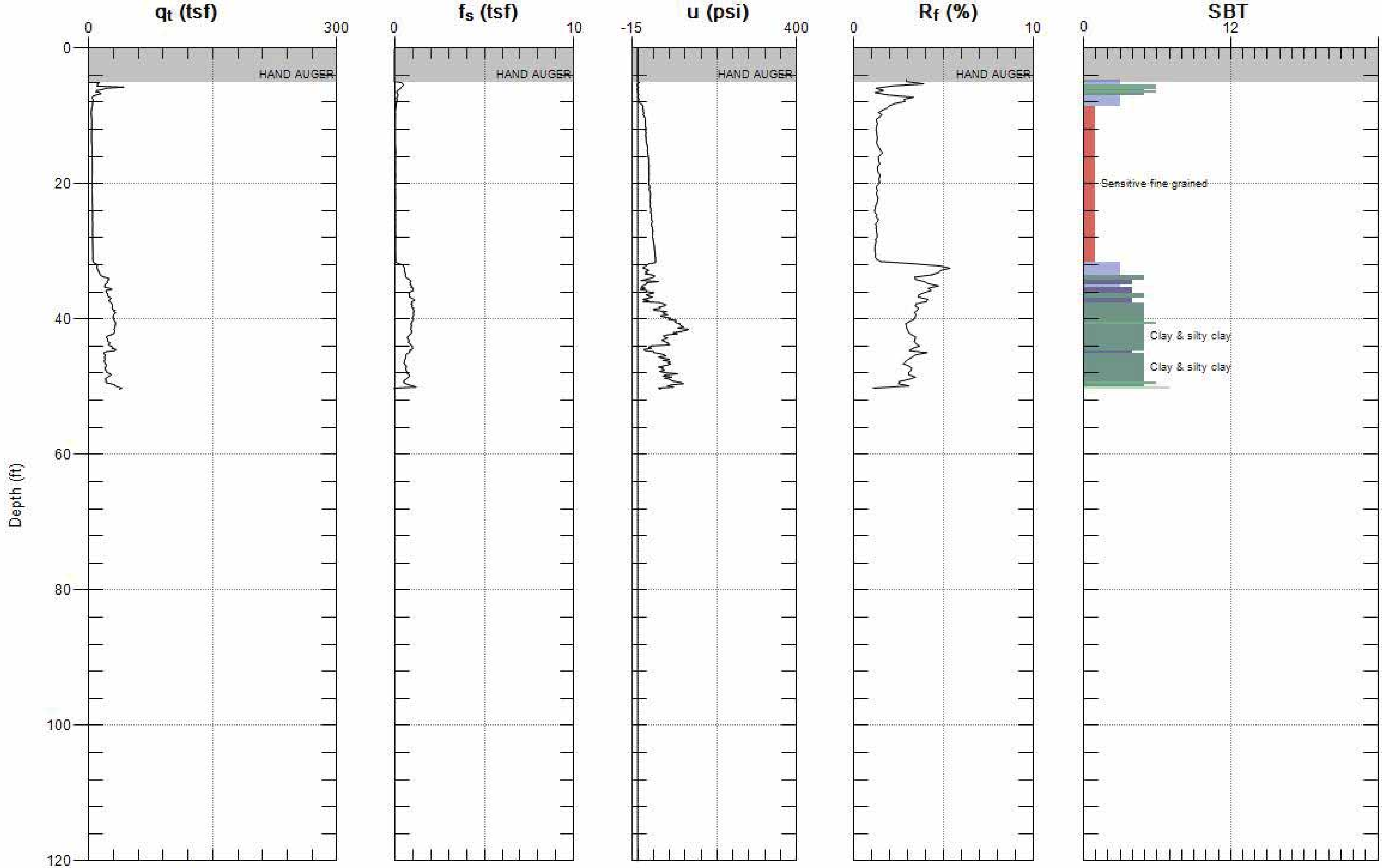
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-28

Engineer: C.COUTU

Date: 10/17/2015 01:58



Max. Depth: 50.361 (ft)

Avg. Interval: 0.328 (ft)

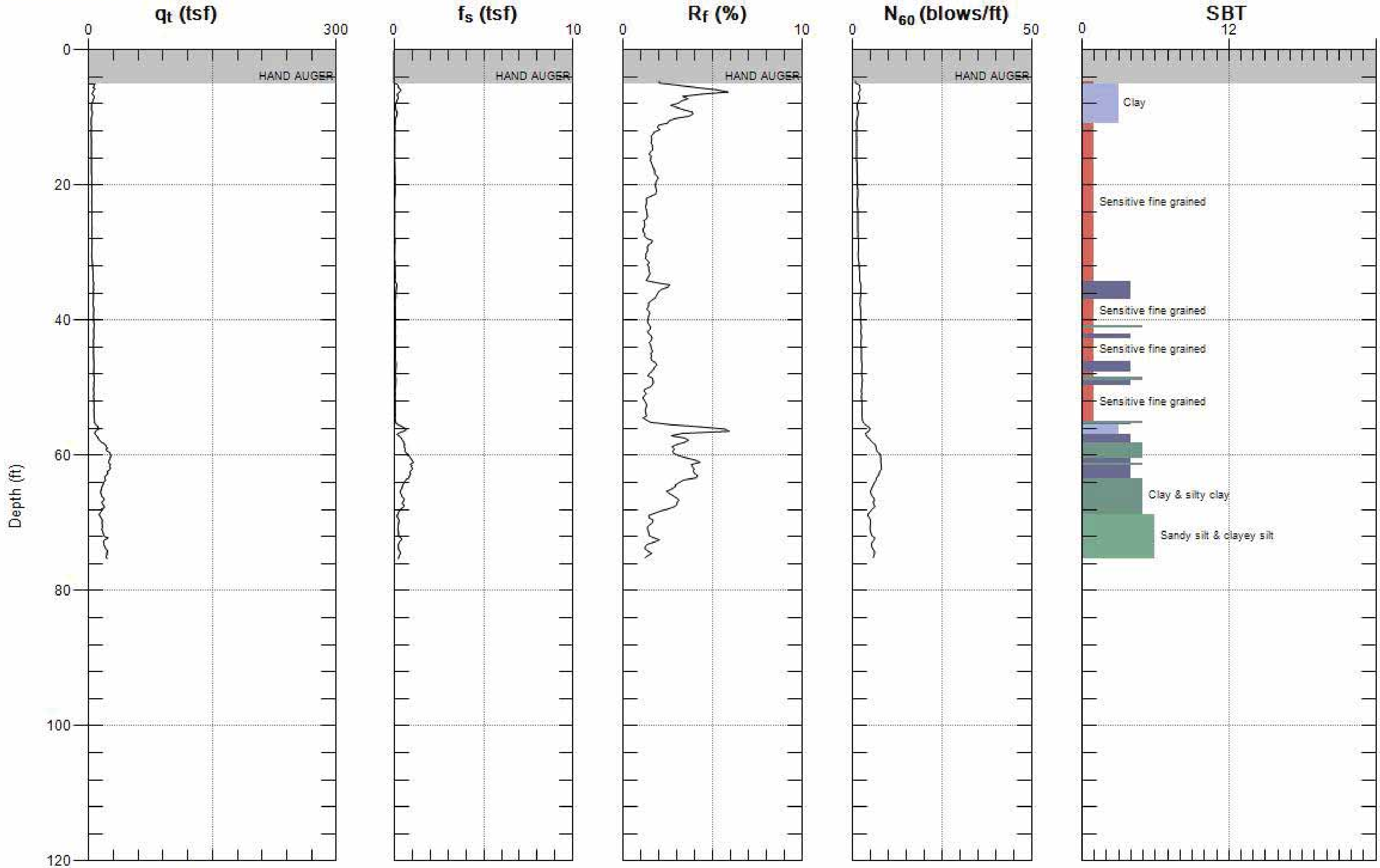
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FOSTER WHEELER

Site: PROJECT ZEUS
Sounding: CPT-29

Engineer: C.COUTU
Date: 10/14/2015 08:08



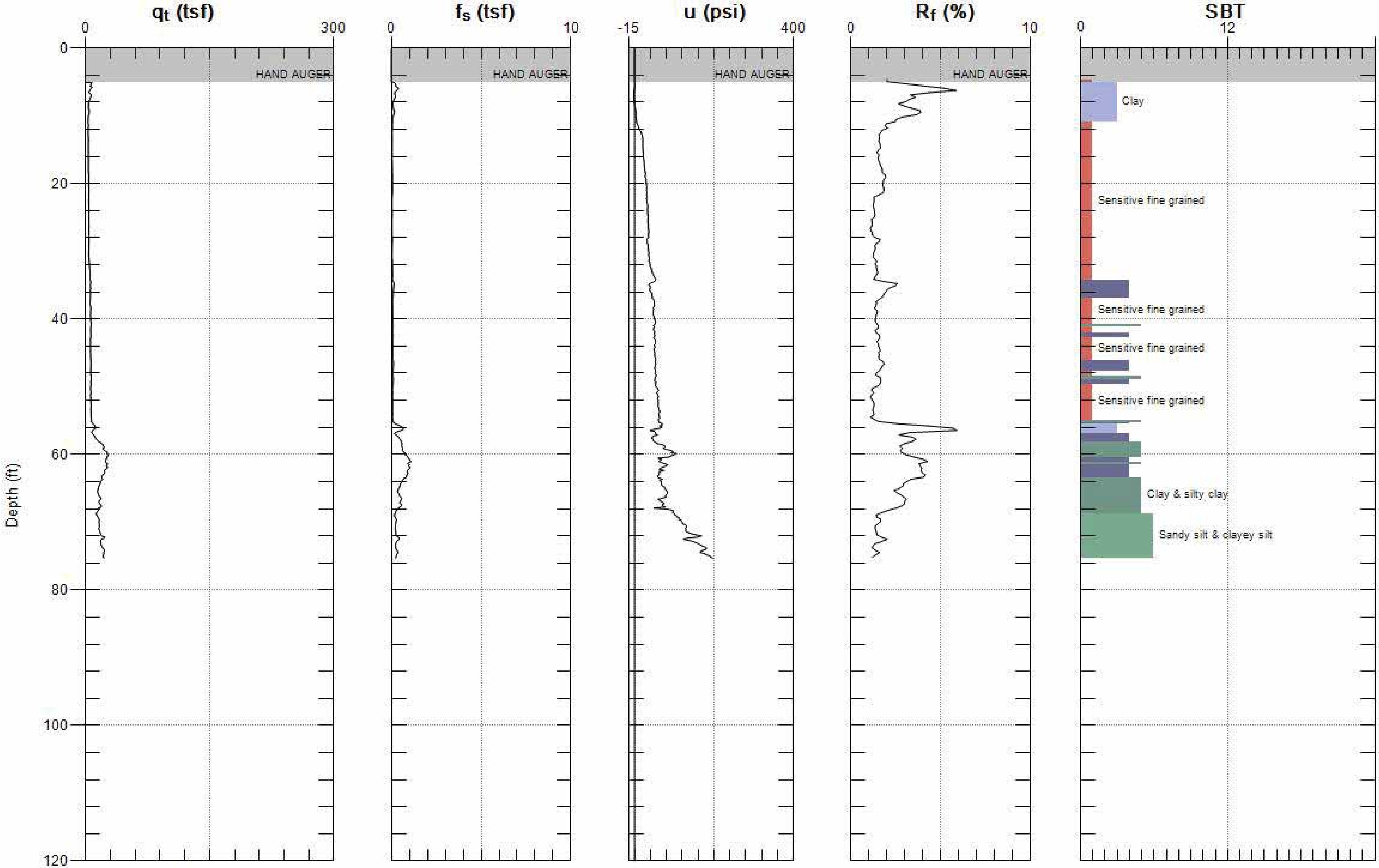
Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **AMEC FOSTER WHEELER**

Site: PROJECT ZEUS
Sounding: CPT-29

Engineer: C.COUTU
Date: 10/14/2015 08:08



Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

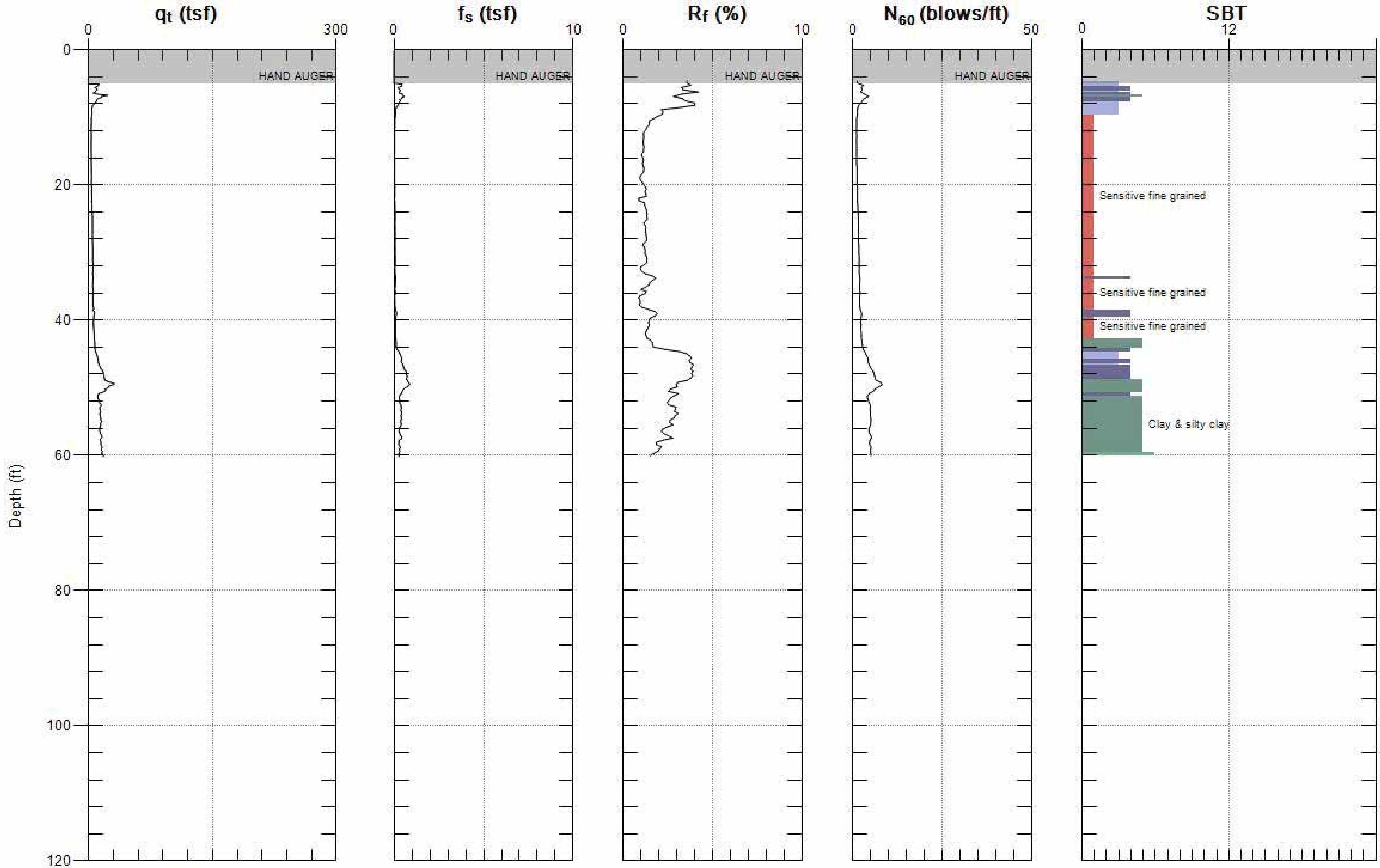
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-30

Engineer: C.COUTU

Date: 10/9/2015 09:53



Max. Depth: 60.203 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

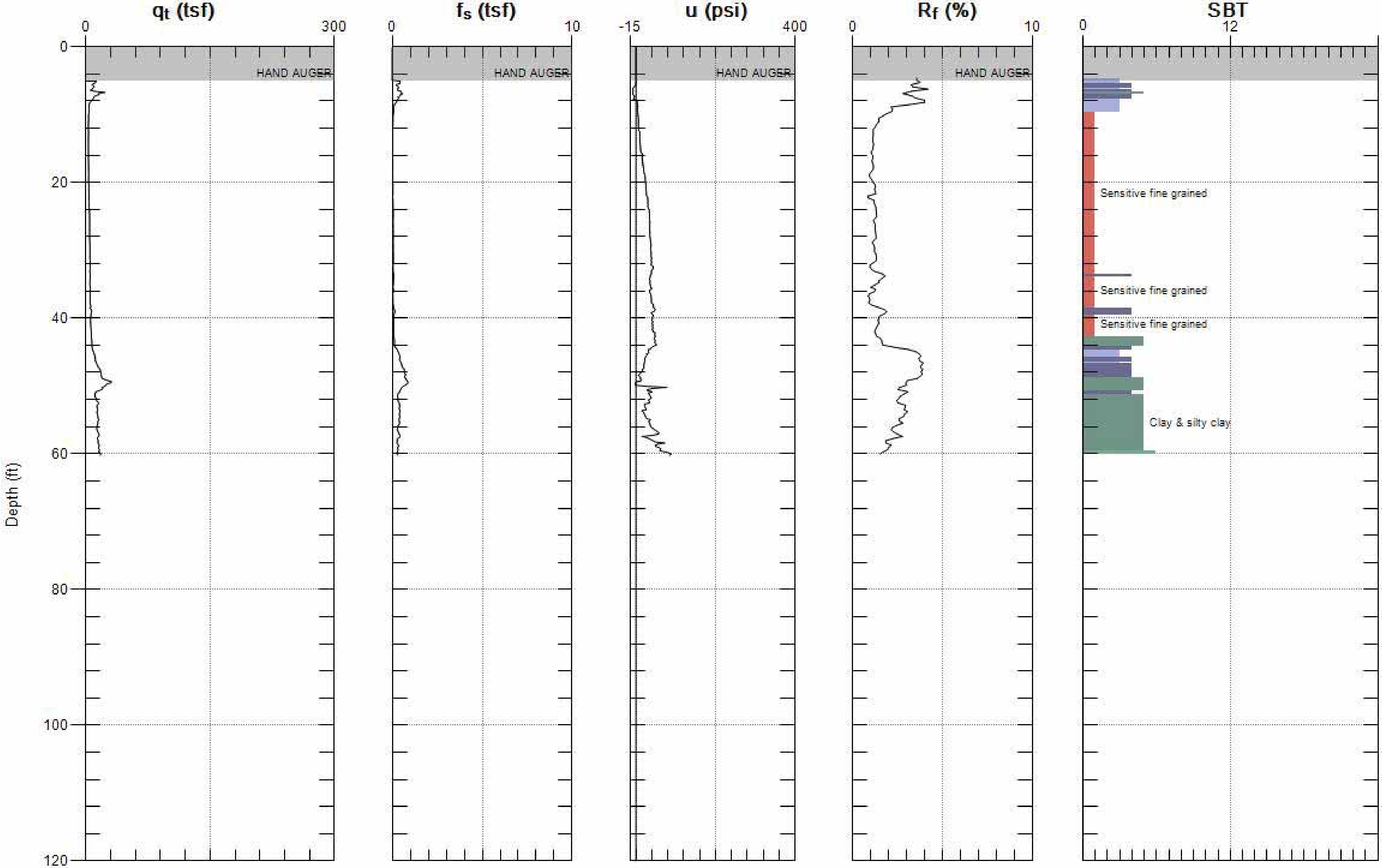
AMEC FOSTER WHEELER

Site: PROJECT ZEUS

Sounding: CPT-30

Engineer: C.COUTU

Date: 10/9/2015 09:53



Max. Depth: 60.203 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

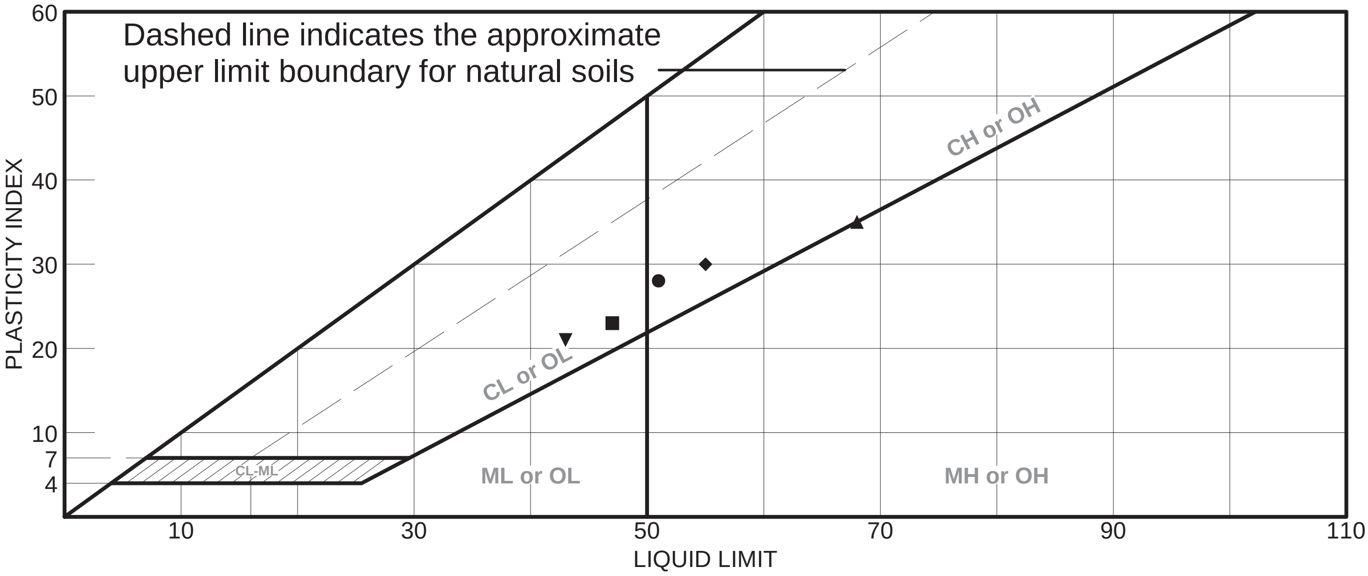
Geotechnical Investigation Report
Project Zeus
Mare Island, Vallejo, California

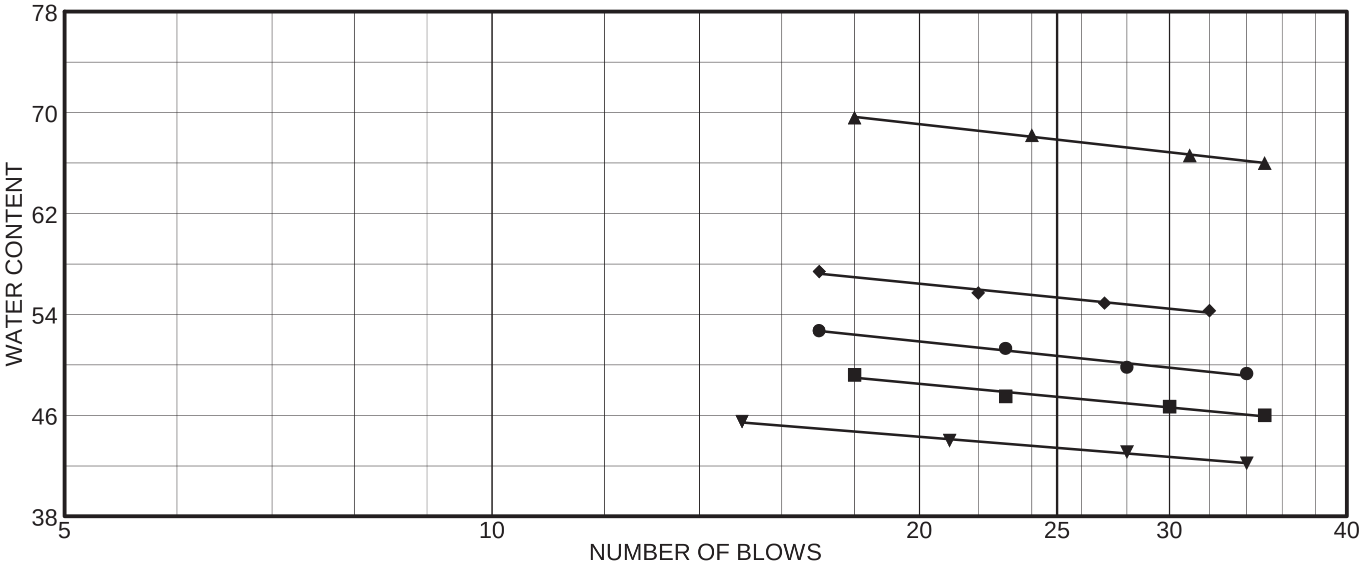
Laboratory tests were performed on selected samples of soil to assess their engineering properties and physical characteristics. The following tests were performed by Cooper Testing Labs of Palo Alto, California, in general accordance with standards of the American Society of Testing and Materials (ASTM), the California Department of Transportation (CAL), or the Uniform Building Code (UBC) on soil samples obtained from the site:

- Moisture Density (ASTM D 7263b)
- Sieve Analysis / 200 Sieve Wash (ASTM D 422 / 1140)
- Atterberg Limit Test (ASTM D 4318)
- Modified Procter (ASTM D 1557)
- Consolidation (ASTM D 2435)
- Minimum Resistivity, pH, Chloride, and Sulfate Tests (CAL 643, 417, 422m)
- Expansion Index Testing
- R-Value (CAL 301)
- Direct Shear (Modified ASTM D 3080)
- Triaxial UU Testing (ASTM D 2850)

Geotechnical laboratory test results are presented in this appendix. Results of moisture density, Atterberg limits, R-value, and triaxial UU test are also reported on the boring logs in Appendix A.

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Very Dark Brown Gravelly Fat CLAY w/ Sand	51	23	28	68.5	60.2	CH
■	Greenish Gray Gravelly Lean CLAY	47	24	23			
▲	Greenish Gray Fat CLAY (Bay Mud)	68	33	35			
◆	Olive Gray Fat CLAY	55	25	30			
▼	Olive Gray Lean CLAY	43	22	21			

Project No. 109-758 **Client:** Amec Foster Wheeler

Project: Project Zeus - 6166150082

● **Source:** B-1	**Sample No.:** S-1-A	**Elev./Depth:** 0-5'
■ **Source:** B-1	**Sample No.:** S-4	**Elev./Depth:** 7.5-9.2'
▲ **Source:** B-1	**Sample No.:** S-6	**Elev./Depth:** 25-27.4'
◆ **Source:** B-1	**Sample No.:** S-10	**Elev./Depth:** 50-52.5'
▼ **Source:** B-1	**Sample No.:** S-11-3	**Elev./Depth:** 53-53.5'

Remarks:
●
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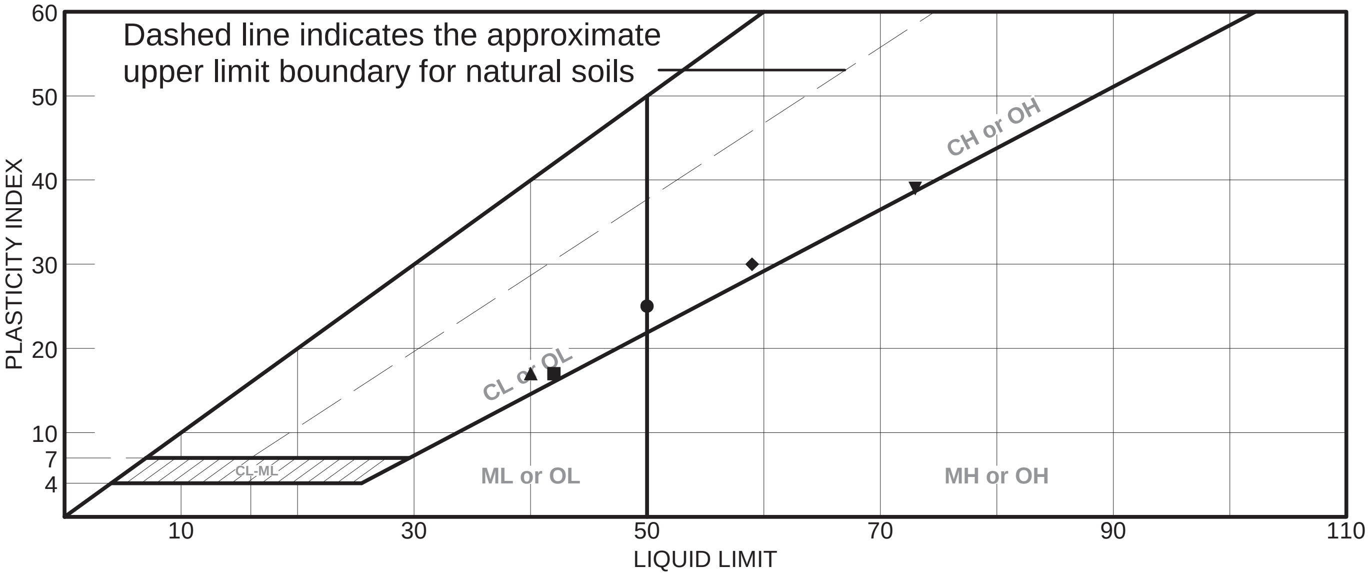
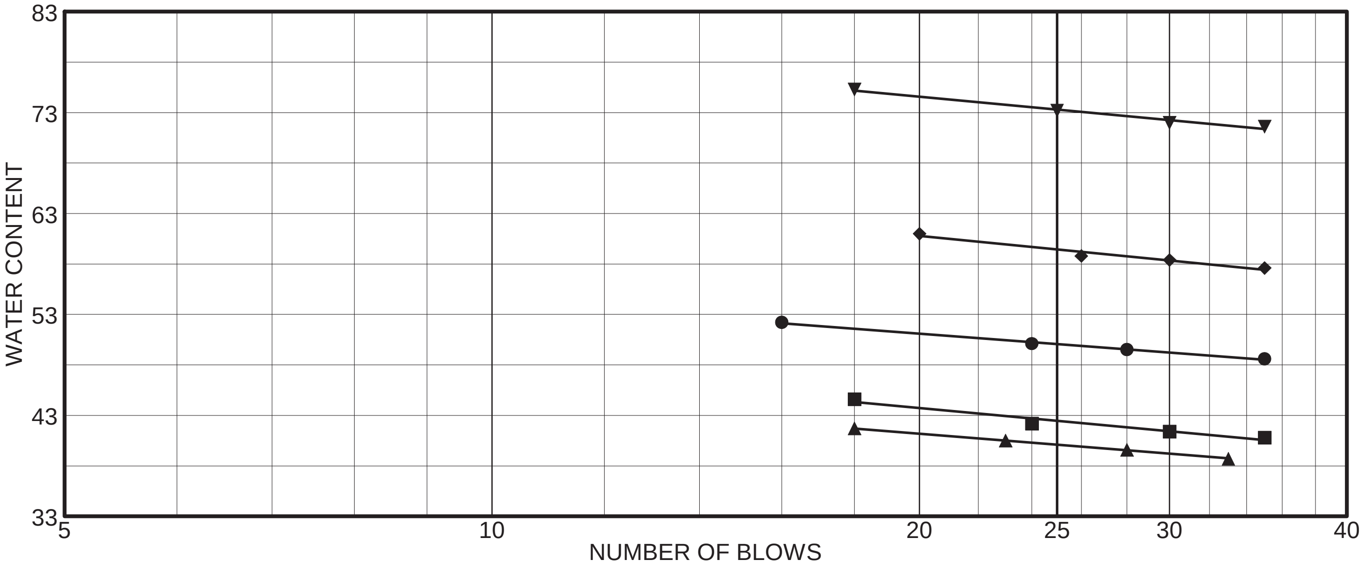
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Olive Gray Lean CLAY	50	25	25			
■	Olive Brown Lean Clayey GRAVEL	42	25	17	42.8	40.7	GC
▲	Yellowish Brown Lean CLAY w/ Sand	40	23	17			
◆	Dark Gray Fat Clayey SAND w/ Gravel	59	29	30			
▼	Dark Olive Gray Fat CLAY (Bay Mud)	73	34	39	100.0	99.9	CH

Project No. 109-758 **Client:** Amec Foster Wheeler

Project: Project Zeus - 6166150082

● **Source:** B-1 **Sample No.:** S-11-4 **Elev./Depth:** 53.5-54'

■ **Source:** B-2 **Sample No.:** S-2-4 **Elev./Depth:** 6-6.5'

▲ **Source:** B-3 **Sample No.:** S-1-3 **Elev./Depth:** 6-7.5'

◆ **Source:** B-3 **Sample No.:** S-2-4 **Elev./Depth:** 8-8.5'

▼ **Source:** B-3 **Sample No.:** S-4 **Elev./Depth:** 13-15.3'

Remarks:

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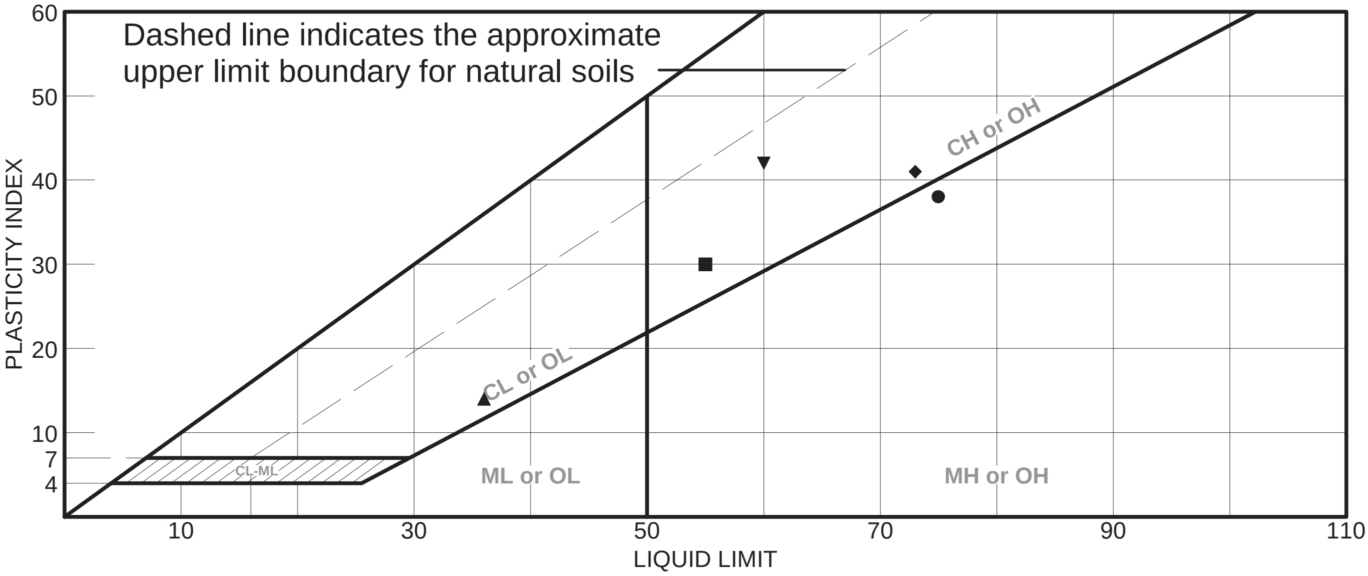
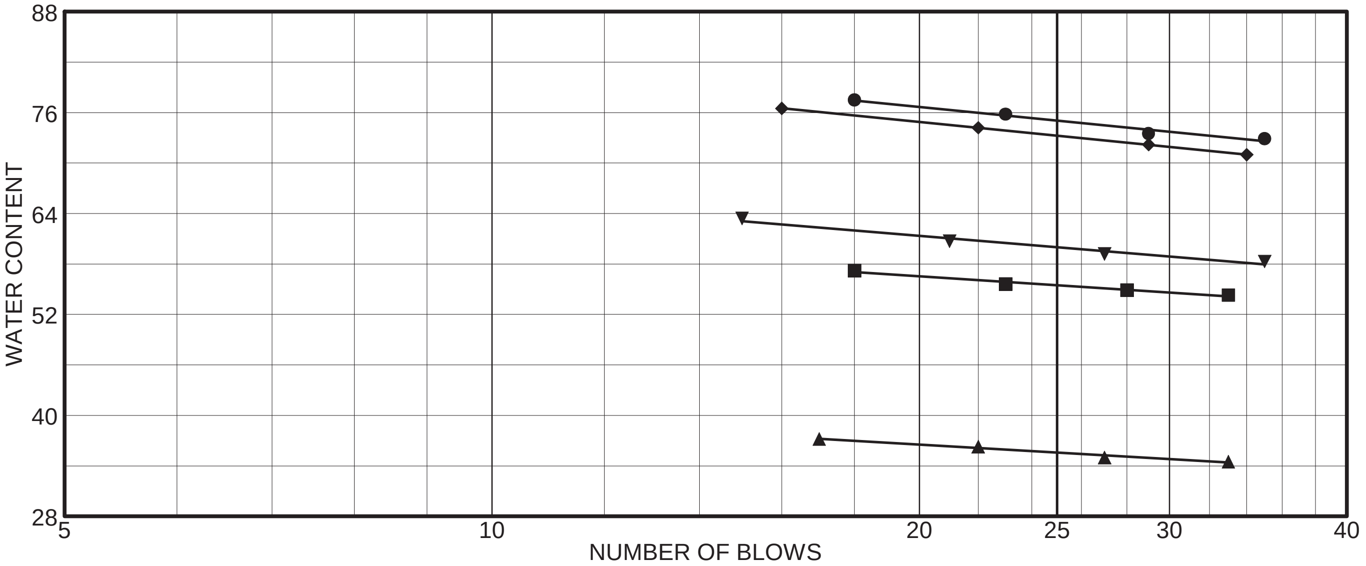
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Greenish Gray Elastic SILT w/ organics & shell fragments (Bay Mud)	75	37	38			
■	Light Gray Fat CLAY	55	25	30			
▲	Dark Olive Brown Lean Clayey SAND w/ Gravel	36	22	14	62.3	43.6	SC
◆	Dark Greenish Gray Fat CLAY (Bay Mud)	73	32	41			
▼	Olive Fat CLAY w/ Sand	60	18	42			

Project No. 109-758 **Client:** Amec Foster Wheeler

Project: Project Zeus - 6166150082

● **Source:** B-3 **Sample No.:** S-5 **Elev./Depth:** 25-25.5'

■ **Source:** B-3 **Sample No.:** S-9-3 **Elev./Depth:** 65.5-66'

▲ **Source:** B-4 **Sample No.:** S-1-A **Elev./Depth:** 0-5'

◆ **Source:** B-4 **Sample No.:** S-4 **Elev./Depth:** 10-12.5'

▼ **Source:** B-4 **Sample No.:** S-6-4 **Elev./Depth:** 47-47.5'

Remarks:
●
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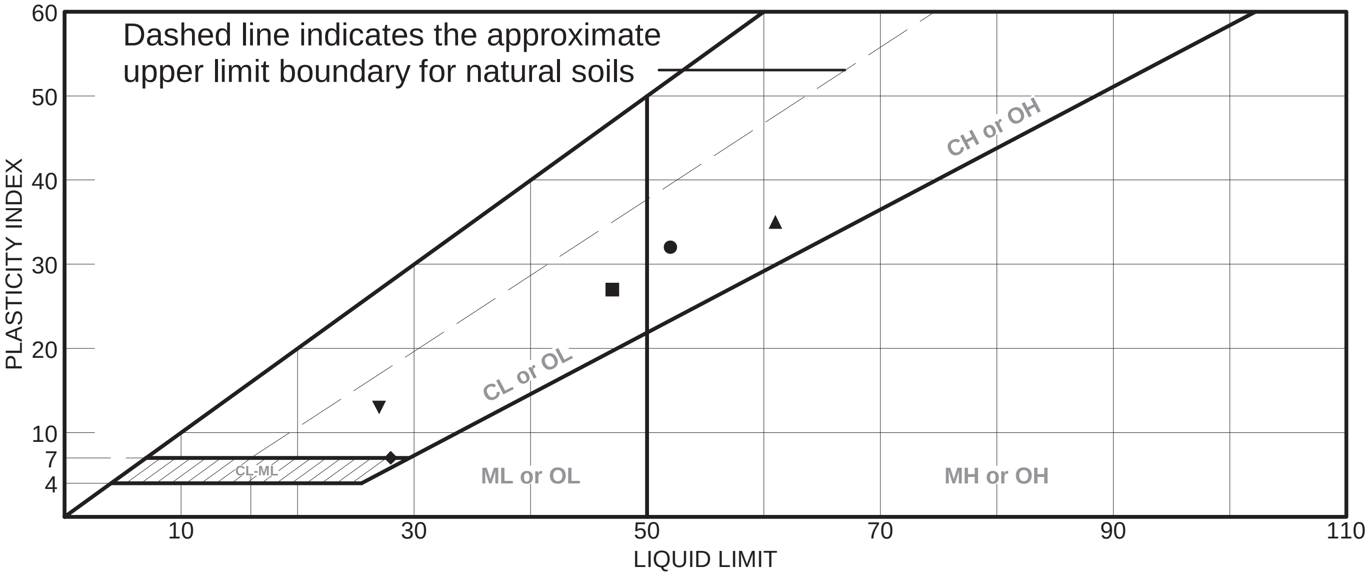
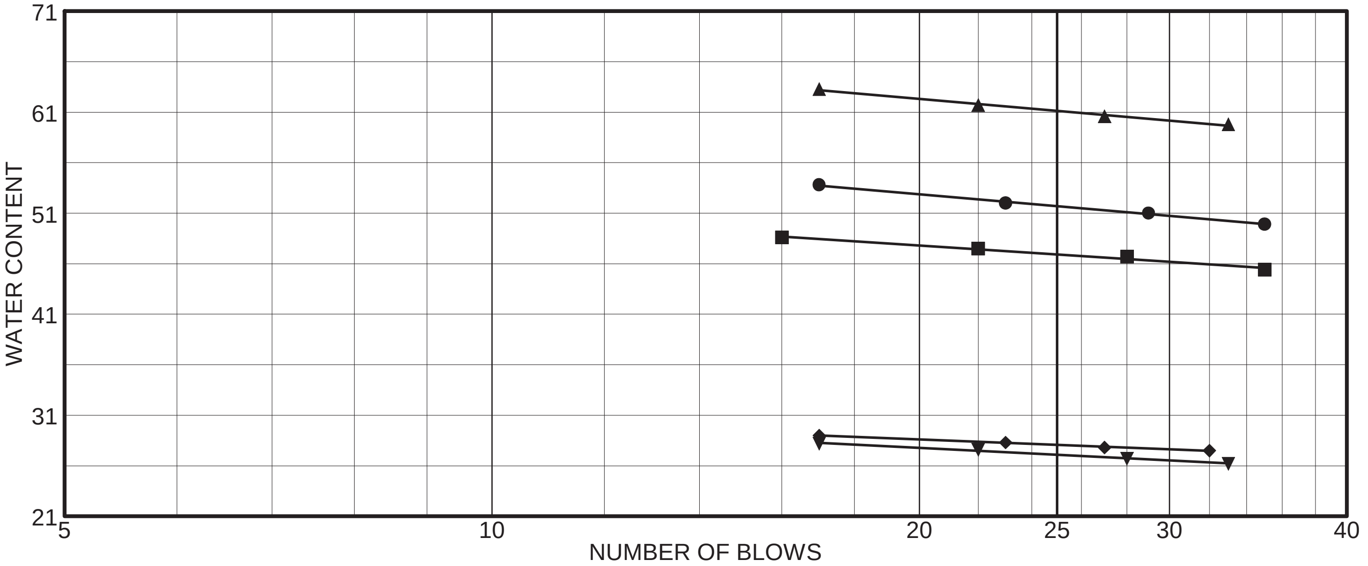
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Olive Gray Fat CLAY	52	20	32			
■	Olive Lean CLAY	47	20	27	99.9	97.5	CL
▲	Greenish Gray Fat CLAY	61	26	35			
◆	Dark Olive Brown Silty, Clayey SAND w/ Gravel	28	21	7	60.6	37.3	SC-SM
▼	Olive Brown Sandy Lean CLAY	27	14	13			

Project No. 109-758 **Client:** Amec Foster Wheeler

Project: Project Zeus - 6166150082

● **Source:** B-4 **Sample No.:** S-9-4 **Elev./Depth:** 62-62.5'
■ **Source:** B-4 **Sample No.:** S-10-3 **Elev./Depth:** 76.5-77'
▲ **Source:** B-4 **Sample No.:** S-12-3 **Elev./Depth:** 95-95.5'
◆ **Source:** B-5 **Sample No.:** S-1-A **Elev./Depth:** 0-5'
▼ **Source:** B-5 **Sample No.:** S-6-3 **Elev./Depth:** 75.5-76'

Remarks:
●
■
▲
◆
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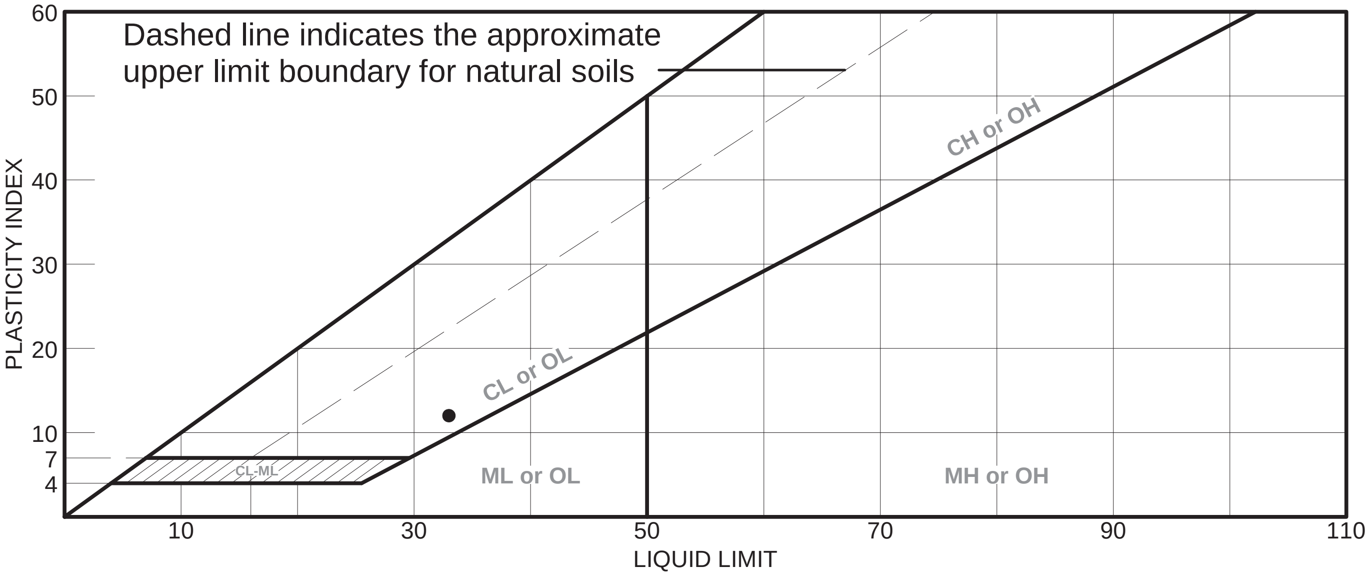
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Yellowish Brown Lean Clayey GRAVEL w/ Sand	33	21	12	30.2	19.6	GC

Project No. 109-758 **Client:** Amec Foster Wheeler

Project: Project Zeus - 6166150082

● **Source:** B-6 **Sample No.:** S-1-A **Elev./Depth:** 0-5'

Remarks:

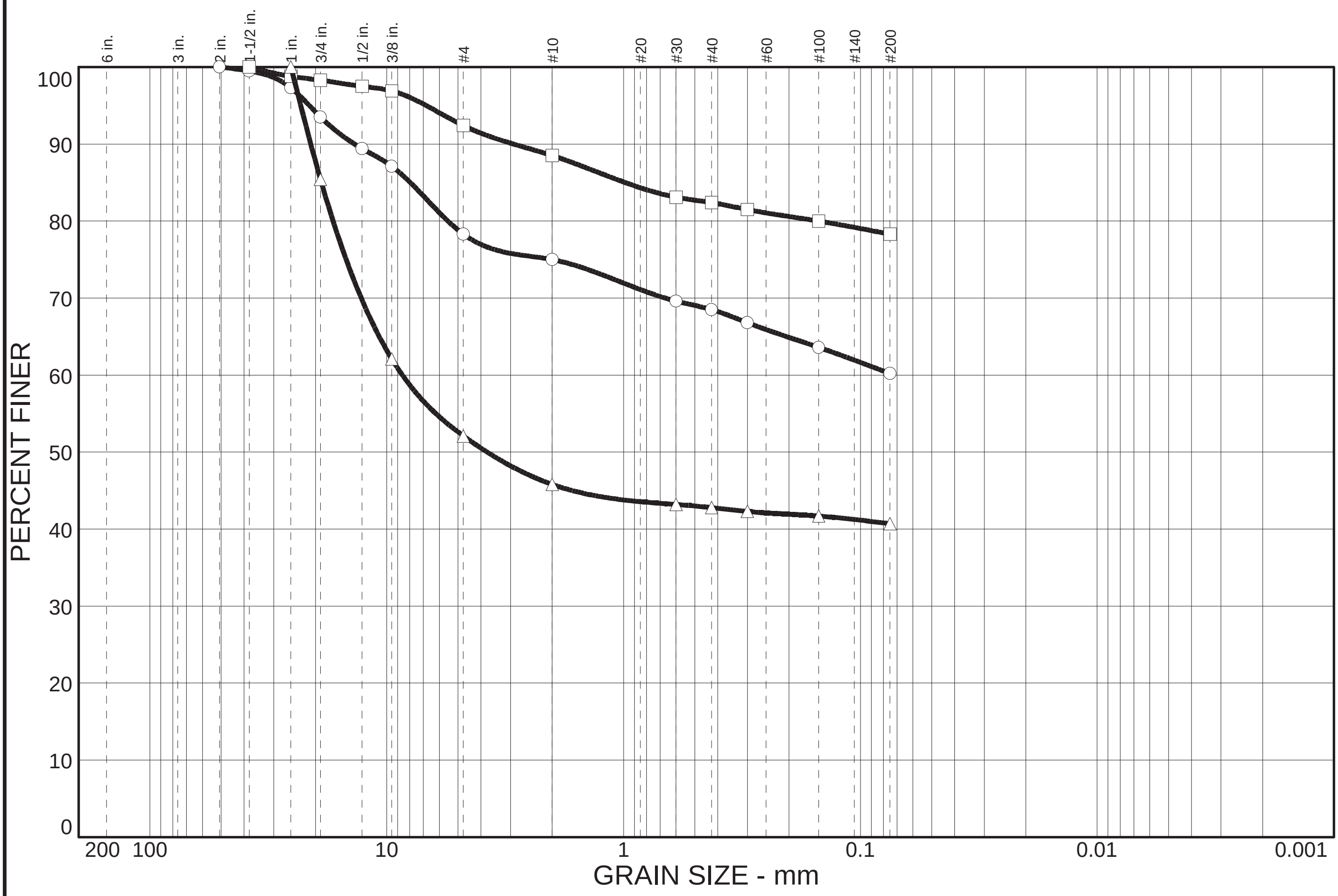
●

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		21.7	18.1	60.2		CH		23	51
□		7.6	14.1	78.3					
△		47.9	11.4	40.7		GC		25	42

SIEVE inches size	PERCENT FINER ○	□	△
2	100.0		
1.5"	99.5	100.0	
1"	97.3	98.8	100.0
3/4"	93.5	98.3	85.4
1/2"	89.4	97.5	
3/8"	87.1	96.9	62.1
GRAIN SIZE			
D_{60}			8.59
D_{30}			
D_{10}			
COEFFICIENTS			
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	78.3	92.4	52.1
#10	75.0	88.5	45.8
#30	69.6	83.1	43.2
#40	68.5	82.4	42.8
#50	66.8	81.5	42.3
#100	63.6	80.0	41.7
#200	60.2	78.3	40.7

SOIL DESCRIPTION

○ Very Dark Brown Gravelly Fat CLAY w/ Sand

□ Yellowish Brown CLAY w/ Sand

△ Olive Brown Lean Clayey GRAVEL

REMARKS:

○

□

△ Due to the small sample size, relative to the largest particle size, this data should be considered to be approximate.

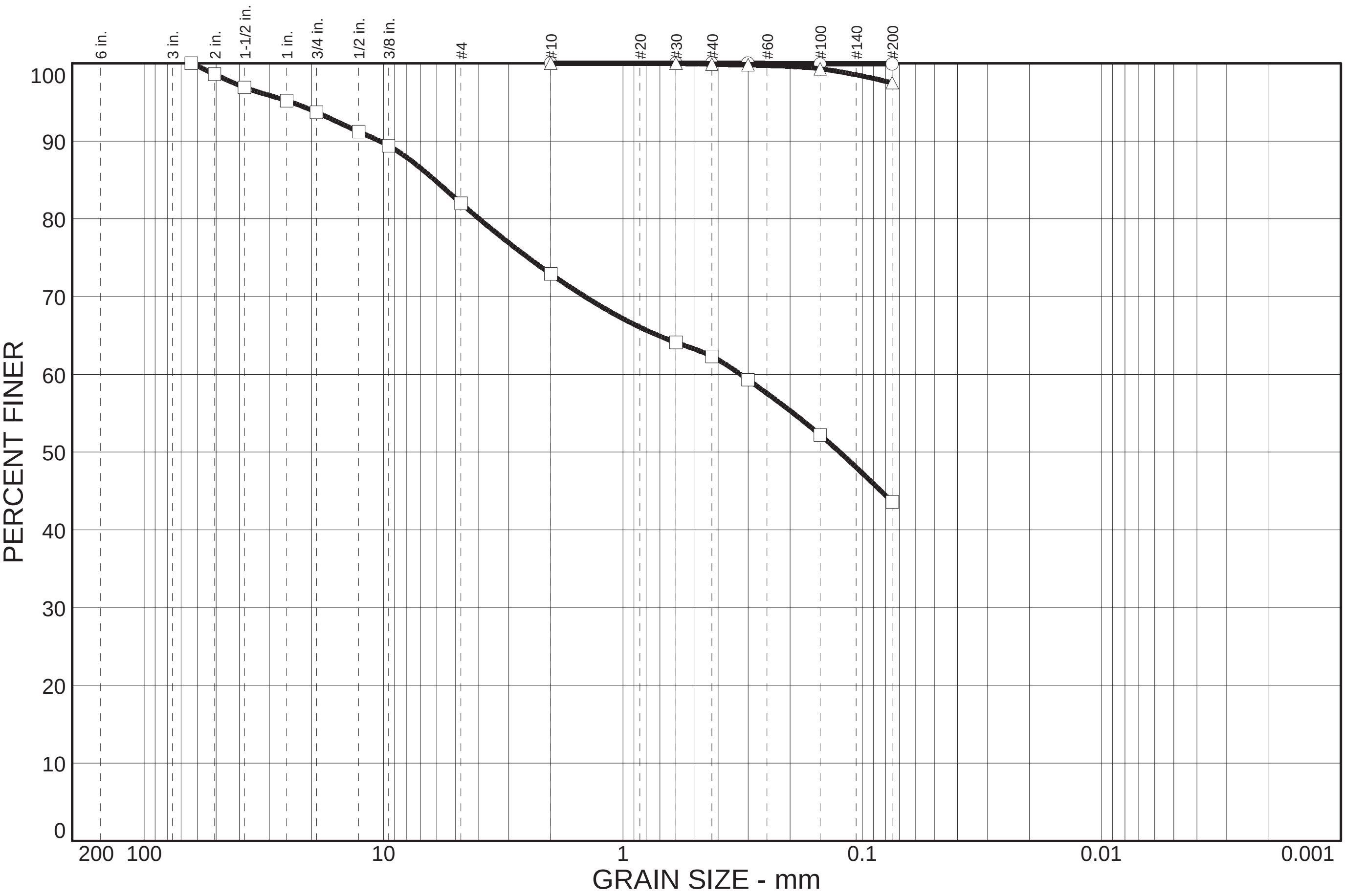
○ Source: B-1 Sample No.: S-1-A Elev./Depth: 0-5'
□ Source: B-2 Sample No.: S-1-A Elev./Depth: 0-5'
△ Source: B-2 Sample No.: S-2-4 Elev./Depth: 6-6.5'

COOPER TESTING LABORATORY

Client: Amec Foster Wheeler
Project: Project Zeus - 6166150082

Project No.: 109-758 Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○			0.1	99.9		CH		34	73
□		18.0	38.4	43.6		SC		22	36
△			2.5	97.5		CL		20	47

SIEVE inches size	PERCENT FINER ○	□	△
2.5		100.0	
2		98.6	
1.5"		96.9	
1"		95.2	
3/4"		93.7	
1/2"		91.2	
3/8"		89.4	
	GRAIN SIZE		
D_{60}		0.323	
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4		82.0	
#10	100.0	72.9	100.0
#30	100.0	64.1	100.0
#40	100.0	62.3	99.9
#50	100.0	59.3	99.8
#100	99.9	52.2	99.3
#200	99.9	43.6	97.5

SOIL DESCRIPTION

○ Dark Olive Gray Fat CLAY (Bay Mud)

□ Dark Olive Brown Lean Clayey SAND w/ Gravel

△ Olive Lean CLAY

REMARKS:

○

□

△

○ Source: B-3 Sample No.: S-4 Elev./Depth: 13-15.3'

□ Source: B-4 Sample No.: S-1-A Elev./Depth: 0-5'

△ Source: B-4 Sample No.: S-10-3 Elev./Depth: 76.5-77'

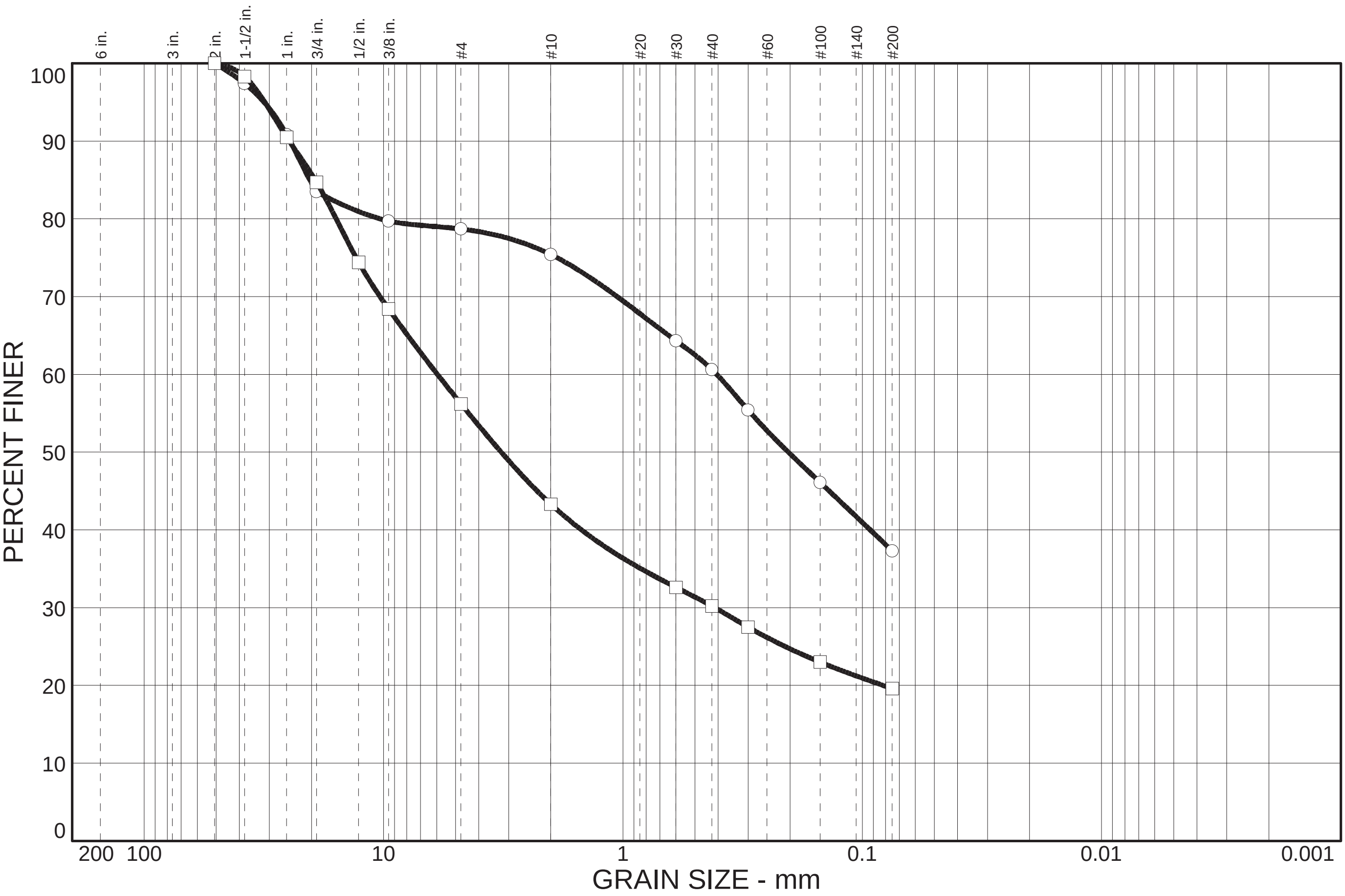
COOPER TESTING LABORATORY

Client: Amec Foster Wheeler

Project: Project Zeus - 6166150082

Project No.: 109-758 Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		21.3	41.4	37.3		SC-SM		21	28
□		43.8	36.6	19.6		GC		21	33

SIEVE inches size	PERCENT FINER ○	PERCENT FINER □	
2	100.0	100.0	
1.5"	97.4	98.3	
1"	90.8	90.5	
3/4"	83.5	84.7	
1/2"		74.4	
3/8"	79.7	68.4	
	GRAIN SIZE		
D_{60}	0.406	5.96	
D_{30}		0.414	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	PERCENT FINER □	
#4	78.7	56.2	
#10	75.4	43.3	
#30	64.3	32.6	
#40	60.6	30.2	
#50	55.4	27.5	
#100	46.1	23.0	
#200	37.3	19.6	

SOIL DESCRIPTION

○ Dark Olive Brown Silty, Clayey SAND w/ Gravel

□ Yellowish Brown Lean Clayey GRAVEL w/ Sand

REMARKS:

○

□

○ Source: B-5 Sample No.: S-1-A Elev./Depth: 0-5'

□ Source: B-6 Sample No.: S-1-A Elev./Depth: 0-5'

COOPER TESTING LABORATORY

Client: Amec Foster Wheeler

Project: Project Zeus - 6166150082

Project No.: 109-758 Figure

#200 Bulk Sieve Wash Analysis
ASTM D 1140m

Job No.: 109-758	**Project No.:** 6166150082	**Run By:**	MD
Client: Amec Foster Wheeler	**Date:** 11/19/2015	**Checked By:**	DC
Project: Project Zeus			

Boring:	B-1								
Sample:	S-4								
Depth, ft.:	7.5-9.2								
Soil Type:	Greenish Gray Gravelly Lean CLAY								
Bulk Sample wt. lb.	932.4								
Wt of Dish & Dry Soil <#4,gm	454.8								
Weight of Dish, gm	193.9								
Weight of Dry Soil <#4, gm	260.9								
Wt. Ret. on #4 Sieve, lb	292.5								
Wt. Ret. on #200 Sieve, gm	30.3								
% Gravel	**31.4**								
% Sand	**8.0**								
% Silt & Clay	**60.7**								

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine the percentage, especially if there is only a trace amount, (5% or less).

#200 Sieve Wash Analysis
ASTM D 1140

Job No.:	109-758a	**Project No.:**	6166150082	**Run By:**	MD
Client:	Amec Foster Wheeler	**Date:**	11/19/2015	**Checked By:**	DC
Project:	Project Zeus				

Boring:	B-1	B-1	B-1	B-3	B-3	B-3	B-4	B-4
Sample:	S-11-3	S-11-4	S-15-4	S-1-3	S-2-4	S-9-3	S-3-3	S-6-3
Depth, ft.:	53-53.5	53.5-54	96-96.5	6-7.5	8-8.5	65.5-66	6-6.5	46.5-47
Soil Type:	Olive Gray Lean CLAY	Olive Gray Lean CLAY	Olive Gray CLAY	Yellowish Brown Lean CLAY w/ Sand	Dark Gray Fat Clayey SAND w/ Gravel	Light Gray Fat CLAY	Dark Gray CLAY	Mottled Olive Gray CLAY
Wt of Dish & Dry Soil, gm	653.7	620.6	604.0	483.3	523.5	610.4	378.3	484.7
Weight of Dish, gm	337.5	329.5	310.3	299.8	173.0	324.1	176.3	303.2
Weight of Dry Soil, gm	316.2	291.2	293.7	183.5	350.6	286.3	202.0	181.5
Wt. Ret. on #4 Sieve, gm	0.0	0.0	0.0	16.7	82.8	0.0	0.0	0.0
Wt. Ret. on #200 Sieve, gm	18.9	14.7	1.3	52.4	180.6	1.4	1.1	2.3
% Gravel	**0.0**	**0.0**	**0.0**	**9.1**	**23.6**	**0.0**	**0.0**	**0.0**
% Sand	**6.0**	**5.0**	**0.4**	**19.5**	**27.9**	**0.5**	**0.5**	**1.2**
% Silt & Clay	**94.0**	**95.0**	**99.6**	**71.4**	**48.5**	**99.5**	**99.5**	**98.8**

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine the percentage, especially if there is only a trace amount, (5% or less).

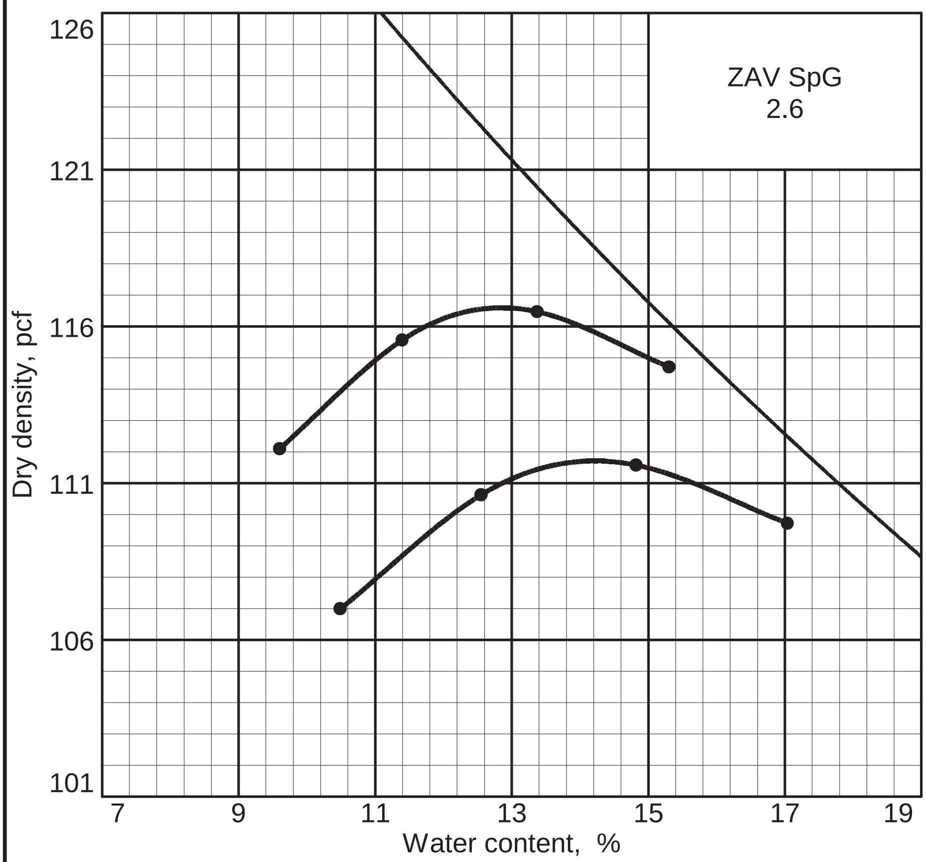
#200 Sieve Wash Analysis
ASTM D 1140

Job No.:	109-758b			**Project No.:**	6166150082		**Run By:**	MD
Client:	Amec Foster Wheeler			**Date:**	11/19/2015		**Checked By:**	DC
Project:	Project Zeus							

Boring:	B-4	B-4	B-4	B-5				
Sample:	S-9-4	S-11-3	S-12-3	S-7-3				
Depth, ft.:	62-62.5	88.5-89	95-95.5	95.5-96				
Soil Type:	Olive Gray Fat CLAY	Olive Gray CLAY	Greenish Gray Fat CLAY	Mottled Olive Clayey SAND				
Wt of Dish & Dry Soil, gm	522.9	640.0	663.8	561.0				
Weight of Dish, gm	171.6	311.5	299.7	304.4				
Weight of Dry Soil, gm	351.3	328.6	364.1	256.6				
Wt. Ret. on #4 Sieve, gm	0.0	0.0	17.0	0.0				
Wt. Ret. on #200 Sieve, gm	11.6	44.3	44.9	145.7				
% Gravel	**0.0**	**0.0**	**4.7**	**0.0**				
% Sand	**3.3**	**13.5**	**7.7**	**56.8**				
% Silt & Clay	**96.7**	**86.5**	**87.7**	**43.2**				

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine the percentage, especially if there is only a trace amount, (5% or less).

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-00 Method B Modified
Oversize correction applied to each point

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 25
Mold Size: .03333 cu.ft.

Test Performed on Material Passing 3/8 in. **Sieve**

Soil Data

NM ____ **Sp.G.** 2.7
LL 51 **PI** 28
%>3/8 in. 12.9 **%<#200** 60.2
USCS CH **AASHTO** ____

TESTING DATA

	1	2	3	4	5	6
WM + WS	8.67	8.79	8.46	8.80		
WM	4.52	4.52	4.52	4.52		
WW + T #1	965.00	691.40	774.20	920.00		
WD + T #1	893.60	639.60	728.70	832.00		
TARE #1	324.70	290.10	294.90	315.50		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	11.4	13.4	9.6	15.3		
DRY DENSITY	115.6	116.5	112.1	114.7		

ROCK CORRECTED TEST RESULTS	UNCORRECTED	Material Description
Maximum dry density = 116.6 pcf	111.7 pcf	Very Dark Brown Gravelly Fat CLAY w/ Sand
Optimum moisture = 12.8 %	14.2 %	

Project No. 109-758 **Client:** Amec Foster Wheeler
Project: Project Zeus - 6166150082

● **Source:** B-1 **Sample No.:** S-1-A **Elev./Depth:** 0-5'

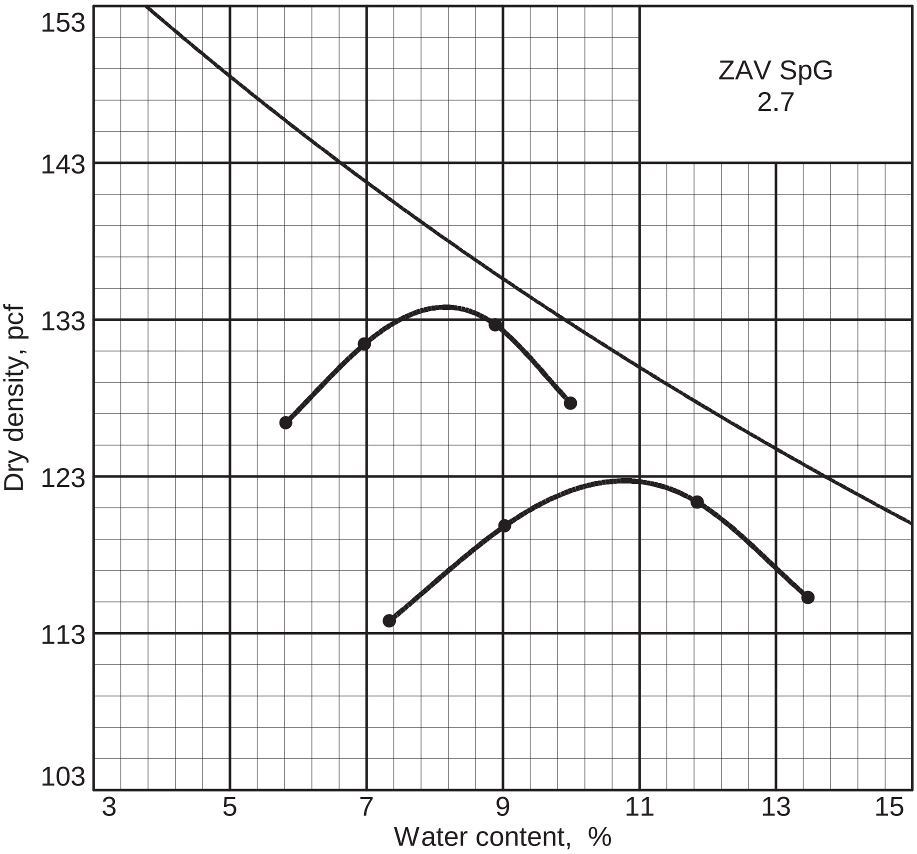
Remarks:

COMPACTION TEST REPORT

COOPER TESTING LABORATORY

Figure

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-00 Method C Modified
Oversize correction applied to each point

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 56
Mold Size: .075 cu.ft.

Test Performed on Material Passing 3/4 in. **Sieve**

Soil Data

NM ______ **Sp.G.** 2.7
LL ______ **PI** ______
%>3/4 in. 32.0 **%<#200** ______
USCS ______ **AASHTO** ______

TESTING DATA

	1	2	3	4	5	6
WM + WS	16.54	16.16	15.52	16.17		
WM	6.36	6.36	6.36	6.36		
WW + T #1	1080.50	1134.20	1199.90	1157.00		
WD + T #1	997.50	1067.30	1138.10	1057.90		
TARE #1	297.10	326.30	295.60	322.30		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	8.9	7.0	5.8	10.0		
DRY DENSITY	132.7	131.4	126.4	127.7		

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 133.8 pcf	122.7 pcf
Optimum moisture = 8.2 %	10.8 %

Material Description
Olive Gray Clayey SAND

Project No. 109-758 **Client:** Amec Foster Wheeler
Project: Project Zeus - 6166150082

● **Source:** B-3 **Sample No.:** S-0-A **Elev./Depth:** 0-5'

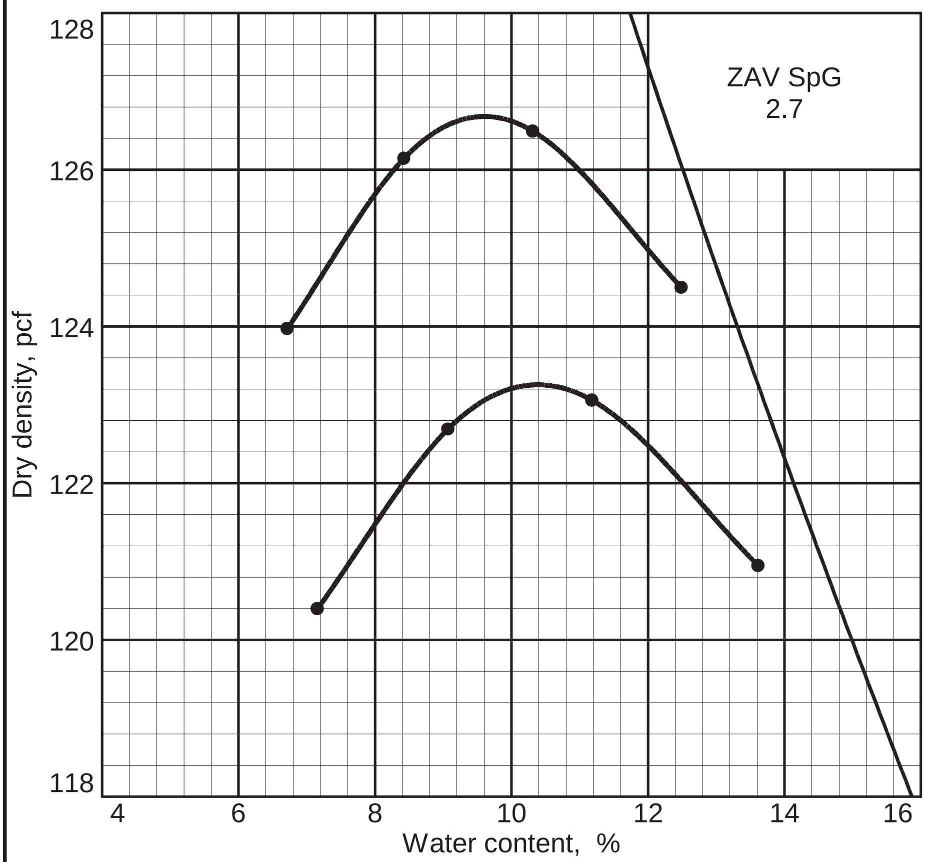
Remarks:
Due to small sample size available several points were run with reused material.

COMPACTION TEST REPORT

COOPER TESTING LABORATORY

Figure

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-00 Method B Modified
Oversize correction applied to each point

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 25
Mold Size: .03333 cu.ft.

Test Performed on Material Passing 3/8 in. **Sieve**

Soil Data

NM ______ **Sp.G.** 2.7
LL 36 **PI** 14
%>3/8 in. 10.6 **%<#200** 43.6
USCS SC **AASHTO** ______

TESTING DATA

	1	2	3	4	5	6
WM + WS	8.82	8.98	9.08	9.10		
WM	4.52	4.52	4.52	4.52		
WW + T #1	827.70	833.10	740.80	559.00		
WD + T #1	794.10	791.10	698.40	503.50		
TARE #1	324.60	327.90	319.10	95.80		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	6.7	8.4	10.3	12.5		
DRY DENSITY	124.0	126.1	126.5	124.5		

ROCK CORRECTED TEST RESULTS	UNCORRECTED	Material Description
Maximum dry density = 126.7 pcf	123.3 pcf	Dark Olive Brown Lean Clayey SAND w/ Gravel
Optimum moisture = 9.6 %	10.4 %	

Project No. 109-758 **Client:** Amec Foster Wheeler
Project: Project Zeus - 6166150082

● **Source:** B-4 **Sample No.:** S-1-A **Elev./Depth:** 0-5'

Remarks:

COMPACTION TEST REPORT

COOPER TESTING LABORATORY

Figure

Moisture-Density-Porosity Report

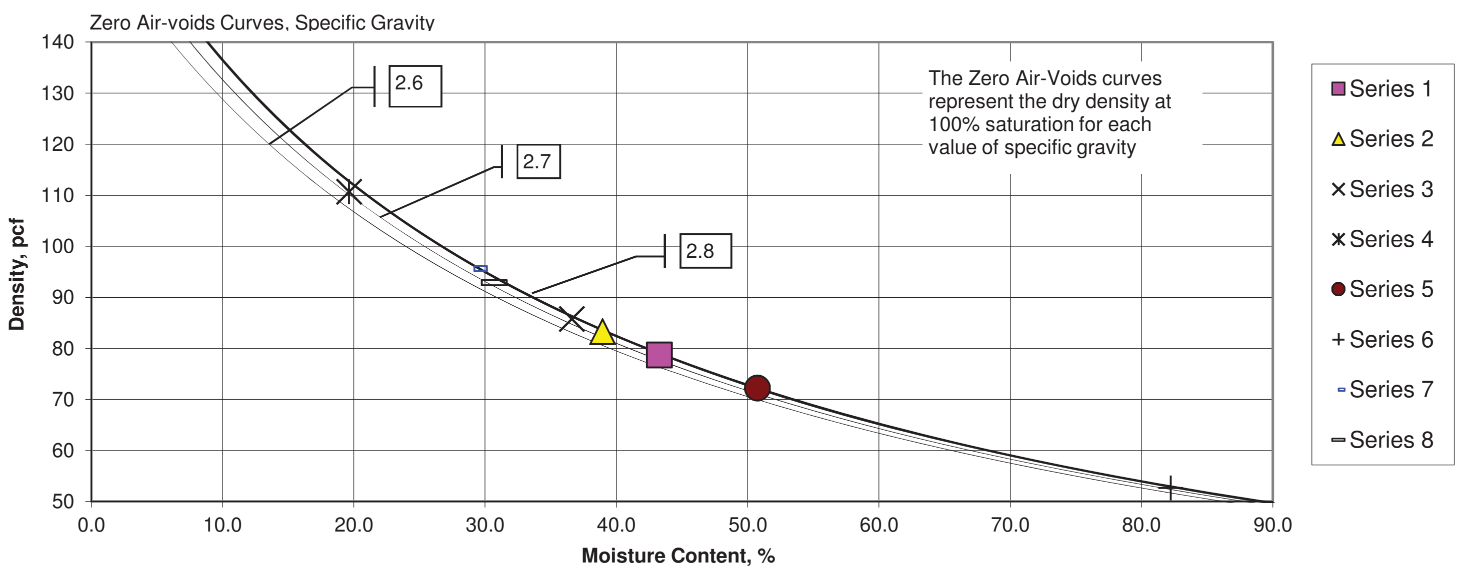
Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No:	109-758a	**Project No.**	6166150082	**By:**	RU
Client:	Amec Foster Wheeler	**Date:**	11/11/15		
Project Name:	Project Zeus	**Remarks:**			

Boring:	B-1	B-1	B-1	B-1	B-3	B-4	B-4	B-4
Sample:	S-11-3	S-11-4	S-15-4	S-16-3	S-9-3	S-3-4	S-6-4	S-9-4
Depth, ft:	53-53.5	53.5-54	96-96.5	110.5-111	65.5-66	6.5-7	47-47.5	62-62.5
Visual Description:	Olive Gray Lean CLAY	Olive Gray Lean CLAY	Olive Gray Sandy CLAY	Light Olive Gray Sandy CLAY	Light Gray Fat CLAY	Gray CLAY	Olive Fat CLAY w/ Sand	Olive Gray Fat CLAY
Actual G_s								
Assumed G_s	2.80	2.80	2.80	2.80	2.80	2.80	2.80	2.80
Moisture, %	43.3	38.9	36.6	19.6	50.7	82.2	29.2	30.7
Wet Unit wt, pcf	112.7	115.7	117.1	132.4	108.9	95.9	123.5	121.3
Dry Unit wt, pcf	78.7	83.2	85.8	110.7	72.2	52.6	95.6	92.8
Dry Bulk Dens.pb, (g/cc)	1.26	1.33	1.37	1.77	1.16	0.84	1.53	1.49
Saturation, %	99.1	99.0	98.5	94.7	99.9	99.1	98.5	97.2
Total Porosity, %	55.0	52.4	51.0	36.7	58.7	69.9	45.3	46.9
Volumetric Water Cont, Θw,%	54.5	51.9	50.2	34.8	58.7	69.3	44.7	45.6
Volumetric Air Cont., Θa,%	0.5	0.5	0.7	1.9	0.1	0.7	0.7	1.3
Void Ratio	1.22	1.10	1.04	0.58	1.42	2.32	0.83	0.88
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

Moisture-Density



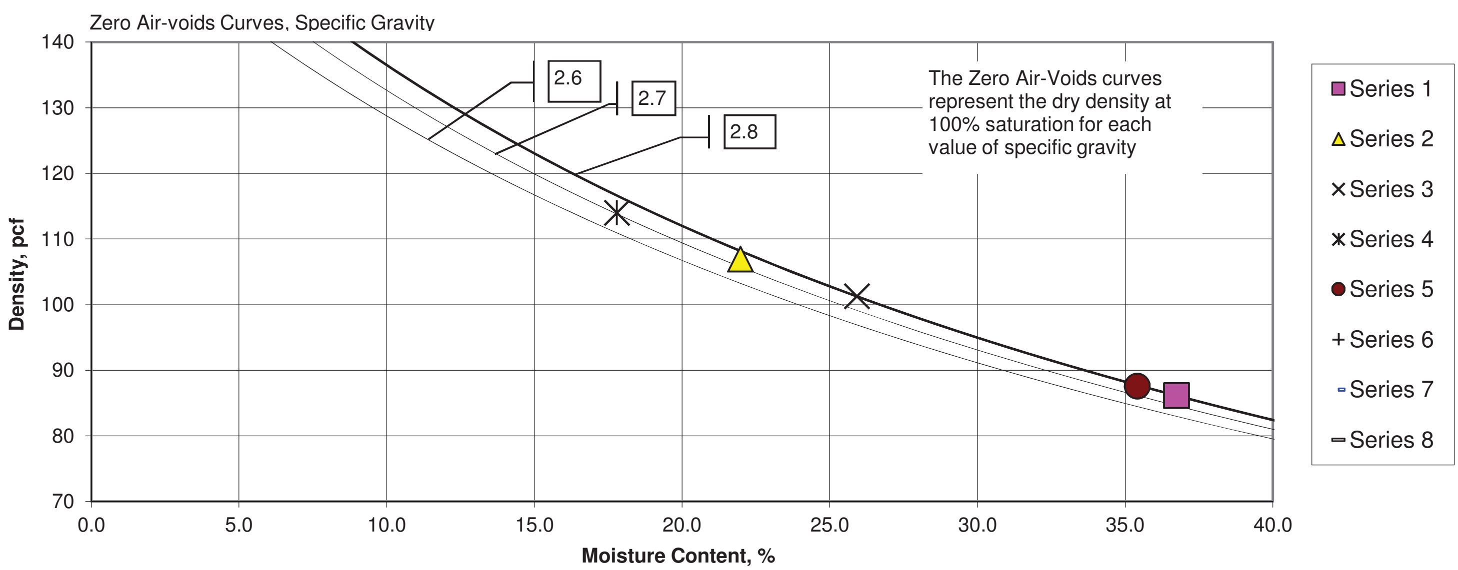
Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No:	109-758b	**Project No.**	6166150082	**By:**	RU
Client:	Amec Foster Wheeler	**Date:**	11/11/15		
Project Name:	Project Zeus	**Remarks:**			

Boring:	B-4	B-4	B-4	B-5	B-5			
Sample:	S-10-3	S-11-3	S-12-3	S-6-3	S-7-3			
Depth, ft:	76.5-77	88.5-89	95-95.5	75.5-76	95.5-96			
Visual Description:	Olive Lean CLAY	Olive Gray CLAY w/ Sand	Greenish Gray Fat CLAY	Olive Brown Sandy Lean CLAY	Mottled Olive Clayey SAND			
Actual G_s								
Assumed G_s	2.80	2.80	2.80	2.80	2.80			
Moisture, %	36.7	22.0	25.9	17.8	35.4			
Wet Unit wt, pcf	117.8	130.4	127.5	134.3	118.6			
Dry Unit wt, pcf	86.1	106.9	101.3	114.0	87.6			
Dry Bulk Dens.pb, (g/cc)	1.38	1.71	1.62	1.83	1.40			
Saturation, %	99.8	96.7	99.8	93.2	99.4			
Total Porosity, %	50.8	38.9	42.1	34.8	49.9			
Volumetric Water Cont,Θw,%	50.6	37.6	42.0	32.5	49.6			
Volumetric Air Cont., Θa,%	0.1	1.3	0.1	2.4	0.3			
Void Ratio	1.03	0.64	0.73	0.53	1.00			
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

Moisture-Density



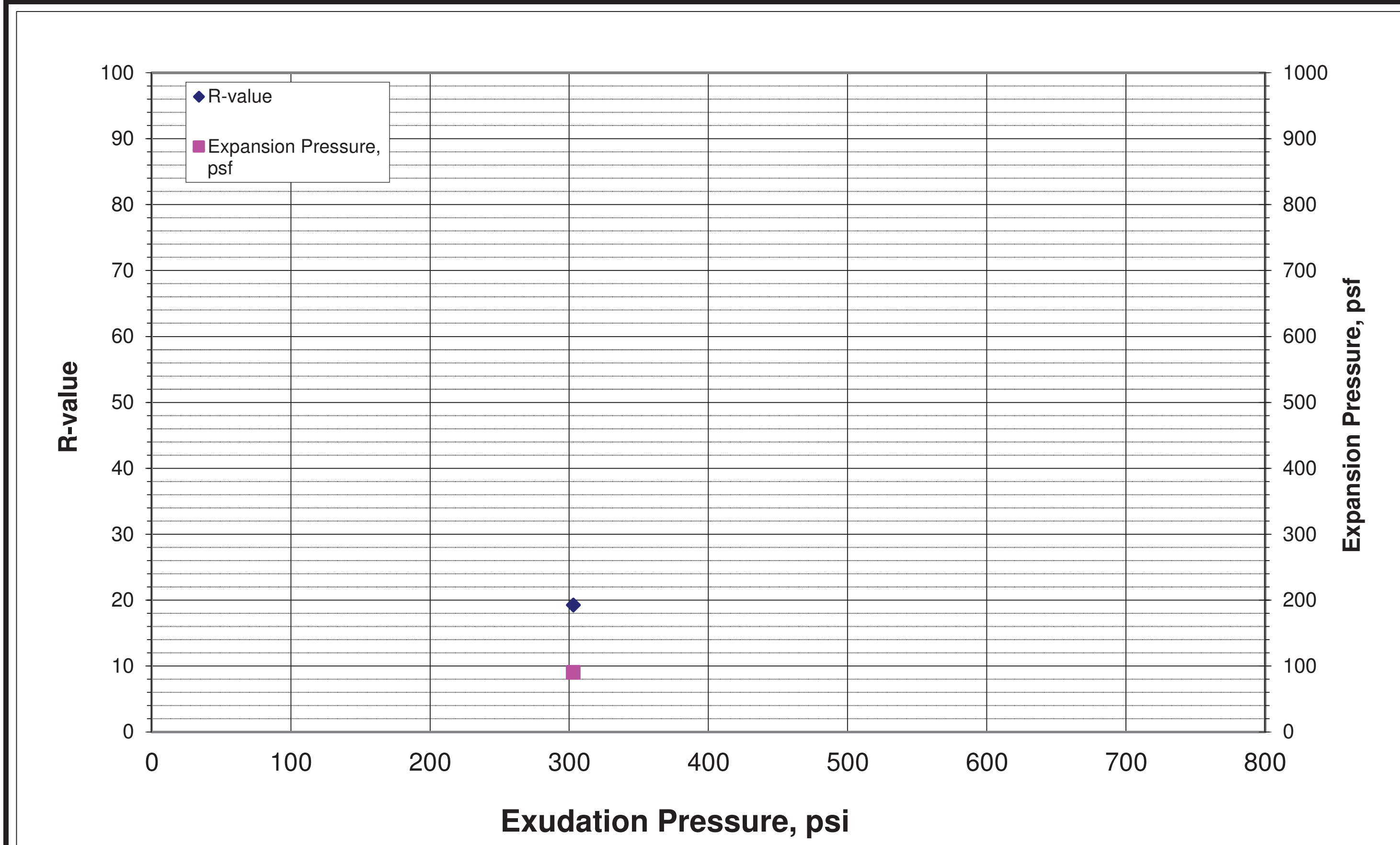
R-value Test Report (Caltrans 301)

Job No.:	109-758	**Date:**	11/11/15	**Initial Moisture,**	15.6%
Client:	Amec Foster Wheeler	**Tested**	MD	**R-value by Stabilometer**	<5
Project:	Project Zeus - 6166150082	**Reduced**	RU		
Sample	B-1,S-1-A @ 0-5'	**Checked**	DC	**Expansion Pressure**	psf
Soil Type:	Very Dark Brown Gravelly Fat CLAY w/ Sand				

Specimen Number	A	B	C	D
Exudation Pressure, psi	303			
Prepared Weight, grams	1200			
Final Water Added, grams/cc	81			
Weight of Soil & Mold, grams	3120			
Weight of Mold, grams	2078			
Height After Compaction, in.	2.55			
Moisture Content, %	23.4			
Dry Density, pcf	100.3			
Expansion Pressure, psf	90.3			
Stabilometer @ 1000				
Stabilometer @ 2000	128			
Turns Displacement	2.62			
R-value	19			

Remarks:

Soil extruded from the mold giving a false exudation pressure. Per Caltrans, the R-Value test was terminated and an R-Value of less than 5 was reported.

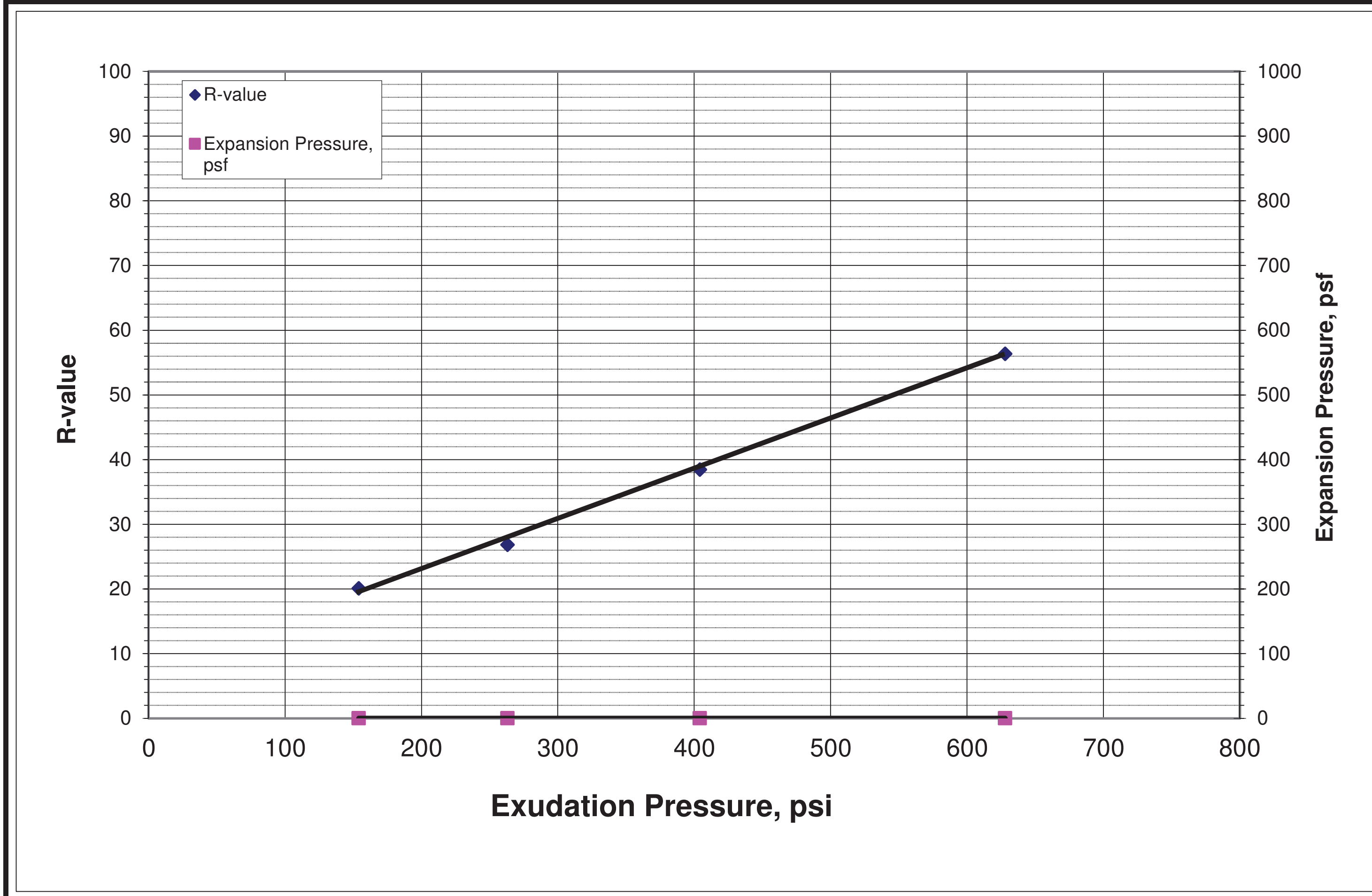


R-value Test Report (Caltrans 301)

Job No.:	109-758	**Date:**	11/11/15	**Initial Moisture,**	8.1%
Client:	Amec Foster Wheeler	**Tested**	MD	**R-value by Stabilometer**	31
Project:	Project Zeus - 6166150082	**Reduced**	RU		
Sample	B-6;S-1-A @ 0-5'	**Checked**	DC	**Expansion Pressure**	0 psf
Soil Type:	Yellowish Brown Lean Clayey GRAVEL w/ Sand				

Specimen Number	A	B	C	D
Exudation Pressure, psi	263	154	628	404
Prepared Weight, grams	1300	1300	1300	1300
Final Water Added, grams/cc	42	52	30	35
Weight of Soil & Mold, grams	3261	3269	3187	3235
Weight of Mold, grams	2102	2106	2076	2078
Height After Compaction, in.	2.51	2.6	2.41	2.55
Moisture Content, %	11.6	12.4	10.6	11.0
Dry Density, pcf	125.3	120.5	126.2	123.8
Expansion Pressure, psf	0.0	0.0	0.0	0.0
Stabilometer @ 1000				
Stabilometer @ 2000	110	122	66	100
Turns Displacement	3.1	3.34	2.51	2.4
R-value	27	20	56	38

Remarks:



Expansion Index

ASTM D-4829-07 X

CTL Job No.:	109-758	**Boring:**	B-3	**Date:**	11/12/2015
Client:	AMEC Foster Wheeler	**Sample:**	S-0-A	**By:**	PJ
Project Name:	Project Zeus	**Depth:**	0-5'		
Project No:	6166150082				

Visual Description: Olive Gray Clayey SAND

Processing:

Percent Passing #4 Sieve

Total Air Dry Weight:	N/A
Wt. Retained on #4 Sieve:	N/A
% Retained	N/A
% Passing #4 Sieve:	N/A

Sample Dimensions

Height (in.)= 1.001 **Diameter (in.) =** 4.017

Moisture Calcs

	Initial	Final
Tare #		
Wet Wt. + Tare, (gm)	687.0	730.9
Dry Wt. + Tare, (gm)	644.7	644.7
Tare Wt., (gm)	307.7	307.7
Wt. Of Water, (gm)	42.3	86.2
% Water	12.6	25.6

Remolding:

Tamp two lifts, 15 blows/lift @ slightly below optimum moisture content

	Initial	Final	
Ring & Sample:	574.9	618.9	grams
Ring:	195.6	195.6	grams
Remolded Wet Wt.:	379.3	423.3	grams
Wet Density	113.9	120.0	pcf
Dry Density	101.2	95.6	pcf
% Sat. = (2.7)(dry dens.)(m/c) / 168.48 - (dry dens.)	51.0	90.5	UBC Saturation range 49-51% ASTM Saturation range 48-52%

Expansion Test:

Date	Time	Dial	Delta h, %
11/10/2015	16:14	0.0000	0.000
11/10/2015	17:35	-0.0550	5.495
11/11/2015	11:11	-0.0592	5.914
11/11/2015	12:11	-0.0592	5.914
		Total Dial	5.9

Tested with 1 psi Surcharge

Remarks:

Expansion Index

Results

$$\frac{\text{initial dial - final dial}}{\text{initial sample height}} \times 1000$$

EI =	59

This test is a simplified index test and may not show the full potential for expansion and/or shrinkage. Use result with caution! See ASTM D 3877 or D4546

Expansion Index

ASTM D-4829-07 X

CTL Job No.:	109-758	**Boring:**	B-4	**Date:**	11/12/2015
Client:	AMEC Foster Wheeler	**Sample:**	S-1-A	**By:**	PJ
Project Name:	Project Zeus	**Depth:**	0-5'		
Project No:	6166150082				

Visual Description: Dark Olive Brown Lean Clayey SAND w/ Gravel

Processing:

Percent Passing #4 Sieve

Total Air Dry Weight:	N/A
Wt. Retained on #4 Sieve:	N/A
% Retained	N/A
% Passing #4 Sieve:	N/A

Sample Dimensions

Height (in.)= 1.001 **Diameter (in.) =** 4.017

Moisture Calcs

	Initial	Final
Tare #		
Wet Wt. + Tare, (gm)	700.4	734.1
Dry Wt. + Tare, (gm)	660.3	660.3
Tare Wt., (gm)	310.8	310.8
Wt. Of Water, (gm)	40.1	73.8
% Water	11.5	21.1

Remolding:

Tamp two lifts, 15 blows/lift @ slightly below optimum moisture content

	Initial	Final	
Ring & Sample:	585.3	619.1	grams
Ring:	195.7	195.7	grams
Remolded Wet Wt.:	389.6	423.4	grams
Wet Density	117.0	124.3	pcf
Dry Density	105.0	102.6	pcf
% Sat. = $\frac{(2.7)(\text{dry dens.})(\text{m/c})}{168.48 - (\text{dry dens.})}$	51.2	88.8	UBC Saturation range 49-51% ASTM Saturation range 48-52%

Expansion Test:

Date	Time	Dial	Delta h, %
11/10/2015	17:00	0.0000	0.000
11/10/2015	17:36	-0.0205	2.048
11/11/2015	11:11	-0.0232	2.318
11/11/2015	12:11	-0.0232	2.318
		Total Dial	2.3

Tested with 1 psi Surcharge

Remarks:

Expansion Index **Results**

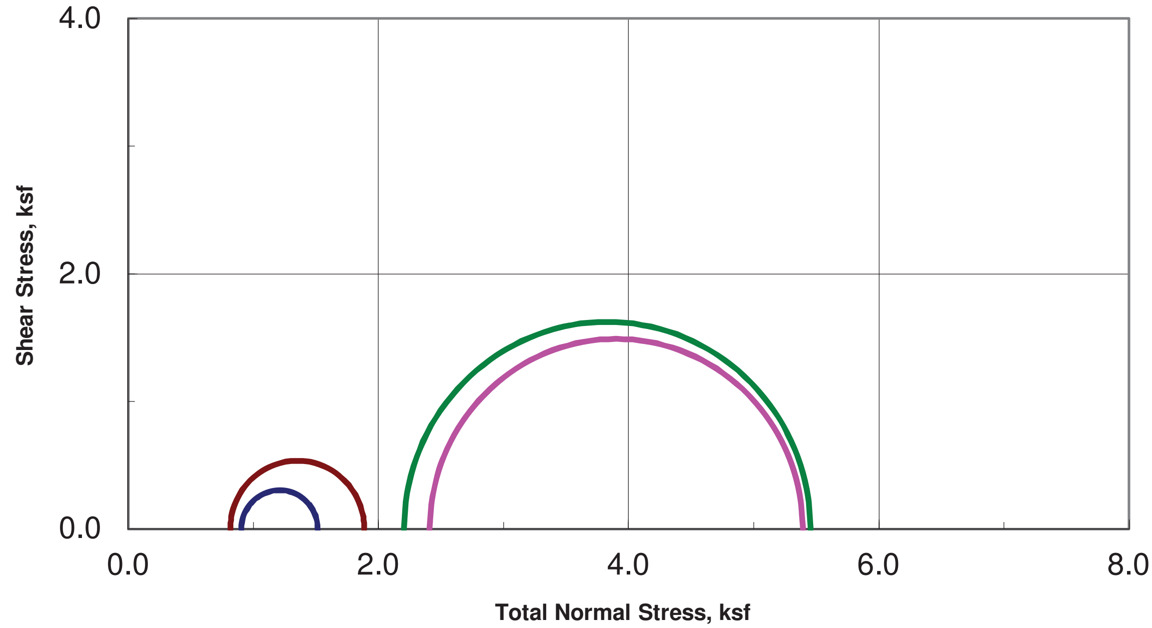
$\frac{\text{initial dial - final dial}}{\text{initial sample height}} \times 1000$

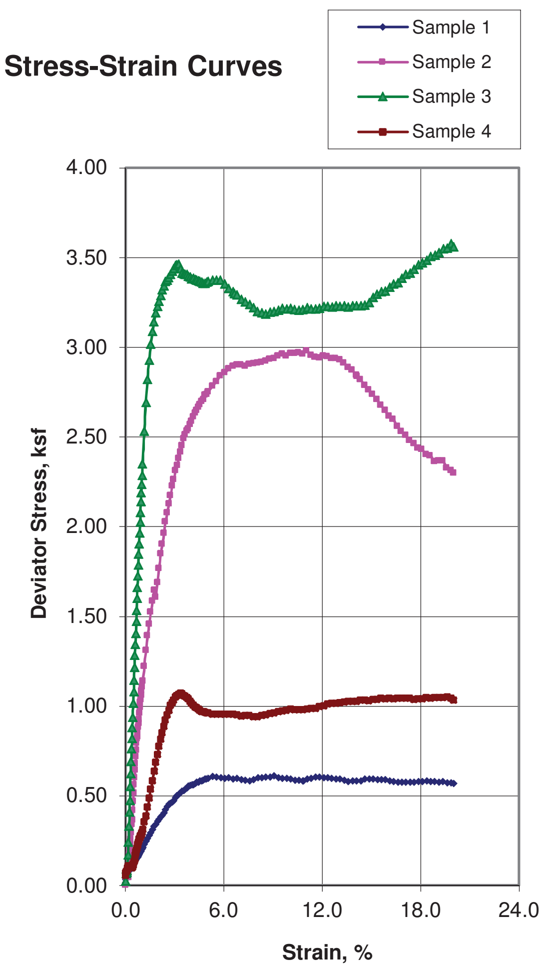
EI = 23

This test is a simplified index test and may not show the full potential for expansion and/or shrinkage. Use result with caution! See ASTM D 3877 or D4546

Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test
ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	79.9	44.7	34.8	59.2
Dry Den,pcf	52.8	75.6	85.3	61.2
Void Ratio	2.194	1.231	0.976	1.755
Saturation %	98.4	98.1	96.3	91.1
Height in	6.09	6.09	6.09	6.10
Diameter in	2.88	2.87	2.87	2.87
Cell psi	6.3	16.7	15.3	5.7
Strain %	9.06	11.05	15.00	3.34
Deviator, ksf	0.611	2.982	3.248	1.071
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.061	0.061	0.061	0.061
Job No.:	**109-758**			
Client:	**Amec Foster Wheeler**			
Project:	**Project Zeus - 6166150082**			
Boring:	B-3	B-3	B-4	B-5
Sample:	S-4	S-7	S-7	S-2
Depth ft:	13-15.3	53-55.5(Tip-1")	48-49.8(Tip-5")	10-12.5(Tip-11")

Visual Soil Description	
Sample #	
1	Dark Greenish Fat CLAY (Bay Mud)
2	Light Greenish Gray CLAY
3	Light Greenish Gray SILT w/ Sand
4	Dark Greenish Gray CLAY (Bay Mud)

Remarks:

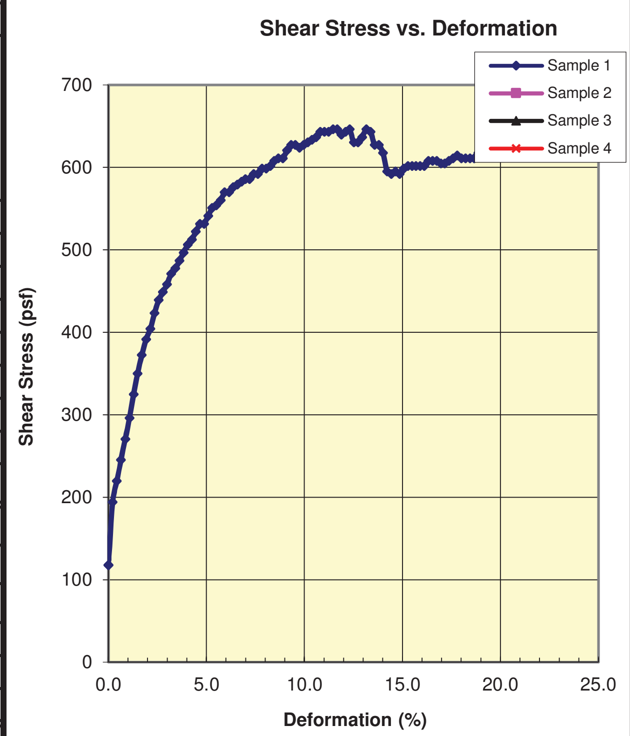
Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

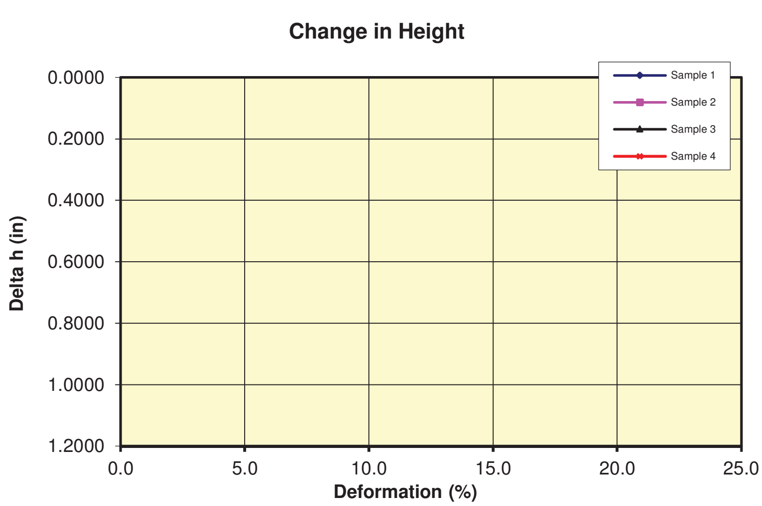
Consolidated Undrained Direct Shear (ASTM D3080M)

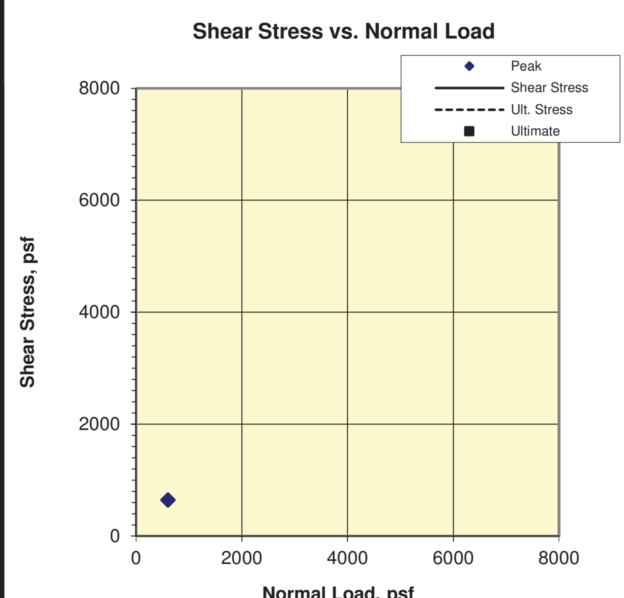
CTL Job #:	109-758	Project #:	6166150082	By:	MD
Client:	Amec Foster Wheeler	Date:	11/13/2015	Checked:	PJ
Project Name:	Project Zeus	Remolding Info:			

Phi (deg)		Ult. Phi (deg)	
Cohesion (psf)		Ult. Cohesion (psf)	

Specimen Data	1	2	3	4
Boring:	B-3			
Sample:	S-1-3			
Depth (ft):	6-7.5			
Visual Description:	Yellowish Brown Lean CLAY w/ Sand			
Normal Load (psf)	600			
Dry Mass of Specimen (g)	116.4			
Initial Height (in)	1.01			
Initial Diameter (in)	2.40			
Initial Void Ratio	0.743			
Initial Moisture (%)	22.6			
Initial Wet Density (pcf)	118.6			
Initial Dry Density (pcf)	96.7			
Initial Saturation (%)	82.3			
ΔHeight Consol (in)	0.0147			
At Test Void Ratio	0.718			
At Test Moisture (%)	25.3			
At Test Wet Density (pcf)	123.0			
At Test Dry Density (pcf)	98.2			
At Test Saturation (%)	95.2			
Strain Rate (%/min)	1.1			
Strengths Picked at	Peak			
Shear Stress (psf)	646			
ΔHeight (in) at Peak				
Ultimate Stress (psf)				







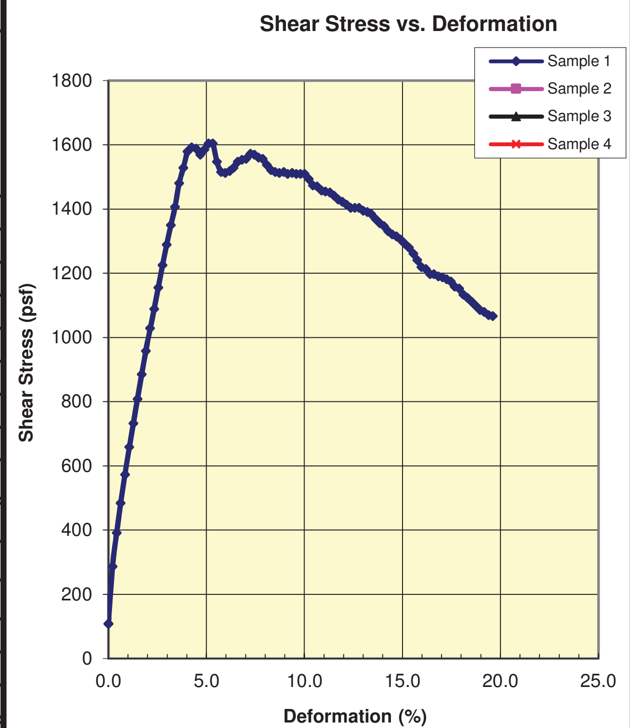
Remarks: *DS-CU* A fully undrained condition may not be attained in this test. ΔH is not measured during undrained direct shear tests.

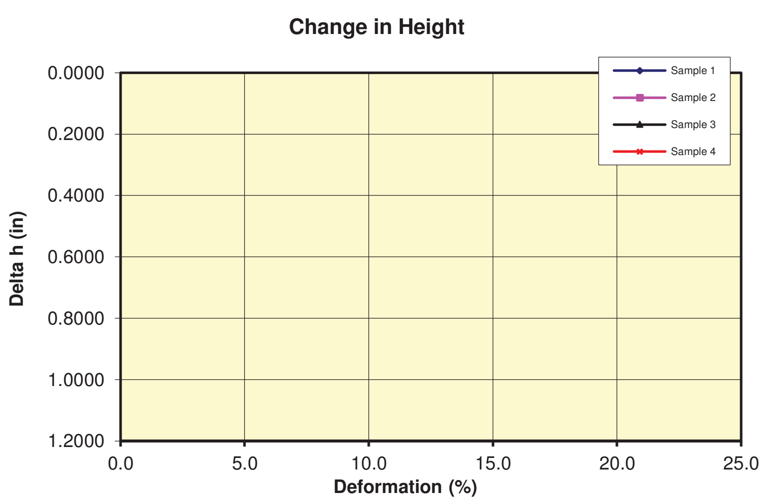
Consolidated Undrained Direct Shear (ASTM D3080M)

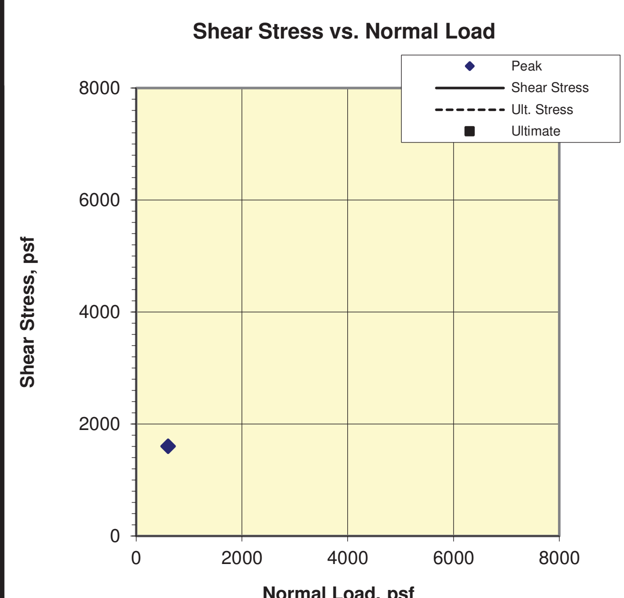
CTL Job #:	109-758	Project #:	6166150082	By:	MD
Client:	Amec Foster Wheeler	Date:	11/13/2015	Checked:	PJ
Project Name:	Project Zeus	Remolding Info:			

Phi (deg)		Ult. Phi (deg)	
Cohesion (psf)		Ult. Cohesion (psf)	

Specimen Data	1	2	3	4
Boring:	B-6			
Sample:	S-2-4			
Depth (ft):	6-6.5			
Visual Description:	Olive Clayey SAND			
Normal Load (psf)	600			
Dry Mass of Specimen (g)	124.7			
Initial Height (in)	1.02			
Initial Diameter (in)	2.40			
Initial Void Ratio	0.660			
Initial Moisture (%)	21.8			
Initial Wet Density (pcf)	126.0			
Initial Dry Density (pcf)	103.4			
Initial Saturation (%)	91.1			
ΔHeight Consol (in)	0.0113			
At Test Void Ratio	0.641			
At Test Moisture (%)	23.0			
At Test Wet Density (pcf)	128.7			
At Test Dry Density (pcf)	104.7			
At Test Saturation (%)	98.5			
Strain Rate (%/min)	1.0			
Strengths Picked at	Peak			
Shear Stress (psf)	1604			
ΔHeight (in) at Peak				
Ultimate Stress (psf)				



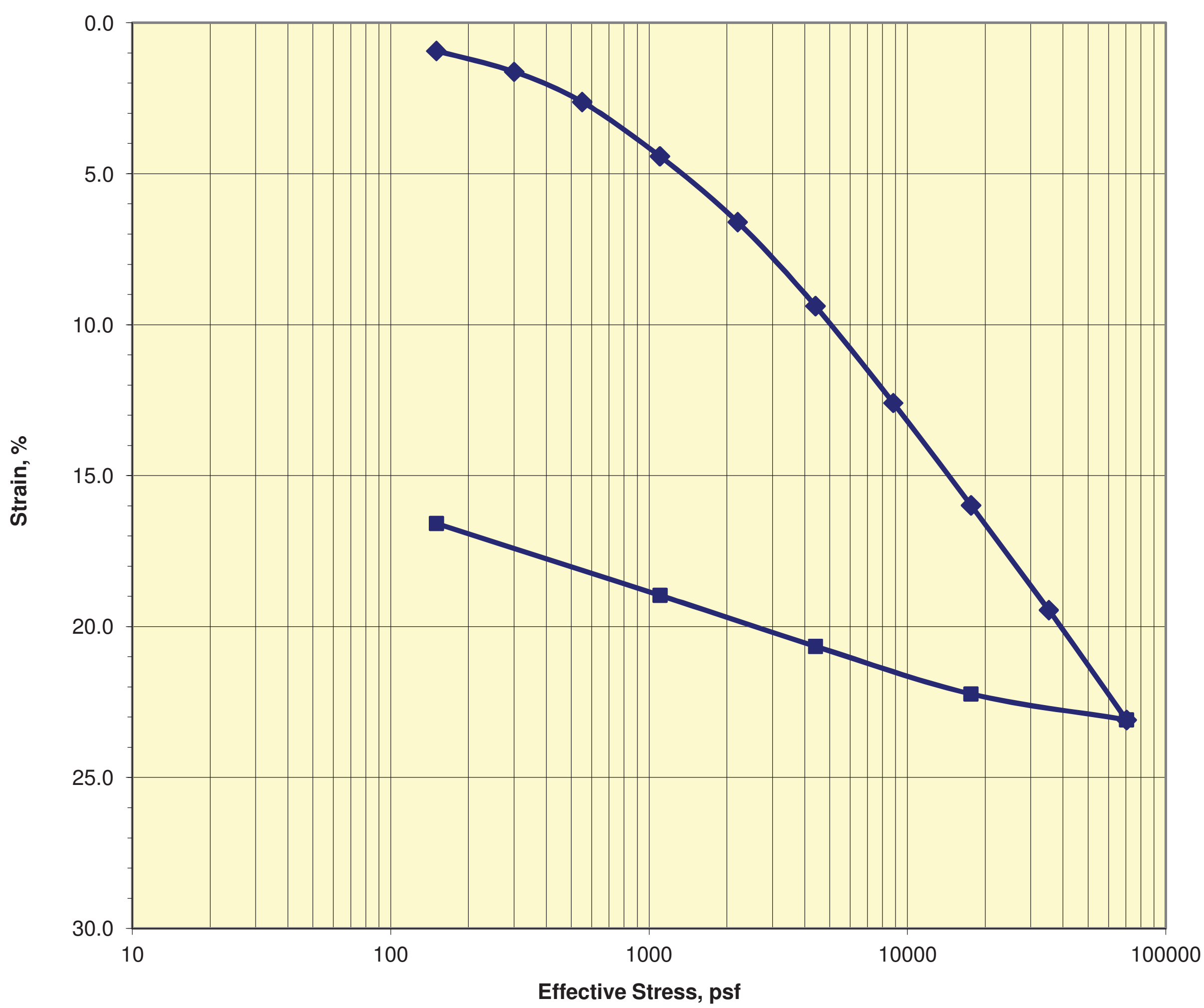




Remarks: *DS-CU* A fully undrained condition may not be attained in this test. ΔH is not measured during undrained direct shear tests.

Consolidation Test
ASTM D2435

Job No.:	109-758	**Boring:**	B-1	**Run By:**	MD
Client:	AMEC	**Sample:**	S-4	**Reduced:**	PJ
Project:	Zeus - 6166150082	**Depth, ft.:**	7.5-9.2(Tip-5")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray Gravelly Lean CLAY			**Date:**	11/20/2015



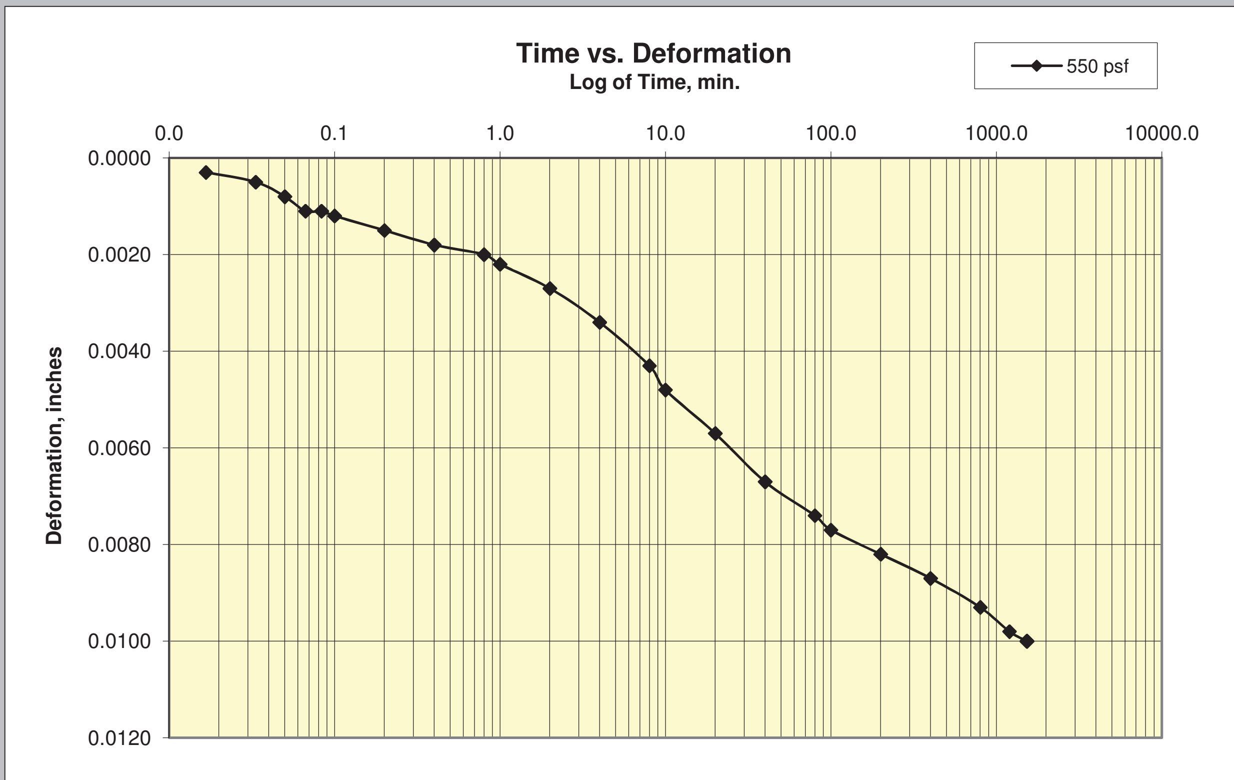
Assumed Gs 2.75	Initial	Final
Moisture %:	32.0	22.3
Dry Density, pcf:	88.9	106.5
Void Ratio:	0.932	0.613
% Saturation:	94.3	100.0

Remarks:

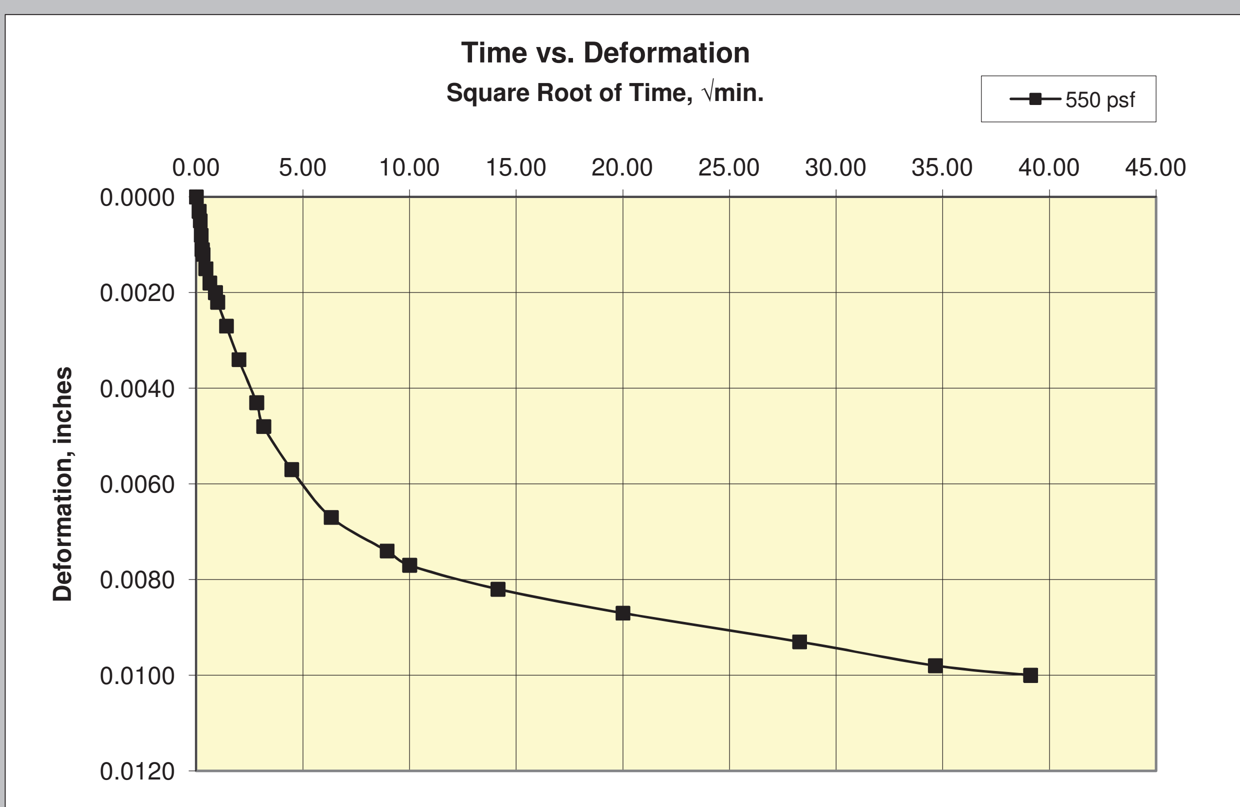
Cooper Testing Labs, Inc.

Load 3

550 psf



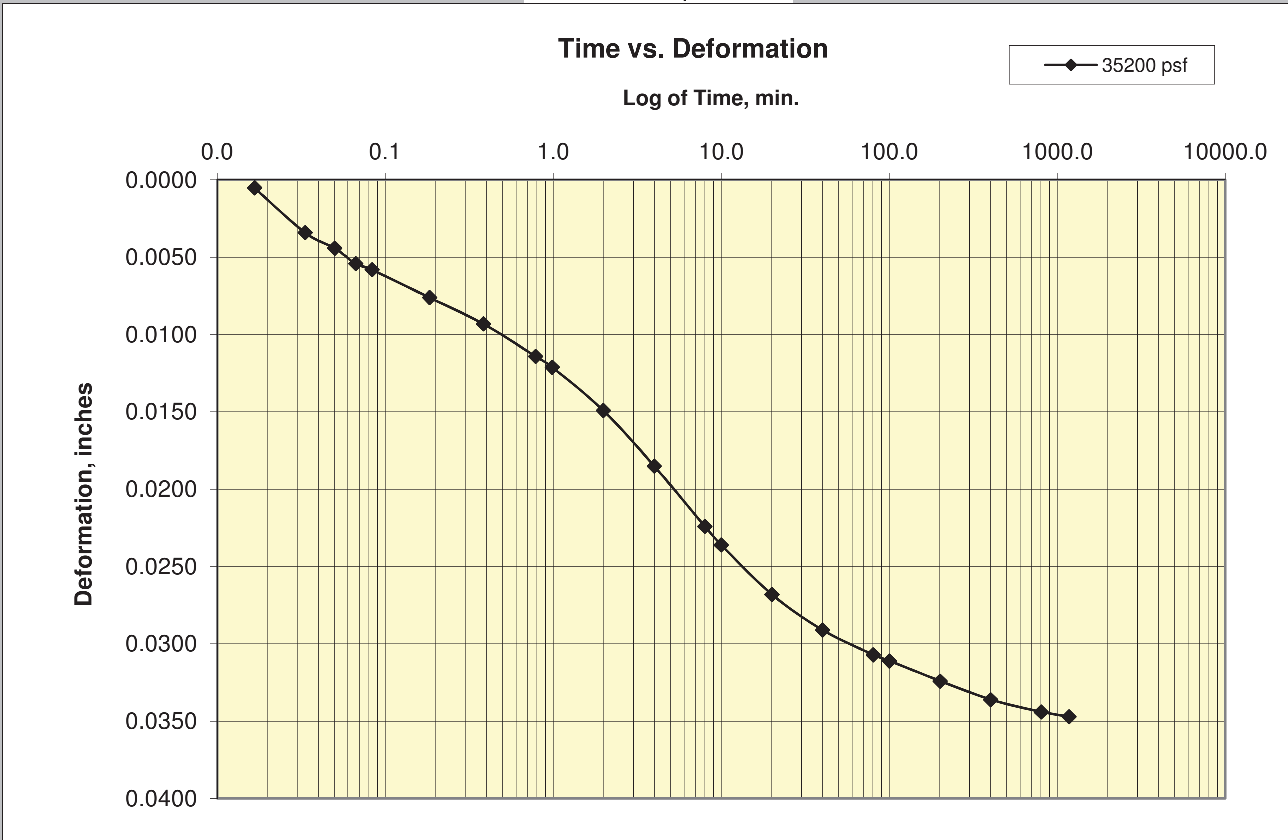
550 psf



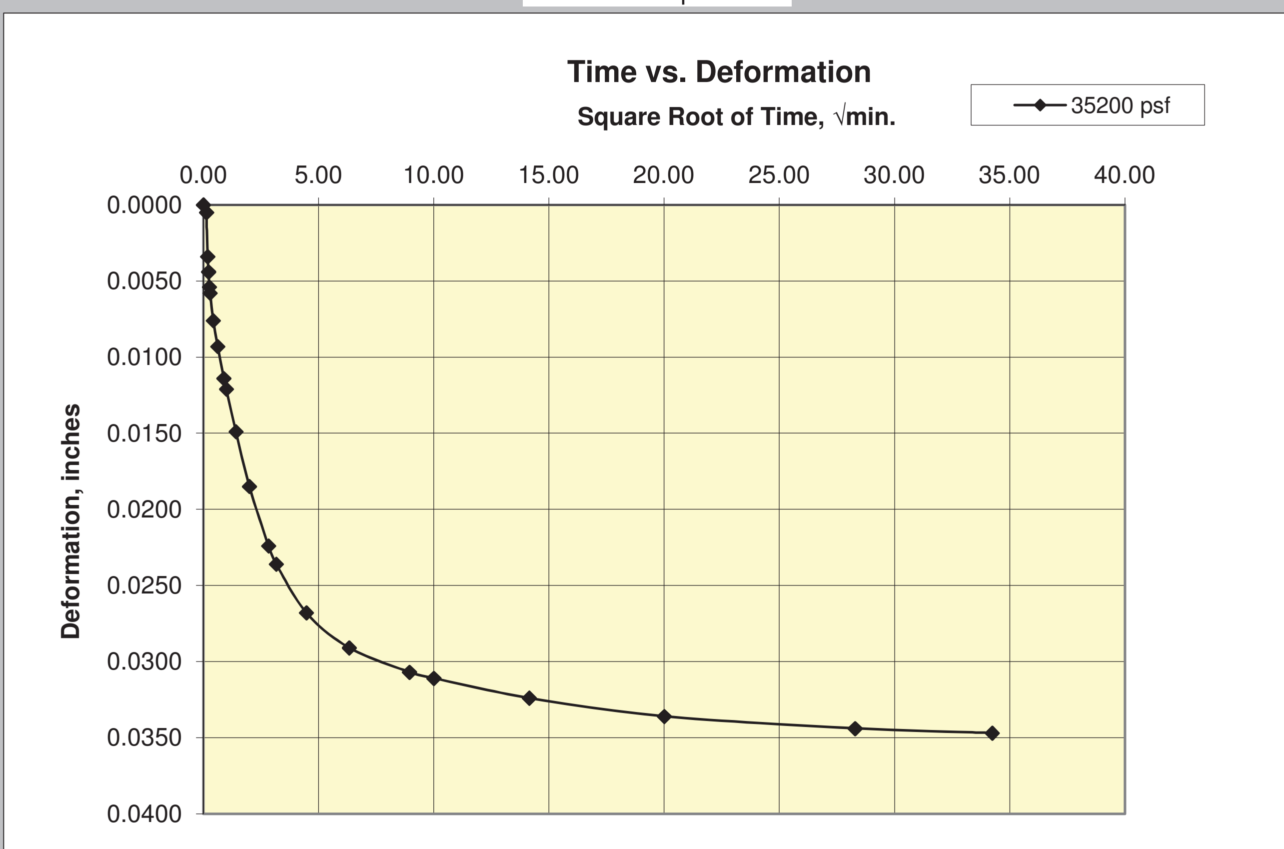
Cooper Testing Labs, Inc.

Load 9

35200 psf

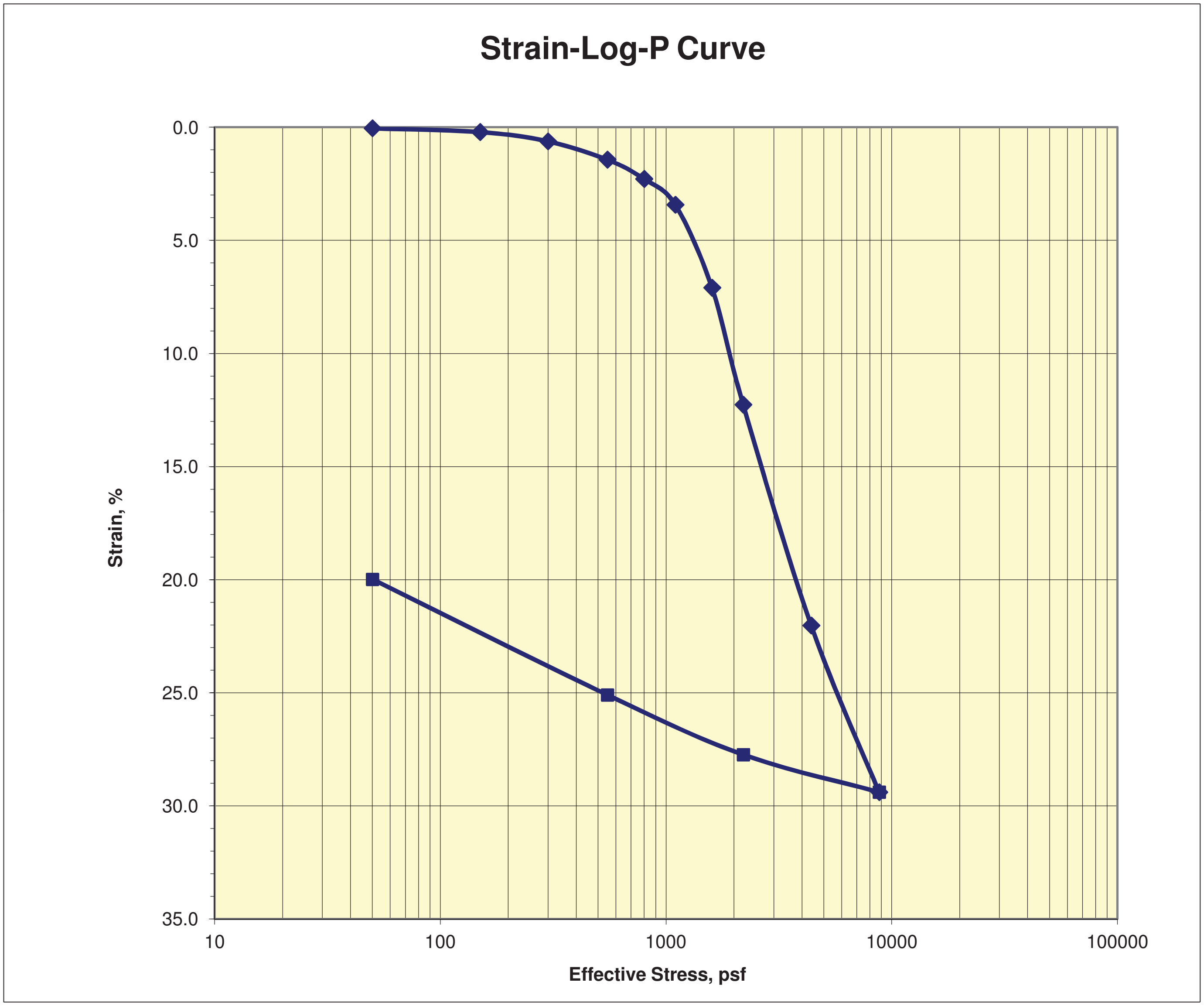


35200 psf



Consolidation Test
ASTM D2435

Job No.:	109-758	**Boring:**	B-1	**Run By:**	MD
Client:	AMEC	**Sample:**	S-6	**Reduced:**	PJ
Project:	Zeus - 6166150082	**Depth, ft.:**	25-27.5(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray Fat CLAY (Bay Mud)			**Date:**	11/18/2015

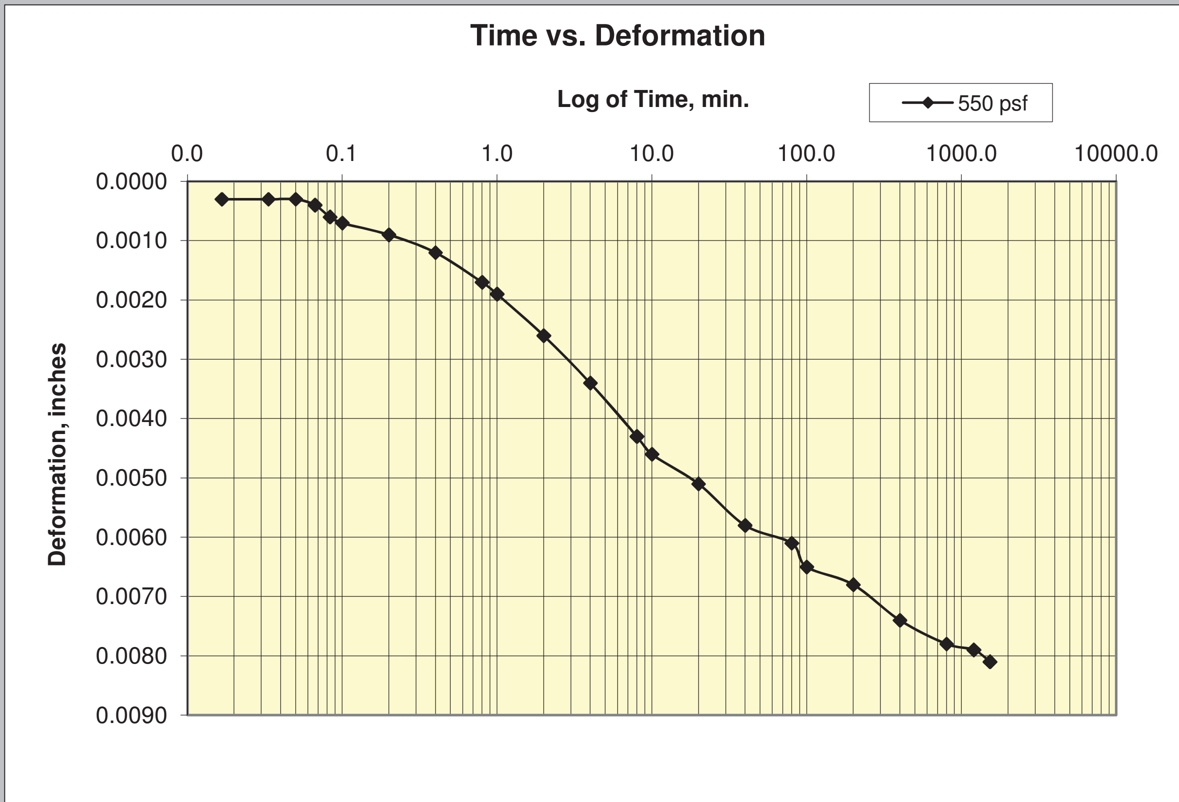


Assumed Gs 2.7	**Initial**	**Final**
Moisture %:	87.2	62.5
Dry Density, pcf:	49.7	62.7
Void Ratio:	2.392	1.688
% Saturation:	98.4	100.0

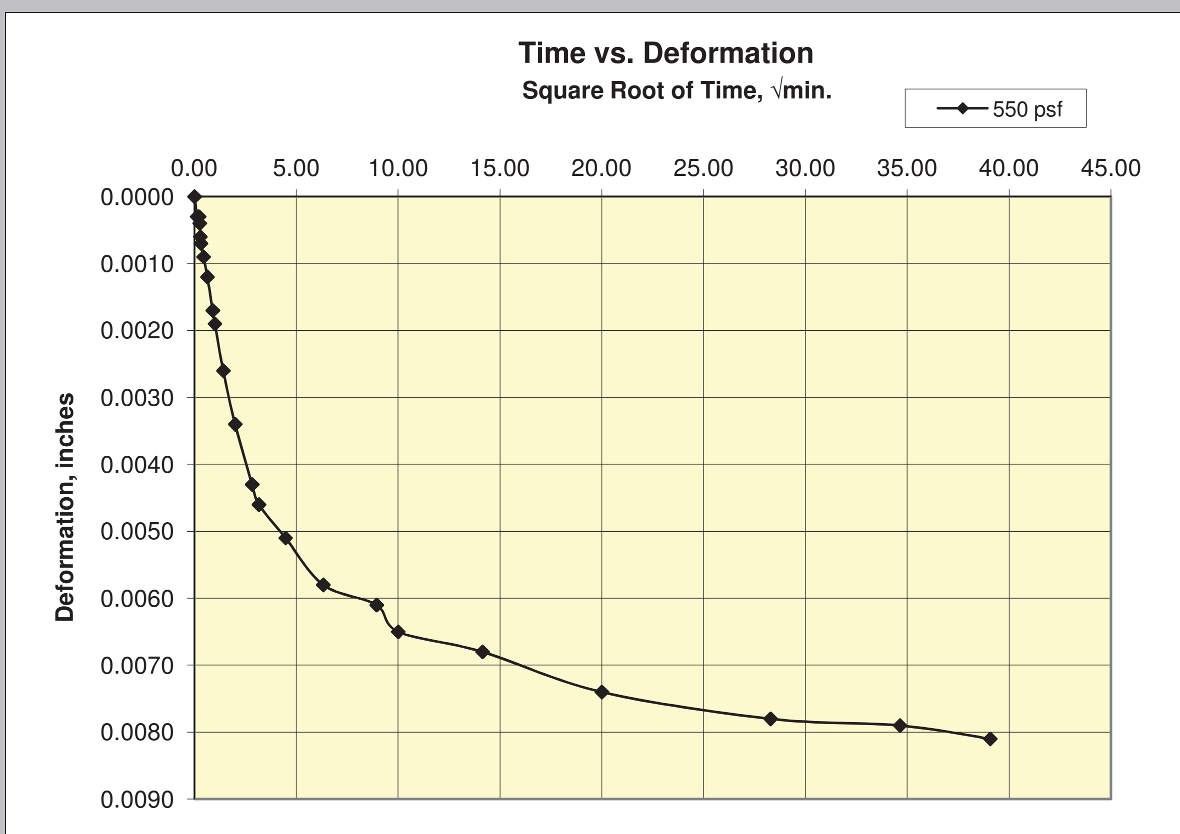
Remarks:

Cooper Testing Labs, Inc.

Load 4 | 550 psf



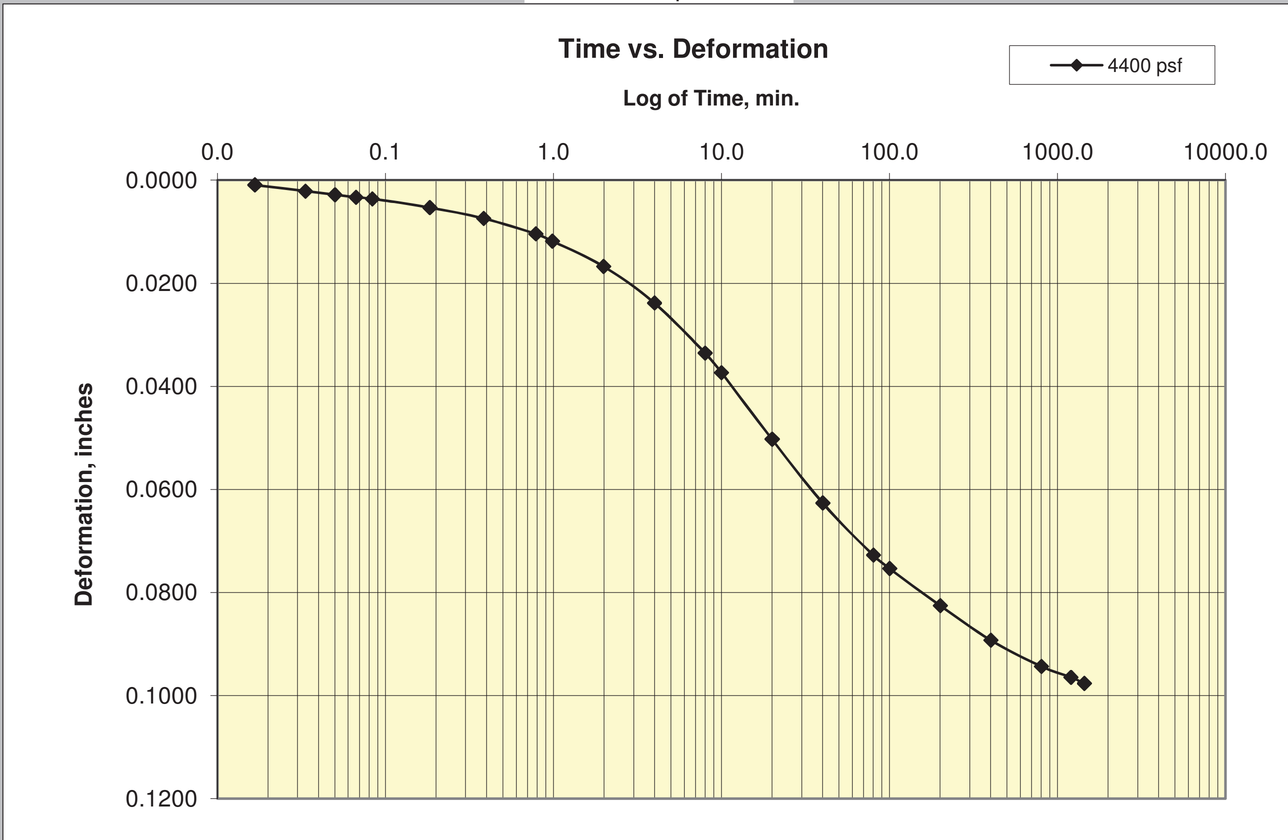
550 psf



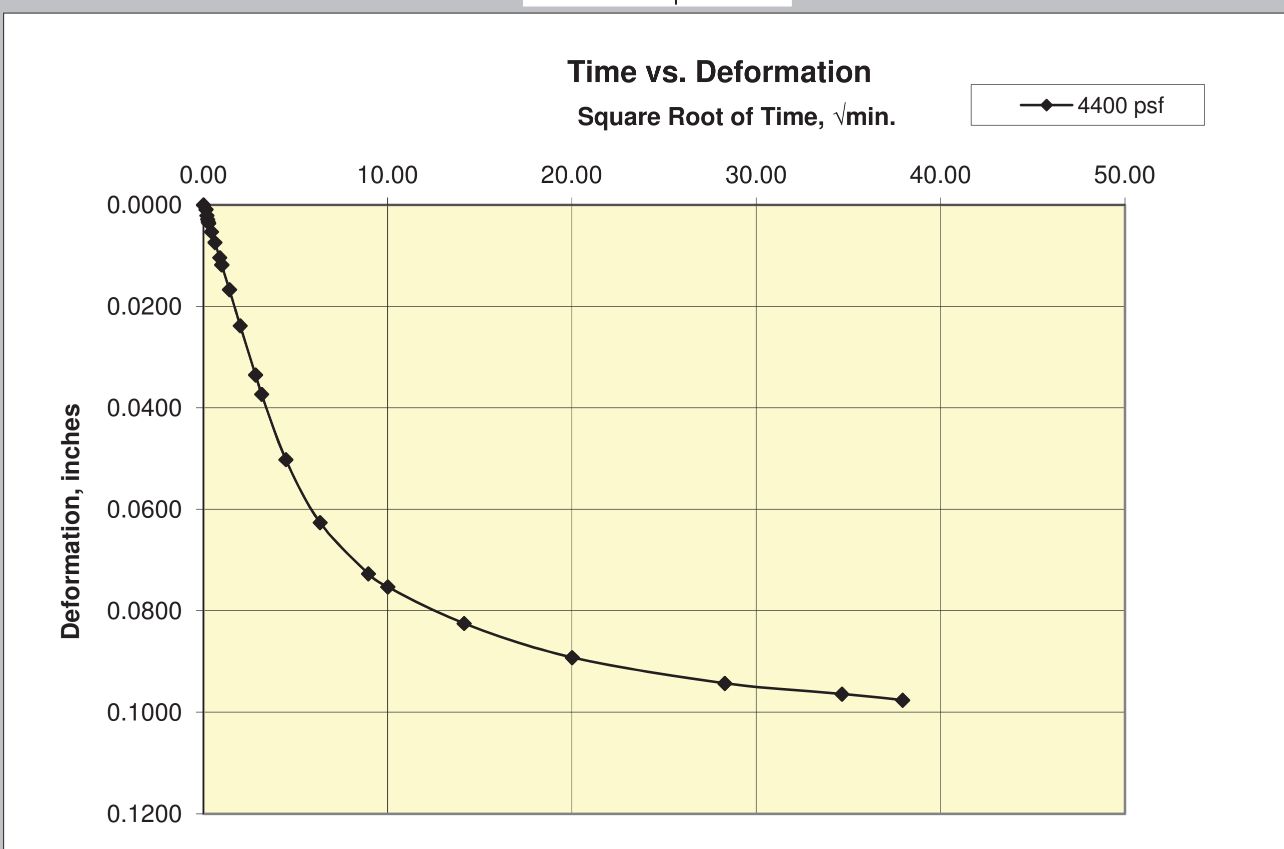
Cooper Testing Labs, Inc.

Load 9

4400 psf

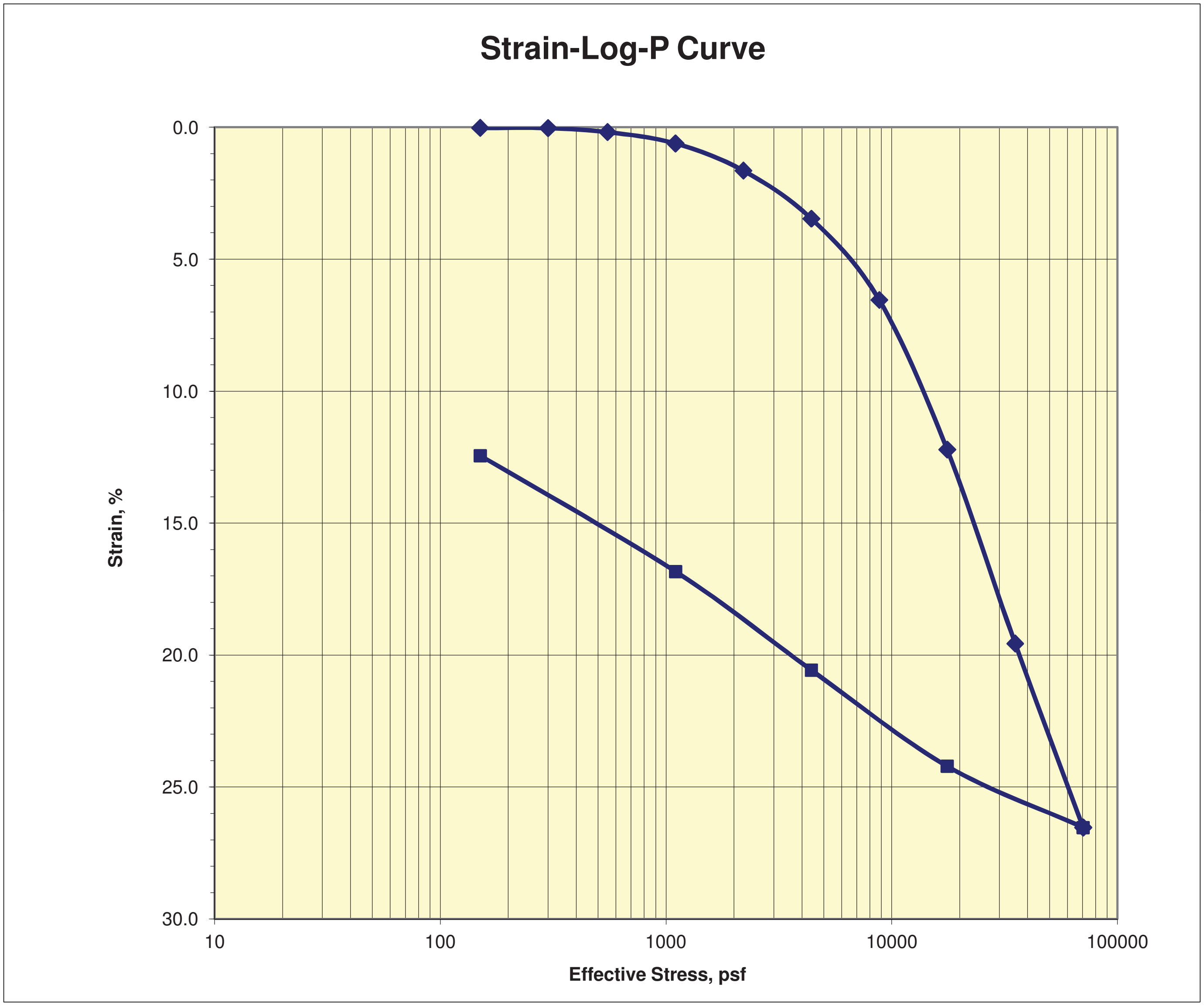


4400 psf



Consolidation Test
ASTM D2435

Job No.:	109-758	**Boring:**	B-1	**Run By:**	MD
Client:	AMEC	**Sample:**	S-10	**Reduced:**	PJ
Project:	Zeus - 6166150082	**Depth, ft.:**	50-52.5(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Olive Gray Fat CLAY			**Date:**	11/18/2015



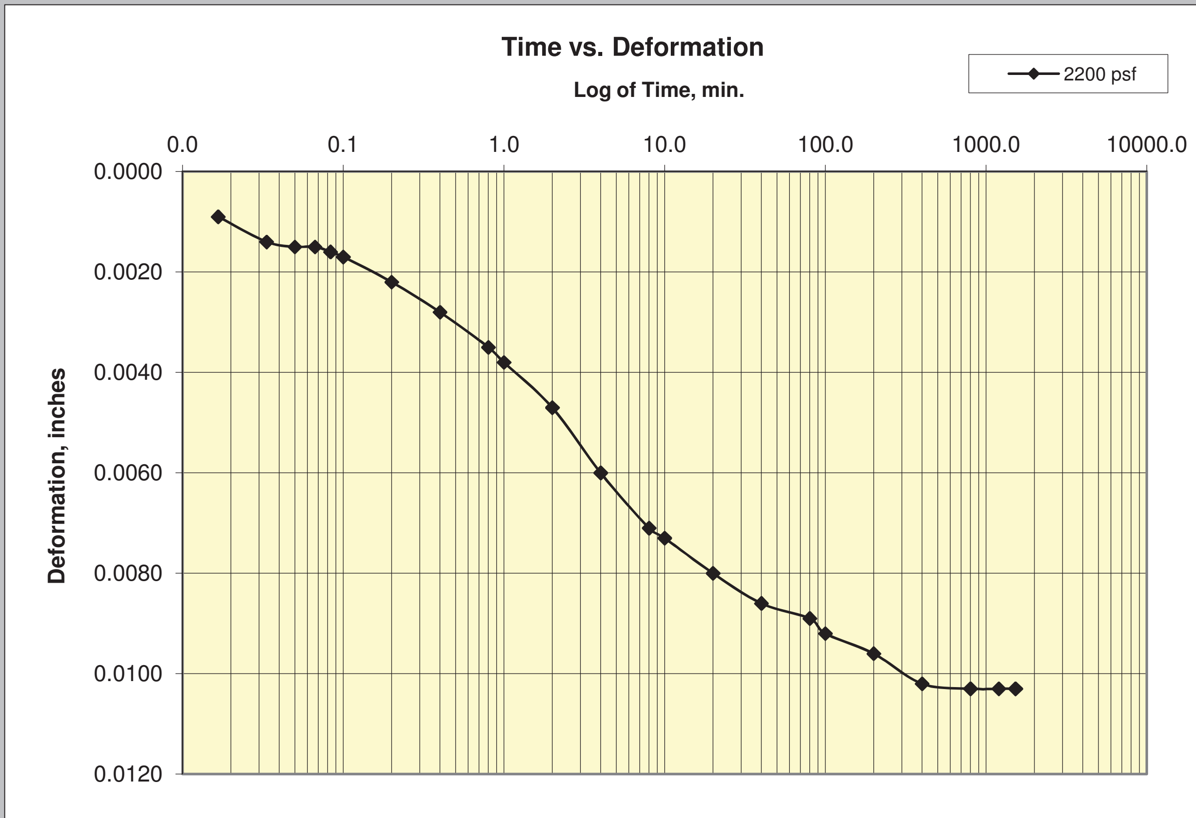
Assumed Gs 2.75	**Initial**	**Final**
Moisture %:	43.5	34.1
Dry Density, pcf:	77.6	88.6
Void Ratio:	1.211	0.937
% Saturation:	98.7	100.0

Remarks:

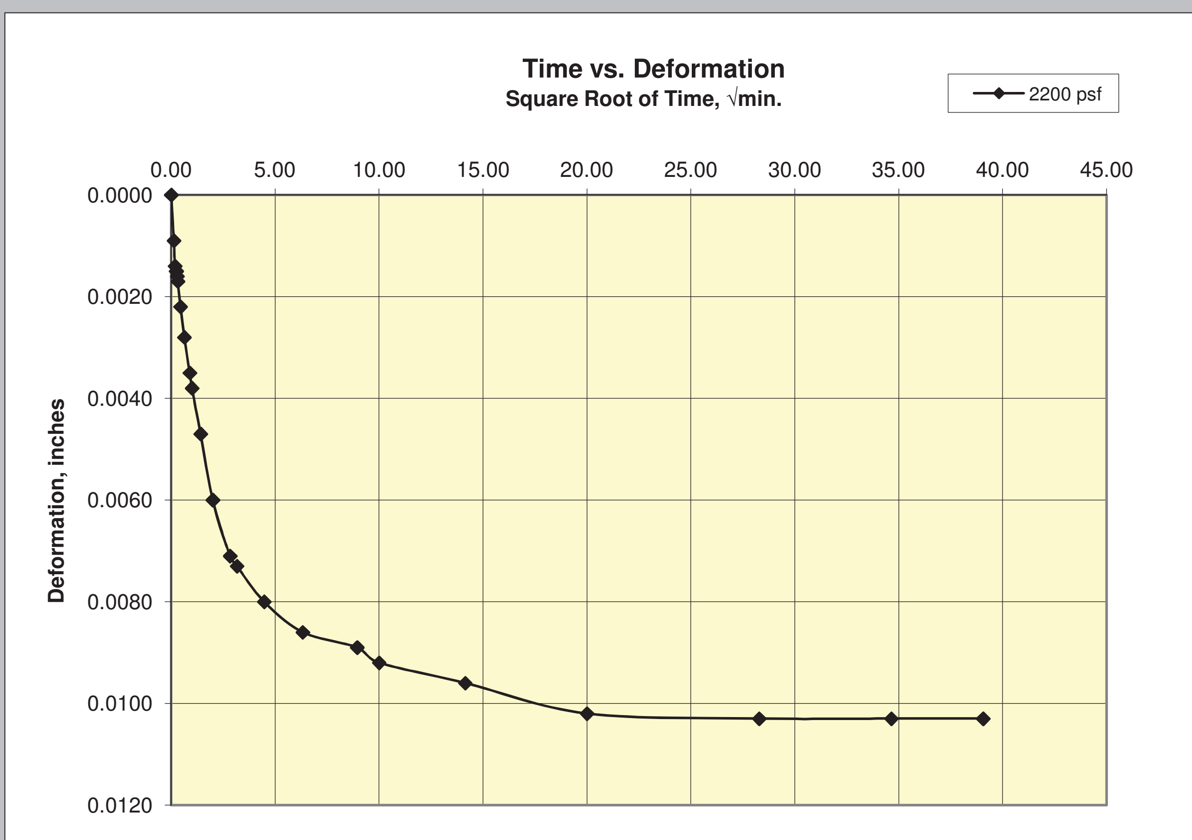
Cooper Testing Labs, Inc.

Load 5

2200 psf



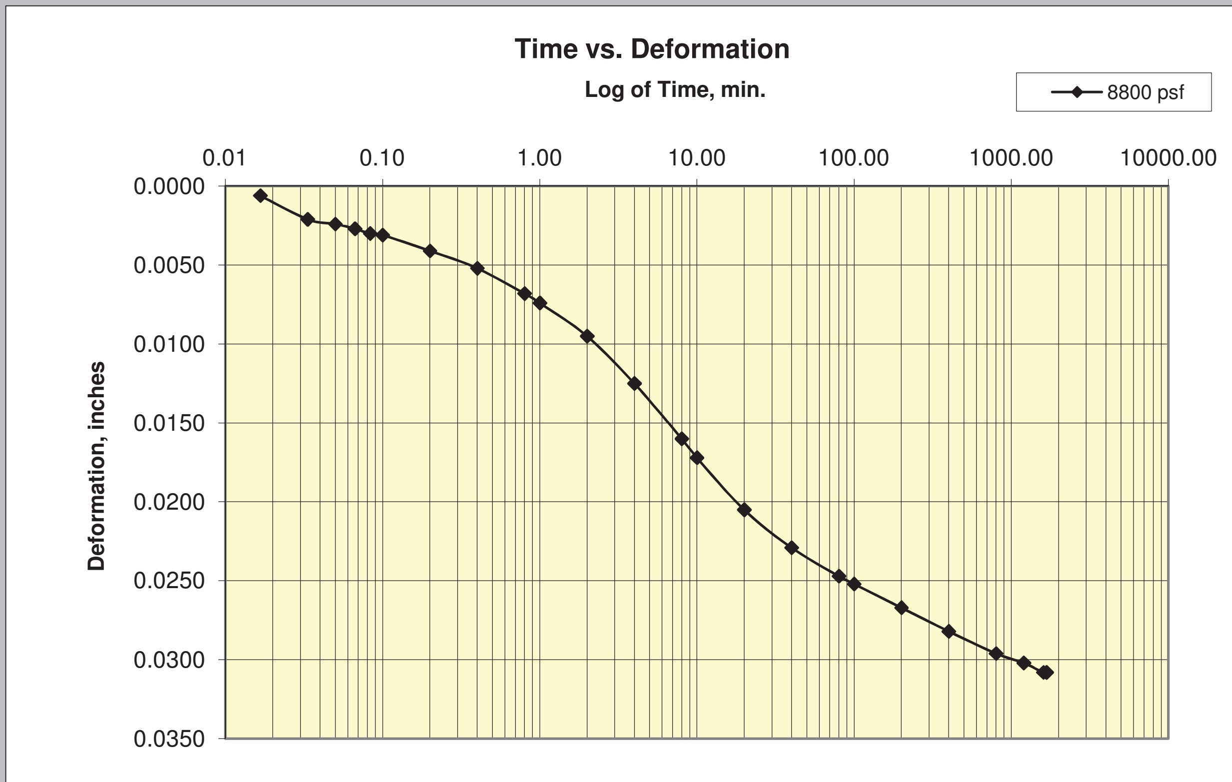
2200 psf



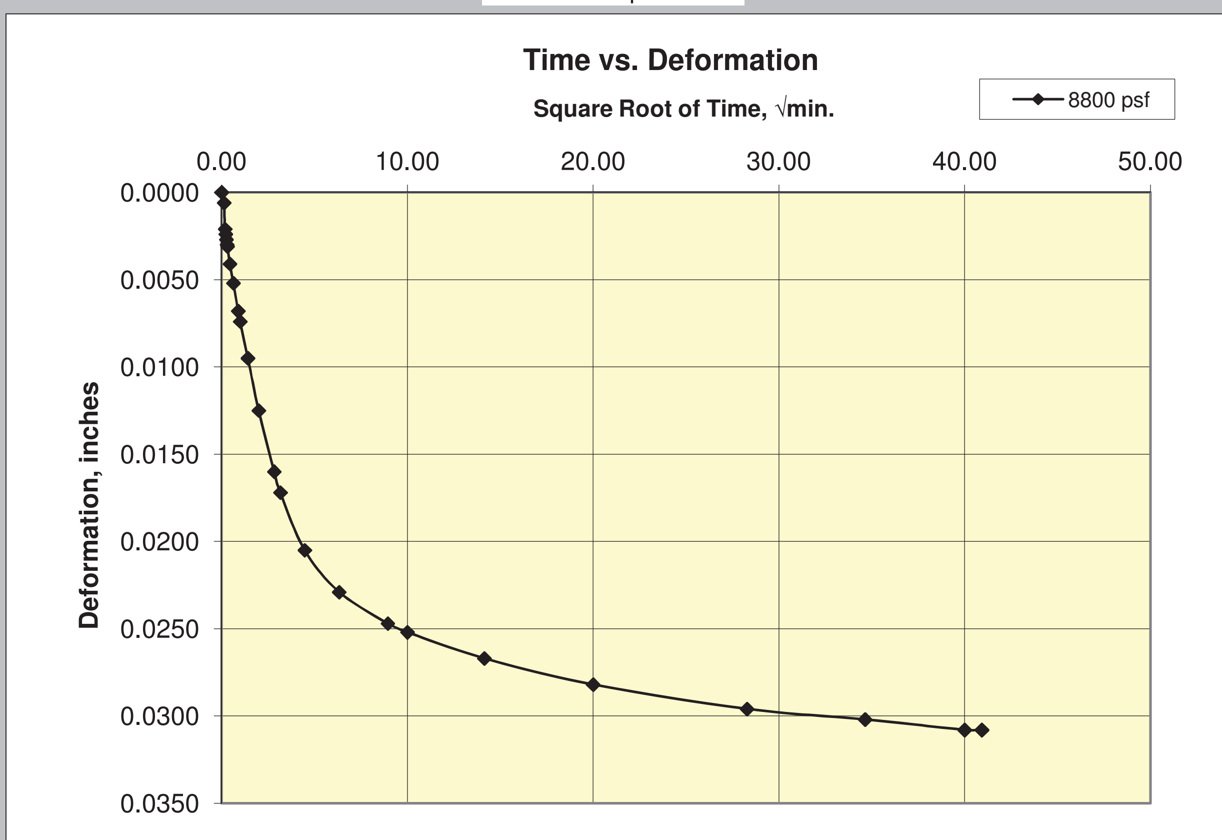
Cooper Testing Labs, Inc.

Load 7

8800 psf



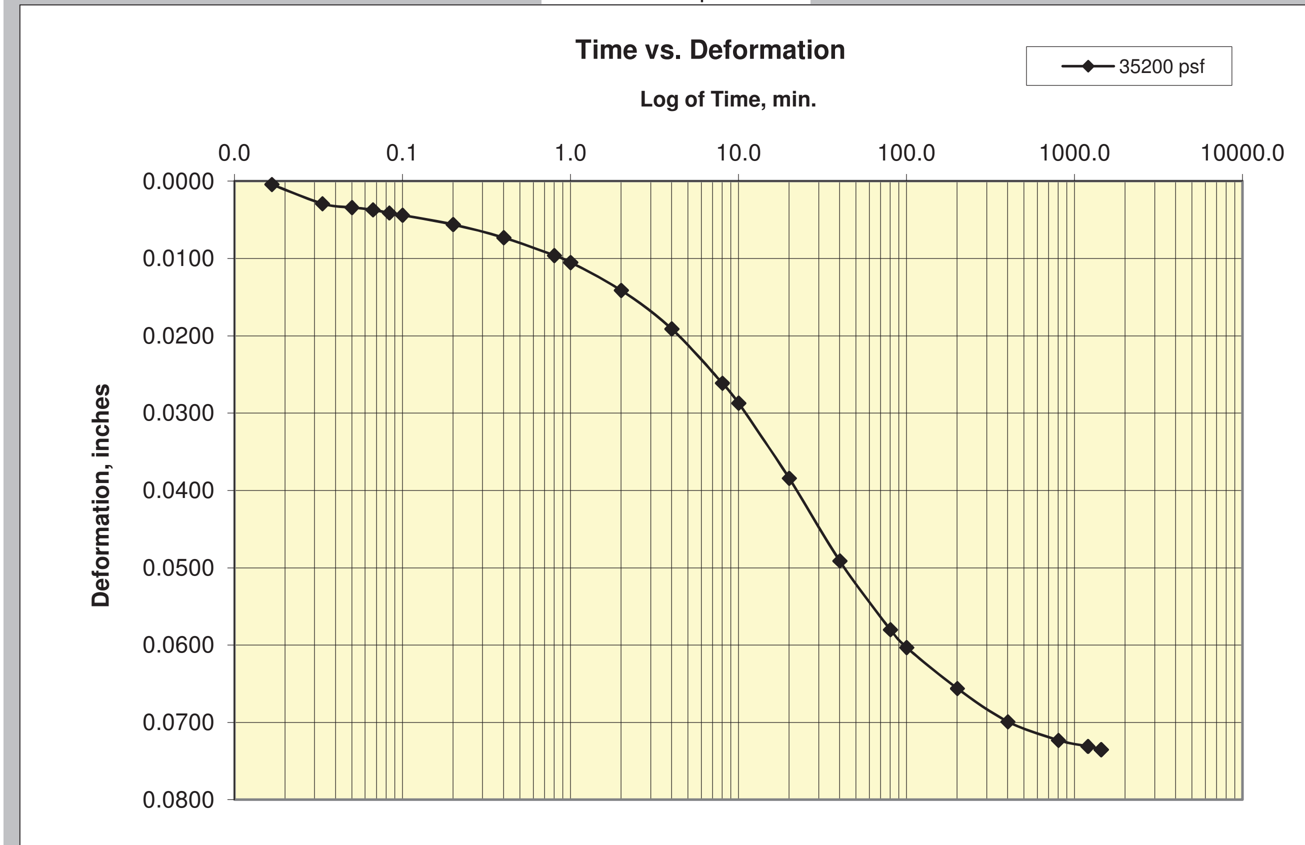
8800 psf



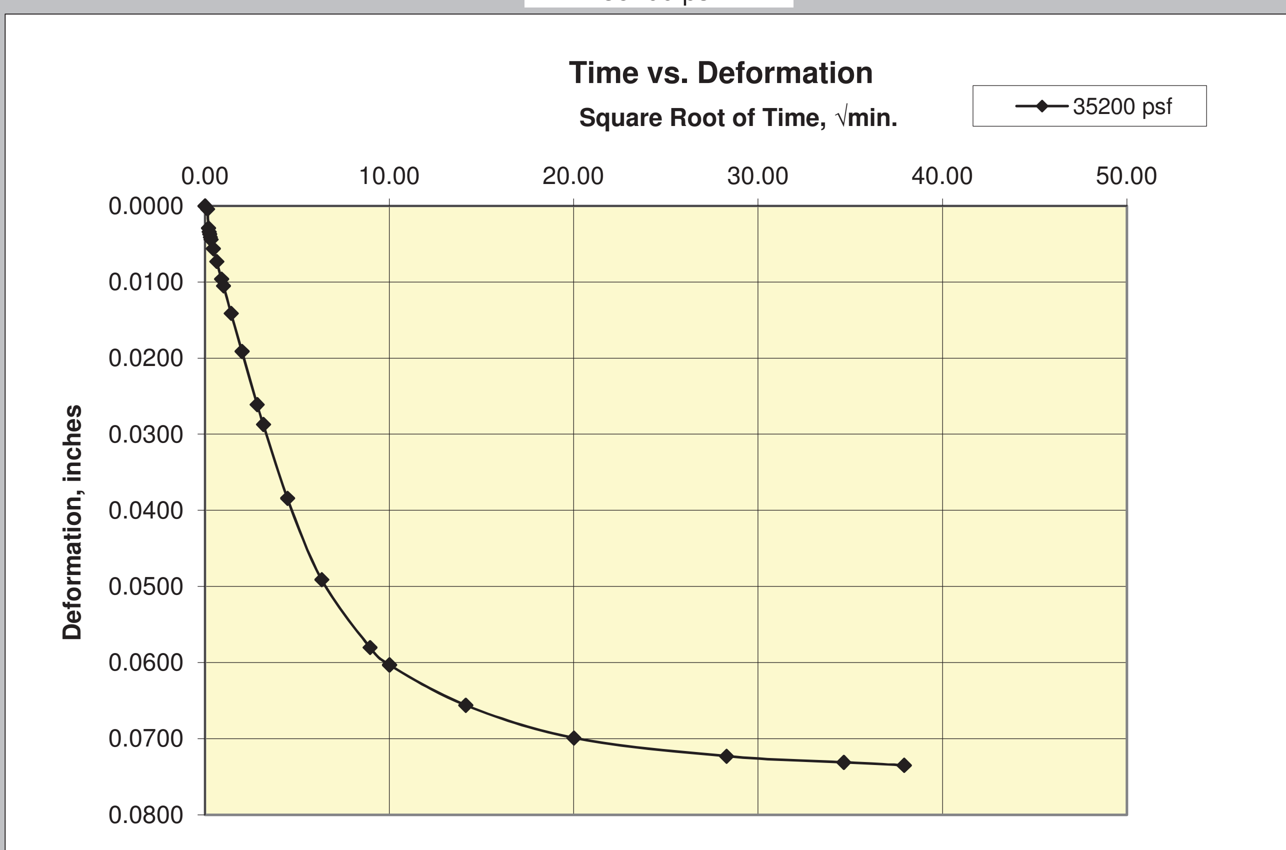
Cooper Testing Labs, Inc.

Load 9

35200 psf



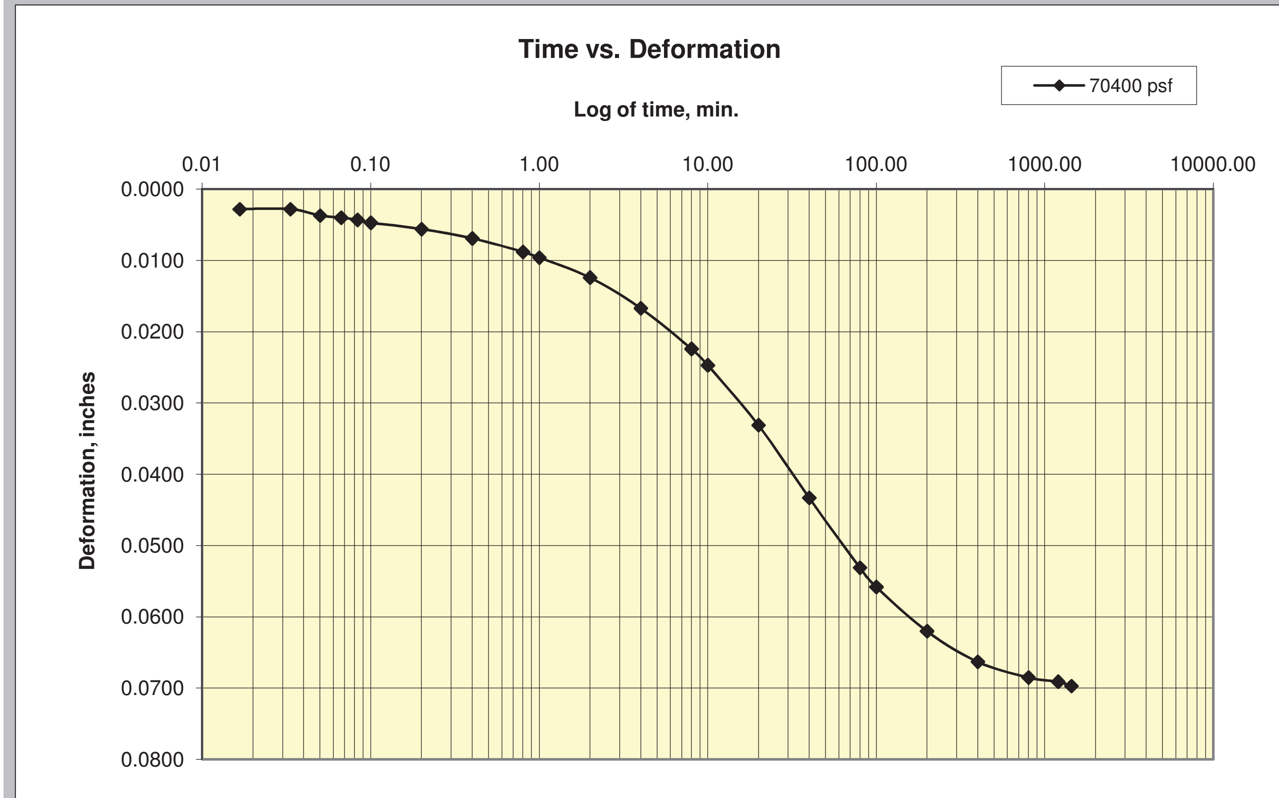
35200 psf



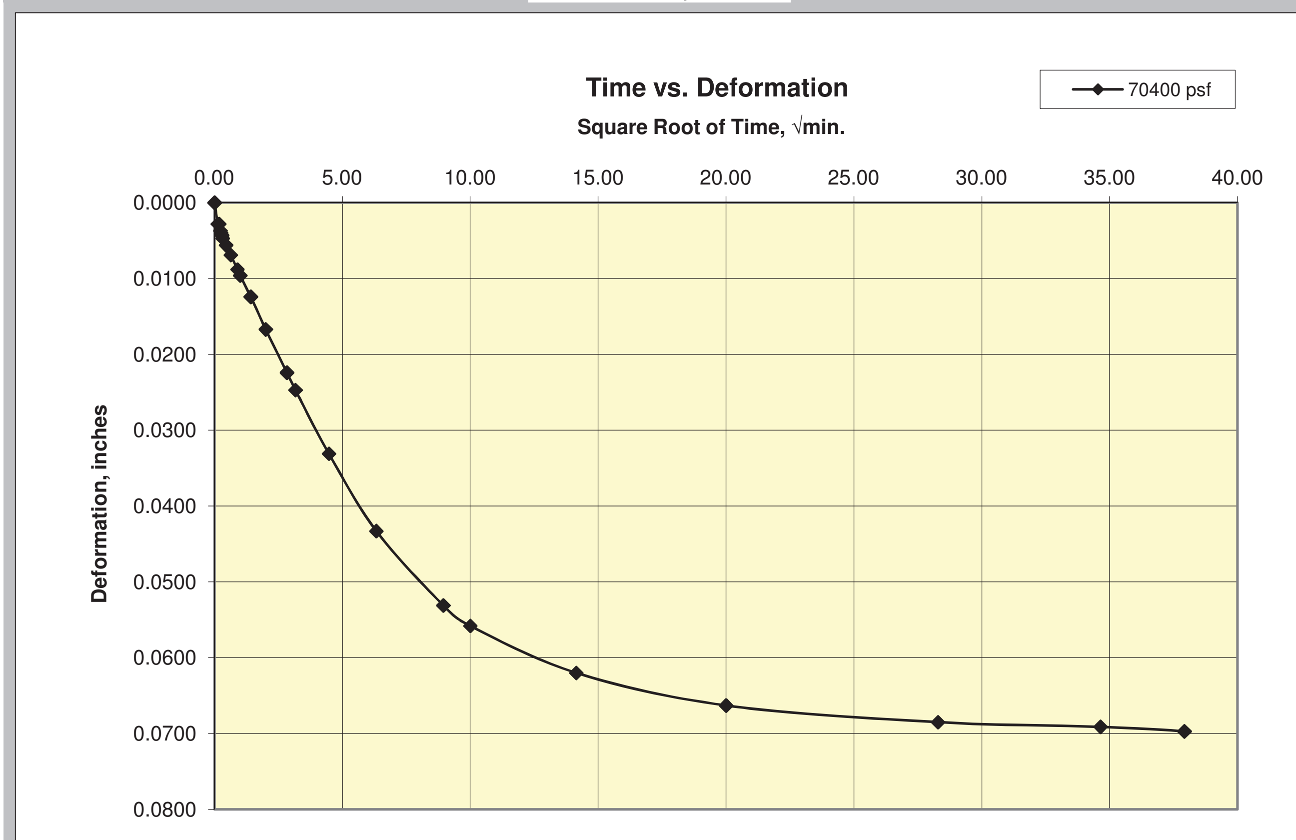
Cooper Testing Labs, Inc.

Load 10

70400 psf

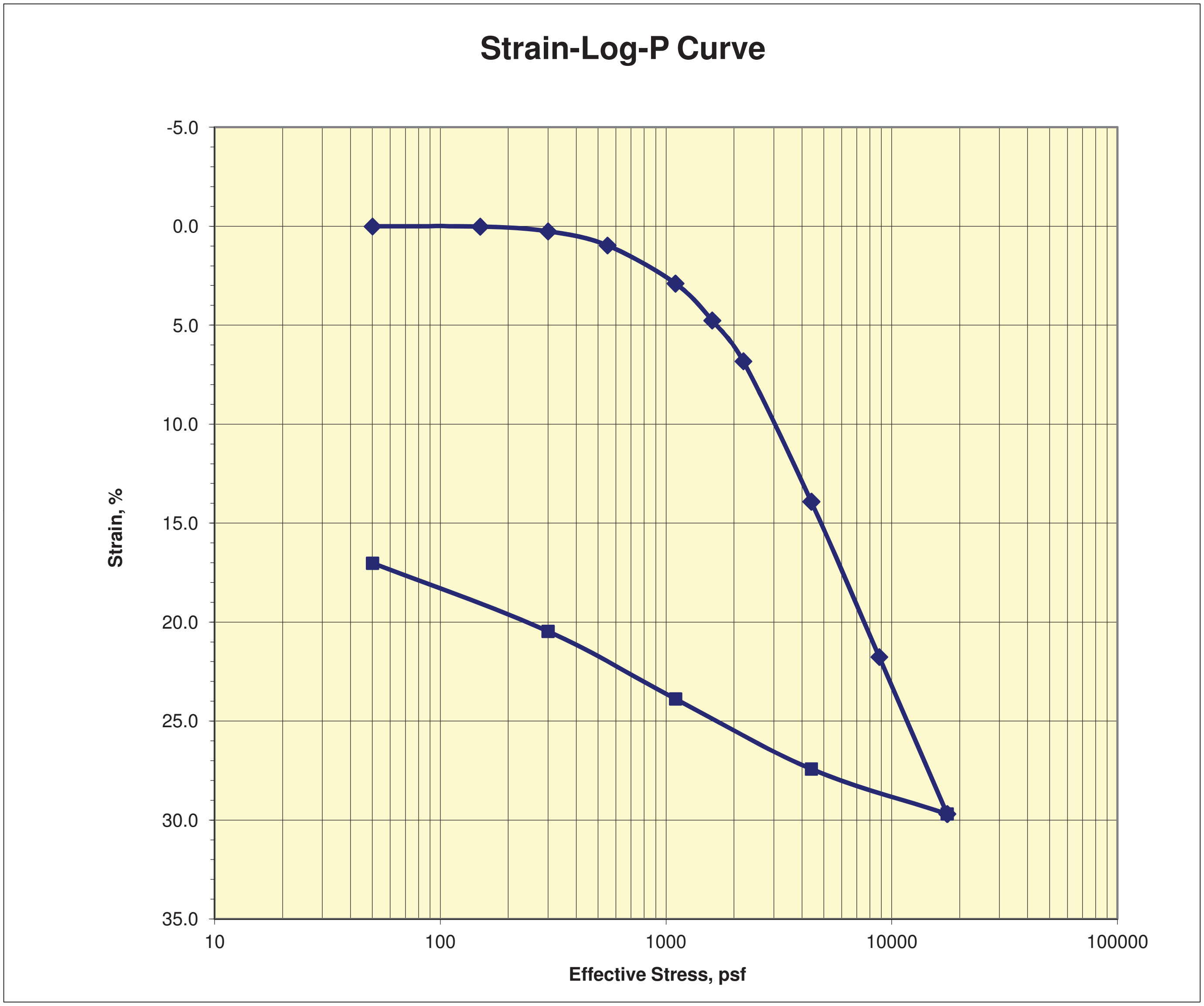


70400 psf



Consolidation Test
ASTM D2435

Job No.:	109-758	**Boring:**	B-2	**Run By:**	MD
Client:	AMEC	**Sample:**	S-3	**Reduced:**	PJ
Project:	Zeus - 616615082	**Depth, ft.:**	12-13(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray CLAY w/ organics			**Date:**	11/18/2015



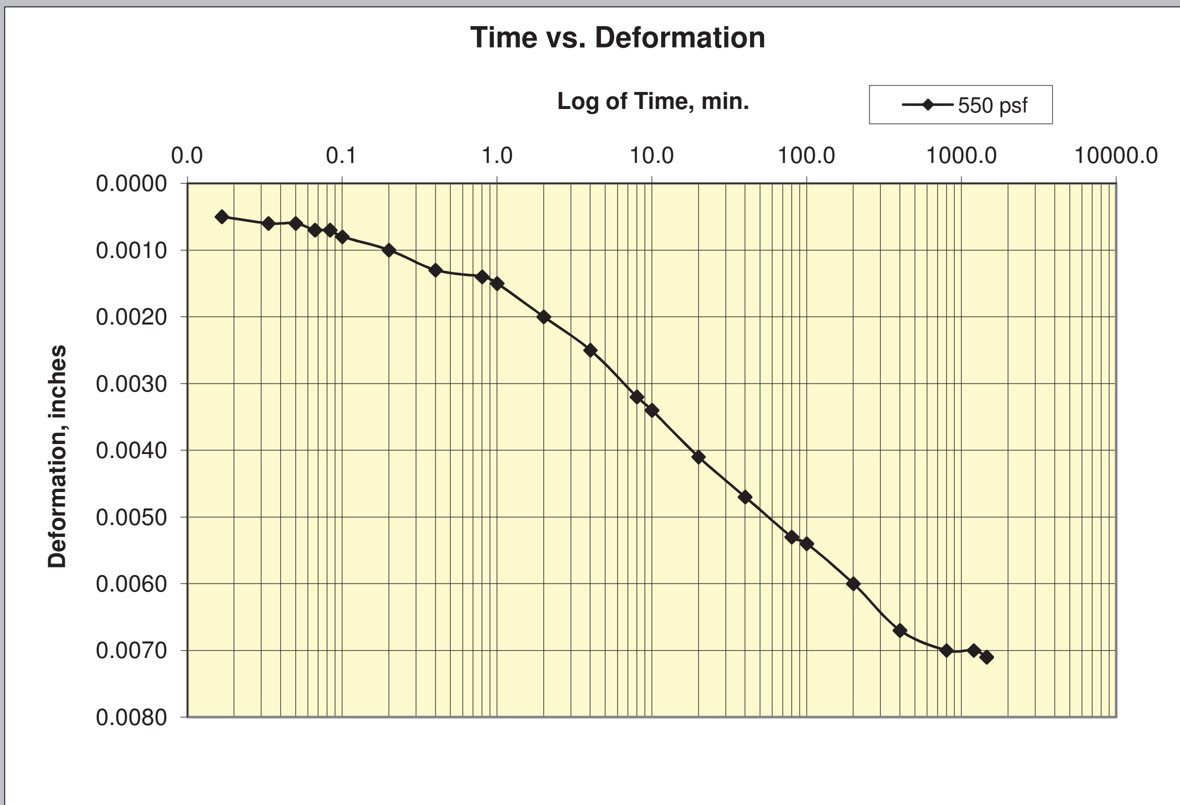
Assumed Gs 2.7	Initial	Final
Moisture %:	80.7	62.8
Dry Density, pcf:	51.9	62.5
Void Ratio:	2.246	1.697
% Saturation:	97.0	100.0

Remarks:

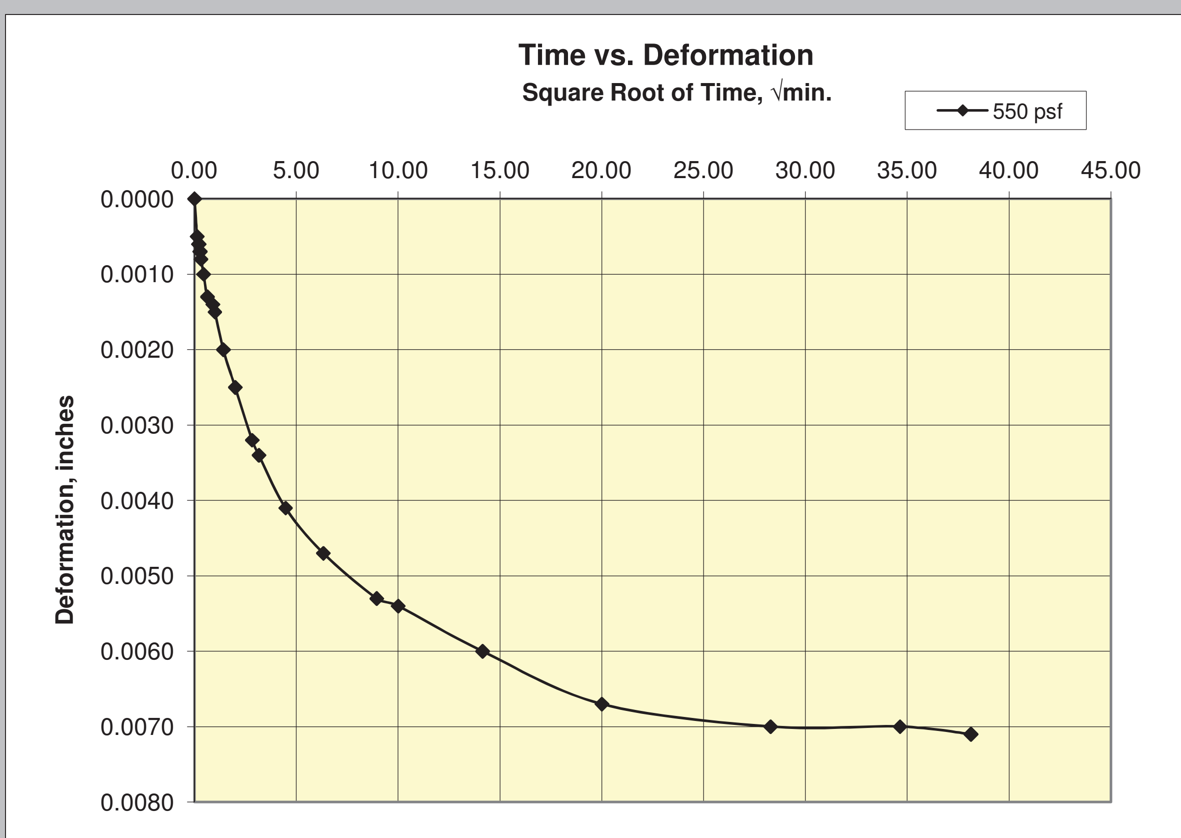
Cooper Testing Labs, Inc.

Load 4

550 psf



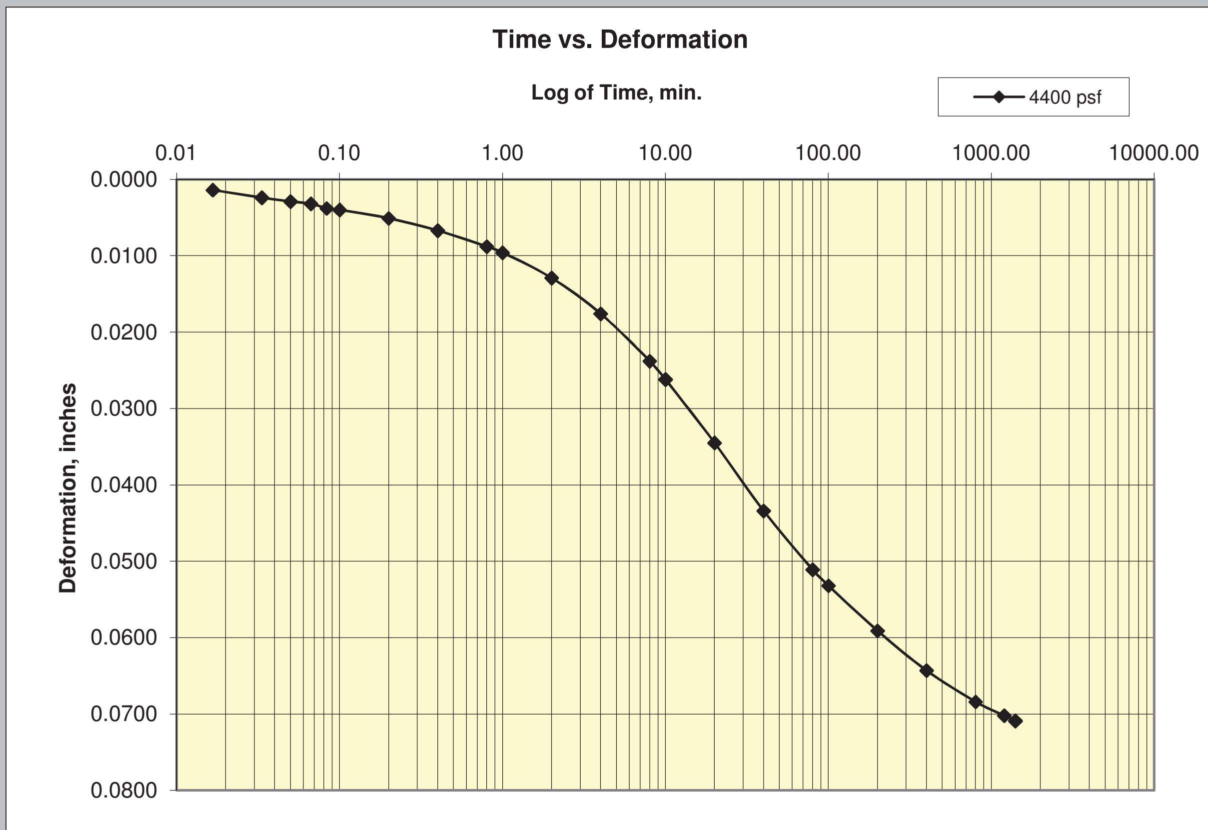
550 psf



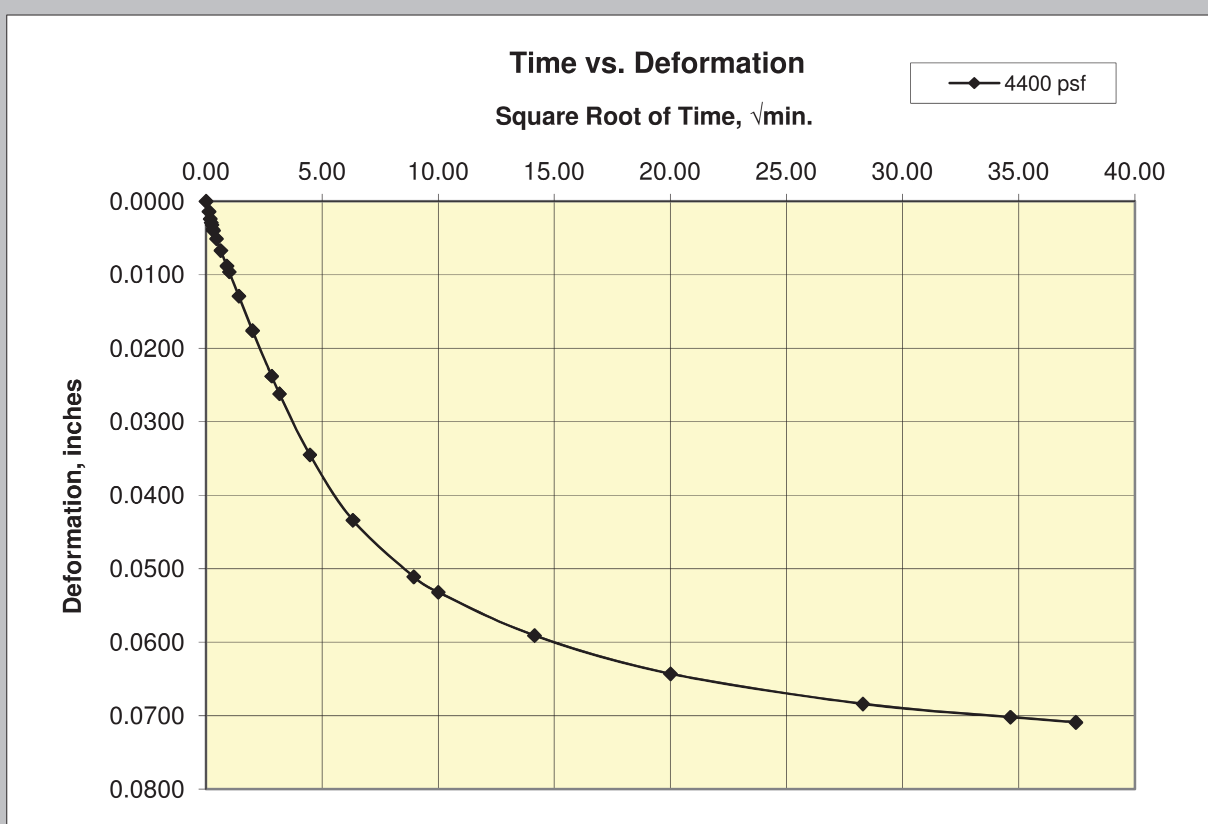
Cooper Testing Labs, Inc.

Load 8

4400 psf

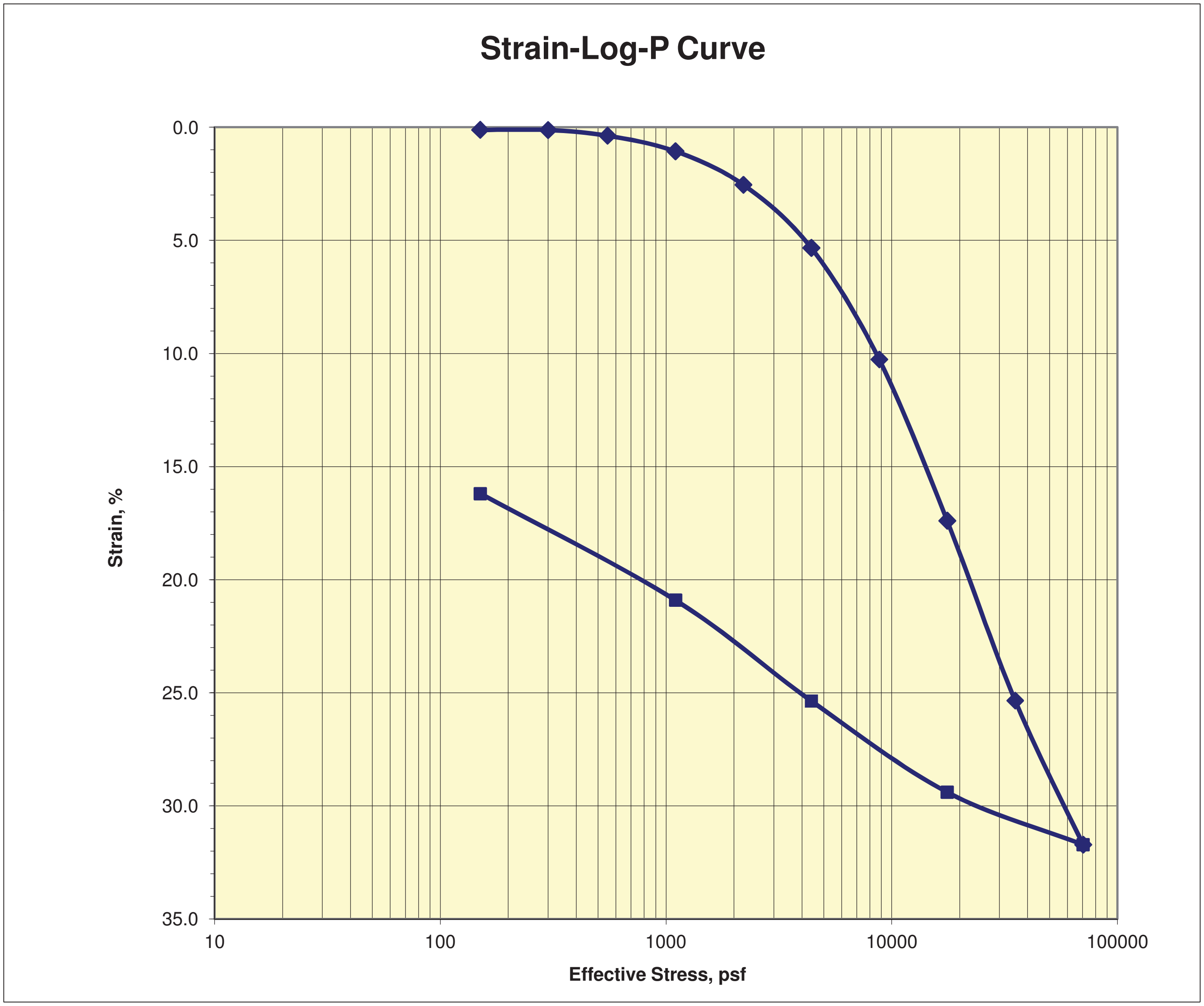


4400 psf



Consolidation Test
ASTM D2435

Job No.:	109-758	**Boring:**	B-3	**Run By:**	MD
Client:	AMEC	**Sample:**	S-3	**Reduced:**	PJ
Project:	Zeus - 6166150082	**Depth, ft.:**	9-10.4(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Dark Gray CLAY			**Date:**	11/19/2015

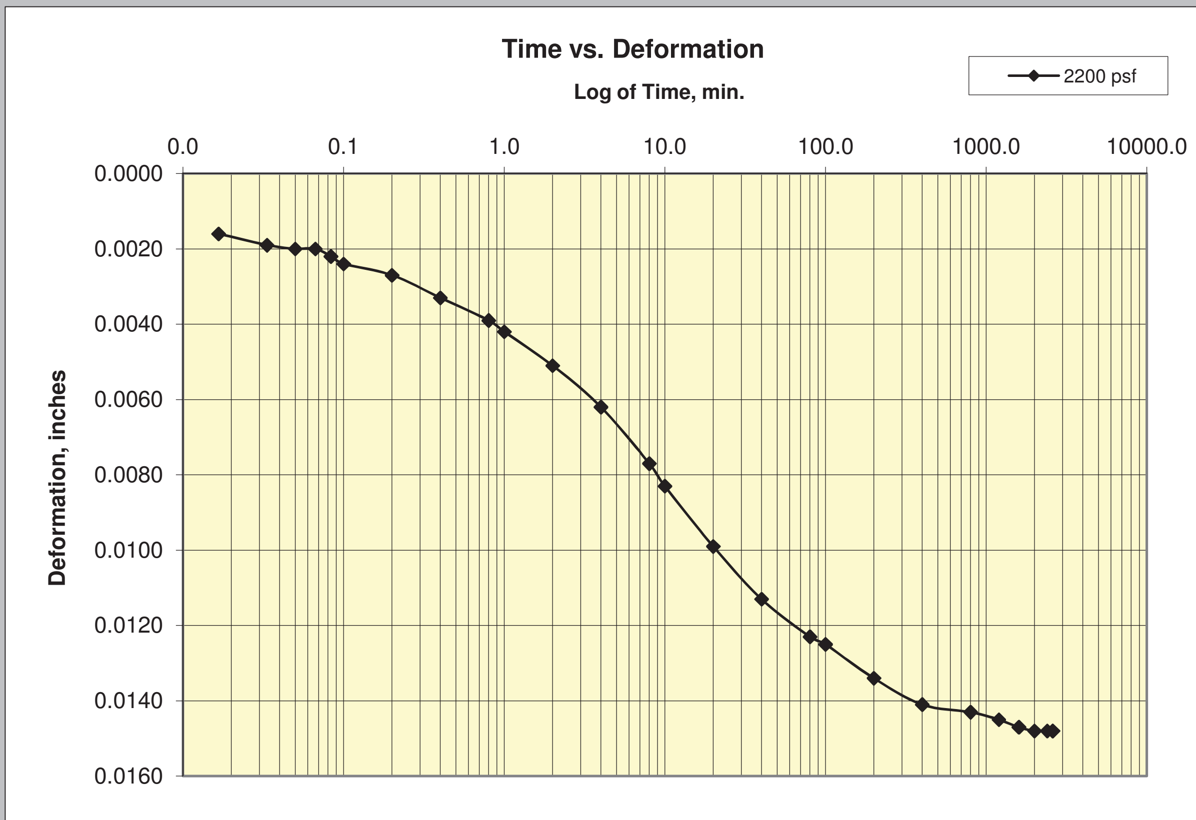


Assumed Gs 2.7	**Initial**	**Final**
Moisture %:	63.1	48.8
Dry Density, pcf:	61.2	72.7
Void Ratio:	1.754	1.319
% Saturation:	97.1	100.0

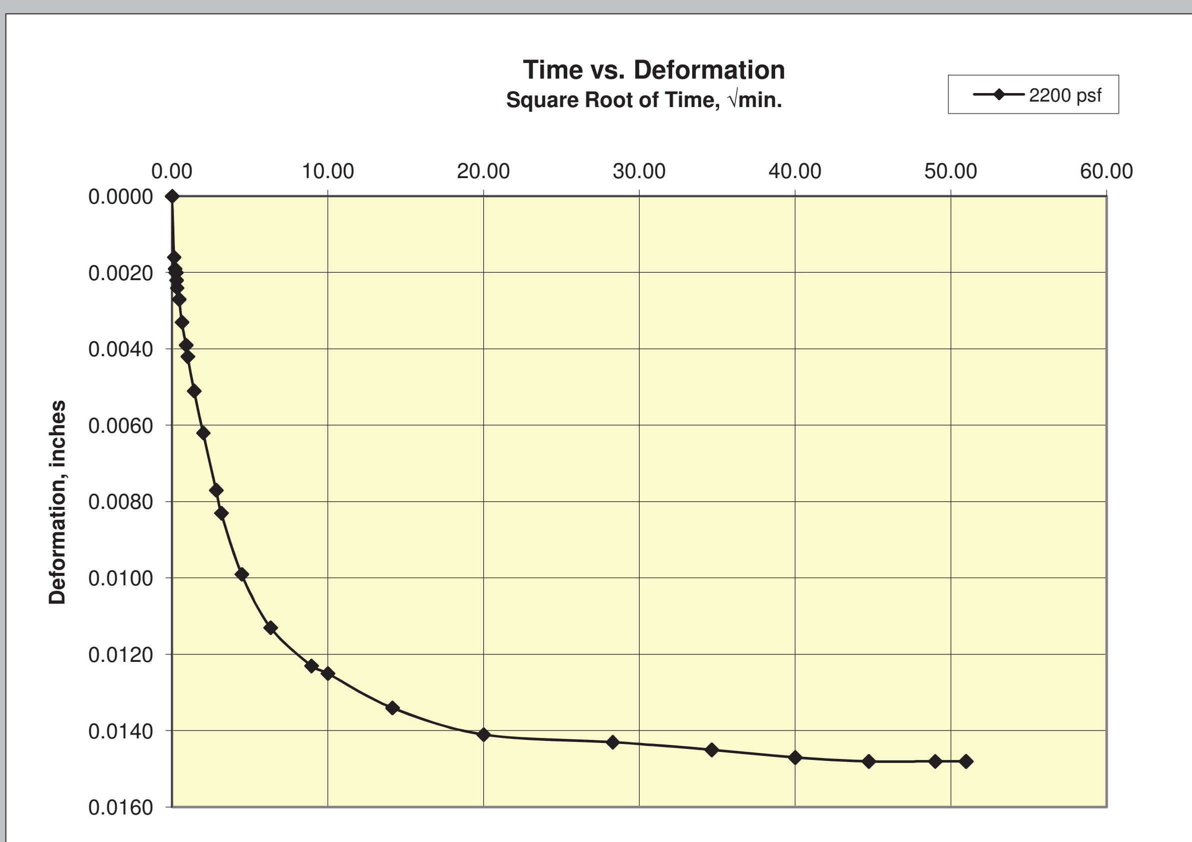
Remarks:

Cooper Testing Labs, Inc.

Load 5 2200 psf



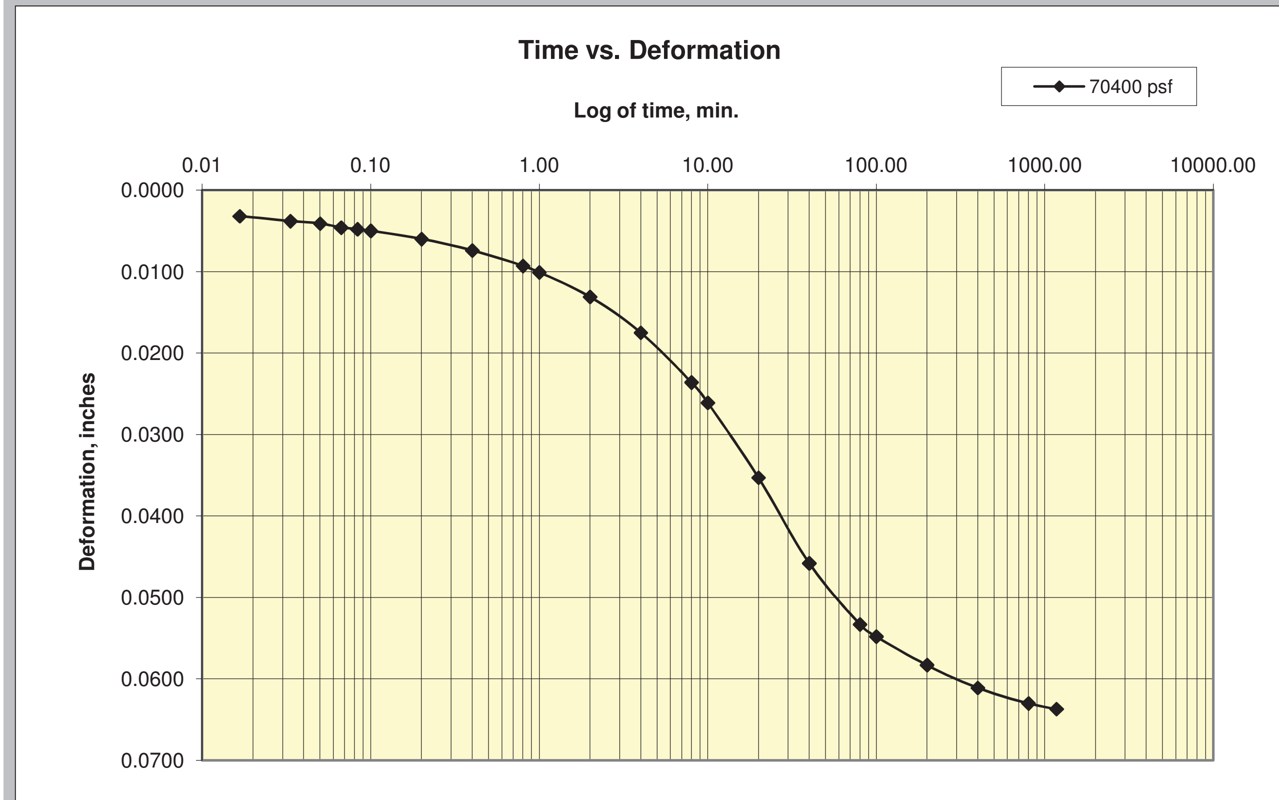
2200 psf



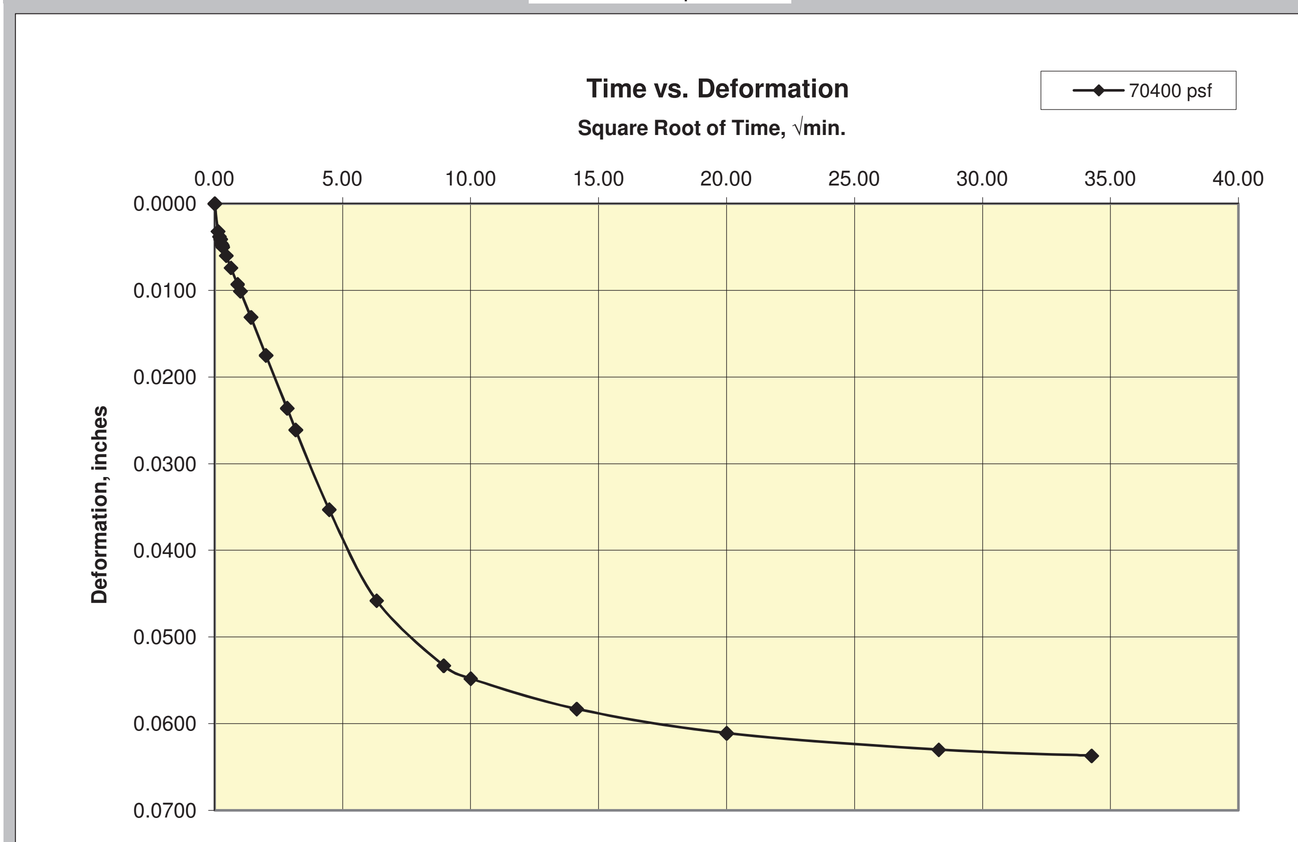
Cooper Testing Labs, Inc.

Load 10

70400 psf

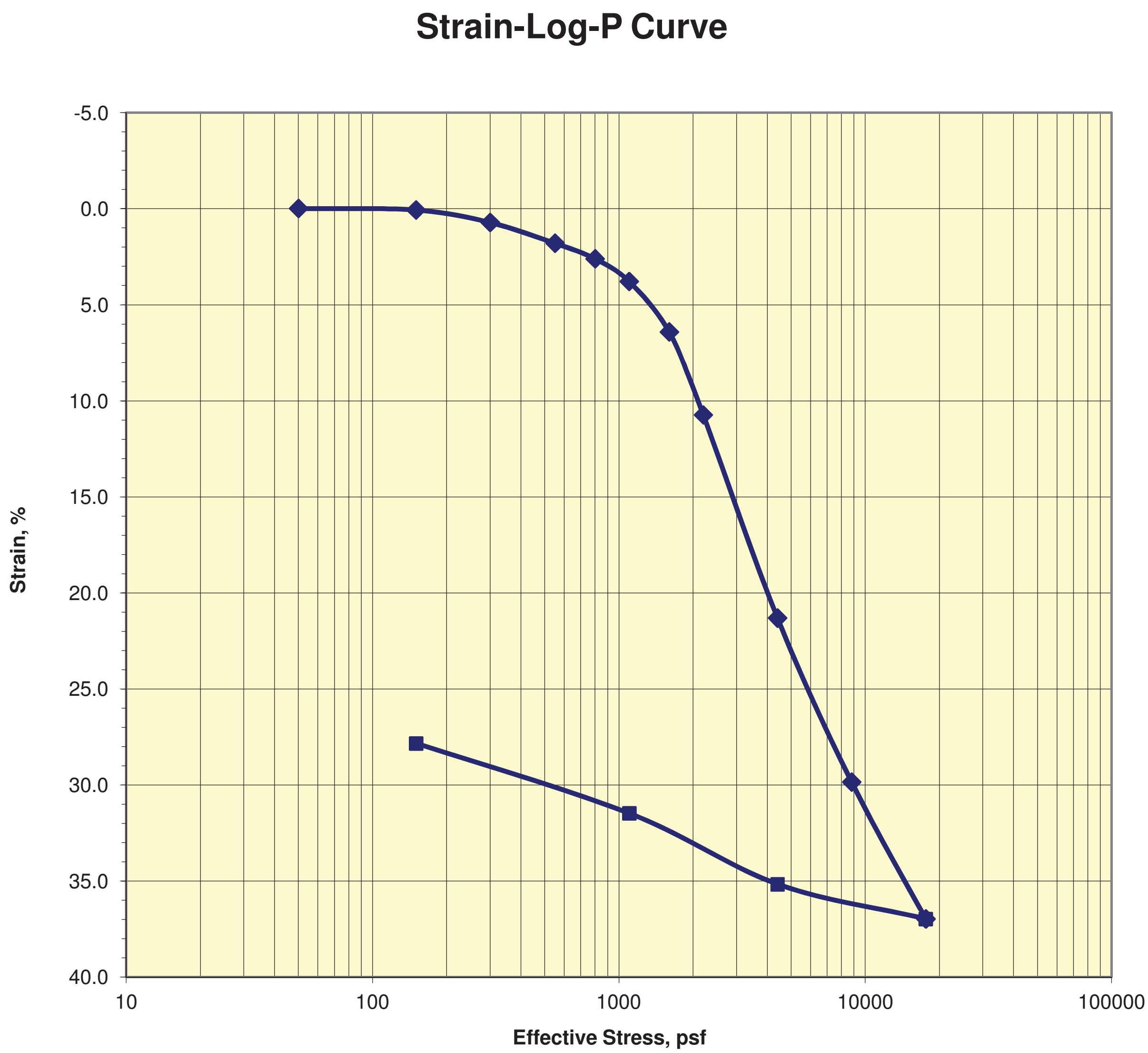


70400 psf



Consolidation Test
ASTM D2435

Job No.:	109-758	**Boring:**	B-3	**Run By:**	MD
Client:	AMEC	**Sample:**	S-5	**Reduced:**	PJ
Project:	Zeus - 6166150082	**Depth, ft.:**	25-25.5(Tip-5")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray Elastic SILT w/ organics & shell fragments (Bay Mud)			**Date:**	11/19/2015



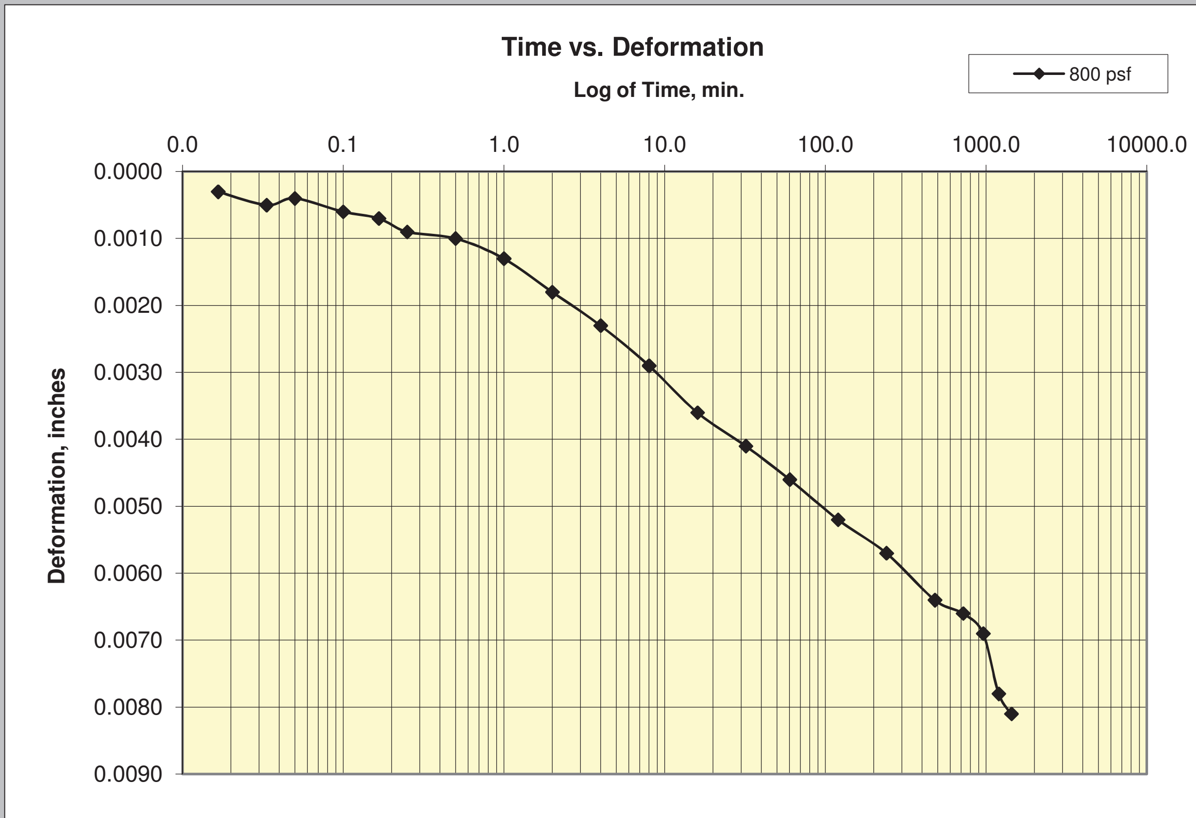
Assumed Gs 2.65	Initial	Final
Moisture %:	84.6	52.9
Dry Density, pcf:	49.6	68.9
Void Ratio:	2.335	1.402
% Saturation:	96.0	100.0

Remarks:

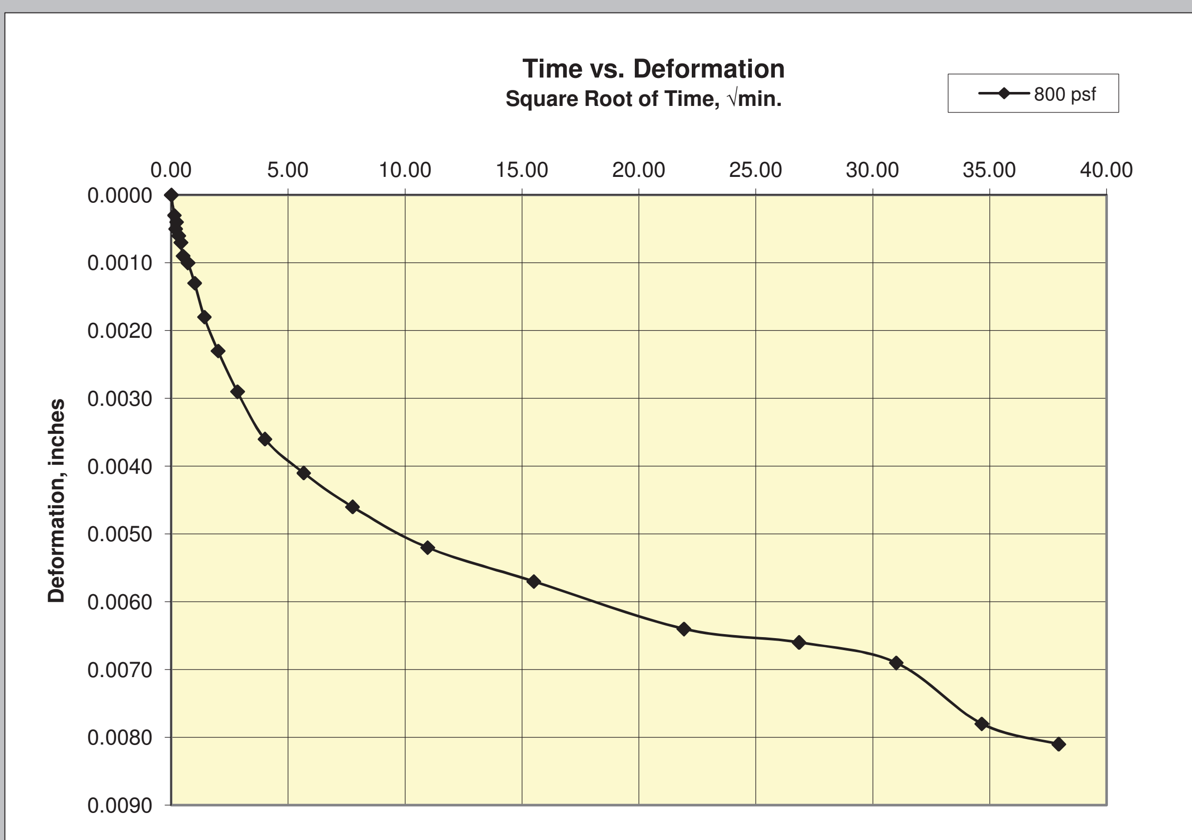
Cooper Testing Labs, Inc.

Load 5

800 psf



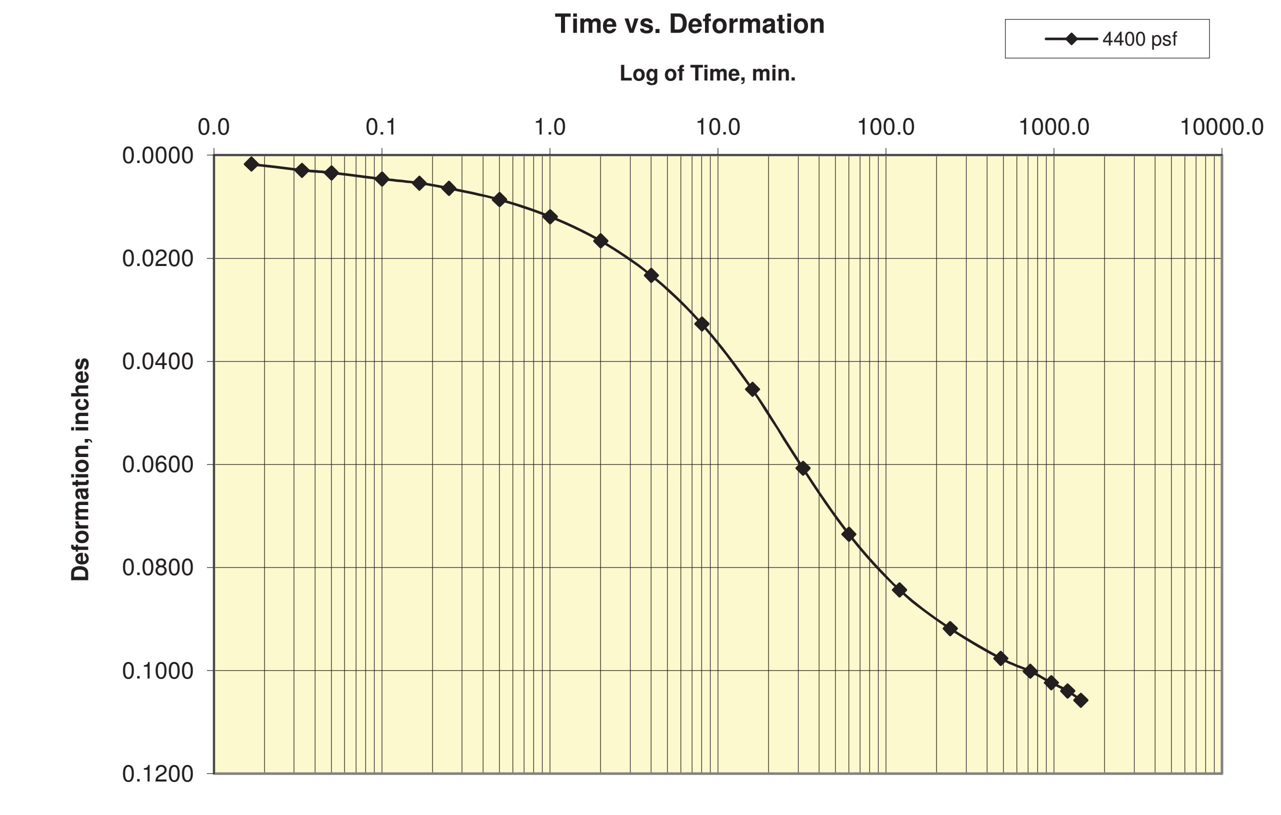
800 psf



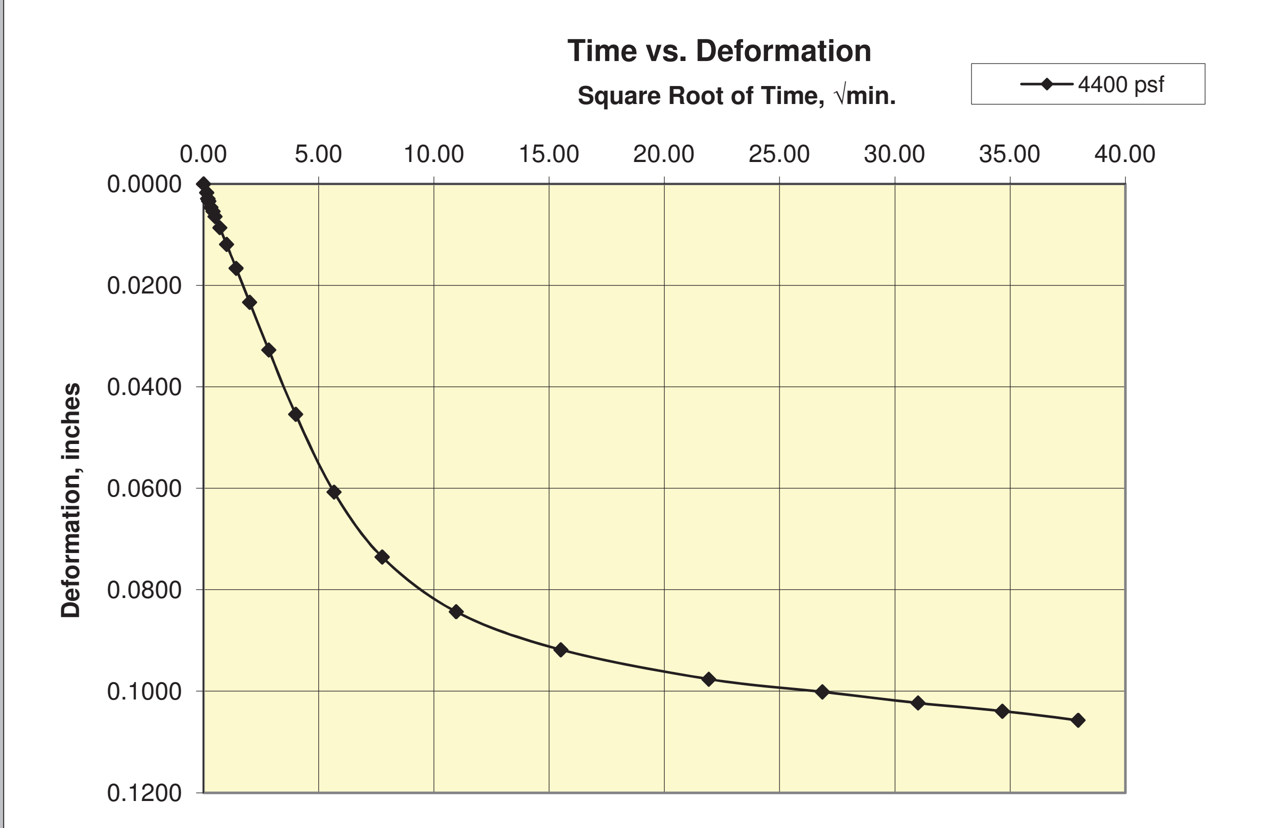
Cooper Testing Labs, Inc.

Load 9

4400 psf



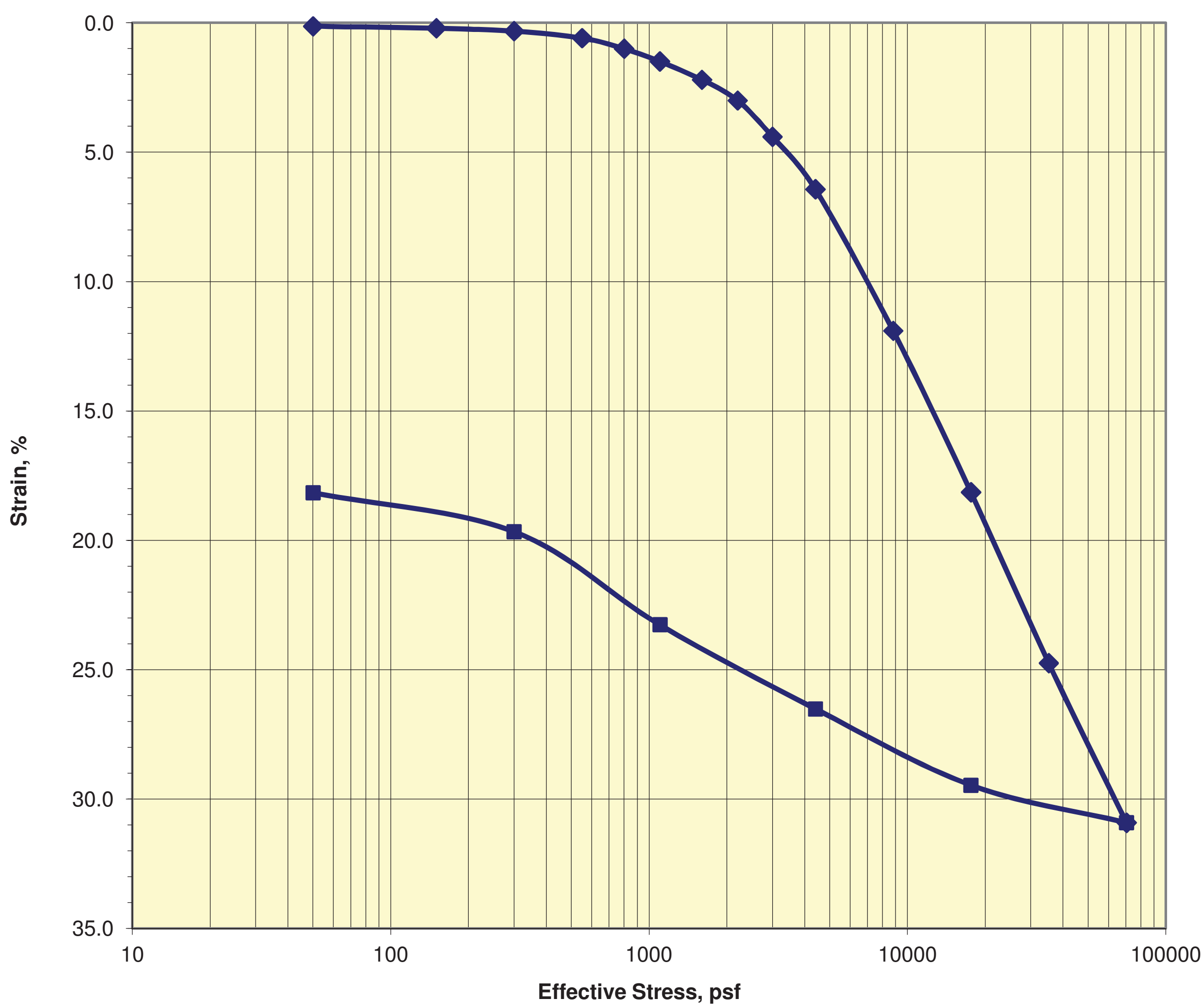
4400 psf



Consolidation Test
ASTM D2435

Job No.:	109-758	**Boring:**	B-4	**Run By:**	MD
Client:	AMEC	**Sample:**	S-4	**Reduced:**	PJ
Project:	Zeus - 6166150082	**Depth, ft.:**	10-12.5(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Dark Greenish Gray Fat CLAY (Bay Mud)			**Date:**	11/25/2015

Strain-Log-P Curve



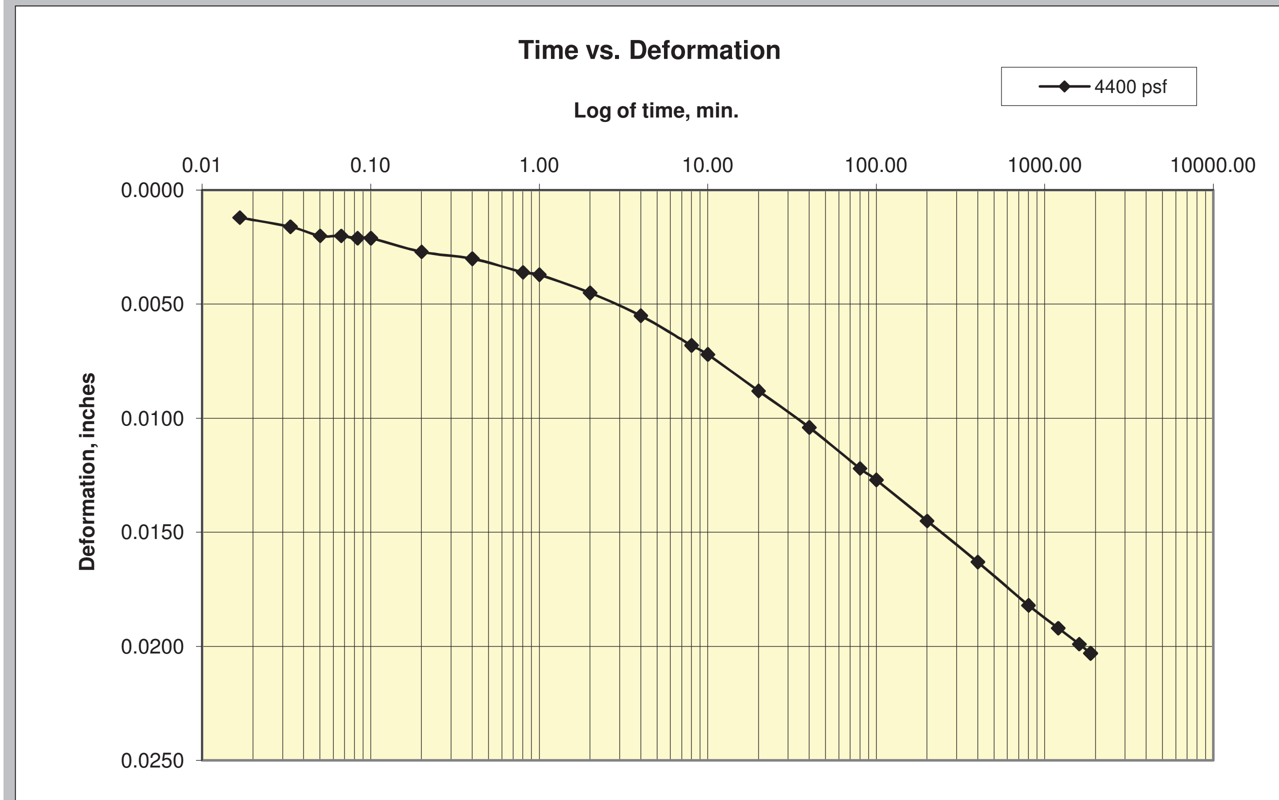
Assumed Gs 2.75	**Initial**	**Final**
Moisture %:	53.7	39.3
Dry Density, pcf:	67.7	82.5
Void Ratio:	1.537	1.080
% Saturation:	96.1	100.0

Remarks:

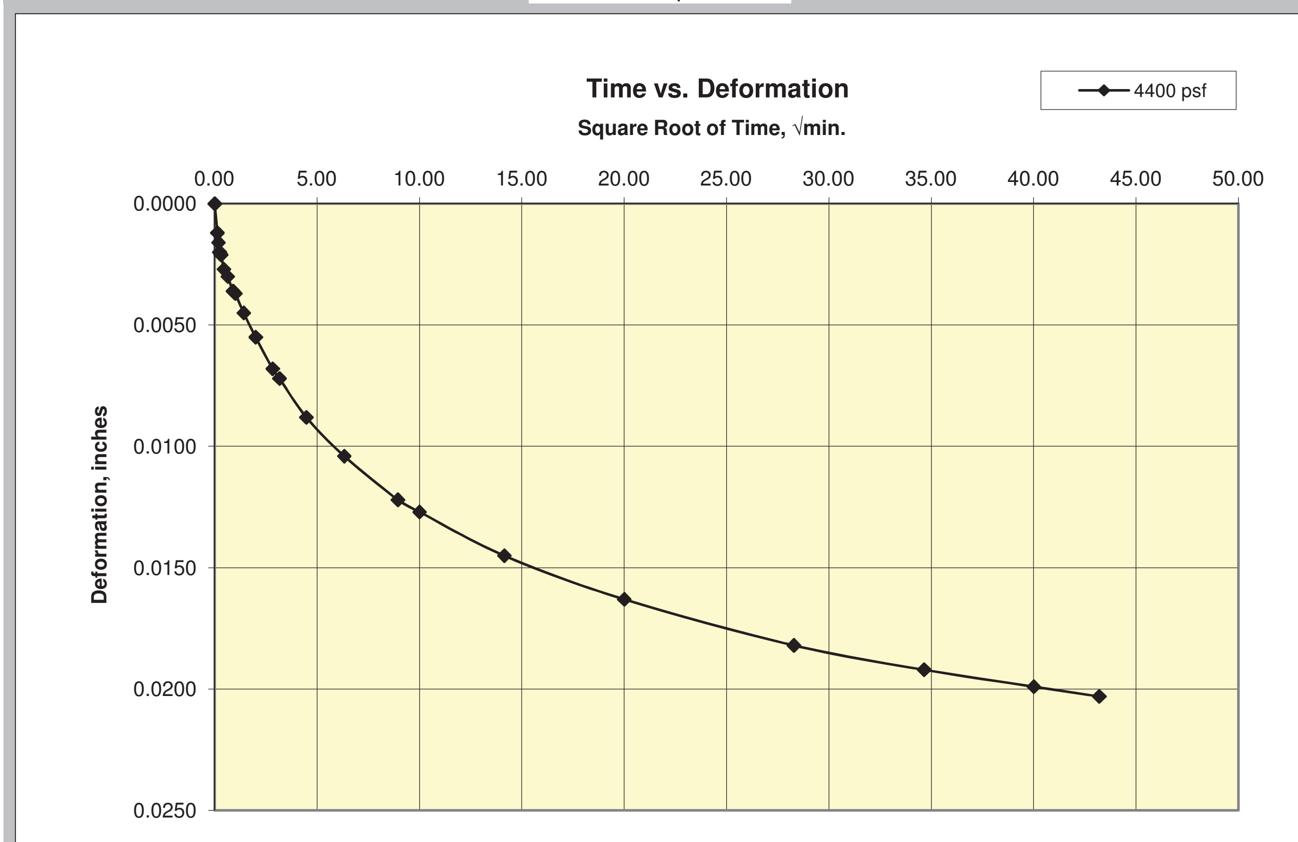
Cooper Testing Labs, Inc.

Load 10

4400 psf



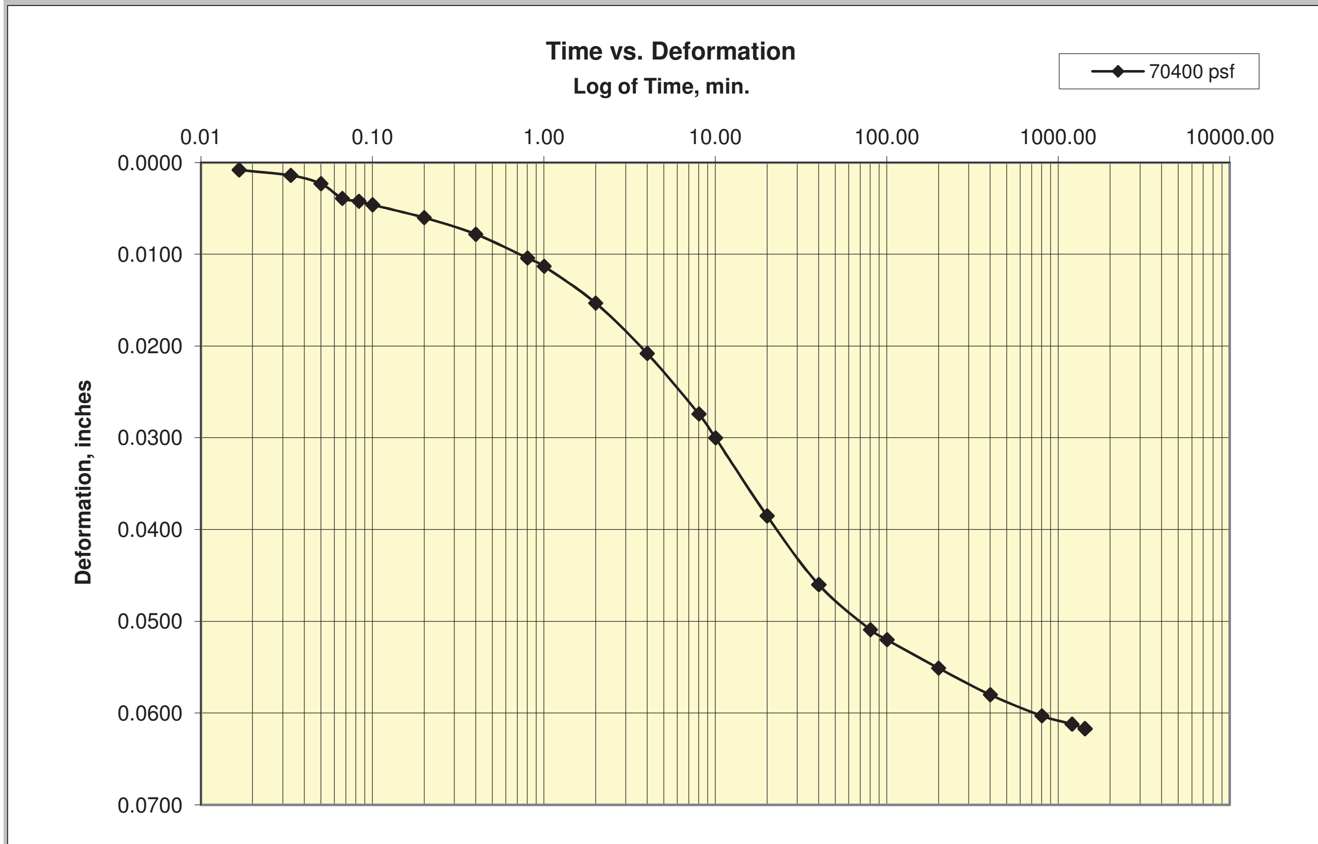
4400 psf



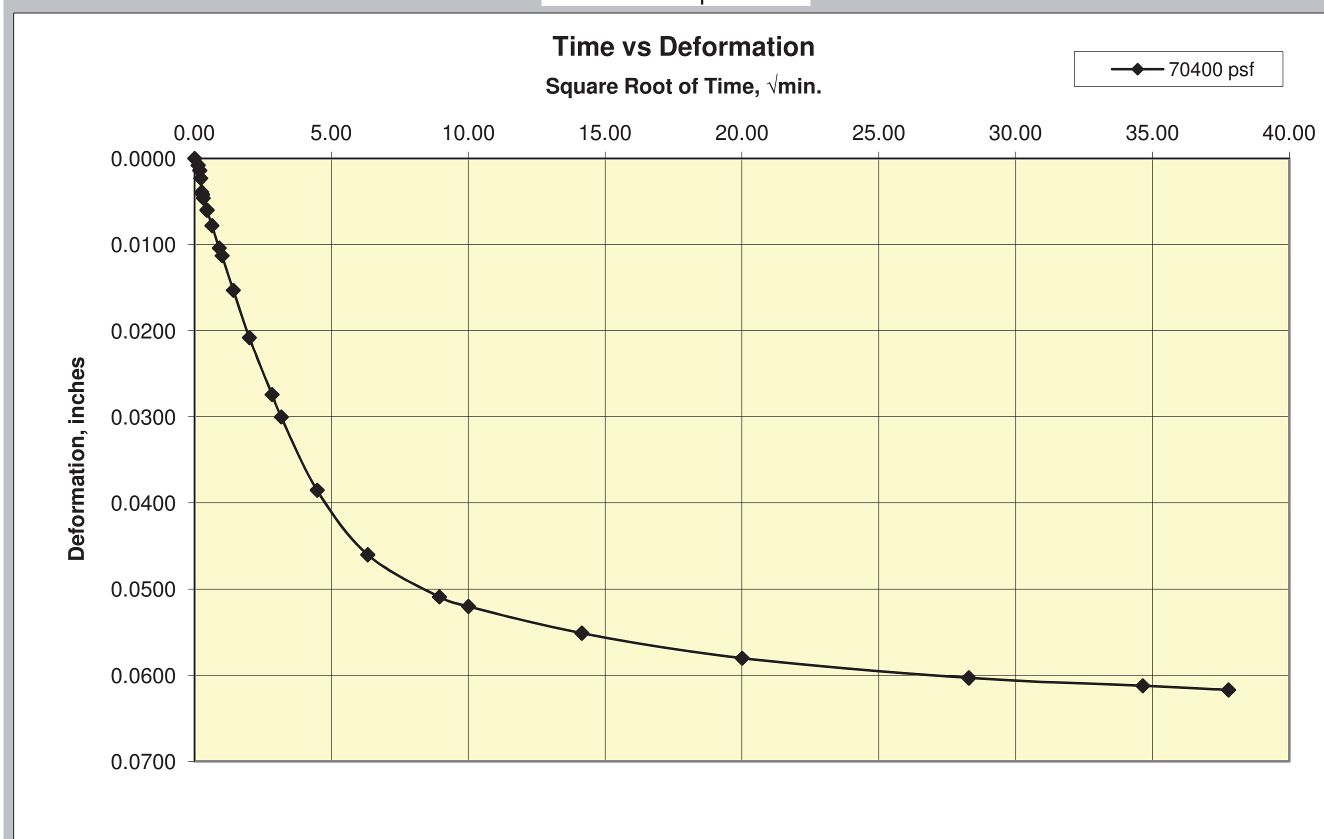
Cooper Testing Labs, Inc.

Load 14

70400 psf

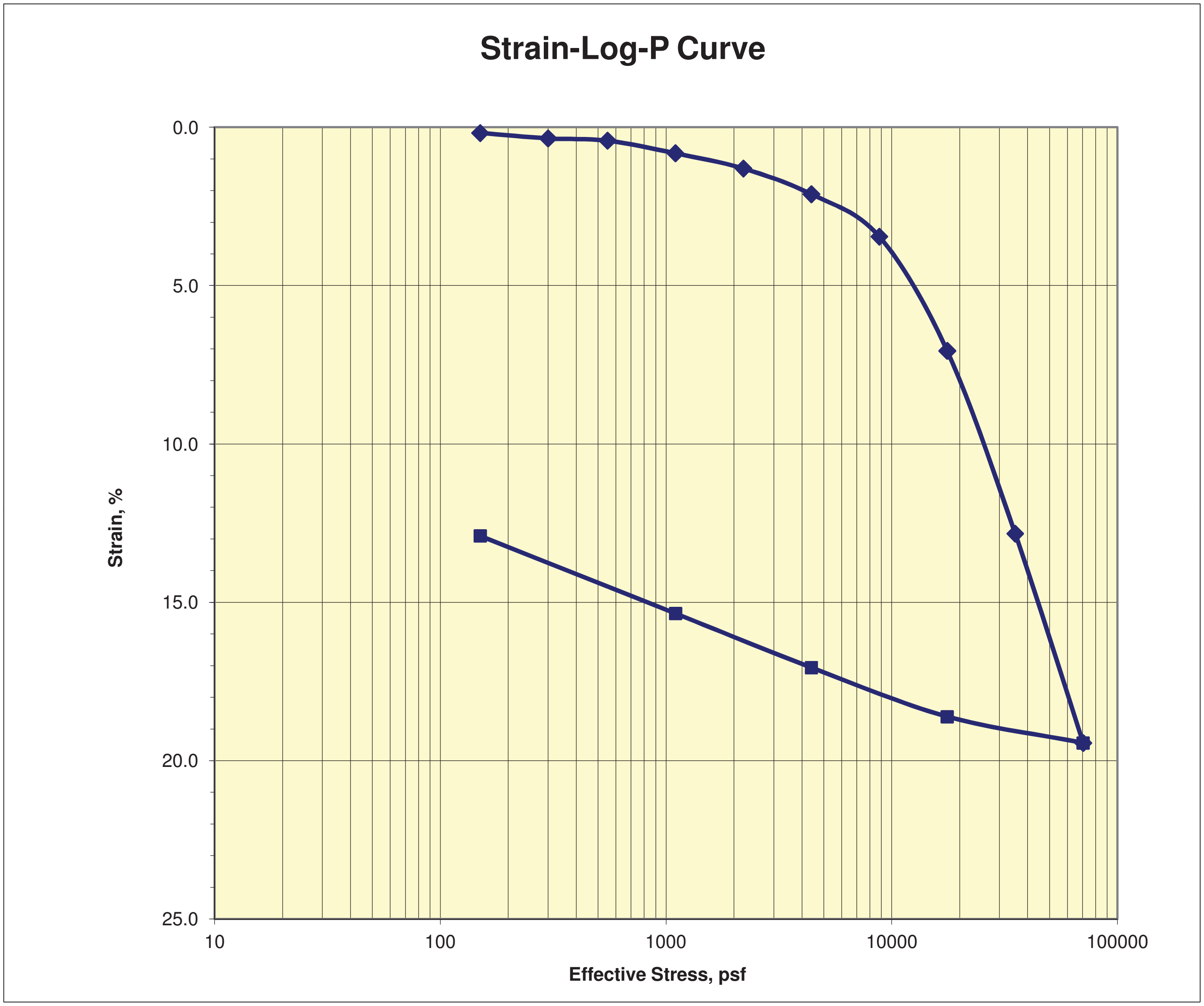


70400 psf



Consolidation Test
ASTM D2435

Job No.:	109-758	**Boring:**	B-4	**Run By:**	MD
Client:	AMEC	**Sample:**	S-7	**Reduced:**	PJ
Project:	Zeus - 6166150082	**Depth, ft.:**	48-49.8(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray SILT w/ Sand (slightly plastic)			**Date:**	11/20/2015



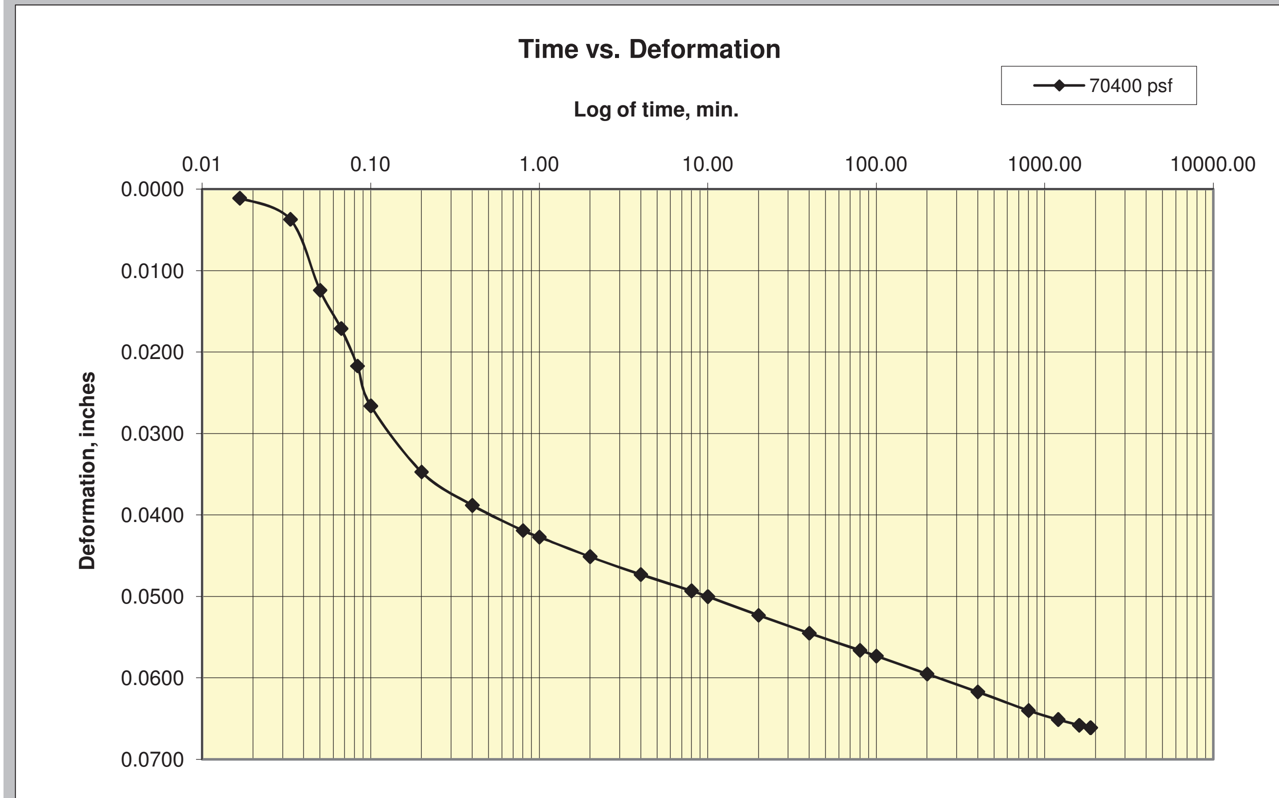
Assumed Gs 2.75	**Initial**	**Final**
Moisture %:	34.4	28.3
Dry Density, pcf:	84.7	96.6
Void Ratio:	1.026	0.777
% Saturation:	92.2	100.0

Remarks:

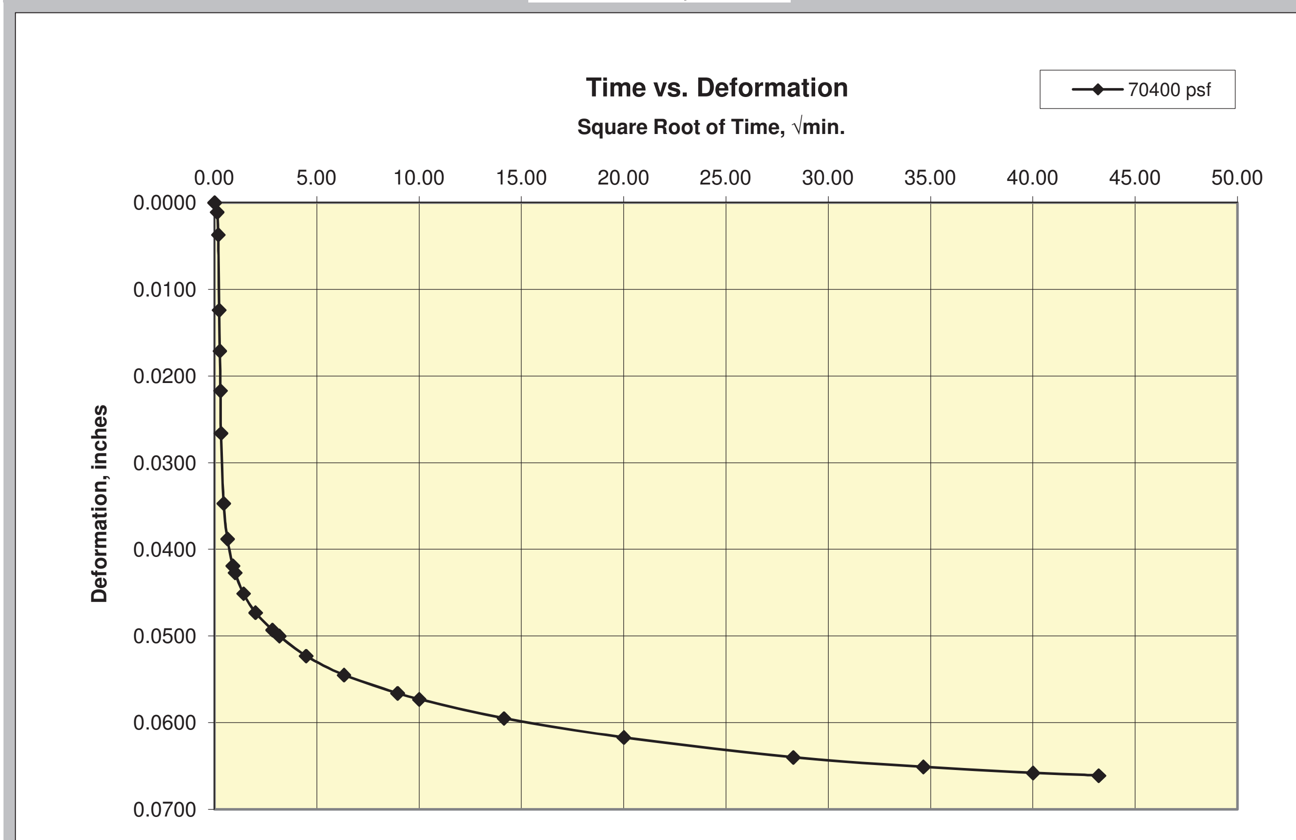
Cooper Testing Labs, Inc.

Load 10

70400 psf

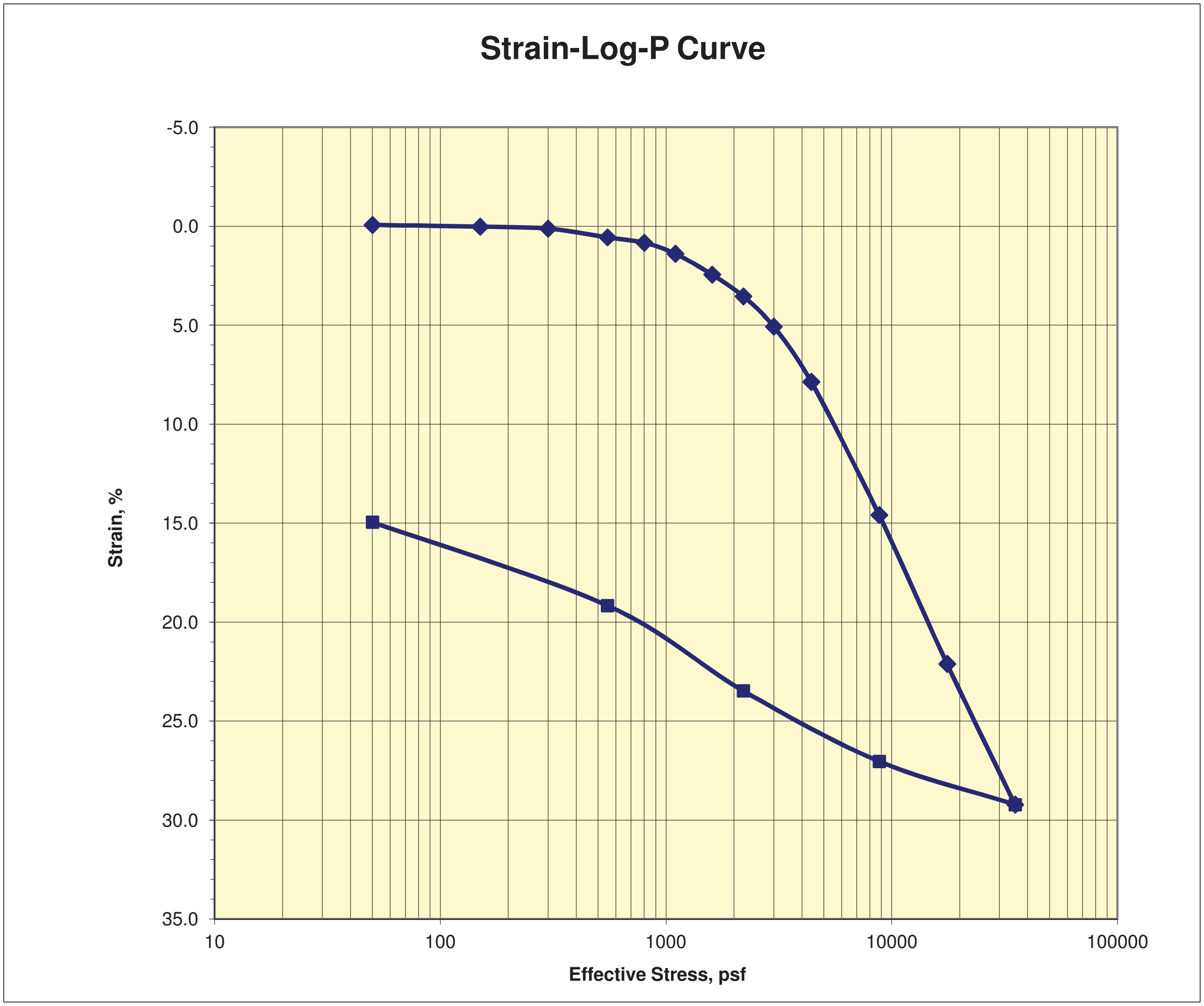


70400 psf



Consolidation Test
ASTM D2435

Job No.:	109-758	**Boring:**	B-5	**Run By:**	MD
Client:	AMEC	**Sample:**	S-2	**Reduced:**	PJ
Project:	Zeus - 6166150082	**Depth, ft.:**	10-12.5(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray CLAY (Bay Mud)			**Date:**	11/23/2015



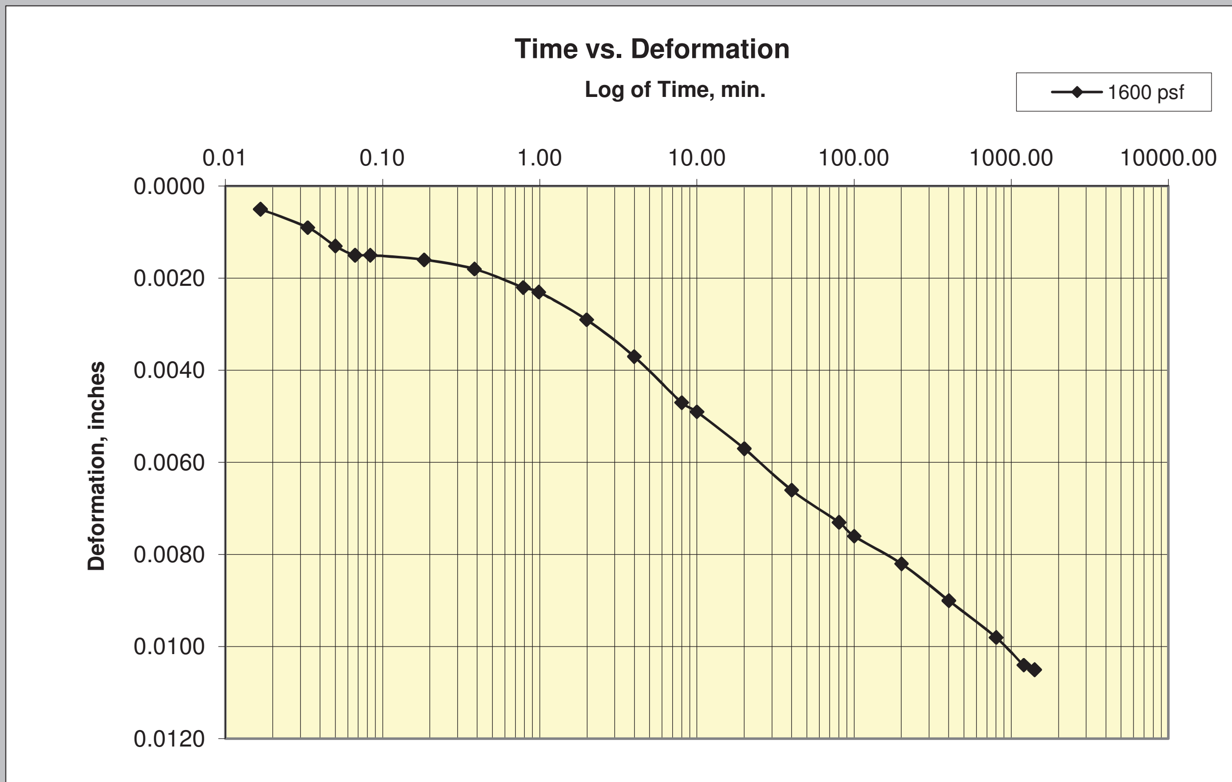
Assumed Gs 2.7	**Initial**	**Final**
Moisture %:	64.5	50.5
Dry Density, pcf:	60.4	71.3
Void Ratio:	1.792	1.363
% Saturation:	97.2	100.0

Remarks:

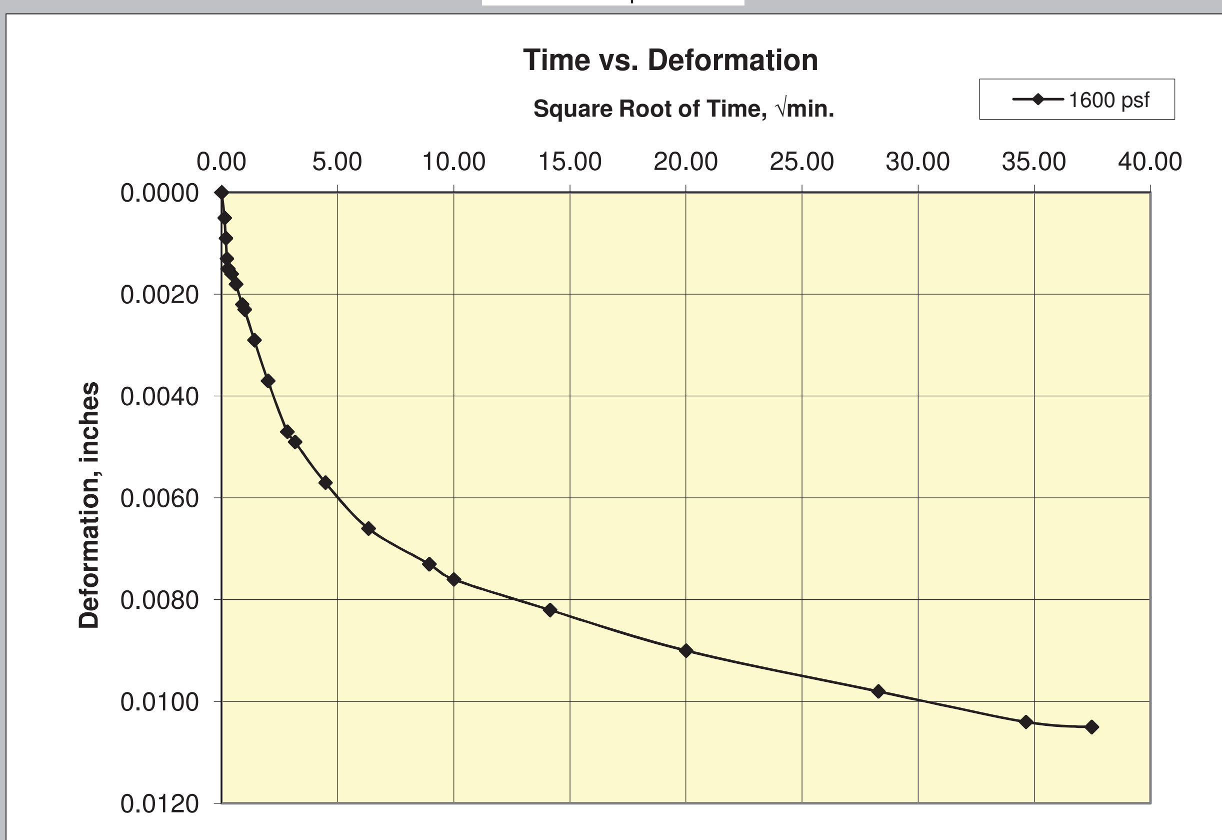
Cooper Testing Labs, Inc.

Load 7

1600 psf

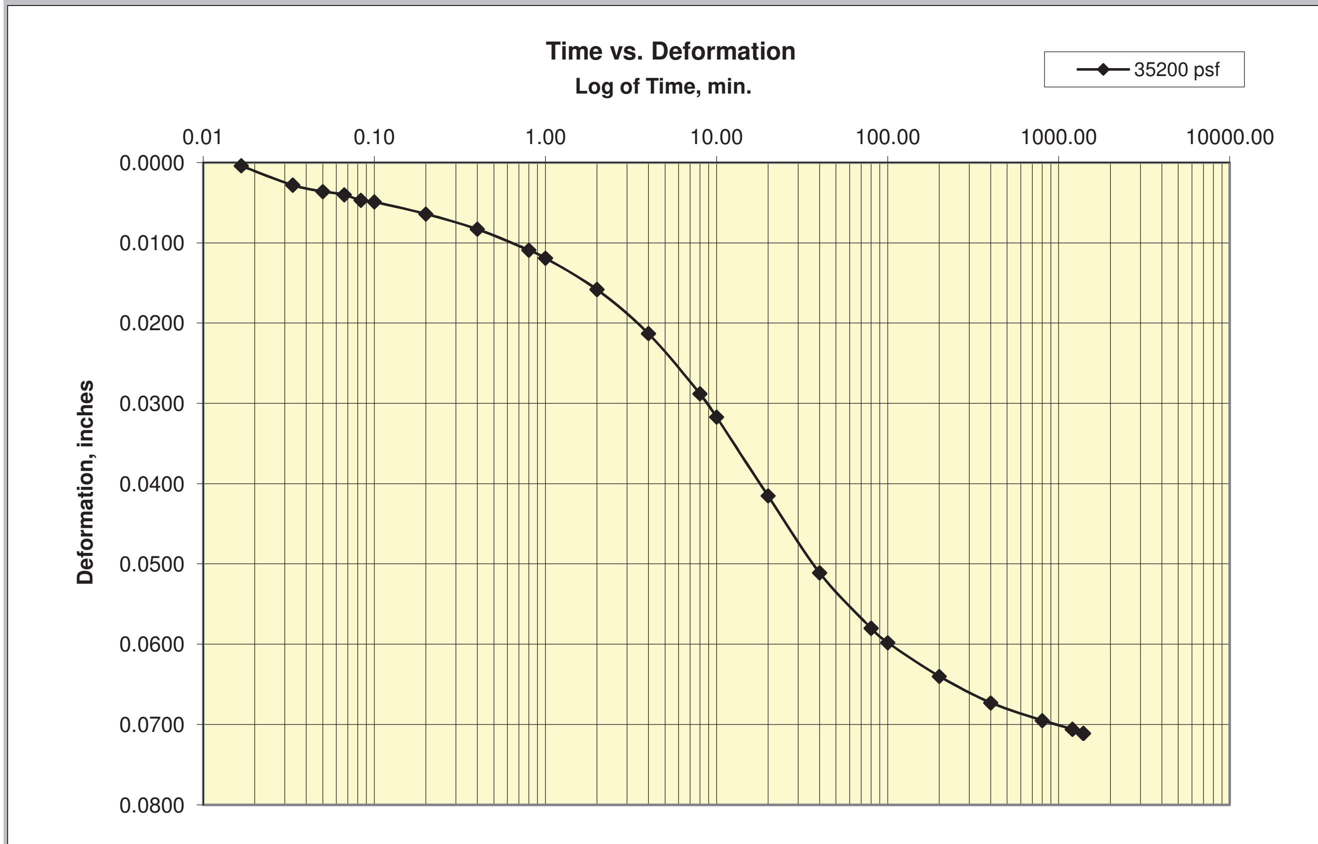


1600 psf

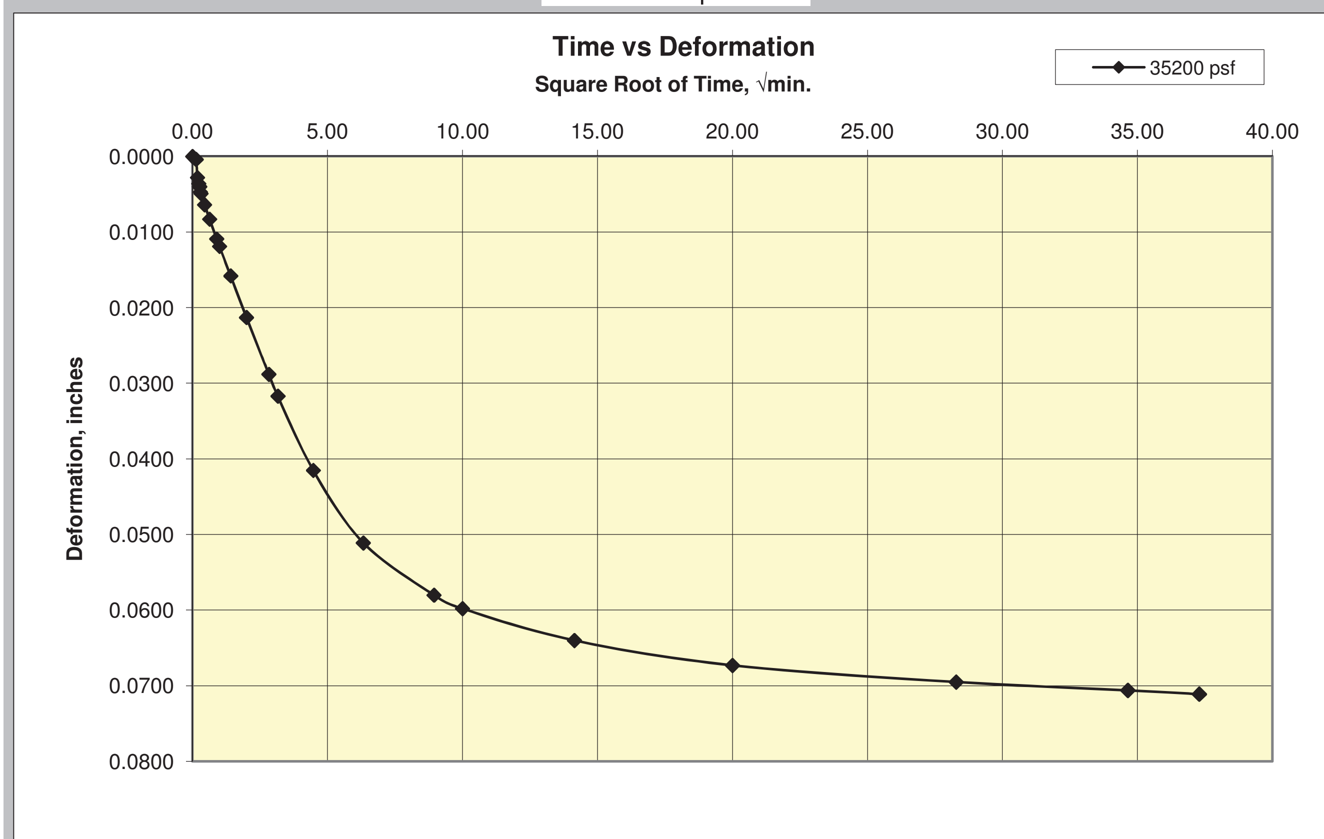


Cooper Testing Labs, Inc.

Load 13 | 35200 psf

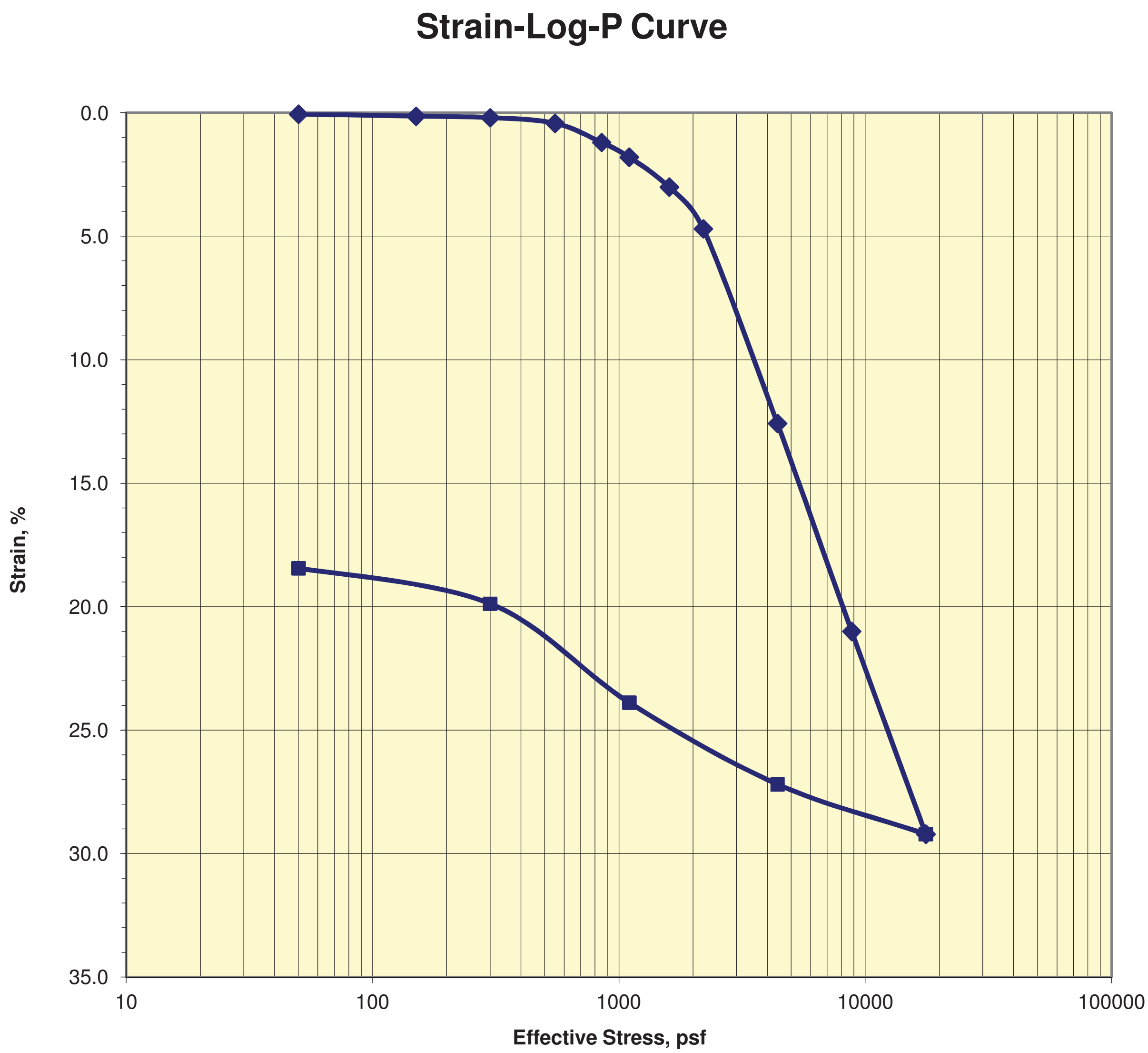


35200 psf



Consolidation Test
ASTM D2435

Job No.:	109-758	**Boring:**	B-5	**Run By:**	MD
Client:	AMEC	**Sample:**	S-3	**Reduced:**	PJ
Project:	Zeus - 6166150082	**Depth, ft.:**	15-17.5(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray CLAY (Bay Mud)			**Date:**	11/23/2015

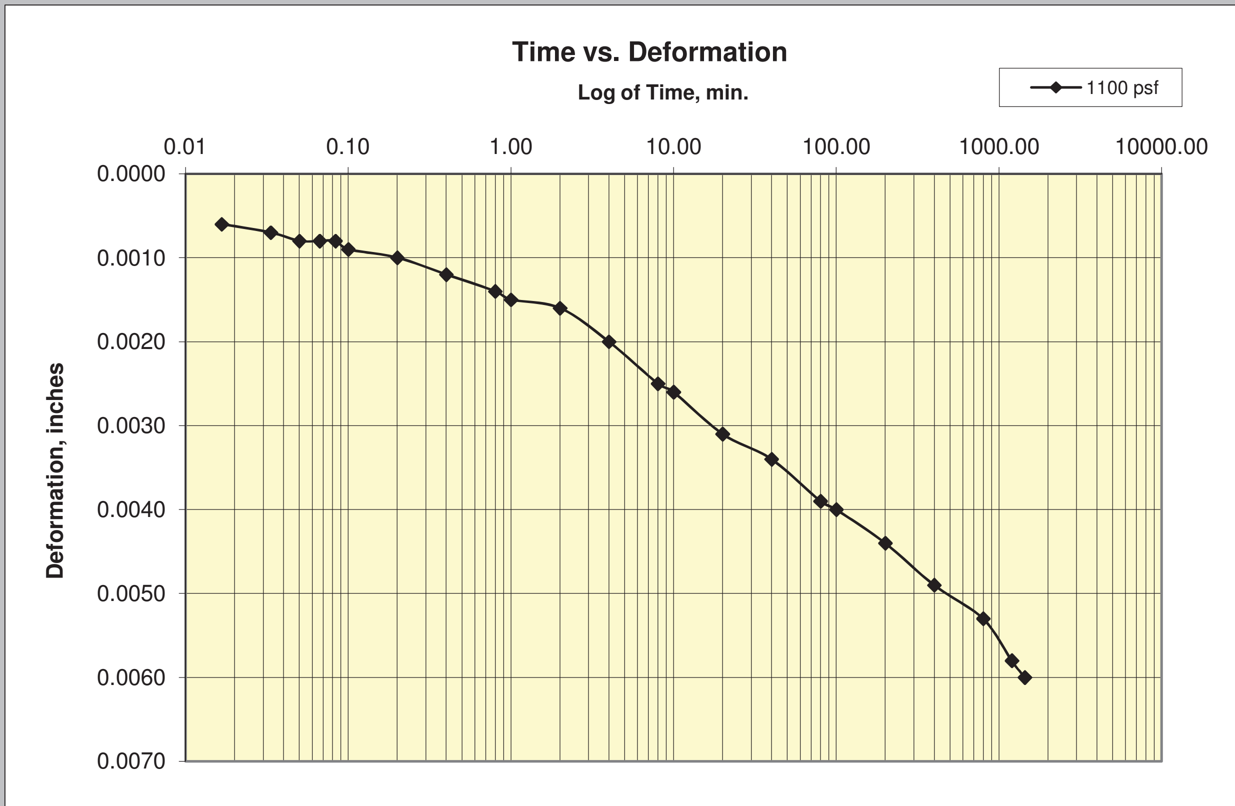


Assumed Gs 2.75	**Initial**	**Final**
Moisture %:	72.8	54.2
Dry Density, pcf:	56.2	68.9
Void Ratio:	2.053	1.490
% Saturation:	97.5	100.0

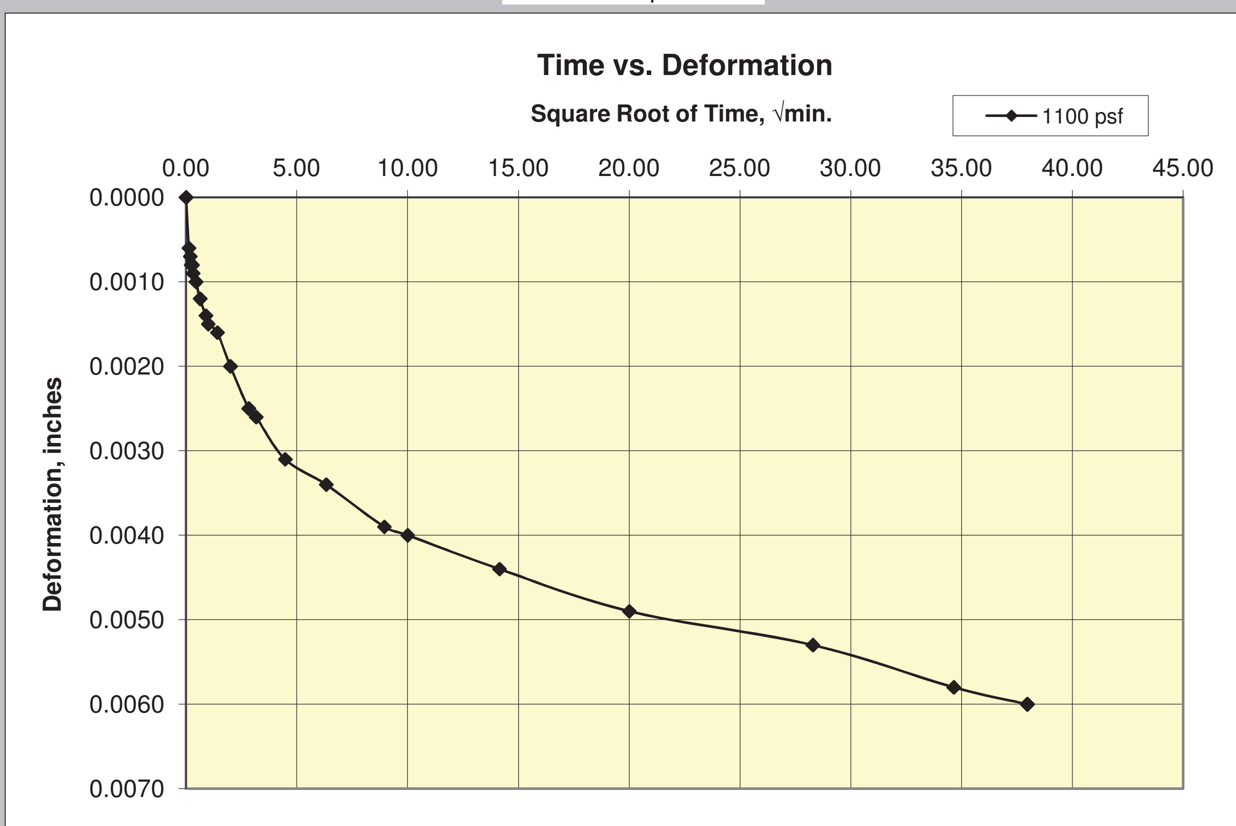
Remarks: The 800 psf point was adjusted to 850 to smooth the curve. It is not uncommon for pneumatic air regulators to drrift as much as 50 psf

Cooper Testing Labs, Inc.

Load 6 1100 psf

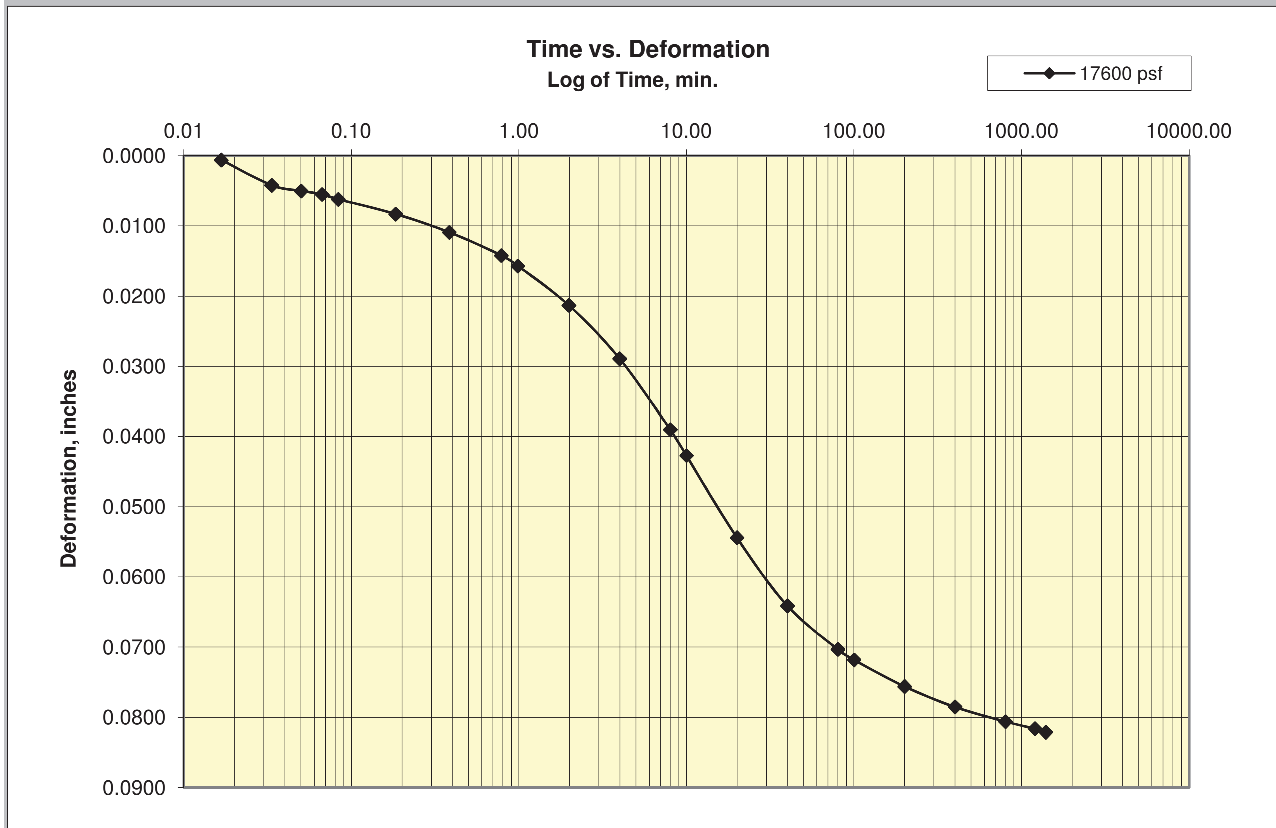


1100 psf

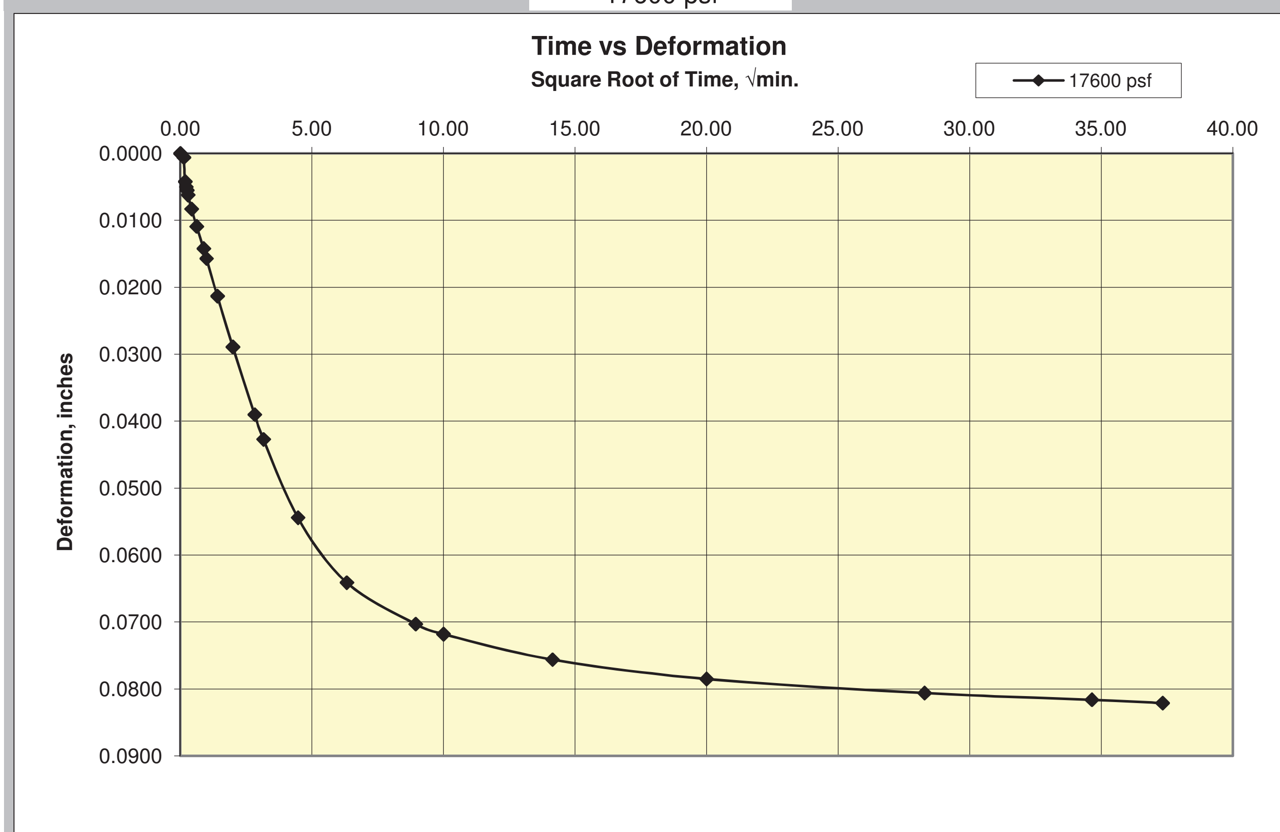


Cooper Testing Labs, Inc.

Load 11 17600 psf



17600 psf



Corrosivity Test Summary

CTL # 109-758 **Date:** 11/11/2015 **Tested By:** PJ **Checked:** PJ
Client: AMEC Foster Wheeler **Project:** Project Zeus **Proj. No:** 6166150082
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-1	S-1-A	0-5	-	976	-	41	82	0.0082	8.5	-	7.0	Very Dark Brown Gravelly Fat CLAY w/ Sand
B-2	S-1-A	0-5	-	1322	-	9	57	0.0057	8.3	-	6.7	Yellowish Brown CLAY w/ Sand
B-3	S-0-A	0-5	-	893	-	9	298	0.0298	8.1	-	7.0	Olive Gray Clayey SAND
B-6	S-1-A	0-5	-	2822	-	10	71	0.0071	8.2	-	4.1	Yellowish Brown Lean Clayey GRAVEL w/ Sand

APPENDIX C

Field Reconnaissance Photo Log

APPENDIX C

FIELD RECONAISSANCE PHOTO LOG

Geotechnical Investigation Report
Project Zeus
Mare Island, Vallejo, California

From May through December of 2015, Amec Foster Wheeler staff took photographs of discrete conditions observed on-site during various visits to the Project Zeus Mare Island site. A log of the photos, including a brief description and date for each photo, are presented in this appendix.

Photograph 1 Foundation and visible void near Mare Island Straight. (05/18/2015)

Photograph 2 Settlement induced damage and visible void adjacent to duct bank near south west corner of site. (10/06/2015)

Photograph 3 Abandoned buried structure typical on north west of site near Amec Foster Wheeler soil boring B-1. Groundwater observed at approximately 4 feet below ground surface. (10/06/2015)

Photograph 4 CPT equipment used by Gregg Drilling during geotechnical exploration activities. (10/09/2015)

Photograph 5 Fraste Multidrill XL drilling equipment used by Pitcher Drilling at Amec Foster Wheeler soil boring B-4. (10/21/2015)

Photograph 6 Vane shear testing at Amec Foster Wheeler soil boring B-2. (10/22/2015)

Photograph 7 Pavement distress, cracking, and potholing over duct bank near Amec Foster Wheeler soil boring B-15. (12/04/2015)

Photograph 8 Pothole in pavement near Amec Foster Wheeler soil boring B-15. (12/04/2015)

Photograph 9	Observed settlement around utility and building structures near Amec Foster Wheeler soil boring B-7. (12/04/2015)

Photograph 10	Observed settlement adjacent to duct bank near Amec Foster Wheeler soil boring B-7. (12/04/2015)

Photograph 11 Observed settlement and pavement distress around abandoned structure on east side of site near Mare Island Straight. (12/04/2015)

APPENDIX D

Seismic Hazard Evaluation Methodology and Results

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SEISMIC HAZARD EVALUATION METHODOLOGY AND RESULTS

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This appendix presents a description of the methodology and results for the ground motion hazard analysis performed for this study. The elements of the site-specific ground motion assessment presented in this appendix are the seismic source characterization (Section 1.0), selection of attenuation relationships (Section 2.0), calculation of frequency of exceedance (Section 3.0) and results of the probabilistic seismic hazard analysis (Section 4.0), and development of uniform hazard equal hazard response spectra (Section 5.0).

1.0 SEISMIC SOURCE CHARACTERIZATION

Two types of potential crustal seismic sources, faults and areal source zones, are included in the seismic hazard model. The assessment of maximum magnitudes and earthquake recurrence rates for both types of sources are described in this section.

Faults that were considered as potential seismic sources include all mapped active and potentially active faults in the San Francisco Bay Region. The major fault sources that could produce large earthquakes capable of causing strong ground shaking at the Mare Island site include the Hayward-Rodgers Creek, Concord-Green Valley, Franklin, West Napa, Greenville, Calaveras, Mount Diablo Thrust, and San Andreas faults.

For this project, active faults are defined as those that have had displacement or seismic activity during the Quaternary period (i.e., 2.6 million years before present [Ma] to the present). These faults are considered likely to be active in the future. All known local and regional faults significant to the site in terms of ground shaking hazard have been identified and characterized as input for the PSHA (Tables 1 and 2). For the present study, Amec Foster Wheeler reviewed previous probability studies (Working Group on California Earthquake Probabilities [WGCEP], 2003, 2008), and the most recent probability study, UCERF3 (Field et al., 2013), as a basis for identifying current information regarding characterization of seismic sources and approaches in evaluating the magnitude and recurrence of earthquakes in the San Francisco Bay Area. The updated SSC model for this study modified the WG03/WG08 rupture source model to give more weight to longer ruptures, and also includes alternative linked fault sources that allow for the occurrence of larger earthquakes on some faults significant to the hazard at Mare Island.

In seismic hazard analysis, areal source zones are used to model the seismicity that cannot be associated with specific geologic structures. This includes random background seismicity and larger events that occur on faults that are not explicitly included in the seismic source model. In the San Francisco Bay Area, the fault sources include the major strike-slip faults and other reverse and oblique-slip faults discussed in the previous section. Other minor faults are assumed to have maximum expected magnitudes equal to or less than the surrounding geologically-based source zones and are assumed to be represented by the seismicity occurring within that source zone. The geometry of these structures is used to define the source geometry of potential earthquakes that occur within regional areal source zones (Figures 4 and 5). In general, the areal source zones are the regions lying between the principal faults of the Bay Area where the seismicity is broadly distributed (Figure 5). For each of the areal source zones, we evaluated the recurrence of earthquakes up to the maximum magnitude based on the seismicity occurring in the zone. This approach assumes a uniform distribution of seismicity throughout the specified source zones. The model also assumes that historical seismicity within the zone reflects the overall level of earthquake hazard, but does not necessarily reflect the spatial distribution of this hazard. The recurrence parameters for the source zones are computed from the historical and instrumental seismicity within each zone using the maximum likelihood methodology developed by Weichert (1980). The recurrence rates for the source zones are based on a truncated exponential model (Cornell and Van Marcke, 1969).

2.0 GROUND MOTION PREDICTION EQUATIONS

A ground motion attenuation model relates the amplitudes of peak ground acceleration and response spectral acceleration to earthquake magnitude and source-to-site distance. Different attenuation models are required for different types of seismic sources. Past studies of strong motion data indicate that the ground motions from the various types of earthquake sources considered in this analysis exhibit different characteristics in terms of the scaling of ground motion amplitudes with magnitude, source-to-site distance, and period of vibration.

For each seismic source, alternative ground motion prediction relationships (GMPEs) were utilized. The uncertainty in the predicted value of a ground motion parameter for each attenuation relationship was modeled by assigning a statistical distribution around the median value relationships in accordance with values given by the authors of the respective GMPEs used in this study.

The GMPEs selected for use in this analysis are those developed for Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation (NGA) West 2 project. These models provide estimates of spectral accelerations in the period range of 0.01 seconds to 10 seconds (spectral frequencies of 0.1 to 100 Hz), representing the median horizontal component of ground motions. The GMPEs are defined in terms of **M** (moment magnitude). All five of the models

(Abrahamson et al., 2014 [ASK14]; Boore et al., 2014 [BSSA14]; Campbell and Bozorgnia, 2014 [CB14]; Chiou and Youngs, 2014 [CY14]; and Idriss, 2014 [ID14]) provide ground motion estimates as a function of the average shear wave velocity of the top 30 meters of the site, $V_{S30.}$ The Idriss (2014) model does not include parameters for sites where the V_{S30} is less than about 1,500 feet per second (450 meters per second), or for normal slip earthquakes; all other models are applicable for softer sites where V_{S30} is equal to 590 feet per second (180 meters per second) (or lower for two models) and for all types of crustal earthquakes. These relationships were developed on the basis of statistical analyses of ground motions recorded during earthquakes at many locations in California as well as in other parts of the western United States and foreign countries having similar tectonic environments.

As described in Section 5.2 of the main report, the shear wave velocity (V_{S30}) identified for use in site response analysis is 550 m/s, representing the top of weathered Great Valley Group sedimentary rock at an average depth of xxx m at the project site (as described in Section xx of the main report), this velocity is taken at a depth of xxx m based on the shear wave evaluations obtained for the site. A lower V_{S30} of 285 m/s corresponding to the transition from old alluvium to Old Bay Sediments was used to prepare preliminary ground motion response spectra for the site; in this analysis, we used the four applicable NGA GMPEs (ASK14, BSSA14, CB14, and CY14) for this site condition. For continuity with these preliminary analyses, these same four GMPEs were used in the final PSHA and DSHA calculations performed for the top of weathered rock with V_{S30} of 550 m/s. These relationships were weighted equally in the hazard analysis because the scientific community considers all of them to be valid scientific models.

3.0 CALCULATIONS OF FREQUENCY OF EXCEEDANCE

The mathematical formulation used in most PSHAs assumes that the occurrence of damaging earthquakes can be represented as a Poisson process. Under this assumption, the probability that a ground motion parameter, Z, will exceed a specified value, z, in time period t is given by:

$$P(Z > z | t) = 1 - e^{-\nu(z) \cdot t} \leq \nu(z) \cdot t \qquad \text{(D-1)}$$

where $\nu(z)$ is the average frequency during time period t at which the level of ground motion parameter Z exceeds value z at the site from all earthquakes on all sources in the region. Equation (D-1) is valid provided that $\nu(z)$ is the appropriate average value for time period t. In this study, the hazard results are reported in terms of the frequency of exceedance $\nu(z)$.

The frequency of exceedance, $\nu(z)$, is a function of the frequency of earthquake occurrence, the randomness of size and location of future earthquakes, and the randomness in the level of ground motion they may produce at the site. It is computed by the expression:

$$\nu(z) = \sum_n \alpha_n(m^0) \int_{m^0}^{m^u} f(m) \left[\int_0^{\infty} f(r|m) \cdot P(Z > z|m,r) \cdot dr \right] \cdot dm \qquad \text{(D-2)}$$

where $\alpha_n(m^0)$ is the frequency of earthquakes on any given source n above a minimum magnitude of engineering significance, m^0; $f(m)$ is the probability density of earthquake size between m^0 and a maximum earthquake the source can produce, m^u; $f(r|m)$ is the probability density function for distance to an earthquake of magnitude m occurring on source n; and $P(Z>z|m,r)$ is the probability that, given an earthquake of magnitude m at distance r from the site, the peak ground motion will exceed level z. The frequency of earthquake occurrence, $\alpha_n(m^0)$, and the size distribution of earthquakes, $f(m)$, were determined by the earthquake recurrence relationships developed by Cornell and Van Marcke (1969). The spatial distribution for the distance between the earthquake rupture and the site was determined by the geometry of the seismic sources defined in Tables 1 and 2, and shown on Figure 5. The conditional probability of exceedance, $P(Z >z|m,r)$, was determined using the GMPEs described above. The GMPEs define the level of ground motion in terms of a lognormal distribution.

The hazard was computed considering the contributions of earthquakes of magnitude **M** 5 and larger (m^0 = 5); **M** 5 is commonly considered to be the lower bound of earthquake magnitude that is capable of producing damage to reasonably well-engineered (design and construction) facilities. At each ground motion level, the complete set of results forms a discrete distribution for frequency of exceedance, $v(z)$. The computed distributions were used to obtain the mean frequency of exceeding various levels of ground motion (mean hazard curve).

4.0 RESULTS OF THE PROBABILISTIC SEISMIC HAZARD ANALYSIS

The basic results of the PSHA are presented in terms of annual frequency of exceedance versus spectral acceleration (commonly referred to as hazard curves). The frequencies of exceedance of various values of peak ground acceleration and response spectral acceleration at the site for given structural periods were calculated by combining, for each fault and then for all the faults:

- the annual frequency of earthquakes of various magnitudes on a fault obtained from the fault recurrence relationships;
- given an earthquake of a certain magnitude on a certain fault, the probability distribution of the location of the earthquake on the fault obtained using the selected rupture area versus magnitude relationship and assuming equal likelihood of rupture along the length and some prescribed probabilities along the depth of the fault; and
- given an earthquake of a certain magnitude occurring at a certain distance from the site, the probability distributions of ground motion at the site obtained from the selected GMPEs. For this study, we used GMPEs corresponding to peak ground acceleration (0.01 second) and twenty structural periods (ranging from 0.02 to 10 seconds) at a damping ratio of 5 percent.

Having obtained the annual frequency of exceedance of a certain level of horizontal response spectral acceleration, the probability of exceeding that level within any time period of interest is then obtained assuming a Poisson distribution, as follows:

$$P_E = 1 - \exp(-\mu t)$$

in which "P_E" is the probability of exceedance, "μ" is the annual frequency of events that exceed that level of ground motion, and "t" is the specified time period of interest.

5.0 UNIFORM HAZARD RESPONSE SPECTRA FOR SELECTED GROUND MOTION EXEEDANCE PROBABILITIES

PSHA results for the Mare Island site assessed during this study were obtained for various hazard levels and 21 spectral periods at five-percent damping. The mean hazard curves for each spectral period analyzed were interpolated to obtain values of spectral acceleration associated with probabilities of exceedance of approximately 20%, 10%, 5%, 2%, and 1% in 50 years (corresponding to equivalent return periods of approximately 225, 475, 975, 2,475 years, and 4,975 years, respectively). These values at the 21 spectral periods analyzed were then utilized to construct uniform hazard response spectra (UHRS) representative of the estimated ground shaking hazard at the sites (Figure 12). The spectral ordinates calculated for peak ground acceleration (PGA) and additional 20 periods for the 2% probability of exceedance of in 50 years are shown in Table 4.

6.0 REFERENCES

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