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REPORT
GEOTECHNICAL STUDY
UPRR BRIDGE STRUCTURE,
ASSOCIATED RETAINING WALLS,
AND APPROACH EMBANKMENTS
12300 SOUTH DESIGN-BUILD PROJECT
*HPP-STP-0071(12)0
DRAPER, UTAH

Submitted To:

GRW
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Suite 108
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Submitted By:

AMEC Earth & Environmental, Inc.
Salt Lake City, Utah

December 6, 2002

Job No. 2-817-004066/4062

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GRW
12257 South Business Park Drive
Suite 108
Draper, Utah 84020

Attention: **Mr. Con Wadsworth**

Gentlemen:

Re: Report
Geotechnical Study
UPRR Bridge Structure, Associated Retaining Walls, and Approach Embankments
12300 South Design-Build Project
*HPP-STP-0071(12)0
Draper, Utah

1. GENERAL

1.1 INTRODUCTION

This report summarizes geotechnical design recommendations for the planned Union Pacific Railroad (UPRR) Bridge structure over 12300 South Street in Draper, Utah, and associated retaining walls and approach embankments. The general location of the site with respect to major topographic features and existing facilities, as of 1999, is presented on Figure 1, Vicinity Map. The general layout of the Corner Canyon area, the bridge crossing, and portions of the embankment walls are presented on Figures 2A, Exploration Location Plan Key; Figure 2B, Corner Canyon; and Figure 2C, Bridge Structure and Retaining Walls. The locations of the subsurface explorations conducted in conjunction with this and previous studies, are also presented on Figures 2B and Figure 2C. A more detailed layout of the bridge site showing the planned location and profile of the bridge, and the location of the existing roadway, is presented on Figure 3, Bridge Site Plan.

The general conclusions and recommendations described herein were provided to the design-build team during the course of this study verbally, via email, and in memoranda dated September 16, 2002[1], September 17, 2002[2], November 8, 2002[3], and November 15, 2002[4]. Please

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[1] "Memorandum, Geotechnical Recommendations, UPRR Bridge Structure, 12300 South Design-Build Project, *HPP-STP-0071(12)0, Draper, Utah," AMEC Job No. 2-817-004062.

[2] "Memorandum, Geoseismic Criteria, UPRR Bridge Structure, 12300 South Design-Build Project, *HPP-STP-0071(12)0, Draper, Utah," AMEC Job No. 2-817-004062.

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note that the conclusions and recommendations provided herein are subject to review and comment by external agencies and may be revised accordingly.

1.2 PROJECT DESCRIPTION

The 12300 South Design-Build Project (Project) will consist of the widening of 12300/12600 South Street from the Bangerter Highway in Riverton, Utah, to 700 East Street in Draper, Utah. The project will include elevating the existing UPRR tracks from the existing 12300 South Street at-grade crossing in Draper, Utah. To separate the railroad and street grades at the crossing, the railroad alignment will be raised above existing alignment grades and 12300 South Street will be lowered below existing street grades. In order to maintain railroad traffic during construction, a temporary detour alignment, referred to as a "shoofly," with an at-grade crossing will be constructed to the west of the existing mainline alignment. The shoofly will consist of a single mainline track over the entire alignment, with a siding track at the south end. The railroad construction will include extending an existing siding track located south of 12300 South Street further to the south and across Corner Canyon.

The railroad alignment will cross 12300 South via a bridge structure. Based on the available bridge design information, including layout and cross section drawings of the bridge and the mainline track profile, the bridge will be a two-span structure with a total span length of about 109 feet and a width of about 79 feet. The bridge will be supported on abutments at the north and south ends and an interior bent located along the centerline of 12300 South Street. The abutments will have a maximum height of about 28 feet measured between the planned grades along 12300 South Street and the top-of-rail. The abutments will also act as earth-retaining structures, supporting backfill placed between the abutments and temporary roadway cuts, and fills placed to construct the bridge approach embankments. The interior bent will be pier-supported above the foundation.

The bridge structure will support two mainline UPRR tracks, a future commuter rail track, and a railroad maintenance-of-way access road. Based on preliminary load information, the total dead loads at the abutments and the interior bent are projected to be about 2,900 and 3,500 kips, respectively. Total live loads, not including impact loads, at the abutments and the interior bent are projected to be about 1,700 and 3,400 kips, respectively. With additional impact loads included, the total live loads at the abutments and the interior bent are projected to be about 2,200 and 4,400 kips, respectively.

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[3] "Draft Memorandum, Geotechnical Recommendations, UPRR Embankment Retaining Walls, 12300 South Design-Build Project, *HPP-STP-0071(12)0, Draper, Utah," AMEC Job No. 2-817-004062.

[4] "Memorandum, Revised Geoseismic Criteria, UPRR Bridge Structure, 12300 South Design-Build Project, *HPP-STP-0071(12)0, Draper, Utah," AMEC Job No. 2-817-004062.

To limit the height of the bridge approach embankments, the finished grades along 12300 South Street will be lowered approximately 10 feet. Based on preliminary embankment cross sections, the maximum height of the approach embankments (to top of sub-ballast) will range from about 15 to 16 feet above existing grades. To transition between the crest of the embankments and the existing grades, reinforced concrete wing walls of varying height will be constructed to the east and west of each abutment to support the embankment fills and backfills between the walls and the temporary road cuts. We understand that these wing walls will be constructed as stand-alone, cantilevered, concrete retaining structures. Based on available information, we project that the footings for these wing walls will be established at least three feet below the lowest adjacent grade for frost protection and to provide for cover over the top of the footings. Based on a minimum roadway elevation of about 4404 feet at the bridge, the base of the wing wall foundations could be established at a maximum elevation around 4401 feet.

Between the ends of the cantilevered concrete wing walls and the ends of the road cuts, mechanically stabilized earth (MSE) retaining walls will be constructed to provide the grade transition along the 12300 South Street roadway cuts. These will be permanent MSE structures typically referred to as single-stage MSE wall systems. An MSE wall specialty firm will design the MSE walls.

The shoofly railroad track will have an overall length of about 7,800 feet. Except for two short segments, the shoofly alignment will cross relatively flat terrain requiring minor cuts or fills generally ranging up to about three feet. Where the shoofly alignment crosses 12300 South Street, cuts up to six feet will be required. Up to about 14 feet of embankment fill will be required where the southern portion of the alignment crosses a drainage ravine, and up to about 17 feet of embankment fill will be required where the northern portion crosses Willow Creek. Note that these cut and fill heights are based on the top of the track sub-ballast and do not include the railroad track ballast, which we understand will be about two feet thick. Based on discussions with the design-build team, the shoofly fill embankment side slopes will range from as steep as one and one-half horizontal to one vertical (1.5H:1.0V) to two horizontal to one vertical (2.0H:1.0V), with the 1.5H:1.0V preferred. The shoofly alignment will cross at least three irrigation ditches requiring installation of temporary culverts or siphon piping, and will cross over existing 36-inch and 48-inch diameter reinforced concrete pipes at Willow Creek.

The mainline track construction will extend northward from south of Corner Canyon to just north of Willow Creek. The total length of the railroad construction will be about 13,400 feet. Except for three segments, the mainline alignment crosses relatively flat terrain requiring minor cuts or fills generally ranging up to about six feet. Wedge fills up to about 37 feet high will be required at Corner Canyon, and wedge fills up to 17 feet high and cuts 10 feet high, will be required at or near Willow Creek. The approach embankments to the bridge will require fills ranging in height from a few feet to as much as 22 feet at Willow Creek. Note that these cut and fill heights are based on the top of the track sub-ballast and do not include the railroad track ballast, which we understand will be about two feet thick.

The mainline fill embankment side slopes generally will be constructed at 2.0H:1.0V. Due to right-of-way constraints, an approximately 500-foot long embankment fill section at Corner Canyon will be constructed with slopes at 1.5H:1.0V. These steeper slopes may require special erosion control measures. Also due to right-of-way constraints, retaining walls will be required along segments of the east side of the fill embankments extending from about 800 feet south of 12300 South Street to almost 2,800 feet north of 12300 South. Based on preliminary design information, these retaining walls will range from about 4 to 17 feet in height. These retaining walls will generally be founded on the descending 2.0H:1.0V embankment fill slopes.

The mainline alignment crosses a concrete box culvert at Corner Canyon, at least two existing CMP culverts, and a pair of existing, large-diameter (78-inch and an 84-inch) steel pipes that carry Willow Creek beneath the existing embankment. Construction will include extending the Willow Creek culverts further west to a new channel also to be constructed as part of the Project.

Site earthwork will include surface preparation, excavations for utility relocation and foundation construction, excavations for the shoofly and mainline alignments, benching of existing mainline embankment fills to receive new fills, and placement of new embankment fills for the shoofly and mainline tracks. Along 12300 South Street, the maximum excavation depths for lowering the roadway grade and abutment wing wall footings are projected to range from about 13 to 15 feet. Deeper excavations to depths of up to about 18 feet may be required for construction of the bridge abutment foundations. Temporary cuts up to 10 feet high may be required in existing embankment fill slopes to bench in new embankment fills, particularly at Corner Canyon.

2. PREVIOUS REPORTS AND INVESTIGATIONS

As part of preliminary planning for the Project, the Utah Department of Transportation (UDOT) authorized Kleinfelder Consultants in 2002 to complete one soil boring (B-5) and one cone-penetration test (CPT) probe (CPT-6) at the existing railroad crossing. Subsurface information from those explorations, as well as reported data from limited laboratory testing conduced on samples obtained from the borings, was provided by UDOT in the Project Request-for-Proposal (RFP). The approximate locations of these explorations are indicated on Figures 2B and 2C. The locations of the boring and CPT probe are based on drawings provided in the RFP.

3. EXISTING FACILITIES

Existing facilities within the project area consist of the mainline tracks, siding tracks extending through and south of the crossing at 12300 South Street, and miscellaneous switching and signaling equipment. Significant embankments supporting the mainline track are located at Willow

Creek at the north end of the project area, at Corner Canyon near the south end of the project area, and at a minor drainage ravine to the north of Corner Canyon. The existing embankment at Corner Canyon extends to heights approaching 40 feet.

The existing alignment crosses a pair of large-diameter (78-inch and 84-inch) steel pipes that channel Willow Creek from east-to-west through the existing embankment, as well as other miscellaneous drainage culverts. Extending north and south of 12300 South Street are gravel-surfaced maintenance roads located along the existing tracks.

The majority of the railroad right-of-way is surrounded by agricultural land, although scattered residential and commercial developments are located along the alignment. The area immediately surrounding the crossing ranges from agricultural land to developed commercial properties. The property to the southeast of the crossing is occupied by a large, relatively new commercial development with paved areas extending almost to the railroad right-of-way. To the northeast of the crossing, the Jordan and Salt Lake Canal extends up to and parallels the east side of the right-of-way along a portion of the mainline alignment. There are no substantial improvements on properties to the west of the right-of-way or to the northeast of the crossing (except for the canal).

Several underground utilities extend along 12300 South Street and through the crossing. These utilities include a Questar gas line, numerous telephone and fiber optic lines, a sanitary sewer line, a culinary water line, and possibly a storm water line. Most, if not all, of these underground utilities will be relocated as part of the Project.

4. FINDINGS

4.1 SITE SURFACE CONDITIONS

The shoofly alignment and existing mainline alignment cross relatively flat terrain except at Willow Creek, Corner Canyon, and the minor drainage ravine north of Corner Canyon. The mainline portion of the project will follow the existing alignment, which is characterized by the existing track structure and embankments. The shoofly alignment will cross mostly undeveloped agricultural land, as well as 12300 South Street and part of the Willow Creek drainage.

At the bridge crossing, the site is relatively flat and currently occupied by the existing track structure, gravel-surfaced railroad maintenance roads, and 12300 South Street. Vegetation along the alignment consists mostly of native grasses and minor brush with occasional trees.

4.2 GEOLOGY

The site is located at the eastern edge of the Bonneville Basin, a deep, sediment-filled structural basin along the eastern edge of the Basin and Range physiographic province. The Bonneville

Basin covers most of northwestern Utah and is bounded immediately to the east of the site by the Wasatch Range of the Middle Rocky Mountain physiographic province.

The topography and near-surface geology at the site are the result of deposition and erosion processes, as well as tectonic activities, over the past approximately 32,000 years. During the Bonneville Lake cycle, which began approximately 32,000 years ago, Lake Bonneville began a slow transgression (rise), with several fluctuations and pauses, before reaching its highest level at about 5,200 feet above sea level about 16,000 years ago. Approximately 14,500 years ago, the lake level dropped rapidly to a level of about 4,880 feet above sea level as the result of catastrophic down cutting of the lake's natural dam at Red Rock Pass in southeastern Idaho. Isostatic rebound and rapid erosion of shoreline sands and gravels and deltaic deposits of the Bonneville phase generally accompanied the rapid decline in the lake level. Additional down cutting at Red Rock Pass and isostatic rebound, as well as changing climatic conditions, reduced the lake level to that near the present day level (4,210 feet above sea level) of the Great Salt Lake about 11,000 years ago.

During deeper lake periods, relatively thick lacustrine sequences of clays, silts, and laminated clays and silts were deposited in the Bonneville basin. As the lake level dropped, streams down-cut through the lake deposits along the mountain front, resulting in complex alluvial and deltaic deposits of granular silt, sand, and gravel that fingered out into the receding lake. In addition, shoreline processes resulted in beach sand and gravel deposits at various stages of the Lake Bonneville regression, and earthquake activity during the Holocene resulted in areas of liquefaction and consequent deformation of predominantly lacustrine sediments.

Based on geologic mapping, subsurface conditions at the site consist predominantly of soils of lacustrine origin. Based on conditions encountered at the exploration locations, the lacustrine soils are underlain at depths of about 105 feet by coarse-grained non-lacustrine soils. These coarse-grained non-lacustrine soils were deposited during interglacial periods similar to today. The lacustrine soils are fine-grained, consisting of layers of clays, silts, and fine sands. Within the site vicinity, the lacustrine soils include near-surface zones of alluvial soils consisting primarily of stream deposits of Holocene age. Where mapped, these alluvial soils typically consist of sand, silt, and minor amounts of clay and gravel. The mapped alluvium is poorly to moderately sorted and typically contains parallel and cross bedding.

4.3 GEOLOGIC HAZARDS

Site geologic hazards are considered to be primarily associated with site seismic ground motion (versus fault hazard). There is a slight possibility of flooding of Willow Creek under an extreme storm event. Due to the site location, geologic hazards such as landslides, rock falls, and mud and debris flows are not expected to occur.

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4.4 FAULTS AND SEISMICITY

In general, the site area is situated within the Intermountain Seismic Belt (Arabasz et al., 1992; Pechmann and Arabasz, 1995). This seismic zone extends from southern Nevada through the state of Utah northward to the Yellowstone area, and north along the mountainous part of western Montana. The Intermountain Seismic Belt is characterized by infrequent, yet moderate to large magnitude earthquakes with relatively shallow focal depths (typically less than 11 miles). Historically, the epicenter of the largest earthquake to occur within the Intermountain Seismic Belt was located near Hebgen Lake, Montana in 1959. That earthquake had a Moment magnitude of 7.3.

Geologic and geomorphic evidence shows that repeated, normal-slip surface faulting has occurred in the Salt Lake Valley through late Pleistocene to Holocene time. The Wasatch fault zone is the most obvious and continuous structural element of the prominent transition zone between the Basin and Range province to the west and the Colorado Plateau and Middle Rocky Mountain provinces to the east (Machette, et al., 1992). The Wasatch fault zone extends from Malad City, Idaho on the north to Fayette, Utah on the south, a distance of about 240 miles. Studies of the fault zone suggest that it is composed of 10 discrete segments, each of which may rupture independently during a major earthquake. Because the length of surface rupturing is generally dependent on earthquake magnitude, the largest earthquake on a single segment within the Wasatch fault zone is estimated to be a magnitude 7.3 event.

In general, earthquakes with magnitudes less than about 6.5 are not considered to cause surface rupturing. Therefore, the occurrence of these earthquakes is not limited to areas where faults have been mapped on the surface. Earthquakes with magnitudes between about 6.5 and 7.3 are expected in the region and are generally associated with identifiable faults, such as the Salt Lake segment of the Wasatch fault zone, that show evidence of recent seismic activity.

The Salt Lake Segment extends for about 28 miles from the Traverse Mountains to the south to the Salt Lake Salient to the north (Warm Springs area). The overall alignment is located between three and three-eighths and three and five-eighths miles west-northwest of the closest portion of the Salt Lake segment of the Wasatch fault zone, which presents the largest source of seismicity in the immediate project vicinity.

4.5 SUBSURFACE SOIL CONDITIONS

Our understanding of site subsurface conditions is based primarily on the results of our current field-exploration and laboratory-testing programs, which were focused on the bridge and mainline walls and embankments. Field explorations were not conducted for the shoofly.

Our field exploration program consisted of drilling three borings (AB-9, AB-20, and AB-21[5]) to depths ranging from about 30.5 to 115.5 feet below existing site grades, advancing five CPT probes (ACPT-13 through ACPT-17[6]) to depths of about 30 feet each, and advancing one CPT probe (ACPT-3) to refusal at a depth of about 108 feet. In addition, two test pits (ATP-1 and ATP-2[7]) were excavated at Corner Canyon to depths of 6 and 18 feet, respectively, to evaluate fill and natural soil conditions along the existing embankment. The approximate locations of the explorations are indicated on Figures 2B and 2C. The subsurface exploration program included installation of temporary piezometers in Borings AB-9, AB-20, and AB-21. A discussion of the field exploration procedures, together with our boring logs, is presented in Appendix A. The CPT logs and associated data are presented in Appendix D.

The information from our current field-exploration and laboratory-testing programs has been supplemented by subsurface information from other geotechnical studies previously conducted by AMEC in the alignment vicinity. In addition, explorations conducted at the bridge location for UDOT in 2002 during the preliminary planning process for the Project (provided in the RFP) were used. The UDOT subsurface explorations extended to maximum depths of about 145 feet below existing site grades. Copies of the logs of the boring and CPT probe accomplished for UDOT are presented in Appendix B for reference.

In general, the soils encountered at the exploration locations along the mainline railroad alignment consist primarily of natural cohesive and cohesionless lacustrine soils, which extend to depths of about 105 feet below ground surface at the bridge location. The deeper explorations at the bridge location encountered cohesionless, predominantly granular, non-lacustrine soils beneath the lacustrine soils. At the boring locations, the natural lacustrine soils are overlain by about three feet of loose fills consisting of sandy silts and silty sands and gravels. We presume that these fills are related to past grading activities within the site vicinity.

The bridge site is underlain to depths of around 40 feet by moderately thick, inter-layered zones of sands, sandy silts, and slightly to moderately plastic clayey silts and silty clays. These zones range in thickness from about 2 feet to as much 15 feet. Within each zone, the primary soil types are often inter-layered with other soil types. The surficial soils to a depth of about 16 feet generally consist of slightly to moderately plastic, clayey silts to silty clays containing fine sand and zones of sandy silts. Below a depth of about 10 feet, the silts and clays are layered with fine sands containing varying amounts of silt. Below a depth of about 16 feet, and extending to depths of about 40 feet, the explorations predominantly encountered fine to medium sands and occasional zones of layered clayey silts to silty clays and layered silts, clays, and silty sands. The sands contain varying amounts of silt and occasional layers of clayey silt to silty clay. Based on drive

5 The designation "AB" indicates an AMEC boring.
6 The designation "ACPT" indicates an AMEC CPT probe.
7 The designation "ATP" indicates an AMEC test pit exploration.

sampler penetration resistance, the fine-grained cohesive soils (clayey silts and silty clays) to depths of about 40 feet range from stiff to very stiff, and the cohesionless silts and sands vary from loose to medium dense.

Below depths of about 40 feet, and extending to depths of about 105 feet, the bridge location is underlain by fine-grained cohesive soils consisting of slightly to moderately plastic, clayey silts and silty clays with sand layers. These clayey silts and silty clays often are inter-layered and contain frequent seams and thin layers of sandy silt and silty fine sand. Based on drive sampler penetration resistance, these fine-grained cohesive soils are considered to be stiff to very stiff.

Below depths of about 105 feet, and extending to the maximum depth explored of about 145 feet, the bridge location is underlain by dense to very dense, non-lacustrine sands and gravels. CPT probe ACPT-3 and the CPT probe CPT-6 conducted for UDOT in 2002 both refused in these soils at depths of about 108 and 111 feet, respectively.

Based on subsurface information from other explorations in the site vicinity, the conditions described above are projected to be relatively consistent across the project vicinity, including the mainline railroad and shoofly construction areas. The supplemental geotechnical investigations upon which these observations are based are being retained in our files.

4.6 GROUNDWATER

Piezometers were installed in Borings AB-9, AB-20, and AB-21 to measure changes in static groundwater levels at the site. Following installation, stabilized groundwater levels were measured on three separate dates. The results of the groundwater level measurements are summarized in the following table along with the corresponding approximate groundwater elevations (based on available ground surface elevation data provided in the RFP).

Stabilized Groundwater Levels (ft)

	AB-9		AB-20		AB-21	
Date	**Depth**	**Elevation**	**Depth**	**Elevation**	**Depth**	**Elevation**
August 8, 2002	11.6	4402	10.5	4404	10.2	4405
September 7, 2002	10.5	4404	10.9	4403	10.7	4404
October 22, 2002	13.5	4401	14.2	4400	11.8	4403

As shown above, the stabilized groundwater levels at the crossing have dropped over a nearly three-month period. It should be noted that these measurements were obtained near the end of summer and beginning of fall when groundwater levels are typically at their highest level, and within the fourth year of an extended drought cycle. Seasonal and long-term groundwater levels are expected to fluctuate by one to two feet, with the lowest groundwater levels typically occurring during the late fall and winter months.

5. EARTHQUAKE CONSIDERATIONS

5.1 SEISMIC HAZARDS

Site faulting and seismicity are discussed in Section 4.4 of this report. Seismic hazards that could be expected at the site would include ground shaking and liquefaction.

5.2 DESIGN CRITERIA

5.2.1 Site Class

As previously summarized herein, the soils underlying the site to depths of 100 feet or more (about 105 feet at the bridge site) are of lacustrine origin and consist of sandy silts, sands, and slightly to moderately plastic clayey silts and silty clays. The clayey silts and silty clays are considered to be generally medium stiff to very stiff with some soft near-surface zones, and they contain varying amounts of fine sand. The plasticity index value for these cohesive soils varies slightly with depth. To depths of between 10 and 15, the plasticity index value ranges from 18 to 21. For cohesive soils between the depths of about 50 and 90 feet, the plasticity index value varies from 12 to 14. Moisture contents are typically below 40 percent (two samples had moisture contents slightly over 40 percent). Based on drive sampler penetration resistance (blow counts), the cohesionless sandy silts and sands underlying the site to depths of about 50 feet range from loose to medium dense.

Soil shear wave velocities were measured at the bridge site during advancement of CPT probe ACPT-3. The measured shear wave velocities ranged from about 450 feet per second within 13 feet of the existing ground surface to 1,075 feet per second at a depth of about 102 feet. Below a depth of about 13 feet, the measured shear wave velocities exceeded 600 feet per second. The average shear wave velocity to a depth of 100 feet is about 740 feet per second.

Based on the subsurface conditions encountered in the deep borings, and on the shear wave velocity data, the bridge site is considered to meet the criteria for Site Class D (stiff soil profile) conditions as described in Table 1615.1.1 of the 2000 International Building Code (IBC 2000). According to criteria outlined in the American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering, a Site Class D profile would correspond to a Soil Type 2 (as defined in Table 9-1-6 of the AREMA Manual).

5.2.2 Ground Motion

According to the UDOT Geotechnical Manual of Instruction (MOI), all bridges must be designed to meet the ground motions generated by the 2 percent in 50-year event (10 percent in 250-year hazard level), along with all walls located within about 50 feet of a bridge foundation and/or that affect the performance or integrity of the bridge. All other walls and embankments must be designed to meet the ground motions generated by the 10 percent in 50-year event. According to the AREMA Manual, a three-level design approach is used that includes consideration of the 100-year, 475-year, and 2,375-year return events, which correspond to the 40 percent in 50-year, the 10 percent in 50-year, and the 2 percent in 50-year events, respectively.

The United States Geologic Survey (USGS), through the National Earthquake Hazards Reduction Program (NEHRP, 1997), has evaluated and mapped the general seismic characteristics of the conterminous United States, particularly the western United States. The MOI requires that the NEHRP ground motion data be used in seismic design of highway structures. The NEHRP ground motion data are probabilistic peak horizontal ground accelerations associated with points mapped on a grid system. The ground motion data for a site can be determined based on the latitude and longitude coordinates of the site. The acceleration values apply to the dense soil to rock boundary between Site Classes B and C (Site Class B-C Boundary), as defined by NEHRP 1997.

The coordinates of the railroad crossing are 40.5267 degrees north latitude and 111.9038 degrees west (negative) longitude. The ground motion values for the USGS grid point closest to the site location are summarized in the following table. The ground motion values in the table incorporate soil amplification factors for a Site Class D soil profile.

Acceleration Criteria	Probabilistic Peak Horizontal Ground Accelerations (percent g) Mapped USGS Values With Soil Amplification		
	40% in 50-yr Event (100-yr return period)	**10% in 50 yr Event (475-yr return period)**	**2% in 50 yr Event (2475-yr return period)**
PGA* (g)	0.10	0.30	0.52
0.2 sec SA** (g)	NA	0.74	1.30
1.0 sec SA** (g)	NA	0.35	0.95

* PGA - Peak Ground Acceleration (horizontal)
** SA - Spectral Acceleration (for response spectrum)

The AREMA design manual provides a risk-based approach for modifying the seismic acceleration coefficients. Based on information provided by HW Lochner, this risk-based approach results in shortening the return periods of the three levels of seismic analysis required by AREMA. The following table summarizes the AREMA modified acceleration coefficients for the railroad crossing.

Probabilistic Peak Horizontal Ground Accelerations (percent g)
AREMA Modified Values with Soil Amplification

Acceleration Criteria	40% in 50-yr Event (95-yr return period)	10% in 50-yr Event (373-yr return period)	2% in 50-yr Event (1980-yr return period)
Peak Ground Acceleration	0.10*	0.26	0.46

* Recommended value. Reduction technically valid only for events with probabilities of occurrence of between 10% and 2% in 50 years.

5.2.3 Liquefaction

Liquefaction is defined as the condition when saturated, loose, and cohesionless, sand-type soils lose their support capabilities because of excessive pore water pressure that develops during a seismic event. Several conditions are necessary for liquefaction to develop: loose cohesionless soils, a moderate to high groundwater table, and sufficient seismicity to cause the soils to liquefy.

We have analyzed the liquefaction potential for the bridge site for both the 10 percent in 50-year and the 2 percent in 50-year seismic events (based on the USGS ground motion values). Subsurface and seismic conditions at the bridge site are considered to be generally representative of the overall railroad alignment, and the general results of our liquefaction analyses should be applicable over the alignment. The proposed construction at the bridge site will result in a distinct topographic break along 12300 South Street through the lowering of 12300 South and the placement of the railroad embankment fills. Accordingly, three separate liquefaction cases were evaluated for each event:

1. The bridge site as it currently exists (prior to construction).
2. Portions of the bridge site where overburden will be removed (along the base of the abutments and associated wing walls).
3. The portions of the bridge site where up to 18 feet (including ballast) of embankment fills will be placed behind abutments and portions of the wing walls.

Our analyses were conducted using methods that correlate liquefaction primarily to soil density determined from drive sampler driving resistance (standard penetration test (SPT) N-values). To confirm those results, we also conducted a liquefaction analysis based on the data from CPT probe ACPT-3.

The results of our liquefaction analyses indicate that, for all three cases, localized liquefaction of a few layers may occur during the 2 percent in 50-year event, but would not occur during the 10 percent in 50-year event. Our analyses indicate that liquefaction could occur within predominantly cohesionless silt and sand layers between the depths of about 17 to 28 feet below current site grades. The maximum thickness of these zones of potentially liquefiable soil ranges from about five feet to ten feet based on information from the borings. The results of our CPT-based liquefaction analysis indicate that the layers of potentially liquefiable soil within these zones are actually relatively thin (about 1 to 2 feet thick) and are separated by layers of non-liquefiable, predominantly cohesive soils. Specialized treatment of such thin layers is difficult and generally not warranted.

The primary effect of potential liquefaction at the site under the 2,475-year event will be both total and differential, post-earthquake, ground-surface settlement. Calculations based on soil density relationships indicate that potential ground-surface settlements ranging from about seven-eighths of an inch to about two inches may occur. Based on the CPT data and the results of the CPT-based liquefaction analysis, the total thickness of potentially liquefiable soils is considered to be significantly less than that assumed for the density-based (SPT "N" value) analyses. Accordingly, we project that total post-earthquake, ground-surface settlements will be about one inch or less. This calculation does not consider the benefit provided by the non-liquefiable deposits overlying the potentially liquefiable layers. Based on these considerations, we do not consider that special ground improvement measures will be necessary for either the approach embankments or bridge abutments.

Another significant effect of liquefaction is "lateral spreading," which can occur on sloping sites, sites with abrupt vertical faces intersecting liquefiable zones, or sites where the liquefiable soils are relatively thick and close to the ground surface. The site of the railroad crossing is essentially flat, and the potentially liquefiable soil zones are moderately deep and will be even after grades are lowered along 12300 South Street. Accordingly, the potential for liquefaction-induced lateral spreading at the site is considered to be negligible.

6. LABORATORY TEST DATA

6.1 LABORATORY TESTING

A series of laboratory tests were performed on disturbed and undisturbed samples from our current explorations to assess geotechnical properties of the soils along the mainline alignment. Laboratory testing included evaluation of soil index properties and determination of soil strength and compressibility characteristics. A description of our laboratory program, including test procedures and results, is presented in Appendix C.

Classification testing was performed to determine soil index properties including natural moisture content, in-situ dry density, Atterberg Limits, and grain-size distribution. The index properties were determined for use in soil classification, correlation of field and other laboratory test data, and specific analyses, including liquefaction potential.

Laboratory vane shear strength testing was conducted on 10 relatively undisturbed samples to determine the undrained shear strength of fine-grained cohesive soils encountered in the explorations. Strength values from relevant tests were used to evaluate the bearing capacity of the near-surface fine-grained cohesive soils and embankment stability, and in preparing recommendations for temporary cut slopes in those soils. The results of the vane shear tests indicate that the undisturbed fine-grained cohesive soils underlying the site possess moderate shear strengths ranging from about 1,500 to 2,350 pounds per square foot.

One-dimensional consolidation testing was conducted on 8 relatively undisturbed samples of fine-grained cohesive soils to determine their compressibility characteristics. The results of these tests were used to evaluate settlement of the bridge wing walls, associated MSE wall systems, and the mainline embankments. The results of the consolidation tests indicate that the fine-grained cohesive soils underlying the site are moderately to highly over-consolidated near the surface, with over-consolidation ratios (OCR) (ratio of past maximum effective stress to current effective stress) ranging from about 4.1 to 7.5 down to a depths of about 10 feet. These fine-grained cohesive soils become slightly to moderately over-consolidated with depth, with OCR's ranging from about 1.3 to 1.6 from depths of about 50 to 105 feet. When loaded below the past maximum consolidation pressure, the fine-grained cohesive soils will exhibit relatively low compressibility characteristics.

Chemical tests were also conducted on selected samples to determine the aggressiveness of those soils with respect to concrete and buried metals. To determine if the site soils will react detrimentally with concrete, pH and soluble sulfates tests were performed on four representative samples of the site soils. The results of those tests indicate that the site soils are mildly to moderately alkaline and contain negligible amounts of water-soluble sulfates. Based on the above values, the potential of the site soils, particularly near-surface soils, to react detrimentally with concrete are considered to be negligible.

To determine if the site soils are corrosive to buried metals, resistivity tests were performed on four representative samples. The results of those tests indicate that the site soils have low to moderate resistivity values ranging from 769 to 2,040 ohm-centimeters. In general, soils with resistivity values less than 1,000 ohm-centimeters are considered to be severely corrosive to buried metals. Soils with resistivity values between 1,100 and 3,000 ohm-centimeters are considered to be moderately corrosive.

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7. STRUCTURES AND EMBANKMENTS

7.1 GENERAL

The following sections provide geotechnical recommendations for design of the bridge structures, including the bridge wing walls, and the mainline embankments, embankment retaining walls, and MSE wall systems along 12300 South Street and the Jordan and Salt Lake Canal. Note that recommendations related to the MSE wall systems are limited to allowable bearing pressure, settlement, and lateral resistance. We understand that a firm specializing in such structures will design the MSE walls.

7.2 BRIDGE FOUNDATION DESIGN CRITERIA

7.2.1 Pile Vertical Capacities

As required by the Project RFP, all bridge structure foundations will be supported on driven piles. The presence of near-surface zones of medium dense sands will result in moderate end-bearing capacities for piles terminated at depths of between 30 and 40 feet below current site grades. However, the tips of piles terminated within this relatively shallow depth zone will be almost immediately underlain by thick, moderately compressible cohesive soils. Piles (and pile groups) founded above the compressible cohesive soils are projected to experience moderate to high vertical settlements similar in order of magnitude to settlements that would be experienced by a shallow spread foundation system. Accordingly, we do not recommend founding the bridge structure on piles terminated above the compressible cohesive soils. We recommend driving the piles through the compressible cohesive soils and into the relatively dense, non-lacustrine sands encountered at depths of about 105 feet below current site grades.

We have calculated the vertical and lateral capacities of driven, steel, closed-end pipe with outside diameters of 12.75 and 16.0 inches. The calculated capacities are based on a top-of-pile elevation of about 4404 feet (about 10 feet below current ground surface). The vertical capacities are based on an installed center-to-center spacing equal to or greater than three times the pile diameter. The calculated ultimate vertical capacities for 12.75- and 16.0-inch diameter piles are summarized in the following table, along with the recommended maximum pile tip elevation.

Pile Diameter (inches)	Ultimate Vertical Capacities for Closed-End Pipe Piles (kips) Compressive	Uplift	Maximum Tip Elevation (ft)
12.75	625	360	4307
16.00	875	480	4307

The indicated ultimate uplift capacities do not include the weight of the piles.

Both UDOT and AREMA define factors of safety to be applied to the ultimate axial capacities to determine allowable compressive and uplift capacities. UDOT requires that the allowable axial capacities be determined in accordance with the factors of safety established by AASHTO. AASHTO has a sliding scale for compressive capacity that ranges from 3.5 to 1.9, depending on the confidence level with respect to capacity achieved through analysis and testing during and after pile installation. UDOT requires that at least one pile installation be monitored dynamically using a Pile Driving Analyzer (PDA), which will provide supplemental information that can be used to increase the confidence level. For uplift, AASHTO requires a factor of safety of 3 without a load test and 2 with a load test (provided actual capacity is achieved).

According to the AREMA Manual, the allowable axial compressive capacity is generally based on a factor of safety of 2. For uplift loads, the factor of safety is either 2 or 3 depending on the combination of loads being resisted.

We understand that a wave equation analysis will be completed prior to pile installation. Accordingly, we recommend using a factor of safety of 2.25 for compressive capacities and 3.0 for uplift, applied to the ultimate capacities. The factor of safety for uplift should not be applied to the weight of the piles.

In order to develop the appropriate compressive capacities, the piles must be driven at least one to two feet into the relatively dense, non-lacustrine sands (to a maximum pile tip elevation of 4307 feet), or to practical refusal, which is defined as a driving resistance of ten or more blows per inch of tip penetration. The maximum pile tip elevation recommended herein is based on a project ground surface elevation of 4414 feet at the deep boring locations. If the actual ground surface elevation is different, the maximum pile tip elevation must be revised accordingly.

7.2.2 Settlement

For piles installed as recommended above, the total settlement of individual piles and pile groups is estimated to be less than one-quarter inch. Differential settlement across the pile caps and between the abutments and the interior bent should be negligible.

7.2.3 Pile Lateral Capacities

Lateral capacities for closed-end, concrete-filled, steel pipe piles were calculated using the latest version of the software program "L-Pile," which can appropriately model a steel-concrete composite section. The table on the next page summarizes the calculated lateral capacities of 12.75-inch and 16.0-inch diameter piles for a range of pile deflections.

Deflection at Top of Pile (inches)	Ultimate Lateral Pile Capacity (kips)	
	12.75-Inch Diameter	16.0-Inch Diameter
1.0	73.0[1]	103.3[1]
0.5	47.7[1]	66.5[1]
0.25	31.4	43.0
Yield[2]	36.0	52.8

[1] Above yield
[2] Seismic

7.2.4 Pile Down Drag

Based on our embankment settlement analyses, a reduction for potential down drag forces due to embankment settlements is not warranted at this location. We do, however, recommend that the embankment construction be completed to the extent possible four to five weeks prior to pile installation.

7.3 RETAINING WALL DESIGN CRITERIA

7.3.1 General

Site retaining walls include the bridge abutment wing walls, embankment retaining walls along the east side of the mainline alignment, and MSE wall systems along 12300 South Street and the Jordan and Salt Lake Canal. We understand that the abutment wing walls, and possibly the embankment retaining walls, will be constructed as cast-in-place, concrete, cantilevered retaining walls supported on shallow spread footings. An alternate, proprietary retaining wall system, the T-Wall system, is also under consideration for the embankment retaining walls. The T-Wall system, if used, and the MSE wall systems along 12300 South Street are considered gravity structures that will be founded at shallow depths.

The abutment wing walls, the MSE walls, and the embankment wall at Willow Creek will be founded on near-surface natural soils. All other embankment walls will be founded on embankment fills.

7.3.2 Allowable Foundation Bearing Pressure

In evaluating the allowable bearing pressure for large shallow footings and MSE walls (wide bearing footprint) founded on natural soils, the controlling subsurface soils were considered to be the relatively stiff cohesive soils encountered within 40 feet of the existing ground surface. Based on laboratory testing, these cohesive soils possess minimum undrained shear strengths of at least 1,500 pound per square foot.

We recommend that shallow spread foundations and MSE walls founded on natural near-surface soils be designed using a maximum allowable bearing pressure of 2,900 pounds per square foot under real load conditions. This value includes a factor of safety of 3 against bearing capacity failure. This allowable bearing pressure may be increased by one-third for total load conditions (dead plus transient loads).

The embankment retaining walls will either be conventional cast-in-place concrete cantilevered walls or gravity-type walls constructed of pre-cast T-Walls (a proprietary, segmental wall system). In evaluating the allowable bearing pressures for the large (4 feet minimum width) spread footings supporting the cast-in-place walls, and the retained T-Wall soil block, the controlling subsurface soils were considered to be compacted embankment fills consisting of embankment for bridge fills (AASHTO A-1 material) within 300 feet of the railroad bridge abutments and on-site materials for embankments beyond the 300 foot zones. On-site materials are projected to consist of clays or clayey silts (AASHTO A-5 to A-7 materials) and cohesionless silty sands and sandy silts (AASHTO A-2 to A-4 materials).

Outside of the 300-foot bridge embankment zones, the type of compacted fills supporting the walls will vary and cannot be predicted. Accordingly, our recommendations are based on materials consisting of imported bridge embankment materials and on-site clay and clayey silt borrow. For walls supported on imported embankment for bridge fills, we recommend that the wall foundations with a minimum width of 4 feet and a minimum embedment of 30 inches be designed using a maximum allowable bearing pressure of 4,000 pounds per square foot. For walls supported on compacted on-site clay or clayey silt borrow, or borrow not meeting the requirements for embankment for bridge materials, we recommend that the wall foundations of similar minimum width and embedment be designed using a maximum allowable bearing pressure of 3,000 pounds per square foot. These allowable bearing pressures include a factor of safety of 3. These allowable bearing pressures are for real load conditions and may be increased by one-third for uniformly distributed total load conditions (dead plus transient loads), but not for increased edge pressures.

Note that the allowable bearing pressures for wall foundations on embankment fills include a reduction for the descending 2H:1V slope at the face of the walls. The toe of all retaining walls founded on slopes no steeper than 2H:1V must have a horizontal offset of at least 4 feet measured horizontally from the face of the footing toe to the finished slope. This offset applies regardless of supporting embankment fill material. Reduced bearing pressures can be provided for walls founded closer to the face of the slope, if requested. Alternately, the depth of the footing may be extended downward.

The following table summarizes the allowable bearing pressures recommended for walls associated with the railroad bridge and embankments, and cut retention along 12300 South Street at the crossing. As stated above, the allowable bearing pressures for wall footings are based on a minimum footing width of four feet.

Wall Location	Bearing Soil	Maximum Allowable Bearing Pressure (psf)	Controlling Conditions
Abutment wing walls and embankment walls at Willow Creek	Undisturbed, relatively stiff, natural cohesive soils	2,900	Relatively large footing width; natural subgrade soils
MSE walls along 12300 South and the Jordan and Salt Lake Canal	Undisturbed, relatively stiff, natural cohesive soils	2,900	Relatively wide footprint; natural subgrade soils
Embankment retaining walls	Compacted AASHTO A-1 material extending to undisturbed natural soils	4,000	Footing widths of four feet or more; minimum four-foot horizontal off-set from face of slope; bearing reduction due to 2H:1V slope below footing
Embankment retaining walls	Compacted common borrow	3,000	Footing widths of four feet or more; minimum four-foot horizontal off-set from face of slope; bearing reduction due to 2H:1V slope below footing

7.3.3 Minimum Embedment

For frost protection, the base of the footings for cast-in-place walls must be embedded at least 30 inches below the lowest adjacent finished grade. A minimum base embedment for frost protection is not required for the MSE walls.

7.3.4 Settlement

Based on a maximum uniform bearing pressure of 2,900 pounds per square foot, we estimate the maximum settlement along the bridge wing walls will be about one and three-quarter inches. This settlement includes the effect of the wing wall backfill associated with the embankment fills. Differential settlements along the wing walls should be less than one-half of the total settlements.

Total and differential settlements along the embankment walls will be governed primarily by the overall embankment settlements, which are discussed in Section 8.3, Embankment Settlements, of this report. Minor additional settlement of the embankment walls is also expected to occur due to consolidation of the underlying fill under the wall footing loads.

Maximum MSE wall settlements are estimated to vary from about one-quarter of an inch to one inch for wall heights varying from about 4 feet to 12 feet. These settlements do not include additional settlement that could result from the loads applied by the adjacent railroad bridge approach embankment fills. This range in settlements should allow for single-stage MSE wall construction.

Based on the over-consolidated condition of the natural cohesive soils underlying the alignment, stresses due to the embankment and retaining wall loads are not expected to exceed the past consolidation pressures of those soils. Accordingly, wall settlements will be elastic and most of the expected settlements should be basically complete within four to five weeks after construction of embankments and walls.

7.3.5 Lateral Resistance

For footings and MSE embankment walls established directly on undisturbed natural cohesive soils, we recommend using a coefficient of 0.40 for determining base sliding lateral resistance. For footings established on at least two feet of compacted granular borrow, we recommend using a coefficient of 0.50 for determining base sliding lateral resistance. For MSE walls and embankment T-Walls established on at least two feet of compacted granular borrow, we recommend using a coefficient of 0.65 for determining base sliding lateral resistance. The base-sliding coefficient for the MSE walls and T-Walls is based on a maximum internal friction angle of 34 degrees for the granular borrow and the fill used to backfill the T-Walls and construct the MSE reinforced fill.

If used in combination with resistance from passive earth pressures, the base-sliding coefficient must be reduced by a factor of safety of 1.5. Note that passive earth pressures cannot be used with retaining walls established on descending embankment slopes.

7.3.6 Foundation Installation

The footings for the abutment wing walls and the embankment wall at Willow Creek, and the MSE walls, may be established directly upon undisturbed natural cohesive soils or on compacted granular backfill borrow extending to undisturbed natural cohesive soils. The embankment retaining walls may be established directly upon properly placed and compacted embankment fills.

Under no circumstances can footings or gravity walls (MSE walls and T-Walls) be installed over non-engineered fills or upon soft, wet, or disturbed soils, construction debris, frozen soil, or within ponded water. If natural cohesive soils upon which footings or gravity walls are to be established become soft, wet, or disturbed, they must be removed to firm, undisturbed natural cohesive soils and replaced with compacted granular backfill borrow. If compacted cohesive embankment fills upon which footings or gravity walls are to be established become soft, wet, or disturbed, they must be removed to firm, undisturbed fills or natural soils and be replaced with compacted embankment fill. If granular backfill borrow or embankment for bridge fills upon which the footings or gravity walls are to be established become disturbed, they must be re-compacted to the appropriate requirements.

The width of the zone of granular backfill borrow placed below footings and MSE walls should extend laterally at least six inches beyond the edges of the footings, or beyond the MSE wall footprint, in all directions for each foot of fill placed beneath the footings or MSE walls. For example, if the width of a footing is two feet and the thickness of the structural fill zone beneath the footing is one foot, the structural fill zone would extend six inches beyond the edges of the footing for a total width of three feet.

The natural, fine-grained, cohesive subgrade soils may degrade significantly during site preparation activities, especially where the base of the excavation is at or near the groundwater level and during wet periods of the year. Where saturated or nearly saturated, the fine-grained subgrade soils will be easily disturbed during placement of forms and reinforcing steel, or other construction activities. Fine-grained embankment fills will also be easily disturbed under similar circumstances. Where these conditions occur, soft or disturbed subgrade materials should be carefully removed under observation by qualified geotechnical personnel. To limit construction-related disturbance of exposed fine-grained, natural subgrade soils or fine-grained embankment fills, particularly if saturated or nearly saturated, or where the work will occur during wet weather, we recommend removing those soils to a depth of at least four inches and replacing those soils with compacted granular stabilizing fill and/or granular backfill borrow.

7.4 LATERAL EARTH PRESSURES

The abutments and the abutment wing walls will retain backfill placed between the structures and the temporary excavation cuts, as well as fill for the approach embankments. We understand that the abutments will be unrestrained even after the bridge girders are installed. Accordingly, the abutments would be able to rotate away from the fill, resulting in an active earth pressure state. If for some reason the abutments are unrestrained before they are backfilled, the abutments would be subject to an at-rest earth pressure state. Cast-in-place embankment retaining walls will be able to rotate away from the fill, and MSE walls and T-Walls will be able to translate away from retained fills. Accordingly, those walls will be subjected to active earth pressures.

Given the anticipated thickness of fills to be retained by the abutments, the abutment wing walls, and the embankment retaining walls, all wall backfill must be compacted to a relatively high degree of compaction to limit self-weight settlement of the backfill during its service life. UDOT requires that all backfill placed within about 300 feet of the bridge structure consist of an A-1 material (per AASHTO M-145) compacted to at least 96 percent of the maximum dry density as determined by the AASHTO T-180 test procedure. Relatively high compaction is also expected for fills outside of those 300-foot zones. Compaction of wall backfill to these criteria will result in very high lateral earth pressures, particularly for the at-rest state.

To limit lateral earth pressures on the abutments and other retaining walls, we recommend that the backfill placed zone immediately behind the abutments and other retaining walls, as described in the following paragraph, consist of a select granular material meeting the requirements of Section 9.3.3, Select Wall Backfill, of this report. The select wall backfill could consist of a Type 1 backfill as described in the AREMA *Manual for Railway Engineering* (2002) if that material meets the general criteria of Section 9.3.3, Select Wall Backfill, of this report.

The select wall backfill zone immediately behind the abutments and other retaining walls should be roughly defined by the back of the abutment or each retaining wall and an imaginary line extending upward from the base of the abutment or retaining wall stem. The line should extend upward at an angle of 30 degrees measured from the vertical back of the abutment or retaining wall stem. In addition, this select wall backfill zone must have a minimum horizontal width (perpendicular to the back of the wall) of ten feet at the abutments and three feet for all other retaining walls. The three-foot wide zone should extend upward from the top of the footing to the intersection with the 30-degree line.

To limit lateral earth pressures due to compaction, it is essential that this select wall backfill zone be compacted in thin lifts of six inches or less to the required dry density using light, hand-operated compaction equipment. Compaction of each loose lift should begin at the back of the abutment or wall and proceed away from the back of the abutment or wall.

In evaluating active and at-rest lateral earth pressure parameters, we have assumed that the select backfill material behind the abutments and retaining walls will control and have assigned that backfill material a moist unit weight of 130 pounds per cubic foot and an internal friction angle of 38 degrees. In evaluating passive lateral earth pressure parameters, we have assumed that the soils controlling passive resistance will consist of granular backfill borrow or general embankment fills because of the length of the passive wedge. Accordingly, we have assigned the resisting soils a moist unit weight of 130 pounds per cubic foot and an internal friction angle of 34 degrees.

The table on the next page lists the lateral earth pressure criteria for both static and seismic conditions. The seismic lateral earth pressures are based on USGS horizontal ground acceleration values of 0.30 and 0.52, which correspond to average ground motion return periods of 475 and 2,475 years. The acceleration values were reduced by 50 percent to obtain the horizontal coefficients used in the calculations. These horizontal coefficients include amplification for a Site Class D soil profile.

	Equivalent Fluid Density (pcf) Based on Mapped USGS Acceleration Values		
Load Condition	**Static**	**475-Year Event**	**2,475-Year Event**
Active (free to rotate)	31	38[1]	49[1]
At-Rest (restrained)	50	39[2]	68[2]
Passive	460[3]	486[4]	345[4]

[1] Total value Including static equivalent fluid density.
[2] Seismic value only. Must be added to static value.
[3] Factor of safety of 1.5
[4] Factor of safety of 1.0

The following table lists the lateral earth pressure criteria for static conditions and seismic conditions based on the AREMA modified horizontal ground acceleration coefficients of 0.10, 0.26, and 0.46, which correspond to average ground motion return periods of 95, 373, and 1,980 years, respectively. These acceleration coefficients include amplification for a Site Class D soil profile.

	Equivalent Fluid Density (pcf) AREMA Modified Acceleration Values			
Load Condition	**Static**	**95-Year Event**	**373-Year Event**	**1,980-Year Event**
Active (free to rotate)	31	31[1]	36[1]	45[1]
At-Rest (restrained)	50	13[2]	32[2]	57[2]
Passive	460[3]	620[4]	513[4]	383[4]

[1] Total value Including static equivalent fluid density
[2] Seismic value only. Must be added to static value.
[3] Factor of safety of 1.5
[4] Factor of safety of 1.0

It must be reiterated that the lateral earth pressure criteria provided in the preceding tables do not include compaction induced horizontal stresses. These criteria apply only for materials meeting the requirements of Section 9.3.3, Select Wall Backfill, of this report, and which are placed and compacted as described in this section. If materials other than those described above are used as backfill immediately adjacent to the abutments or other retaining walls, or the backfill materials are placed and compacted differently than recommended herein, the above equivalent fluid densities must be modified. Modified equivalent fluid densities could be significantly higher.

It should be noted that the equivalent fluid density values provided above are based on the assumption that the backfill materials will not become saturated. The equivalent fluid density values may be decreased by 50 percent if the backfill becomes saturated. However, full hydrostatic water pressures will have to be included, and hydro-dynamic pressures may need to be considered.

In determining the lateral earth pressures acting on the bridge structures and other retaining walls, we recommend the following approaches:

1. Static active earth pressures alone are determined using the above static equivalent fluid density value and are applied using a triangular distribution that increases with depth below top of wall. The total static active force is applied at a point located one-third of the height of the soil retained above the base of the wall.

2. Seismic active earth pressures alone are applied using an "inverted" triangular distribution that decreases with depth below the top of the wall. The total seismic active force is applied at a point above the base of the wall equivalent to six-tenths the height of the retained soil.

3. The total active seismic earth pressure is the summation of the static and seismic components. The total static and seismic active earth pressures may also be approximated using the above seismic equivalent fluid densities applied using a uniform distribution based on one-half the height of the wall times the seismic equivalent fluid density. The total active force is then applied at a point above the base of the wall equivalent to one-half the height of the retained soil.

4. The total at rest seismic earth pressure is the summation of the static and seismic components. At-rest static and seismic earth pressures are determined using the above equivalent fluid densities. The total static at-rest force is determined and applied using a distribution appropriate for braced conditions. The total seismic at-rest force is determined using an inverted triangular pressure distribution that decreases with depth. The maximum pressure is based on the height of the wall times the seismic equivalent fluid density. The total seismic at-rest force is applied at a point above the base of the wall equivalent to six-tenths one-half the height of the retained soil.

5. The passive static and seismic earth pressures can be determined using the above equivalent fluid densities and are both applied using a normal triangular pressure distribution.

7.5 GLOBAL STABILITY OF RETAINING WALLS

Generalized global stability analyses were conducted for selected embankment and MSE wall configurations utilizing the microcomputer program WINSTABL (Version 2.08). Conditions analyzed included short-term static (construction), long-term static, and seismic. The modified Bishop's method for a circular failure surface was used in the analyses.

Embankment wall configurations that were analyzed included a typical wall established at the crest of the embankment fill slope and the approximately 22-foot high wall planned at the Willow Creek crossing. For the embankment walls, the groundwater level was considered to be ten feet below the base of the embankment fill. The following embankment cross sections were included in the analyses.

Cross Section Station	Wall Height	Wall Location	Subgrade Conditions
161+00	8	Top of slope	Embankment fill
176+89	22	Vertical transition	Natural soils

The global stability analyses of the MSE walls along 12300 South Street were based on a 12-foot wall height and a 7-foot deep reinforced soil zone, with level ground surfaces above and in front of the wall. The groundwater table for the MSE wall was considered to be at the base of the wall.

The soil parameters on the next page were used for the analyses. The long-term strength parameters for the natural soils are based on the assumption that the soils consist of cohesive soils or predominantly cohesive soil mixtures of relatively low plasticity. The long-term strength parameters for the embankment fills are based on the assumption that the fills possess relatively high internal friction angles or consist of mixtures of granular and cohesive soils of relatively low plasticity.

Soil	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)	Short-Term Strength Parameters (Construction/Seismic) Internal Friction Angle (degrees)	Short-Term Strength Parameters (Construction/Seismic) Cohesion (psf)	Long-Term Strength Parameters (Drained) Internal Friction Angle (degrees)	Long-Term Strength Parameters (Drained) Cohesion (psf)
Natural Cohesive	120	125	0	1,500	31	200
Embankment Fill[1]	115	120	34	100	34	100
MSE Retained fill[2]	125	125	30	1,500	31	200

[1] Common borrow considered to be the controlling fill material. Unit weights are conservative.
[2] Retained soils are assumed to be natural cohesive soils.

Global stability of the embankment and MSE walls under seismic conditions was analyzed for both the 10 percent and 2 percent in 50-year events (475-year and 2,475-year return periods). A pseudo-static approach was used to analyze seismic stability, with a pseudo-static coefficient of 0.15 for the 10 percent in 50-year event and a pseudo-static coefficient of 0.26 for the 2 percent in 50-year event. It should be noted that the pseudo-static coefficient is not the true horizontal ground motion acceleration. A value of 50 percent of the horizontal ground acceleration is commonly used to represent the pseudo-static coefficient.

The following table summarizes the results of our embankment and MSE wall stability analyses, along with required minimum factors of safety defined by UDOT.

Wall Location/Case	Factor of Safety: Short-term static (construction)	Long-term static	10-percent in 50-year seismic event	2-percent in 50-year seismic event
Station 161+00	10.4	10.1	7.2	5.9
Station 172+89	2.6	2.1	1.9	Not Required
MSE – 12300 South	5.8	2.1	3.5	2.7
Minimum UDOT (No impact to/impact to bridge abutments)	1.3/1.1	1.5/1.5	1.0/1.0	1.1/1.1

The calculated global stability factors of safety for static and seismic load conditions are considered acceptable for the embankment and MSE walls.

8. EMBANKMENTS

8.1 GENERAL

As previously described in Section 1.2, Project Description, of this report, embankments will be constructed to support the shoofly track and mainline track structures. Construction of the shoofly will require embankment fills ranging in height from a few feet to as much as about 17 feet at the Willow Creek crossing. Construction of the mainline track will require wedge fills as high as about 37 feet high at Corner Canyon and as high as about 17 feet at or near the Willow Creek crossing. The approach embankments to the bridge will require fills ranging in height from a few feet to as much as 22 feet at the Willow Creek crossing. Note that these embankment fill heights are based on the top of the track sub-ballast and do not include the railroad track ballast, which we understand will be about two feet thick.

Based on discussions with the design-build team, the shoofly fill embankment side slopes will range from as steep as one and one-half horizontal to one vertical (1.5H:1V) to two horizontal to one vertical (2H:1V), with the 1.5H:1V preferred. The mainline fill embankment side slopes will be constructed at 2H:1V. Due to right-of-way constraints, an approximately 500-foot long embankment fill section at Corner Canyon will be constructed with slopes as steep as about 1.5H:1V.

As discussed in the previous sections of this report, right-of-way constraints dictate that retaining walls will be required along segments of the east side of the mainline fill embankments extending from about 800 feet south of 12300 South Street to almost 2,800 feet north of 12300 South. These retaining walls will generally be founded on the 2H:1V embankment fill side slopes.

The shoofly alignment will cross at least three irrigation ditches requiring installation of temporary culverts or siphon piping, and will cross over existing 36-inch and 48-inch diameter reinforced concrete pipes at Willow Creek. The mainline track alignment crosses the existing Willow Creek culvert pipes (78-inch and 84-inch diameter steel pipes).

8.2 EMBANKMENT STABILITY

Stability analyses of the planned mainline embankments were performed for typical sections of the embankments utilizing the microcomputer program WINSTABL (Version 2.08). The modified Bishop's method for a circular failure surface was used within the program. For the embankment stability analyses, the groundwater level was considered to be ten feet below the base of the embankment fill. The embankment cross sections on the next page were used in the analyses.

Cross Section Station	Embankment Height	Embankment Slope Inclination	Embankment Conditions
97+00	37	1.5H:1V	Wedge fill on existing 2H:1V embankment slope
161+00	18	2.0H:1V	Typical embankment; level ground at toe
172+00	12	Vertical Wall	Embankment toe on descending natural slope

The following soil parameters were used for the analyses. The assumptions made in assigning long-term strength parameters are similar to those described in Section 7.5, Global Stability with Retaining Walls, of this report.

			Short-Term Strength Parameters (Construction/Seismic)		Long-Term Strength Parameters (Drained)	
Soil	**Moist Unit Weight (pcf)**	**Saturated Unit Weight (pcf)**	**Internal Friction Angle (degrees)**	**Cohesion (psf)**	**Internal Friction Angle (degrees)**	**Cohesion (psf)**
Natural Cohesive	120	125	0	1,500	31	200
Embankment fill[1]	115	120	0	600	31	200
Embankment Fill[2]	115	120	34	100	34	100

[1] Clays and clayey silts, sands, and gravels. Unit weights are conservative

[2] Common borrow considered to be the controlling fill material. Unit weights are conservative.

Global stability of the embankments under seismic conditions was analyzed for the 10 percent in 50-year event (475-year return period). A pseudo-static approach was used to analyze seismic stability, with a pseudo-static coefficient of 0.15 for the 10 percent in 50-year event. It should be noted that the pseudo-static coefficient is not the true horizontal ground motion acceleration. A value of 50 percent of the horizontal ground acceleration is commonly used to represent the pseudo-static coefficient.

Embankment Location/Case	Factor of Safety: Short-term static (construction)	Long-term static	10-percent in 50-year seismic event
Station 97+00*	1.9	1.7*	1.5*
Station 161+00	2.1	2.1	1.5
Station 172+00	2.6	1.6	1.9
Minimum UDOT - No impact to/impact to bridge abutments	1.3/1.1	1.3	1.0

* New wedge fill to consist of clays or clayey silts, sands, and gravels meeting the requirement of Section 9.3.4. Minimum horizontal fill thickness of 10 feet from toe to crest.

The calculated global stability factors of safety for static and seismic load conditions are considered acceptable for the embankment and MSE walls. Note that the stability of 1.5H:1.0V side slopes along the shoofly embankments will be similar to that of the mainline fill slope at Station 97+00 (Corner Canyon) provided the outer portion of the shoofly slopes are constructed of similar clayey material meeting the requirements of Section 9.3.4, Embankment Side Slope Fills, of this report. These side slope fill materials must be placed to a minimum horizontal thickness of ten feet on all slopes steeper than 2.0H:1.0V.

8.3 EMBANKMENT SETTLEMENT

Settlement analyses were conducted for several typical embankment cross sections along the railroad mainline alignment. For each cross section, the settlement at the base of the fill (natural ground surface) was calculated at points directly below the toe of the embankment, the crest of the embankment side slope, and the center of the embankment.

The following embankment cross sections were used in the settlement analyses:

Cross Section Station	Embankment Height	Embankment Conditions
139+00	5	Side fill to existing embankment
147+00	11	Symmetrical embankment over existing 4-foot embankment
153+00	17	Symmetrical embankment over existing 4-foot embankment
161+00	18	Near-symmetrical embankment over existing 4-foot embankment; 8-foot wall on east side
163+15	16	Near-symmetrical embankment at bridge south abutment; 6-foot wall on east side

164+90	15	Symmetrical embankment at bridge north abutment
172+00	12	Embankment over existing 3-foot embankment; vertical wall on east side; toe on descending natural slope
176+89	22	Embankment at Willow Creek culvert over existing 15-foot embankment; vertical wall on east side
178+00	25	Symmetrical embankment over existing 15-foot embankment

Embankment loads were based on a moist unit weight for the new and existing fill of 125 pounds per cubic foot. The following table summarizes the results of our settlement analyses.

	Maximum Calculated Settlement (inches)			
Cross Section Station	**Toe of 2H:1V Fill Slope**	**Crest of Fill Slope**	**Centerline of Embankment**	**Below Fill Retaining Wall**
139+00	3/8 to 1/2	1/2 to 3/4	NA	NA
147+00	1/2 to 3/4	1-3/8 to 1-3/4	1-1/8 to 1-1/2	NA
153+00	3/4 to 1	2-1/8 to 2-1/2	1-3/4 to 2	NA
161+00	7/8 to 1	2-1/4 to 2-1/2	2 to 2-1/4	1-7/8 to 2
163+15	7/8 to 1	2-1/8 to 2-1/2	2-1/4 to 2-1/2	1-7/8 to 2
164+90	3/4 to 1	2 to 2-1/4	2-1/4 to 2-1/2	NA
172+00	3/4 to 1	1-3/4 to 2	2-1/8 to 2-1/2	NA
176+89	7/8 to 1	2-3/8 to 2-1/2	1-3/8 to 1-1/2	1-1/2 to 1-3/4
178+00	7/8 to 1	2-1/4 to 2-1/2	1-1/8 to 1-1/2	NA

Note that settlements calculated at Stations 163+15 and 164+90 do not include "end-of-fill" effects, which are expected to reduce the settlement magnitudes at the abutments from those indicated in the table.

To evaluate the impact of the embankment on adjacent developed property, ground surface settlements were also calculated for points beyond the east railroad right-of-way at Station 163+15. Settlements calculated for points located 5, 15, and 25 feet east of the right-of-way were five-eighths to seven-eighths, one-half to five-eighths, and three-eighths to one-half inch, respectively.

Based on the results of our consolidation testing at the bridge site, and settlement data from construction of the existing I-15 embankments, settlements are projected to be within the elastic range. Accordingly, secondary settlement is not considered to be an issue, and surcharging will not be required. Almost all of the embankment settlements are expected to be complete within four to five weeks after construction of the embankments.

9. EARTHWORK

9.1 SITE PREPARATION

Site preparation will include demolition of existing pavements, track structures, and other miscellaneous structures, removal of existing vegetation, and stripping of topsoil and any other deleterious materials encountered. All demolition debris, vegetation, topsoil, and any other deleterious materials encountered must be totally removed from beneath planned structures and embankment fills. Existing track embankment fills may be left in place. Demolition debris and vegetation, and any other deleterious materials encountered must be removed from the site. Topsoil may be stockpiled for shoofly surface restoration and reuse along the mainline alignment and other areas.

Following removal of demolition debris, vegetation, and topsoil, all subgrade areas supporting retaining walls or receiving new embankment fills should be proof-rolled and recompacted in accordance with UDOT requirements. To avoid disturbance of potentially wet and soft subgrade soils, excavation to planned subgrade should be accomplished using smooth-lipped equipment. Placement of granular backfill borrow, and stabilizing fill, if necessary, should proceed immediately after excavation is complete to avoid unnecessary subgrade disturbance.

9.2 TEMPORARY EXCAVATIONS

Excavations will be required to construct portions of the shoofly and mainline, to lower 12300 South Street at the crossing, to bench in new embankment fills, and to install utilities, the abutment pile caps, the abutment wing wall footings, and the MSE walls. Excavations to depths of up to 20 feet or more are projected to primarily encounter undisturbed, natural, fine-grained, cohesive soils. The undisturbed, natural, cohesive soils possess relatively high strengths, and temporary excavations within these soils are expected to remain stable even when cut at vertical or nearly vertical slopes.

Temporary construction excavations in the undisturbed, natural, cohesive soils not exceeding 4 feet in depth and not overlain by existing fills may be constructed with near-vertical side slopes. Excavations up to 15 feet in depth in undisturbed, natural, cohesive soils may be constructed with side slopes no steeper than one-half horizontal to one vertical (0.5H:1.0V). Excavations up to 20 feet in depth in undisturbed, natural, cohesive soils may be constructed with side slopes no steeper than three-quarters horizontal to one vertical (0.75H:1.0V).

Excavations extending deeper than ten feet below existing grades are expected to encounter saturated conditions and layers and zones of water bearing cohesionless granular soils. If cohesionless granular soils and groundwater are encountered, significantly flatter side slopes, shoring and bracing, and/or dewatering will be required. Groundwater seepage, if present, is expected to be moderate and manageable using typical diversion ditches, small local sumps, and portable pumping equipment.

Qualified personnel must inspect all excavations periodically. If any signs of instability or excessive sloughing are noted, immediate remedial action must be initiated.

9.3 STRUCTURAL FILL MATERIALS AND COMPACTION

9.3.1 General

In general, structural fill materials and their placement must conform to UDOT specifications. We anticipate that most fill materials used at the bridge abutments or to construct the approach embankment segments located within 300 feet of the bridge abutments will conform to UDOT's requirements for granular backfill borrow or embankment for bridge. Fill materials used to construct embankment segments located more than 300 feet from the bridge abutments are expected to consist of common fill per UDOT's specifications. However, fills used to stabilize soft or saturated subgrade soils, for select wall backfill materials, or to construct the outer portion of embankment slopes steeper than 2H:1V must meet the following requirements.

9.3.2 Stabilizing Fill

Stabilizing fill would be used to stabilize soft subgrade conditions or where structural fill is required below a level one foot above the water table at the time of construction. Stabilizing fill should consist of a mixture of coarse gravels and cobbles or a gap-graded, angular, 1.5 to 2.0-inch-minus gravel. Stabilizing fill, if utilized, should be end-dumped, spread to a maximum loose lift thickness of 15.0 inches, and compacted by dropping a backhoe bucket onto the surface continuously at least twice. As an alternative, the stabilizing fill may be compacted by at least two passes of moderately heavy construction equipment or large self-propelled compaction equipment. Subsequent fill materials placed over coarse stabilizing fills should be adequately compacted so that the finer fraction of the overlying fills is "worked into" the voids in the underlying coarser stabilizing fills.

9.3.3 Select Wall Backfill

Select structural backfill placed immediately behind the temporary abutment walls must meet UDOT's general criteria for free-draining granular backfill borrow. In addition to UDOT's requirements, the backfill material must be angular to sub-angular, with zero percent material by weight finer than 0.075 millimeter. The select wall backfill could consist of a Type 1 backfill as described in the AREMA Manual for Railway Engineering (2002) if that material meets the preceding criteria.

The select wall backfill zone immediately behind the abutments and other retaining walls should be roughly defined by the back of the abutment or each retaining wall and an imaginary line extending upward from the base of the abutment or retaining wall stem. The line should extend upward at an angle of 30 degrees measured from the vertical back of the abutment or retaining wall stem. In addition, this select wall backfill zone must have a minimum horizontal thickness of ten feet at the abutments and three feet for all other retaining walls. The three foot wide zone should extend upward from the top of the footing to the intersection with the 30-degree line.

Select backfill placed within the above-described backfill zones should be placed in maximum six-inch thick loose lifts and compacted to a non-yielding condition using light, hand-operated compaction equipment. Compaction of each lift must begin at and progress away from the back of the abutment or retaining walls.

9.3.4 Embankment Side Slope Fills

Material placed in the outer ten feet of embankment side slopes steeper then 2H:1V must consist of a clay or clayey material meeting UDOT's general criteria for common clay fill.

10. EXISTING UTILITIES/CULVERTS

We anticipate that most of the utilities along 12300 South Street will be relocated or replaced as part of construction. Deep remaining utilities will be subjected to minor settlements induced by the approach embankments.

Existing culverts at Corner Canyon and Willow Creek will remain in place. The Corner Canyon culvert is projected to experience a minor increase in earth pressures and a minor amount of embankment related settlement. The Willow Creek culvert pipes will experience a moderate to significant increase in overburden pressure and the following range of estimated settlements:

Location	Settlement (inches)
Toe of West Side Slope	7/8 to 1
Crest of West Side Slope	2-3/8 to 2-1/2
Centerline of New Embankment	1-3/8 to 1-1/2
Retaining Wall (East Side of Embankment)	1-1/2 to 1-5/8
Right-of-way Line	3/4 to 7/8

We understand that the embankment retaining wall at Willow Creek will bridge the existing culvert pipes. It is preferred to bring the wall footing (currently to be established at an elevation near or at that of the top of the pipes) as close to the north and south sides of the pipe zone as possible.

To limit stresses on the existing culvert pipes due to the retaining wall foundation bearing pressures, we recommend locating the bottom edge of the toe of each abutment footing such that the springline of the pipes are located above an imaginary, 45-degree line extending downward and outward from the closest bottom edge of the wall footing. If this placement cannot be achieved without lowering the entire footing, we recommend using a narrow (12- to 24-inch wide), down-turned edge along the face of the footing to extend the bottom of the footing to a sufficient depth. Note that this approach may have practical structural limitations on the depth to which the down-turned edge may extend. The structural engineer must determine those limitations, if they exist. If the footings are positioned as recommended, the pipes should experience maximum footing-related pressures of between 5 and 10 percent of the actual footing bearing pressure.

11. PROFESSIONAL STATEMENTS

Supporting data upon which our discussions and recommendations are based are presented in preceding sections and Appendices of this report. Recommendations presented herein are governed by the physical properties of the soils encountered in the subsurface explorations, projected groundwater conditions, and the layout and design data discussed in Section 1.2, Project Description, of this report. If subsurface conditions other than those described in this report are encountered and/or if design and layout changes are implemented, AMEC must be informed so that our recommendations can be reviewed and amended, if necessary.

Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices in use locally at this time.

We appreciate the opportunity of providing this service for you. If you have any questions or require additional information, please do not hesitate to contact us.

Respectfully submitted,

AMEC Earth & Environmental, Inc.

J. Wade Gilbert, State of Utah No. 367656
Sr. Geotechnical Engineer

Reviewed By:

William J. Gordon, State of Utah No. 146417
Professional Engineer

JWG/:ka

Encl. Figure 1, Vicinity Map
Figure 2a, Exploration Location Plan Key
Figure 2b, Corner Canyon
Figure 2c, Bridge Structure and Retaining Walls
Figure 3, Bridge Site Plan
Appendix A, Field Explorations and Instrumentation
Appendix B, Explorations by Others
Appendix C, Laboratory Testing
Appendix D, Cone Penetration Test (CPT) Data

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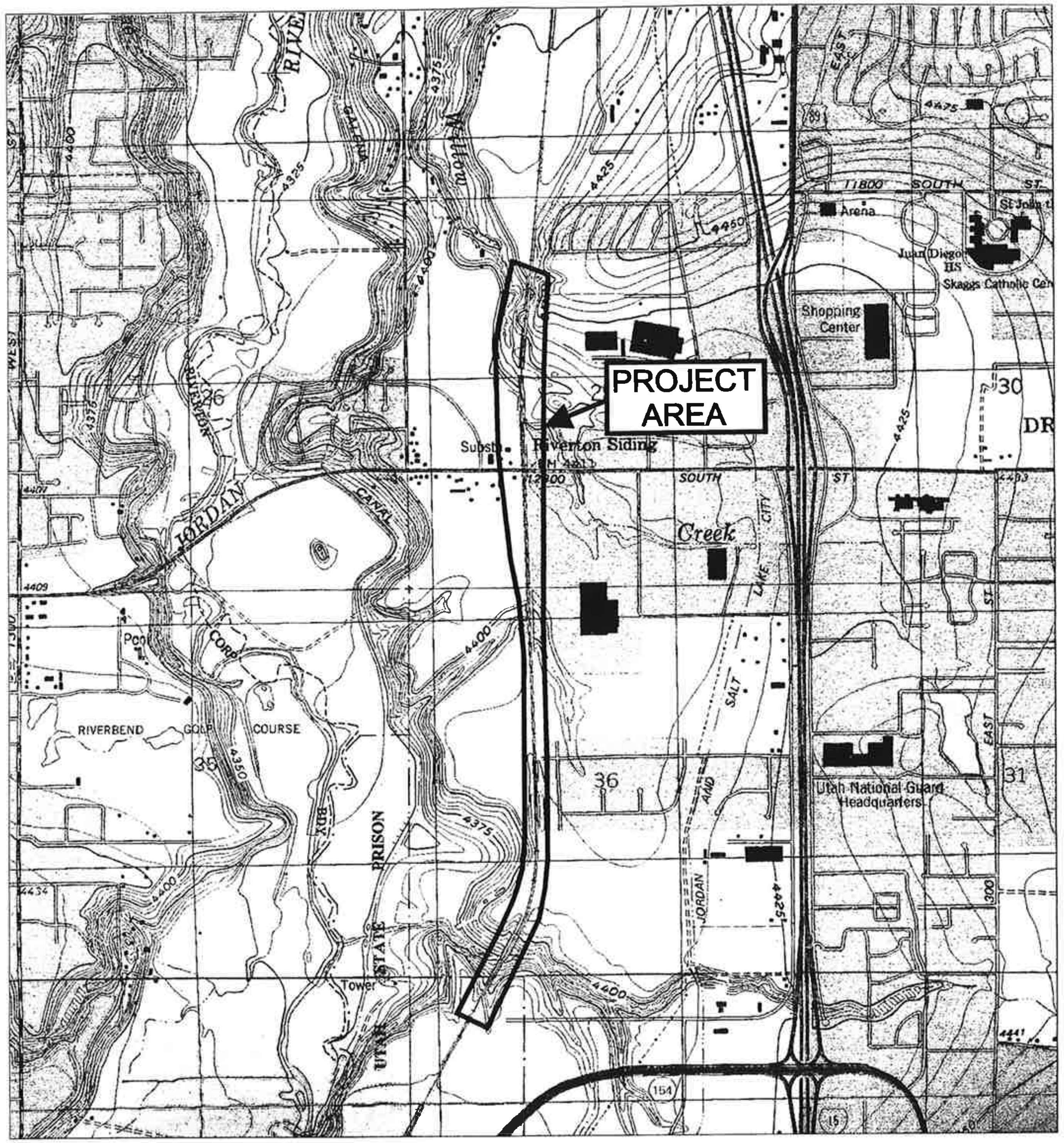
STATE OF UTAH

12300 SOUTH DESIGN-BUILD PROJECT *HPP-STP-0071(12)0
AMEC JOB NO. 2-817-004066 (4062)



SCALE IN FEET
1000 0 1000 2000

FIGURE 1
VICINITY MAP

REFERENCE:
USGS 7.5 MINUTE TOPOGRAPHIC QUADRANGLE MAP
TITLED "MIDVALE, UTAH"
DATED 1999

12300 SOUTH DESIGN-BUILD PROJECT *HPP-STP-0071(12)0
AMEC JOB NO. 2-817-004066(4062)

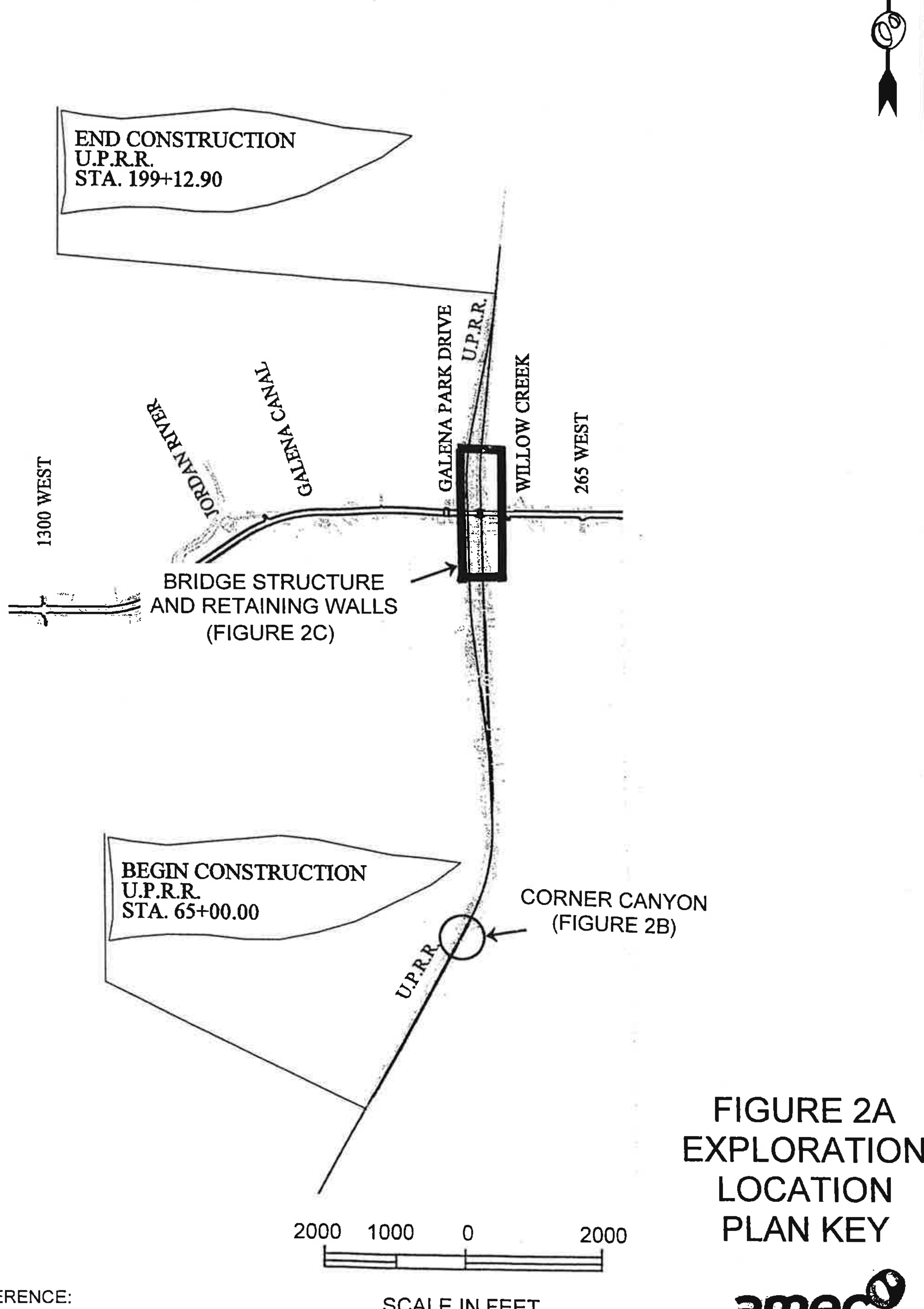


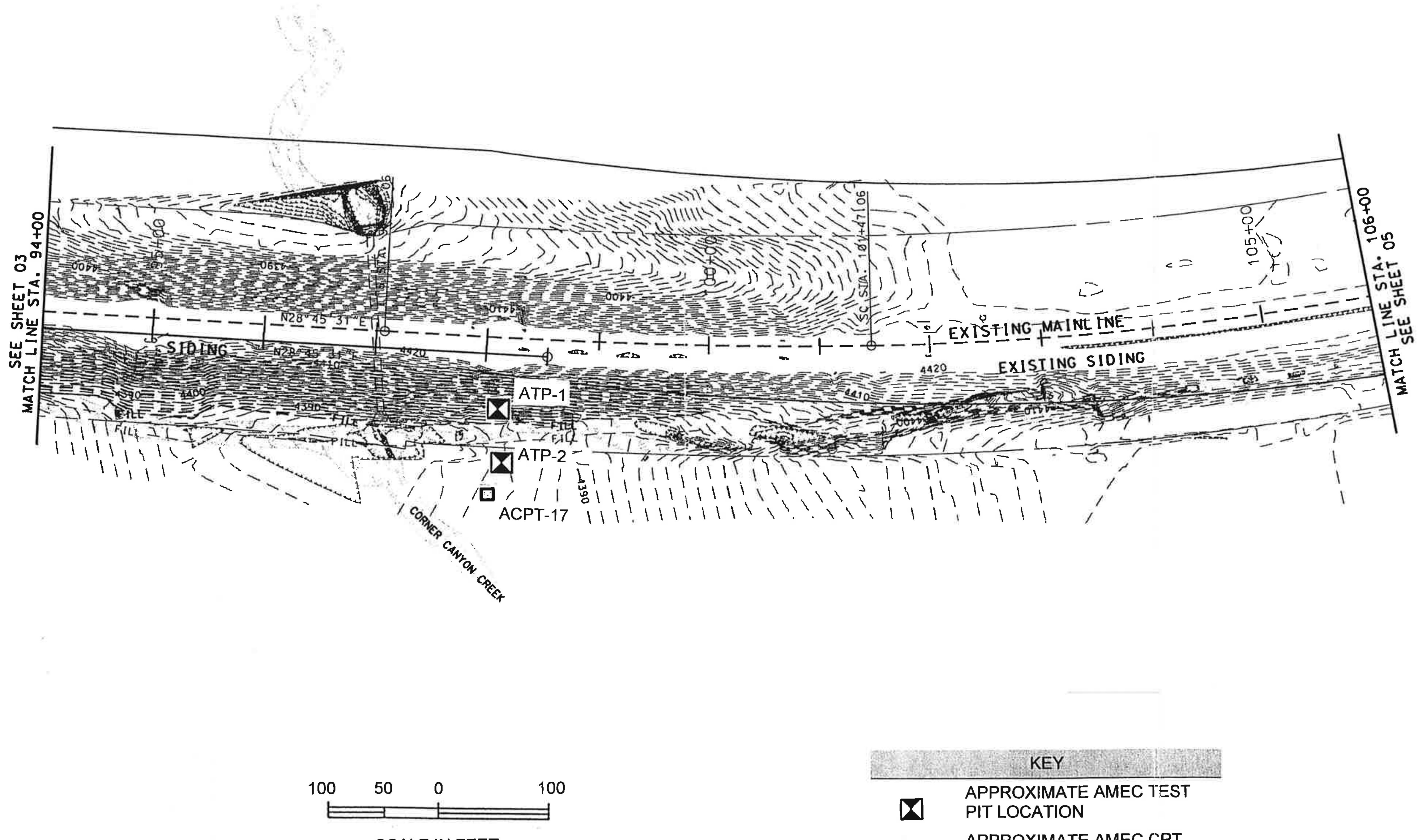
FIGURE 2A
EXPLORATION
LOCATION
PLAN KEY

amec

REFERENCE:
ADAPTED FROM DRAWING
PROVIDED BY CLIENT

N073408

12300 SOUTH DESIGN-BUILD PROJECT *HHP-STP-0071(12)0
AMEC JOB NO. 2-817-004066(4062)



KEY

Symbol	Description
☒	APPROXIMATE AMEC TEST PIT LOCATION
▫	APPROXIMATE AMEC CPT PROBE LOCATION

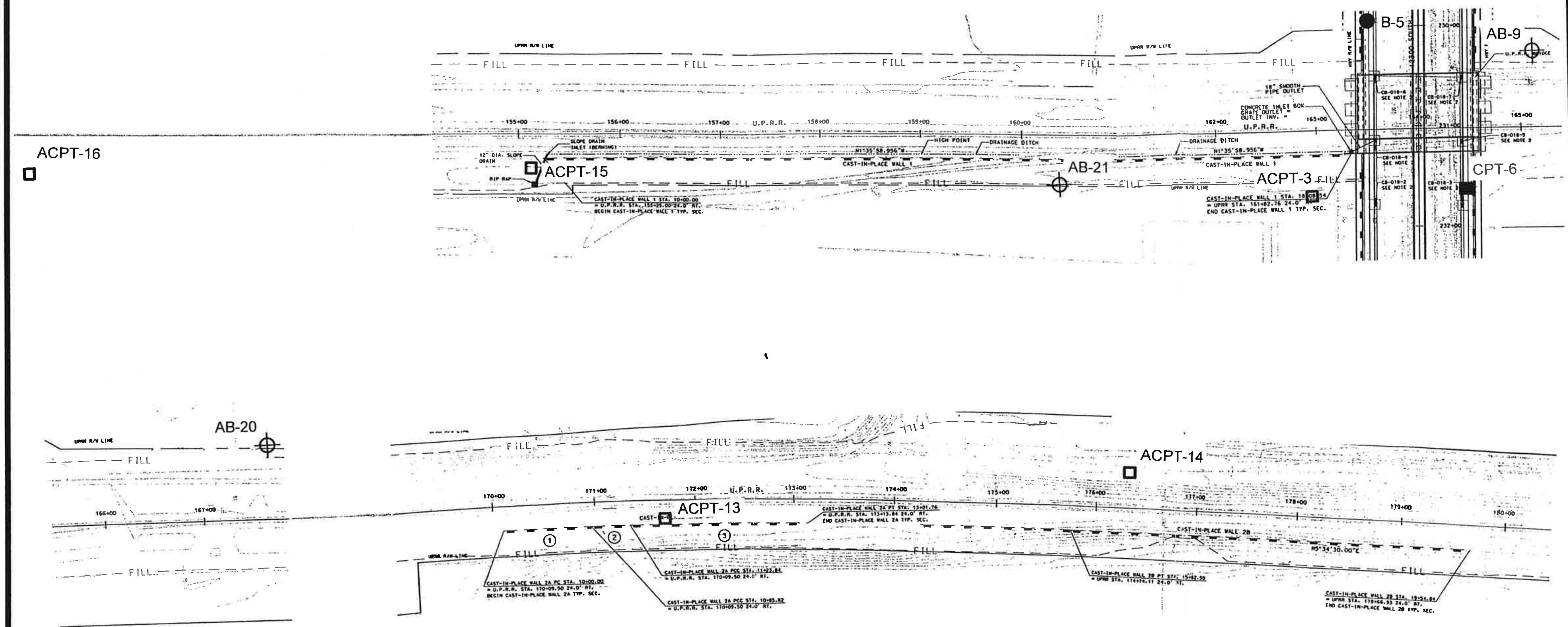
FIGURE 2B
CORNER CANYON

amec

REFERENCE:
ADAPTED FROM DRAWING
PROVIDED BY UDOT

N06854

12300 SOUTH DESIGN-BUILD PROJECT *HHP-STP-0071(12)0
AMEC JOB NO. 2-817-004066(4062)



100 50 0 100

SCALE IN FEET

KEY	
⌖	APPROXIMATE AMEC BORING LOCATION
□	APPROXIMATE AMEC CPT PROBE LOCATION
●	APPROXIMATE LOCATION PREVIOUS BORING
■	APPROXIMATE LOCATION PREVIOUS CPT PROBE

FIGURE 2C BRIDGE STRUCTURE AND RETAINING WALLS

amec

REFERENCE:
ADAPTED FROM DRAWING
PROVIDED BY UDOT

N068544

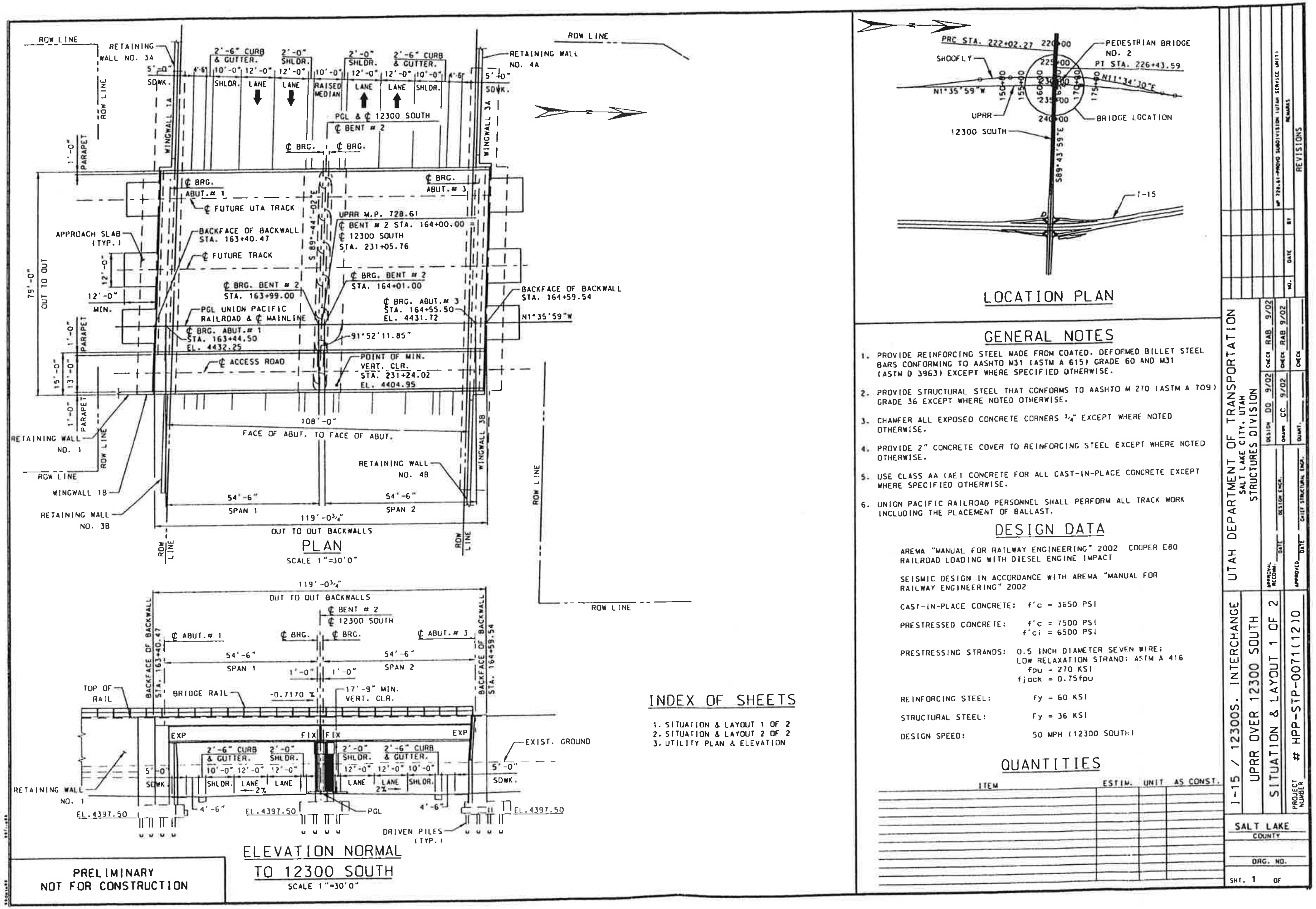


FIGURE 3
BRIDGE
SITE PLAN

REFERENCE:
DRAWING PROVIDED
BY UDOT

N068544

APPENDIX A

Field Explorations and Instrumentation

APPENDIX A
FIELD EXPLORATIONS AND INSTRUMENTATION

1. FIELD EXPLORATIONS

Subsurface soil and groundwater conditions along the mainline alignment were explored by drilling three borings (AB-9, AB-20, and AB-21) to depths ranging from about 30.5 to 115.5 feet below existing site grades, advancing five CPT probes (ACPT-13 through ACPT-17) to depths of about 30.0 feet each, and advancing one CPT probe (ACPT-3) to refusal at a depth of about 108.0 feet. In addition, two test pits (ATP-1 and ATP-2) were excavated at the base of the existing embankment at Corner Canyon to depths of 6.0 and 18.0 feet, respectively.

Firms under subcontract to AMEC conducted the explorations. L & L Drilling advanced the borings using a truck-mounted, Diedrich D120 drill rig equipped with hollow-stem augers. ConeTec, Inc. conducted the CPT probes. Skyline Contractors, using a JCB 214S all-wheel drive backhoe equipped with an extended boom, excavated the test pits. The approximate locations of the explorations are indicated on Figure 2B, Corner Canyon, and Figure 2C, Bridge Structure and Retaining Walls.

The field portion of our study was under the direct control and continual supervision of an experienced member of our geotechnical staff. Our representative coordinated and monitored the drilling and excavating activities, and the installation of piezometers in all three borings. Our representative maintained a continuous log of the subsurface conditions encountered at each boring and test pit location and obtained representative samples of the soils encountered in the borings and test pits for subsequent laboratory testing and examination.

Relatively undisturbed samples of the typical soils encountered in the borings were obtained at 2.5- or 5.0-foot intervals using a D&M split-barrel sampler of the type illustrated on Figure A-1. The sampler was driven into the undisturbed soil ahead of the auger bit with a 140-lb, automatic drop hammer falling a distance of 30 inches. The number of blows required to drive the sampler for the final foot of soil penetration, or part thereof, is noted on the boring logs adjacent to the appropriate sample notation.

The soils encountered in the borings and test pits were classified in the field based upon visual and textural examination. These classifications were supplemented by subsequent examination and testing in our laboratory. Soils were classified in general accordance with ASTM D-2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure)*. Detailed graphical representations of the subsurface conditions encountered at the boring locations are presented on Figures A-2 through A-4, Log of Test Borings, and A-5 and A-6, Log of Test Pits. The exploration logs represent our interpretation of the field logs and the results of our laboratory classification testing. Figure A-7, Unified Soil Classification System, provides a key to the soil descriptions on the logs. The CPT logs and associated data are presented in Appendix D.

The explorations were located in the field by hand taping or pacing from existing physical features. The ground surface elevations shown on the boring and test pit logs were determined by superimposing the exploration locations on a map of site topographic contours provided by UDOT in the Project RFP. Both the exploration locations indicated on Figures 2B and 2C and the ground surface elevations indicated on the boring and test pit logs should be considered approximate.

2. INSTRUMENTATION

2.1 PIEZOMETERS

Following completion of drilling operations, one and one-quarter-inch diameter slotted PVC pipes were installed to depths of about 20 feet in Borings AB-9, AB-20, and AB-21 to provide a means of monitoring groundwater fluctuations.

Stabilized groundwater levels were measured in the piezometers on three separate dates after installation. The results of the groundwater level measurements are summarized in the following table along with the corresponding approximate groundwater elevations (based on available ground surface elevation data provided in the RFP).

Stabilized Groundwater Levels (ft)

	AB-9		AB-20		AB-21	
Date	Depth	Elevation	Depth	Elevation	Depth	Elevation
August 8, 2002	11.6	4402	10.5	4404	10.2	4405
September 7, 2002	10.5	4404	10.9	4403	10.7	4404
October 22, 2002	13.5	4401	14.2	4400	11.8	4403

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER

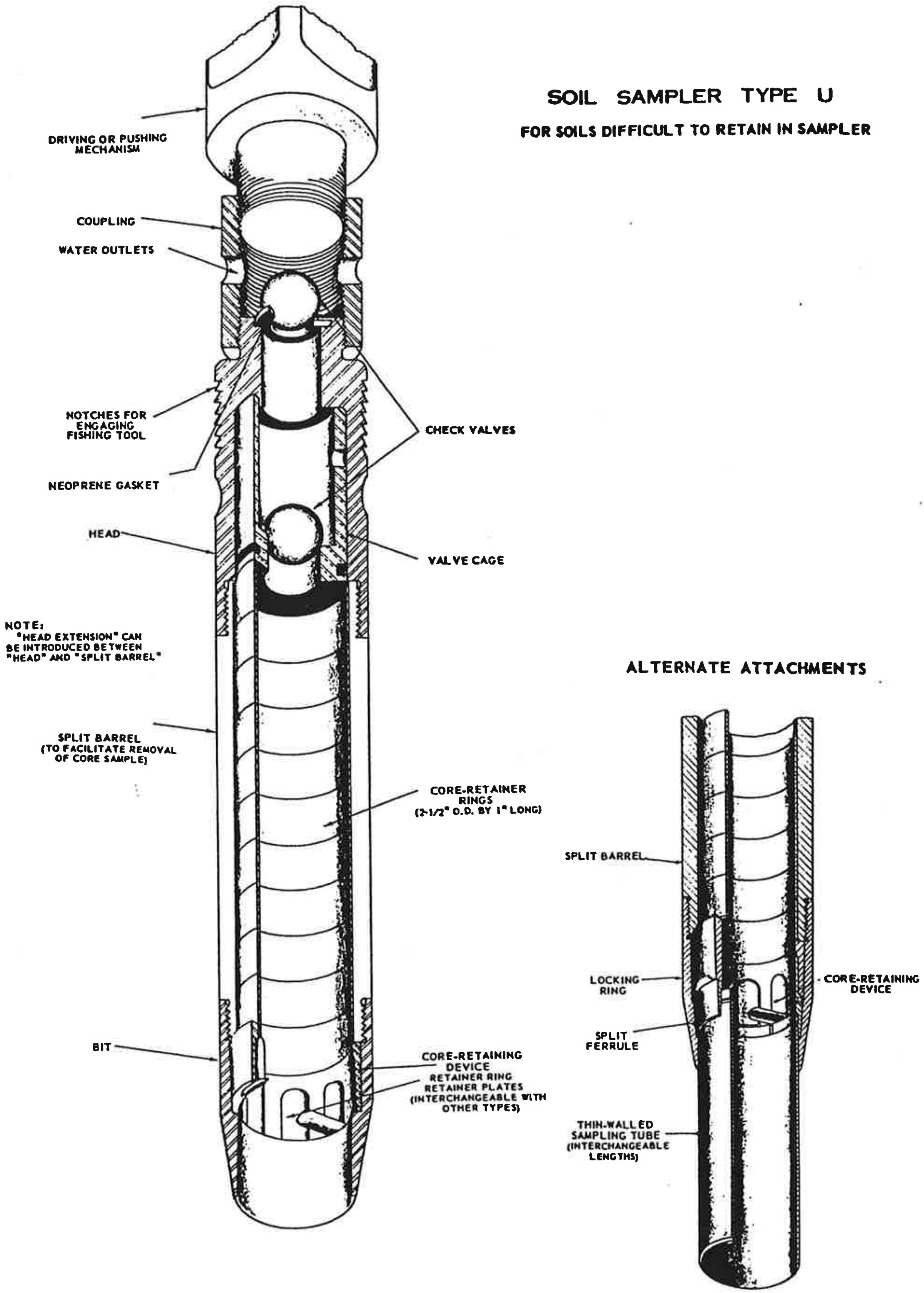


FIGURE A-1

PROJECT 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT

LOG OF TEST BORING NO. AB-9

JOB NO. 2-817-004066 DATE 08-01-02

RIG TYPE Diedrich D120 (L&L)
BORING TYPE 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV. 4414'+/-
LOGGED BY Matt Gallegos

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0								GP/FILL		**COARSE GRAVEL**; loose; FILL
								SM/ ML FILL	moist, very loose	**SILTY FINE SAND/SANDY SILT**; dark brown; FILL
				D	10					
								ML	moist, loose	**FINE SANDY SILT** with trace clay; brown
5				D	21	84	31.7			grades brown-gray/gray-brown with rust-brown mottles
10				D	10	80	37.9		medium stiff	grades to clayey silt with fine sand and occasional silty clay layers to 1" thick and silty fine sand seams to 1/8" thick
								CL/ ML/ SM	very moist to saturated, very stiff/ medium dense	**LAYERED SILTY CLAY, CLAYEY SILT, AND SILTY FINE SAND**; 1" to 2" layers; light brown to gray-brown
15				D	29					
								SM	saturated, medium dense	**SILTY FINE TO MEDIUM SAND**; brown
20				D	29		24.1			grades with occasional clayey silt and silty clay layers 1/4" to 1" thick
25										

GROUNDWATER

	DEPTH	HOUR	DATE
▽	11.6	9:40	08-08-02
▼	13.5		10-22-02

SAMPLE TYPE
A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-2

amec

PROJECT 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT

LOG OF TEST BORING NO. AB-9

JOB NO. 2-817-004066 DATE 08-01-02

RIG TYPE Diedrich D120 (L&L)
BORING TYPE 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV. 4414'+/-
LOGGED BY Matt Gallegos

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
25				D	32	105	21.4			grades with less silt
								CL/ ML/ SP	saturated, medium dense/ very stiff	**LAYERED SILTY CLAY/CLAYEY SILT/FINE SAND**; layers to thicker than 6"; brown
30				D	50					grades to 1" to 3" layers with fine to medium sand layers and occasional silty fine sand layers to 1" thick
								SM/ ML	saturated, medium dense	**SILTY FINE SAND/SANDY SILT** with occasional clayey silt/silty clay layers to 1/2" thick; brown
35				D	53	107	21.7			
40				D	60	103	21.1			grades gray
								CL/ ML	saturated, very stiff	**SILTY CLAY/CLAYEY SILT** with fine sand and occasional silty fine sand layers to 3" thick; gray to dark gray with black mottles
45				D	22					
50										grades without sand layers

GROUNDWATER

	DEPTH	HOUR	DATE
▽	11.6	9:40	08-08-02
▼	13.5		10-22-02

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-2
(con't)

amec

PROJECT 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT

LOG OF TEST BORING NO. AB-9

JOB NO. 2-817-004066 DATE 08-01-02

RIG TYPE Diedrich D120 (L&L)
BORING TYPE 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV. 4414'+/-
LOGGED BY Matt Gallegos

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
50				D	23	91	31.1			grades dark gray to black
								ML	saturated, stiff	**CLAYEY SILT** with fine sand and occasional fine sandy silt layers to 1" thick; gray to dark gray
55				D	20	102	23.2			
60				D	15					
								ML/ CL	saturated, stiff	**CLAYEY SILT/SILTY CLAY** with fine sand and numerous clayey silt layers 1/4" to 1/2" thick; gray to dark gray with black mottles
65				D	16	88	33.9			grades dark gray and black
								ML	saturated, very stiff	**CLAYEY SILT** with fine sand and numerous silty fine sand layers 1/4" to 1" thick; gray to dark gray
70				D	35				stiff, layered drilling	
									softer drilling	
75										

GROUNDWATER

	DEPTH	HOUR	DATE
▽	11.6	9:40	08-08-02
▼	13.5		10-22-02

SAMPLE TYPE
A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-2
(con't)

amec

PROJECT 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT

LOG OF TEST BORING NO. AB-9

JOB NO. 2-817-004066 DATE 08-01-02

RIG TYPE Diedrich D120 (L&L)
BORING TYPE 4-1/4" ID Hollow-Stem Auger
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LOGGED BY Matt Gallegos

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
75				D	34	98	26.0			grades to occasional silty fine sand and silty clay layers 1/4" to 1" thick
80				D	17	86	41.2		stiff	grades with occasional fine to coarse sand seams and layers 1/8" to 1/4" thick
85				D	21	86	33.2			grades without sand layers
								CL	saturated, very stiff	**SILTY CLAY** with fine sand; dark brown-gray
90				D	27	86	35.6			
95				D	30					
								ML	saturated, very stiff	**CLAYEY SILT** with some fine sand; brown-gray
100										

GROUNDWATER

	DEPTH	HOUR	DATE
▽	11.6	9:40	08-08-02
▼	13.5		10-22-02

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-2
(con't)

amec

PROJECT 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT

LOG OF TEST BORING NO. AB-9

JOB NO. 2-817-004066 DATE 08-01-02

RIG TYPE Diedrich D120 (L&L)
BORING TYPE 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV. 4414'+/-
LOGGED BY Matt Gallegos

Depth in Feet	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
100	D	44	96	25.6			
105	D	154					
			107	22.5	SP/ SM	saturated, very dense	**FINE SAND** with silt; gray-brown
110	D	100/4"					
115	D	100/5"					
120							
125							

Stopped drilling at 115.0'.

Stopped sampling at 115.5'.

Installed 1-1/4" diameter slotted PVC pipe to 20.0'.

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary to a proper understanding of the nature of the subsurface materials.

GROUNDWATER

DEPTH	HOUR	DATE
11.6	9:40	08-08-02
13.5		10-22-02

SAMPLE TYPE
A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-2
(con't)

amec

LOG OF TEST BORING NO. AB-20

PROJECT 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT

JOB NO. 2-817-004066 DATE 08-05-02

RIG TYPE Diedrich D120 (L&L)
BORING TYPE 4-1/4" Hollow-Stem Auger
SURFACE ELEV. 4414'+/-
LOGGED BY Matt Gallegos

Depth in Feet	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0					SM/ GM FILL	moist, "loose"	**SILTY SAND AND GRAVEL**; sawdust at surface; fine to coarse sand; fine and coarse gravel; no roots or topsoil; brown; FILL
	D	11					
					SM/ ML	moist, loose/ medium stiff	**LAYERED SILTY SAND AND CLAYEY AND SANDY SILT**; fine sand; 1" to 2" thick layers; brown
5					CL	moist, stiff	**SILTY CLAY** with fine sand; brown to brown-gray
	D	12	85	30.6			
	D	7	76	40.9		very moist, medium stiff	grades with clayey silt zones; gray to brown-gray with brown mottles
10							
					CL/ ML/ SM	saturated, stiff/loose	**LAYERED CLAYEY SILT, SILTY CLAY, AND SILTY FINE TO MEDIUM SAND**; 1/2" to 2" thick layers; brown/gray/rust-brown
	D	20					
15							
					SM	saturated, medium dense	**SILTY FINE TO MEDIUM SAND**; brown
	D	54		20.2			
20					CL/ ML/ SM	saturated, very stiff/ medium dense	**LAYERED SILTY CLAY, CLAYEY SILT, AND SILTY FINE SAND**; brown-gray
	D	26					
25							

GROUNDWATER

DEPTH	HOUR	DATE
10.5		08-08-02
14.2		10-22-02

SAMPLE TYPE
A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-3

amec

LOG OF TEST BORING NO. AB-20

PROJECT 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT

JOB NO. 2-817-004066 DATE 08-05-02

RIG TYPE Diedrich D120 (L&L)
BORING TYPE 4-1/4" Hollow-Stem Auger
SURFACE ELEV. 4414'+/-
LOGGED BY Matt Gallegos

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
25										
				D	50					
30										
										Stopped drilling at 29.0'. Stopped sampling at 30.5'. Installed 1-1/4" diameter slotted PVC pipe to 20.0'.
35										
40										
45										
50										The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary to a proper understanding of the nature of the subsurface materials.

GROUNDWATER

DEPTH	HOUR	DATE
10.5		08-08-02
14.2		10-22-02

SAMPLE TYPE
A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-3
(con't)

amec

PROJECT 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT

LOG OF TEST BORING NO. AB-21

JOB NO. 2-817-004066 DATE 08-05-02

RIG TYPE Diedrich D120 (L&L)
BORING TYPE 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV. 4415'+/-
LOGGED BY Matt Gallegos

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0								SM/ GM FILL	moist, "loose"	**SILTY SAND AND GRAVEL**; fine to coarse sand; fine and coarse gravel; brown; FILL
								ML	moist to very moist, soft	**CLAYEY SILT** with some fine sand; brown to gray-brown
5										
				D	4	89	32.9			
				D	4	89	29.7			
10										
								CL/ SP	saturated, very stiff/ medium dense	**LAYERED SILTY CLAY WITH FINE SAND AND FINE TO MEDIUM SAND** with occasional fine sandy silt seams and layers to 1/4" thick; major sand layers 4" to 6" thick; clay mottled brown and gray; sand brown
				D	50	99	25.4			
15										
				D	27					grades with numerous clayey silt layers to 1" thick; red-brown
20										
								SP/ SM	saturated, loose	**FINE TO MEDIUM SAND** with some silt; brown
				D	25				no sample recovery	
25										

GROUNDWATER

	DEPTH	HOUR	DATE
▽	10.2		08-08-02
▼	11.8		10-22-02

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-4

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LOG OF TEST BORING NO. AB-21

PROJECT: 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT

JOB NO. 2-817-004066 DATE 08-05-02

RIG TYPE: Diedrich D120 (L&L)
BORING TYPE: 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV.: 4415'+/-
LOGGED BY: Matt Gallegos

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
25										
				D	50					
30						100	22.1	SM/ML	moist, medium dense	**SILTY SAND/SANDY SILT** with some clay; fine sand; olive-gray/brown
35										
40										
45										
50										

Stopped drilling at 29.0'.

Stopped sampling at 30.5'.

Installed 1-1/4" diameter slotted PVC pipe to 20.0'.

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary to a proper understanding of the nature of the subsurface materials.

GROUNDWATER

	DEPTH	HOUR	DATE
▽	10.2		08-08-02
▼	11.8		10-22-02

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-4 (con't)

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LOG OF TEST PIT NO. ATP-1

PROJECT 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT

JOB NO. 2-817-004066 DATE 10-30-02

GROUNDWATER

DEPTH	HOUR	DATE
	*	

BACKHOE TYPE JCB 214S (Skyline)
LOCATION Corner Canyon
ELEVATION
LOGGED BY Matt Gallegos

Depth in Feet	Graphical Log	Sample	Sample Type	Moisture in % / Dry Density Pcf	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0					ML FILL	moist, "medium dense"	**FINE SANDY SILT**; loose top 6" to 12"; numerous pinholes to 2.0'; brown; EMBANKMENT FILL
			TW	19.0/ 78.0			
			D		ML/ SM/ CL FILL	moist, "medium dense"/ "stiff"	**LAYERED FINE SANDY SILT (BROWN), SILTY FINE TO MEDIUM SAND (BLACK), AND SILTY CLAY (GRAY)**; 1" to 3" thick; EMBANKMENT FILL
5					CL	moist, "stiff"	**SILTY CLAY**; gray
							Stopped excavation at 6.0'.
							No sidewall caving.
							* Groundwater not encountered.
10							
15							
20							
							The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary to a proper understanding of the nature of the subsurface materials.
25							

SAMPLE TYPE
B - Bucket Sample
D - Disturbed Bulk Sample
SW - Sidewall Sample
TW - Thinwall Sample

FIGURE A-5

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PROJECT 12300 South Design-Build Project
12300 South UPRR Crossing, Draper, UT
JOB NO. 2-817-004066 DATE 10-30-02

LOG OF TEST PIT NO. ATP-2

GROUNDWATER

	DEPTH	HOUR	DATE
▽	11.0		10-30-02
▼			

BACKHOE TYPE JCB 214S (Skyline)
LOCATION Corner Canyon
ELEVATION
LOGGED BY Matt Gallegos

Depth in Feet	Sample Type	Moisture in % / Dry Density Pcf	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0	D		ML FILL	slightly moist, "stiff"	**CLAYEY AND FINE SANDY SILT** with pockets of native clay and tree debris; disturbed to 12"; brown; FILL
			SP	moist, "medium dense"	**FINE TO COARSE SAND**; stream channel deposits; brown
5	TW	20.3/ 100.0	CL/ ML	moist, "stiff"	**MASSIVE AND LAYERED SILTY CLAY AND CLAYEY SILT** with some fine sand; 2" to 4" thick fine sand dike cutting diagonally through test pit; gray and light brown
10 ▽	D			very moist to saturated	grades to blocky clay in 2" to 8" layers with occasional to numerous fine to medium sand layers 1/2" to 1" thick
				moist	grades without sand layers
15	D				
20					Stopped excavation at 18.0'. No sidewall caving. The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary to a proper understanding of the nature of the subsurface materials.
25					

SAMPLE TYPE
B - Bucket Sample
D - Disturbed Bulk Sample
SW - Sidewall Sample
TW - Thinwall Sample

FIGURE A-6

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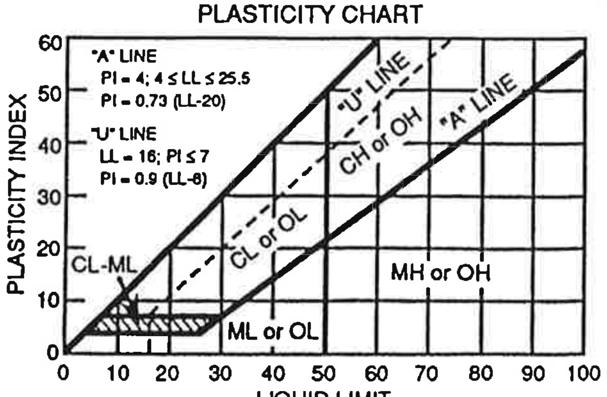
UNIFIED SOIL CLASSIFICATION SYSTEM

Soils are visually classified for engineering purposes by the Unified Soil Classification System. Grain-size analyses and Atterberg Limits tests often are performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. Graphic symbols are used on boring logs presented in this report. For a more detailed description of the system, see "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)" ASTM Designation: 2488-84 and "Standard Test Method for Classification of Soils for Engineering Purposes" ASTM Designation: 2487-85.

MAJOR DIVISIONS				GRAPHIC SYMBOL	GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (50% or less of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)			GW	Well graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures
					GP	Poorly graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		GM	Silty gravels, gravel-sand-silt mixtures
			Limits plot above "A" line & hatched zone on plasticity chart		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)			SW	Well graded sands, gravelly sands
					SP	Poorly graded sands, gravelly sands
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		SM	Silty sands, sand-silt mixtures
			Limits plot above "A" line & hatched zone on plasticity chart		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS Limits plot below "A" line & hatched zone on plasticity chart	SILTS OF LOW PLASTICITY (Liquid Limit less than 50)			ML	Inorganic silts, clayey silts of low to medium plasticity
		SILTS OF HIGH PLASTICITY (Liquid Limit 50 or more)			MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
	CLAYS Limits plot above "A" line & hatched zone on plasticity chart	CLAYS OF LOW PLASTICITY (Liquid Limit less than 50)			CL	Inorganic clays of low to medium plasticity, gravelly, sandy, and silty clays
		CLAYS OF HIGH PLASTICITY (Liquid Limit 50 or more)			CH	Inorganic clays of high plasticity, fat clays, sandy clays of high plasticity
	ORGANIC SILTS AND CLAYS	ORGANIC SILTS AND CLAYS OF LOW PLASTICITY (Liquid Limit less than 50)			OL	Organic silts and clays of low to medium plasticity, sandy organic silts and clays
		ORGANIC SILTS AND CLAYS OF HIGH PLASTICITY (Liquid Limit 50 or more)			OH	Organic silts and clays of high plasticity, sandy organic silts and clays
ORGANIC SOILS		PRIMARILY ORGANIC MATTER (dark in color and organic odor)			PT	Peat

NOTE: Coarse-grained soils with between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone on the plasticity chart have dual classifications.

PLASTICITY CHART



DEFINITION OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Boulders	Above 12 in.
Cobbles	12 in. to 3 in.
Gravel	3 in. to No. 4 sieve
Coarse gravel	3 in. to 3/4 in.
Fine gravel	3/4 in. to No. 4 sieve
Sand	No. 4 to No. 200 sieve
Coarse sand	No. 4 to No. 10 sieve
Medium sand	No. 10 to No. 40 sieve
Fine sand	No. 40 to No. 200 sieve
Fines (silt and clay)	Less than No. 200 sieve

FIGURE A-7

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APPENDIX B

Explorations by Others

APPENDIX B
EXPLORATIONS BY OTHERS

This appendix provides logs of explorations conducted at the site by others. These explorations include one soil boring, B-5, and one cone-penetration test (CPT) sounding, CPT-6, completed at the existing railroad crossing for UDOT by Kleinfelder in 2002. The approximate locations of these explorations are shown on Figure 2C, Bridge Structure and Retaining Walls. These exploration logs are provided for reference only.

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4137 South 500 West
Salt Lake City, Utah 84123
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12300 South Street Improvements
700 East to Bangerter Highway

KLEINFELDER

Project No. 13894-85D

FIELD TEST BORING LOG

Boring: **B-5**

Sheet 1 of 3

Boring: B-5
Sheet 1 of 3

Elevation (ft)	SAMPLE DESCRIPTION * (ASTM D 2488)	Depth (ft)	Type	Recovery (in)	Blows per 6 in. (N)	USCS	AASHTO	Moisture, %	% Passing No. 200
	Lean CLAY - stiff, moist, brown	0				CL	A-6		
4410									
		5	SPT	13	4 5 6				
4405									
	Fat CLAY - soft, moist, brown	10	SPT	15	1 1 2	CH	A-7-6		
4400									
	Silty SAND - medium dense, wet, gray	15	SPT	16	3 10 10	SM	A-2-4		
4395 ∇									
		20	SPT	10	3 4 16				
4390									
		25	SPT	10	2 9 6			19	46
4385									
	Sandy SILT - very stiff, wet, gray, fine grain, with frequent clay seams and layers	30	SPT	9	3 6 12	ML	A-4	20	70
4380									
		35	SPT	13	7 10 13			24	54
4375									
	Lean CLAY - soft to medium stiff, wet, gray with black	40	SPT	18	1 1 1	CL	A-6		
4370									
		45	SPT	18	1 2 3				
4365									
		50	SPT	18	1 1 3				
4360									
		55	SPT	18	1 3 5				
4355									
		60	SPT	18	1 0 1				
4350									
		65	SPT	18	1 2 2				
4345									

Other column headings: Graphic Log; SAMPLE; Soil Classification *; ● SPT (N) Blows/ft; ○ SPT (N) Value greater than 50 blows/ft (scale 0, 25, 50); Test Results: S_u, psf (torvane in italics); Dry Density, lb/ft^3; Liquid Limit; Plasticity Index; Other Tests

Logged by:	M. Ivers
Date Start:	3/29/02
Date Finish:	4/2/01
Northing (ft):	7,362,671.3
Easting (ft):	1,528,207.3
Elevation (ft):	4,412.9
Total Depth Drilled (ft):	145.0
Drill Contractor:	ConeTec
Driller:	B. Mercur
Rig Type:	MARL M10
Drilling Method:	Hollow-Stem Auger
Hammer Type:	Automatic
Rod Type:	AWJ
Boring Diameter:	8 inches

LEGEND/NOTES

Elevations based upon North American Vertical Datum of 1988 (NAVD '88)

Coordinates are NAD '83

∇ = Observed Groundwater depth at time of drilling

Blows = Number of blows required to drive split spoon sampler 6 inches or distance shown

USCS = Unified Soil Classification System

AASHTO = American Association of State Highway and Transportation Officials

* = Classification based on visual-manual observations only (ASTM D 2488)

SAMPLE TYPE

SPT = Standard Penetration Test, 1.375 in. ID and 2 in. OD split spoon sampler

MCAL = Modified California Sampler, 2 in. ID and 2.5 in. OD split spoon sampler

P = Piston Sampler, 3 in. OD

SH = Shelby Tube, 3 in. OD, pushed

BAG = Bulk Sample

UTDOT ENGLISH 35022032.GPJ 6/3/02

12300 South Street Improvements
700 East to Bangerter Highway
KLEINFELDER
Project No. 13894-85D

FIELD TEST BORING LOG

Boring: **B-5**

Sheet 2 of 3

Boring: B-5
Sheet 2 of 3

Elevation (ft)	SAMPLE DESCRIPTION * (ASTM D 2488)	Depth (ft)	Type	Recovery (in)	Blows per 6 in. (N)	USCS	AASHTO
	- some sand Lean CLAY - soft to medium stiff, wet, gray with black (continued)		SPT	18	2 2 4		
4340							
		75	SPT	18	1 1 2		
4335							
		80	SPT	18	2 2 3		
4330							
		85	SPT	18	2 3 3		
4325							
		90	SPT	18	2 3 5		
4320							
		95	SPT	11	3 5 6		
4315							
	SILT - hard, wet, gray, with frequent sand	100	SPT	18	8 12 23	ML	A-4
4310							
		105	SPT	18	8 18 20		
4305							
	Silty SAND - loose to very dense, wet, gray	110	SPT	18	6 23 27	SM	A-2-4
4300							
		115	SPT	18	3 3 4		
4295							
		120	SPT	10	13 50/5"		
4290							
		125	SPT	5	50/5"		
4285							
		130					
4280							
	- Gravel (Crew drilled to 145 feet to recover sampler.)	135			2 50/2"		
4275							

● SPT (N) Blows/ft
○ SPT (N) Value greater than 50 blows/ft

0 25 50

Test Results: Su, psf (torvane in italics) | Dry Density, lb/ft³ | Moisture, % | Liquid Limit | Plasticity Index | % Passing No. 200 | Other Tests

Logged by:	M. Ivers
Date Start:	3/29/02
Date Finish:	4/2/01
Northing (ft):	7,362,671.3
Easting (ft):	1,528,207.3
Elevation (ft):	4,412.9
Total Depth Drilled (ft):	145.0
Drill Contractor:	ConeTec
Driller:	B. Mercur
Rig Type:	MARL M10
Drilling Method:	Hollow-Stem Auger
Hammer Type:	Automatic
Rod Type:	AWJ
Boring Diameter:	8 inches

LEGEND/NOTES

Elevations based upon North American Vertical Datum of 1988 (NAVD '88)

Coordinates are NAD '83

∇ = Observed Groundwater depth at time of drilling

Blows = Number of blows required to drive split spoon sampler 6 inches or distance shown

USCS = Unified Soil Classification System

AASHTO = American Association of State Highway and Transportation Officials

* = Classification based on visual-manual observations only (ASTM D 2488)

SAMPLE TYPE

SPT = Standard Penetration Test, 1.375 in. ID and 2 in. OD split spoon sampler

MCAL = Modified California Sampler, 2 in. ID and 2.5 in. OD split spoon sampler

P = Piston Sampler, 3 in. OD

SH = Shelby Tube, 3 in. OD, pushed

BAG = Bulk Sample

UTDOT ENGLISH 3502Z032.GPJ 6/2/02

Elevation (ft)	Boring: B-5 Sheet 3 of 3 SAMPLE DESCRIPTION * (ASTM D 2488)	Depth (ft)	Graphic Log	SAMPLE Type	Recovery (in)	Blows per 6 in. (N)	Soil Classification * USCS	AASHTO	● SPT (N) Blows/ft ○ SPT (N) Value greater than 50 blows/ft (0 25 50)	Su, psf (torvane in italics)	Dry Density, lb/ft 3	Moisture, %	Liquid Limit	Plasticity Index	% Passing No. 200	Other Tests
4270	(Crew drilled to 145 feet to recover sampler.)	145														
4265		150														
4260		155														
4255		160														
4250		165														
4245		170														
4240		175														
4235		180														
4230		185														
4225		190														
4220		195														
4215		200														
4210		205														
4205																

UTDOT ENGLISH 38022032.GPJ 8/2/02

12300 South Street Improvements
700 East to Bangerter Highway
KLEINFELDER
Project No. 13894-85D

FIELD TEST BORING LOG

Boring: **B-5**

Sheet 3 of 3

Logged by: M. Ivers
Date Start: 3/29/02
Date Finish: 4/2/01
Northing (ft): 7,362,671.3
Easting (ft): 1,528,207.3
Elevation (ft): 4,412.9
Total Depth Drilled (ft): 145.0
Drill Contractor: ConeTec
Driller: B. Mercur
Rig Type: MARL M10
Drilling Method: Hollow-Stem Auger
Hammer Type: Automatic
Rod Type: AWJ
Boring Diameter: 8 inches

LEGEND/NOTES

Elevations based upon North American Vertical Datum of 1988 (NAVD '88)

Coordinates are NAD '83

∇ = Observed Groundwater depth at time of drilling

Blows = Number of blows required to drive split spoon sampler 6 inches or distance shown

USCS = Unified Soil Classification System

AASHTO = American Association of State Highway and Transportation Officials

* = Classification based on visual-manual observations only (ASTM D 2488)

SAMPLE TYPE

SPT = Standard Penetration Test, 1.375 in. ID and 2 in. OD split spoon sampler

MCAL = Modified California Sampler, 2 in. ID and 2.5 in. OD split spoon sampler

P = Piston Sampler, 3 in. OD

SH = Shelby Tube, 3 in. OD, pushed

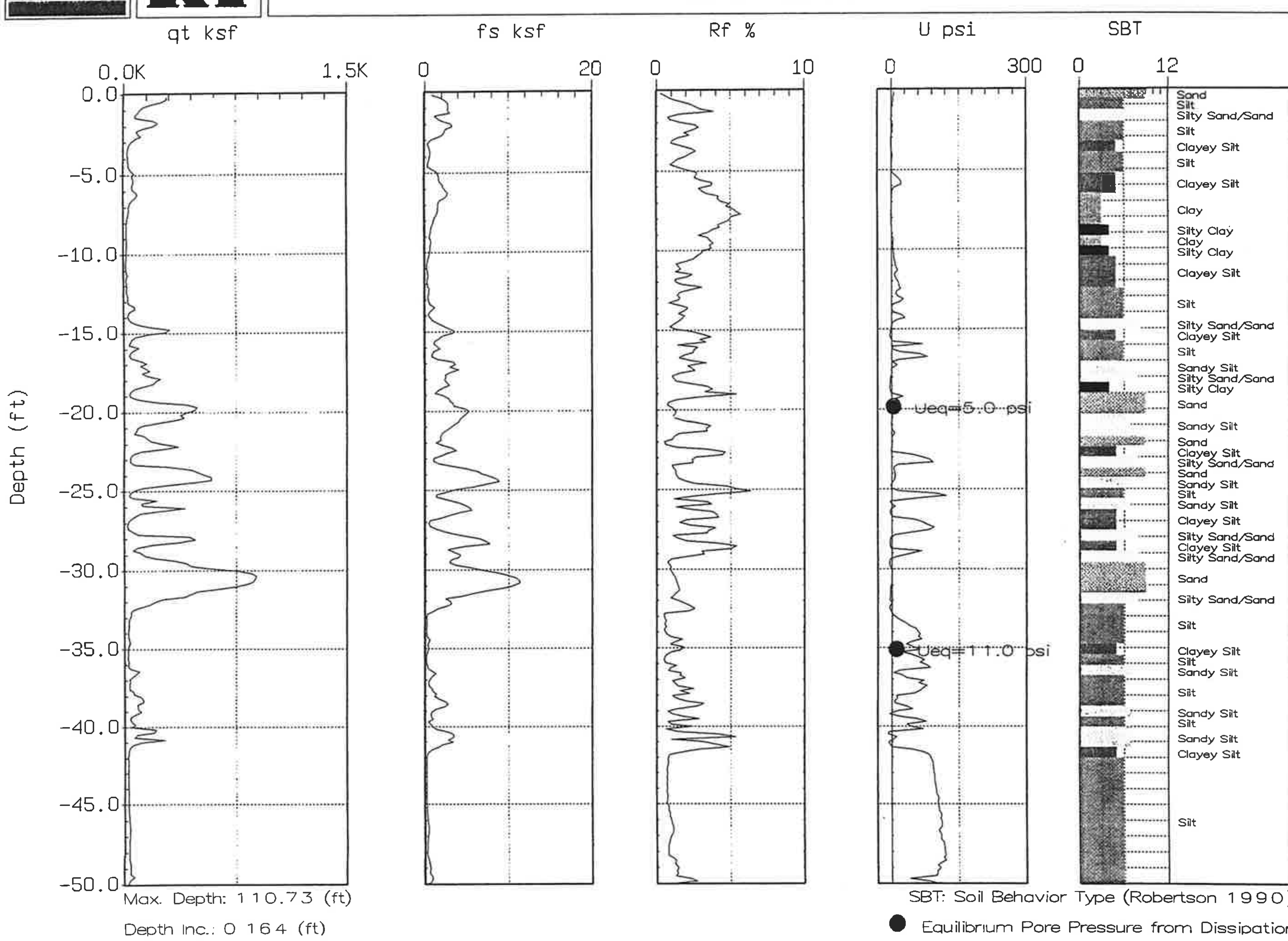
BAG = Bulk Sample

CONETEC

12300 South St.

Hole No.: CPT–6
Location: Railroad

Cone: 20 TON A 070
Date: 04:02:02 13:16

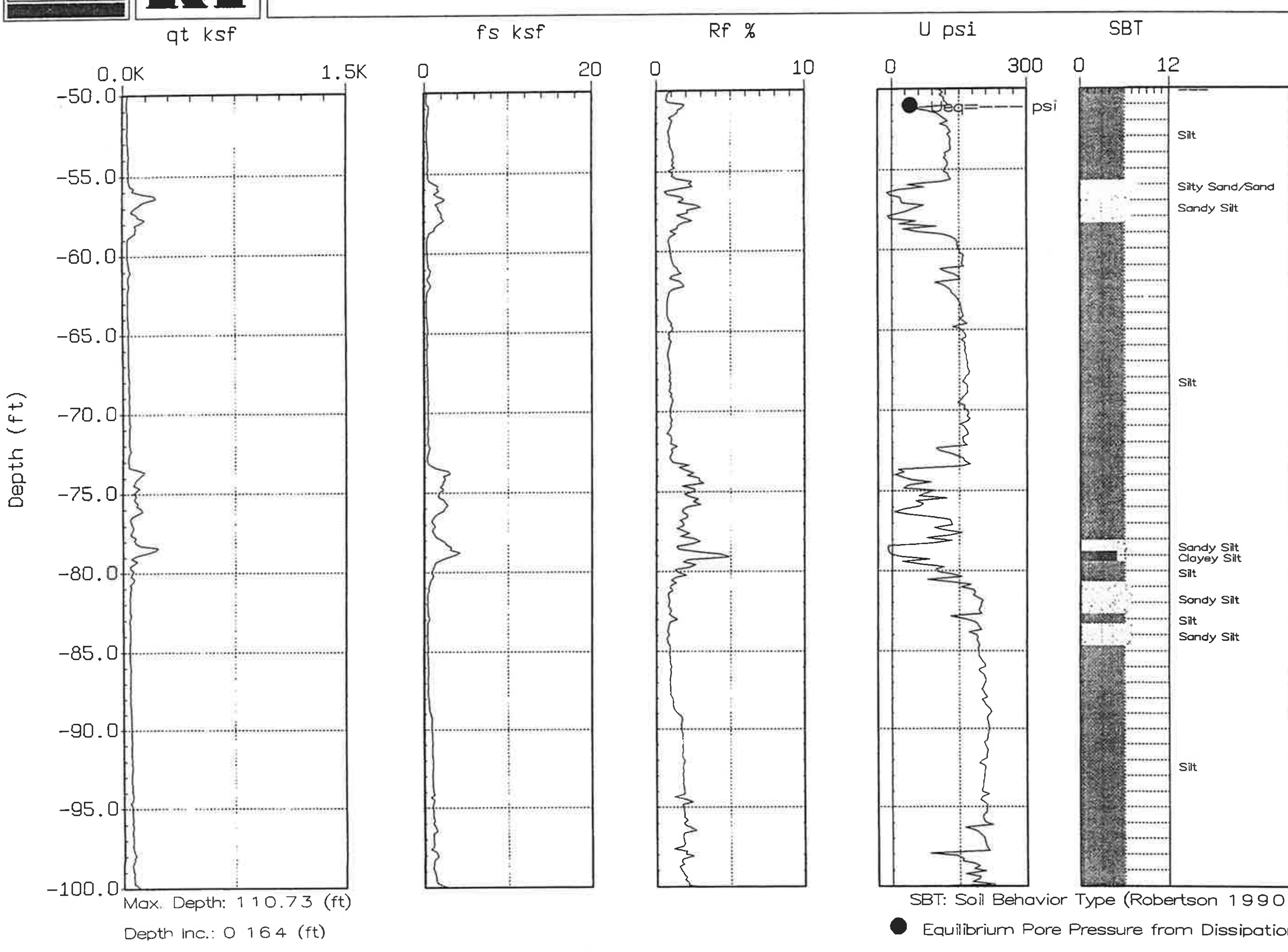


CONETEC

12300 South St.

Hole No. CPT-6
Location: Railroad

Cone: 20 TON A 070
Date: 04:02:02 13:16



Max. Depth: 110.73 (ft)
Depth Inc.: 0 164 (ft)

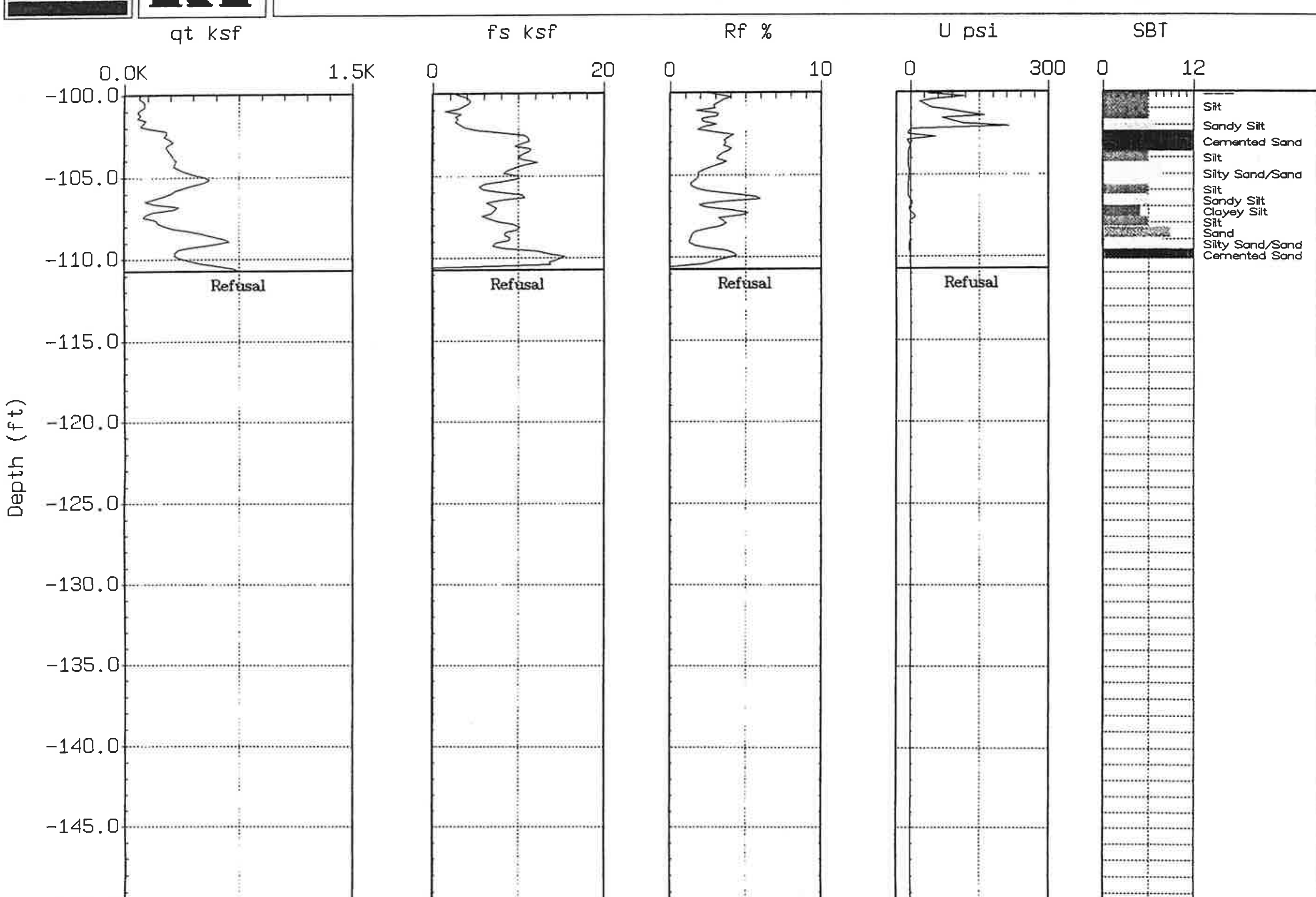
SBT: Soil Behavior Type (Robertson 1990)

● Equilibrium Pore Pressure from Dissipation

12300 South St.

Hole No.: CPT–6
Location: Railroad

Cone: 20 TON A 070
Date: 04:02:02 13:16



Max. Depth: 110.73 (ft)
Depth Inc.. 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation

APPENDIX C

Laboratory Testing

APPENDIX C
LABORATORY TESTING

1. GENERAL

Laboratory tests were performed on representative samples of the soils encountered in Borings AB-9, AB-20, and AB-21, and in Test Pits ATP-1 and ATP-2, to evaluate pertinent physical characteristics, aid in classifying the soils, and correlate other test data. The laboratory program included sample inspection to confirm AMEC's field soil descriptions and classification testing to determine natural moisture content, in-situ soil density, Atterberg limits, and grain-size distribution. Selected samples were evaluated for strength and consolidation characteristics by performing laboratory vane shear strength tests and one-dimensional consolidation tests. Chemical tests were also conducted on selected samples to determine the aggressiveness of those soils with respect to concrete and steel.

Identification of the test procedures and summaries of selected test results are presented in the following sections of this appendix. An overall summary of the classification, strength, and consolidation test results is presented in Table C-1.

2. CLASSIFICATION TESTS

2.1 MOISTURE AND DENSITY TESTS

Determination of natural moisture content was performed in general accordance with ASTM D-2216 test procedures. Determination of the in-situ dry density of selected, relatively undisturbed samples was performed in general accordance with ASTM D-4564 test procedures. Natural moisture content and dry density, where determined, are presented adjacent to the corresponding sample notation on the boring and test pit logs included in Appendix A.

It should be noted that some relatively undisturbed samples were subjected to a variety of test procedures, resulting in more than one set of natural moisture and, where determined, in-situ dry density values. For these samples, the natural moisture and in-situ dry density test values obtained in conjunction with individual test procedures are listed in the summaries of those test procedures presented in the following sections. For samples with more than one set of test results for natural moisture and in-situ dry density, the averaged values of those test results are presented on the boring and test pit logs and in Table C-1.

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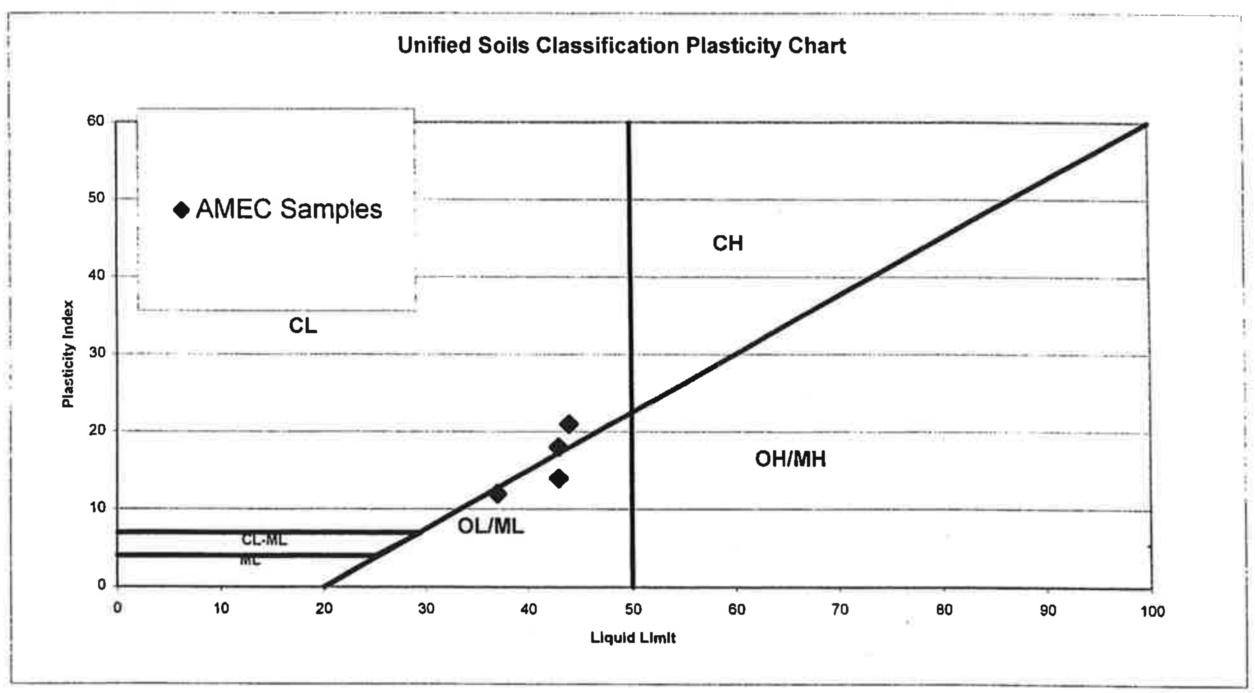
4137 South 500 West
Salt Lake City, Utah 84123
Tel +1 (801) 266-0720
Fax +1 (801) 266-0727 www.amec.com

2.2 ATTERBERG LIMITS TESTS

Determination of the Atterberg limits of selected samples was performed in general accordance with ASTM D-4318. Results from Atterberg limits testing are summarized in the following table and in Table C-1, and are illustrated on the following Plasticity Chart.

Boring No.	Sample Depth (ft)	Unified Soil Classification System Group Symbol*	In situ Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
AB-9	10.5	CL	35.5	44	23	21
AB-9	50.5	CL/ML	29.9	37	25	12
AB-9	55.5	ML	22.4	--	--	NP
AB-9	65.5	ML/CL	34.1	37	25	12
AB-9	75.5	ML	25.5	--	--	NP
AB-9	80.5	ML	41.0	43	29	14
AB-9	85.5	ML	28.0	37	25	12
AB-9	100.5	SP-SM	23.8	--	--	NP
AB-20	6.5	CL	29.6	43	25	18

* Based upon portion of the sample passing the No. 40 sieve.



2.3 GRAIN SIZE ANALYSES

Full and partial grain-size analyses were performed in general accordance with ASTM C-117, C-136, and D-1140 test procedures. Partial grain-size analyses were performed to determine the fines content (percent material by weight passing the No. 200 sieve). Results from all of the grain-size analyses are summarized in the following table. The results of the fines content analyses are also included in Table C-1.

Boring No.	Sample Depth (ft)	Percent Passing by Weight				Unified Soil Classification System Group Symbol
		No 4	No. 40	No. 100	No. 200	
AB-9	20.5	100	98	58	25	SM
AB-9	25.5				13	SM
AB-9	35.5	100	100	87	47	SM/ML
AB-9	40.5				43	SM
AB-9	105.5	100	100	56	10	SP-SM
AB-20	19.5	100	91	49	18	SM
AB-21	29.5				47	SM/ML
ATP-1	1.5	100	99	85	68	ML

3. ONE-DIMENSIONAL CONSOLIDATION TESTS

One-dimensional consolidation testing was performed on ten relatively undisturbed, representative samples of the fine-grained cohesive soils encountered in the borings. Consolidation testing was performed in general accordance with the ASTM D-2435 test procedures. Detailed results of the tests are maintained within our files and can be provided at your request.

4. LABORATORY VANE SHEAR STRENGTH TESTS

Undrained shear strength tests were performed on relatively undisturbed samples of fine-grained cohesive soils in general accordance with the ASTM D-4648 test procedure. Results from the vane shear strength tests are summarized in the table on the next page and in Table C-1.

Boring No.	Sample Depth (ft)	Unified Soil Classification System Group Symbol*	In situ Moisture Content (%)	Dry Density (pcf)	Average Peak Undrained Shear Strength (psf)
AB-9	50.5	CL	31.9	90	1880
AB-9	55.5	CL	23.9	102	1880
AB-9	65.5	CL	33.8	88	1725
AB-9	75.5	MH/CH	26.5	98	1665
AB-9	85.5	MH/CH	35.2	86	1975
AB-9	90.5	CL	35.8	85	1860
AB-20	9.5	ML	42.8	74	2350
AB-21	6.5	CL/ML	32.2	90	1500
AB-21	9.5	ML	29.4	90	1500
ATP-2	4.5	ML	20.3	100	4250

5. CHEMICAL TESTS

5.1 pH AND SOLUBLE SULFATES TESTS

To determine if the site soils will react detrimentally with concrete, pH and soluble sulfates tests were performed on four representative samples of the site soils. The results of those tests are presented below:

Boring No.	Sample Depth (ft)	Unified Soil Classification System Group Symbol	pH	Soluble Sulfates Content (Percent by weight)
AB-9	15.5	CL/ML/SM	8.2	<0.001
AB-9	45.5	CL/ML	7.6	0.056
AB-20	9.5	CL	8.4	<0.001
AB-21	9.5	ML	8.1	<0.001

The above test results indicate that the site soils are mildly to moderately alkaline and contain negligible amounts of water-soluble sulfates. Based on the above values, the potential of the site soils, particularly near-surface soils, to react detrimentally with concrete are considered to be negligible.

5.2 RESISTIVITY TESTS

To determine if the site soils are corrosive to buried metals, resistivity tests were performed on four representative samples. The results of those tests are presented below:

Boring No.	Sample Depth (ft)	Unified Soil Classification System Group Symbol	Resistivity (ohm-centimeter)
AB-9	5.5	ML	2,040
AB-9	15.5	CL/ML/SM	1,830
AB-20	45.5	CL/ML	769
AB-21	9.5	ML	1,370

The above test results indicate that the site soils have low to moderate resistivity values. In general, soils with resistivity values less than 1,000 ohm-centimeters are considered to be severely corrosive to buried metals. Soils with resistivity values between 1,100 and 3,000 ohm-centimeters are considered to be moderately corrosive. Note that these guidelines are very qualitative and do not provide information regarding corrosion rates, etc. Note that for soils with pH values between 4 and 10, the corrosion rate is considered similar to that for neutral soils.

Table C-1

12300 South
Laboratory Testing Summary Sheet
12300 South - UPRR Crossing
12/04/02

Boring No.	Sample No.	Sample Depth (ft)	Moisture Content (%)	Dry Density (pcf)	Moist Density (pcf)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	P'c (psf)	P'o (psf)	OCR (Pc/Po)	-200 (%)	Vane Shear (psf)	AASHTO Classification
AB-9	1	2.5												
	2	5.5	31.7	84.2	110.9									
	3	10.5	37.9	79.5	109.6	44	23	21	5400	1210	4.46			A-7-6
	4	15.5												
	5	20.5	24.1									25.2		A-2-4
	6	25.5	21.4	105.0	127.4							12.7		A-2-4
	7	30.5												
	8	35.5	21.7	107.0	130.2							46.7		A-4
	9	40.5	21.1	102.5	124.1							43.3		A-4
	10	45.5												
	11	50.5	31.1	91.2	119.5	37	25	12	5600	3380	1.66		1880	A-6
	12	55.5	23.2	102.3	126.0			NP					1880	
	13	60.5												
	14	65.5	33.9	87.7	117.4	37	25	12	7000	4440	1.58		1725	A-6
	15	70.5												
	16	75.5	26.0	98.0	123.5			NP					1665	
	17	80.5	41.2	79.7	112.5	43	29	14						A-7-6
	18	85.5	33.2	85.7	114.1	37	25	12	7900	5530	1.43		1975	A-6
	19	90.5	35.6	85.5	115.8				7300	5800	1.26		1860	
	20	95.5												
	21	100.5	25.6	96.1	120.7			NP						
	22	105.5	22.5	107.1	131.2							10.0		A-3
	23	110.0												
	24	115.0												
AB-20	1	3.5												
	2	6.5	30.6	85.2	111.3	43	25	18						A-7-6
	3	9.5	40.9	76.4	107.5				7000	1050	6.67		2350	
	4	14.5												
	5	19.5	20.2									17.9		A-2-4
	6	24.5												
	7	29.5												

Table C-1

12300 South
Laboratory Testing Summary Sheet
12300 South - UPRR Crossing
12/04/02

Boring No.	Sample No.	Sample Depth (ft)	Moisture Content (%)	Dry Density (pcf)	Moist Density (pcf)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	P'c (psf)	P'o (psf)	OCR (Pc/Po)	-200 (%)	Vane Shear (psf)	AASHTO Classification
AB-21	1	6.5	32.9	88.9	118.1				5800	770	7.53		1500	
	2	9.5	29.7	89.1	115.6				4600	1120	4.11		1500	
	3	14.5	25.4	98.8	123.9									
	4	19.5												
	5	29.5	22.1	99.8	121.8							46.5		A-4
ATP-1	1	1.5	19.0	78.4	93.3							68.2		A-4
	2	3.5												
ATP-2	1	1.0												
	2	4.5	20.3	99.7	119.9								4250	
	3	12.0												
	4	18.0												

APPENDIX D

Cone Penetration Test (CPT) Data

APPENDIX D
CONE PENETRATION TEST (CPT) DATA

This appendix provides logs of the cone penetration test (CPT) probes and associated data. ConeTec, Inc. advanced six CPT probes along the mainline alignment. Those probes include five probes (ACPT-13 through ACPT-17) along the mainline to depths of about 30.0 feet each, and one probe (ACPT-3) at the bridge site to refusal at a depth of about 108.0 feet. The approximate locations of the CPT probes are indicated on Figure 2B, Corner Canyon, and Figure 2C, Bridge Structure and Retaining Walls.

CONETEC

ConeTec, Inc.

Geotechnical and Environmental Site Investigation Contractors

3589 West 500 South, Suite 3, Salt Lake City, UT 84104 • PO Box 22082, Salt Lake City, UT 84122
Tel: (801) 973-3801 • Fax: (801) 973-3802 • Web: www.conetec.com • Email: ctecslc@attglobal.net

August 20, 2002

Job No.: 02-351

Mr. Wade Gilbert
AMEC Earth & Environmental, Inc.
4137 South 500 West
Salt Lake City, UT 84123

Re: CPT Results
12300 South Interchange Reconstruction Project
Salt Lake City, Utah

Dear Wade,

Per your request, we have completed the CPT investigation for the above referenced project. Enclosed is one set of standard CPT plots, seismic CPT plots, pore pressure dissipation plots and a data diskette. The diskette contains data files for the CPT plots (*.cor files), the files for the PPD plots (*.ppd files). The following table summarizes the work performed at the site.

CPT Location	CPT Filename	Maximum Depth (ft)	PPD Depth (ft)	PPD Duration (sec)	Ueq (ft)	Comments
ACPT-3	351CP03	108.43	101.38	200	39.0	Seismic & Refusal
ACPT-4	351CP04	53.15	35.43	700	28.8	Refusal
ACPT-5	351CP05	30.02	22.64	300	19.3	
ACPT-6	351CP06	20.18	No PPD			Refusal
ACPT-7	351CP07	30.02	30.02	600	19.0	
ACPT-8	351CP08	25.92	No PPD			Refusal
ACPT-9	351CP09	30.02	25.59	800	~8.0	
ACPT-11	351CP11	60.04	60.04	1000	36.5	
ACPT-12	351CP12	51.02	No PPD			Refusal

Many correlations have been developed for design parameters based on CPT data. The interpretations are presented only as a guide for geotechnical use and should be carefully scrutinized for consideration in any geotechnical design.

Job No.: 02-351

Mr. Wade Gilbert
AMEC Earth & Environmental, Inc.
August 20, 2002
Page Two

Assumptions have been made regarding soil unit weights, groundwater level and interpretational methods, which may or may not apply to this site. The following table summarizes the values assigned to the specific soil behavior type zones.

Zone	SPT Qt/N	Unit Wt. (kN/m^3)	Unit Wt. (pcf)	K (cm/s)	Description
0	1.0	19.5	124.1	$1x10^{-15}$	Undefined
1	2.0	12.5	79.6	$1x10^{-7}$	Sensitive Fines
2	1.0	17.5	111.4	$1x10^{-15}$	Organic Soil
3	1.0	17.5	111.4	$5x10^{-8}$	Clay
4	1.5	18.0	114.6	$5x10^{-7}$	Silty Clay
5	2.0	18.0	114.6	$5x10^{-6}$	Clayey Silt
6	2.5	18.0	114.6	$5x10^{-5}$	Silt
7	3.0	18.5	117.8	$5x10^{-4}$	Sandy Silt
8	4.0	19.0	120.9	$5x10^{-3}$	Silty Sand/Sand
9	5.0	19.5	124.1	$5x10^{-2}$	Sand
10	6.0	20.0	127.3	5	Gravelly Sand
11	1.0	20.5	130.5	$1x10^{-15}$	Stiff Fine Grained
12	2.0	19.0	120.9	$1x10^{-5}$	Cemented Sand

We appreciate the opportunity of providing these services to you. If you have any questions regarding the enclosed material or if, we can be of additional assistance, please contact us.

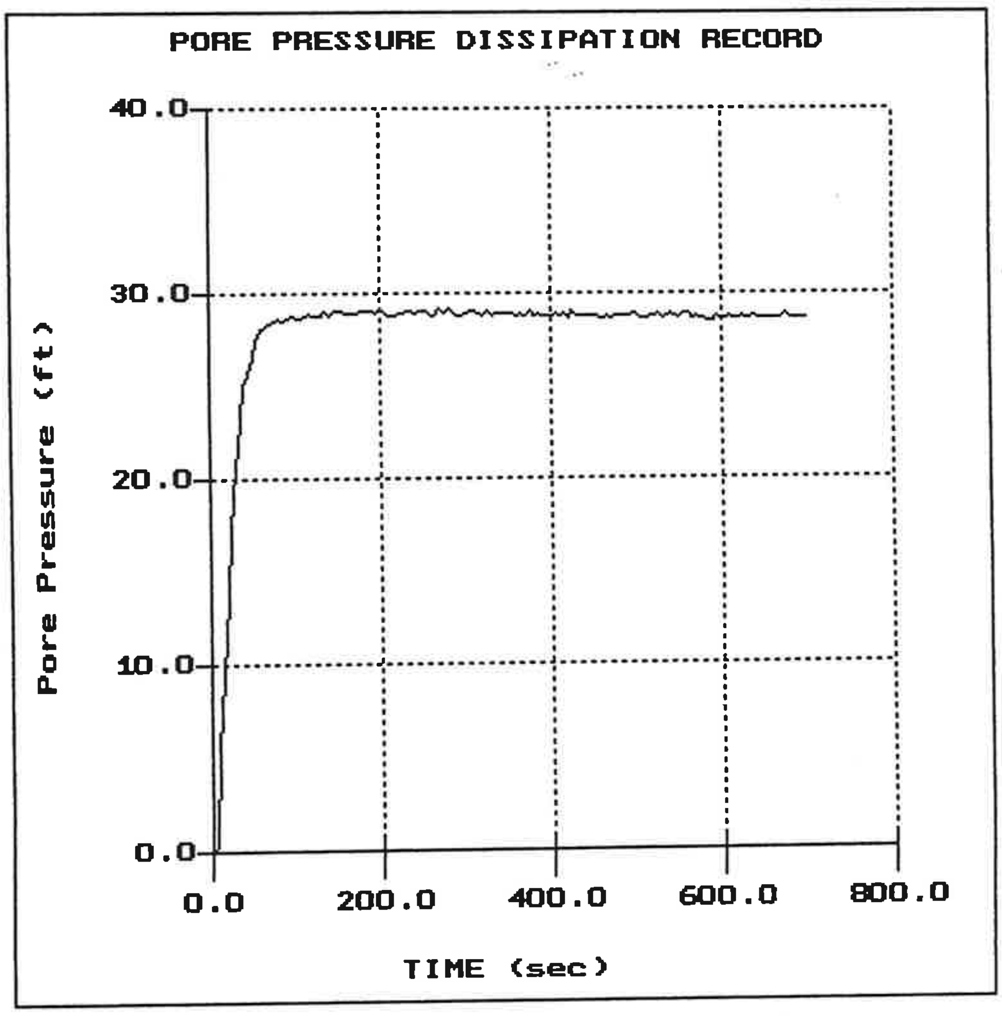
Sincerely,

ConeTec, Inc.

Shawn D. Steiner, P.E.
Manager

Enclosures

AMEC E&E	Hole No.:ACPT-04 Location:DRAPER	Cone: 20 TON A 122 Date:08:05:02 14:45



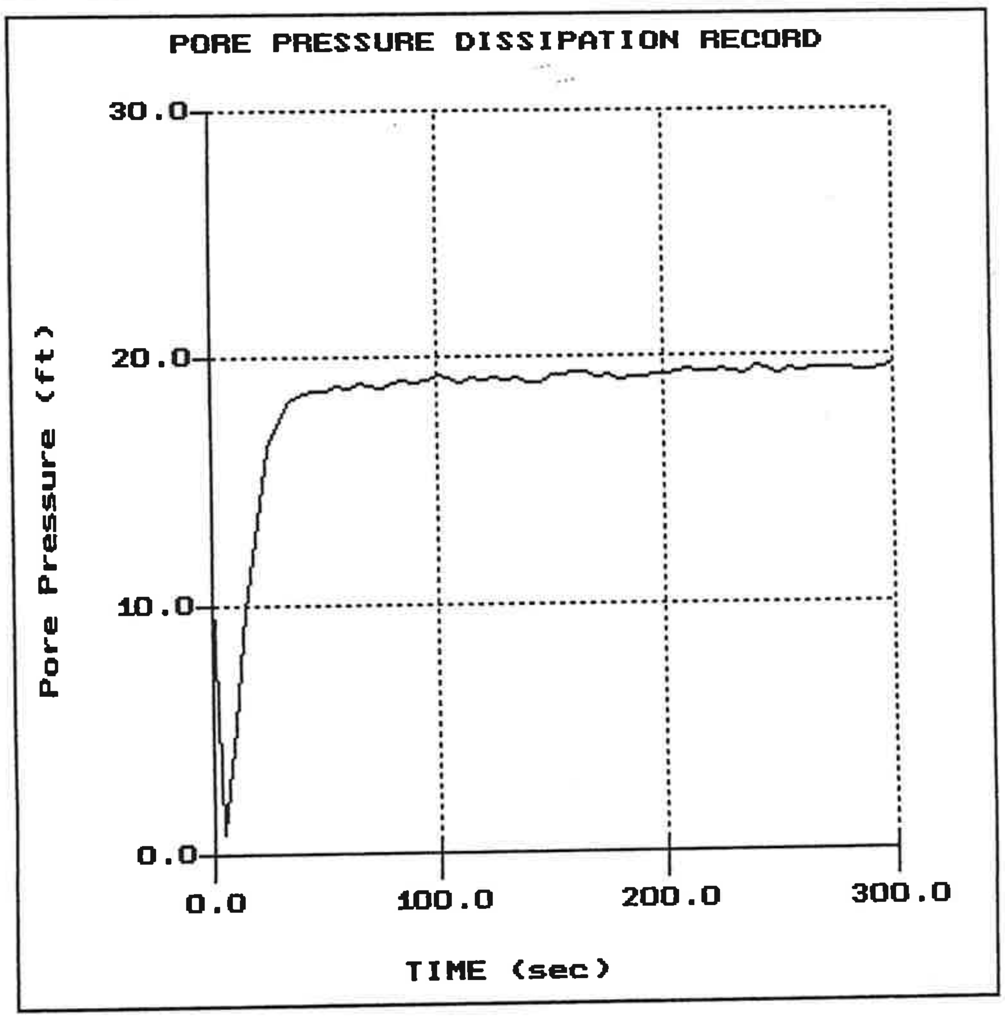
File: 351CP04.PPD
Depth (m): 10.80
(ft): 35.43
Duration : 700.0s
U-min: -3.66 0.0s
U-max: 29.14 280.0s

AMEC E&E

Hole No.:ACPT-05
Location:DRAPER

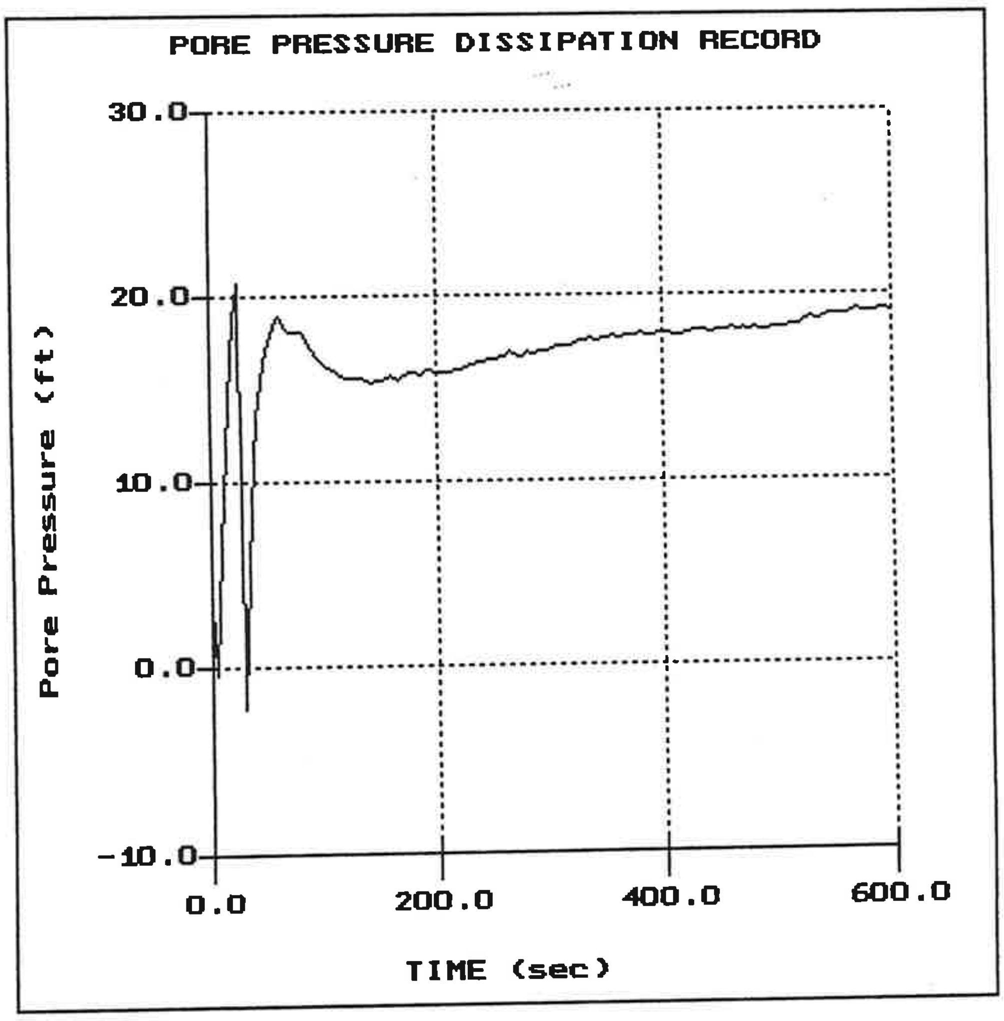
Cone: 20 TON A 122
Date:08:05:02 10:33

File: 351CP05.PPD
Depth (m): 6.90
(ft): 22.64
Duration : 300.0s
U-min: 0.84 5.0s
U-max: 19.71 300.0s



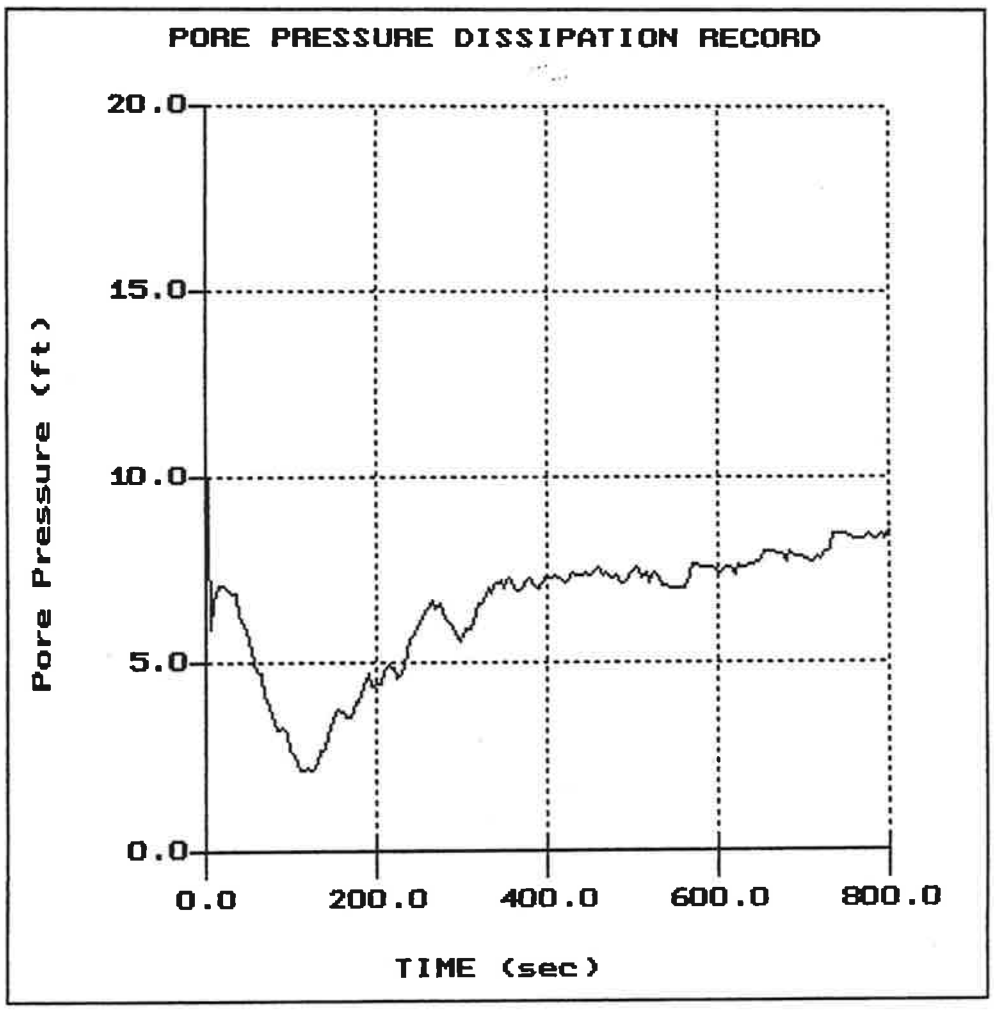
AMEC E&E	Hole No.:ACPT-07 Location:DRAPER	Cone: 20 TON A 122 Date:08:05:02 13:16

File: 351CP07.PPD
Depth (m): 9.15
(ft): 30.02
Duration : 605.0s
U-min: -2.21 30.0s
U-max: 20.74 25.0s



AMEC E&E	Hole No.:ACPT-09 Location:DRAPER	Cone: 20 TON A 122 Date:08:05:02 11:31

File: 351CP09.PPD
Depth (m): 7.80
(ft): 25.59
Duration : 800.0s
U-min: 2.06 110.0s
U-max: 11.36 0.0s

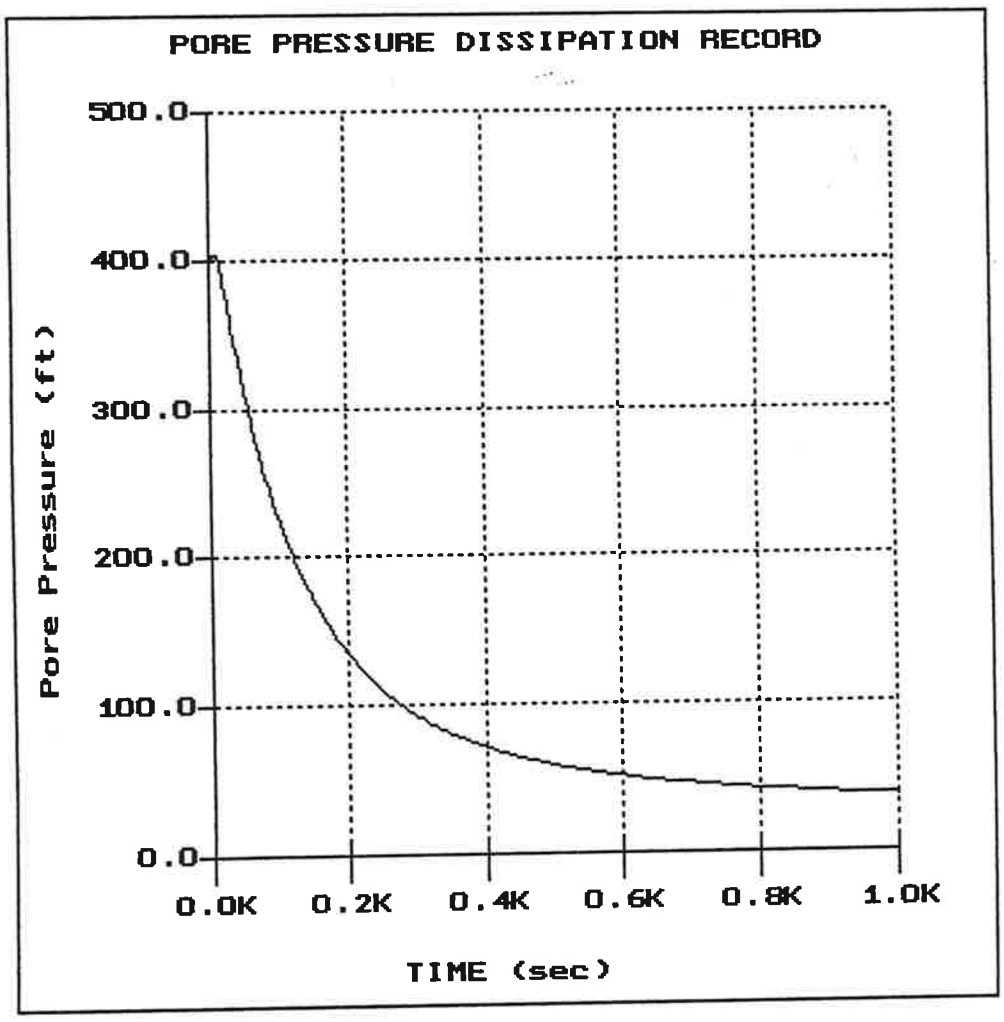


AMEC E&E

Hole No.:ACPT-11
Location:DRAPER

Cone: 20 TON A 122
Date:08:05:02 16:20

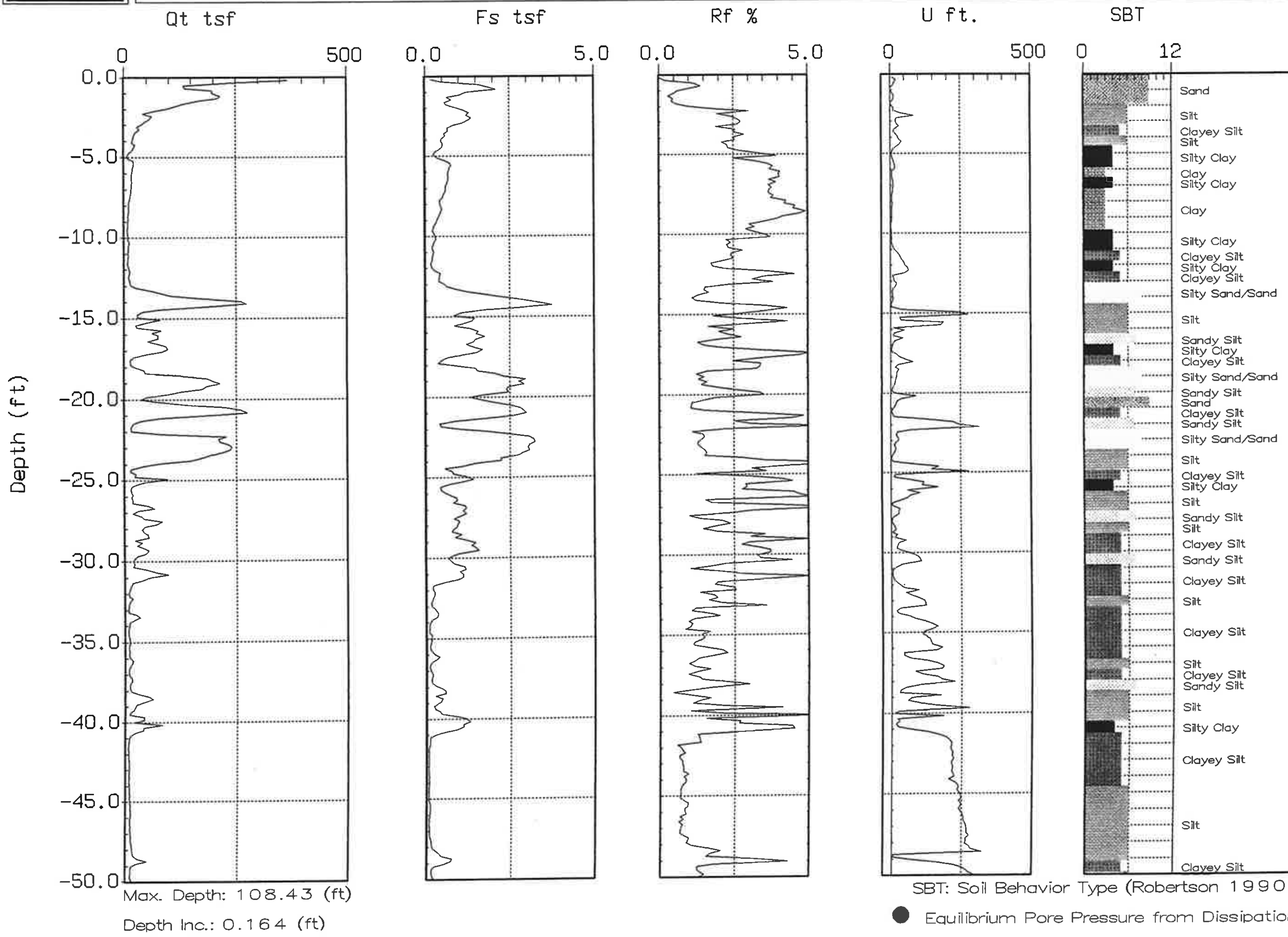
File: 351CP11.PPD
Depth (m): 18.30
(ft): 60.04
Duration : 1000.0s
U-min: 35.43 1000.0s
U-max: 403.31 5.0s



AMEC E & E

Hole No.: ACPT-3
Location: 12300 SOUTH

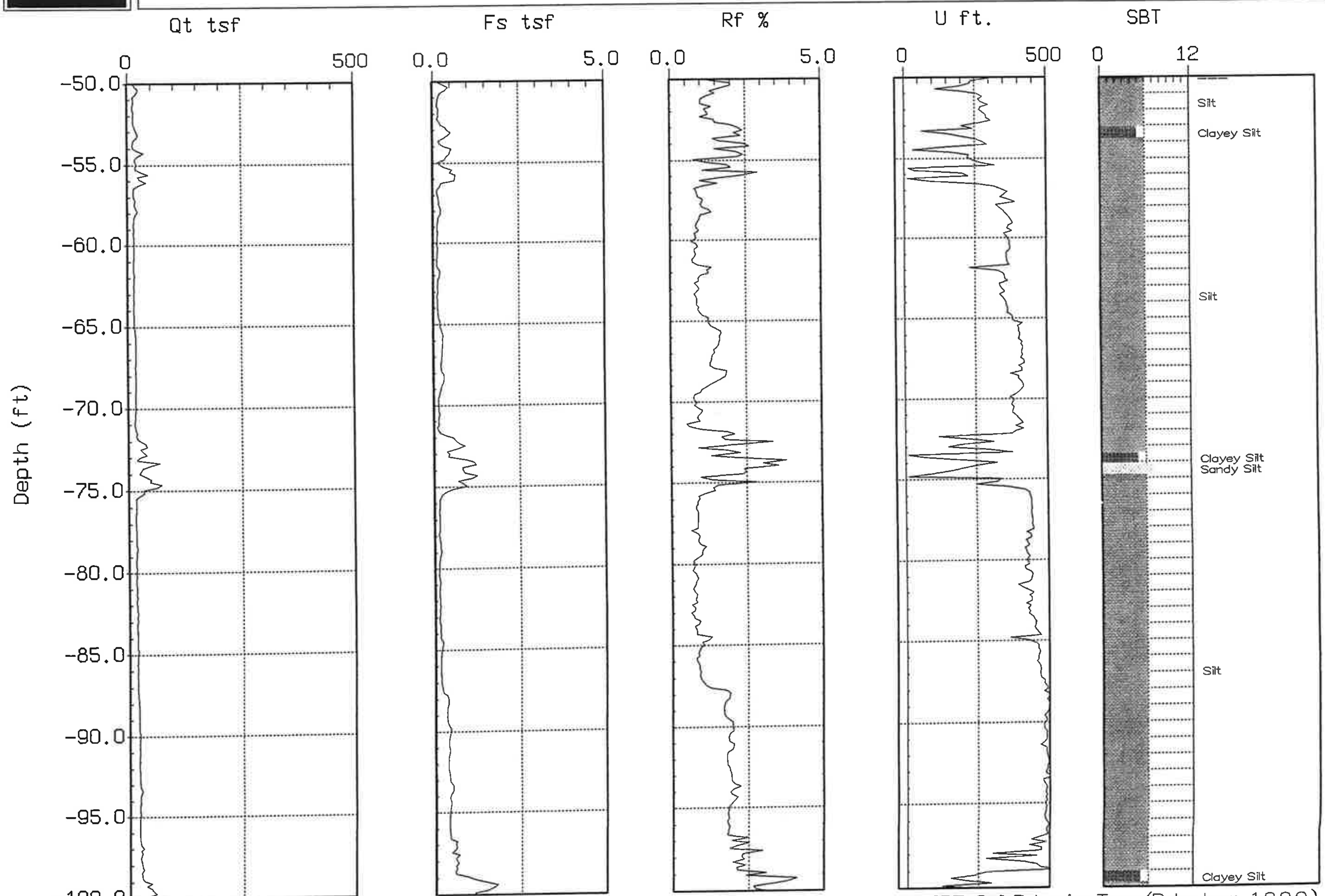
Cone: 20 TON A 122
Date: 08:02:02 14:27



AMEC E & E

Hole No.: ACPT–3
Location: 12300 SOUTH

Cone: 20 TON A 122
Date: 08:02:02 14:27



Max. Depth: 108.43 (ft)
Depth Inc.: 0.164 (ft)

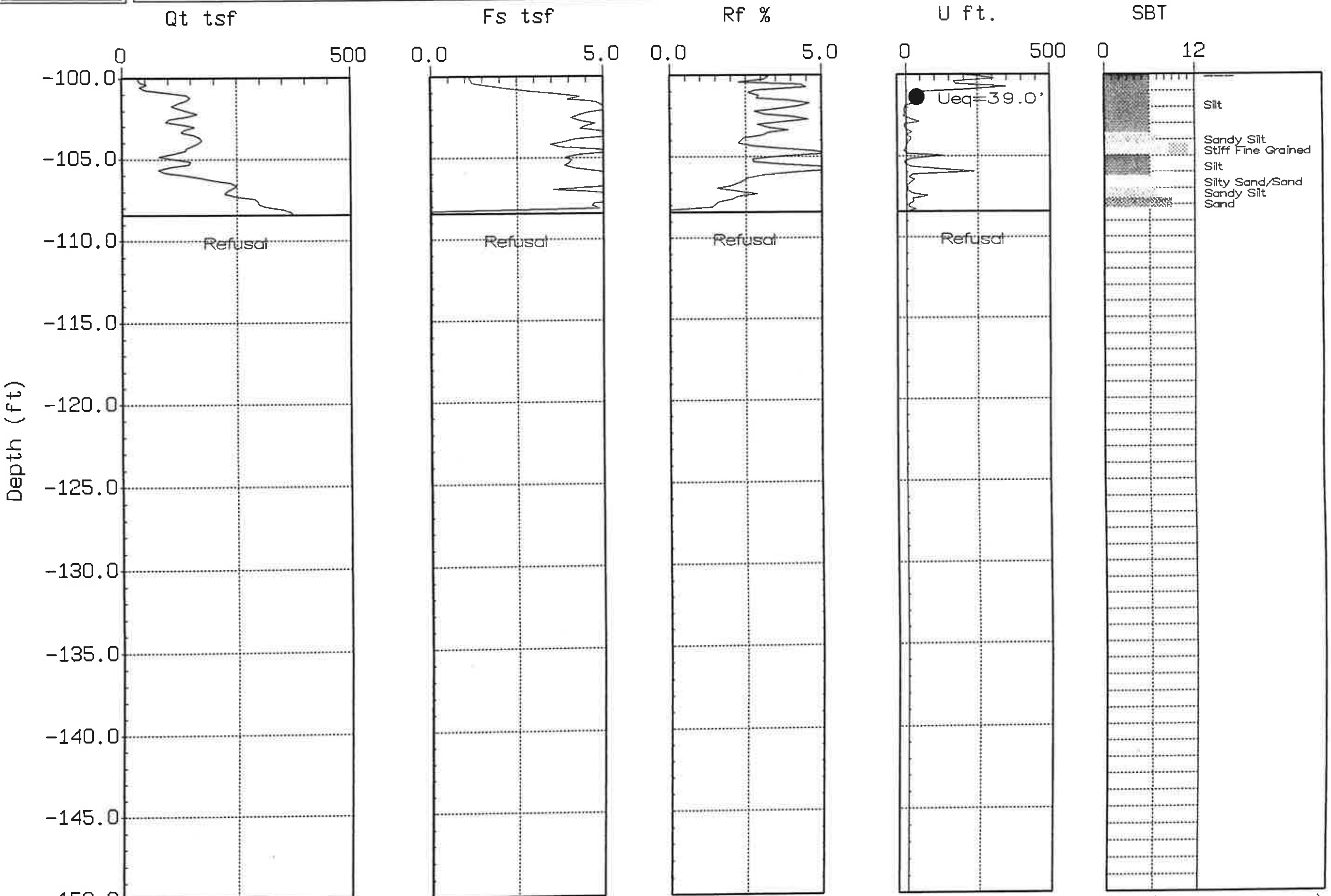
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation

CONETEC

AMEC E & E

Hole No.: ACPT–3
Location: 12300 SOUTH

Cone: 20 TON A 122
Date: 08:02:02 14:27



Max. Depth: 108.43 (ft)
Depth Inc.: 0.164 (ft)

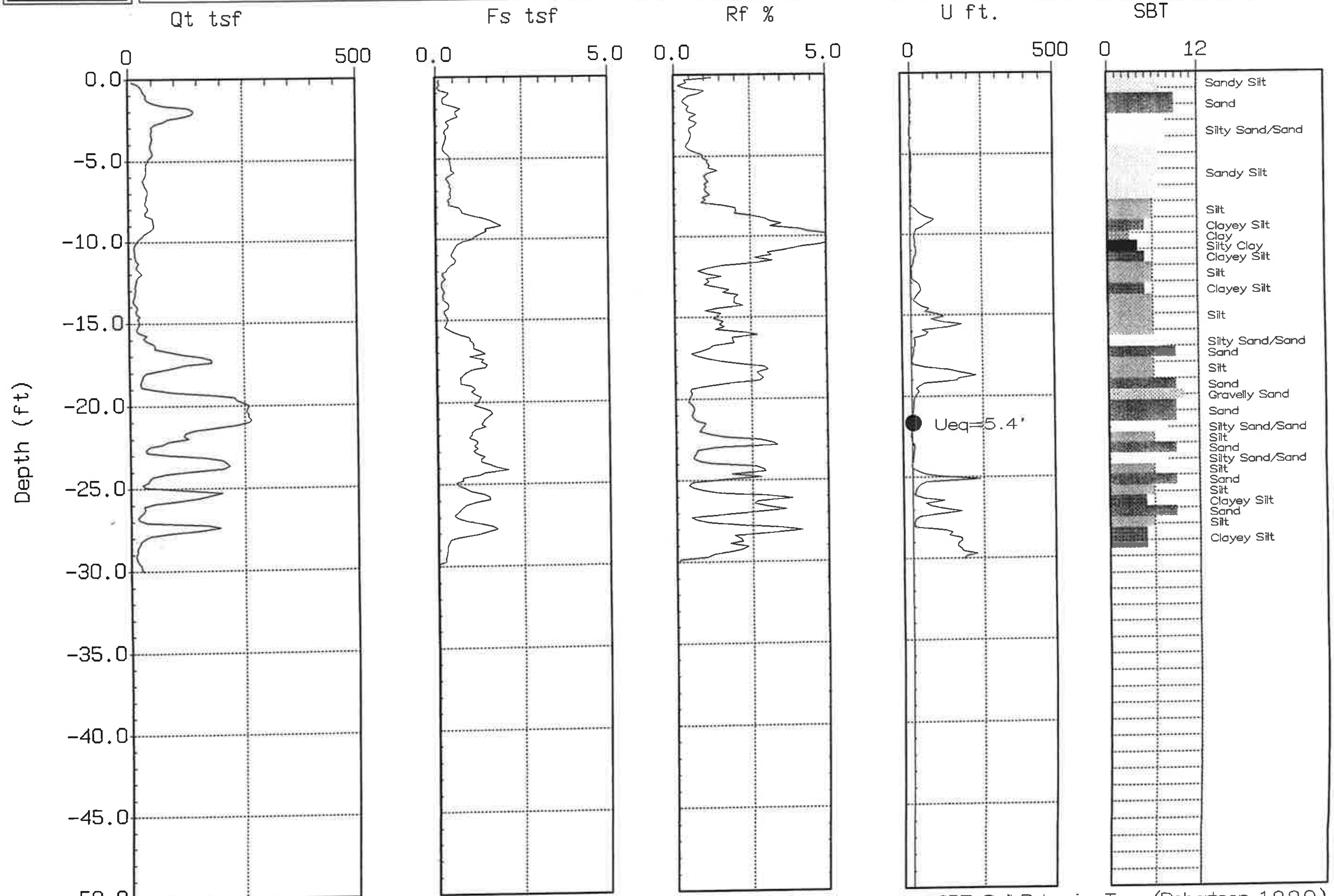
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation

CONETEC

AMEC E & E

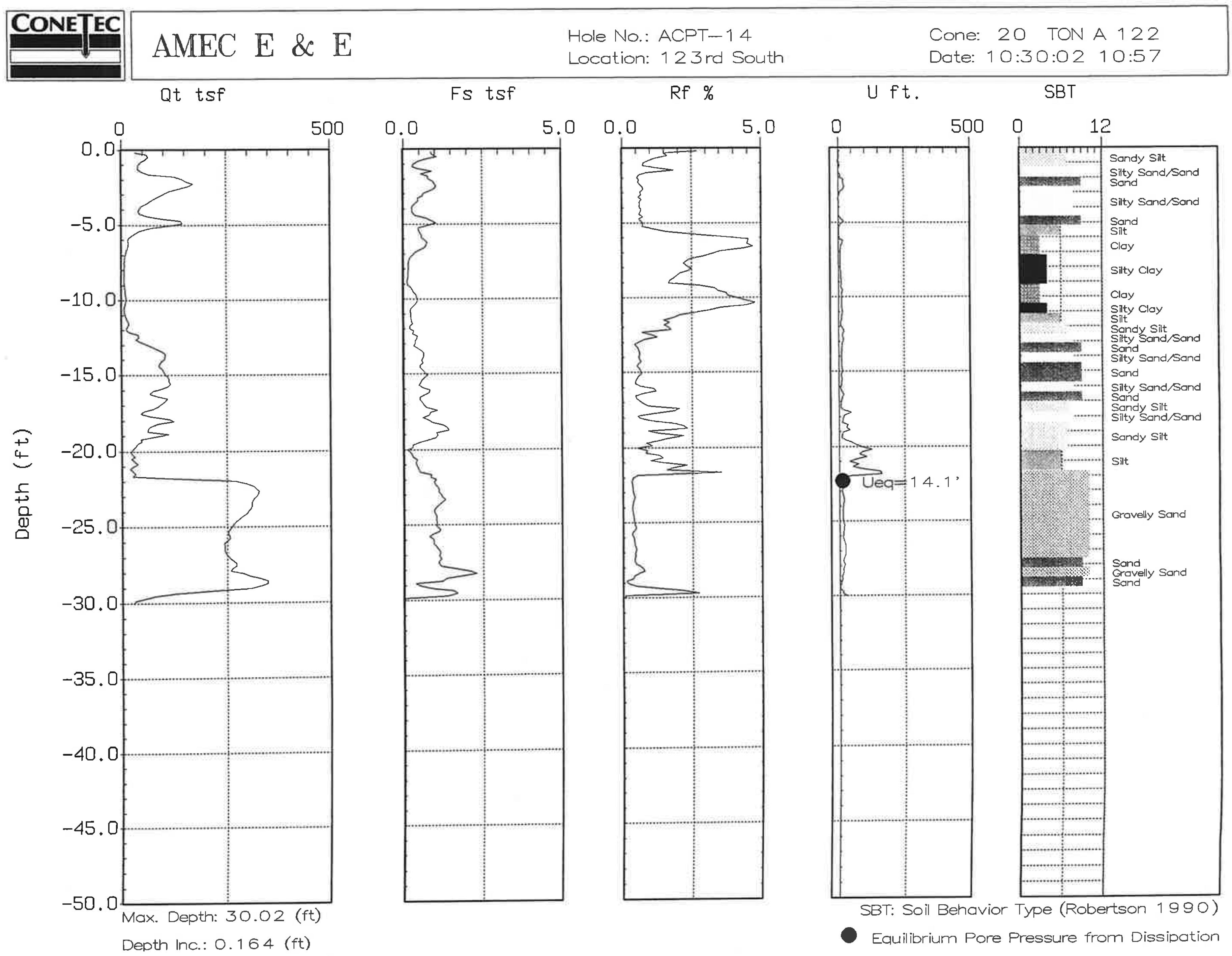
Hole No.: ACPT-13
Location: 123rd South

Cone: 20 TON A 122
Date: 10:30:02 09:56



Max. Depth: 30.02 (ft)
Depth Inc.: 0.164 (ft)

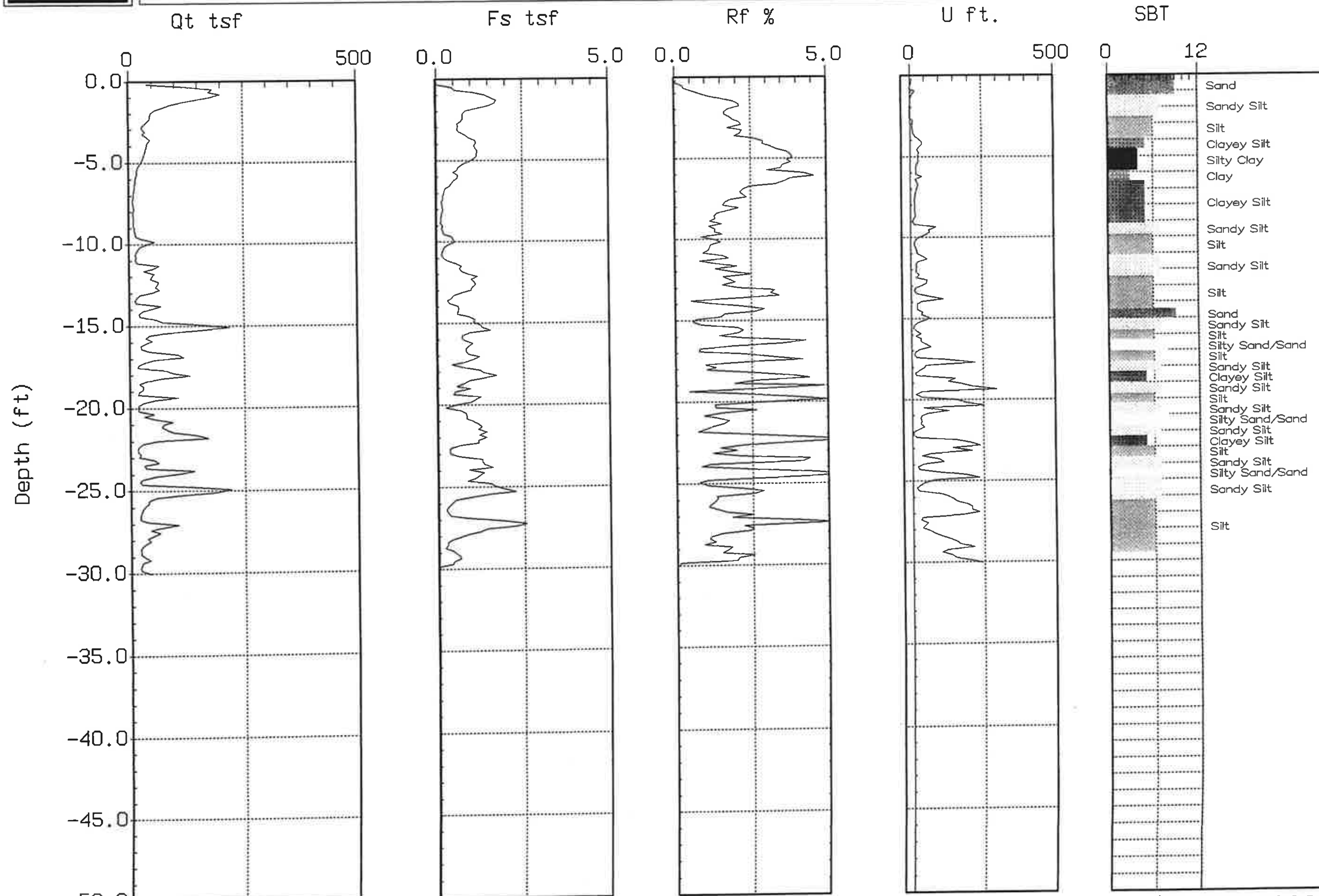
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation


CONETEC
AMEC E & E
Hole No.: ACPT-14
Location: 123rd South
Cone: 20 TON A 122
Date: 10:30:02 10:57
Qt tsf
Fs tsf
Rf %
U ft.
SBT
Depth (ft)
Ueq=14.1'
Sandy Silt
Silty Sand/Sand
Sand
Silty Sand/Sand
Sand
Silt
Clay
Silty Clay
Clay
Silty Clay
Silt
Sandy Silt
Silty Sand/Sand
Sand
Silty Sand/Sand
Sand
Silty Sand/Sand
Sand
Sandy Silt
Silty Sand/Sand
Sandy Silt
Silt
Gravelly Sand
Sand
Gravelly Sand
Sand
Max. Depth: 30.02 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
Equilibrium Pore Pressure from Dissipation

AMEC E & E

Hole No.: ACPT-15
Location: 123rd South

Cone: 20 TON A 122
Date: 10:30:02 11:31



Max. Depth: 30.02 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

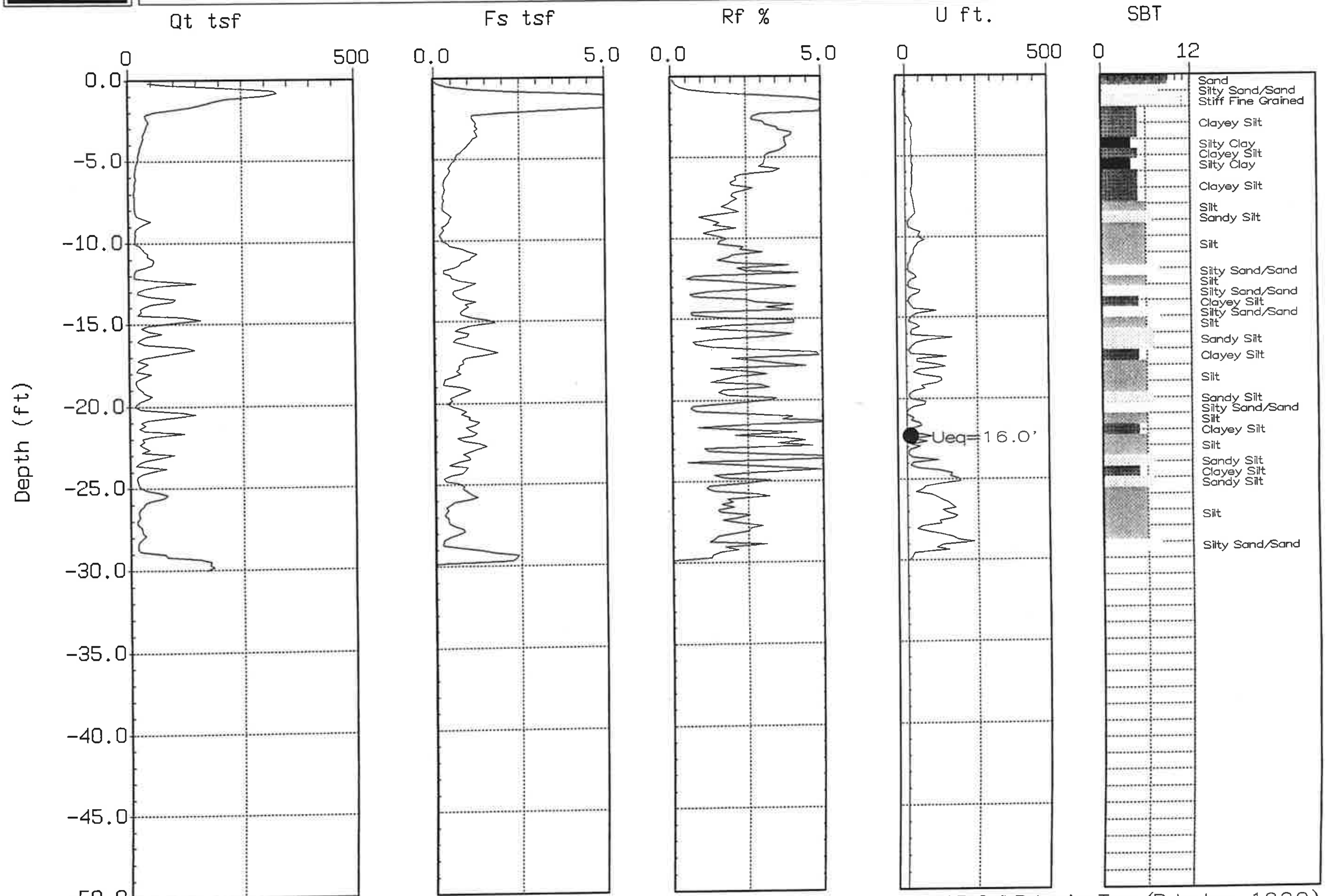
● Equilibrium Pore Pressure from Dissipation

CONETEC

AMEC E & E

Hole No.: ACPT-16
Location: 123rd South

Cone: 20 TON A 122
Date: 10:30:02 11:48



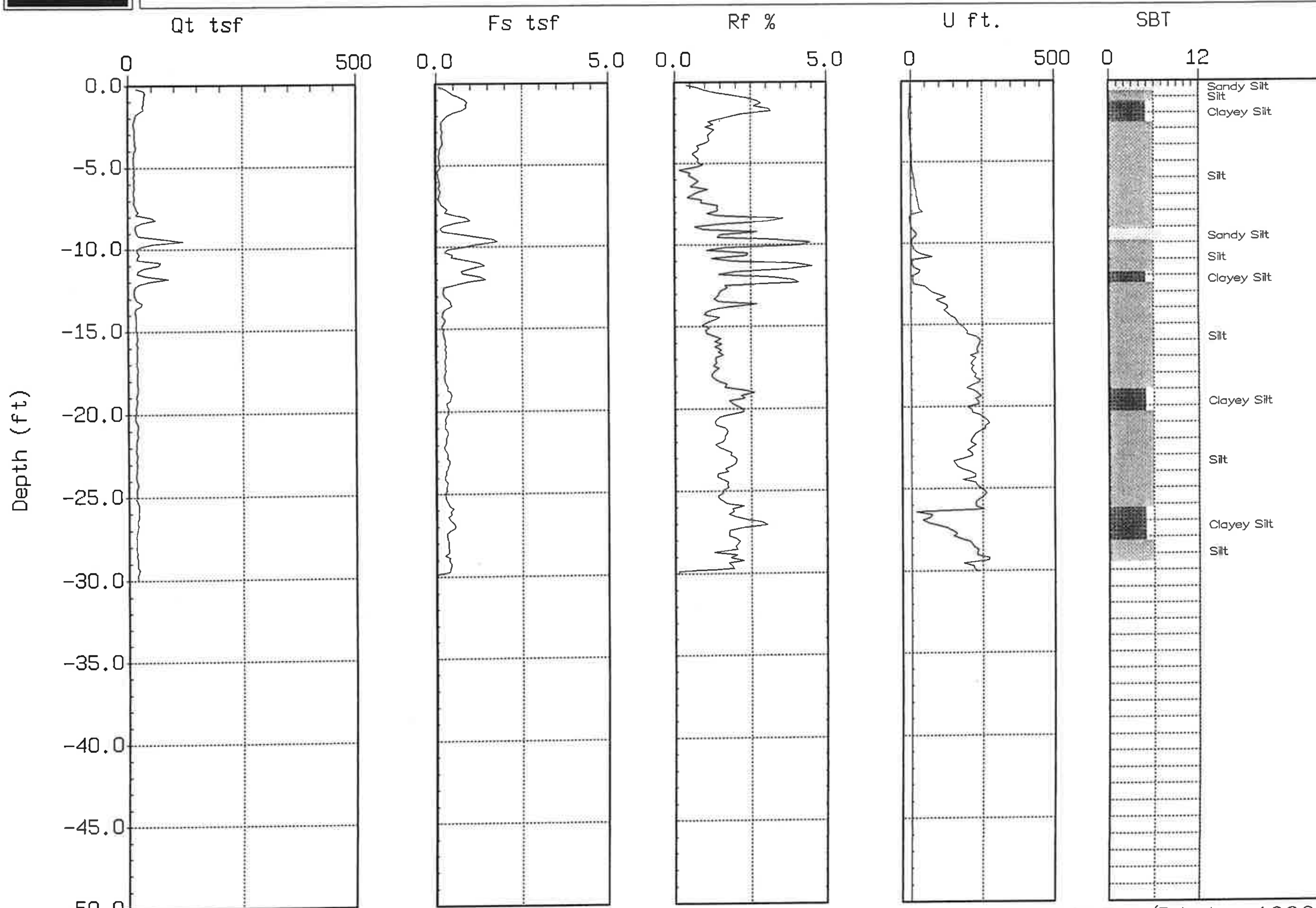
Max. Depth: 30.02 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation

AMEC E & E

Hole No.: ACPT–17
Location: 123rd South

Cone: 20 TON A 122
Date: 10:30:02 12:47



Max. Depth: 30.02 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

● Equilibrium Pore Pressure from Dissipation