PRELIMINARY GEOTECHNICAL REPORT
KNIK ARM BRIDGE PROJECT

PREPARED FOR:

Alaska Dept. of Transportation & Public Facilities

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EXECUTIVE SUMMARY

This report presents the results of field explorations and testing, surface reconnaissance, laboratory testing, and preliminary engineering analyses for the Knik Arm Bridge Project, Anchorage, Alaska. In support of this effort, an overwater geophysical survey was also performed by Golder Associates (Golder). The purpose of this work was to define subsurface conditions across a likely water corridor north of Cairn Point for estimating concept level pile sizes, capacities and embedment depths for bridge piers. The findings can then be used for completing updated construction costs for the bridge. The data will also be useful predesign information for performing follow-on planning, feasibility, and alignment studies. Also included are a preliminary ground response analysis of the site, a brief liquefaction evaluation, and a preliminary embankment stability evaluation of the soils along the two mile highway corridor on the eastside shoreline between the proposed east bridge abutment and the Port of Anchorage.

For this study, subsurface cross-sections were developed from the drilling, testing, and reconnaissance to represent our interpretation of subsurface conditions across this part of Knik Arm and along the east shoreline to the Port of Anchorage. This work included sixteen borings, two cone penetration tests, shear wave velocity measurements at one location, and Golder's geophysical survey. Additionally, a brief reconnaissance of both bluffs was carried out to highlight slope conditions and determine the soils potential for borrow material for causeway construction at the bridge ends.

The 200- to 300-foot borings showed that the channel is made up largely of hard clay-like and very dense fine sand deposits altered by glacier action, large tide changes, and strong currents. In the middle of the channel, the soils, in descending order, comprise about 20 to 40 feet of loose to medium dense marine fine sands, and 150 feet or more of dense to very dense fine sands and very stiff to hard silty clays overlying slightly gravelly silty clays as the basement material. Locally, these deeper clays are hard, registering standard penetration resistance values in excess of 100 blows per foot (bpf). The geophysical survey results in Golder's 2004 report, indicate that the hard basement clays are relatively thick below the borings and eventually reach sands and gravelly soils with bedrock being greater than 600 feet below the channel.

In the offshore areas, foundations to support the bridge piers at the selected crossing points would be constructed in over 100 feet of water in the middle of a one mile wide channel area, and must extend through the thin, weaker soil units, and derive foundation support in the deep, underlying glacial deposits. For these conditions, and to accommodate a reasonable water clearance, bridge piers below water are tentatively envisioned to be a group of six or more large diameter pipe piles driven 60 to 250 feet below the mudline and deriving support in both skin friction and end bearing. The pile cap or pile tops for each pier are envisioned to be near the mid elevation of the tide range and protected from ice forces with a cone shaped cover or jacket.

For purposes of estimating costs in this concept-level geotechnical study, 4- and 8-foot-diameter pile piles were analyzed and the results indicated that the above embedment depths ultimate axial pile capacities of 10,000 to 15,000 kips and 3,000 to 5,000 kips are possible for these two pile sizes, respectively. Actual tip elevations to achieve these capacities at various locations in the channel are summarized in the profile in Figure 11 of this report. This figure provides an easy method for estimating pile lengths at concept pier locations and/or determining total piling footage and its approximate cost. Other foundation designs should be evaluated during the design stage to develop the most cost effective design.

From these studies, we generally concluded from limited drivability studies and local experience that deep penetration of these large piles using large hammers is possible in these dense or hard glacial units, but higher than normal driving stresses and boulder obstructions are possible and may require thicker walls and high strength steel in the piles. Of particular concern is a shallow very dense till-like gravelly cap that will have to be penetrated in some parts of the channel as well as a few gravelly zones or local boulders. Our preliminary studies suggest that 1- to 1.5-inch and 2-inch wall thickness should be appropriate for 4- and 8-foot-diameter piles, respectively, to penetrate into or through these very dense layers with possible variable or less thick walls in other areas.

Additional borings at each pier have been recommended for final design to define subsurface conditions along a final preferred alignment and refine the conclusions reached in this concept level study. As the design evolves, follow-on studies may reveal that a test pile program may prove to be a cost-effective way to evaluate soil/pile setup characteristics, refine wall thickness requirements, confirm that suitable capacities and embedment can be achieved in these dense/hard soils using large hammers, and serve as a demonstration project to pile contractors of the difficulties of driving large piles in these compact/gravelly soils. This latter effort, if the added costs can be justified, will remove much of the guesswork in pile driving and should lead to lower construction bids for the production piles.

A preliminary ground response analysis was conducted at the bridge site for conceptual bridge design. The analysis was based on the shear wave velocities measured at the site and in the vicinity, regional probabilistic ground motion hazard studies and Uniform Hazard Spectrum (UHS) by the United States Geological Survey (USGS), and a single earthquake time history, representing a near-by shallow crustal earthquake that was spectrally matched to the UHS. Based on the results of the preliminary site response analyses, the response spectrum prescribed by the American Association of State Highway and Transportation Officials (AASHTO) for Soil Type II, and the Anchorage AASHTO Soil Type II spectrum is appropriate for conceptual bridge design.

Our boring data reveals that the soils in the channel crossing are generally dense to very dense or very stiff to hard and, as such, are not susceptible to liquefaction or strength losses with one minor exception. The recent sediments in the center of channel and are loose to medium dense fine sands in the upper 30 to 40 feet and in all probability will liquefy and lose their strength during strong earthquake shaking. Since the skin friction of piles penetrating loose sands at shallow depths is small, liquefaction of this thin unit will not seriously impact the total axial carrying capacities or estimated lengths of the piles themselves. The temporary loss of strength in this shallow unit will, however, cause reduced lateral support and force added stiffness in the pile to transmit these loads to the deeper deposits.

Borings drilled along the east shoreline between the east abutment and the Port of Anchorage reveal generally stiff to hard gravelly clays, and silty clays or very dense, silty sands at shallow depths. Preliminary calculations reveal adequate bearing support and slope stability for 2 Horizontal to 1 Vertical (2H:1V) fill slopes and embankments to at least 15 feet high over the mudflats to elevate the approach highway above the high tide line. The foundation soils are likely stronger than a granular embankment fill making the fill the weak link in the stability analyses. Additional more in depth explorations will likely be required for final design to confirm the above conclusions.

Approach causeways are planned at both ends with the objective of shortening the bridge length to reduce construction costs. As discussed in the report, the soils in the tide zone are dense or hard and generally suited for support of high embankment fills. Embankment reinforcement techniques such as geotextiles may be evaluated during the design stage of the project. Future hydrology studies are, however, recommended to refine the feasible causeway lengths and scour/deposition characteristics for the causeway and bridge piers, before final geotechnical studies can be developed with foundation recommendations.

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ABBREVIATIONS & ACRONYMS

°F	Degrees Fahrenheit
AASHTO	American Association of State Highway and Transportation Officials
ADOT&PF	Alaska Department of Transportation & Public Facilities
API	American Petroleum Institute
bpf	Blows per foot
cm/yr	Centimeters per year
CPT	Cone Penetration Test
FS	Factor of safety
Golder	Golder Associates
H	Horizontal
HDR	HDR Alaska, Inc.
HLA	Harding Lawson Associates
km	kilometer
ksi	Kips per square inch
LRFD	Load and resistance factor design
m	meter
Mat-Su	Matanuska-Susitna
MLLW	Mean Lower Low Water
mm/yr	Millimeters per year
NCEER	National Center for Earthquake Engineering Research
NOAA	National Oceanic Atmospheric Administration
OD	Outside diameter
PB	Parsons Brinckerhoff Construction Services, Inc.
PDA	Pile Driving Analyzer
PSHA	Probabilistic Seismic Hazard Analyses
psi	Pounds per square inch
RMDT	Robert Miner Dynamic Testing, Inc.
SPT	Standard Penetration Test
tsf	Tons per square foot
UHS	Uniform Hazard Spectrum
USGS	United States Geological Survey
V	Vertical
ybp	Years Before Present

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical study conducted in the vicinity of the Knik Arm where a bridge crossing is tentatively planned. The location of this alignment is shown on Figure 1. This is the fifth geotechnical study of this area and comprised of drilling nine deep borings across the roughly 12,000-foot wide channel supported by two cone penetration tests (CPT), shear wave velocity measurements at one location and seven shallow on shore borings near the high tide line south from the east abutment to the Port of Anchorage.

It should be emphasized that this is a concept level study with limited explorations aimed at estimating pile lengths and project construction costs and is not intended for final design. After a preferred alignment is chosen, the information can be used as additional information and a guide for planning future explorations for final design of bridge piers, causeways, and the new shoreline road needed to tie the bridge structure into the existing road system.

To compliment this field exploratory and testing effort, an overwater geophysical survey was also conducted by Golder Associates (Golder) to further evaluate subbottom conditions in the channel crossing area and to the north. This combined geotechnical program was the largest of the prior studies and was moved up in the normal planning/design schedule to take advantage of a jack up platform that had been mobilized to the Anchorage area to study future development concepts at the nearby Port of Anchorage. This equipment permitted deeper borings to be drilled in these waters where strong currents and large tides would hamper drilling efforts from a floating platform.

The purpose of this work was to better understand the geology and subsurface conditions in the channel area north of Cairn Point so that pile lengths and foundation costs could be better estimated. The latest prior field study was conducted in 1984 to support siting and comprised the drilling of three shallower borings with overwater geophysics. These and other prior studies left large data gaps in defining the geology and estimating lengths for large diameter high capacity piles for the many overwater piers that will support this bridge. This lack of subsurface information forced an interpretation of this limited information, the results having a possible significant influence on the estimated construction costs. The focus of this current effort was to help define foundation conditions so that the construction costs could be refined. The information would also be available for use in the follow-on planning, predesign, and future design phases for the project.

The shallower borings were added to the current program to address embankment stability concerns especially any mudflats toe buttressing requirements for a new east shoreline approach highway from the Port of Anchorage.

1.1 Prior Studies

Four prior geotechnical studies of limited extent were performed north of Cairn Point in efforts aimed at evaluating various alignment crossings. These studies are listed in the references at the back of the report and include Dames & Moore, 1970, Alaska Department of Highways, 1970, Shannon & Wilson, 1971, and Harding Lawson Associates (HLA), 1984.

Geotechnical work on the Knik Arm Bridge project was started in the early 1970s and produced the first three reports above. Dames & Moore performed an overwater geophysical survey of this area of the waterway while the Alaska Department of Highways and Shannon & Wilson, Inc., conducted limited overwater drilling and shoreline reconnaissance activities respectively. The focus of these studies was an early attempt at identifying the most promising crossing locations rather than collecting data to establish foundation types or construction costs.

In 1984, HLA conducted more intensive studies at three potential locations, one starting on the shoreline near downtown Anchorage and two alignments further north up the Knik Arm beyond Cairn Point. The closest crossing studied is referred to as the Elmendorf Crossing and is generally situated about ¼ miles north of the current alignment on the west side and ¾ miles north of this alignment on the east shoreline. The location of the current crossing is shown on Figure 1. Three borings, HLA 4, 5, and 6, also shown on Figure 1, were drilled in the Elmendorf Crossing corridor to generally define subsurface conditions.

This 1984 information was then reviewed again in Parsons Brinckerhoff Construction Services, Inc./HDR Alaska, Inc., 2003, (PB/HDR, 2003) and revealed broad data gaps in the information when used for estimating the foundation requirements and construction costs. Each existing boring encountered different geologic units making it difficult to develop a subsurface cross section that could be used for estimating reasonable foundation costs. The intent of the follow-on study was to fill in these data gaps where the feasibility and construction costs could be revisited and would not have to rely so heavily on broad interpolations of limited subsurface data. The current alignment was shifted south of the Elmendorf Crossing to shorten the access roads and be less costly.

1.2 Current Scope

The 2003 technical review and bridge cost studies have narrowed the crossings to the single area generally shown in Figure 1. The current work effort was focused at this site and consisted of a field and laboratory program to collect site soils data followed by preliminary analysis of this information. The actual field work comprised drilling nine deep borings across the channel supplemented by CPTs at a few boring locations, shear wave velocity, measurements at one location, and seven shallow borings along the east side tide flats south to the Port of Anchorage. Also rod energy transfer studies were conducted during Standard Penetration Testing (SPT) with various length drill rods. The bulk of the field effort was on drilling and sampling the seven overwater borings using a jack up platform system that had been initially mobilized to the Port of Anchorage for expansion studies. The remaining nine abutment/shoreline borings were drilled near high tide using a local track-mounted drill to compliment the offshore borings.

To supplement this program, an overwater bathymetry/acoustical (geophysical) survey was added to better understand subsurface conditions between and below the borings. The results of this survey are contained in Golder's 2004 report.

Soil samples recovered during the above drilling were returned to our Anchorage soils laboratory for selective index, strength, and consolidation testing, as appropriate. The combined field, laboratory and geophysical results were then used to prepare subsurface profiles across the channel and along the east shoreline and evaluated the likely foundation requirements for a water crossing with a bridge and/or partial causeway in this vicinity of the Knik Arm.

The analysis, contained herein, is a concept level evaluation recognizing that the alignment can change and many of the bridge plans are not yet well developed. The focus of this analysis was primarily on the pile capacity vs. embedment depths for large diameter piles likely to be driven for pier support. These limited analyses provide for both identifying pile driving issues and estimating approximate pile lengths for the various bridge piers across the channel for cost estimating purposes only.

As a part of the analysis effort, a limited site-specific ground response analysis was conducted using the measured shear wave velocities, boring data, and the computer program ProShake (EduPro Civil Systems, 1999). The results of the site-specific analyses were then compared to an American Association of State Highway and Transportation Officials (AASHTO) code based spectrum to aid in estimating the lateral loads on the bridge piers during a major future seismic event. Additionally, the liquefaction potential and its impact on project development are briefly addressed.

Since softer shoreline muds and loose fills were encountered at the Port of Anchorage and had to be stabilized with large earth toe buttresses, limited stability studies were also completed using our findings in the shallower borings to evaluate the impact of embankment loading on the foundation soils along the east shoreline.

1.3 Report Organization

This data report is organized into eight main sections. Section 1 is introductory in nature consisting of general information regarding the project, prior studies, the current scope of work and the authorization and limitations of our studies. Sections 2, 3, and 4 contain general descriptions of the site and project and a brief summary description of the field explorations and laboratory tests performed. Details of this work are provided in the appendices.

Section 5 is devoted to a summary discussion of the geology, tectonics, and seismicity of the area. This information is also discussed in PB/HDR, 2003. This section is followed by Section 6, which gives a description of the subsurface conditions based upon the exploration and testing program and Golder's geophysical survey (Golder, 2004). Backup logs and test results for this latter section are contained in the appendices.

Section 7 summarizes the results of our limited geotechnical analyses including pile capacity and embedment results, pile drivability concerns, the ground response analysis findings, liquefaction, and shoreline embankment stability. The final section, Section 8, is a brief discussion of recommended additional explorations and studies.

Seven appendices accompany the main text and figures. Appendix A contains the results of our geological reconnaissance including a general description of bluff slope conditions with photographs. Appendix B provides discussions of the major field drilling and sampling work including on and offshore drilling and sampling equipment and procedures. Also included within this appendix are the results of current and prior drilling efforts including 21 detailed boring logs. The results of CPTs at two boring locations and shear wave velocity measurements are contained in Appendices C and D respectively, and include cone logs and a velocity depth profile.

Appendix E summarizes the results of measured energy transfers from the surface SPT hammer to the sampler for various rod lengths. Appendix F describes the laboratory test procedures on the recovered soil samples and the results. The focus of this testing was on evaluating soil shear strengths for pile design, although basic index and a few consolidation tests were also performed.

Appendix G contains pile capacity vs. embedment depth plots for two pile sizes at each of the nine borings drilled to define the soils beneath the channel crossing. This backup data was used to compile the summary profile (Figure 11) for determining pile lengths in the main report. The last appendix, Appendix H, contains important information about your geotechnical report and is intended to aid the planners and users in understanding the use and limitations of our geotechnical work.

1.4 Authorization

This work was performed in general accordance with our Subconsultant Agreement dated August 15, 2003, with subsequent amendments aimed at completing an expanded work scope. PB, the prime consultant, and the project representatives from the Alaska Department of Transportation and Public Facilities (ADOT&PF) approved the general scope of the geotechnical work at several meetings held at the start and as the work progressed.

1.5 Limitations

The subsurface conditions described in this report were extrapolated and interpreted from our explorations and testing along a tentatively proposed water alignment and shoreline corridor and are assumed to be typical of the subsurface conditions throughout the channel and east shoreline, i.e., the subsurface conditions along other sections to either side of this alignment or corridor are not significantly different from those disclosed by the geotechnical studies completed to date. It is possible (and likely) that some of our measured sediment thicknesses and material properties may vary over time due to ongoing scour and future possible alignment shifts as a final alignment is being selected. The strong currents induced by the large tide changes will also likely result in continuous scouring and deposition of the main channel with bottom vertical changes.

Unanticipated soil conditions are commonly encountered and cannot fully be determined with a limited exploratory program. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, we recommend that this above information be used for its intended cost, and planning purposes, and that for final design additional site specific explorations and testing be conducted as deemed appropriate once more precise alignments and pier locations and spans are established.

2.0 SITE AND PROJECT DESCRIPTION

The Knik Arm Bridge Project is intended to provide a connection between the west and east shore of the Knik Arm, and a two mile stretch of undeveloped east shoreline to the Port of Anchorage.

2.1 Site Description

Anchorage is located in Southcentral Alaska, is the largest city in the state, and accounts for nearly half of the state's population. The Port of Anchorage and the Ted Stevens Anchorage International Airport serve as major transportation hubs for goods entering Alaska and/or serving the Pacific Rim countries. The shorelines of Anchorage are characterized as large mudflats in the intertidal zones and 50 to 150 foot high bluffs above high tide. Similar conditions exist in Knik Arm and at the proposed crossing area. A site map showing the tentative crossing area, the onland and offshore topography, and the locations of borings drilled in the area is presented in Figure 1.

Knik Arm is an approximately 34 mile long narrow water body that is orientated approximately northeast by southwest. It is a 1.6- to 5-mile wide waterway that is characterized by strong currents, deep water, and large tides, as well as strong winds, winter storms, and sea ice. The water is also murky with glacial silt making visibility for divers and construction workers limited to a few feet or less. These work conditions make construction of a highway bridge across this channel one of the more challenging projects in Alaska.

As shown in Figure 1, the tentative crossing on the east side is about a mile north of Cairn Point and merges with Point MacKenzie Road on the west with an overwater distance of about 2 ¼ miles (about 11,900 feet). The width of the channel at Cairn Point is only about 8,500 feet, however, the water depths, based on Figure 2, are about 100 feet deeper than at the proposed crossing location. The Figure 2 National Oceanic Atmospheric Administration (NOAA) map showing bottom contours in the chosen alignment reveal maximum water depths of about 130 feet at extreme high tides or a mudline of roughly Elevation –96 feet (Mean Lower Low Water [MLLW] Datum). This bottom elevation is deeper than suggested by the contours in Figure 1. We understand that NOAA is currently studying changing bottom conditions in Knik Arm by comparing old and new soundings taken over many years and that a report on this important issue should be coming out in the near future.

Tides, as noted above, are large and range between Elevation +34.1 feet (MLLW Datum) at high tide and –6.1 feet at low tide for a total maximum change of 40.2 feet. MLLW has been taken as the project elevation datum and is used throughout most of the remainder of the report.

The corresponding high and low elevations for Mean Sea Level Datum are +17.6 and –22.6 feet, respectively.

Currents created at tide changes vary with the location in the channel and when, during the tide cycles, the measurements are made. During our explorations, maximum currents of 5.6 knots (9.5 feet per second [ft/sec]) on the ebb tide were measured at the water surface at one location. At most other locations, more typical maximum values were in the 4 to 5 knot (6.8 to 8.4 ft/sec) range. When comparing our flow measurements with those found in the tide tables during our explorations, we generally found that the published flows (Nobeltec Tides & Currents, Version 3.0 software) were about 1.5 knots (2.5 ft/sec) too low. These published comments are based on NOAA predictions from gages situated south of the study area. Since our exploration work was conducted when the tide changes and currents were lowest, higher values, approaching 8 to 9 knots (13.5 to 15 ft/sec) are possible in our opinion or values higher than reported by the Corps of Engineers and estimated in PB/HDR's 2003 report.

2.2 Geography

Knik Arm in part is a glacially scoured valley. The local topography above high tide consists of high near-vertical cliffs along much of the coast including both sides of Knik Arm with intertidal mudflats reaching about ¼ and ½ miles seaward on the west and east sides, respectively, at extremely low tides. Based on our reconnaissance survey in Appendix A and the photographs in Figure 3 the east and west bluffs are roughly 70 and 100 feet high, respectively. These bluffs are both part of the Elmendorf Moraine or gravel deposits left as a result of prior glacier retreats.

From limited reconnaissance of both bluffs and as shown in Figure 3, they appear to be in a state of marginal stability as erosion from tides and strong currents seem to be slowly cutting away at the toe of slopes on both sides of the Knik Arm. This toe erosion results in progressive slumping and/or sloughing of the bluffs, the tailings of which are eventually washed away at high tides or waves again exposing the toe for more erosion. Freeze thaw effects, infrequent strong earthquakes, and bank seepage also appear to contribute some to this erosion process. Bank regression studies of the Anchorage bluff at Point Woronzof indicate an average regression rate of about 2 feet per year. Similar regression rates probably occur here as the sea face exposure, slope heights and bank materials appear similar.

As shown by the offshore topographic contours in Figures 1 and 2, the channel bottom at low tide continues seaward at a gentle grade for some distance or as a inclined terrace before dropping rather abruptly on both sides (particularly the west submarine bank) to form an approximately one-mile wide main channel. Our current soundings during our offshore drilling program, the Figure 2 NOAA 2001 contours, and the Sheet 2 Geophysical Survey (Golder, 2004) confirm that these steep banks do exist.

The climate is predominantly cool maritime with mild winters and cool summers. Average annual precipitation is about 15 inches. Strong winds are common especially in winter and cloud cover is persistent. Average annual temperature is about 38 degrees Fahrenheit (°F) with a mean January temperature of about 35°F and a mean August temperature of almost 59°F.

2.3 Project Description

The purpose of the Knik Arm Bridge project is to construct a hard link or bridge from Anchorage northwest over the Knik Arm to the Matanuska-Susitna (Mat-Su) Borough where the road will interconnect with the existing roads. On the east side the bridge will pass to a causeway and then to an embankment fill which parallels the undeveloped shoreline south about 2 miles to the Port of Anchorage and existing roads in the area. For this report, only the water crossing of Knik Arm and to a limited extent the embankment south to the Port were studied.

For cost estimating purposes, the location of the bridge shown on Figure 1, was generally established far enough north to avoid the deep waters at Cairn Point and to avoid the need for a high level bridge that would require extra clearance for future shipping traffic. This boat traffic may serve the existing Port MacKenzie Dock; a future planned deep water dock and/or a potential ferry landing at this location. As shown in Figure 2, the Port MacKenzie dock is situated roughly a mile south of the bridge's west abutment.

Cost update studies were completed in PB/HDR's 2003 report to establish technical feasibility and the relative costs of different sizes of bridges. Various bridge options with and without gravel approach causeways at the ends were studied along with two-, three-, and four-lane steel and/or concrete bridges with and without a rail line over the bridge. Provisions for a railroad on this bridge are not given. However, we understand that the future design may include railroad compatibility as to vertical and horizontal control.

All options studied were determined feasible; however, the costs varied considerably, one sensitive factor being the cost of the bridge's pier foundations. For lack of adequate geotechnical data, conservative foundations had to be assumed in the cost evaluation which may have penalized all of the options and made the validity of the estimate questionable. This geotechnical study was undertaken to eliminate or refine this unknown in the cost estimate. The scope of this work is generally outlined in Section 1.2 and in the Executive Summary. The results are presented in the text and appendices, which follow.

3.0 FIELD EXPLORATIONS AND TESTING

The geotechnical evaluation involved the five specific field tasks listed below:

1. Geological reconnaissance of both bluffs,
2. Drilling and sampling of nine deep and seven shallow borings,
3. Select CPT Testing at two locations,
4. Downhole shear wave velocity measurements at one location,
5. SPT energy transfer tests on drill rods at six points in two borings.

An overwater geophysical survey was also completed as a sixth task in support of these studies. This work was performed by Golder Associates and the results are presented in Golder's 2004 report. The scope of each of these efforts is briefly described below while detailed procedures and the results of the first five tasks are contained in Appendices A through E.

3.1 Geological Reconnaissance

A geologic reconnaissance of the bluffs was conducted along the shorelines in the study area on September 8 and October 28, 2003. The purpose of the reconnaissance was to classify the existing soil near the proposed locations of the new bridge abutments. This data will be used with on and offshore borings in each area to develop a generalized subsurface profile along the alignment and to assess bank material as a potential gravel source for causeway construction and/or future road embankment or prism materials.

From recent borrow source studies by our firm for the Mat-Su Borough on the west side bluffs, we know that fine gravelly sands exist in the upper reaches of the bluff (Shannon & Wilson, 2003). It is in a very dense state, moist and often has less than 10 percent fines (material passing the No. 200 sieve). We also found that this unit contains silty zones with fines reaching 30 percent or more at some depths, but the broad sampling interval (often 20 feet) did not provide a good feel for the amount of high quality granular soils. Our sampling and laboratory tests reveal that there are granular soils with low fines near the bluff, but the extent of areas with higher fines in these better quality soils is not yet well defined. Our borings drilled at elevations of approximately +115 to +310 feet found the sand to be about 50 to 60 feet thick before encountering more silty soils at depth. Much of this material generally meets the requirements for a Select Type A, B, or C fill according to the ADOT&PF Standard Specifications for Highway Construction.

The results of our reconnaissance of each bluff are summarized in the two photographs in Figure 3 and described in detail in Appendix A of this report. Presented with the Appendix A text are additional photographs depicting surface conditions at each bluff site and nearby exposures. In summary, Figure 3 exposures comprise clay and silty sands with varying amounts of gravel in both bluffs. Both of these areas have material with excess fines that are not well suited for reuse as fill in a water area. The granular soils contain thin clay interbeds that prohibit mining of the soils without intermixing it with clay fines. In other words, the immediate bluff materials are dense and/or hard and can be cut and will stand on steep slopes, as noted in the photographs, but would not be suitable for constructing below-water portions of causeway embankments.

3.2 Borings

Sixteen borings, designated A-1 through A-17, but excluding A-3, were advanced to define the subsurface conditions at the proposed crossing alignment and along the interconnecting road embankment south to Anchorage. Seven of the borings were drilled from a jack up platform using rotary and wireline drilling methods while the remaining two abutment and seven road embankment borings were advanced on the mudflats near high tide using a Nodwell track-mounted rotary drill and hollow stem augers.

The drilling work was performed between August and October 2003, with the center borings in deep water, A-2, A-5 and A-10, being advanced when the tides were most favorable for this work (i.e., when high tides and currents were lowest during the drilling schedule). As noted on Figure 1, some of the center borings were not drilled along the profile as they had to be shifted north to shallower water in order to safely set up the platform and maintain its stability during high tide and strong currents. Water depths at the seven overwater locations ranged generally from about 30 to 75 feet. The greater water depths were accommodated by planning and implementing these deeper water borings when the high tide elevations and currents were lowest during the drilling program.

The locations of the 16 current borings and 5 prior borings are shown on the boring location plan in Figure 1 and on the subsurface profiles, Figures 4 and 5. Figure 4 represents subsurface conditions in the crossing area while Figure 5 depicts conditions along the east side road embankment area. The individual logs of all 21 borings are presented in Appendix B. Also presented in this appendix is a detailed description of drilling and sampling procedures for both on and offshore work.

3.3 Cone Penetration Testing

CPTs were performed at two boring locations (A-1 and A-5) to further evaluate the properties of the soils particularly the uniformity of the thick silty clays and fine sands and their relative strength or density properties. The CPT measurements were conducted by Gregg In Situ, Inc. Gregg In Situ frequently works with Gregg Drilling on this type of overwater work. In general, the upper layer (15 to 20 feet) of soil was drilled to create a cased hole to support the CPT rod as it was pushed beyond these depths into the underlying clays or sandy soils. When the casing was set and drilled out, the penetrometer tests began. The cone used for this study was a 2.3 in^2 (10-cm^2) standard electronic cone.

The tests consist of pushing an instrumented, 60 degree piezocone in the soil at a constant 0.8 inch per second rate. The resistance to continuous penetration encountered by the cone tip, and a 31-$inch^2$ sleeve in the cone tip are transmitted electronically through the push rods into a portable data acquisition system at the surface. A pore pressure element is located behind the tip, just in front of the sleeve. During the test, the data was graphically displayed in color on a computer screen showing tip stress, friction stress, pore pressure, and slope plotted against depth. Logs from the cone penetrometer tests and the measured piezometric data are included in Appendix C along with calculated friction ratios, relative strengths, and equivalent Standard Penetration Resistance (or uncorrected N) values and $N_{1(60)}$ values. The $N_{1(60)}$ values are equivalent corrected blow counts (for confining effects) and an assumed 60 percent energy transfer down the drill rods to the sampler.

3.4 Shear Wave Velocity Measurements

While pushing the cone adjacent to Boring A-5, the cone advance was stopped every 5 feet for making shear wave velocity measurements. These measurements are made using conventional downhole procedures and provide useful velocity information for performing a follow on ground response analysis and developing a site specific response spectra. In this test, the energy source, provided and operated by Statewide Blasting and Perforating from Eagle River, is placed on the bottom of the seafloor near the top of the starter casing set to advance this CPT. When advance of the cone is stopped periodically, a blasting cap is fired from the source sending P and S waves into the ground and past a velocity transducer or geophone attached near the CPT tip. The interval travel times of both waves can be measured between the source and geophone at different depths enabling the interval shear wave velocities to be calculated every 5 feet. The results of the velocity changes with depth from these measurements are presented in Appendix D.

3.5 Standard Penetration Test Rod Energy Transfer Measurements

Since the borings drilled for this study were in excess of 100 feet and from a jacked-up platform over 70 feet of water, Standard Penetration Tests had to be performed with very long and large diameter rods. The rods were made of 3.5-inch outside diameter (OD) pipe with 0.188-

inch walls compared to the smaller standard N-rod while rod lengths reached over 300 feet in the deeper borings. Additionally, a normally higher efficiency auto trip hammer was used in lieu of the often still-used standard rope and cathead. These changes from the on-land SPT procedures, from which all of the N-value empiricisms are based, led to concerns about the validity of the recorded blow counts and what corrections were appropriate for determining the density of deep cohesionless soils or the consistency (stiffness) of the cohesive soils.

To evaluate these results, Robert Miner Dynamic Testing, Inc. (RMDT) from Seattle was retained to model the rods as if it were a pile being analyzed using Pile Driving Analyzer (PDA) technology. For this work, a five foot top section of rod was instrumented with four strain sensors and four accelerometers to measure the energy transfer directly from the automatic 140 pound hammer to the rods. Through PDA CAPWAP analysis, the energy transfer from the hammer to the rod tip or the SPT sampler was calculated. **CAPWAP** is a signal matching program that uses the measured force and velocity from a Pile Driving Analyzer. Actual field measurements were completed at six depths while taking SPT samples in Borings A-6 and A-10.

The results of these measurements are summarized in Table 1 while RMDT's full report, with calculations is included as Appendix E. The report shows an efficiency of about 83 to 89 percent (average 85 percent) energy transfer at the top of the rods. The report also shows relatively constant energy losses with depth or transfer efficiency drops of about 1.0 to 1.2 percent for each additional 10 feet of rod. Two hundred twenty feet of rod length would have an energy drop of about 1.1 percent x 22 rods + 15 percent, or about 40 percent total or closely approaching the 60 percent system efficiency normally used for on land borings taking SPTs N_{60} values with a cathead and rope (i.e., the increase in efficiency of the auto hammer offsets the energy losses due to rod length and increased diameter at about 220 feet of rod or roughly Elevation -170 feet). This means that above about Elevation -170 feet, the measured N values by the current method are too low and need to be increased to match normal N_{60} values. Correspondingly, below Elevation -170 feet, the measured values are too high and have to be lowered to reflect normal N_{60} values.

3.6 Geophysical Survey

From our overwater field exploratory work, it became apparent that the actual contours on Figure 1 were old, extremely inaccurate, and generally deeper than anticipated. Other measurements by two prior geophysical studies (HLA, 1984, and Dames & Moore, 1970) and the 2001 NOAA charts in Figure 2 likewise showed different depths, suggesting a need for current bathymetry information recognizing that the bottom conditions in the channel were likely change with time. Additionally, the complex and abrupt changes in soil units between borings along the alignment and those offset to the north indicated that geophysics may provide insight into these conditions.

An overwater geophysical and hydrographic survey was conducted at the site in November 2003, by Golder to establish current bottom contours in the area, determine sediment thicknesses, and define the continuity of major soil units, to the extent possible. It was hoped that the abrupt changes in soil types between borings could be understood to build an even better Profile A-A' in Figure 4 and delineate what lies below the borings for support of our ground response analysis. This work comprised hydrographic and acoustical subbottom profiling from a survey vessel using both bathymetric and acoustic profiling equipment.

The results of this survey are presented in Golder's 2004 companion report titled *Knik Arm Geophysical Investigation*. The focus of this survey was along the alignment shown in Figure 1 and four additional crossing areas and five longitudinal lines to the north. These lines were conducted to determine whether better foundation conditions could be expected in this area with a slightly different crossing alignment. The survey was not extended south of the study alignment because the deeper water and steeper subbottom slopes in this area would likely result in increased construction difficulties and costs. The reader is referred to this report for the detailed results from this survey. Select technical information from that study has been used to assist us with our preliminary analyses and both bottom elevations and geophysical interpretations of the continuity of units between borings have been incorporated into Figure 4.

4.0 LABORATORY TESTING

Laboratory tests were performed on selected soil samples from the borings to determine the pertinent physical characteristics and engineering properties. The laboratory testing program on the soils was formulated with special emphasis on the determination of their strength properties for estimating pile capacities. Additionally, index tests namely moisture contents, gradation and Atterberg limits together with a few consolidation tests were conducted to better establish the behavior characteristics of these soils. The parameters from these tests, combined with visual examination of the sample's consistency during drilling, the penetration resistance values from the Standard Penetration tests, and other field measurements provide the information needed for our preliminary engineering analysis of the soils.

Results of the soil tests performed on samples from each boring are presented in Appendix F, together with a brief description of each test. The results of these tests have been used to define the physical characteristics of the major soil units discussed in Section 6, "Subsurface Conditions."

5.0 LOCAL GEOLOGY, TECTONICS AND SEISMICITY

5.1 Local Geology

Approximately two major glaciation events have occurred in the upper Cook Inlet within the last 75,000 years. During the Knik Glaciation (30,000 to 75,000 years ago), thick sequences of sediment, known as the Knik Ground Moraine, were deposited as glaciers retreated. These deposits extend from the Eagle River Valley to Point Woronzof and can be observed in the Eagle River channel and south of Fort Richardson in the Anchorage Bowl area. During the time of deposition, fresh water lakes and ponds formed and were subsequently filled by peat and clay.

The Naptowne Glaciation (11,000 to 30,000 years ago) is responsible for the majority of glacial deposits currently encountered in the Anchorage area. At its maximum, the Naptowne Glaciation extended across the Anchorage Bowl area from the north and terminated at Point Woronzof and Point Campbell (Dilley, 2000). The Bootlegger Cove Clay was formed during this time in the ice-free areas of the basin starting around 18,000 years ago. Thick units of this clay were deposited throughout the upper Cook Inlet region. Prior to and concurrent with the deposition of the Bootlegger Cove Clays, material was being shed out of the uplifting Chugach Mountains through alluvial processes (Hamilton, 1994). Wedges of sand and gravel interfinger with and underlie the clay in many areas.

Towards the end of the Naptowne Glaciation, meltwater from the Knik-Matanuska glacier flowed across the Anchorage area towards the south as large braided stream channels containing sand and gravel. These sands and gravels were bound to the northeast portion of the Anchorage basin by the glacier lobe and deposited as the Mountain View Fan, which underlies parts of Government Hill, Mountain View, and Downtown Anchorage (Dilley, 2000). This deposit overlies much of the Bootlegger Cove Formation.

Approximately 14,000 years ago, the Knik-Matanuska lobe of the glacier retreated to roughly the location of the present day Eagle River and remained in that location for the next 2,000 years (Hamilton, 1994). During this time, large amounts of material were shed from the retreating glacier and subsequently formed the Elmendorf Moraine. The Elmendorf Moraine blocked drainage of the ancestral Eagle River creating a large lake within the lower part of the valley. Major deposition ended when ancestral Eagle River cut through the Elmendorf Moraine and drained the bound lake (Dilley, 2000).

By about 11,500 years ago, glacial ice had retreated approximately 30 miles up the Knik Arm and Anchorage was ice-free. By 10,000 years ago, many mountain passes were ice-free and glaciers were near their present locations (Hamilton, 1994). Since this time, glaciers have fluctuated slightly with small surges and retreats. The waters of Cook Inlet have risen in response to a worldwide sea level increase due to melting glaciers and have subsequently flooded the valley of the Knik-Matanuska River system creating Knik Arm.

5.2 Tectonics and Seismicity

The project region is one of the most seismically active areas in the U.S. and historically subjected to large earthquakes. More than 5,000 small and large earthquakes have been reported in the Alaska region since 1898. A list of earthquakes according to depth was obtained from the Alaska Earthquake Information Center, the locations of which are plotted on Figure 6.

The seismicity and tectonics of southern Alaska is the result of ongoing relative motion between two lithospheric plates; the Pacific plate moves about 5 to 6 centimeters per year (cm/yr) northwestward relative to the North American Plate. This relative motion between plates results in two different styles of deformation in southern Alaska. Along the Alaska panhandle and eastern margin of the Gulf of Alaska the horizontal relative movement between the plates is accommodated primarily by high-angle strike-slip faults. Along the northern margins of the Gulf of Alaska (including the Kenai Peninsula and extending westward parallel with the Aleutian Islands), the convergent relative motion between plates is accommodated by underthrusting of the Pacific plate beneath the North American plate.

This underthrusting forms a northwestward-dipping subduction zone and, from compression in the overlying crust, results in folds, high-angle reverse faults, and thrust faults. The Aleutian trench marks the surface expression of this subduction zone and is located approximately 186 miles (300 kilometers [km]) southeast of Anchorage.

Within this tectonic framework, four broad seismogenic sources of historical seismicity are identified, namely:

- Strike-Slip earthquakes on or associated with the transform boundary between the North American and Pacific Plates along the eastern margin of the Gulf of Alaska.
- Interplate or megathrust earthquakes between the North American and subducting portion of the Pacific Plate along the northern margin of the Gulf of Alaska.
- Deep intraslab earthquakes within the subducting Pacific plate (i.e., in the Benioff zone).
- Shallow crustal earthquakes within the North American Plate north of the Gulf of Alaska as a result of stresses induced by the plate interactions.

Due to their proximity to the site, the interplate, intraslab and shallow crustal sources are the most significant ground motion hazard sources for the site. These sources and associated seismicity are briefly described below.

5.2.1 Interplate Earthquakes

The interplate source extends from east of the Kenai Peninsula and west along the Aleutian Islands approximately 2,300 miles (3,701 km). The Aleutian Trench defines the up-dip extent. The down dip extent or width of the interplate source varies and is widest in the site region as the down-dip edge of the zone approaches to within a horizontal distance of approximately 18.6 miles (30 km) of the site. As shown on Figure 7, at this closest approach the depth to the interplate rupture is approximately 18.6 miles (30 km), HLA Associates, 1984. The largest historical earthquake to affect the site region was the M_W 9.2 1964 Alaska Earthquake, which ruptured the east end of the interplate zone beneath the Kenai Peninsula and Kodiak Island.

5.2.2 Intraslab Earthquakes

Deep intraslab earthquakes occur within the subducting Pacific plate (i.e., in the Benioff Zone) at depths of about 20 to 75 miles (about 32 to 120 km). This zone is parallel and extends north of the interplate source zone and is located beneath the site. A review of historical seismicity in the vicinity reveals that most of the seismic events are at focal depths of greater than about 20 miles (about 32 km). The depth of these events suggests that these are occurring in the Benioff zone below the crust.

5.2.3 Shallow Crustal Earthquakes

Shallow crustal seismicity has also been recorded in the site region. However, no shallow crustal earthquakes have been directly correlated with shallow crustal fault that has ruptured the ground surface.

Within the Anchorage region, two crustal faults have been identified that may present significant ground motion hazards at the proposed Knik Arm Crossing. They are:

1. The Castle Mountain Fault, and
2. the Border Ranges Fault zone.

Castle Mountain Fault

The Castle Mountain Fault has been mapped over a total length of approximately 295 miles (475 km). As shown in the upper map in Figure 6, the fault trends east-northeast/west-southwest approximately parallel to the northwest shore of the Cook Inlet. At its closest approach, it is about 25 miles (40 km) northwest of the site. Evidence of Holocene (11,000 years before present [ybp]) displacement has been observed along a 50 mile (80 km) long portion of the fault in the Susitna lowland north of Anchorage.

The fault displays evidence of both right-lateral strike-slip and reverse slip components. The north side is displaced upward relative to the south side along a steep, north-dipping fault plane. Slip during the Holocene Epoch on the Castle Mountain Fault has been predominately strike-slip with a component of dip-slip movement indicated by displacement of Holocene features and sediments. In the Susitna lowland, a Holocene sand ride is displaced 23 feet (7 meters [m]) in a right-lateral sense while near-surface sediments have been displaced vertically 7.5 feet (2.3 m).

Because there is no documented evidence for displacement along the Castle Mountain Fault during historical time, the maximum earthquake magnitude was estimated from available seismological and geological data. A magnitude M_s[(*)] 7.0 earthquake occurred in the vicinity of the Castle Mountain Fault west of Anchorage in 1933. Due to the poor accuracy of epicenter location at the time and a lack of surface displacement investigations, it is not known if the earthquake was related to the Castle Mountain Fault.

Using Slemmons (1982) relationship between maximum earthquake magnitude and source parameters (maximum surface rupture length, total length, fault area, or displacement per event), Woodward-Clyde Consultants (1982) have estimated the maximum magnitude for the fault to be about 7.5. Assuming an average slip rate of approximately 5 millimeters per year (mm/yr), they also estimate the average recurrence interval for a maximum magnitude 7.5 on the Castle Mountain Fault to be approximately 235 years. Wesson, et al. (1999) in their probabilistic ground motion hazard study for the State of Alaska also determined that a likely maximum magnitude for this fault is about 7.5, but they used a slip rate of 0.5 mm/yr to estimate a recurrence rate of 1,300 years.

[(*)] M_s surface wave magnitude that relies on the amplitude of the surface waves with periods of 20 seconds, which are recorded at great distances.

Border Ranges Fault

The Border Ranges Fault is mapped as a north-dipping reverse fault separating upper Paleozoic and lower Mesozoic rocks on the north from Upper Mesozoic and Tertiary rocks on the south. As shown on Figure 6, the Border Ranges Fault can be traced approximately 620 miles (1,000 km) northeastward from Kodiak Island, across the Kenai Peninsula, and along the northern front of the Chugach Mountains. At its closest approach, the Border Ranges Fault is about 7.5 miles (12 km) southeast of the site. The Fault is interpreted to be an ancient subduction zone (suture zone) of Mesozoic to early Tertiary age. The active subduction zone has since migrated southeastward to the Aleutian trench.

Geologic mapping in the southern Kenai Peninsula by John Kelley (1981) suggests possible reactivation by more youthful faulting along a small portion of the ancient Border Ranges zone. This would be consistent with the faulted basin margins and fore-arc tectonic model of the area as proposed by Dickinson and Seeley (1979).

Investigations pertaining to the activity of the Border Ranges Fault zone are still in progress and there is no means to directly assess its earthquake potential. However, if the Border Ranges Fault is part of the same tectonic system as the Castle Mountain Fault, then a similar maximum magnitude (i.e., M_s 7.5) and recurrence interval (i.e., hundreds to a few thousand years) could be likely.

6.0 SUBSURFACE CONDITIONS

The following discussion is based upon the field and laboratory results from 16 new borings, two CPT soundings, shear wave velocity measurements at one location, a reconnaissance of the shoreline bluffs, Golder's geophysical survey, and any pertinent prior boring data. The detailed results of this work are presented in Appendices A through F and Golder's 2004 report. Available bathymetry data for the channel area north of Cairn Point is very limited. In addition to the Golder study we identified one survey conducted as part of a previous Knik Arm Bridge study and a recent survey conducted the National Ocean and Atmospheric Administration (NOAA). This survey, conducted in August 2001 and titled Hydrographic Survey H-11031, has not yet been released to the public, although we were provided with preliminary information.

6.1 Channel Crossing Soils

The soils across the channel and in the bluffs are of glacial or marine origin and, except for near surface deposits in the main channel bottom, are generally dense to very dense or very stiff to hard. Channel Crossing Profile A-A' in Figure 4 illustrates our broad interpretation of the soils across this section of the channel. This profile shows both an expanded 1 Horizontal to 1 Vertical (1H:1V) scale to illustrate the changes in soil types with depth as well as a natural scale profile to show the actual flatter grades of the channel bottom. Figure 1 shows the location of this profile. The glacial geology in this area appears to be complex and has developed as a result of a number of glacier advances and retreats, scouring and redeposition as tills and in both glacial lake and marine environments, and consolidation of deposits due to glacier over riding. Based on surface exposures, these depositional characteristics are not only present below the waters of Knik Arm and the mudflats, but also exist in the steep bluffs on both sides of the channel.

As summarized in Figure 4, about four basic geologic units appear to have been penetrated with the deep borings and are summarized in descending order as follows:

1. Recent Channel Marine Deposits,
2. Glacial Till or Moraine Deposits,
3. Glacial Lake Clays or Marine/Alluvial Sands, and
4. Possible Knik Tills.

6.1.1 Recent Channel Marine Deposits

Up to 40 feet of loose to medium dense silty to clean fine sands are present in the center of the main channel as shown on Figure 4. Locally these loose sand deposits appear to thin on the east side to less than 10 or 15 feet and are absent on the west side as water depths diminish. We believe these are recent marine deposits that are somewhat mobile and tend to shift over time as sand dunes with the changing currents and tides. They are likely present on the east side because of slightly lower currents and flatter bottom slopes and absent on the west mudflats for the opposite conditions.

Measured uncorrected N values from the two borings (A-2 and A-10) that penetrated deeply into this deposit were between 5 and 10 blows per foot (bpf) with an average value of about 7 bpf. When corrected for rod length/auto hammer effects per Section 3.5 and Appendix E and confining pressure effects, the average corrected N value or $N(1)_{60}$ is about 10 or 11 bpf or a material which is boarder line between loose and medium dense. These properties and the generally low N values above indicate that the possibility of these recent deposits liquefying under strong earthquake shaking is high.

A shallow gravel cover has also been deposited on the mudflats near the eroding toe of the east bluff. This surficial unit is generally less than 10 feet thick and appears to be remnant particles eroded from the east bluff till-like soils, but are too coarse to be transported out of the area. The general lack of noticeable thicknesses of similar gravelly soils on the west side mudflats, but the presence of boulders and coarse gravel on the surface, suggest that these mudflat slopes are steeper and subject to stronger erosive forces than the east side. The east side beach gravel particles in Photograph 2 in Figure 3 are fine to coarse and rounded to subrounded indicating that they are of glacial origin, likely from the bluff tills.

6.1.2 Glacial Till or Moraine Deposits

This unit mantles much of the side channel bottom areas, extends back into and is exposed in both bluffs, but appears to have been eroded away in the center of the channel. Based on Figure 4 and the upgradient Borings A-5 and HLA 5, it is both thick and thin up and down the channel with estimated maximum thicknesses of over 100 feet, particularly near both abutments. Its general lack of apparent bedding or any well-defined structure suggests that it is a glacial till. In addition to its lack of structure, it is characterized as both a gravelly clay and sand because of its changing mixture of particle sizes noted on the grain size plots in Appendix F. It is locally a gravelly, silty clay, particularly on the east side and a silty, gravelly sand with thick gravelly clay zones or layers on the west side. Gravel is generally present in this material even though in small quantities compared to its finer grained matrix materials. It is also consistently very dense or hard with Standard Penetration Resistances generally in excess of 50 bpf and frequently in excess of 100 bpf.

Water contents range widely between 10.7 and 31.2 percent, and where it was cohesive, Atterberg limit tests show that it is a CL according to the Unified Soil Classification System (Appendix F, Table F-2) or has low plasticity characteristics.

More detailed soil descriptions and test results on this unit can be obtained from the reconnaissance photographs in Appendix A, the boring logs in Appendix B, and the laboratory test results on select samples in Appendix F.

This till mantle in our opinion is one of the stronger support soils at the site, but where thin, its pile carrying capacity will be limited, requiring that piles penetrate it to achieve the needed higher design capacities. Pile tip damage can also occur while attempting to penetrate this very dense unit or, where thick, its high density/consistency may make achieving a suitable minimum embedment difficult. Both these factors need to be considered in determining pile sizes, lengths, and wall thicknesses to handle the possible high driving stresses in these soils. Though rare, based on this limited drilling program, the possibly exists for encountering boulders and causing the piles to stop short of its intended tip depth or be damaged.

6.1.3 Glacial Lake Clays or Marine/Alluvial Sands

Once the upper till-like unit or the loose marine deposits are penetrated, the borings encountered a thick clay/sand deposit or probably the most dominant geologic unit beneath the channel. The average and ranges in engineering properties of these sands and clays are summarized in Table 2 or presented in detail in Appendices B and F. As shown on Figure 4, the sand is thin near the west end or absent, but thickens to over 160 feet to the east, and then changes into a 200- to 250-foot massive silty clay stratum over the eastern one third of the channel.

The continuity of the sand clay stratum across the site in Figure 4 were determined by placing the boundary results from the geophysical survey (Golder, 2004) on the profile, and performing minor adjustments in the sub bottom data to match conditions in each boring on the alignment. It shows that normal straight line interpolation methods between borings may not be an accurate representation of actual conditions in this case.

Inconsistencies in subsurface conditions were noted along the crossing alignment between Borings A-1 and A-2, when attempting to project the north offset HLA borings (HLA 4 and HLA 5) and Borings A-5 south onto Profile A-A’ in Figure 4 and match them with the geophysical results in Golder’s 2004 report. It appears that the boring north offset distances of 2,000 to 4,000 feet (see Figure 1) are just too great to obtain single cross section results that are meaningful at the current profile location. The logs of these three north offset borings as well as Boring A-10, are included as support data for interpretation, but deleted from Profile A-A’ in Figure 4 to avoid misrepresenting conditions in this vicinity. These extra boring data, however, suggest the following.

1. The alluvial sands in both geophysical surveys (HLA, 1984, and Golder, 2004) appear to lie in a well defined smaller channel than is suggested by projecting these northern borings onto Profile A-A’, particularly Boring A-5.
2. The borings along Profile A-A’ indicate that the alluvial sands and clays are covered by glacial tills on both sides of the channel while the geophysical results suggest that they are present over the clays, but stop at the edge of the alluvial sands.
3. Boring HLA 5 shows gravelly till soils in the upper 100+ feet where the alluvial sands would be expected on Profile A-A’ between Borings A-1 and A-2. This suggests that the till cap could be thicker than anticipated in the middle of the channel especially if the crossing alignment is moved further to the north.
4. Boring A-5 shows a thin till cap overlying 200’ of silty sands with silty clay interbeds, where HLA, 1984, suggests clays should be present and Golder’s 2004

report data indicates that the deeper Knik Tills are much shallower in this part of the channel section.

5. Boring A-5 drilled 2,000 feet north of Profile A-A' encountered a thick deposit of silty sands interbedded with thin very stiff silty clay layers. As discussed below, this together with similar consistency/density and index properties, suggests that the thick alluvial sands and glacial lake clays in Figure 4 were formed about the same time and the transition from sand to clay is more gradual than indicated by the geophysical data.

These above results support a conclusion that the geology is complex and not well defined in this channel, but our borings, summarized in Figure 4, support that 1) the shallow tills overlie both the deep alluvial sands and glacial lake clays in the channel, 2) the alluvial sands are more extensive and (from A-5 results) may form a wider and deeper channel than suggested by the geophysics, and 3) the Knik Tills are much deeper over the eastern one third of the channel.

This unit is distinguished from the till-like soils by its general lack of gravel particles with the exception of a few gravelly zones.

a. Alluvial Sands

The sand is classified as a dense to very dense, gray, clean to silty fine sand generally grading into a silty sand or sandy silt to the east. From gradation and limit results in Appendix F, Figures F-1 and 2, the Unified Soil Classification symbol of the fine sand is largely an SP or SP-SM and the silty sand to sandy silt an SM or ML. Locally at depth the fine sand appears to be deposited as a glacial rock flour, and except in the sand/clay interbedded zone noted in Boring A-5 seldom exceeds 20 percent fines, has little apparent cohesion, and is nonplastic in many cases.

Cone data were taken adjacent to Boring A-5 to check the density and uniformity of the sands. Measured CPT tip resistances in the upper 100 foot depth range were relatively uniform and generally between 40 and 50 tons per square foot (tsf) increasing to 70 tsf below. Friction ratios values are about 1 percent (unnormalized) and 2 percent or slightly more (normalized) which is typical of a granular soil. The inferred soil behavior classification, based on the CPT data, is silt sand and sand using non-normalized data and silt mixtures using normalized data. As noted below, when compared with the CPT A-1 measurements in clay soils and average properties in Table 2, the differences in strength and index properties seem to be minimal and the results amazing similar further supporting that the clays and sands are from the same geologic unit deposited about the same time under only slightly different conditions.

The average density properties of this granular unit are best taken from Borings A-2, A-5 and A-10 as each penetrates a thick part of this unit. Uncorrected N values plotted with elevation for the sands are plotted on Figure 8 along with the calculated N(60) plots from the CPT measurements. The (60) indicates that a 60 percent energy loss in hammer/rod efficiency was assumed in the calculation while the data in Section 3.5 and Appendix E indicates it is more in the 70 to 80 percent range. None of the N values are reduced to N(1) for confining pressure influences.

The CPT N(60) data and the A-5 N values in Figure 8 in our opinion reflect lower values for the sands because of higher real transfer energy than used in the CPT calculations and both are in the siltier or interbedded sand and clay deposits in Boring A-5 area where lower N values should be expected. Figure 8 also reflects a consistent increase in density with depth and with corrections applied will probably still show that most of the fine sands below about Elevation -130 or roughly 70 feet below mudline are mostly near the borderline of dense to very dense becoming very dense with depth.

The average shear velocities were about 1,135 ft/sec in the siltier sands with clay interbeds at Boring A-5 and are probably several hundred ft/sec higher in the more massive sand unit found in the center of the channel.

b. Glacial Lake Clays

The clay beneath the eastern part of the channel is classified as a stiff to hard, gray, silty clay with generally low plasticity characteristics. Typical shear strengths vs. elevation for this clay unit are summarized on Figure 9. This figure provides a summary plot of the laboratory shear strength results including unconfined compression tests, triaxial tests and pocket penetrometer measurements and generally shows consistent strengths with depth with most values falling in the 2 to 5 kip per square foot (very stiff to hard) range with slightly lower strengths at about Elevation -250 feet.

A summary of the Mohr Circles from numerous unconsolidated undrained triaxial and unconfined compression tests is presented on Figure 10. The triaxial tests are presented in detail in Appendix F, Figure F-3, and were conducted with a confining pressure close to the in situ effective confining pressure. These results show similar results to the Figure 9 data or average shear strengths of 1.75 to 2 tsf (3.5 to 4 ksf), but also reflect local hard zones or layers with strengths several times the average values.

The index properties of the clay portion of this unit are also reflected from the laboratory tests on clay samples in Borings A-1 and A-6 summarized on Table 2. These data show an average water content of about 23 percent which is only slightly above the average plastic limit (about 19 percent) and well below the liquid limit (about 37 percent). The Atterberg Limit test results in Appendix F, Figure F-2, consistently plot above the A-Line indicating a CL Uniform

Soil Classification symbol or a clay with low plasticity characteristics. Based on measurements made on numerous undisturbed test specimens, the wet unit weight of the clay is about 138 pounds per cubic foot.

Limited cone data were taken adjacent to Boring A-1 to check the strength and uniformity of the clays. Measured CPT tip resistances in the 23 to 107 foot depth range were generally between 40 and 50 tons per square foot and had friction properties that are typical of a competent cohesive soil as opposed to a granular unit. Using Nkt values of 12 to convert the CPT measurements in Appendix C to strength resulted in calculated undrained shear strength in the 5.5 to 6 ksf range or values slightly higher than the laboratory data in Figure 9. Similarly, using Nkt values of 15 resulted in calculated undrained shear strength in the 4.5 to 5 ksf range or values close to the laboratory data in Figure 9. This indicates that a Nkt value of 15 is appropriate to have values closer to those strengths in Figure 9. More importantly, the tip results also show very uniform strengths with depth even though both hard and less stiff zones were found to exist in the boring at other depths.

Low calculated friction ratios of between 2 and 2.5 percent and an inferred soil behavior classification suggests that, based on CPT A-1 data in the 23 to 107 feet depth range, the clays may have silt, sandy silt and silt mixture properties. As noted above the friction ratios for the sands in CPT A-5 had slightly lower friction ratios of 1 percent (unnormalized) and 2 percent (normalized) and silty sand and sand using non-normalized data and silt mixtures using normalized data. This suggests that the behavior differences between the glacial lake clays and alluvial sands are small and reflective of a larger unit deposited under a similar geologic environment, only one has slightly more fines than the other. A close review of the index properties in Table 2 further confirms this conclusion as the water contents, Atterberg limits (in the clays interbedded in the sands), and unit weights of the sands are the same or slightly less than the average values shown for the clays.

More detailed soil descriptions/parameters and test results on this sand/clay unit can be obtained from the reconnaissance photographs in Appendix A, the boring and cone logs in Appendices B and C, the shear wave velocity and drill rod energy transfer results in Appendices D and E, and the laboratory test results on select samples in Appendix F.

The above sand/clay properties depict a competent soil unit that will provide good skin friction support for the piles beneath piers. They also provide good end bearing assuming reasonable plug development, but the clays offer lower end bearing support and lower total pile capacities compared to the sands.

6.1.4 Possible Knik Tills

This is the deepest unit encountered in the borings. Instead of being the typical very dense sands and gravels found in deep borings throughout the Port and downtown Anchorage areas, it is classified as a hard, gray, gravelly, sandy clay with gravelly and silty clay zones. Average N values were generally over 50 bpf and often in excess of 100 bpf. Triaxial and unconfined tests report compressive strengths of 2.5 to 4.7 tsf, however, its hard solid physical appearance in undisturbed samples and high pocket penetrations values (often greater than 4.5 tsf) indicate that cylindrical test specimen may be failing prematurely, and often as a brittle specimen where the in situ value is probably higher.

This Knik Till unit, like the shallower tills is an excellent soil for pile support, however, its largely clay matrix makes its end bearing capabilities theoretically less than if it were a granular soil. Based on a strong reflector from the geophysical survey (Golder, 2004), Figure 4 shows a deeper basement layer lies below the borings at Elevation -190 feet and deeper near the west side. This is interpreted to be sand and gravel and also likely a part of the underlying Knik Tills.

6.2 East Shoreline Soils

The soils along the east shoreline of Knik Arm between the proposed east bridge abutment and the Port of Anchorage will support a highway embankment and pavement section to access the proposed bridge. East Shoreline Profile B-B’ in Figure 5 illustrates our interpretation of the soils across this section of the channel. Figure 1 shows the locations of this profile.

As shown on Figure 5, similar soils to the above four units except for the marine sands also are present along the east shoreline, although there is a tendency of encountering slightly weaker soils south of Cairn Point as the Port is approached. The dominant soils, however, are the Unit 3 glacial lake or marine sands and clays although zones of dense gravelly till-like deposits are locally present north of Cairn Point and loose to medium dense silty sands and stiff silty clays exist near the Port. Moisture content, limits, gradation, and P200 tests on samples from these shallower borings are presented in Appendix F.

7.0 GEOTECHNICAL ANALYSES AND RECOMMENDATIONS

The key geotechnical components of this project include the foundations for the main bridge crossing of Knik Arm, possible causeways at each bridge abutment and the approach roads to tie the project into the existing road systems. The focus of this study is on the piles in the overwater bridge piers recognizing that the number, size, and embedment depths can have a major impact on the construction cost of this project.

Since the shoreline drilling work was added to our work scope, we have also provided herein a cursory evaluation of the stability of the fill embankment planned along the east shoreline from the east bridge abutment to the Port of Anchorage. A closer evaluation can be completed in final design studies once the prism width of the roadway, the embankment heights, and original ground elevations are better defined.

Additionally, a site-specific ground response analysis was performed as part of this initial effort to compare with an AASHTO Code based spectrum to further guide the planners in assessing the likely seismic design loads on different types of bridge structures. A cursory evaluation of liquefaction was also included as strength loss could reduce the support capabilities of the piles, particularly lateral resistance.

We understand that causeways are tentatively planned at the bridge approaches, but their lengths and water depths/embankment heights have not yet been established. In general, our borings indicate favorable foundation soils for support of sizeable embankments. However, in selecting causeway lengths, we recommend a hydrology study of the constriction be conducted to evaluate future scour of the soils at the causeway ends and in the bridge section. The till thins in this area or is absent and the surficial fine sands in this center area of the channel are highly erodable.

7.1 Pier Description

For the purposes of estimating costs, the bridge foundation considerations from the PB/HDR 2002 study were used. In summary: 1) the bridge concept for the 2002 study was a four-lane concrete or steel bridge, however the current cost estimate also includes a two-lane bridge option 2) the bridge piers will be designed with approximately four to six driven large diameter pipe piles, 3) pier spacing is anticipated to be on the order of 600 feet, and 4) the pier footings will act as pile caps. The pile caps will likely be located in the intertidal zone and protected from sea ice by a sloping jacket or hood.

We understand that the desired ultimate axial pile capacities are on the order of 15,000 kips or more. Preliminary calculations suggest that to obtain those capacities, piles on the order of 8 feet in diameter could be required. Because of the anticipated hard driving to achieve enough embedment into some of the hard till-like gravelly soils or to penetrate this dense cap and achieve deep embedment into the underlying sands and clays, an average wall thickness of 2 inches was assumed. In reality a variable or thinner section may be appropriate in some areas for final design.

To help optimize the pile design, 4-foot diameter piles were also evaluated to provide a range in capacities such that smaller sizes (or intermediate sizes by extrapolation) could also be assessed. For these smaller piles, we have assumed pile ultimate capacities in the range of about 3,000 to 5,000 kips each and, consistent with PB/HDR's 2003 report, wall thickness in the 1- to 1.5-inch range.

7.2 Pile Analysis Methodology

As shown on Figure 1 and described above, four major soil units are encountered in borings along the alignment. Within many of these units, strengths and density changes with depth were common and in some cases significant where use of average strength properties for each unit was not considered appropriate for a proper analysis. Therefore for pile capacity analyses, the conditions in each of the nine deep boring logs were modeled as nine idealized profiles along the over-water portion of the bridge. The soil type, material properties, and geotechnical design parameters adopted for each of the nine idealized profiles for the approximate depth of each boring are summarized on Table 3. Changes in contacts, and material types generally were intended to reflect those conditions noted on each boring log while soil parameters were estimated from the strength, and density results noted from field and laboratory measurements.

The above soil properties were then converted to pile parameters and analyzed by following guidelines presented by the American Petroleum Institute (API) in their manual on recommended practice. Since the API procedures in RP2A were largely developed for large diameter offshore platform piles, this procedure was selected as the preferred analysis with APILE plus as the computer program for carrying out the analysis.

As noted in the API analysis, the procedures for clays are based on the use of undrained shear strength and are largely empirical. Correspondingly, for sands or cohesionless soils the API procedure is also empirical, but effective stress techniques are employed because for long term performance no excess pore water pressures are assumed.

In the analysis both side friction and end bearing are assumed to contribute to the total capacity. We also assumed that to achieve suitable embedment, the piles would have to be driven as a non displacement pile with an open bottom. In calculating tip resistance, two approaches are possible in assessing the extent of plug development. The shear strength (or a remolded strength) on the inside of the pile sidewalls could be assumed to build up this tip resistance gradually or the pile could be assumed to be plugged totally recognizing that the dominant soils are compact, and a substantial fraction of the pile will be cleaned out and replaced with structural concrete. The latter assumption of a full plug was assumed in our analysis recognizing that during driving it will not likely develop for these large piles, particularly the 8-foot diameter piles. If plug development is not provided for per Approach 2 and the first approach is assumed, the calculated total capacity in our analysis may be as much as 20 percent too high.

7.3 Pile Capacity and Embedment Results

7.3.1 Compression Capacity

The results of the calculated ultimate axial pile capacity vs. embedment depths for each of the conditions at the nine borings are presented in Appendix A for both 4- and 8-foot diameter piles. Also shown in each plot is the ultimate side friction (or uplift support) and bearing support or tip resistance, the sum of which is the ultimate axial capacity. The 8-foot diameter capacities vs. embedment depth curves for each of the nine borings are shown in Figures A-1 through A-9, while values for the 4-foot diameter piles are presented in Figures A-10 through 18.

In most cases, the total capacities in Appendix A increase with embedment depth; however, in several instances the curves take a reverse or saw-toothed shape. This drop in capacity with increased embedment occurs when the pile passes from a dense granular soil with high bearing values to a cohesive soil where much lower tip capacities must be used. This drop also occurs if the pile passes through a weaker soil unit with reduced bearing and/or side friction values. Table 3 shows the changes in these soil types and properties with depth used in our analyses.

Using these Appendix A plots, the theoretical embedment depths (or pile lengths) below the mudline can be determined at each boring location for a given pile capacity. As examples, if 3,000 to 5,000 and 10,000 to 15,000 kip ultimate capacities are desired for 4- and 8-foot diameter piles respectively, the required pile embedment depths for the various piers across Knik Arm can be estimated from the pile tip contour lines noted by the blue and green lines in the Figure 11 profile. The top and bottom values reflect capacities for the upper and lower example capacities noted above. Figure 11 may also be used to extrapolate approximate embedment depths for other capacities or even intermediate pile diameters. In using this plot, the capacity and depth results should be checked against the results in Figures A-4 and A-9 to account for any longer piles that may be reflected by data in these two north offset borings (i.e., Borings A-5 and A-10).

Other approaches can involve taking the capacity/embedment depths from the plots in Appendix A directly and grouping or assigning a number of piers to each boring area. With this approach, the Figure 4 Profile A-A' section and Figure 1 should probably be used to estimate the number of piers closest to each boring and total pile embedment lengths for each group of piers. Assuming that the actual pile butt will be situated near the mid point in the tide fluctuation zone (about Elevation +14 feet), actual pile lengths can be estimated as the embedded length plus the water depth to MLLW Elevation 0' datum (both in Figure 11) plus 14 feet to reach the midpoint elevation. Allowing space for a causeway on each end, they show that total pile lengths for the 15,000 kip ultimate capacity will range from about 75 to 150 feet on the west side and up to 255 feet on the east side in the clays (at Boring A-1). The smaller piles to reach 5,000 kip capacities will have about the same lengths or slightly more on the west side and 20 to 70 feet less on the east side.

For load and resistance factor design (LRFD), AASHTO's 2003 Standard Specification for Highway Bridges recommends pile resistance factors of 0.7 for operating loads with pile driving analyzer tests or 0.8 for an actual load test.

7.3.2 Uplift Capacity

Calculated ultimate uplift capacity changes with pile embedment below the mudline are also summarized in Appendix A as the skin friction on each plot. In general, the skin friction component of the total capacity typically represents greater than 50 percent (and as much as 85 percent) of the total capacity in clay soils and in granular soils where the pile embedment is greater than 100 feet. In the upper 100 feet of granular soils where penetration to achieve high total capacities is limited (like in the till soils at Borings A-6, A-7, and A-9), the uplift resistance can be small and may control over compression capacity in estimating the pile lengths in these areas unless these shallow water areas are covered with a causeway.

7.4 Pile Settlements

The total pier settlements that can be permitted for this bridge structure depend on many factors including the actual loads that are applied and/or the span and pier loads, the permissible amount of pile load transfer in skin friction and end bearing for each of the main soil types (clays, sands, and tills), possible group action or battering, and seismic considerations. Since the channel bottom soils are generally very stiff to hard or dense to very dense excluding the marine sediments, we believe that total and differential settlements can be kept relatively small and within tolerable limits for suitable long term performance of a pile supported bridge at this location. Refined settlements analyses will need to be performed in the future once a bridge width and design concept for a typical span and pier cap are better defined.

7.5 Pile Drivability

The static pile calculations above and in Appendix A do not consider drivability and assume that the stated capacities and pile embedments can be achieved if sufficiently large hammers are available to drive the piles to the prescribed depths without encountering refusal or obstructions such as boulders. Boulders were reported on our boring logs and while rare are known to be present at the site. If very large boulders are encountered, refusal to pile driving will occur, which would require pile cleanout and/or the need to core through or breakup the boulder. A suitably equipped pile top-drive drilling rig (likely required for pile cleanout in order to place structural concrete) should be available during this effort.

The following table summarizes the pile sizes, pile sections, and hammers considered in a previous drivability analyses (PB/HDR, 2003).

Pile Diameter (feet)	Pile Wall Thickness (inches)	Hydraulic Hammer - Rated Energy
8	2	1180 kip-ft
8	3	1180 kip-ft
10	3 1/8	1180 kip-ft
4	1	148 kip-ft
4	1 ½	369 kip-ft

Since the soil's density and consistency characteristics considered in this earlier analysis are very similar to what was actually encountered, the results of the preliminary pile drivability evaluations are still considered applicable and concluded that:

1. Deeper penetrations (and therefore higher capacity) can be achieved for a given pile size with a larger pile wall thickness (i.e., driving stresses will be less with piles with thicker sidewalls).
2. The larger diameter (8-foot diameter) piles provide pile capacities that are three to four times greater than the intermediate (4-foot diameter) piles.
3. Wall thicknesses in the 2- to 3-inch range are appropriate for the 8-foot diameter piles while 1- to 1.5-inch walls should work for the 4-foot piles.

4. If the shallow tills can be penetrated, average embedments for 8-foot diameter piles of 273 to 290 feet were predicted in hard clay soils while 123 to 173 feet of embedment was calculated in very dense granular soils, the larger embedments occurring with the thicker sidewalls. For the 4-foot piles, maximum embedments were 138 to 205 feet in the clay soils and 80 to 143 feet in the very dense granular soils.

A comparison of these above maximum embedment depths with those in Figure 11 support that the depths in Figure 11 are theoretically achievable, especially with the thicker walls.

Case histories exist from the offshore experience in Cook Inlet (with somewhat similar soils conditions) for: 1) piles on the order of 34- to 84-inches in diameter, 2) piles driven to penetrations ranging from 60 to 125 feet, and 3) use of air-steam hammers with rated energies on the order of 870 kip-ft. In some instances, pile cleanout was required to achieve design penetration. Similarly, 42-inch diameter by ¾-inch wall, high strength pile piles were driven with a Delmag D 125-13 diesel hammer (350 kip-ft) to penetration depths of over 220 feet including up to 30 feet into similar very dense till-like soils at the Glenn Parks Highway Interchange Project near the head of Knik Arm. This local experience also supports that long piles and reasonable penetration of dense soils are possible with large hammers.

Compressive driving stresses calculated from additional drivability studies using GRL WEAP, a 1,180 kip foot hydraulic hammer, and the above 8 foot piles were in the 32 to 40 kips per square inch (ksi) range with the higher values occurring in the thinner pile sections or in the more granular soils, as noted above. This suggests that if thinner pile sections (2-inches or possibly or 1- or 1.5-inch walls) are used for 8-foot diameter piles, steel with yield strengths greater than 36 ksi will be needed. Comparable wall thicknesses for 4-foot diameter piles would be 1 or 1.5 inches, as noted in the table above.

A further check of the drivability of shorter 8-foot diameter piles into and/or through the very dense till-like soils with high end bearing resistance reveal even higher driving stresses (exceeding 60 ksi) for the thinner sections (i.e., if a 1- or 1.5-inch wall pile is selected to penetrate the high density tills and achieve high capacities). The use of thinner steel sections increase the likelihood of possible pile damage. Thus, for piles penetrating the till-like soils, pending more refined analyses and a test pile program, 2-inch wall thicknesses with 56,000 pounds per square inch (psi) or better strength steel should be assumed for these larger piles for any future concept studies. A-56 steel was used on the 42-inch piles for the Glenn Parks Project with a driving shoe to penetrate hard or gravelly layers.

In summary, the deep penetration of large diameter pipe piles into these soils appears feasible. However, large hammers and piles with thick side walls and higher strength steels may be required to achieve the desired penetration and high capacities. Recognizing that the calculated drivability results in dense or hard soils are highly sensitive to the input assumptions and boulders may be present, further studies and/or a test pile program with PDA measurements and/or static load tests may prove valuable as part of future design studies to give contractors bidding the construction work confidence that driven piles are feasible and that suitable pile penetration can be achieved by driving alone.

7.6 Ground Response Analyses

Preliminary site specific ground response analyses were performed based on the measured shear wave velocities and the subsurface data from the boring logs and laboratory testing. The analyses included the following steps:

1. Develop rock uniform hazard spectrum (UHS) for 475-year return period ground motions.
2. Spectrally match a previously recorded rock motion to the rock UHS.
3. Develop soil profile geometry and select dynamic soil properties of the soil models to be analyzed.
4. Use the program ProShake (Edu Pro Civil Systems, 1999) to perform one-dimensional site specific ground response analyses.
5. Compare the results of the ProShake analyses to the design response spectra prescribed by AASHTO (2002).

The rock UHS for 475-year return period ground motions is shown in Figure 12. This return period was selected as the basis for the site response analyses to be consistent with the hazard level in AASHTO 2002 (i.e., 475 years). The spectrum is based upon probabilistic seismic hazard analyses (PSHA) performed by the U.S. Geological Survey and is regional in nature (Wesson, et al., 1999). For comparison, the AASHTO 2002 design spectrum for rock (Soil Profile Type I) is also shown in Figure 12.

We then evaluated the deaggregation of the United States Geological Survey PSHA to determine what types of earthquake events contribute most significantly to the ground motion hazard at the Knik Arm Bridge site. At periods of one to two seconds, contributions from nearby crustal faults as well as offshore subduction events are significant. Since our site specific ground response study is preliminary, we chose to base our analyses on a crustal event similar to motions that might be expected due to movement on the Border Ranges Fault located approximately 7.5 miles (12 km) from the site.

We based our selection of the time history for the site response analyses to be as consistent as practical with a maximum credible earthquake on this above fault and distance between the fault and the site (i.e., magnitude 7.5 reverse or thrust event, recorded approximately 7.5 miles [12 km]from the fault on the footwall). Consequently, we selected a ground motion recorded on rock during the 1999 Chi-Chi Earthquake 6 miles (10 km) from the fault on the footwall. The north component of the acceleration time history was modified to be spectrum compatible with the rock UHS using the program RSPMATCH (Abrahamson, 1994). The response spectrum of the motion after spectrum matching is shown in Figure 12. The acceleration time history with the response spectrum shown in Figure 12 was used as input for the ground response analyses.

The soil model was developed based upon the measured shear wave velocities by seismic CPT in Boring A-5 to a depth of about 225 feet. These results are presented in detail in Appendix D. Below the depth of measured velocities, the shear wave velocities were estimated in the Knik Till based upon measurements at Gould Hall located on the campus of Alaska Methodist University, now Alaska Pacific University in Anchorage (Shannon & Wilson and Agbabian Associates, 1980). The resulting shear wave velocities profile used in our analyses is presented in Figure 13.

The soils in Boring Log A-5 indicate predominantly cohesionless soils; however, the results of the cone penetration at Boring A-5 indicate a mix of sands, silts and clays. Based on the CPT and boring logs in Appendices B and C, it appears that the cone penetration was pushed in a zone of interfingering cohesionless and cohesive materials. We modeled the soils in the profile as all cohesionless, using the modulus degradation and damping curves by EPRI (1993). We then ran a second model in which all of the soil was modeled as cohesive using the plasticity index dependent modulus degradation and damping curves by Vucetic and Dobry (1991) and plasticity index of 20.

The results of the ground response analyses are presented in Figure 14 in the form of 5 percent damped response spectra of the surface motions. The yellow and blue curves represent responses due to modeling of the soils as cohesive or cohesionless material, respectively. The recommended design response spectrum prescribed by AASHTO for Soil Type II and the Anchorage area is also presented in Figure 14. As can be seen on this figure, the spectra calculated from the site response analysis are generally less than or equal to the AASHTO Soil Type II spectrum for periods less than 0.2 seconds and greater than 1.3 seconds. Between 0.2- and 1.3-second periods, the spectra calculated from the site response analysis are typically equal to or greater than the AASHTO Soil Profile Type II spectrum and may exceed the AASHTO spectrum by as much as 30 percent in some relatively narrow period ranges. Therefore, based on the results of the preliminary site response analyses, conceptual bridge design can be based on the AASHTO Soil Profile Type II spectrum.

Final ground response studies should include further variations of the soil model and input rock motions that take into account all types of earthquake sources that affect the bridge site.

7.7 Liquefaction Considerations

Liquefaction of the soils under future earthquake shaking could reduce the axial and lateral support for the piles. It generally occurs in granular soils, typically loose saturated sand and silty sands, due to a rapid buildup of pore water pressure and subsequent decrease in effective stress and significant loss of strength. As discussed previously, the recent fine sands or marine deposits encountered at relatively shallow depths in the borings (above 30 to 40 feet below mudline) were loose to medium dense and appear to have the highest liquefaction potential. Conversely as shown in Figure 8, much of the deeper alluvial sands encountered by our borings (with a few isolated exceptions) are largely medium dense to very dense and are therefore not likely to liquefy in a future earthquake.

To confirm the above statements, liquefaction analyses were performed on the soils encountered by Borings A-2, A-5, and A-10, or those borings containing mostly deep sandy soils. Our liquefaction analyses generally followed the steps outlined in Youd, T.L. and Idriss, I.M., 2001. The only deviation from these procedures is that we used the hammer energy efficiencies measured during our field effort and shown in Appendix E to refine the corrected blow count value calculations performed for the analyses. A summary of these results is presented in Appendix H, Figures H-1, H-3, and H-5, as the factor of safety against liquefaction vs. depth. Note that the depths in these figures are really MLLW elevations and the upper part of these results reflect some water as shown in the subsurface profile to the left of the factor of safety plots.

In this assessment, the potential soil shear strength reductions in the non-cohesive and low-plasticity soils considered residual soil shear strengths for soils with a factor of safety less than one under the design earthquake. For each boring, the liquefaction potential of the soils was evaluated using Seed's simplified empirical procedure and in accordance with National Center for Earthquake Engineering Research (NCEER) technical report NCEER-97-0022 (Youd and Idriss, 1997). For the liquefaction calculations, and consistent with our above ground response analyses, a site peak ground acceleration of 0.36g was assumed. Reduced soil shear strengths were then estimated for the soils with a factor of safety less than 1.0 using the empirical relationships by Seed and Harder, 1990, and assuming strengths approximately ¼ above the lower bound of this relationship. Detailed results of these analyses are included in Appendix H as Figures H-2, H-4, and H-6.

As shown in the summary figures in Appendix H, the majority of the sandy soils evaluated for liquefaction potential have a factor of safety of greater than 1.0 against liquefaction. According to our analyses, the loose to medium dense sand in the upper 35 to 40 feet below mud line of Boring A-2 are liquefiable under our model conditions. Two deeper samples from this boring were determined to be liquefiable. However, these soils are isolated within liquefaction-stable soils and, in our opinion, should have a negligible effect on pile foundations. Our analyses of Boring A-5 revealed only one sample at approximately 113 feet below mud line that was liquefiable under our assumed conditions. In Boring A-10, we found that the soils associated with the top layer of loose to medium dense, slightly silty sand (from 0 to 25 feet below mudline) were generally liquefiable.

The above analysis of the sandy soils confirms that only the recent marine deposits in the upper approximately 40 feet may liquefy under strong earthquake shaking. Therefore, we recommend that for this cost evaluation and estimating pile lengths, it should be assumed that these marine deposits in this upper maximum 30- to 40-foot zone will contribute no axial or lateral support for the pier. Fortunately, this is not considered a serious limitation to the feasibility of placing a bridge at this location as the axial support provided from these weak soils is small and any extra lateral resistance can be achieved by increasing the stiffness of the piles in this zone and/or battering the piles. The risk of a change in pile lengths or foundation costs is small and reduced even further, in our opinion, by using large diameter, high capacity piles.

Liquefaction analyses were not performed on the clay soils because it was not considered necessary. Atterberg Limit results in Appendix F, Figure F-2, indicate that much of the glacial lake clay soils possess low plasticity characteristics (a Liquid Limit below 33.5 per cent) and may have some potential for liquefaction. We conclude, however, that they have a low or no liquefaction potential under strong earthquakes for two reasons. The undrained shear strengths of the clay soils in Figure 9 are mostly in the very stiff range and the sensitivity is low. Also, recent studies from cyclic tests at the Port of Anchorage for similar, but weaker clay soils, found that the clay "is not sensitive to cyclic loading and strength reduction, does not have the tendency to liquefy under seismic loading conditions and does not exhibit anisotropic strength behavior." Based on these findings, we believe that under strong earthquake shaking, significant liquefaction or strength losses are not likely to occur in the clay part of this unit.

As discussed in Section 6, till like deposits occur in abutment bluffs, at shallow depths in the intertidal zone, and as the site's basement material and comprise a heterogeneous and varying mixture of sands, gravels, silts and clays. These tills are consistently very dense or hard with Standard Penetration Resistances generally in excess of 50 bpf and more frequently in excess of 100 bpf. Because of these high resistance values, these tills are considered to be very compact and stable and in our opinion not susceptible to liquefaction. Thus where they are present, they offer excellent lateral and axial support for piles penetrating and/or bearing in these materials.

7.8 Embankment Stability

About two miles of highway earth embankments will be required along the east shoreline to elevate the road surface above the high tide line (about Elevation +34.1 feet, MLLW Datum). The embankments' location is identified on Figure 1 as Subsurface Profile B-B'. The soils depicted on this profile are shown on Figure 5 and are based on 25- to 30-foot deep borings drilled roughly 30 to 40 feet from the toe of the steep bank directly east of this alignment. Since the banks in this area rise up to 70 feet or more, and are being eroded at the toe and are failing as slump blocks, we assume that this embankment will cover the mudflats near high tide starting at the toe of the bank and extending seaward about 100 feet.

Since the embankment requirements for the highway have not yet been defined, the following assumptions were made in order to evaluate the general stability of an embankment on this mudflat.

1. The highway elevation will be about 6 feet above the extreme high tide (about Elev. +40' MLLW).
2. Embankment slopes will be 2H to 1V.
3. Embankment fill will be granular, well compacted, and placed on the mudflat soils. Strength parameters for this fill were assumed as an internal friction angle, ϕ, of 36 degrees.
4. The mudflats have an assumed seaward grade of about 6 percent.
5. A modest water table is assumed in the fill (see Figures 15 through 17).
6. Riprap will provide slope protection and therefore a surface raveling failure is prevented in the stability modeling.
7. Geotextiles can be added beneath the riprap or fill, if necessary, to improve stability, provide separation of classified soils, and/or to prevent leaching.

For these assumed conditions, the fill height at the crest of the embankment slope should be roughly 15 feet.

A two-dimensional model was then developed using the above assumptions, and strength parameters of the soils from the shoreline borings. The model was then analyzed with GSTABL7, a computer program for analyzing the stability of slopes. In this analysis, a search is conducted of over 100 arcs to locate the most critical failure surface and the lowest factor of safety. The Modified Bishop method with circular arcs was used in completing the analysis. For a seismic evaluation of the embankment slope, a pseudostatic method of analysis is performed where an equivalent horizontal static force of 0.2 g is applied to the critical arc. This seismic coefficient is consistent with local code requirements and has been used for design of structures placed on bluffs throughout Anchorage and along the shorelines at the Port of Anchorage.

Three cases, designated 1 through 3, were evaluated to reflect conditions summarized at the north, central and south parts of the corridor shown on the subsurface profile in Figure 5. For the three different cases, the following factors of safety (FS) were calculated.

	Soil Conditions	**Static FS**	**Seismic FS**
Case 1	Hard Gr.Clay/Till Over Hard Clay	2.56	1.14
Case 2	D. Sand over Hd. Clay	1.63	1.09
Case 3	Stiff Clay over Stiff to Hard silty Clay	2.84	1.95

In general, minimum factors of safety of 1.5 for static loading conditions and 1.1 for seismic loading are considered appropriate for important structures such as bridges, buildings, dams, etc. A highway embankment is not considered a critical structure and lower factors of safety are often accepted for the more severe loading conditions recognizing that the cost to repair a failure is often far less than designing for all conditions. These above results indicate adequate factors of safety for embankment stability.

The soil properties, layer thicknesses, and the location of the ten most critical failure arcs and their factors of safety for static loading conditions for the above cases are summarized on Figures 15, 16, and 17. The bold red arc is the failure arc with the lowest factor of safety. The seismic factor of safety in the above table applies only to the most critical failure arc noted on these figures. Each of these plots show elevations that represent MLLW datum. As noted above, a surface raveling slope failure was prevented by forcing all failure arcs to pass below the thin dashed red line on these figures.

In general a comparison of these cases with the soil conditions along the highway alignment in Figure 5 reveal similar behavior and adequate factors of safety against failure where special buttressing or an earth toe berm will not likely be required. The three figures also indicate the following:

1. Deep seated failure is unlikely as failure arcs for all situations are shallow.
2. Embankment failure, if it were to occur, would be shallow and not encompass the entire highway section (i.e., limited to about 10 feet of shoulder or less). Loss of the entire road section is not likely.
3. Most of the failure arcs remain in the fill and pass just below the upper limit of failure (the dashed red line) meaning the actual factor of safety is more controlled by the assumed shear strength of the fill than the strength of the foundation materials.

In summary, we do not foresee any significant stability difficulties associated with design of 10- to 20-foot high, 2H:1V slopes on these mudflats. Steeper embankment slopes are possible to 1.5H:1V, but not recommended in this area, particularly if tide water covers the slope face and can create a sudden drawdown situation.

7.9 Causeway

A causeway is generally planned at each end of the bridge and is tentatively visualized as an earth embankment extending offshore until water depths become too large. The causeway is less costly than the bridge and to place long causeways greatly reduces the bridge length needed to span this 12,000-foot wide water crossing. Shortened bridge lengths of 5,000 to 10,000 feet have been tentatively suggested, however, hydrology studies in our opinion need to be completed to refine the above numbers.

The hydrology study would help define how the constriction imposed by the causeway would impact channel erosion and deposition patterns both up and downstream, particularly scour at the causeway ends and at bridge piers. Figure 4 shows that the center of the channel contains a wide section of fine sands in the channel bottom. Also, numerous grain size curves in Appendix F reveal low silt fines and a poorly graded material with little apparent cohesion to resist scour. When comparing these gradation results with published stream velocity vs. particle size curves that result in material being transported, eroded, or deposited, it is apparent that these soils are among the more highly erodeable materials.

Once the above hydrology related conditions are better defined, the geotechnical concerns of causeway embankment stability, settlements, and slope protection in deeper water and end structure stability can be addressed. In general, the soils in the mudflats or bridge approach areas and shown on Figure 4 are hard or very dense and for the most part suitable for support of large embankment fills.

8.0 FUTURE GEOTECHNICAL STUDIES & TESTING

It should be emphasized that this is a concept level study with limited explorations aimed at estimating pile lengths and project construction costs and is not intended for final design. After a preferred alignment is chosen, the information can be used as additional information and a guide for planning future explorations for final design of bridge piers, causeways, and the new shoreline road needed to tie the bridge structure into the existing road system. From a geotechnical/foundation engineering perspective, the final design phase of the bridge piers should include the following:

8.1 Final Site Characterization

More detailed site characterization should be conducted to provide pier-specific characterization for the final bridge alignment. Borings should be drilled to depths in excess of the planned pile lengths at each pier location. Laboratory testing on the soil should also be conducted to evaluate the index and range of engineering properties of these materials. This information will help refine pile capacity and embedment depths as well as identify the thickness of the till cap at each pier to help evaluate test and production pile work, the potential for pile damage during driving, and if the need exists for additional pile wall thickness or a variable section to accommodate any excessively high stresses during driving.

As discussed previously, limited on-shore geotechnical and bluff reconnaissance studies have been completed on both sides of Knik Arm for both the current or past studies or projects. Much of this data is summarized in Figures 4 and 5 and depicts generally more favorable foundation conditions compared to the tideland soils found to the south in more developed areas of Anchorage and at the Port of Anchorage. This shoreline data is useful for project planning, alignment selection, and preliminary design, however, it is limited and data gaps likely exist. Once a preferred alignment is selected, more in-depth on shore exploratory work will need to be conducted to fill in gaps and address final foundation design requirements in the intertidal and upland access road areas.

8.2 Earthquake Ground Motion Studies

AASHTO 2002, requires special studies to determine site-specific design motions if the site is located close to an active fault or if long duration earthquakes are expected in the region. While there may be some question as to the activity of the Border Ranges Fault, the site is subject to long duration megathrust events on the subduction zone. Consequently, site-specific ground motion studies will be required for final design. Final ground response studies should include further variations of the soil model and input rock motions that take into account all of the types of earthquake sources (including subduction zone megathrust) that affect the bridge site.

Since the 1964 Great Alaska Earthquake induced an estimated 3 to 7 minutes (probably about 4 minutes) of strong vibratory shaking in the Anchorage area (Shannon & Wilson, 1964), the possibility of experiencing an exceptionally long duration of shaking should be considered in future design analyses. It could induce more extensive liquefaction in any marginal bridge foundation support soils increasing the depth of pile fixity, lateral forces, or the site response. Long durations of shaking may also result in strength losses in causeway fills placed in deep water or result in bluff slumping in abutments or the planned approach highway to the south. Thus, duration, as noted, is an important parameter and should not overlooked in future design studies when considering both vibratory and ground failure effects for both the bridge and adjacent embankments.

8.3 Pile Testing

A pile test program could be implemented as a part of the design process if found to be beneficial for the costs expended. Three possible benefits from a test pile program are: 1) to demonstrate to contractors that driving in or through the till is possible and provide the confidence to secure lower bids; 2) to confirm or refine pile wall thicknesses; and 3) to optimize pile capacity estimates with pile setup/load transfer measurements using PDA technology. As the design evolves, such a program can be evaluated and implemented if the merits gained can justify the high costs of these efforts.

The handling and driving of long, large-diameter piles, with large hammers, in areas of strong currents, frequent winds, dense/hard soils, and large tidal variations together with a short construction season, seasonal sea ice, and murky water will present significant challenges during construction. The uncertainties associated with working in such an environment should be taken into consideration when preparing preliminary cost estimates for this work.

SHANNON & WILSON, INC.

Fred R. Brown, P.E.	Elizabeth A. Karcheski
Sr. Vice President	Project Geologist I

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Table 1

Summary of Drill Rod Energy Transfer Results

Boring	Starting Sample Depth (ft)	Penetration Resistance (blows/set)	Average Measured Transfer Energy EFV (lb-ft)	Average Computed Transfer Efficiency ETR (%)	CAPWAP Results for One Blow		Rod Efficiency at Depth of Sample (%)
					Transfer Energy at Top of Rod (lb-ft)	Transfer Energy Approx. 15 ft above the Spoon (lb-ft)	
A6	31	28/ft	305	87	300	250	83
A10	66	48/ft	293	84	290	220	76
A10	88.5	38/ft	311	89	310	230	74
A10	126	41/ft	299	85	300	210	70
A10	156	72/ft	295	84	290	200	69
A10	166	84/ft	291	83	290	190	65

Table 2
SUMMARY OF ENGINEERING PROPERTIES OF ALLUVIAL SANDS AND GLACIAL CLAYS

Granular Soils: SAND Borings A-2, 5 and 10
Fine grained Soils: CLAY Borings A1 and 6

Soil Type: PROPERTIES	SANDS Range	SANDS *Average*	CLAYS Range	CLAYS *Average*
Water Content, % — See Appendix F, Table F-1	15.9 - 28.5	*23.2*	11.1 - 29.9	*22.9*
Atterberg Limits, % — Plastic	15* - 22*	*19*	26 - 45	*36.6*
Liquid — See Appendix F, Figure F-2	22* - 43*	*32*	16 - 23	*19*
% Passing 200 Sieve — See Appendix B, Boring Logs	6.7 - 96	*19*	20 - 90	*43*
Undrained Shear Strength (ksf) — Unconfined Compression Test			1.75 - 10	*3.1*
All testing results — See Figure 10			5-Jan	*3.5*
Wet Density, pcf — See Appendix F, Table F-1	125* - 162.2*	*132.2**	120.5 - 162.7	*138.4*
Unified Soil Classification Symbol — See Appendix F, Table F-2	SP-SM or SP		CL	
Stiffness or Compactness — See Figures 9,10 & 11	Dense to Very Dense	*Very Dense*	Stiff to Hard	*Very Stiff to Hard*
Shear Wave Velocities, fps — See Appendix D	720 - 1750	*1135*	None Taken	

* Atterberg limits and densities taken from clay units interbedded within sandy soil.

Table 3. Geotechnical Pile Design Parameters (Sheet 1 of 3)

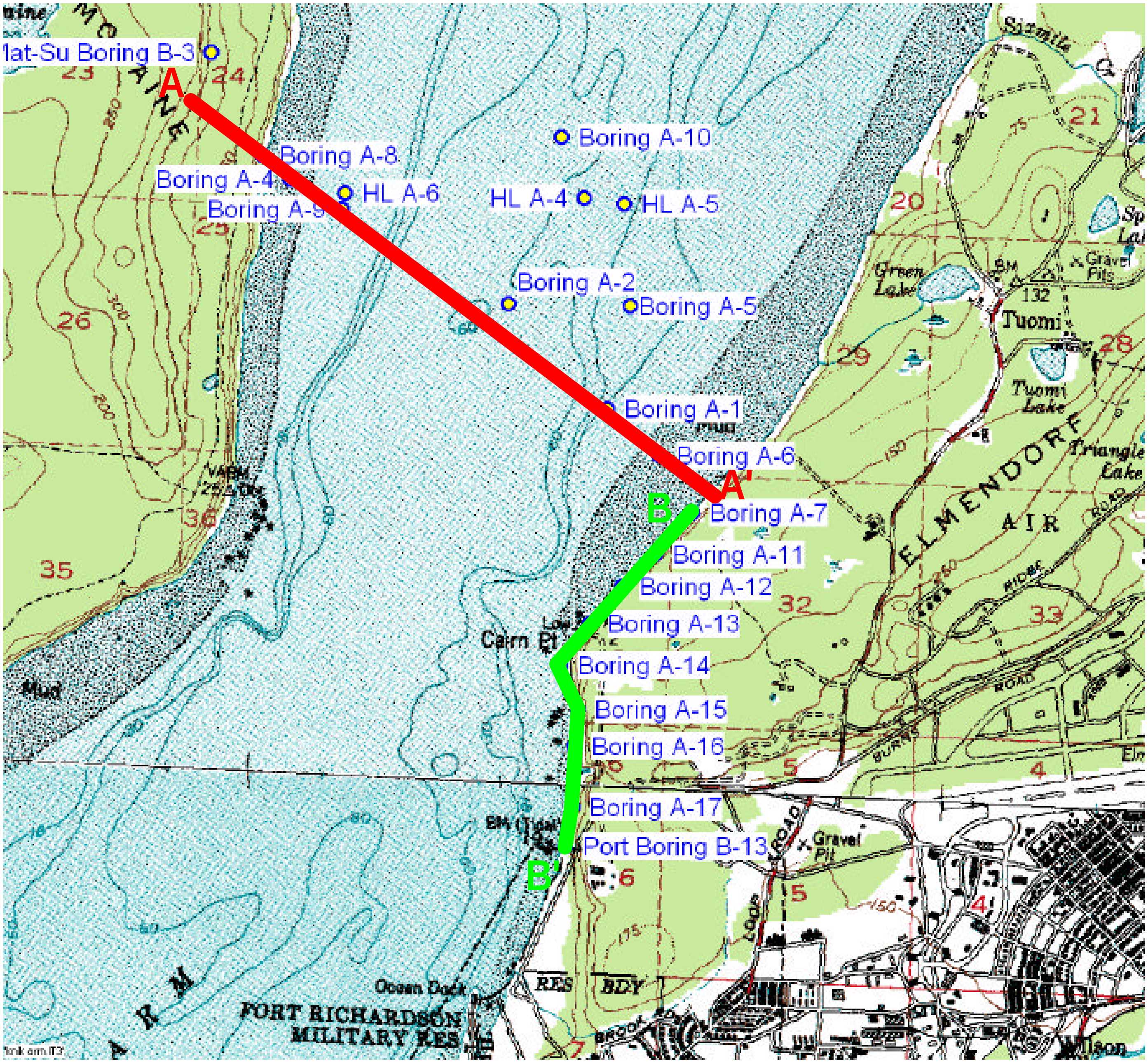
Description	Depth Below Mudline (feet)	Submerged Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (ksf)	Soil-Pile Friction Angle, δ (degrees)	Limiting Skin Friction, F_{max} (kips/ft^2)	Bearing Capacity Factor, N_Q	Limiting Unit End Bearing, Q_{max} (kips/ft^2)
Profile Matching Conditions in Boring A-1							
Medium dense, silty sand	0 – 12	62.6		17.5	1.2	10	50.0
Stiff to hard, silty CLAY	12 – 61	69	3.2		1.6		28.8
Stiff to very stiff, silty clay	61 – 104	65	2.0		1.0		18.0
Very stiff, silty CLAY	104 – 145	64	2.9		1.45		26.1
Very stiff to hard, silty clay	145 – 159	64	4.0		2.0		36.0
Very stiff silty clay	159 – 226	68	2.2		1.1		19.8
Stiff silty clay	226 - 240	67	1.0		.75		9.0
Very stiff to hard silty clay	240 – 270	77	4.0		2.0		36.0
Stiff to very stiff silty clay	270 – 312	68	3.4		1.7		30.6
Hard, gravelly silty clay	312 - 337	74	4.7		2.35		42.3
Profile Matching Conditions in Boring A-2							
Description	**Depth Below Mudline (feet)**	**Submerged Unit Weight, γ' (pcf)**	**Undrained Shear Strength, S_u (ksf)**	**Soil-Pile Friction Angle, δ (degrees)**	**Limiting Skin Friction, F_{max} (kips/ft^2)**	**Bearing Capacity Factor, N_Q**	**Limiting Unit End Bearing, Q_{max} (kips/ft^2)**
Loose sand	0 - 20	58		0	.60	5	25
Medium dense, slightly silty, fine sand	20 - 55	60		0	1.2	10	50
Medium dense to very dense silty sand	55 - 100	68		25	1.7	2	100
Very dense, slightly silty to clean sand	100 - 142	70		30	2.0	40	200
Very dense silty sand and gravel	142- 156	73		35	2.4	50	250
Very stiff to hard silty clay	156 - 182	63.5	3.7		1.85		33.3
Hard slightly sandy silty clay	182 - 191	68		30	2.0	40	200
Very stiff silty clay	191 - 198	63.5	2.5		1.25		22.5
Profile Matching Conditions in Boring A-4							
Description	**Depth Below Mudline (feet)**	**Submerged Unit Weight, γ' (pcf)**	**Undrained Shear Strength, S_u (ksf)**	**Soil-Pile Friction Angle, δ (degrees)**	**Limiting Skin Friction, F_{max} (kips/ft^2)**	**Bearing Capacity Factor, N_Q**	**Limiting Unit End Bearing, Q_{max} (kips/ft^2)**
Dense to very dense silty sand and gravel	0 - 40	78		35	2.4	50	250
Hard gravelly silty clay	40 - 123	65	5		2.5		45
Very dense silty sand	123 – 128	63		30	2.0	40	200
Hard, sandy silty clay	128 – 144	68	5		2.5		45
Hard slightly gravelly silty clay	144 – 200	68	6		3.0		54

Table 3. Geotechnical Pile Design Parameters (continued; Sheet 2 of 3)

Description	Depth Below Mudline (feet)	Submerged Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (ksf)	Soil-Pile Friction Angle, δ (degrees)	Limiting Skin Friction, F_{max} (kips/ft^2)	Bearing Capacity Factor, N_Q	Limiting Unit End Bearing, Q_{max} (kips/ft^2)
Profile Matching Conditions in Boring A-5							
Very stiff to hard clay with sand	0 - 25	65	3.5		1.75		31.5
Very stiff dense clay and sand	25 - 125	68	3.0		1.75		27.0
Very dense and hard sand and clay	125 - 160	68		30	2.00	35	100.0
Profile Matching Conditions in Boring A-6							
Description	**Depth Below Mudline (feet)**	**Submerged Unit Weight, γ' (pcf)**	**Undrained Shear Strength, S_u (ksf)**	**Soil-Pile Friction Angle, δ (degrees)**	**Limiting Skin Friction, F_{max} (kips/ft^2)**	**Bearing Capacity Factor, N_Q**	**Limiting Unit End Bearing, Q_{max} (kips/ft^2)**
Very stiff sandy gravelly silty clay	0 - 48	65	3.3		1.65		29.7
Very dense sandy gravel and cobbles	48 - 65	78		35	2.4	50	200
Very dense, gravelly silty SAND	65 - 80	68		32	2.1	40	200
Very stiff to hard sandy, silty clay	80 - 87	67	3.0		1.5		27
Dense to very dense slightly gravelly silty SAND	87 - 93	68		30	2.1	40	200
Very stiff to hard slightly sand silty clay	93 - 135	67	3.5		1.75		31.5
Very stiff to hard slightly sandy silty clay	135 - 181	68	3.3		1.65		29.7
Hard, gravelly silty clay	181 - 210	76	5.0		2.5		45
Profile Matching Conditions in Boring A-7							
Description	**Depth Below Mudline (feet)**	**Submerged Unit Weight, γ' (pcf)**	**Undrained Shear Strength, S_u (ksf)**	**Soil-Pile Friction Angle, δ (degrees)**	**Limiting Skin Friction, F_{max} (kips/ft^2)**	**Bearing Capacity Factor, N_Q**	**Limiting Unit End Bearing, Q_{max} (kips/ft^2)**
Very dense, silty SAND, gravel and cobbles	0 - 117	78		36	2.5	48	230
Hard, silty clay	117 - 193	68	5		2.5		45
Dense, silty SAND	193 - 220	72		30	2.0	40	200
Profile Matching Conditions in Boring A-8							
Description	**Depth Below Mudline (feet)**	**Submerged Unit Weight, γ' (pcf)**	**Undrained Shear Strength, S_u (ksf)**	**Soil-Pile Friction Angle, δ (degrees)**	**Limiting Skin Friction, F_{max} (kips/ft^2)**	**Bearing Capacity Factor, N_Q**	**Limiting Unit End Bearing, Q_{max} (kips/ft^2)**
Medium dense, silty sand	0 - 10	58		15	1.0	8	40
Very stiff to hard, sandy gravelly clay	10 - 36	65	3.5		1.75		31.5
Very dense, slightly gravelly slightly silty SAND	36 - 61	68		32	2.2	42	210
Hard, gravelly sandy, silty Clay	61 - 162	78	5.0		2.5		60
Hard, slightly sandy silty clay	162 - 220	73	4.4		2.2		39.6

Table 3. Geotechnical Pile Design Parameters (continued; Sheet 3 of 3)

Profile Matching Conditions in Boring A-9							
Description	**Depth Below Mudline (feet)**	**Submerged Unit Weight, γ' (pcf)**	**Undrained Shear Strength, S_u (ksf)**	**Soil-Pile Friction Angle, δ (degrees)**	**Limiting Skin Friction, F_{max} (kips/ft^2)**	**Bearing Capacity Factor, N_Q**	**Limiting Unit End Bearing, Q_{max} (kips/ft^2)**
Very dense slightly silty, gravelly sand	0 - 17	68		35	2.4	50	250
Very dense, sandy gravel	17 - 80	78		38	3.0	60	280
Very dense, silty, gravelly sand	80 - 95	68		35	2.4	50	250
Hard, slightly gravelly sandy clay	95 - 140	70	4.0		2.0		360
Profile Matching Conditions in Boring A-10							
Description	**Depth Below Mudline (feet)**	**Submerged Unit Weight, γ' (pcf)**	**Undrained Shear Strength, S_u (ksf)**	**Soil-Pile Friction Angle, δ (degrees)**	**Limiting Skin Friction, F_{max} (kips/ft^2)**	**Bearing Capacity Factor, N_Q**	**Limiting Unit End Bearing, Q_{max} (kips/ft^2)**
Loose to medium dense, fine sand	0 - 43	63		15	1	8	40
Dense to very dense fine sand	43 - 126	67		30	2	40	200
Medium dense, silty fine sand	126 - 150	77		35	2.4	50	250
Hard, sandy, silty Clay	150 - 190	65	5		2.5		45
Very dense, silty, sandy gravel	190 - 210	78		35	2.4	50	250
Hard, slightly sand gravelly clay	210 - 240	70	6		3.0		65



1927 North American Datum; Topography in Feet
Generated by Big Topo (www.igage.com)
Map compiled from USGS Quads: Tyonek B-1 63K: AK
Anchorage B-8 63K: AK Tyonek A-1 63K: AK
Anchorgae A-8 63K: AK Anchorage A-7 63K: AK

Note: Due to the depth of water and strong currents, Borings A-5 and A-10 had to be moved North of alignment to set up and drill holes in the center of the channel.

LEGEND

- Boring Number and Approximate Location
- Channel Crossing Profile A-A'
- East Shoreline Profile B-B'

Project Datum: MLLW

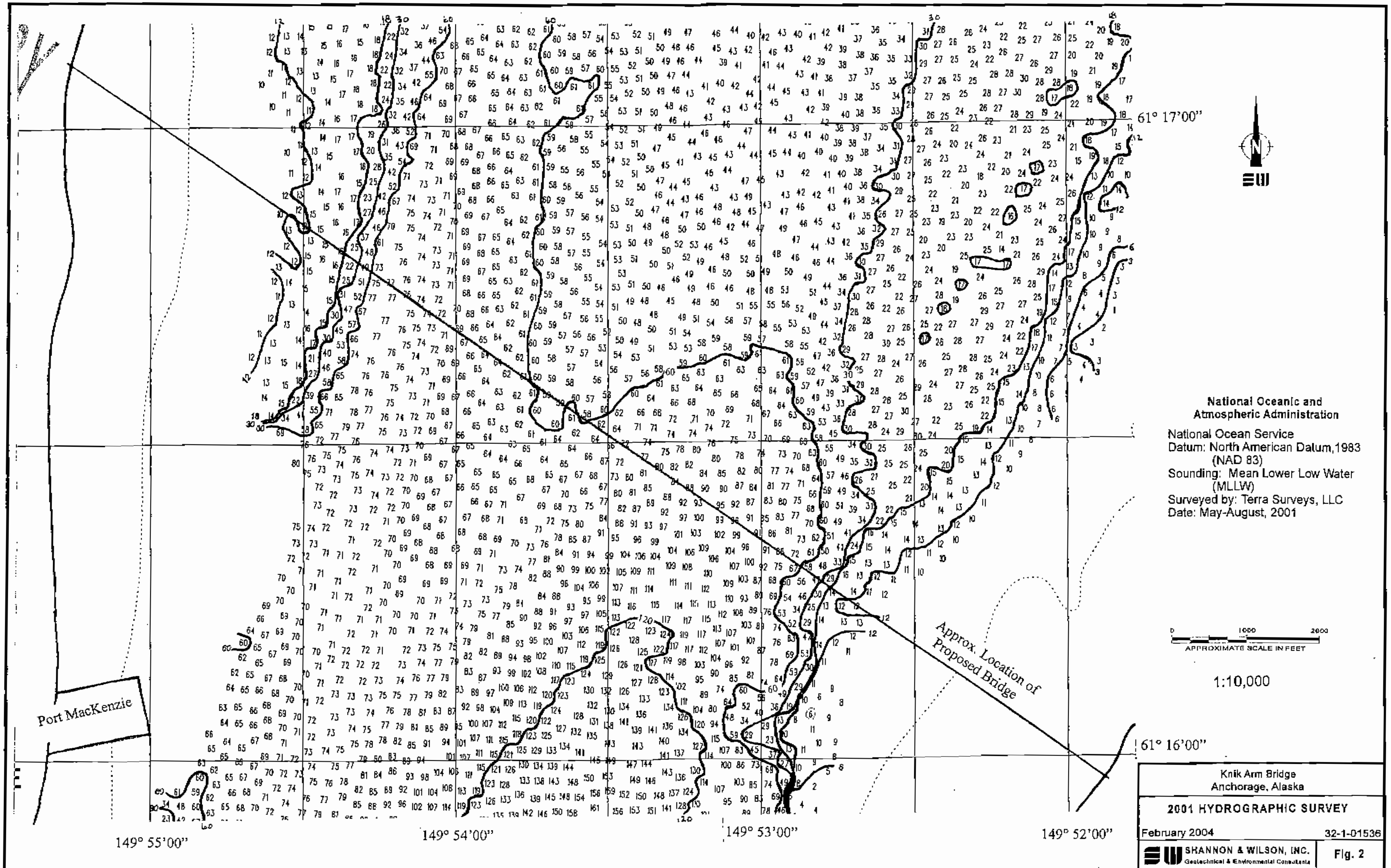
Knik Arm Bridge
Anchorage, Alaska

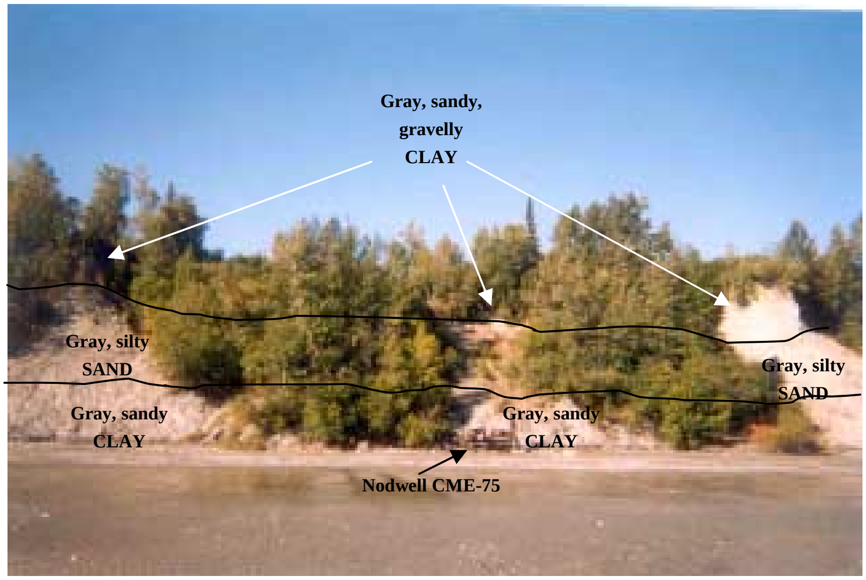
BORING LOCATION PLAN

February 2004 — 32-1-01536

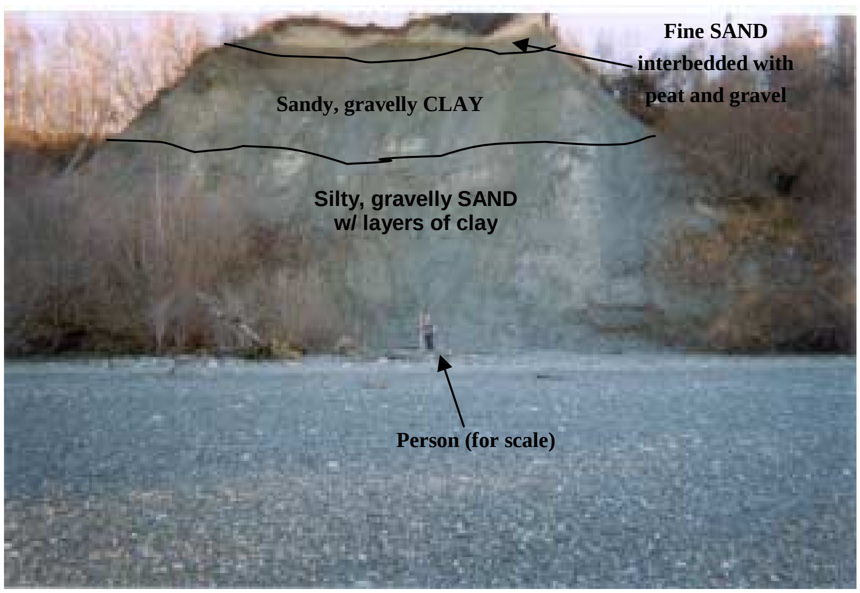
SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. 1


61° 17'00"
61° 16'00"
149° 55'00"
149° 54'00"
149° 53'00"
149° 52'00"
Port MacKenzie
Approx. Location of Proposed Bridge
National Oceanic and Atmospheric Administration
National Ocean Service
Datum: North American Datum,1983 (NAD 83)
Sounding: Mean Lower Low Water (MLLW)
Surveyed by: Terra Surveys, LLC
Date: May-August, 2001
0
1000
2000
APPROXIMATE SCALE IN FEET
1:10,000
Knik Arm Bridge
Anchorage, Alaska
2001 HYDROGRAPHIC SURVEY
February 2004
32-1-01536
SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants
Fig. 2

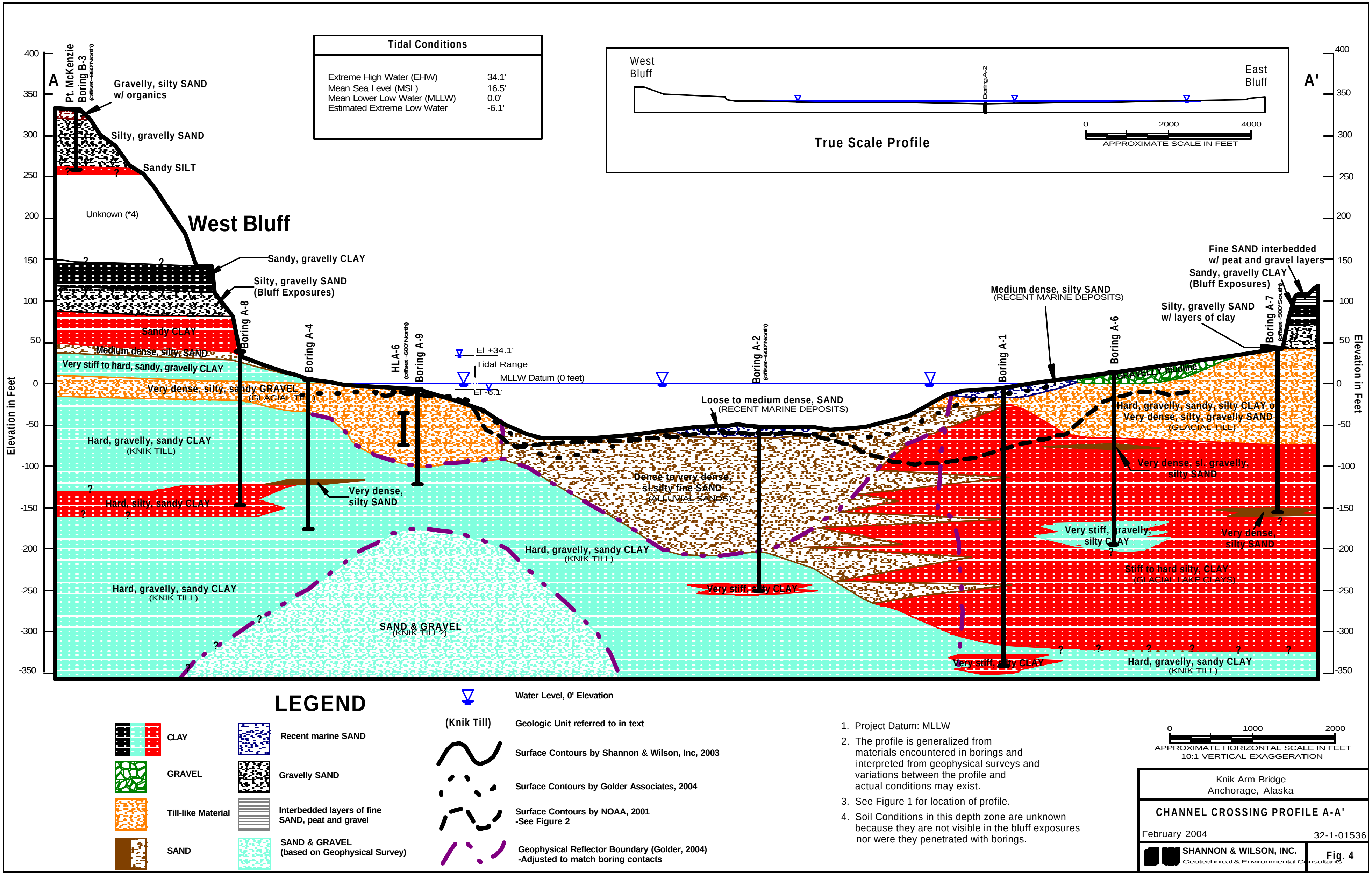


Photograph 1: West bluff geology at Boring A-8.

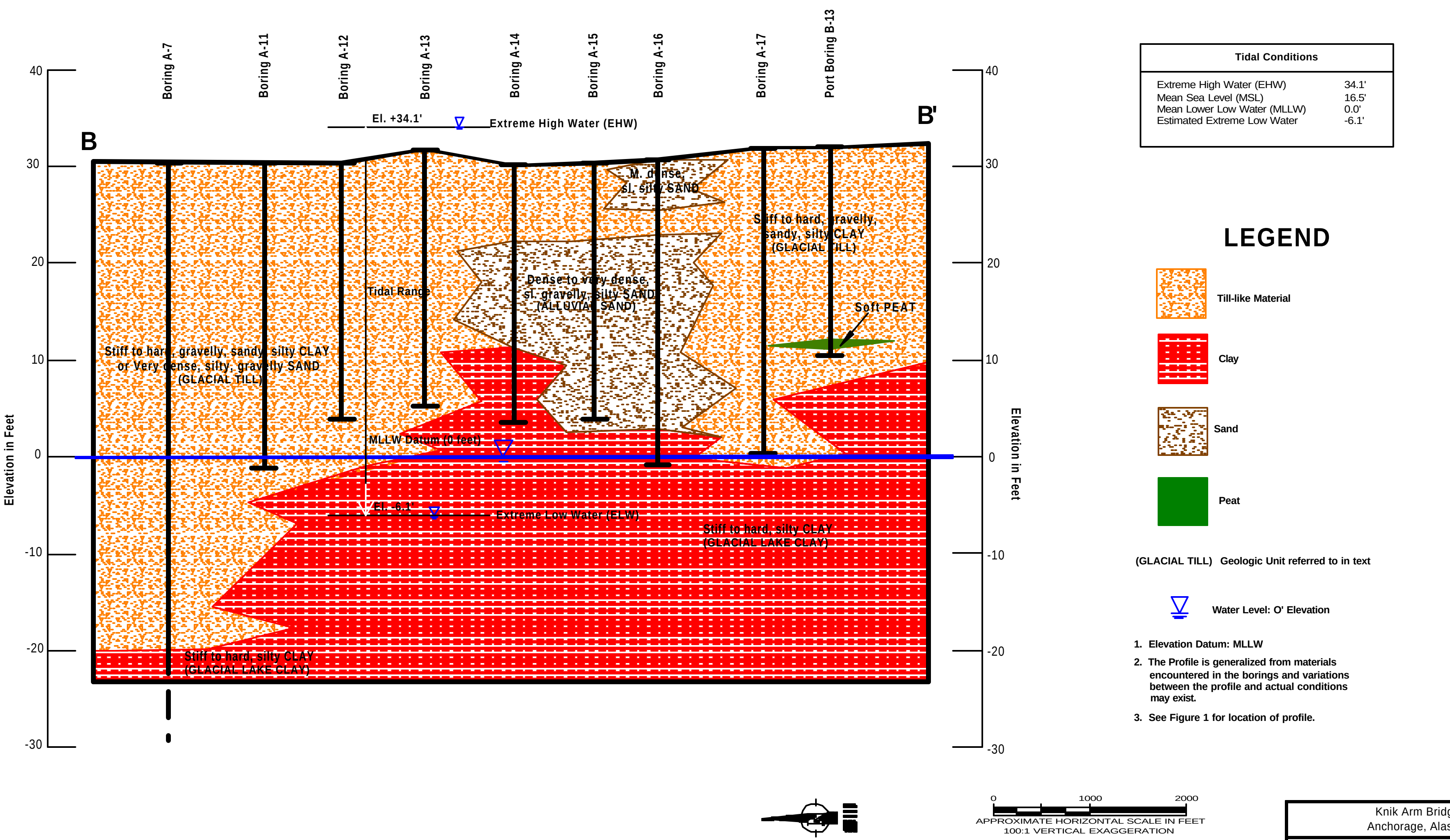


Photograph 2: East Bluff Geology at Boring A-12.

Knik Arm Bridge Anchorage, Alaska
SUMMARY OF BLUFF CONDITIONS
February 2004 — 32-1-01536
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants — Fig. 3



Tidal Conditions
Extreme High Water (EHW) 34.1'
Mean Sea Level (MSL) 16.5'
Mean Lower Low Water (MLLW) 0.0'
Estimated Extreme Low Water -6.1'
West Bluff
East Bluff
Boring A-2
True Scale Profile
0 2000 4000
APPROXIMATE SCALE IN FEET
A
A'
Elevation in Feet
Pt. McKenzie Boring B-3
Gravelly, silty SAND w/ organics
Silty, gravelly SAND
Sandy SILT
Unknown (*4)
West Bluff
Sandy, gravelly CLAY
Silty, gravelly SAND (Bluff Exposures)
Sandy CLAY
Medium dense, silty SAND
Very stiff to hard, sandy, gravelly CLAY
Very dense, silty, sandy GRAVEL (GLACIAL TILL)
Hard, gravelly, sandy CLAY (KNIK TILL)
Hard, silty, sandy CLAY
Very dense, silty SAND
Hard, gravelly, sandy CLAY (KNIK TILL)
SAND & GRAVEL (KNIK TILL?)
Boring A-8
Boring A-4
HLA-6
Boring A-9
El +34.1'
Tidal Range
MLLW Datum (0 feet)
El -6.1'
Boring A-2
Loose to medium dense, SAND (RECENT MARINE DEPOSITS)
Dense to very dense, sl.silty fine SAND (ALLUVIAL SANDS)
Hard, gravelly, sandy CLAY (KNIK TILL)
Very stiff, silty CLAY
Boring A-1
Medium dense, silty SAND (RECENT MARINE DEPOSITS)
GRAVELLY moraine
Boring A-6
Silty, gravelly SAND w/ layers of clay
Boring A-7
Sandy, gravelly CLAY (Bluff Exposures)
Fine SAND interbedded w/ peat and gravel layers
Hard, gravelly, sandy, silty CLAY or Very dense, silty, gravelly SAND (GLACIAL TILL)
Very dense, sl. gravelly, silty SAND
Very stiff, gravelly, silty CLAY
Very dense, silty SAND
Stiff to hard silty CLAY (GLACIAL LAKE CLAYS)
Very stiff, silty CLAY
Hard, gravelly, sandy CLAY (KNIK TILL)
LEGEND
CLAY
GRAVEL
Till-like Material
SAND
Recent marine SAND
Gravelly SAND
Interbedded layers of fine SAND, peat and gravel
SAND & GRAVEL (based on Geophysical Survey)
Water Level, 0' Elevation
(Knik Till) Geologic Unit referred to in text
Surface Contours by Shannon & Wilson, Inc, 2003
Surface Contours by Golder Associates, 2004
Surface Contours by NOAA, 2001 -See Figure 2
Geophysical Reflector Boundary (Golder, 2004) -Adjusted to match boring contacts
1. Project Datum: MLLW
2. The profile is generalized from materials encountered in borings and interpreted from geophysical surveys and variations between the profile and actual conditions may exist.
3. See Figure 1 for location of profile.
4. Soil Conditions in this depth zone are unknown because they are not visible in the bluff exposures nor were they penetrated with borings.
0 1000 2000
APPROXIMATE HORIZONTAL SCALE IN FEET
10:1 VERTICAL EXAGGERATION
Knik Arm Bridge
Anchorage, Alaska
CHANNEL CROSSING PROFILE A-A'
February 2004
32-1-01536
SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants
Fig. 4



Knik Arm Bridge
Anchorage, Alaska

EAST SHORELINE PROFILE B-B'

February 2004

32-1-01536

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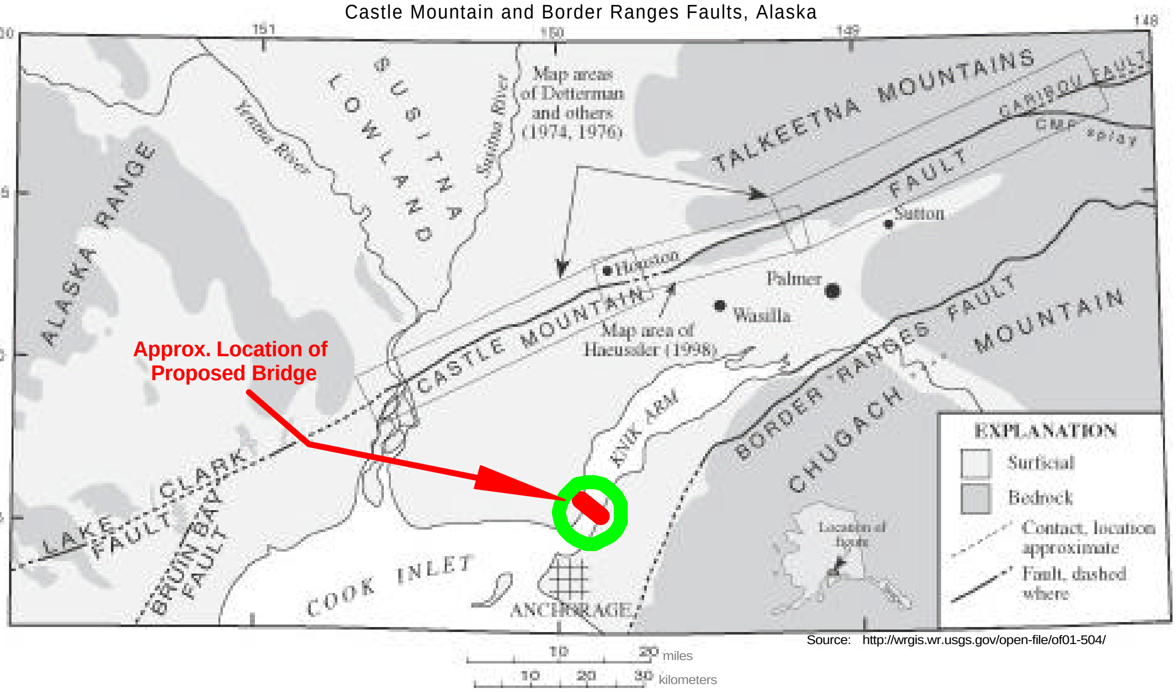
Fig. 5

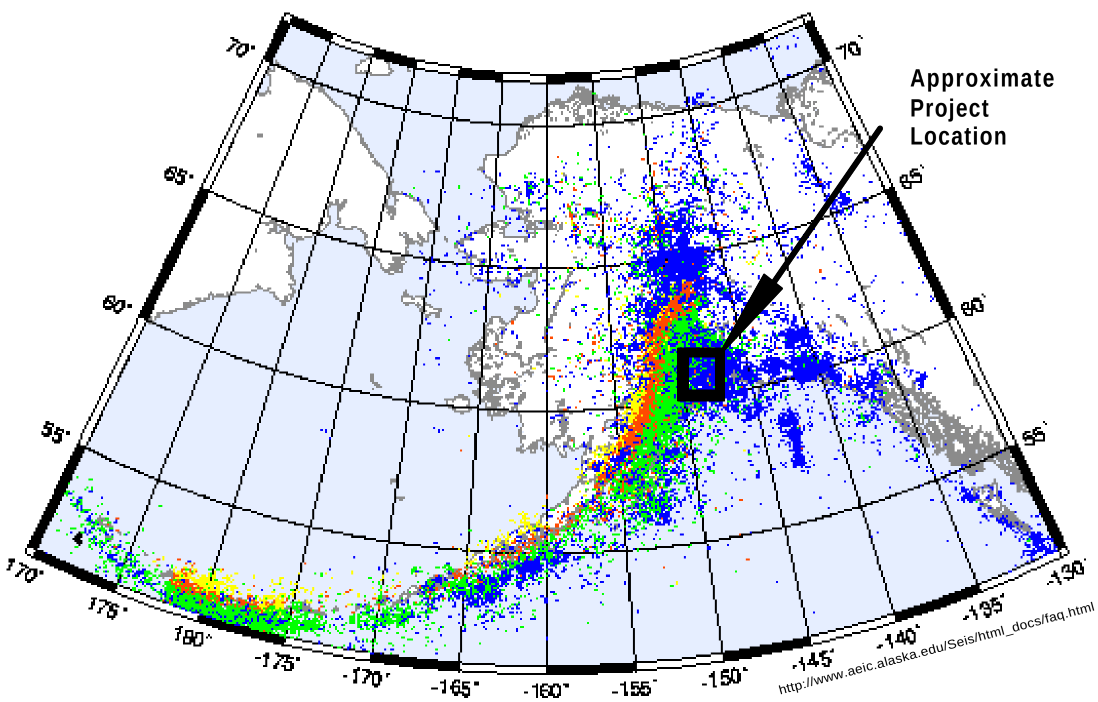
Castle Mountain and Border Ranges Faults, Alaska



Source: http://wrgis.wr.usgs.gov/open-file/of01-504/

Alaska Seismicity 1898-2002

Depth color key: blue = 0-33km, green = 33-75km, red = 75-125km, yellow = 125+km



Knik Arm Bridge
Anchorage, Alaska

FAULTING AND EARTHQUAKE MAP

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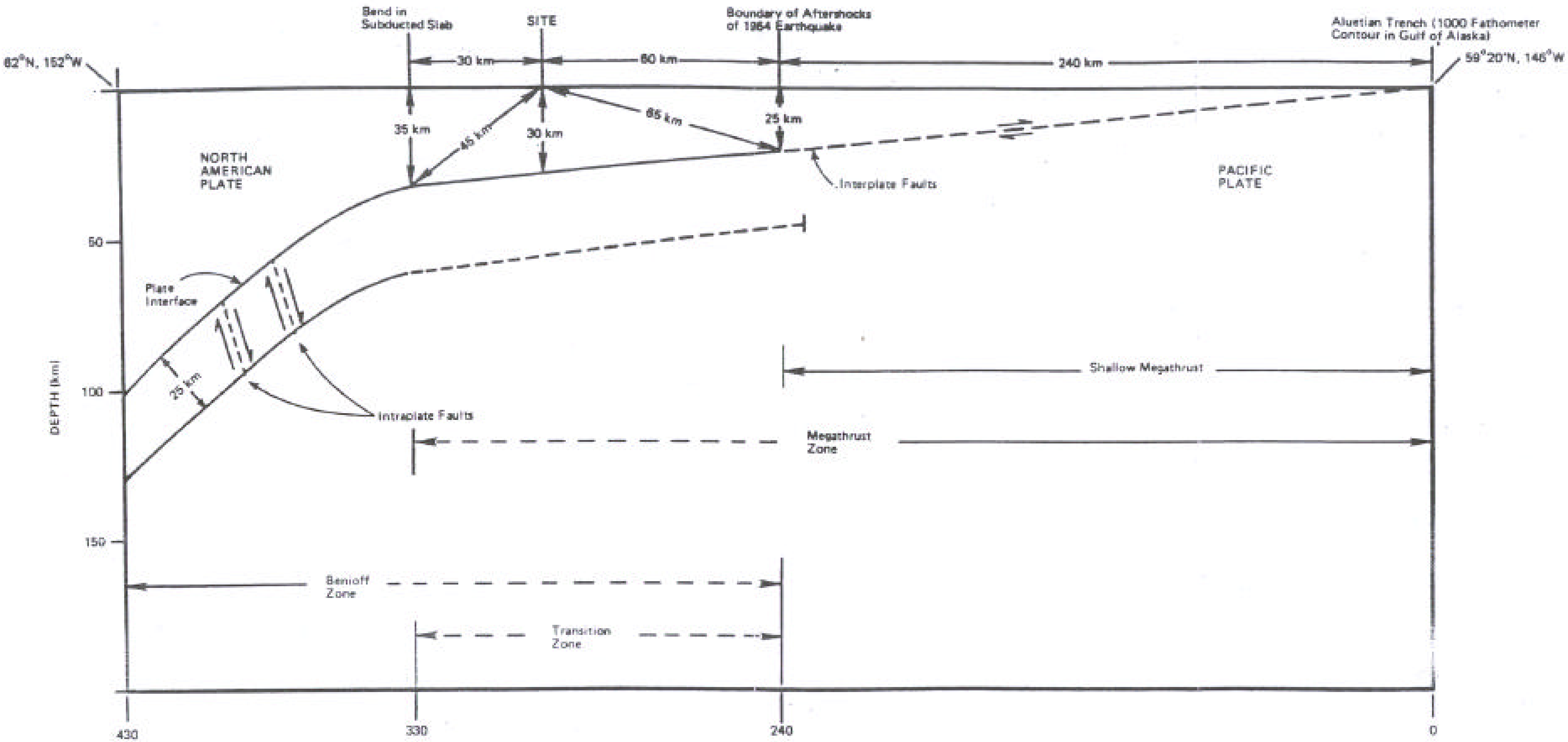
Fig. 6



Taken from Harding Lawson Associates, 1984

Knik Arm Bridge Anchorage, Alaska	
SEISMIC SOURCES IN ANCHORAGE AREA February 2004 32-1-01536	
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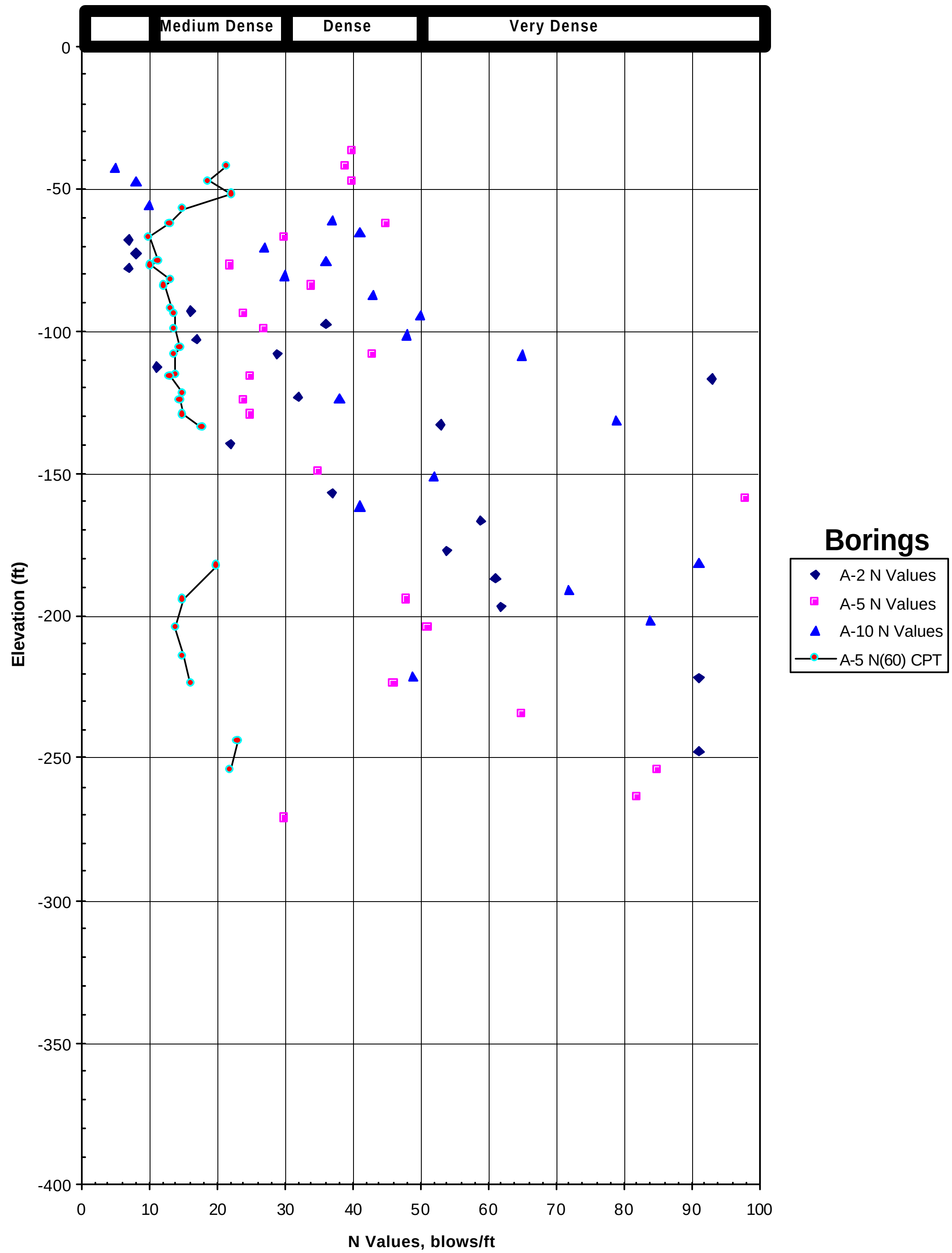

Notes:
1. N values are Standard Penetration Test Results
2. N values in Borings A-2, 5, and 10 are uncorrected for energy loss and confirming pressure effects.
3. N(60) CPT-N values are calculated assuming 60 percent energy loss and are uncorrected for confining pressure effects.
4. Elevation data: MLLW

Knik Arm Bridge
Anchorage, Alaska

SAND DENSITY VALUES

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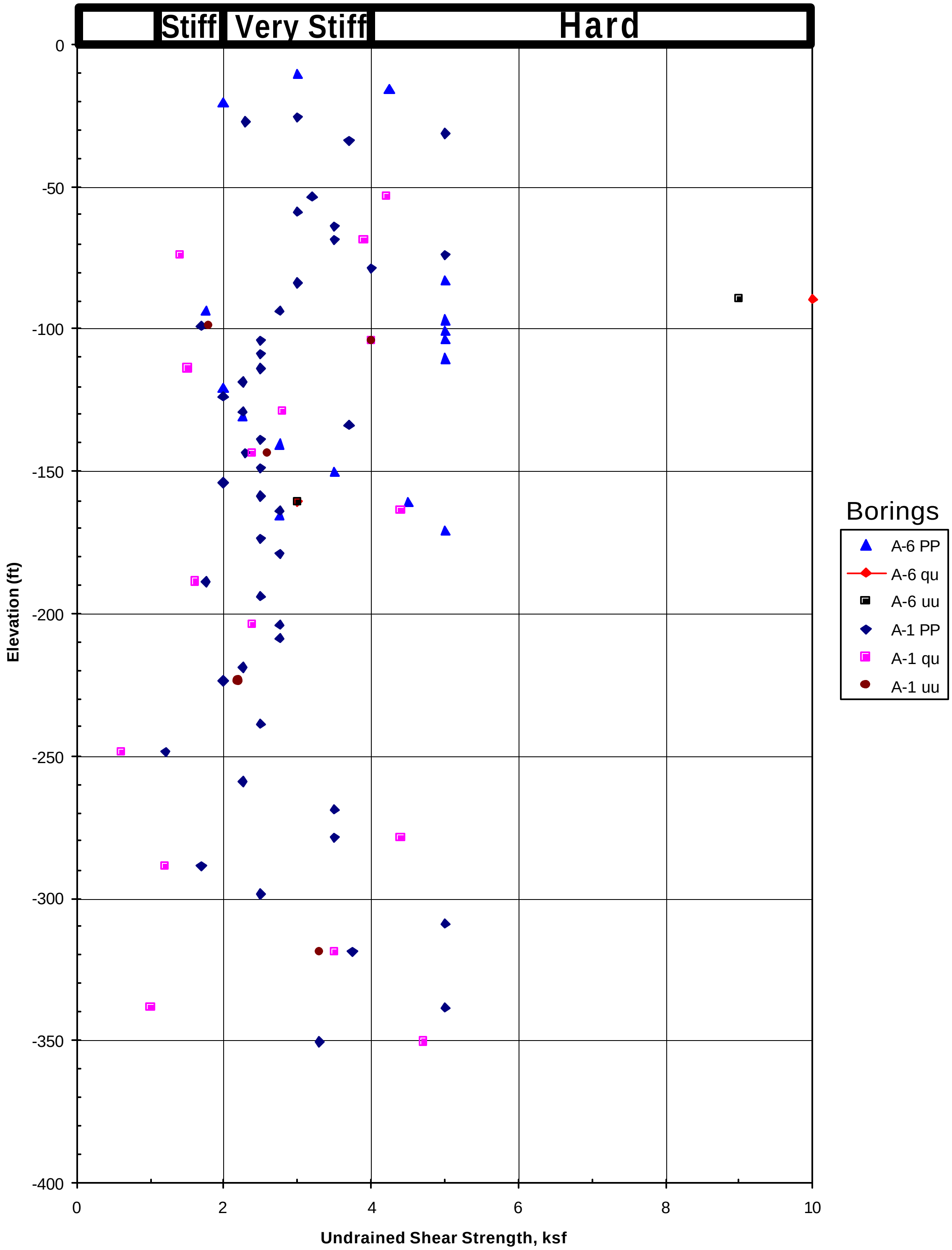
Fig. 8



NOTES:

1. pp denotes 1/2 pocket penetrometer measurements, when values exceed limit of test (<4.5) a value of 5 ksf was plotted.
2. q_u denotes 1/2 unconfined compressive strength.
3. uu denotes $s_1 - s_3/2$
4. Elevation datum: MLLW

Knik Arm Bridge
Anchorage, Alaska

TYPICAL CLAY STRENGTHS

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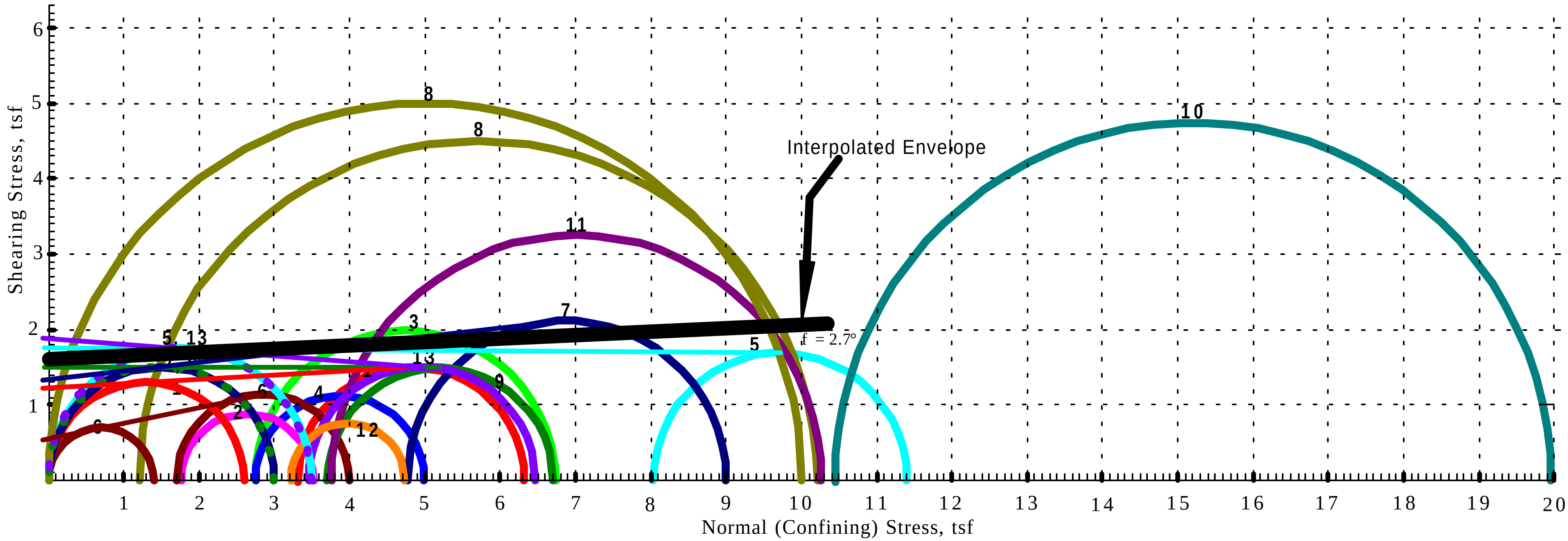
Fig. 9



Curve	Sample	Depth (ft)	Max Stress (tsf) Unconfined	Max Stress (tsf) Triaxial		s1=s+s3		Confining P (tsf)	Description
			s1	s1-s3		s1		s3	
1	A1 S9	35	2.6	3		6.3		3.3	Silty CLAY (CL)
2	A1 S19	85		1.8		3.6		1.8	Silty CLAY (CL)
3	A1 S26	120		3.9		6.78		2.88	Silty CLAY (CL)
4	A1 S40	210		2.2		5.08		2.88	Silty CLAY (CL)
5	A1 S50	305	3.5	3.3		11.4		8.1	Silty CLAY, occasional gravel (CL)
6	A5 S17	83	1.4	2.3		4		1.7	Sandy, silty CLAY with sand seams (CL)
7	A5 S30	183	3	4.2		9		4.8	Gravelly, silty fine SAND (SM)
8	A6 S18	97	10	9		10.2		1.2	Silty CLAY with sand seams (CL)
9	A6 S31	175	3	3		6.7		3.7	Silty CLAY with sand seams (CL)
10	A7 S25	165		9.5		19.9		10.4	Silty CLAY with sand seams (CL)
11	A7 S27	185		6.5		10.3		3.8	Silty CLAY with sand seams (CL)
12	A8 S19	130		1.5		4.7		3.2	Gravelly, sandy, silty CLAY (CL)
13	A10 S23	175.5	3.5	3		6.4		3.4	Gravelly, sandy, silty CLAY (CL)

Knik Arm Bridge
Anchorage, Alaska

MOHR CIRCLE DIAGRAMS

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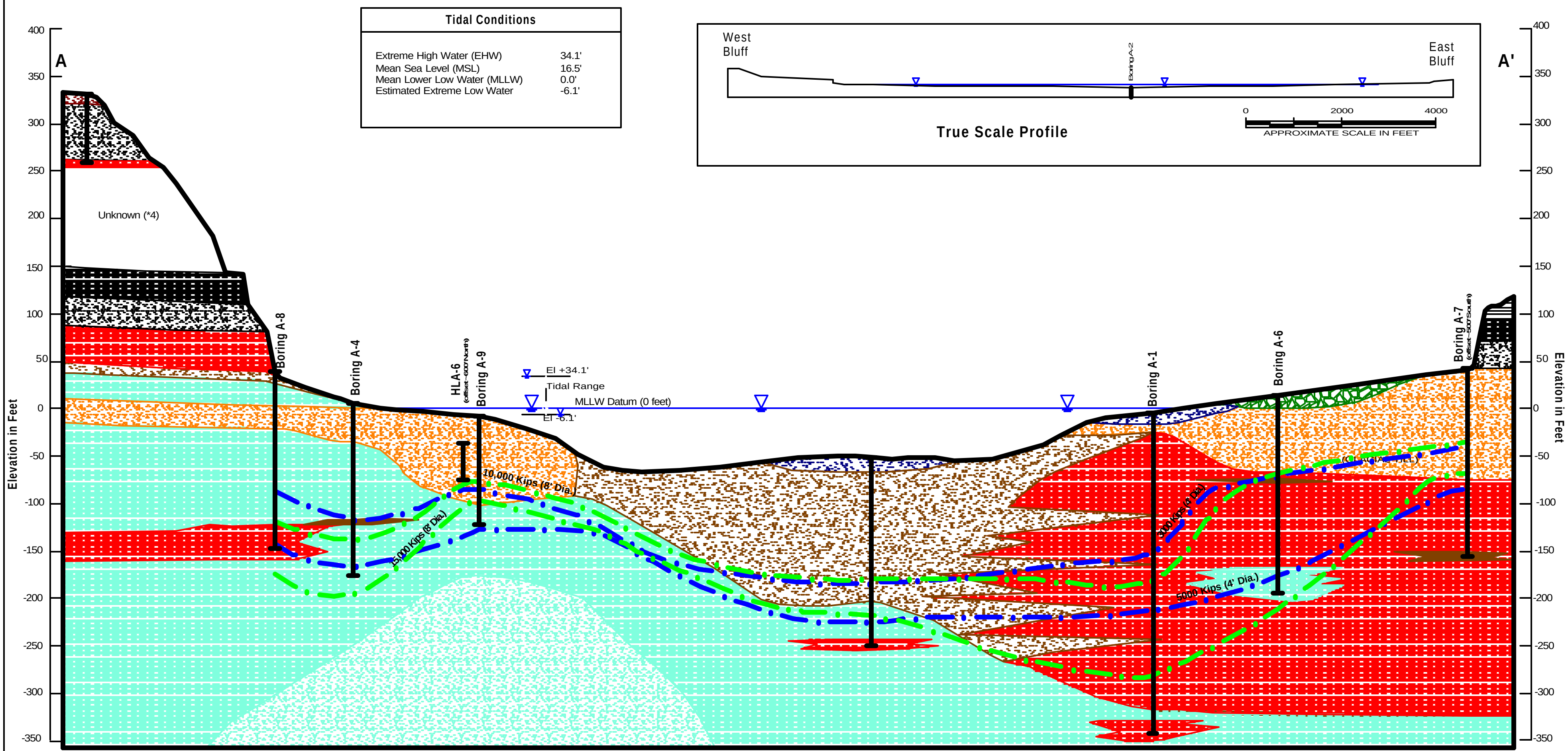
Fig. 10



LEGEND

- CLAY
- GRAVEL
- Till-like Material
- SAND
- Recent marine SAND
- Gravelly SAND
- Interbedded layers of fine SAND, peat and gravel
- SAND & GRAVEL (based on Geophysical Survey)
- Water Level, 0' Elevation
- (Knik Till) Geologic Unit referred to in text
- 8 Foot Diameter Piles
- 4 Foot Diameter Piles

1. Project Datum MLLW
2. The profile is generalized from materials encountered in borings and interpreted from geophysical surveys and variations between the profile and actual conditions may exist.
3. See Figure 1 for location of profile.
4. Soil Conditions in this depth zone are unknown because they are not visible in the bluff exposures nor were they penetrated with borings.

0 1000 2000
APPROXIMATE HORIZONTAL SCALE IN FEET
10:1 VERTICAL EXAGGERATION

Knik Arm Bridge
Anchorage, Alaska

SUMMARY OF PILE EMBEDMENT DEPTHS

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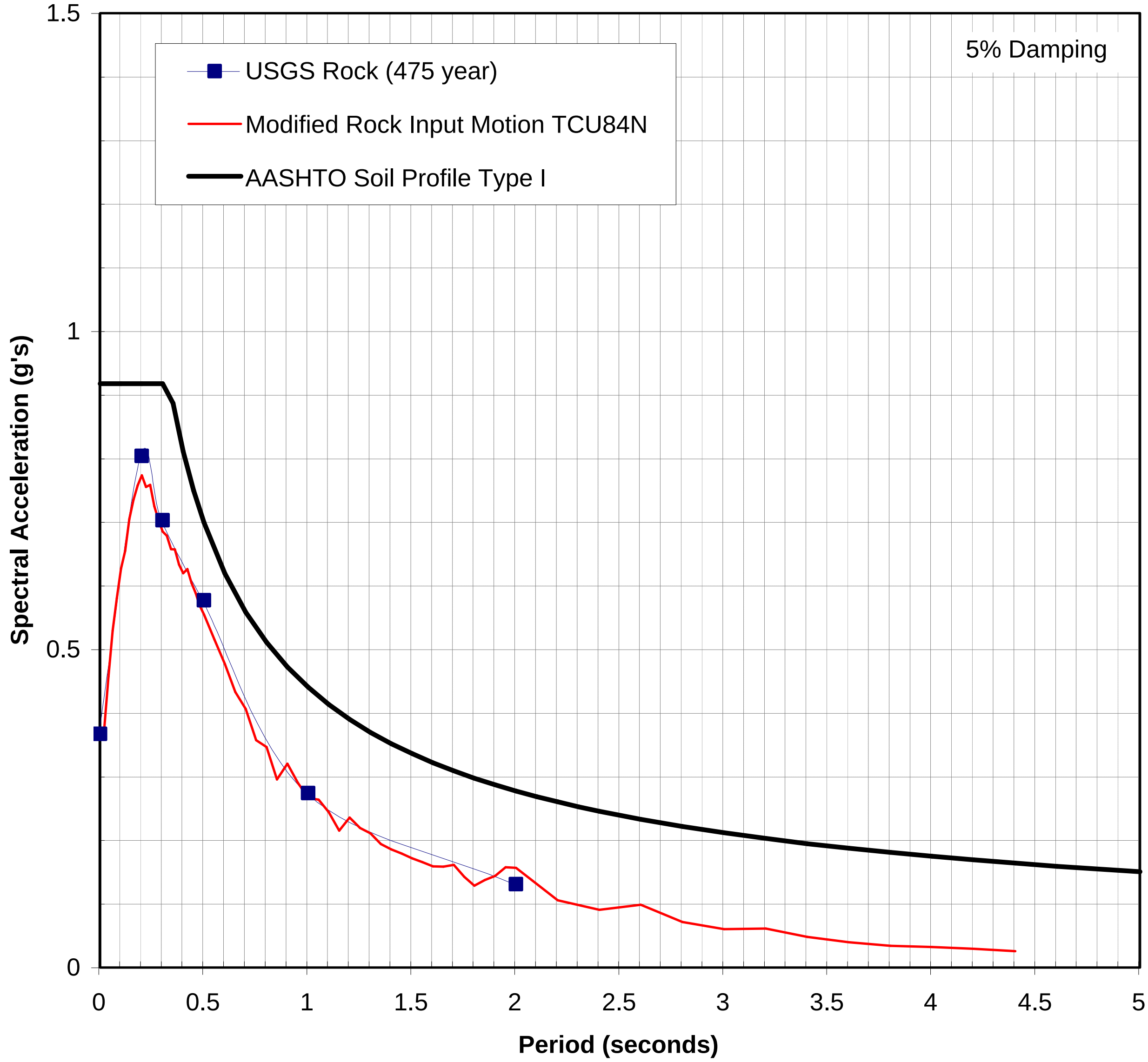
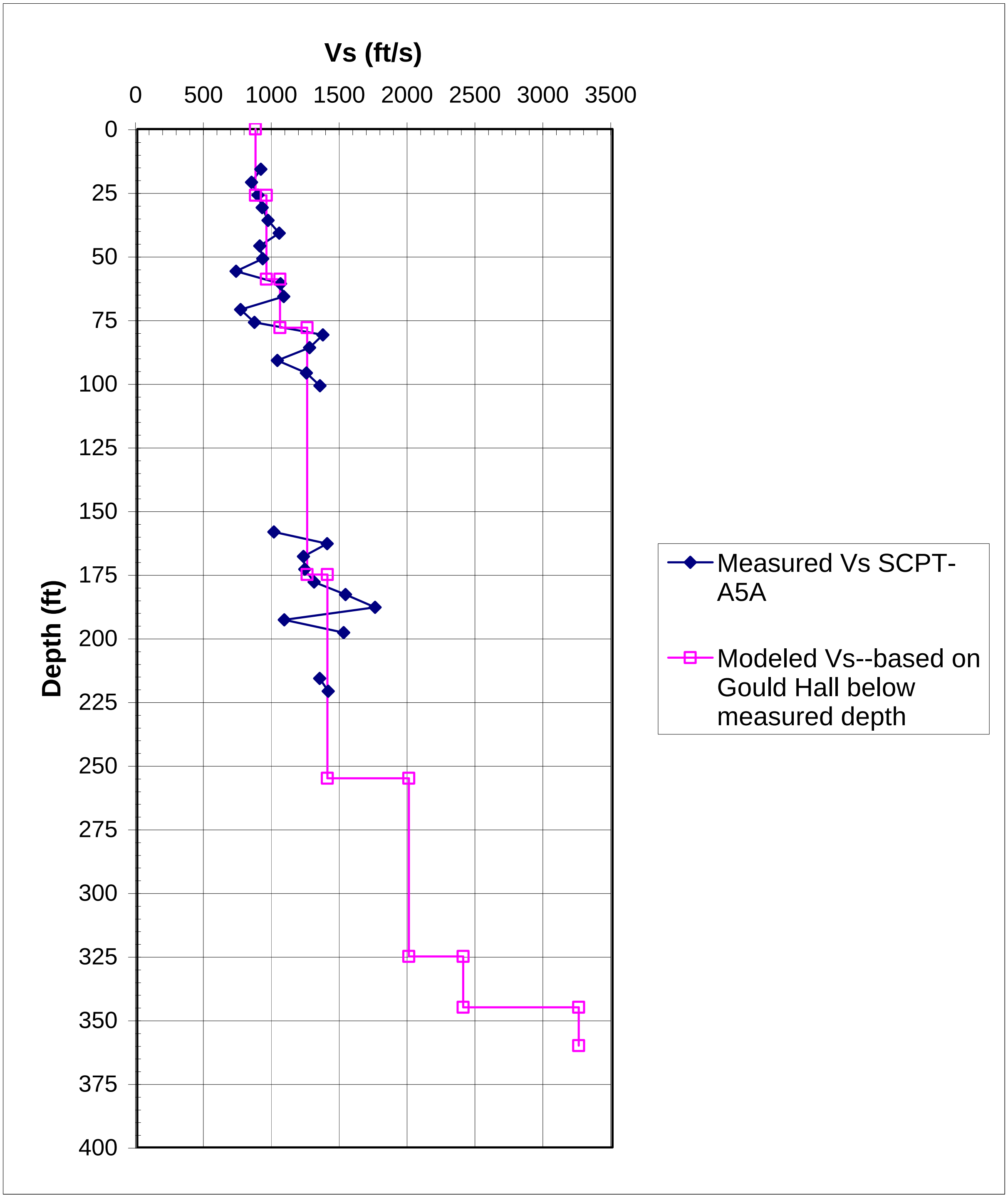
Fig. 11



NOTE

Spectra represent horizontal free-field ground motions at the ground surface.

Knik Arm Crossing Anchorage, Alaska	
ROCK SITE RESPONSE SPECTRA February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 12**



Knik Arm Crossing
Anchorage, Alaska

MEASURED AND MODELED SHEAR WAVE VELOCITY

February 2004

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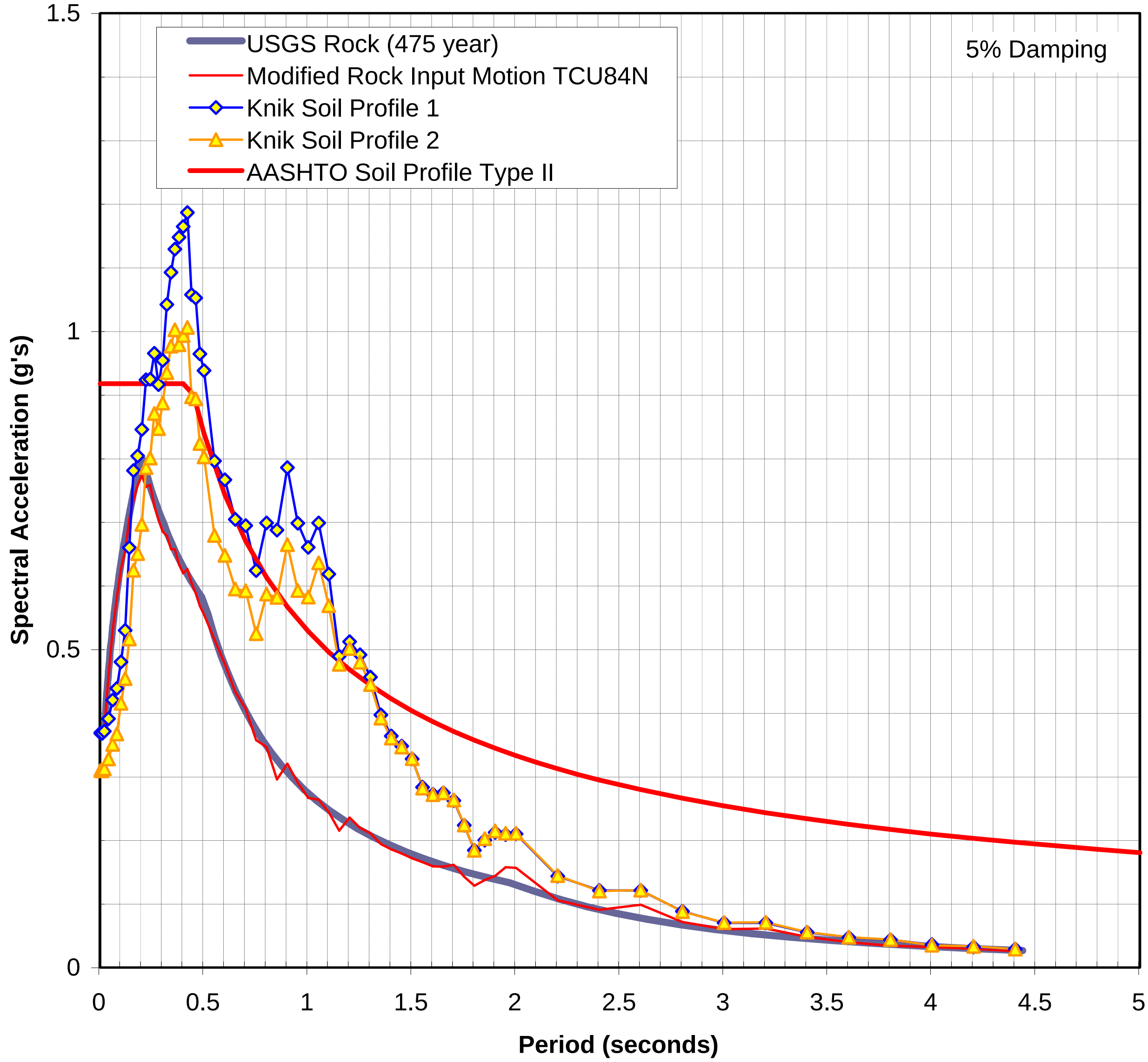
FIG. 13

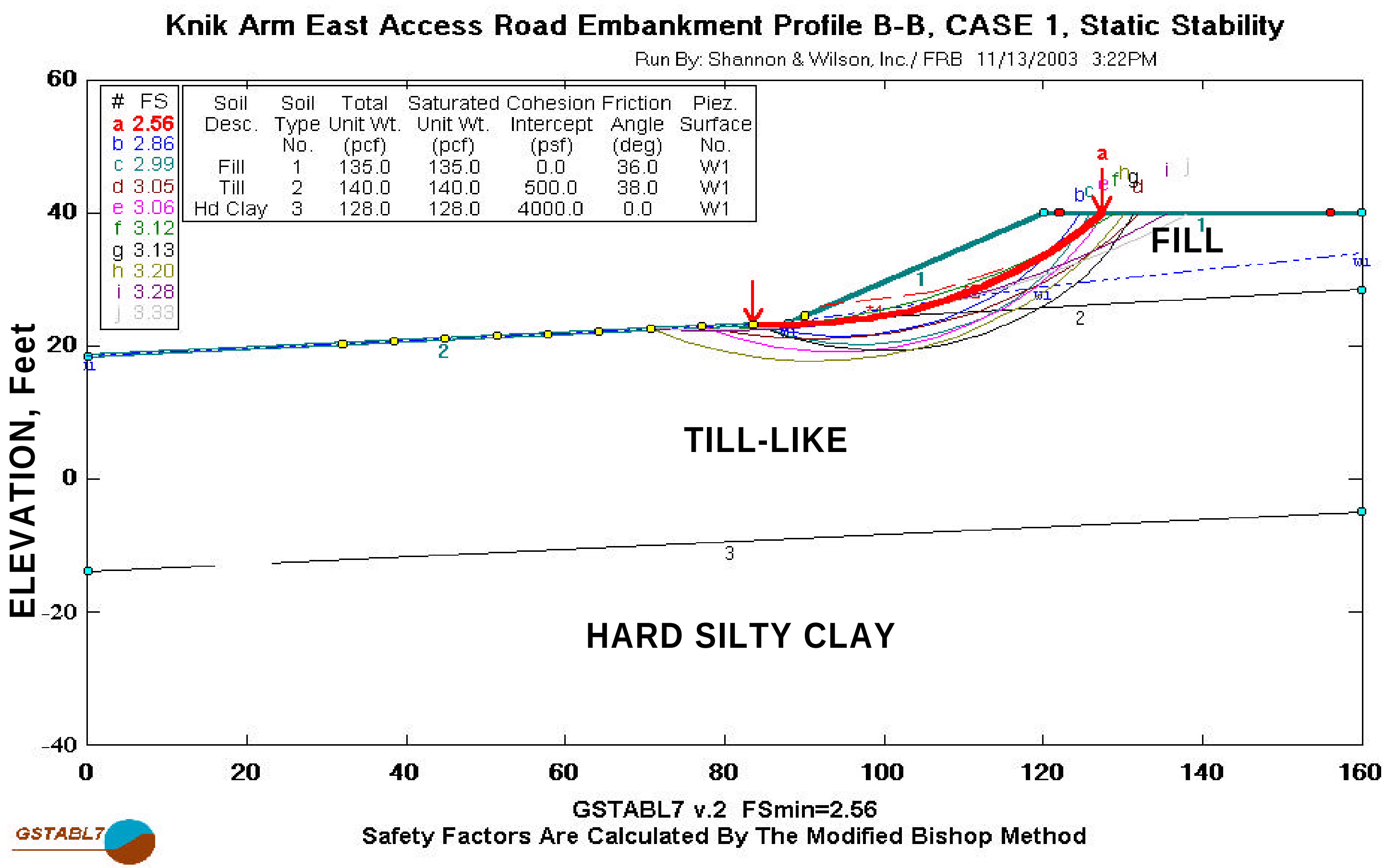


NOTES

1. Spectra represent horizontal free-field ground motions at the ground surface.

2. Knik Soil Profile 1 corresponds to a predominantly cohesionless profile (i.e., sands and gravels). Knik Soil Profile 2 corresponds to a predominatly cohesive profiles (i.e., silts and clays).

Knik Arm Crossing Anchorage, Alaska	
ROCK AND SOIL SITE RESPONSE SPECTRA February 2004	32-1-01536
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 14**



Knik Arm Bridge
Anchorage, Alaska

**STATIC STABILITY
CASE 1**

February 2004

32-1-01536

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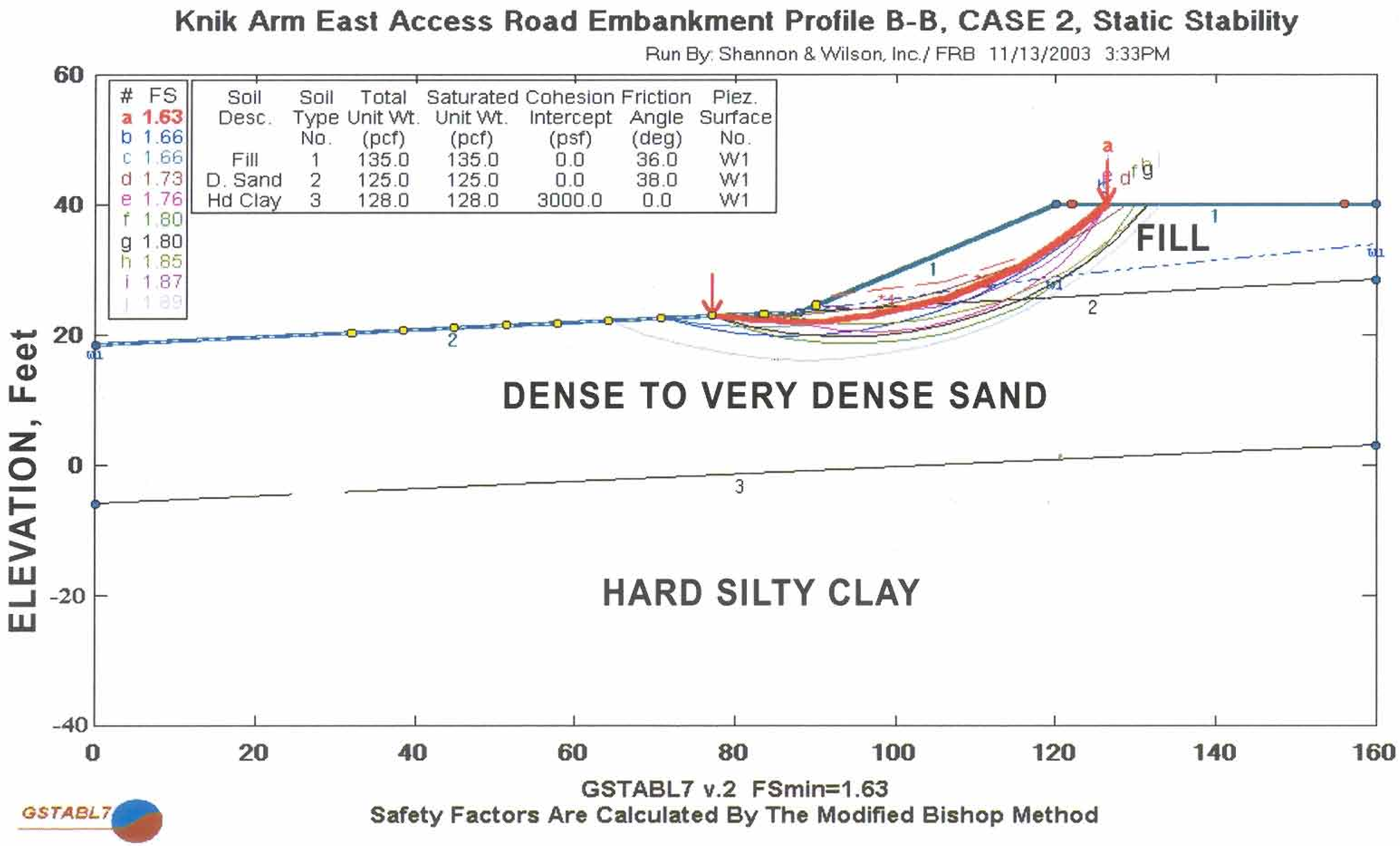
Fig. 15



Knik Arm Bridge
Anchorage, Alaska

STATIC STABILITY
CASE 2

February 2004

32-1-01536

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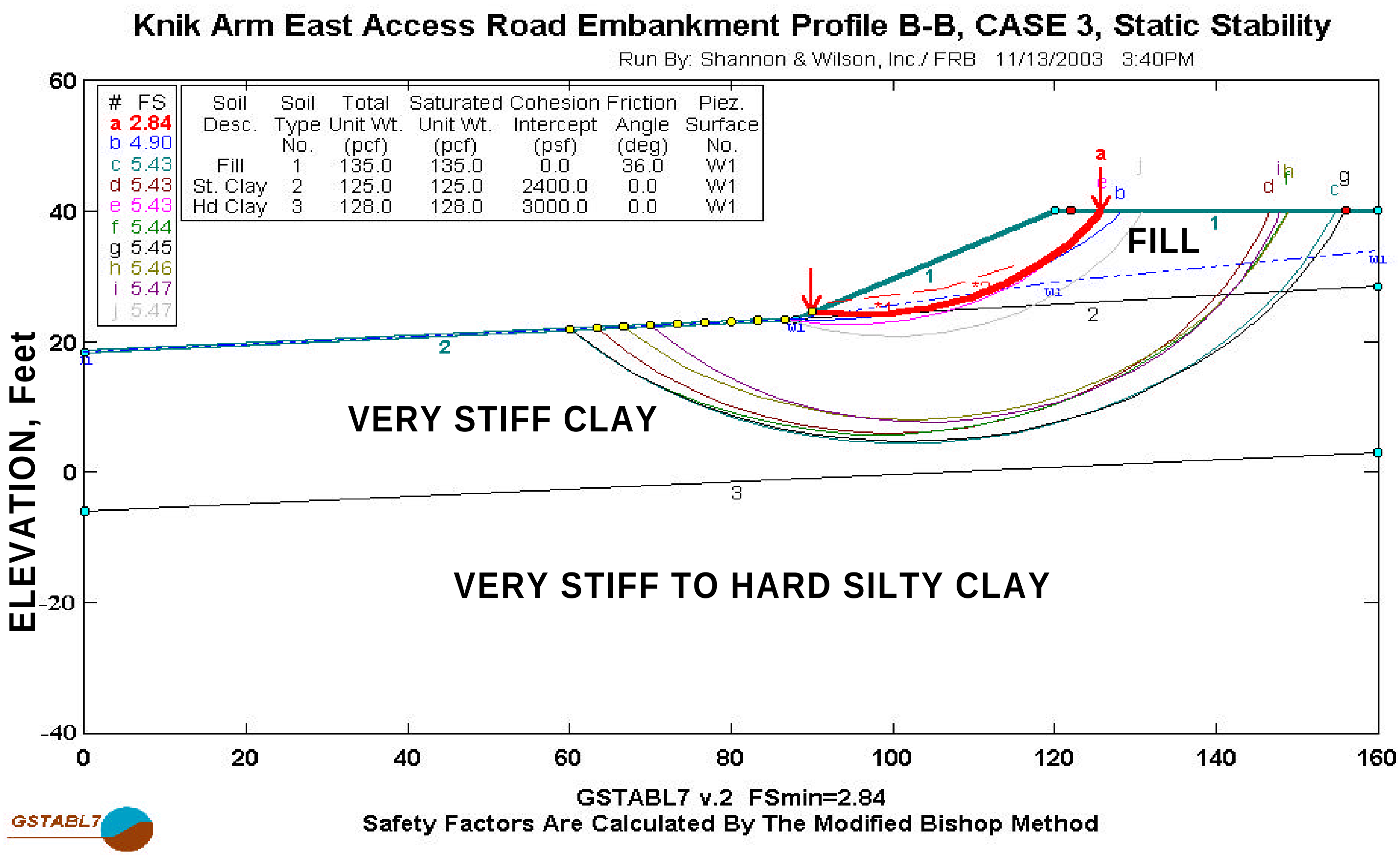
Fig. 16



Knik Arm Bridge Anchorage, Alaska	
SLOPE STABILITY RESULTS **CASE 3** February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	**Fig. 17**

APPENDIX A

GEOLOGICAL RECONNAISSANCE

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APPENDIX A
GEOLOGICAL RECONNAISSANCE

A geologist photographed and mapped bluff conditions along both shores of Knik Arm directly adjacent to the corridor shown in Figure 1. The west and east sides were evaluated on September 8 and October 28, 2003, respectively. Our observations at each of the bluffs and along the shores are described below. A Survey GPS was used to assist in determining bluff heights and the elevations of features of interest.

A-1 East Side Bluff

The east bluff of the proposed Knik Arm Crossing, shown in Photographs A-1 through A-8, is approximately 70 feet high with multiple ravines and slough material along the shoreline. Three main units can be identified in the exposed face of the bluff, shown in Photograph A-2. The lower unit is represented by a 30 feet section of gray, silty, gravelly sand with interbedded layers of clay up to 2 inches thick. This unit consists mostly of medium-grained sand and is heavily stratified, containing multiple clay layers. These structures are indicative of a fluvial system where outwash material has been reworked by stream channels intermingling the sands and clays. At the location of Boring A-7, this unit was obscured by landslide and slough material. Excellent examples of this unit are observed at Boring A-12, shown in Photographs A-3 and A-4, and at Boring A-14, south of the Boring A-7 location along the shoreline.

Overlying the lower sand unit is approximately 22 feet of gray, sandy, gravelly clay. This clay layer lacks structure and bedding, indicating a till-like material. The clay is covered by the remaining upper portion of the bluff.

The upper portion of the east bluff is an assortment of interbedded fine-grained sand, gravel and peat, shown in Photographs A-5 and A-6. The base of this upper unit consists of yellow, fine-grained sand overlain by a thin layer of gravel. The gravel is then overlain by approximately eight inches of organic material (peat). This sequence is repeated with approximately five feet of gray, fine-grained sand, a thin layer of gravel, and roughly two feet of peat at the top of the bluff. The yellow color of the underlying fine-grained sand is most likely a result of leaching from the upper organic layers.

The upper interbedded sand, gravel and peat is characteristic of a glacial lake formed as the glacier retreats and drops blocks of ice. The ice forms a depression, then melts and forms a small lake or pond. After time, this pond fills in with sediment and organics.

The features and units observed in the east bluff are, for the most part, continuous along the shoreline from Boring A-7 south to Cairn Point. Occasional slough and landslide material obscures the bluff in some locations.

Beginning at Cairn Point and extending south toward the Port of Anchorage is a series of landfill deposits. Elmendorf Air Force Base operated a surface dump at the top of the bluff from 1945 to 1957. Over time, debris from the landfill has slumped down slope onto the beach, shown in Photographs A-7 and A-8. Currently there are multiple locations where this landfill debris is visible and continuously being eroded by tide action. The beach is littered with broken glass, dishes and scrap metal. The lower portion of the slumped bluff contains charred rubbish consisting of wood, glass, wasted vehicles, cement blocks, and 55-gallon drums.

At the onset of our explorations, we received a summary of shoreline sweep material. Elmendorf AFB reported removing Ordnance and Explosives, mostly consisting of small arms casings. Pipes containing chrysotile, an asbestos containing material, have also been identified within the bluff debris.

A-2 West Side Bluff

The west bluff of the proposed Knik Arm Crossing, shown in Photograph A-9, is approximately 100 feet high. Three distinct soil units are exposed in the face of the bluff. The lower unit is represented by gray, sandy clay with no apparent bedding or structure indicating till-like material. This unit is at least forty feet thick. It is overlain by a 30-foot thick unit of gray, silty, gravelly sand. This unit contains mostly medium-grained sand. A unit of gray, sandy, gravelly clay, approximately 30 feet thick caps the bluff. This unit lacks structure or bedding, again indicating a till-like material

A 72-foot deep boring was drilled in September 2003 for the Matanuska-Susitna Borough at an elevation of approximately 330 feet above sea level at the top of the Elmendorf Moraine, east of Lake Lorraine. The soil encountered in the boring consisted of 11 feet of brown, gravelly, silty sand with organics overlying 59 feet of gray, silty, gravelly sand. The lower 2 feet of the boring ended in a gray, sandy silt of unknown thickness. This silt is thought to overlie the sandy, gravelly clay, which caps the main portion of the west bluff.

Tide levels encountered during drilling reached approximately 18 inches above the elevation of Boring A-8, as shown on Photograph A-10.

Photograph A-1: Nodwell CME-75 drill rig and east bluff at Boring A-13.

Photograph A-2: East bluff near Boring A-12 (note person in middle for scale).

Knik Arm Bridge
Anchorage, Alaska

PHOTOGRAPHS A-1 AND A-2

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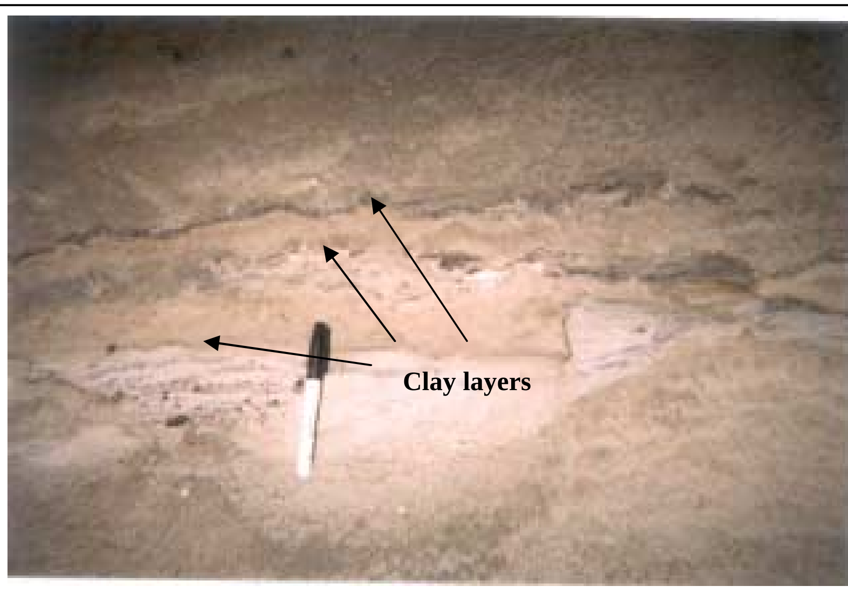
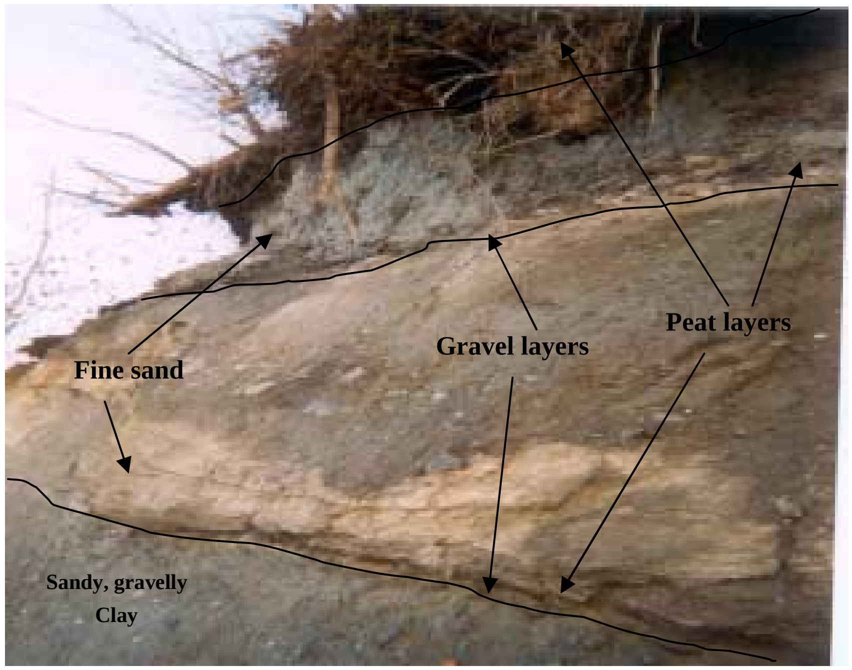
Fig. A-1

Photograph A-3: Interbedding of clay and sand observed in east bluff.

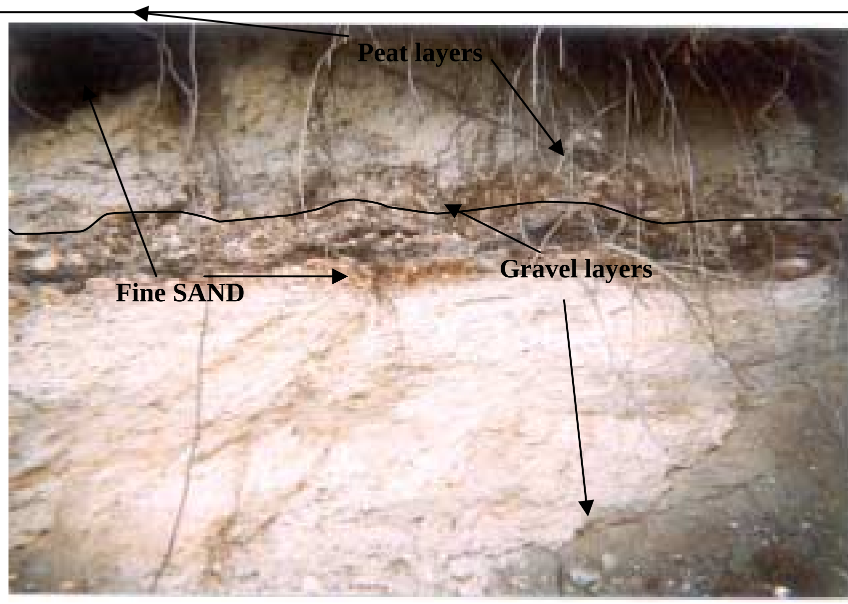


Photograph A-4: Detailed interbedding of sand and clay in east bluff.

Knik Arm Bridge Anchorage, Alaska	
PHOTOGRAPHS A-3 AND A-4	
February 2004	32-1-01536
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. A-2



Photograph A-5: Interbedding of clay and sand observed in east bluff.



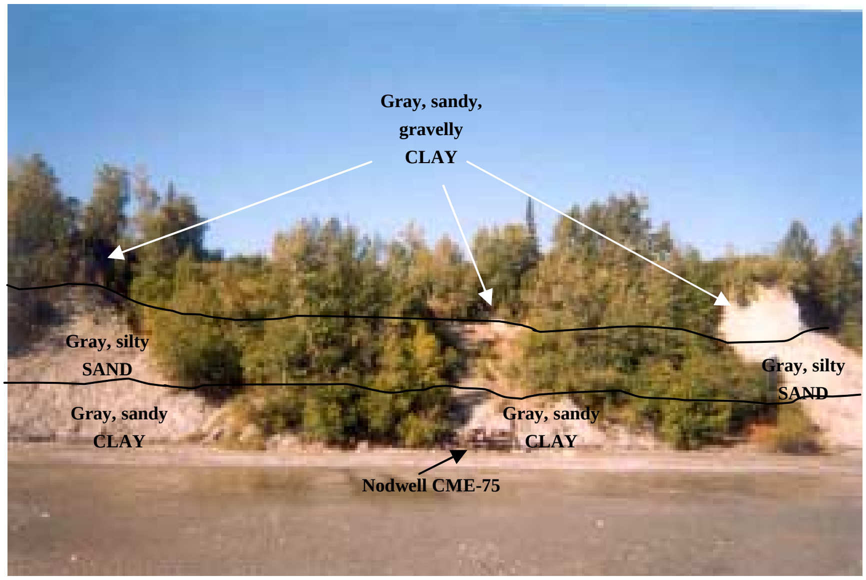
Photograph A-6: Detailed interbedding of fine sand, peat and gravel in east bluff indicating lake deposition.

Knik Arm Bridge Anchorage, Alaska	
PHOTOGRAPHS A-5 AND A-6	
February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. A-3

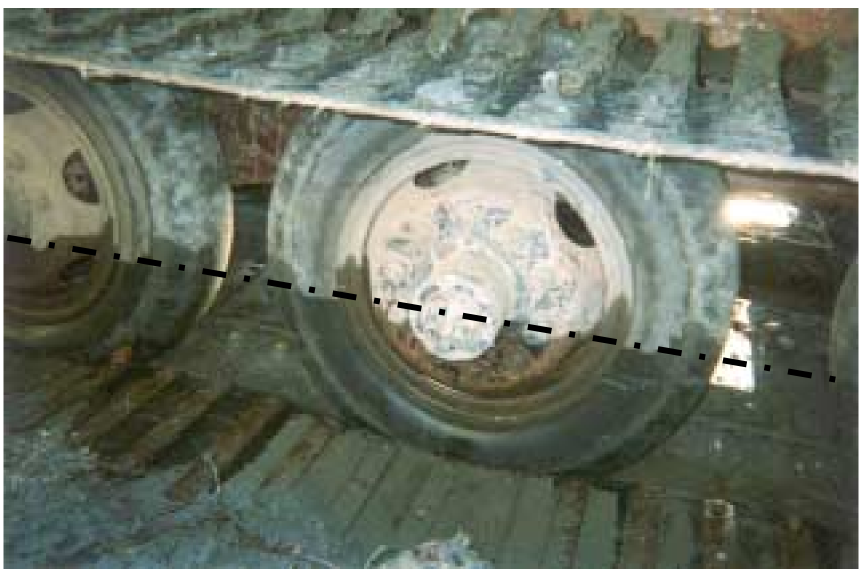
Photograph A-7: East bluff military landfill site, south of Cairn Point.

Photograph A-8: East bluff military landfill strands.

Knik Arm Bridge Anchorage, Alaska	
PHOTOGRAPHS A-7 AND A-8	
February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. A-4



Photograph A-9: Nodwell CME-75 drill rig and west bluff at Boring A-8



Photograph A-10: High tide mark from 11 September, 2003 on Nodwell wheels at west bluff, Boring A-8.

Knik Arm Bridge Anchorage, Alaska	
PHOTOGRAPHS A-9 AND A-10	
February 2004	32-1-01536
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. A-5

APPENDIX B

DRILLING AND SAMPLING PROCEDURES AND RESULTS

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APPENDIX B
DRILLING AND SAMPLING PROCEDURES AND RESULTS

B-1 Overwater Drilling

Seven overwater borings, designated A-1, A-2, A-4 through A-6, A-9, and A-10, were accomplished to depths ranging from 183 to 337 feet below the mudline at the locations shown on Figure 1. The log of each boring is presented in Figures B-1 through B-5 and B-8 and B-9. A total of 1,500 lineal feet of overwater borings were drilled and sampled. All overwater borings were drilled by Gregg Drilling from Signal Hill, California, using a Mobile B80-22 drilling rig with a 22 foot stroke. Support equipment included 12-inch outer starter casing, 6-inch inner drill casing, mud rotary drilling tools with a wireline system of rods and samplers and a jack up platform. The drilling operations were continuously monitored by field engineers or geologists from Shannon & Wilson, Inc.

The overwater drilling was performed from a 50-foot by 50-foot platform with four 100-foot long legs equipped with a center moon pool, a small crane, work and emergency skiffs, a digital global positioning system, a flow meter and a covered work space. This jack up platform, owned by Seacore, LTD from Gweek, England, is a modular unit, which for this project consisted of 6 floats pinned together. The platform is raised and lowered using jacks with a 10-foot stroke operating at a rate of roughly 5 feet per minute. The legs are 30-inches in diameter by 1-inch thick wall steel pipes and were rigged to work in 30 to 70 feet of water and accommodate 3 to 6 knot currents. The operators of the platform indicated that they noted no evidence of scour around the legs, but the feet often sank 1 to 4 feet into the mud during setup. Carl Anderson at the Port of Anchorage provided the tug to move the platform to each drill location.

The drilling work was completed over two work periods to take advantage of the more favorable tide conditions in August and September 2003. The first period started on August 16 and ended on August 22, 2003, during which time three borings, A-1, A-2 and A-4, were completed along with a CPT sounding at Boring A-1. During this time the high tides rarely reached Elevation +25 feet.

The second work period started on September 12 and finished on September 21, 2003, and resulted in four additional borings, one CPT sounding at Boring A-5, and downhole shear wave velocity measurements at Boring A-5. From tide tables, tides during this time ranged between Elevation + 30 feet and 0 feet, but during a two day favorable stretch, the high tide remained below Elevation +23 feet while the low tide was about +3 feet.

Drilling went reasonably smoothly except that some drilling time was lost during moves and casing setups when strong current, and adverse tide conditions forced delays until these conditions improved. In most cases the platform base was jacked to about Elevation +30 feet for drilling with minor height adjustments to accommodate tide conditions that were estimated from local tide tables. Drilling was carried out on a 24-hour basis.

Once the platform was jacked up on its four legs to the elevation needed for stability, each boring was initiated by setting 12-inch and 6-inch casings through the water and seating it into the soil below. The boring was then advanced using mud rotary methods, and a third 4.5 inch diameter HWT drill rod/casing with drill bits and three different wireline and/or drive samplers. The third drill rod/casing was carried down with the hole as drilling advanced to control caving of the borehole walls.

Location control overwater was established by our representatives and the barge/platform supervisor using an onboard differential GPS survey methods tied into markers at the Port of Anchorage. The accuracy for most drill locations is estimated to be less than 10 feet. Vertical depths or mudline elevations were checked by direct measurements from the deck using a weight on a line related to published tide tables and a level survey from the platform deck to range poles set on each shoreline. The shoreline elevations were then tied together with a portable Survey GPS system with an accuracy of less than six inches. Vertical elevations are judged to be accurate to about one foot or less. The elevation datum for these measurements and the project datum was taken as MLLW. The overwater boring elevations and horizontal coordinates are presented on each boring log in Appendix B, Figures B-1 through B-9.

B-2 Overwater Sampling

As a boring was advanced, sampling was generally accomplished at 5-, 7.5-, and 10-foot depth intervals using both disturbed and undisturbed sampling procedures. The three samplers generally used for the offshore work were as follows:

Disturbed Samplers

1. Two-inch OD split spoon sampler using SPT procedures,
2. Push core wireline 5 foot core barrel with a rugged 3 inch inner tube designed to recover large gravelly samples,

Undisturbed Samplers

3. Three inch by 30 inches long thin wall tubes advanced with wireline spring loaded core barrel (similar design to Pitcher Barrel)

With the SPT method, a 2-inch OD split-spoon sampler is advanced 18 inches into the undisturbed soil at the bottom of the advancing boring, with blows of a 140-pound surface auto-hammer falling 30 inches on the drill rods. The number of blows required to produce the final 12 inches of an 18-inch penetration of the hammer, defined as the Standard Penetration Resistance, was recorded for each sample by our representative. When hard or very dense soils, or coarse gravels were encountered, the sampler often could not be driven the full 18 inches, or in some cases even 12 inches. The blow counts, or N values, which are noted on the logs, are uncorrected values and provide a means of evaluating the consistency (stiffness) for cohesive soils and compactness for sands. When a full 18 inches penetration was not achieved, blows and the penetration achieved are recorded on the logs. To aid in evaluating the above uncorrected N values, particularly for sandy soils, energy transfer studies were conducted to measure the energy losses between the hammer and the top of the rods and between the hammer to the bottom of the rods (or near sampler) for various lengths. The results of these measurements are presented on Table 1 and in Appendix E.

The push core wireline sampler was used sparingly or only when recovery of material by other methods was poor. It has a catcher at the bottom and allows recovery of up to a three inch diameter by four foot long disturbed sample. Because this sampler recovers disturbed material and provides no driving resistance or estimate of soil density or consistency, it was used as a final choice when adequate recovery was not possible using the other two methods.

The final sampler is a modified Pitcher Barrel sampler, well suited for taking undisturbed 3-inch thin wall tube samples of stiff to hard clay/silt soils or soft rock. With this sampler, the wireline barrel advances the thin wall tube by a spring loaded piston inside a coring barrel. As the coil spring compresses the rotating outer barrel cuts away the soil around the outside of the tube, reducing side friction and allowing the spring to direct its load to forcing the tube into the undisturbed soil at the bottom of the advancing boring. The barrel’s carbide cutting teeth can usually cut to within an inch or less of the lower tube end in hard soils permitting good recovery of a near full tube of soil. This sampler was chosen over a conventional on-shore piston undisturbed sampler because it operates using wireline equipment and is faster and less costly to recover and for these soils results in a longer and likely better quality sample of undisturbed material for testing in the laboratory.

B-3 Onshore Drilling and Sampling

Field exploratory work for the onshore work included advancing deep borings at the two abutment sites and seven shallower somewhat evenly spaced borings near the high water line between the east abutment and the north side of the Port of Anchorage. The deep borings, A-7 and A-8, were extended to depths of 196 and 186-feet, respectively, while the shoreline borings, A-11 through A-17, were each drilled to depths of between 25 and 30 feet. The approximate locations of these borings are shown on Figure 1. These borings were generally on the mudflats near the high tide line and in most cases within about 40 feet or less of the toe of the bluffs. The two deeper borings were drilled very close to the bluff to prevent water at high tide from rising above the tracks on the rig. Brief delays in the work schedule had to be provided to avoid these high tide time periods as water often rose up to the track rig as shown in Appendix A, Photograph A-10. Detailed logs of the test holes are presented in Appendix B as Figures B-6, B-7, and B-10 through B-16.

Drilling services for the onshore borings were provided by Discovery Drilling of Anchorage, Alaska, using a track-mounted CME 75 drill rig. The borings were advanced with 8-inch outside diameter, 3-1/4 inch inside diameter hollow-stem augers. Pumps were also supplied to add water or drilling mud to the borings to control heave when necessary. An experienced geologist from our firm was present continuously during drilling to locate the borings, observe drill action, collect samples, log subsurface conditions, and monitor any groundwater encountered.

As the borings were advanced, both disturbed and undisturbed samples were recovered at 5 or 10-foot depth intervals. Disturbed samples were taken with a split-spoon sampler using SPT procedures, as described above. The uncorrected N values are shown graphically on the boring logs adjacent to the sample depth, and give a measure of the relative compactness or consistency (stiffness) of the cohesionless and cohesive soils at the site, respectively.

Undisturbed samples were taken by fixing a 3-inch diameter by 30-inch thin wall tube on the end of the drill rods and advancing it with the hydraulic ram into the undisturbed soil at the bottom of the boring as drilling progressed. The recovered tubes were sealed at the ends with plastic caps and returned to our Anchorage laboratory for testing, as necessary.

At the end of drilling, all on shore borings were backfilled with native cuttings. The locations of the borings, shown on Figure 1 and on the boring logs, were determined by the same Survey GPS system used to tie in the offshore borings.

B-4 Prior Borings

Three prior overwater borings, designated HLA 3, HLA 4, and HLA 5, were drilled in the crossing vicinity by Harding Lawson and Associates in 1984 as part of the early Knik Arm Crossing studies (HLA, 1984). The approximate locations of these borings are shown on Figure 1. This drilling work was performed from a floating barge held in place with heavy anchors. Logs of these borings are presented in Figures B-18 through B-20.

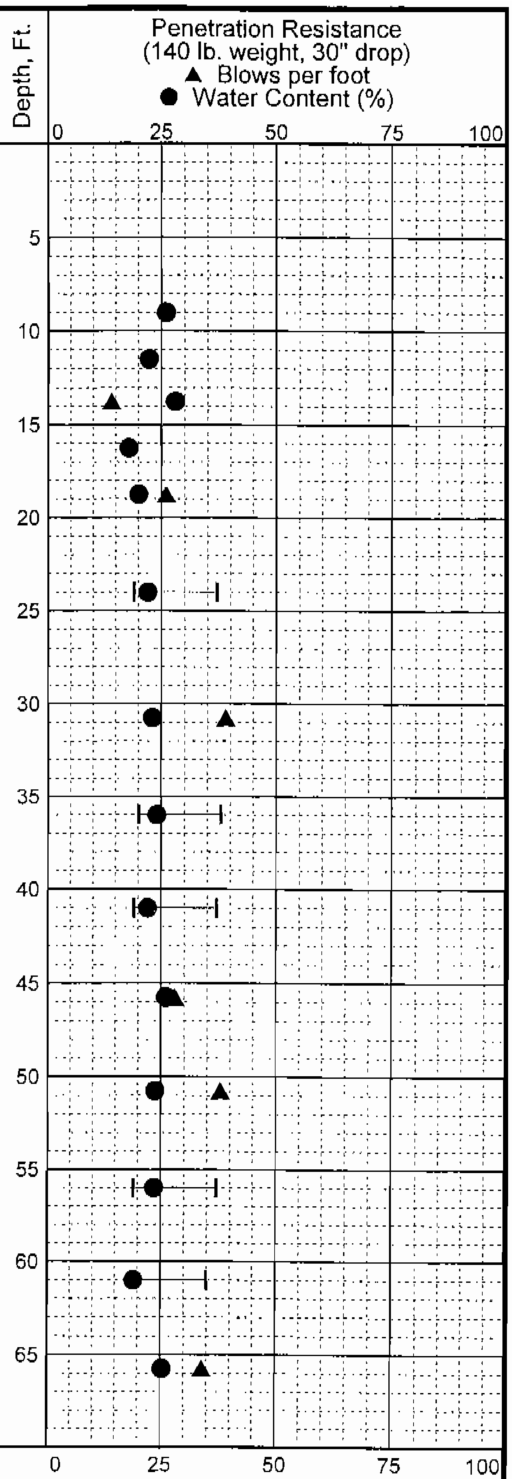
Several borings were drilled on the west abutment bluff in September 2003, for the Mat-Su Borough in conjunction with a sand/gravel borrow source assessment for this area. The location of the closest boring, B-3, is shown on Figure 1 and a log of this boring is presented on Figure 4 and Appendix B, Figure B-21. The drilling work used the same on shore equipment and procedures described in the previous section.

Several borings were drilled on the east shoreline north of the Port of Anchorage between October 24 and November 15, 1996. These borings were advanced as part of north tideland fill and cargo expansion studies for the Port of Anchorage (Shannon & Wilson, 1997). The location of Boring B-13 is shown on Figure 1 and a log of this boring is presented on Figure 5 and Appendix B, Figure B-17. The drilling work used the same equipment type and procedures described in the previous sections.

MATERIAL DESCRIPTION

Approximate Water Depth Above Mudline: 54 Ft.
Approximate Elevation: -13.8 Ft. (MLLW Datum)

MATERIAL DESCRIPTION	Depth, Ft.	Samples	Ground Water
Thin layer of fine gravel on surface (3-6")		S0	▽ 8/16/03
Medium dense, gray, silty SAND, trace of gravel, with occasional lenses of very stiff silty clay; moist to wet			
S1: 3% Gravel, 77% Sand, 20% Fines		S1	
S2: PP = 3		S2	
S3: PP = 2.3	12.0	S3	
Very stiff to hard, gray, silty CLAY with scattered and lenses of gravel; moist			
S4: PP = >4.5		S4	
S5: PP = 3.7	20.0	S5	
S6: PP = 3.3 qu = 2tsf		S6	
Very stiff gray, silty CLAY; moist			
		S7 *	
S8: PP = 3.5		S8	
S9: PP = 3.5 qu = 2.6tsf		S9	
S10: PP = 3.2 qu = 4.2tsf		S10	
S11: PP = 3		S11	
S12: PP = 3.5		S12	
S13: PP = 3.5 qu = 3.9tsf		S13	
S14: PP = >4.5 qu = 1.4tsf	61.0	S14	
S15: PP = 4		S15	
Stiff to very stiff, gray, silty CLAY, with local stiff and a few hard zones and lenses of gravel; moist			



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LEGEND

- * Sample Not Recovered
- ■ Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

▽ Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |——●——| Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. PP (Pocket Penetrometer) tests estimate Unconfined Compressive Strength of Cohesive Soils. qu are direct unconfined compressive strength test results. All measurements in tons per square foot.

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-1
Location: N 61°16.314' W 149°52.57

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-1
Sheet 1 of 5

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 54 Ft. Approximate Elevation: -13.8 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
S16: PP = 3			S16		
Stiff to very stiff, gray, silty CLAY, with local stiff and a few hard zones and lenses of gravel; moist			S17		75
S18: PP = 2.75			S18		80
S19: PP = 1.7 uu = 1.8tsf			S19		85
S20: PP = 2.5 uu = 4tsf			S20		90
S21: PP = 2.5			S21		95
S22: PP = 2.5 qu = 1.5tsf			S22		100
	104.0				
S23: PP = 2.25			S23		105
S24: PP = 2			S24		110
Very stiff, gray, silty CLAY, with a few scattered gravel; moist					
S25: PP = 2.25 qu = 2.8tsf			S25		115
S26: PP = 3.7 uu = 3.9tsf			S26		120
S27a: PP = 2.5			S27a S27b		125
S28: PP = 2.3 qu = 2.8tsf			S28		130
S29: PP = 2.5 qu = 2.4tsf uu = 2.6tsf			S29		135

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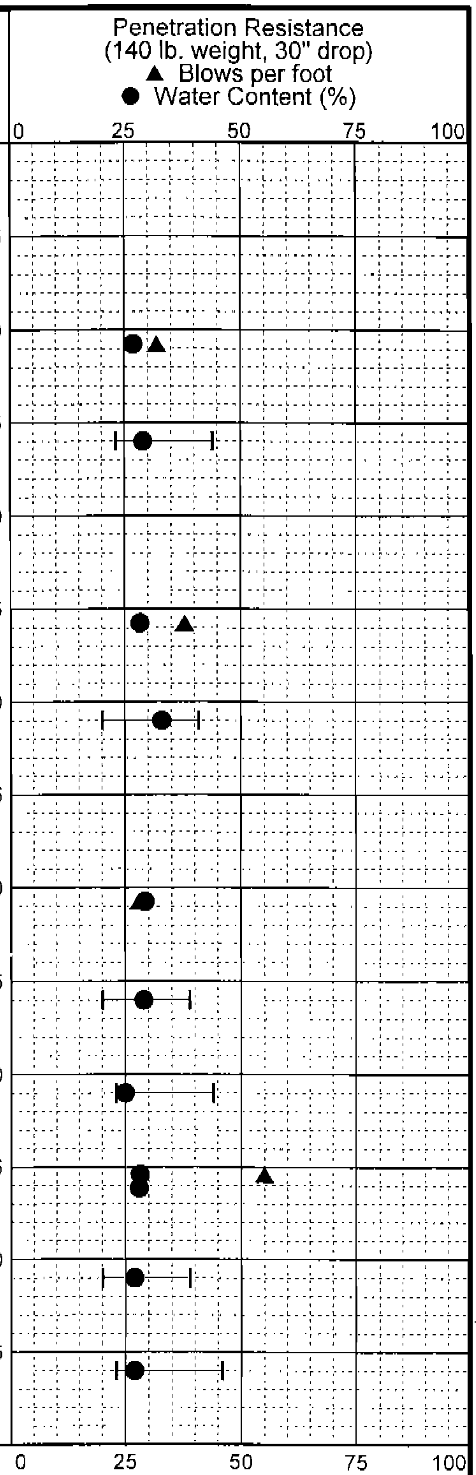

LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
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4. PP (Pocket Penetrometer) tests estimate Unconfined Compressive Strength of Cohesive Soils. qu are direct unconfined compressive strength test results. All measurements in tons per square foot.

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-1
Location: N 61°16.314' W 149°52.57

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-1
Sheet 2 of 5

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 54 Ft. Approximate Elevation: -13.8 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
S30: PP = 2			S30		
	145.0				145
S31: PP = 2.5 Very stiff to hard, gray, silty CLAY, with scattered gravel; moist			S31		
S32: PP = 2.75 qu = 4.4tsf			S32		150
					155
S33: PP = 2.5	159.0		S33		
					160
					165
S34: PP = 2.75 Very stiff, gray, silty CLAY, with a few stiff layers and several 1 foot ± lenses of gravel; moist			S34		
					170
S35: PP = 1.75 qu = 1.6tsf			S35		
					175
					180
S36: PP = 2.5			S36		
					185
S37: PP = 2.75 qu = 2.4tsf			S37		
					190
					195
S38: PP = 2.75			S38		
					200
S39: PP = 2.25			S39		
					205

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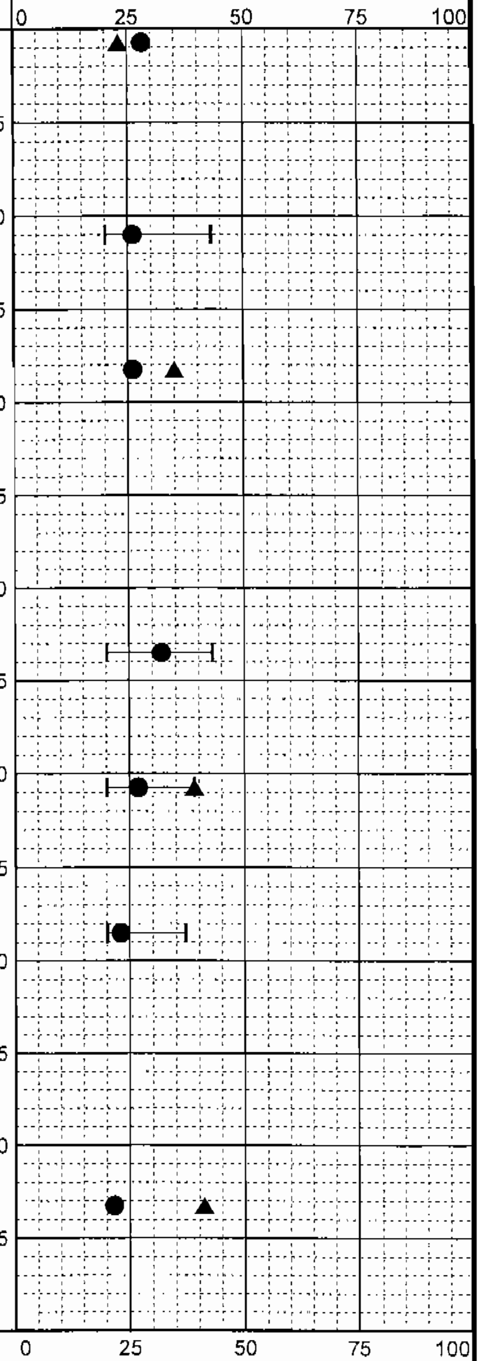
Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |—●—| Liquid Limit
Natural Water Content

NOTES

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2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-1
Location: N 61°16.314' W 149°52.57

February 2004 — 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-1
Sheet 3 of 5

OVERWATER_LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 54 Ft. Approximate Elevation: -13.8 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
S40: PP = 2 uu = 2.2tsf			S40		
Very stiff, gray, silty CLAY, with a few stiff layers and several 1 foot ± lenses of gravel; moist			S41		215 220
S42: PP = 2.5	226.0		S42		225
Stiff, gray, silty CLAY, with a few medium stiff layers; moist					230 235
S43: PP = 1.2 qu = 0.6tsf	240.0		S43		240
S44: PP = 2.25			S44		245
Very stiff to hard, gray, silty CLAY; moist					250
S45: PP = 3.5			S45		255 260
S46: PP = 3.5 qu = 4.4tsf	270.0		S46		265 270
Stiff to very stiff, gray, silty CLAY, with lenses of gravel and thin hard layers; moist					275
S47: PP = 1.7 qu = 1.2tsf			S47		

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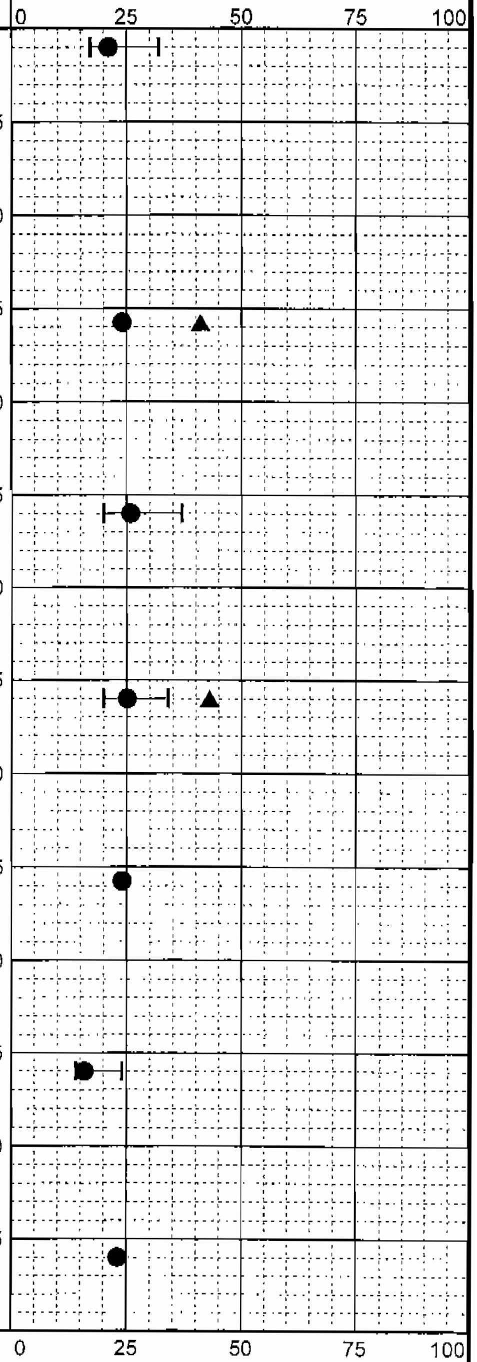
Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- ■ Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |—●—| Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-1
Location: N 61°16.314' W 149°52.57

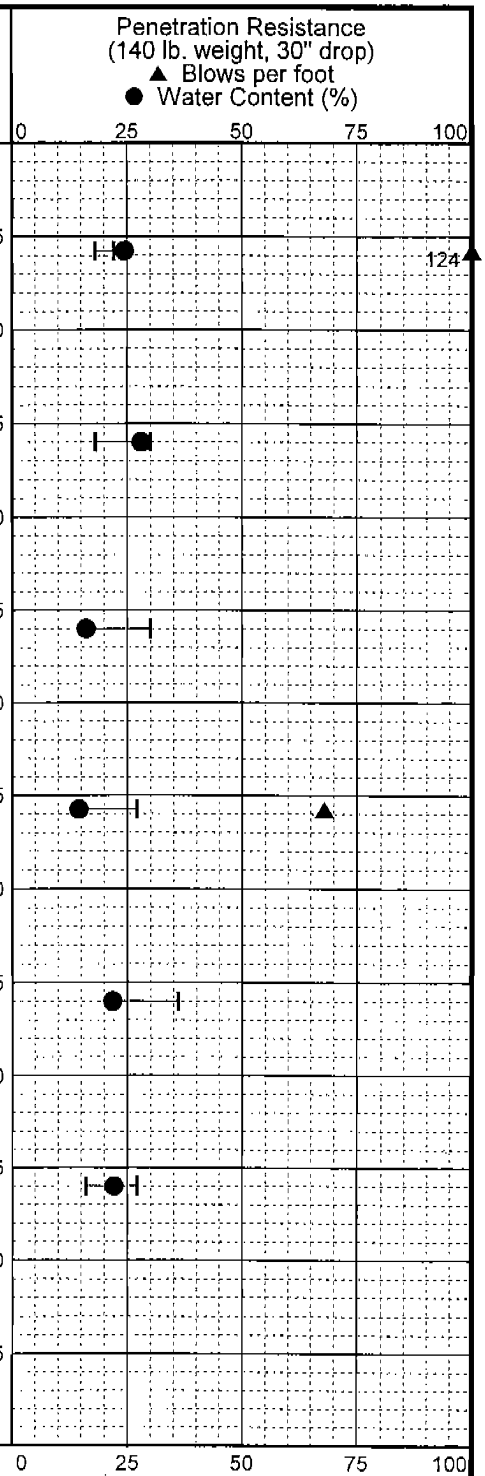
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-1
Sheet 4 of 5

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Approximate Water Depth Above Mudline: 54 Ft. Approximate Elevation: -13.8 Ft. (MLLW Datum)					
S48: PP = 2.5 Stiff to very stiff, gray, silty CLAY, with lenses of gravel and thin hard layers; moist			S48		285
					290
S49: PP = >4.5			S49		295
					300
S50: PP = 3.75 qu = 3.5tsf uu = 3.3tsf			S50		305
					310
Hard, gray, gravelly, silty CLAY; moist	312.0		S51		315
Hard, gray, silty CLAY; moist	319.0				320
S52: PP = >4.5 qu = 1.0tsf			S52		325
					330
S53: PP = 3.3 qu = 4.7tsf			S53		335
Bottom of Boring Boring Completed 8/16/03	337.0				340
					345



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-1
Location: N 61°16.314' W 149°52.57

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-1
Sheet 5 of 5

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 77 Ft. Approximate Elevation: -60.9 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Loose, gray, SAND; wet			S1, S2, S3	▽ 8/20/03	5, 10, 15
S4a: 0% Gravel, 89% Sand, 11% Fines S4b: 10.8% Fines Medium dense, gray, slightly silty, fine SAND; wet S7: 0% Gravel, 92% Sand, 8% Fines	20.0		S4a, S4b, S5, S6*, S7, S8, S9*		20, 25, 30, 35, 40, 45, 50
Medium dense to very dense, gray, clean to silty fine SAND, with layers of gray sand; wet S12: 0% Gravel, 84% Sand, 16% Fines S14: 0% Gravel, 84% Sand, 16% Fines	55.0		S10, S11, S12, S13*, S14		55, 60, 65, 70, 75

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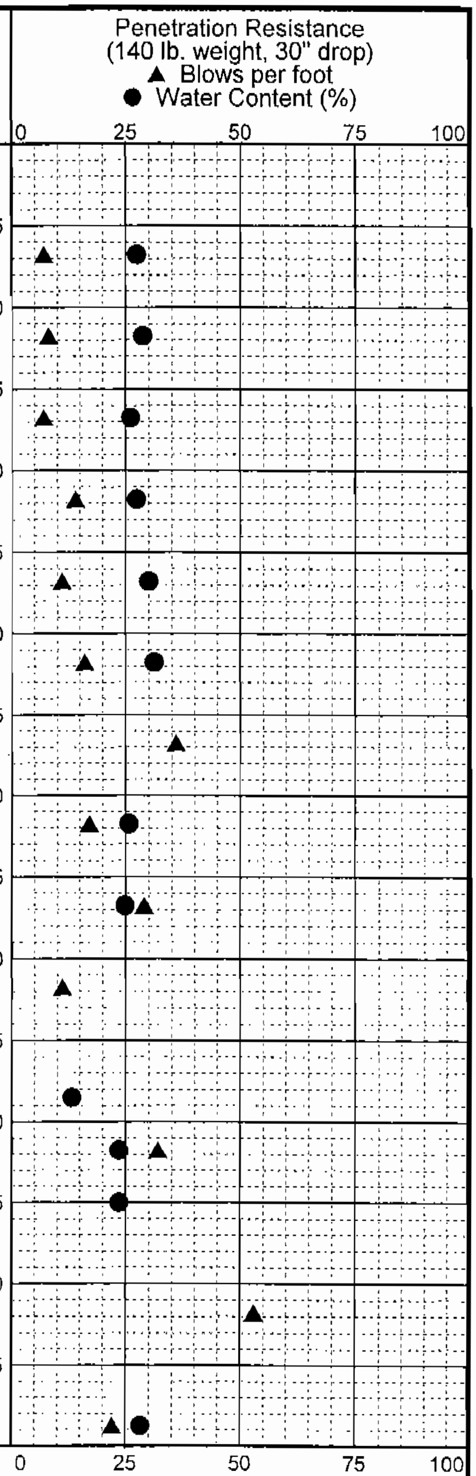

LEGEND

- * Sample Not Recovered
- ■ Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

▽ Ground Water Level At Time Of Drilling Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)

Plastic Limit |—●—| Liquid Limit

Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-2
Location: N 61°16.661' W 149°53.273'

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-2
Sheet 1 of 3

OVERWATER_LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 77 Ft. Approximate Elevation: -60.9 Ft. (MLLW Datum)	Depth, Ft.	Samples
Medium dense to very dense, gray, clean to silty fine SAND, with layers of gray sand; wet S15: 0.3% Fines		S15
		S16 *
	100.0	
Dense, gray, slightly silty to clean fine SAND; wet		S17
S18: 1% Gravel, 92% Sand, 7% Fines		S18
S19: 0.8% Fines		S19
	130.0	
S20: 0% Gravel, 58% Sand, 42% Fines Very dense, gray, silty fine SAND, with trace of gravel; wet		S20
		S21
	148.0	
Dense to very dense, gray, GRAVEL; wet		S22
	156.0	

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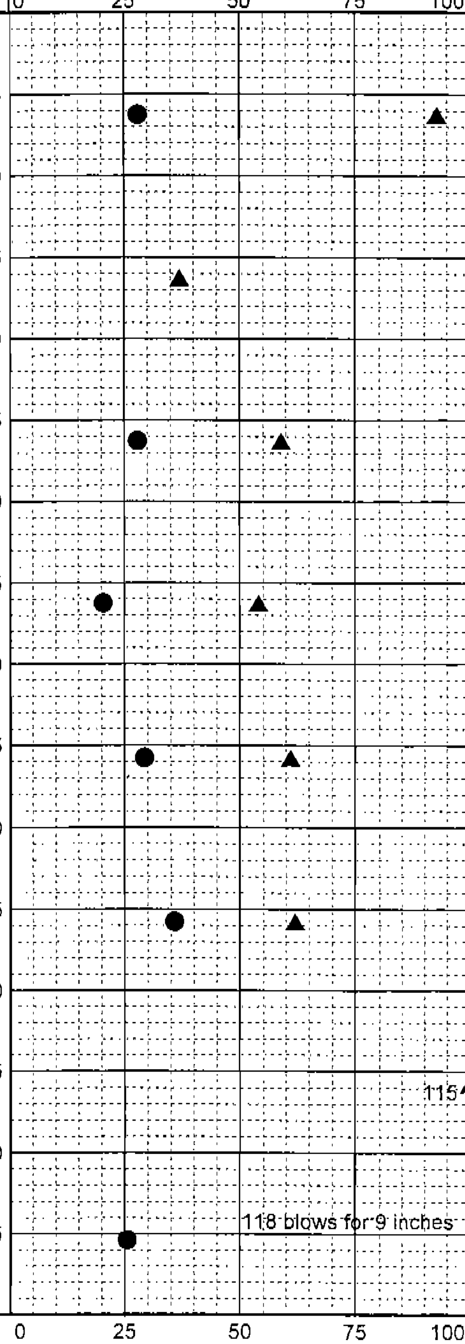
Ground Water | Depth, Ft.

Penetration Resistance (140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-2
Location: N 61°16.661' W 149°53.273'

February 2004 | 32-1-01536

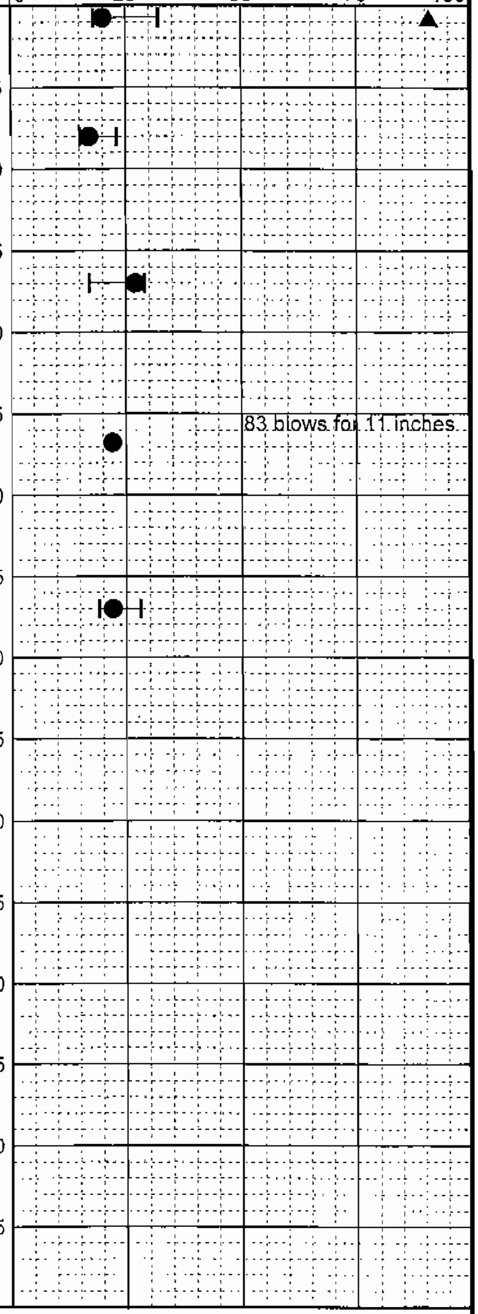
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-2
Sheet 2 of 3

OVERWATER_LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 77 Ft. Approximate Elevation: -60.9 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
S23: PP = 3.5 Very stiff to hard, gray, silty CLAY, with trace of gravel; moist S25: PP = 3.5 S26: PP = 4 qu = 2.5tsf			S23, S24*, S25, S26		165, 170, 175, 180
Hard, gray, slightly sandy, silty CLAY; moist S27: 94.4% Fines	182.0		S27		185, 190
Very stiff, gray, silty CLAY; moist S28: PP = 2.5	191.0		S28		195
Bottom of Boring Boring Completed 8/20/03	198.0				200, 205, 210, 215, 220, 225, 230, 235

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-2
Location: N 61°16.661' W 149°53.273'

February 2004 | 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-2
Sheet 3 of 3

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 34 Ft. Approximate Elevation: -2.9 Ft. (MLLW Datum)	Depth, Ft.	Samples	Ground Water
Boulder			8/18/03 ▽
Very dense, gray, silty, gravelly SAND; moist and non plastic S3: 33% Gravel, 36% Sand, 31% Fines	5.5	S1, S2, S3, S4	
S6: 36% Gravel, 40% Sand, 24% Fines Dense to very dense, gray, interbedded, silty, gravelly SAND and slightly silty GRAVEL; moist S8: 92% Gravel,	20.0	S5, S6, S7, S8, S9	
Hard, gray, slightly sandy, gravelly, silty CLAY with few stiff clay lenses; dry, crumbly and low plasticity, locally non plastic S13: PP = >4.5	40.0	S10, S11, S12, S13, S14, S15	

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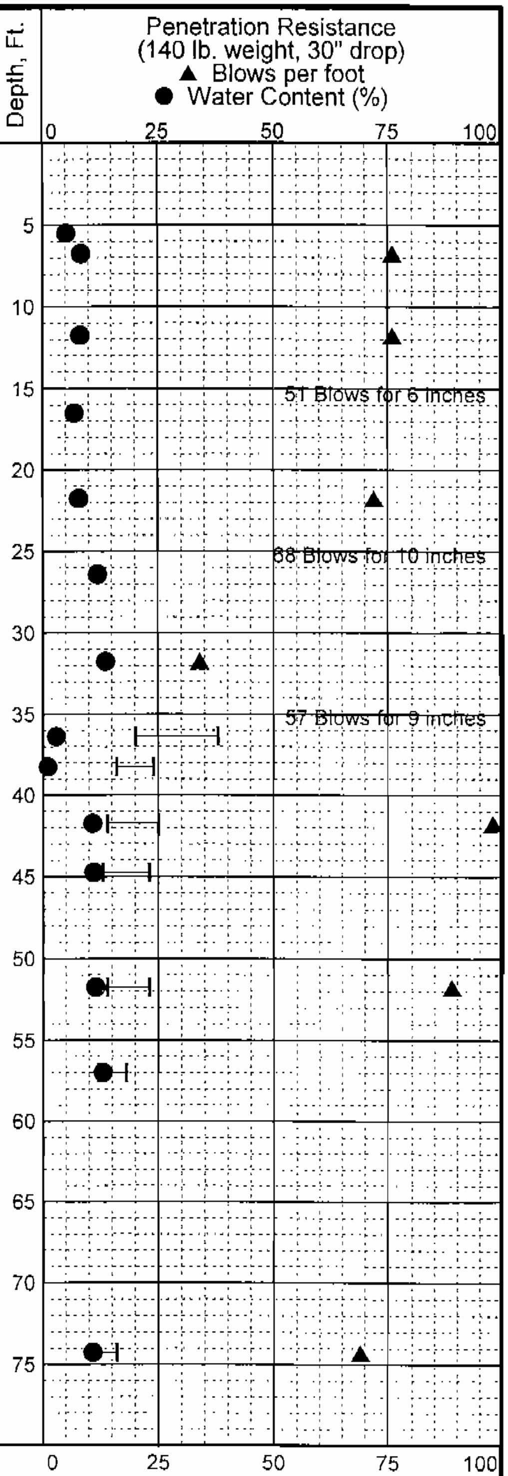

LEGEND

- * Sample Not Recovered
- ■ Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

▽ Ground Water Level At Time Of Drilling Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |—●—| Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

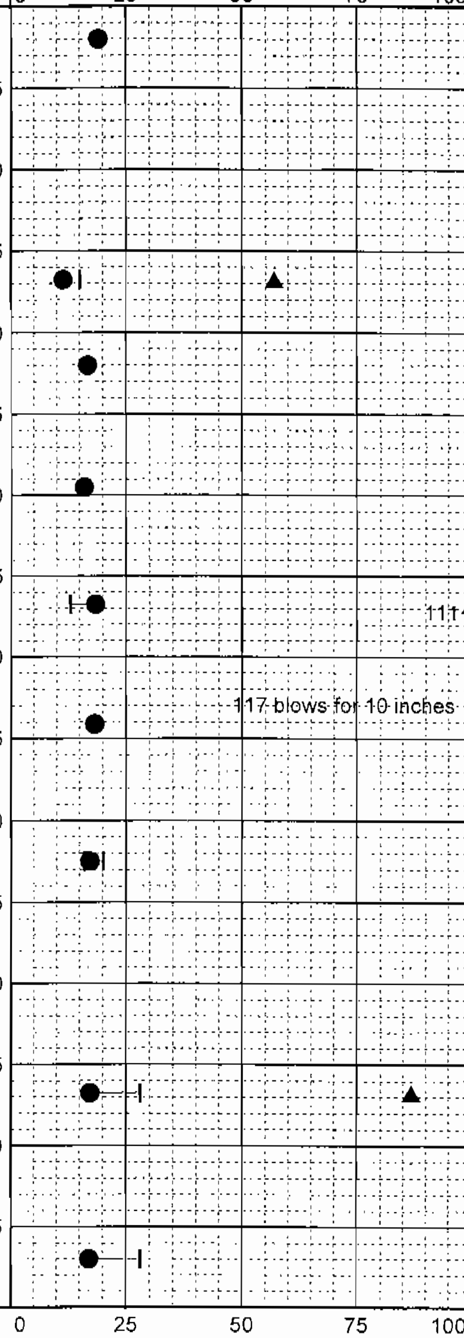
LOG OF BORING A-4
Location: N 61°17.063' W 149°54.890'

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-3
Sheet 1 of 3

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 34 Ft. Approximate Elevation: -2.9 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.	Penetration Resistance (140 lb. weight, 30" drop) ▲ Blows per foot ● Water Content (%)
S16: PP = 4.5 qu = .84tsf			S16		85	
Hard, gray, slightly sandy, gravelly, silty CLAY with few stiff clay lenses; dry, crumbly and low plasticity, locally non plastic			S17		90	
			S18		95	
S19: PP = >4.5			S19		100	
S20: PP = 1.25			S20		105 110	
S21: PP = >4.5			S21		115 120	111
Very dense, gray, silty SAND; wet and non plastic	123.0		S22		125	117 blows for 10 inches
S23: PP = >4.5 Hard, gray, sandy, silty CLAY, most with isolated lenses of gravel 6"- to 1'-thick, spaced randomly; moist with low plasticity S24: PP = >4.5	128.0		S23 S24		130 135 140	
S25: PP = >4.5 Hard, gray, slightly gravelly, silty CLAY; moist with low plasticity S26: PP = >4.5	144.0		S25 S26		145 150 155	



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LEGEND

- * Sample Not Recovered
- ■ Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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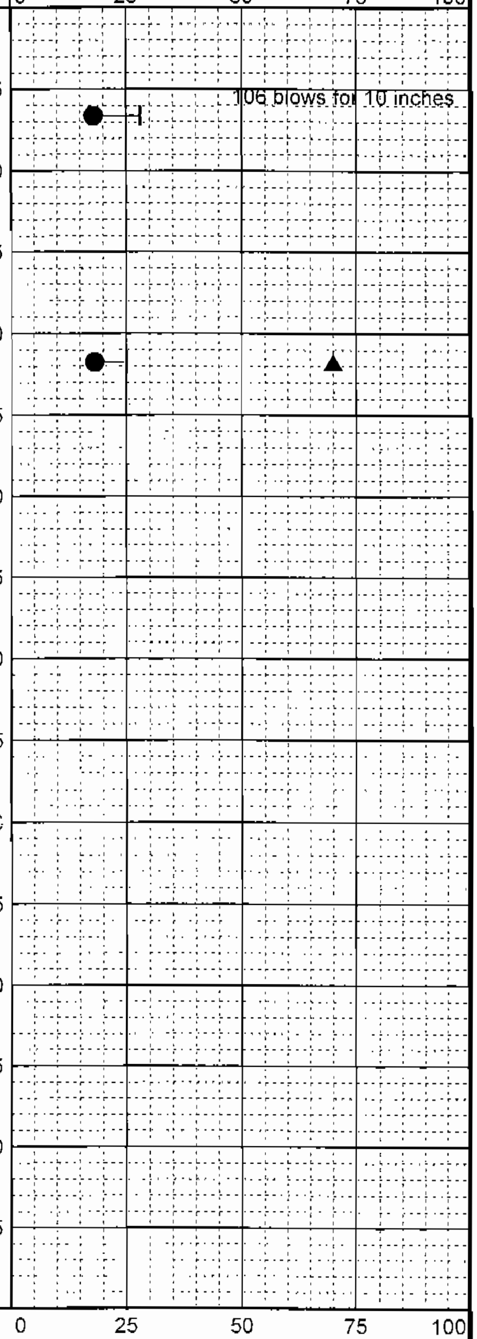
Knik Arm Bridge Anchorage, Alaska	
LOG OF BORING A-4 **Location: N 61°17.063' W 149°54.890'**	
February 2004	32-1-01536
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**Fig. B-3** **Sheet 2 of 3**

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 34 Ft. Approximate Elevation: -2.9 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.	Penetration Resistance (140 lb. weight, 30" drop) ▲ Blows per foot ● Water Content (%)
S27: PP = >4.5 Hard, gray, slightly gravelly, silty CLAY; moist with low plasticity S28: PP = >4.5	182.5		S27 S28		165–235	
Bottom of Boring Boring Completed 8/18/03						



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-4
Location: N 61°17.063' W 149°54.890'

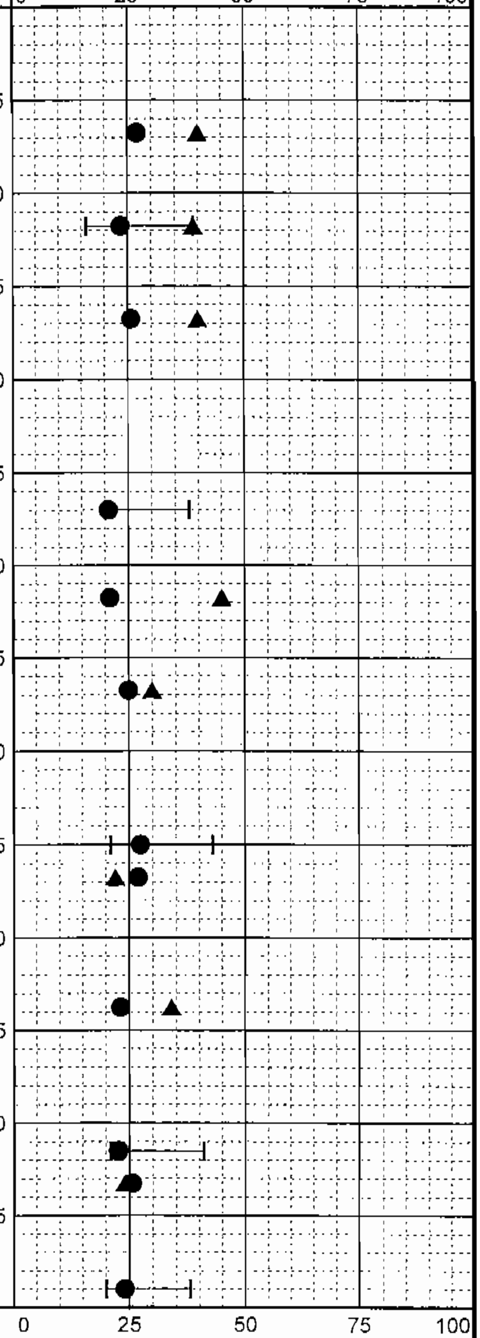
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-3
Sheet 3 of 3

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 54 Ft. Approximate Elevation: -30.1 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
				▽ 9/16/03	
					5
S1: PP = 2.75			S1		
Very stiff to hard, gray, slightly gravelly, slightly sandy CLAY, interbedded with layers of dense, clayey SAND; wet					10
S2: PP = 4.5			S2		
					15
S3: PP = 3.5			S3		
					20
			S4 *		
	25.0				25
S5: PP = 2.5 qu = .54tsf			S5		
					30
S6: PP = >4.5			S6		
					35
S7: PP = 3.5			S7		
Very stiff to hard, gray, sandy CLAY to medium dense to dense, clayey fine SAND; wet					40
S8: PP = 2.5 qu = 2.3tsf			S8		45
S9: PP = 2			S9		
					50
S10: PP = 3.5			S10		
S11: PP = 4			S11		
					55
					60
S12: PP = 2.5 qu = 2.4tsf			S12		
			S13		
					65
			S14		

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



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LEGEND

- * Sample Not Recovered
- ■ Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

▽ Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |——●——| Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-5
Location: N 61°16.674' W 149°52.395'

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-4
Sheet 1 of 4

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 54 Ft. Approximate Elevation: -30.1 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
S14a: PP = 3 qu = 1.4tsf			S14b *		
S15: PP = 3.5 S16: PP = 2.75			S15 S16		75 80
S17: PP = 1 qu = 1.4tsf uu = 2.3tsf S18: PP = 3			S17 S18		85
S19: PP = 2.5			S19 S20		90
Very stiff, gray, sandy CLAY to medium dense to dense, clayey fine SAND; wet, local stiff layers			S21		95 100
S22: PP = 2 qu = 2.0tsf			S22		105 110
S23: PP = 3.5			S23		115 120
	125.0				125
S24: 0% Gravel, 88% Sand, 12% Fines			S24		130
Very dense, gray, silty fine SAND, and interbedded with gray, very stiff to hard, silty CLAY; wet					135

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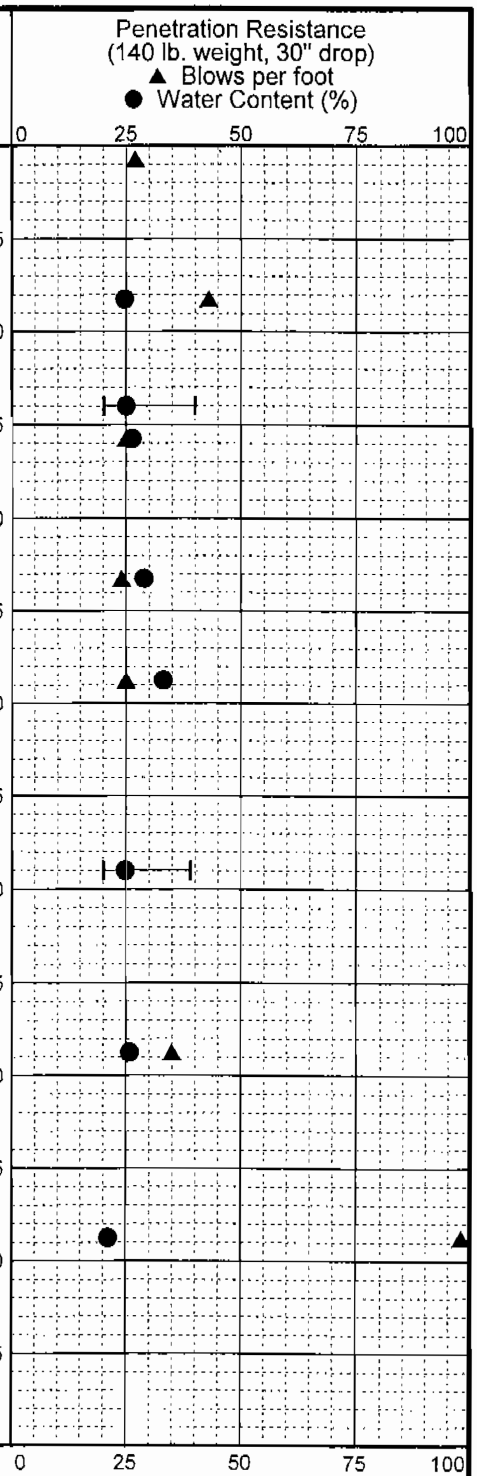

LEGEND

* * Sample Not Recovered
* Shelby Tube
* 2" O.D. Split Spoon Sample
* 4" Push Core Sample
* Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |—●—| Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-5
Location: N 61°16.674' W 149°52.395'

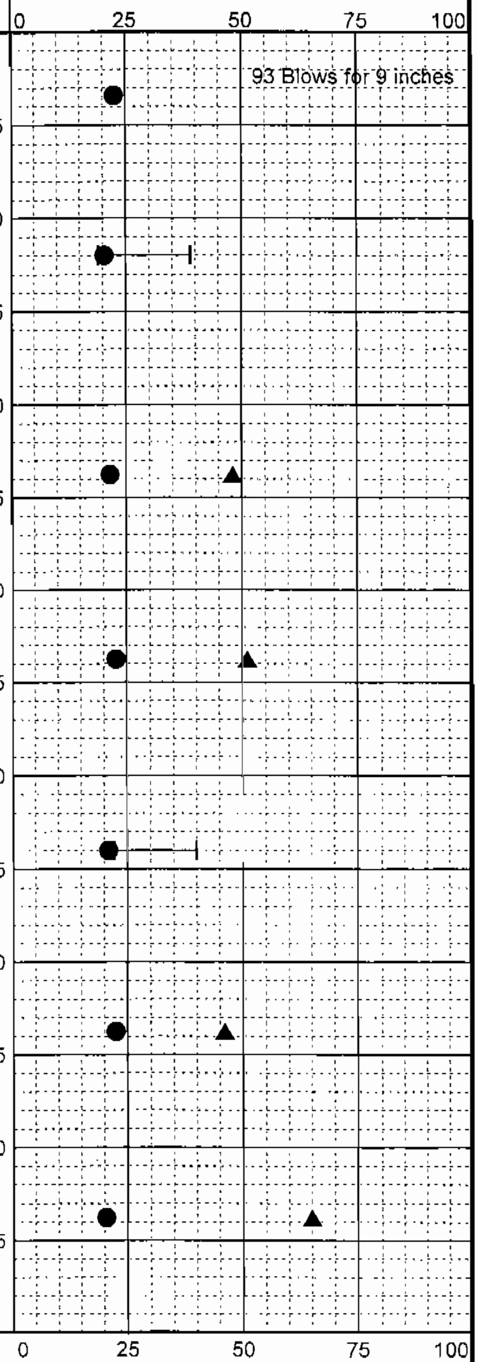
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-4
Sheet 2 of 4

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 54 Ft. Approximate Elevation: -30.1 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.	Penetration Resistance (140 lb. weight, 30" drop) ▲ Blows per foot ● Water Content (%)
S26: PP = 2.25			S26		145	
S27: PP = 4.25 qu = 3.2tsf uu = 6.5tsf			S27		150 155	
S28: PP = 3.5 0% Gravel, 4% Sand, 96% Fines			S28		160 165	
Very dense, gray, silty fine SAND, and interbedded with gray, very stiff to hard, silty CLAY; wet					170	
S29: PP = 3			S29		175	
S30: PP = <1 qu = 3.0tsf uu = 4.2tsf			S30		180 185	
S31: PP = 2.5 2% Gravel, 5% Sand, 93% Fines			S31		190 195	
S32: PP = 3			S32		200 205	



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LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-5
Location: N 61°16.674' W 149°52.395'

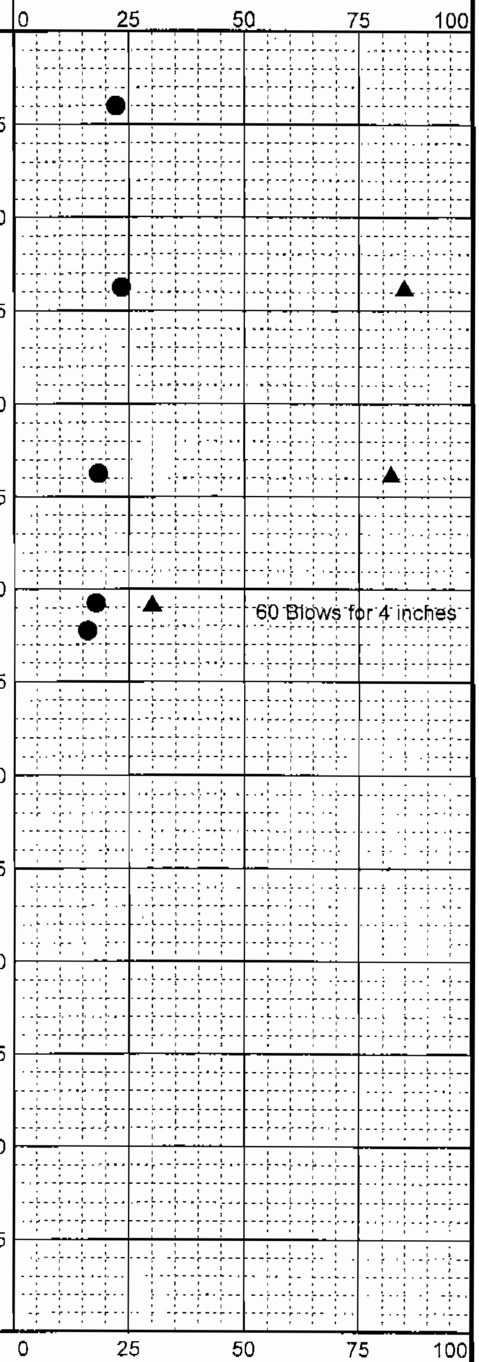
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-4
Sheet 3 of 4

MATERIAL DESCRIPTION	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Approximate Water Depth Above Mudline: 54 Ft. Approximate Elevation: -30.1 Ft. (MLLW Datum)					
S33: PP = 4.5 0% Gravel, 97% Sand, 3% Fines			S33		215
Very dense, gray, silty fine SAND, and interbedded with gray, very stiff to hard, silty CLAY; wet					220
S34: PP = 3.5			S34		225
					230
			S35		235
			S36		240
	242.5		S37		
Bottom of Boring Boring Completed 9/16/03					245
					250
					255
					260
					265
					270
					275

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-5
Location: N 61°16.674' W 149°52.395'

February 2004 32-1-01536

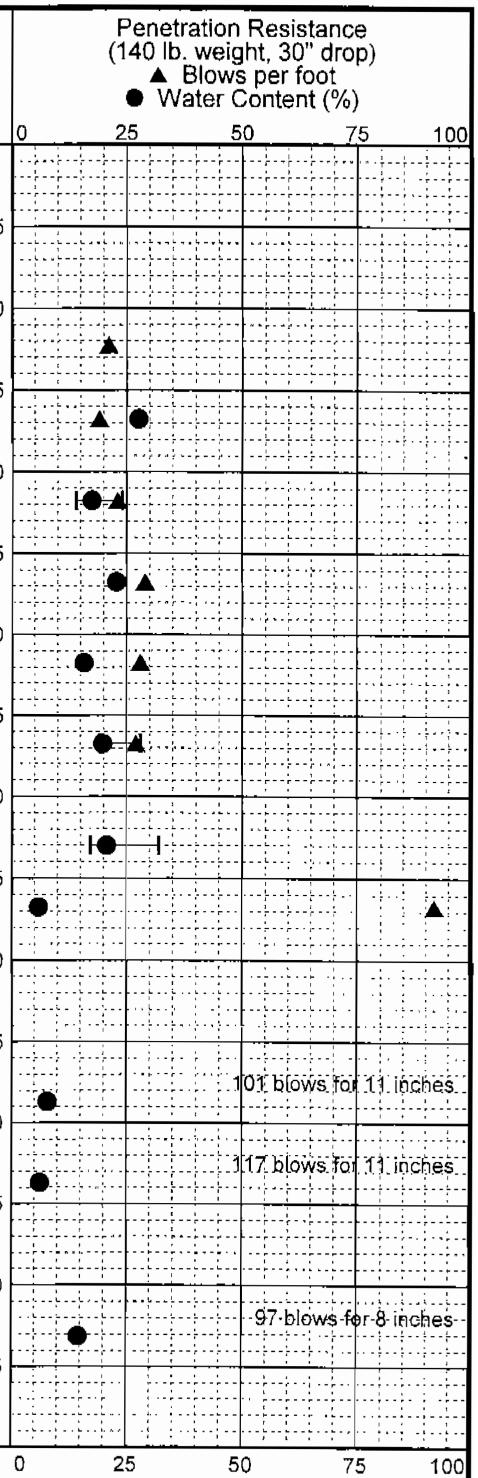
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-4
Sheet 4 of 4

OVERWATER_LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 30 Ft. Approximate Elevation: 4.47 Ft. (MLLW Datum)	Depth, Ft.	Samples
Gravelly mudline		
	3.0	
Very stiff, gray, sandy, silty CLAY; moist		S1 *
S2: PP = 3		S2
S3: PP = 4.25		S3
S4: PP = 2		S4
	27.3	
Medium dense, gray, silty GRAVEL; moist		
	29.0	
Very stiff, gray, gravelly, silty CLAY; moist with sand partings and seams		S5 * S6 S7
	46.0	
Dense, gray, gravelly CLAY to clayey GRAVEL; moist		S8
	50.0	
Very dense, gray-black, gravelly COBBLES; wet		S9
	56.0	
Very dense, gray, slightly silty, sandy GRAVEL; wet		S10 S11
	65.0	
Very dense, gray, gravelly, silty fine SAND; wet		
S12: 17% Gravel, 63% Sand, 20% Fines		S12

Ground Water: 9/14/03



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LEGEND

- * Sample Not Recovered
- ■ Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling. Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |—●—| Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-6
Location: N 61°16.159' W 149°52.139'

February 2004 — 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-5
Sheet 1 of 3

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 30 Ft. Approximate Elevation: 4.47 Ft. (MLLW Datum)	Depth, Ft.	Samples
Very stiff to hard, gray, sandy, silty CLAY; moist	80.0	S13
Dense to very dense, gray, slightly gravelly, silty SAND; moist S14: PP = >4.5 S15: 11% Gravel, 46% Sand, 43% Fines	87.0	S14, S15
S18: PP = 1.75 qu = 10tsf uu = 9tsf S19: PP = >4.5 S20: PP = >4.5 Very stiff to hard, gray, slightly sandy, silty CLAY, with sand seams and partings increasing thickness with depth; moist S21: PP = >4.5 S22: PP = >4.5 S23: PP = 2	93.0	S16*, S17, S18, S19, S20, S21, S22, S23
S24: PP = 2.25 0% Gravel, 10% Sand, 90% Fines Very stiff to hard, gray, slightly sandy, silty CLAY; wet S25: PP = 2.75 S27: PP = 3.5	135.0	S24, S25, S26, S27

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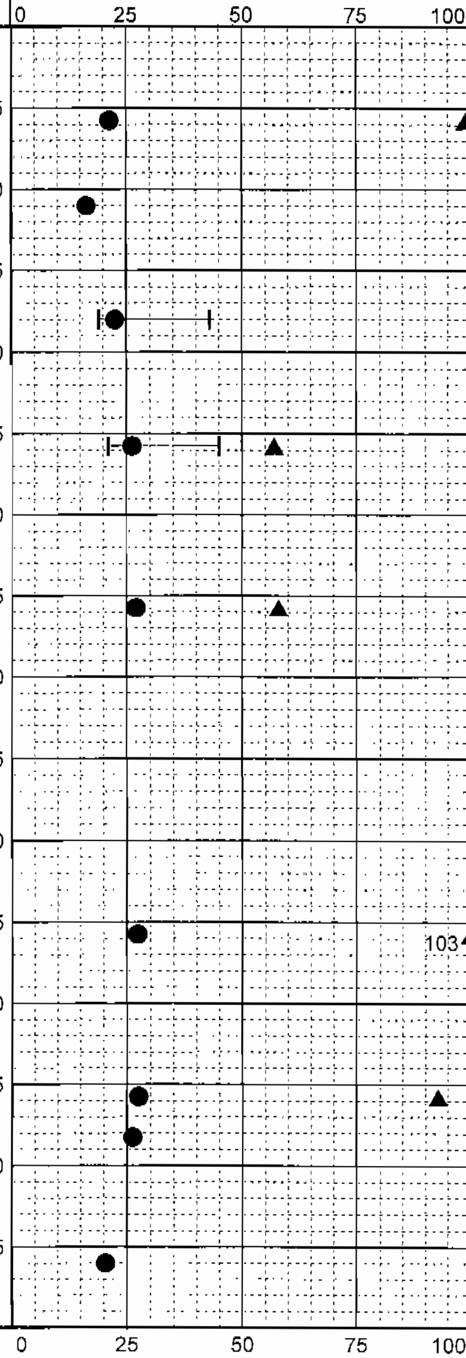
Penetration Resistance (140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-6
Location: N 61°16.159' W 149°52.139'

February 2004 32-1-01536

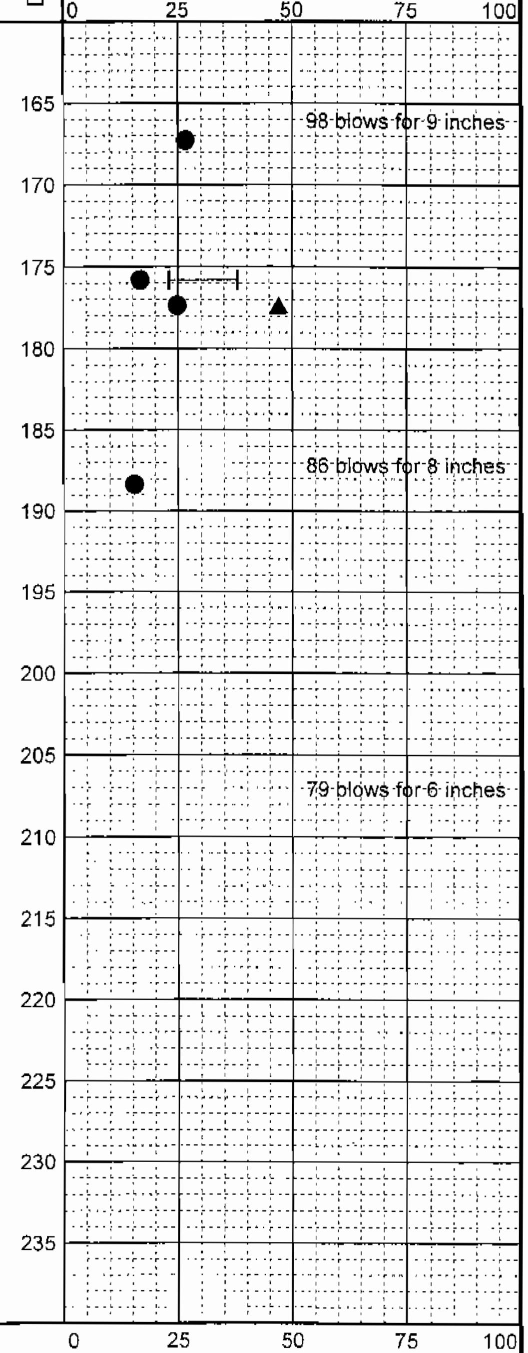
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-5
Sheet 2 of 3

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 30 Ft. Approximate Elevation: 4.47 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water
Very stiff to hard, gray, slightly sandy, silty CLAY; wet S28: PP = >4 S30: PP = 2.75 S31: PP = >4.5 qu = 3tsf uu = 3tsf			S28 S29 S30 S31 S32 S33	
Hard, gray, gravelly, silty CLAY; moist	181.0		S34	
Bottom of Boring Boring Completed 9/14/03	208.5		S35*	

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-6
Location: N 61°16.159' W 149°52.139'

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-5
Sheet 3 of 3

OVERWATER_LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: .5 Ft. Approximate Elevation: 30.5 Ft. (MLLW Datum)	Depth, Ft.	Samples	Ground Water
Hard, gray, gravelly, sandy, silty CLAY; moist S5: 17% Gravel, 25% Sand, 58% Fines		S1 *, S2, S3, S4, S5, S6, S7, S8, S9	
S10: 28.9% Fines Very dense, gray, silty SAND; wet	50.5	S10, S11	10/16/03 ⊻
Very dense, gray, silty, gravelly SAND; wet Gravel content decreases with depth	58.0	S12, S13, S14	
Very dense, gray, slighty gravelly, silty fine SAND; wet	73.0	S15	

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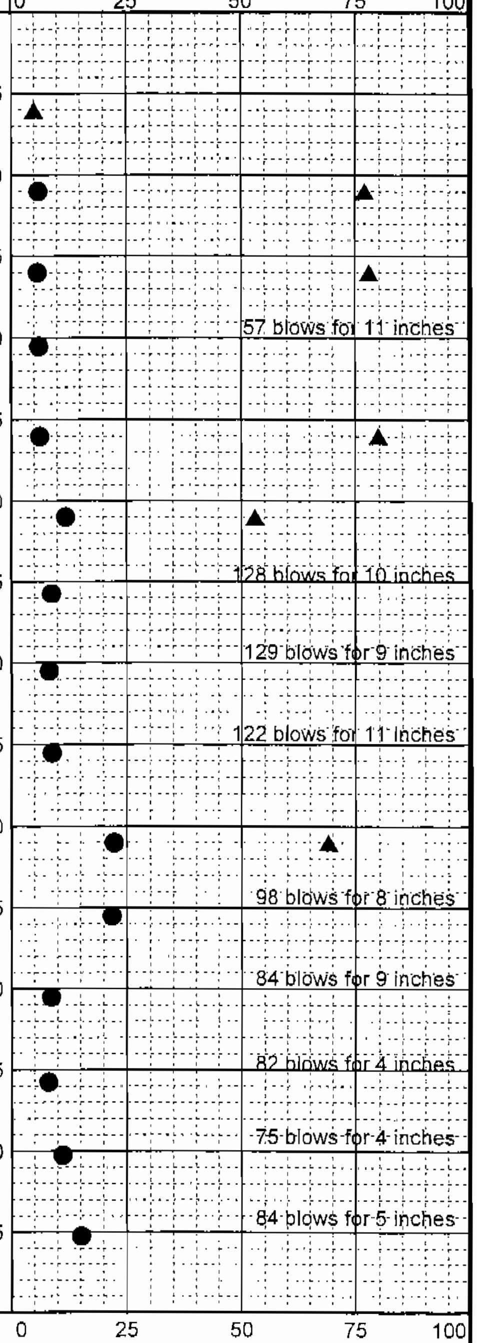
Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- ■ Shelby Tube
- ⌶ 2" O.D. Split Spoon Sample
- ⌶ 4" Push Core Sample
- **I** Rock Core Sample

⊻ Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |——●——| Liquid Limit
Natural Water Content

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-7
Location: N 61°15.966' W 149°51.883'

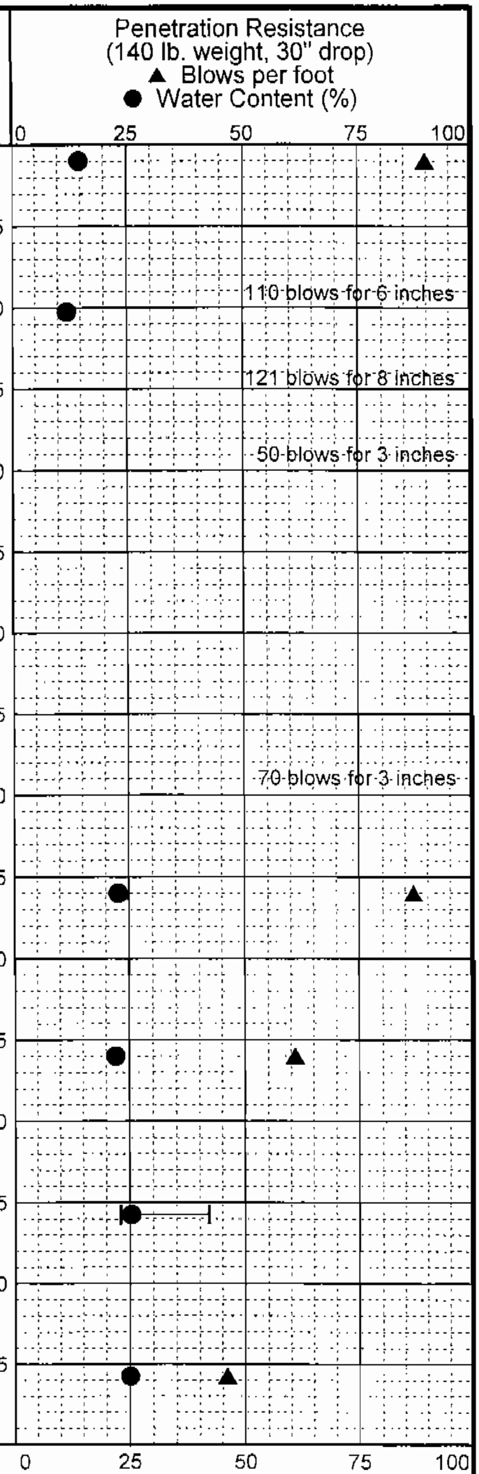
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-6
Sheet 1 of 3

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: .5 Ft. Approximate Elevation: 30.5 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Very dense, gray, slighty gravelly, silty fine SAND; wet			S16, S17		85, 90
Very dense, gray, silty, sandy, gravelly COBBLES; wet	94.0		S18*, S19*		95, 100, 105, 110, 115
S21: PP = >4.5 Hard, gray, silty, CLAY with sand seams increasing in thickness to interbedded clay and clayey sands; moist S22: PP = >4.5 S24: PP = >4.5	117.0		S20, S21, S22, S23, S24		120, 125, 130, 135, 140, 145, 150, 155



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LEGEND

- * Sample Not Recovered
- ■ Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |——●——| Liquid Limit
Natural Water Content

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-7
Location: N 61°15.966' W 149°51.883'

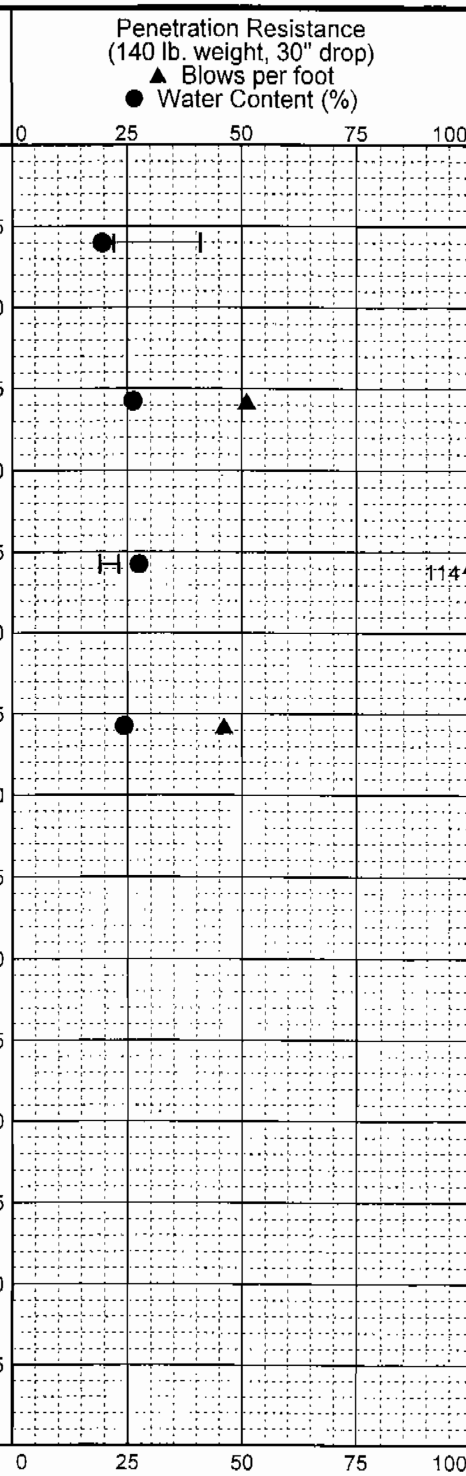
February 2004 | 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-6
Sheet 2 of 3

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: .5 Ft. Approximate Elevation: 30.5 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Hard, gray, silty, CLAY with sand seams increasing in thickness to interbedded clay and clayey sands; moist S25: PP = >4.5 qu = 9.5tsf			S25		165
					170
S26: 93.3% Fines			S26		175
					180
			S27		185
					190
S28: 88.7% Fines	193.0				
Dense, gray, silty fine SAND with sand seams and layers of gray clay; moist	196.5		S28		195
Bottom of Boring Boring Completed 10/16/03					200
					205
					210
					215
					220
					225
					230
					235



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-7
Location: N 61°15.966' W 149°51.883'

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

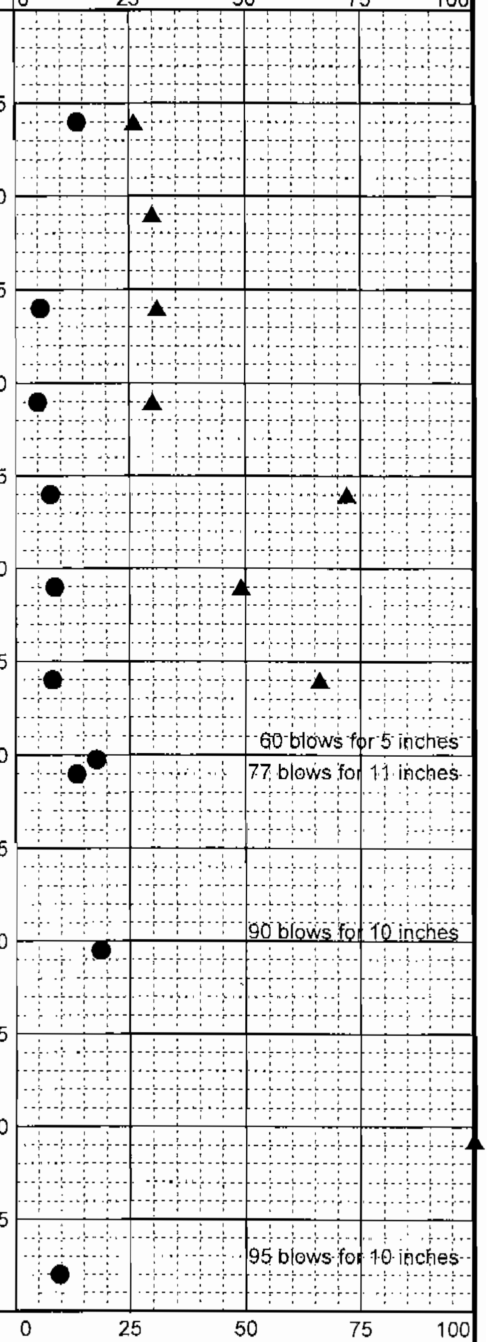
Fig. B-6
Sheet 3 of 3

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 2 Ft. Approximate Elevation: 30.3 Ft. (MLLW Datum)	Depth, Ft.	Samples
Medium dense, brown, silty SAND; wet		S1
Very stiff to hard, gray, sandy, gravelly, silty CLAY; wet	10.0	S2, S3, S4, S5, S6
S9: 8% Gravel, 82% Sand, 9% Fines Very dense, gray, slightly gravelly, slightly silty fine SAND; wet	35.5	S7, S8*, S9, S10
Hard, gray, gravelly, sandy, silty CLAY; moist	60.6	S11, S12*

Ground Water: Groundwater Not Encountered During Drilling on 9/10/03

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



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LEGEND

- * Sample Not Recovered
- ■ Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

⊻ Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-8
Location: N 61°17.142' W 149°55.095'

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-7
Sheet 1 of 3

MATERIAL DESCRIPTION	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.	Penetration Resistance (140 lb. weight, 30" drop) ▲ Blows per foot ● Water Content (%)

Approximate Water Depth Above Mudline: 2 Ft.
Approximate Elevation: 30.3 Ft. (MLLW Datum)

S14: 13% Gravel, 32% Sand, 55% Fines

Hard, gray, gravelly, sandy, silty CLAY; moist

S19: PP = 1
qu = 1.5tsf

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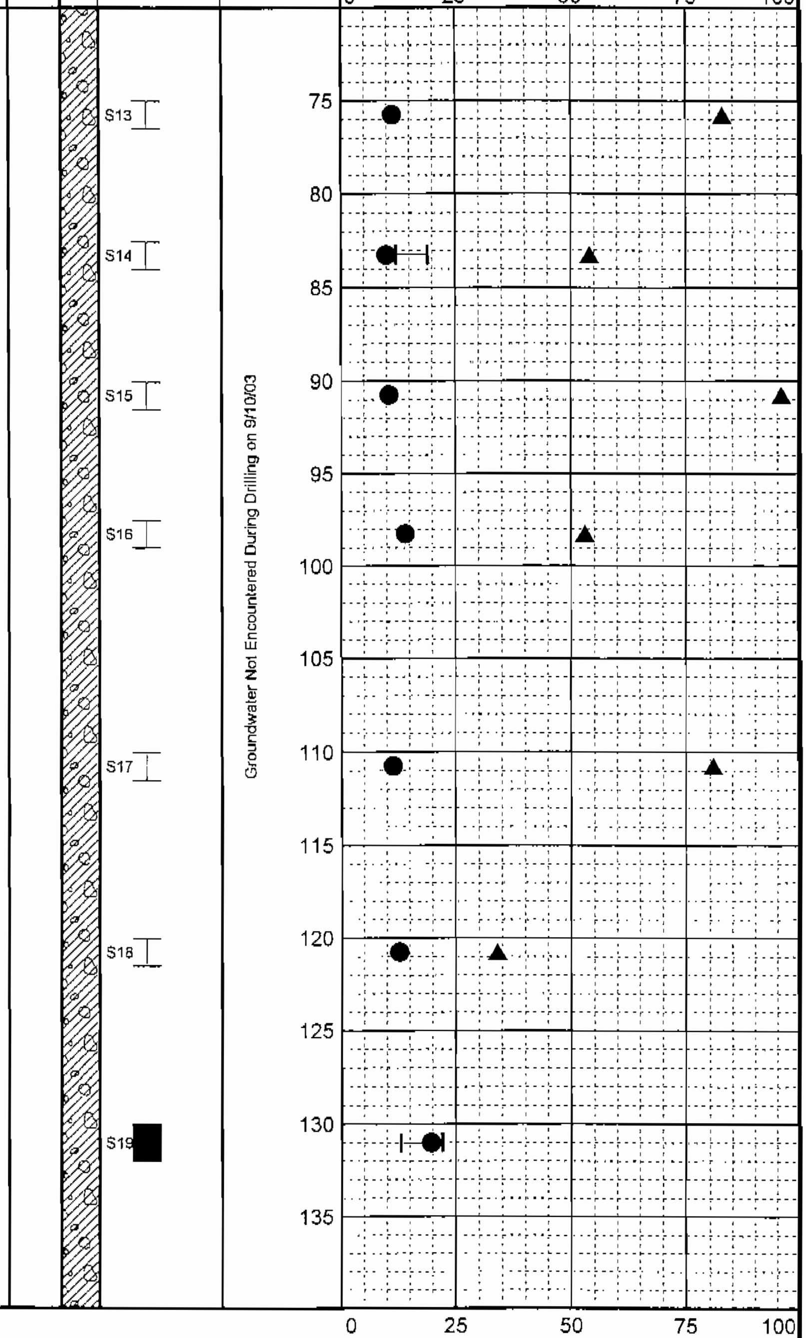

LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |—●—| Liquid Limit
Natural Water Content

NOTES

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2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. PP (Pocket Penetrometer) tests estimate Unconfined Compressive Strength of Cohesive Soils. qu are direct unconfined compressive strength test results. All measurements in tons per square foot.

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-8
Location: N 61°17.142' W 149°55.095'

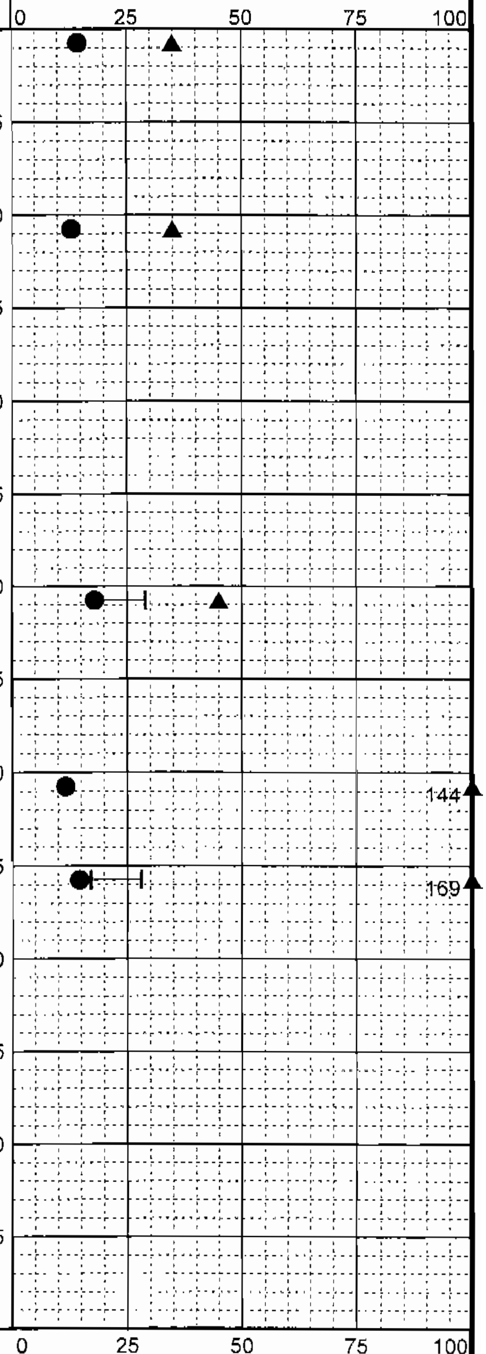
February 2004 | 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-7
Sheet 2 of 3

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 2 Ft. Approximate Elevation: 30.3 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.	Penetration Resistance (140 lb. weight, 30" drop) ▲ Blows per foot ● Water Content (%)
Hard, gray, gravelly, sandy, silty CLAY; moist			S20, S21, S22*	Groundwater Not Encountered During Drilling on 9/10/03	145, 150, 155, 160	
Hard, gray, slightly sandy, silty CLAY; wet	162.0		S23, S24, S25		165, 170, 175, 180, 185	
Bottom of Boring Boring Completed 9/10/03	186.5				190, 195, 200, 205	



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |——●——| Liquid Limit
Natural Water Content

NOTES

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2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
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4. PP (Pocket Penetrometer) tests estimate Unconfined Compressive Strength of Cohesive Soils. qu are direct unconfined compressive strength test results. All measurements in tons per square foot.

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-8
Location: N 61°17.142' W 149°55.095'

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-7
Sheet 3 of 3

OVERWATER_LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 49 Ft. Approximate Elevation: -16.7 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Very dense, gray, slightly silty, gravelly SAND; wet				▽ 9/12/03	5
Hard, gray, slightly sandy, gravelly,silty CLAY; moist	6.0		S1		
S2: 26% Gravel, 67% Sand, 7% Fines Very dense, gray, slightly silty, gravelly SAND; wet	10.0		S2 S3		10 15
S4: 3.8% Fines Very dense, gray, sandy GRAVEL with trace of silt; wet S5: 58% Gravel, 38% Sand, 5% Fines S9: 69% Gravel, 31% Sand, 0% Fines	17.0		S4 S5 S6 S7 S8 S9 S10		20 25 30 35 40 45 50 55 60 65

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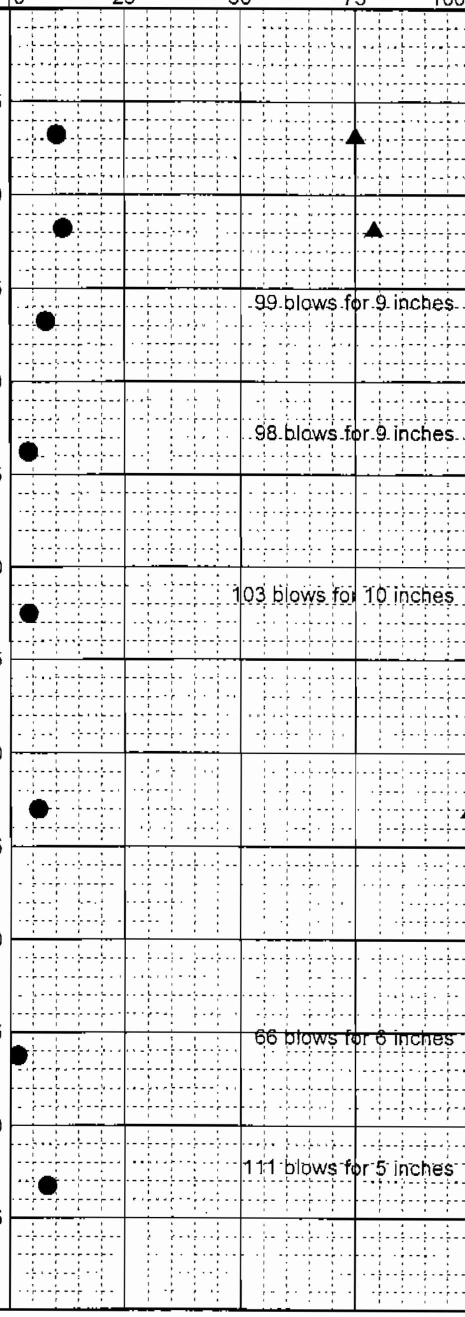
Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

* Sample Not Recovered
■ Shelby Tube
2" O.D. Split Spoon Sample
4" Push Core Sample
Rock Core Sample

▽ Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-9
Loation: N 61°16.967' W 149°54.502

February 2004 | 32-1-01536

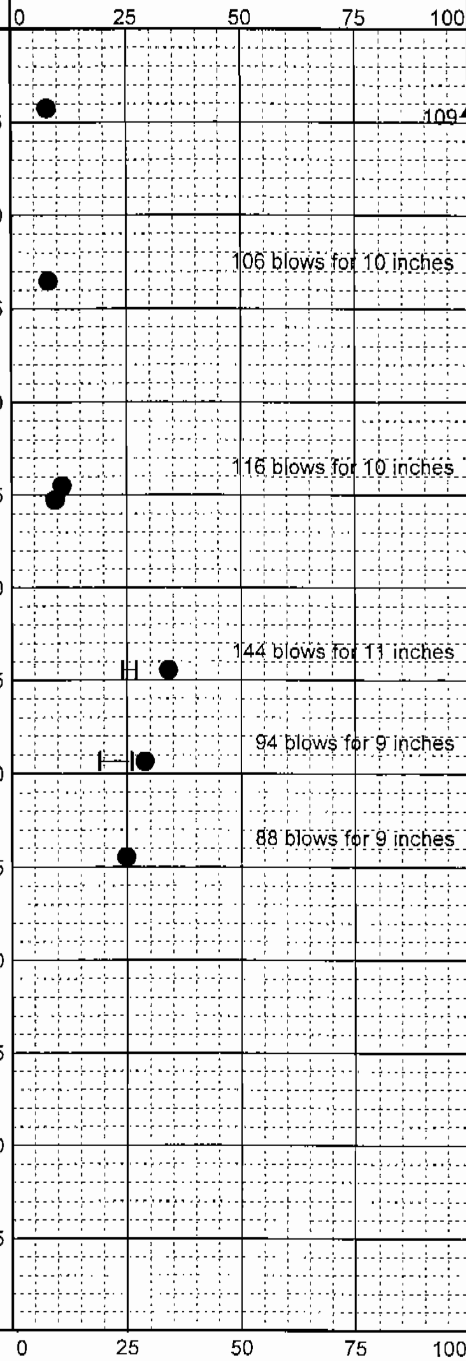
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-8
Sheet 1 of 2

OVERWATER_LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 49 Ft. Approximate Elevation: -16.7 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
S11: 4.5% Fines Very dense, gray, sandy GRAVEL with trace of silt; wet	80.0		S11		75 80
Very dense, gray, silty, gravelly SAND; wet	95.0		S12 S13a S13b		85 90 95
Hard, gray, slightly gravelly, sandy, silty CLAY; moist S15: PP = 2.25 S16: PP = 2.3 S17: 9% Gravel, 22% Sand, 70% Fines	114.9		S14 S15 S16 S17		100 105 110 115
Bottom of Boring Boring Completed 9/12/03					120 125 130 135

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
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4. PP (Pocket Penetrometer) tests estimate Unconfined Compressive Strength of Cohesive Soils. qu are direct unconfined compressive strength test results. All measurements in tons per square foot.

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-9
Loation: N 61°16.967' W 149°54.502

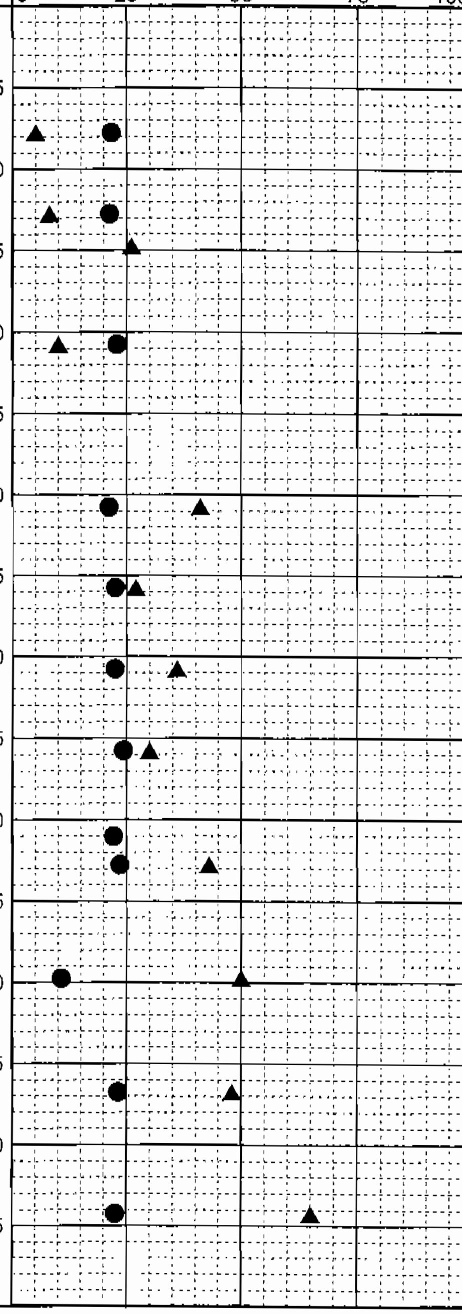
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-8
Sheet 2 of 2

OVERWATER_LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 69 Ft. Approximate Elevation: -34.4 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.	Penetration Resistance (140 lb. weight, 30" drop) ▲ Blows per foot ● Water Content (%)
Loose to medium dense, gray, slightly silty, fine SAND; wet S2: 0% Gravel, 91% Sand, 9% Fines			S1, S2, S3*, S4	9/18/03 ▽	5, 10, 15, 20, 25	
Medium dense, gray, fine SAND, with trace of silt; wet S6: 0% Gravel, 96% Sand, 4% Fines	28.0		S5, S6, S7		30, 35, 40	
S10: 1% Gravel, 90% Sand, 9% Fines Dense to very dense, gray, slightly silty, fine SAND, trace of gravel; wet Gravel content decreases with depth S12: 5.9% Fines S13: 6% Gravel, 85% Sand, 9% Fines	43.0		S8, S9, S10, S11, S12, S13		45, 50, 55, 60, 65, 70, 75	



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LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

▽ Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-10
Location: N 61°17.248' W 149°52.945'

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-9
Sheet 1 of 3

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 69 Ft. Approximate Elevation: -34.4 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
			S14		
					85
S15: 4% Gravel, 87% Sand, 9% Fines			S15		90
Dense to very dense, gray, slightly silty, fine SAND, trace of gravel; wet Gravel content decreases with depth			S16		95
					100
S17: 0% Gravel, 91% Sand, 9% Fines			S17		105
					110
S18: 9.5% Fines			S18		115
					120
					125
S19: 0% Gravel, 86% Sand, 14% Fines Dense, gray, silty fine SAND; wet	126.0		S19		130
Very dense, gray, sandy, silty GRAVEL, with boulders; wet	136.0		S20		135 140 145
Hard, gray, sandy, silty CLAY; moist S21: 67.1% Fines	150.0		S21		150 155

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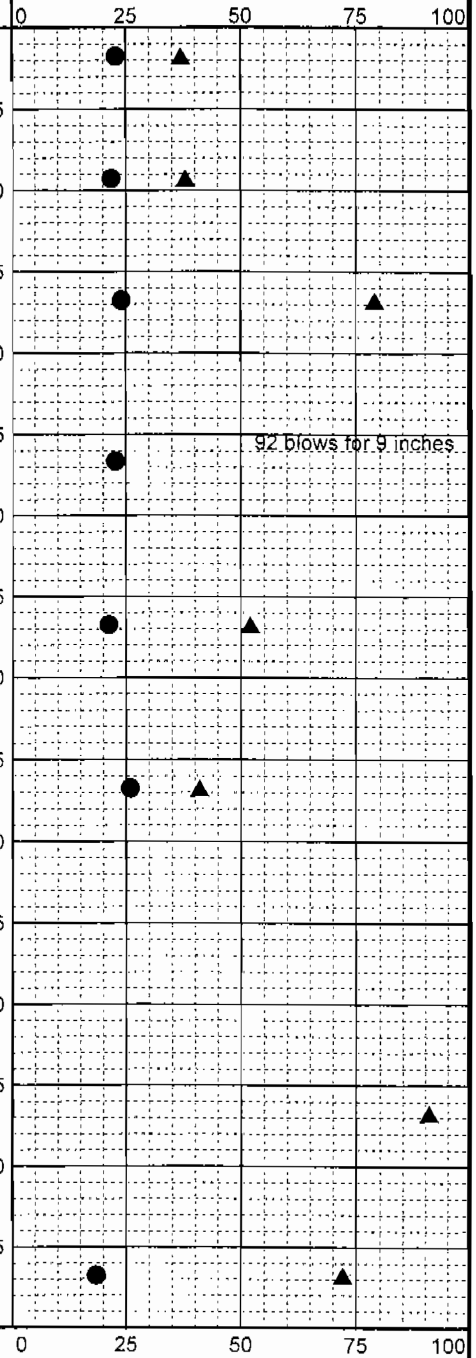
Penetration Resistance (140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-10
Location: N 61°17.248' W 149°52.945'

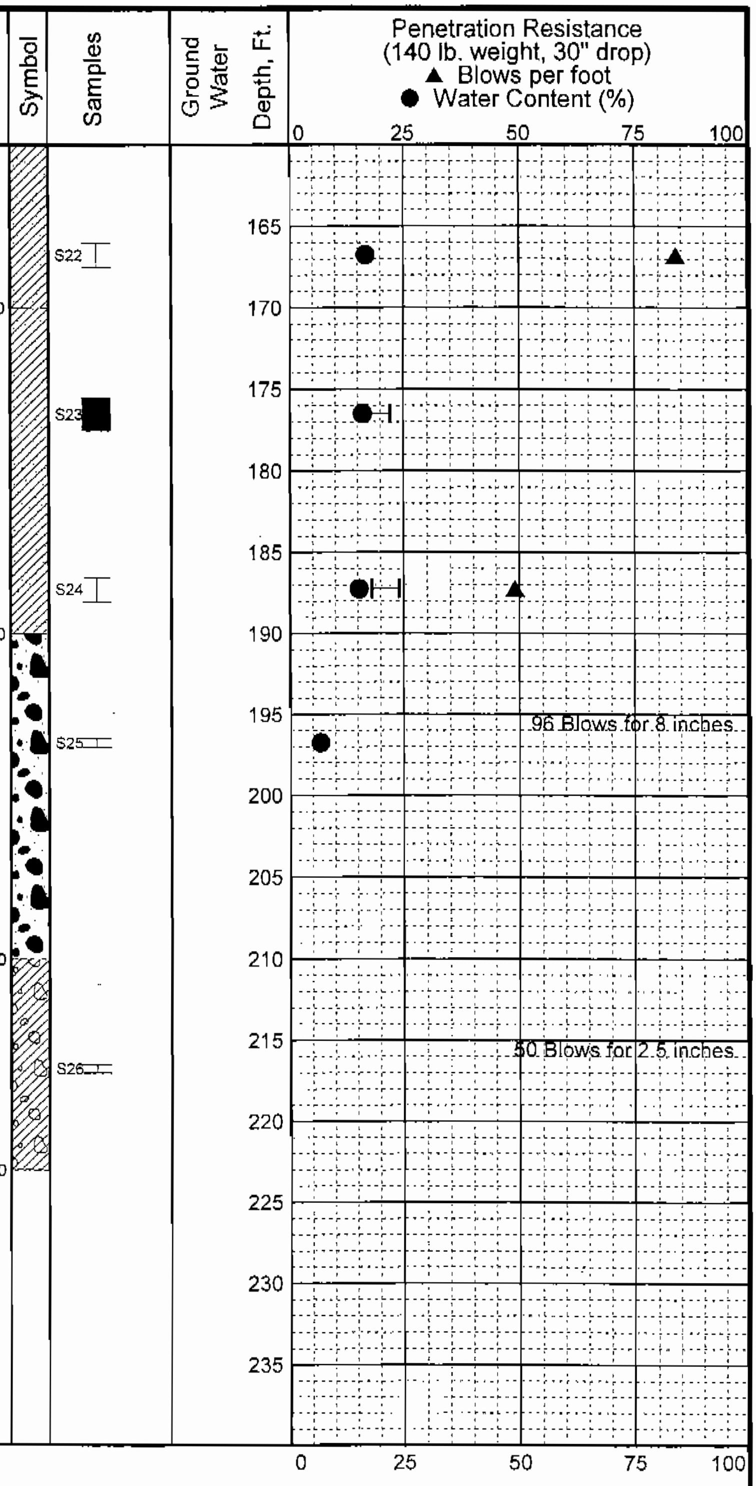
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-9
Sheet 2 of 3

OVERWATER_LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 69 Ft. Approximate Elevation: -34.4 Ft. (MLLW Datum)	Depth, Ft.	Samples
Hard, gray, sandy, silty CLAY; moist		S22
	170.0	
S23: PP = <1 qu = 3.6tsf uu = 3.0tsf Hard, gray, gravelly, sandy, silty CLAY; moist to wet S24: 65.4% Fines		S23 S24
	190.0	
Very dense, gray, silty, sandy GRAVEL, with cobbles; wet		S25
	210.0	
Hard, gray, slightly sandy, gravelly, silty CLAY; moist		S26
	223.0	
Bottom of Boring Boring Completed 9/18/03		



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-10
Location: N 61°17.248' W 149°52.945'

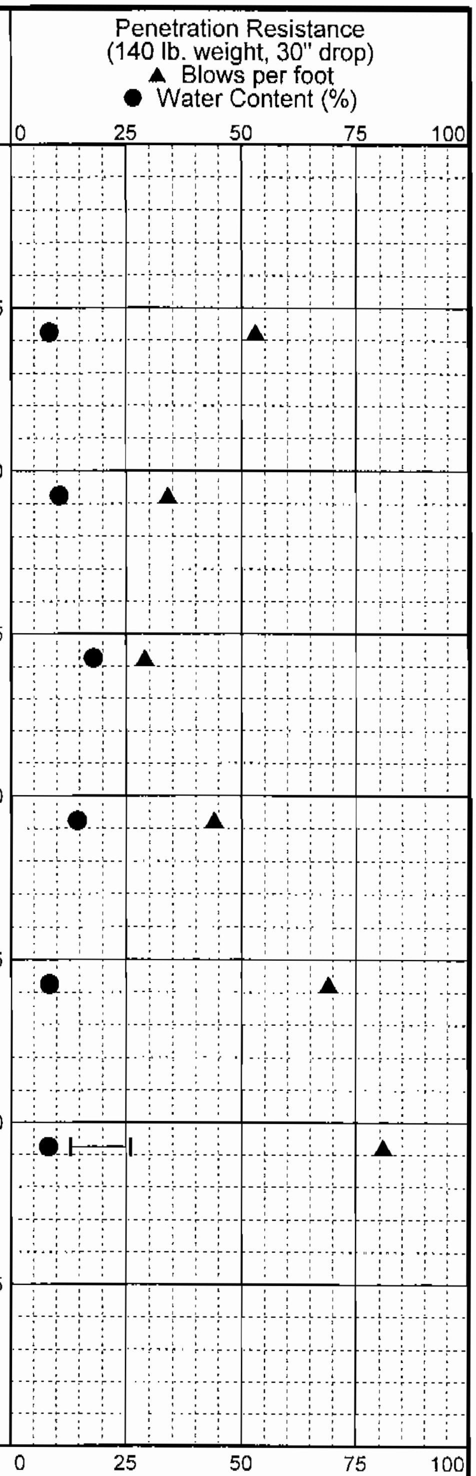
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-9
Sheet 3 of 3

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Elevation: 30.4 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Hard, gray, slightly gravelly, sandy, silty CLAY with sand seams; moist S2: 9% Gravel, 18% Sand, 73% Fines			S1 S2 S3		5 10 15
Hard, gray, gravelly, silty CLAY with sand seams; moist	18.0		S4 S5 S6	10/21/03	20 25 30
Bottom of Boring Boring Completed 10/21/03	31.5				35



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-11
Location: N 61°15.816' W 149°52.133'

February 2004 32-1-01536

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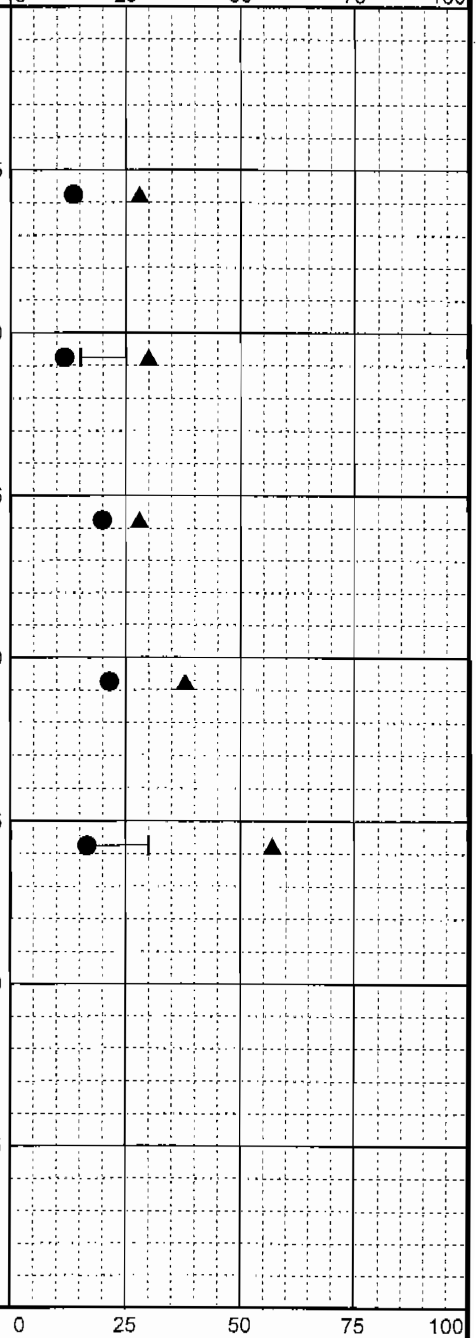
Geotechnical and Environmental Consultants

Fig. B-10

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Water Depth Above Mudline: 45 Ft. Approximate Elevation: 30.1 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Very stiff, gray CLAY with sand seams and trace of gravel; moist			S1		5
S2: 90.2% Fines			S2		10
			S3		15
	18.0				
Hard, gray, slightly gravelly CLAY with sand seams; moist			S4	10/21/03	20
			S5		25
Bottom of Boring Boring Completed 10/21/03	26.5				30
					35

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-12
Location: N 61°15.700' W 149°52.366'

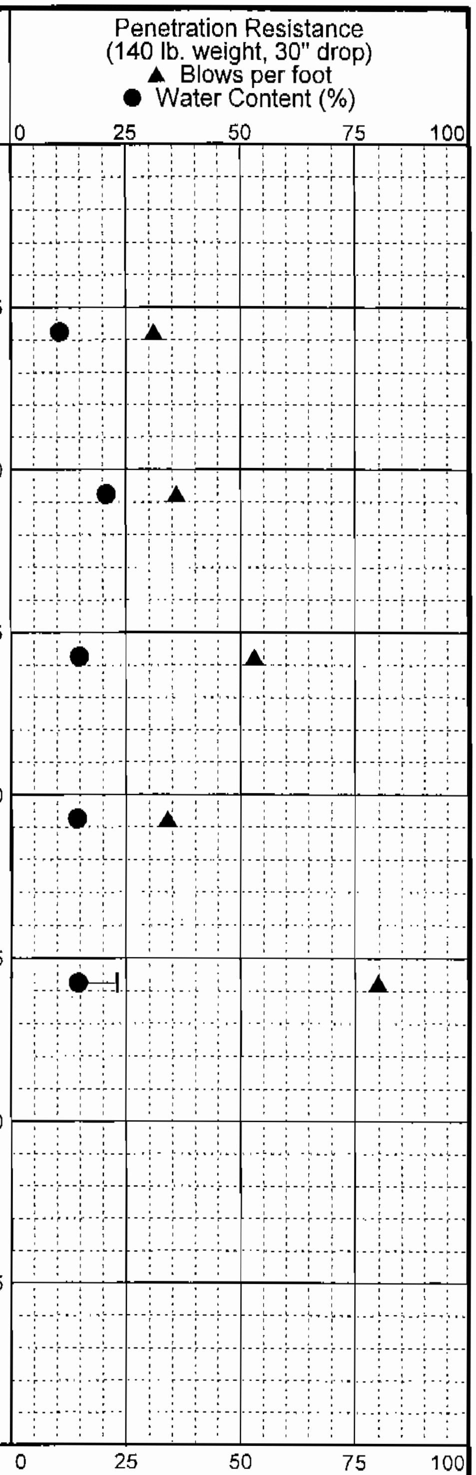
February 2004 — 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-11

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Elevation: 31.3 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Very stiff, gray, gravelly, sandy, silty CLAY; moist S1: 16% Gravel, 32% Sand, 52% Fines			S1		5
Hard, gray, slightly gravelly, sandy, silty CLAY with sand seams and layers; moist	10.5		S2 S3 S4 S5	10/22/03	10 15 20 25
Bottom of Boring Boring Completed 10/22/03	26.5				30 35



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit —●— Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-13
Location: N 61°15.566' W 149°52.583'

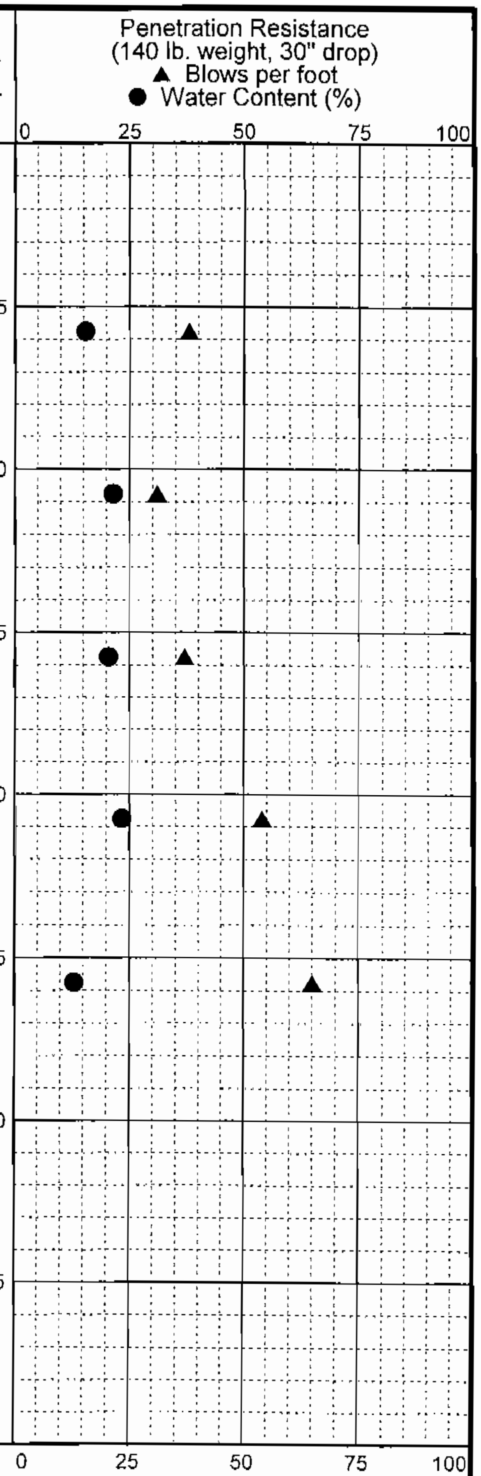
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-12

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Elevation: 29.7 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water
Hard, gray, silty CLAY with trace of gravel; moist to wet			S1	
Dense, gray, silty fine SAND; wet S3: 23.6% Fines	8.0		S2, S3	
Hard, gray, sandy, silty CLAY; moist S5: 78.7% Fines	19.0		S4, S5	10/22/03
Bottom of Boring Boring Completed 10/22/03	26.5			



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-14
Location: N 61°15.416' W 149°52.783'

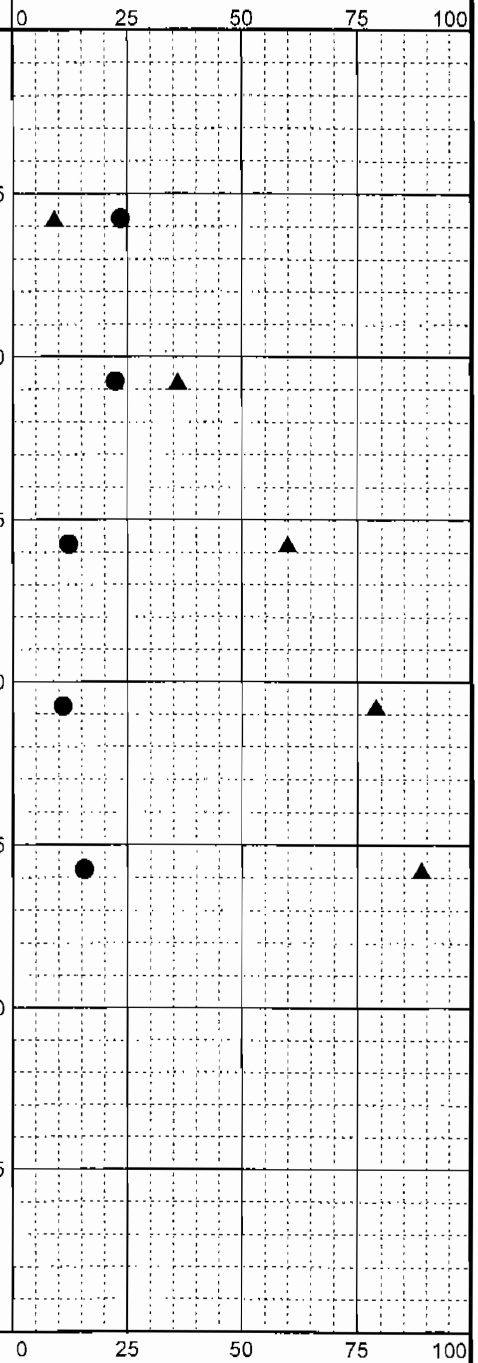
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-13

MATERIAL DESCRIPTION Approximate Elevation: 30.2 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Stiff, gray, sandy, silty CLAY; wet S1: 98.9% Fines			S1		5
Dense to very dense, gray, slightly gravelly, silty SAND; wet S3: 44.5% Fines	8.0		S2 S3 S4 S5	10/22/03	10 15 20 25
Bottom of Boring Boring Completed 10/22/03	26.5				30 35

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |——●——| Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-15
Location: N 61°15.268' W 149°52.648'

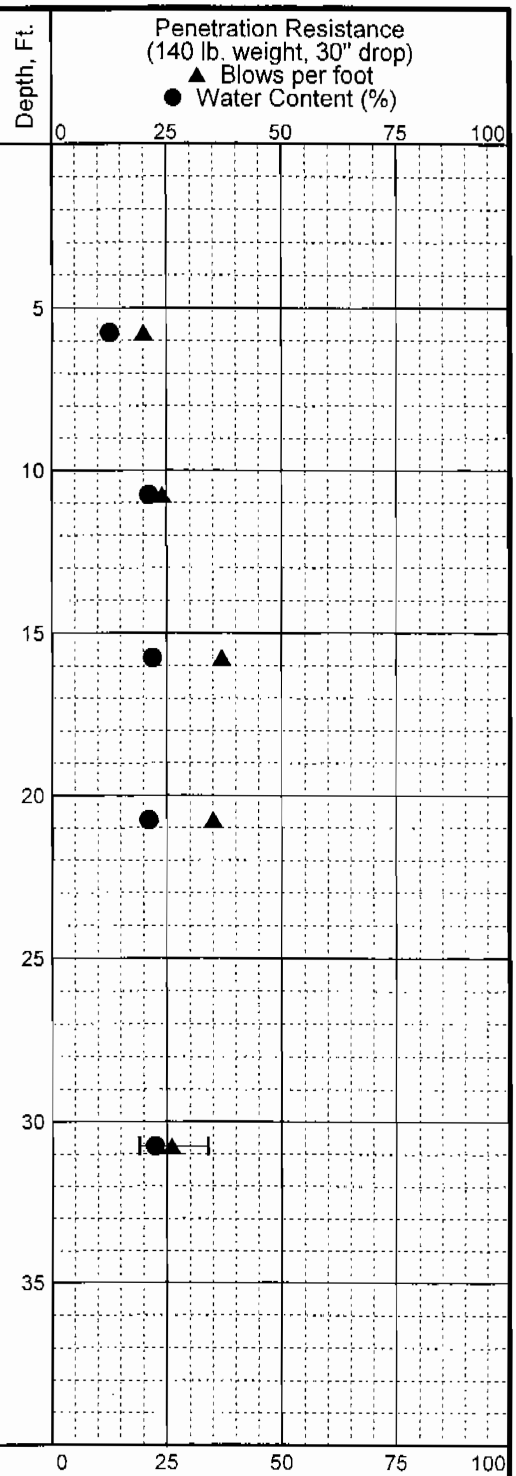
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-14

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Elevation: 30.4 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water
Medium dense, gray, slightly silty SAND; moist				
S1: 10% Gravel, 27% Sand, 63% Fines Very stiff, gray, slightly gravelly, sandy, silty CLAY; moist	5.5		S1	
Dense, gray to brown, silty fine SAND; wet S3: 22.9% Fines	8.0		S2 S3 S4	Groundwater Not Encountered During Drilling on 10/22/03
Very stiff, gray, silty CLAY; moist	28.0		S5	
Bottom of Boring Boring Completed 10/22/03	31.5			



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

∇ Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

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Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-16
Location: N 61°15.138' W 149°52.656'

February 2004 | 32-1-01536

SHANNON & WILSON, INC.
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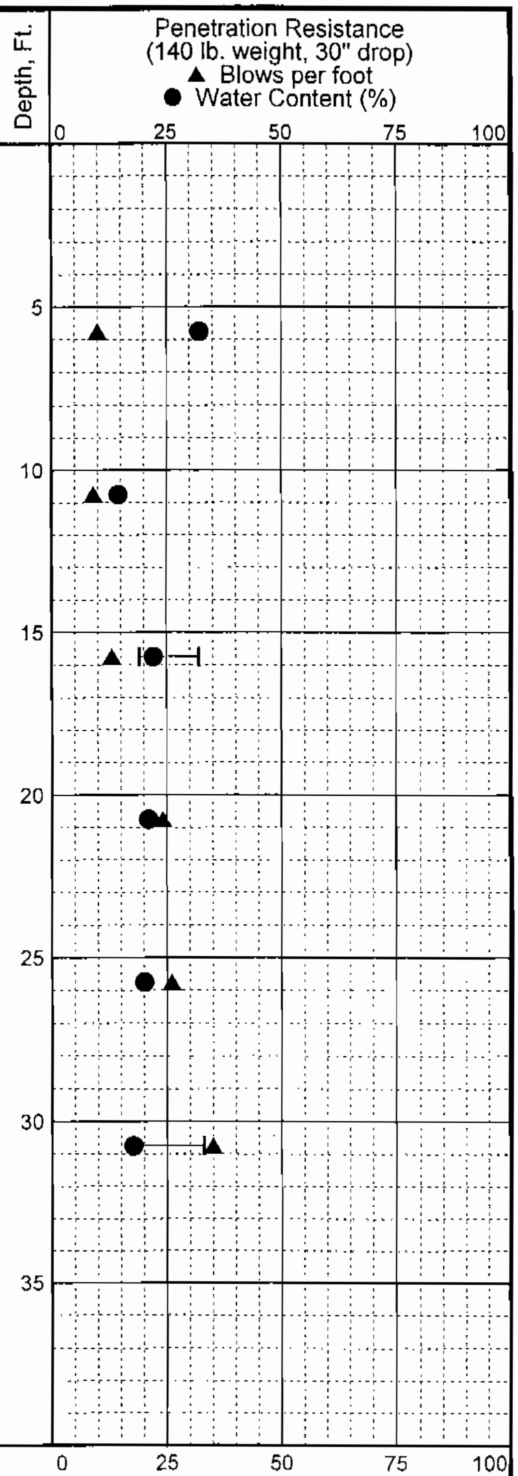
Fig. B-15

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Elevation: 31.5 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water
Stiff, gray, sandy, silty CLAY with trace of gravel; moist			S1	
	8.0			
Stiff, gray, gravelly, sandy, silty CLAY; wet S2: 16% Gravel, 21% Sand, 63% Fines			S2	Groundwater Not Encountered During Drilling on 10/22/03
	13.0			
Very stiff to hard, gray, gravelly, sandy, silty CLAY with ocassional sand seams; moist			S3 S4 S5 S6	
	31.5			
Bottom of Boring Boring Completed 10/22/03				



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. PP (Pocket Penetrometer) tests estimate Unconfined Compressive Strength of Cohesive Soils. qu are direct unconfined compressive strength test results. All measurements in tons per square foot.

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING A-17
Location: N 61°14.925' W 149°52.652'

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

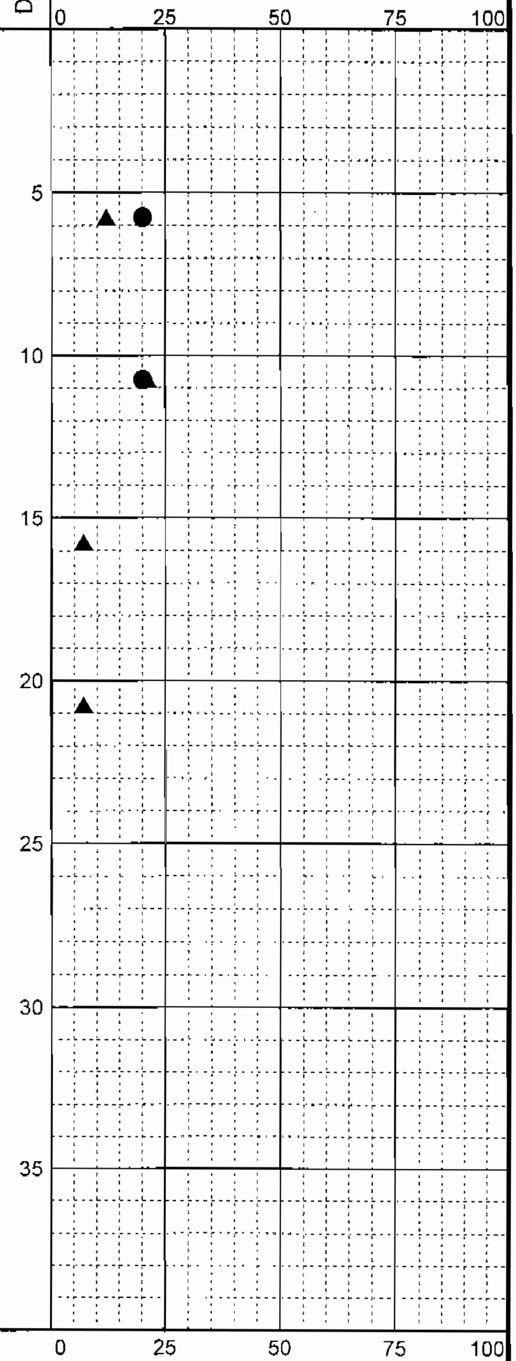
Fig. B-16

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

MATERIAL DESCRIPTION Approximate Elevation: 49.2 Ft. (MLLW Datum)	Depth, Ft.	Symbol	Samples	Ground Water
Medium dense, brown, silty, gravelly, SAND				
Very Stiff, gray, sandy SILT	6.0		S1, S2	
Medium stiff, gray, silty CLAY; moist	15.0		S3	
Soft, brown, fibrous PEAT	20.0		S4	
Medium stiff, gray, silty CLAY; moist	21.0			
Bottom of Boring Boring Completed 11/15/96	21.5			

Groundwater Not Encountered During Drilling on 11/15/96

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- Shelby Tube
- 2" O.D. Split Spoon Sample
- 4" Push Core Sample
- Rock Core Sample

Ground Water Level At Time Of Drilling
Water Depths Are Lisited For High Tide
At Time Of Drilling And May Vary

● Water Content (%)
Plastic Limit |—●—| Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. PP (Pocket Penetrometer) tests estimate Unconfined Compressive Strength of Cohesive Soils. qu are direct unconfined compressive strength test results. All measurements in tons per square foot.

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING :
Port of Anchorage Boring B-13

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. B-17

OVERWATER LOG 1536NOPP.GPJ SWNEW3.GDT 2/11/04

BLOWS/FOOT	DEPTH (FT)	DESCRIPTION	OTHER TESTS
29		no recovery at 0.0'	
		GRAY SAND (SP-SM)	PSA
17		loose to medium dense	
29		silty at 8.5'	
	10		
			PSA
13	20	no recovery at 19.0'	
		GRAY SILTY SAND (SM) medium dense, fine-grained sand, with zones of decreased silt content	
26	30		PSA
26	40		PSA, OLI
15			Minus #200=4.3%
	50		
9			OLI, PSA
	60	trace of fibrous organics at 60.0'	
39	70	becoming dense at 70.0'	OLI, PSA
		fine-grained sand with abundant fibrous organics and scattered gravel at 75.0' to 80.0'	
41	80	peat seam at 80.0' to 80.4'	PSA, OLI OLI, PSA
		becomes gravelly at 87.0' to 90.0'	
51	90	gravel to 3/4"; 1/4" seam of organics at 30° bedding angle	
59		abundant organics and wood pieces in wash returns	Minus #200=9.3%
	100		
85		trace of gravel and 1" seam of wood	PSA, Radiocarbon Date
	110	Boring Terminated at 109.5'; Casing Broke	

MOISTURE CONTENT (%): 0, 25, 50, 75
SHEAR STRENGTH (KSF): 0, 1, 2, 3
SAMPLES, GRAPHIC LOG, DRY DENSITY (PCF)

DATE DRILLED 7/21/84 WATER DEPTH (MLLW) 56 FT.

EQUIPMENT Failing 750 Drill Rig BORING COORDINATES x=521 236 y=2 661 291

SHEAR STRENGTH: ▲ Torvane △ Triaxial Test ■ Lab Vane

MOISTURE CONTENT: Plastic Limit ├──●── Natural ──┤ Liquid Limit

NOTE: Blow counts marked with an asterisk were determined using a 300 lb hammer free falling 30 inches onto a 2.5 inch I.D. (3.0 inch O.D.) split spoon sampler. Unmarked blow counts were determined using standard penetration test methods.

Harding Lawson Associates
Engineers, Geologists & Geophysicists

Log of Boring 4
Knik Arm Crossing
Anchorage, Alaska

PLATE **A4**

DRAWN | JOB NUMBER 9620,016.08 | APPROVED | DATE 8/84 | REVISED | DATE

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING:
Harding Lawson Boring A-4
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. B-18

BLOWS/FOOT	DEPTH (FT)	DESCRIPTION	DRY DENSITY (PCF)	OTHER TESTS
		GRAY SAND (SP) loose easy casing advancement, loose materials at seafloor		
175	10	GRAY SILTY SANDY GRAVEL (GM) very dense, gravel to 1-1/2", fine to coarse sand, with seams of gravelly silty sand (SM)		PSA
	20			
98*	30		144 141	PSA
	40			
		cobbles at 46.0'		
104*	50		141 150	PSA, CLI
	60			
	70	very dense, gravel to 2"		
50/3"				Minus #200=5.0%
	80			
		drilling behavior indicates seams of silty sand at 86.0' to 87.0'		
	90			
144/4"	100	Boring Terminated at 102.0'		Minus #200=2.2%

MOISTURE CONTENT (%) 0 25 50 75
SHEAR STRENGTH (KSF) 0 1 2 3

SAMPLES GRAPHIC LOG

DATE DRILLED 7/23/84-7/25/84 WATER DEPTH (MLLW) 46 FT.

EQUIPMENT Failing 750 Drill Rig BORING COORDINATES x=522 072 y=2 661 197

SHEAR STRENGTH

▲ Torvane △ Triaxial Test ■ Lab Vane

MOISTURE CONTENT

Plastic Limit ├── Natural ●──┤ Liquid Limit

NOTE: Blow counts marked with an asterisk were determined using a 300 lb hammer free falling 30 inches onto a 2.5 inch I.D. (3.0 inch O.D.) split spoon sampler. Unmarked blow counts were determined using standard penetration test methods.

Harding Lawson Associates
Engineers, Geologists & Geophysicists

Log of Boring 5

Knik Arm Crossing
Anchorage, Alaska

PLATE **A5**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	9620,016.08		8/84		

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING:
Harding Lawson Boring A-5

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. B-19

BLOWS/FOOT	DEPTH (FT)	DESCRIPTION	MOISTURE CONTENT (%) 0 25 50 75 / SHEAR STRENGTH (KSF) 0 1 2 3	DRY DENSITY (PCF)	OTHER TESTS
77		hard materials at seafloor, slow casing advancement; wash returns are rock pieces from 0.0' to 3.0' and fine sand below 3.0'			
		GRAY SILTY GRAVEL (GM) dense			
	10	GRAY SILTY SAND (SM)			
		GRAY GRAVELLY SILT (ML) hard			
		GRAY SILTY SANDY GRAVEL (GP-GM) very dense, gravel to 2"			
136					
	20				
	30	very difficult driving, fractured gravel pieces to 2" in wash return			
	40	Boring Terminated at 38.5'; Casing Broke			
	50				
	60				
	70				
	80				
	90				
	100				

DATE DRILLED 7/24/84-7/25/84 WATER DEPTH (MLLW) 36 FT.

EQUIPMENT Failing 750 Drill Rig BORING COORDINATES x=516 152 y=2 661 157

SHEAR STRENGTH

▲ Torvane △ Triaxial Test ■ Lab Vane

MOISTURE CONTENT

Plastic Limit ├── Natural ●──┤ Liquid Limit

NOTE: Blow counts marked with an asterisk were determined using a 300 lb hammer free falling 30 inches onto a 2.5 inch I.D. (3.0 inch O.D.) split spoon sampler. Unmarked blow counts were determined using standard penetration test methods.

Harding Lawson Associates
Engineers, Geologists & Geophysicists

Log of Boring 6
Knik Arm Crossing
Anchorage, Alaska

PLATE **A6**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	9620,016.08		8/84		

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING:
Harding Lawson Boring A-6

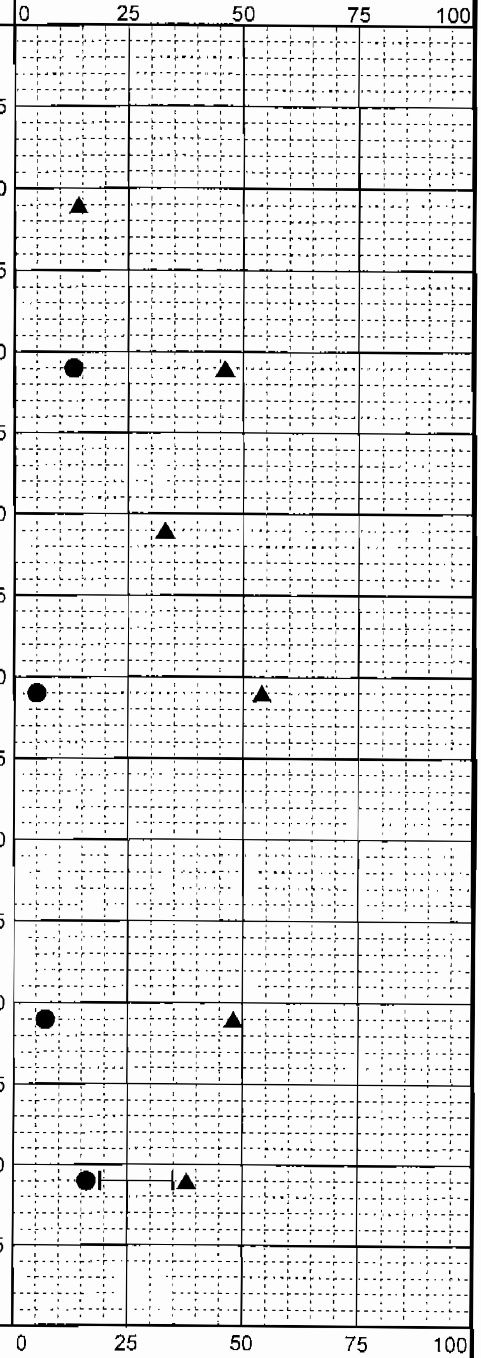
February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. B-20

MATERIAL DESCRIPTION Approximate Elevation:330 Ft.	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Medium dense, brown, slightly silty, gravelly SAND with scattered organics					5
	11.0		S1		10
Dense, brown, slightly silty SAND to silty, gravelly SAND; moist					15
S2: 8% Gravel, 45% Sand, 47% Fines			S2		20
				Groundwater Not Encountered During Drilling on 9/04/03	25
			S3		30
					35
S4: 35% Gravel, 49% Sand, 16% Fines			S4		40
					45
					50
					55
S5: 20% Gravel, 49% Sand, 31% Fines			S5		60
					65
Hard, gray, sandy SILT with trace of gravel and sand seams; moist	70.0		S6		70
	72.0				
Bottom of Boring Boring Completed 9/04/03					75

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- 3" O.D. Split Spoon Sample
- Ground Water Level At Time Of Drilling

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

Knik Arm Bridge
Anchorage, Alaska

LOG OF BORING :
Pt. MacKenzie Boring B-3

February 2004 32-1-01536

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Fig. B-21

GEOTECHNICAL LOG 1536B.GPJ SWNEW3.GDT 2/11/04

APPENDIX C

CONE PENETRATION TEST RESULTS

TABLE OF CONTENTS

Report Prepared by Gregg In Situ, Inc.

LIST OF FIGURES

LIST OF PLOTS

C-1 Cone Penetration Tests

The cone penetration tests (CPTU) with pore pressure measurements were carried out by Gregg In Situ using an integrated electronic cone system.

A 20 ton compression type cone (refer to Figure CPTU) was used for all of the soundings. This cone has a tip area of 15 sq. cm. and a friction sleeve area of 225 sq. cm. The compression cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.85 (based on pressure chamber testing). The porewater pressure filter was located directly behind the cone tip. The 5.0 mm thick filter is made of porous plastic. Each filter was saturated in silicone oil under vacuum pressure prior to penetration. Porewater pressure dissipation data was recorded at 5 second intervals during pauses in penetration as directed by the field representative.

The cone was capable of recording the following parameters at varying depth intervals:

Tip Resistance (q_c)
Sleeve Friction (f_s)
Dynamic Pore Pressure (u)
Temperature (T)
Cone Inclination (I)

A summary of the CPTs carried out is presented in the Project Summary Appendix.

Selected parameters were printed simultaneously on a printer and stored on a floppy disk for future analysis and reference. All cone penetration testing was carried out in accordance with ASTM D-5778-95.

A complete set of baseline readings was taken prior to and at the completion of each sounding to determine temperature shifts and any zero load offsets. Corrections for temperature shifts and zero load offsets can be extremely important, especially when the recorded loads are relative small. In sandy soils, however, these corrections are generally negligible. Graphical depictions of all CPT data are presented in several plots.

The inferred stratigraphic profile at each CPT test location is included with this report. The stratigraphic interpretations are based on relationships between cone bearing (q_t); Sleeve Friction (f_s); and dynamic pore pressure (u). The friction ratio, R_f (100 x f_s /q_t), is a calculated parameter, which is used to identify the type of soil and hence gives an indication of its behavior. Generally, soft cohesive soils have big friction ratios, low cone bearing pressures, and generate large porewater pressures during penetration. Cohesionless soils have lower friction ratios, high cone bearing pressures, and generate little in the way of excess porewater pressure during penetration. The classification of soils is based on correlations summarized by Robertson (1990) as shown in Figure SBT. It is not always possible to clearly identify a soil type based on q_t and f_s alone. Experience, judgment and analyses of porewater pressure generation during penetration and subsequent dissipation tests should be used in arriving at soil type in these ambiguous situations.

Stratigraphic interpretations using CPTU data using a normalized (stress corrected) soil behavior type chart (Robertson, 1990 – Figure SBTn) are also included in this report. The Robertson publication emphasizes when normalized stratigraphic interpretation is appropriate.

C-2 Gregg Digital File Formats

CPT Data Files

Unless otherwise required by the client, Gregg CPT data files are named such that the first 3 characters contain the job number, the next two characters are typically CP followed by two characters indicating the sounding number. The last DOS character position is reserved for the letters a, b, c, d, etc., to uniquely identify multiple sounds at the same location. The CPT sounding file has the extension COR and pore pressure dissipation files have the extension PPD. As an example, for job number 99-127 the first sounding will have file names 127CP01.COR and 127CP01.PPD.

The CPT (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. Ends of data marker
4. Units information

Header Lines

Line 1: Columns 1-6 are blank (future use)
Columns 7-21 contain the sounding Date and Time
Columns 22-36 contain the sounding Operator

Line 2: Columns 1-16 contain the Job Location
Columns 17-31 contain the Cone ID
Columns 32-47 contain the sounding number

Data Records

The data records contain 4 or more columns of data in floating point format. A comma (and spaces) separates each data item:

Column 1: Sounding depth (m)

Column 2: Tip (q_c) data uncorrected for pore pressure effects. Recorded in units selected by the operator.

Column 3: Sleeve (f_s) data. Recorded in units selected by the operator

Column 4: Dynamic pore pressure readings. Recorded in units selected by the operator

Column 5: Exists only if specialty modules (resistivity and/or UVIF) have been used.

End of Data Marker

After the last line of data a line containing ASCII 26 (CTL-Z) and a new line (carriage return/line feed) character. This is used to mark the end of data.

Units Information

The last section of the file contains information about the units that ere selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth, q_c, f_s, and u. The third line contains the conversion values required for Gregg's software to convert the recorded data to an internal set of base units (bar for q_{c1}, bar for f_s, and meters for u).

CPT Dissipation Files

CPT Dissipation files have the same naming convention as the CPT sounding files and have the extension PPR. PPR files consist of the following components:

1. Two lines of header information
2. Data records

Header Lines (same as COR file):

Line 1: Columns 1-6 are blank (future use)

Columns 7-21 contain the sounding Date and Time

Columns 22-36 contain the sounding Operator

Line 2: Columns 1-16 contain the Job Location
Columns 17-31 contain the Cone ID
Columns 32-47 contain the sounding number

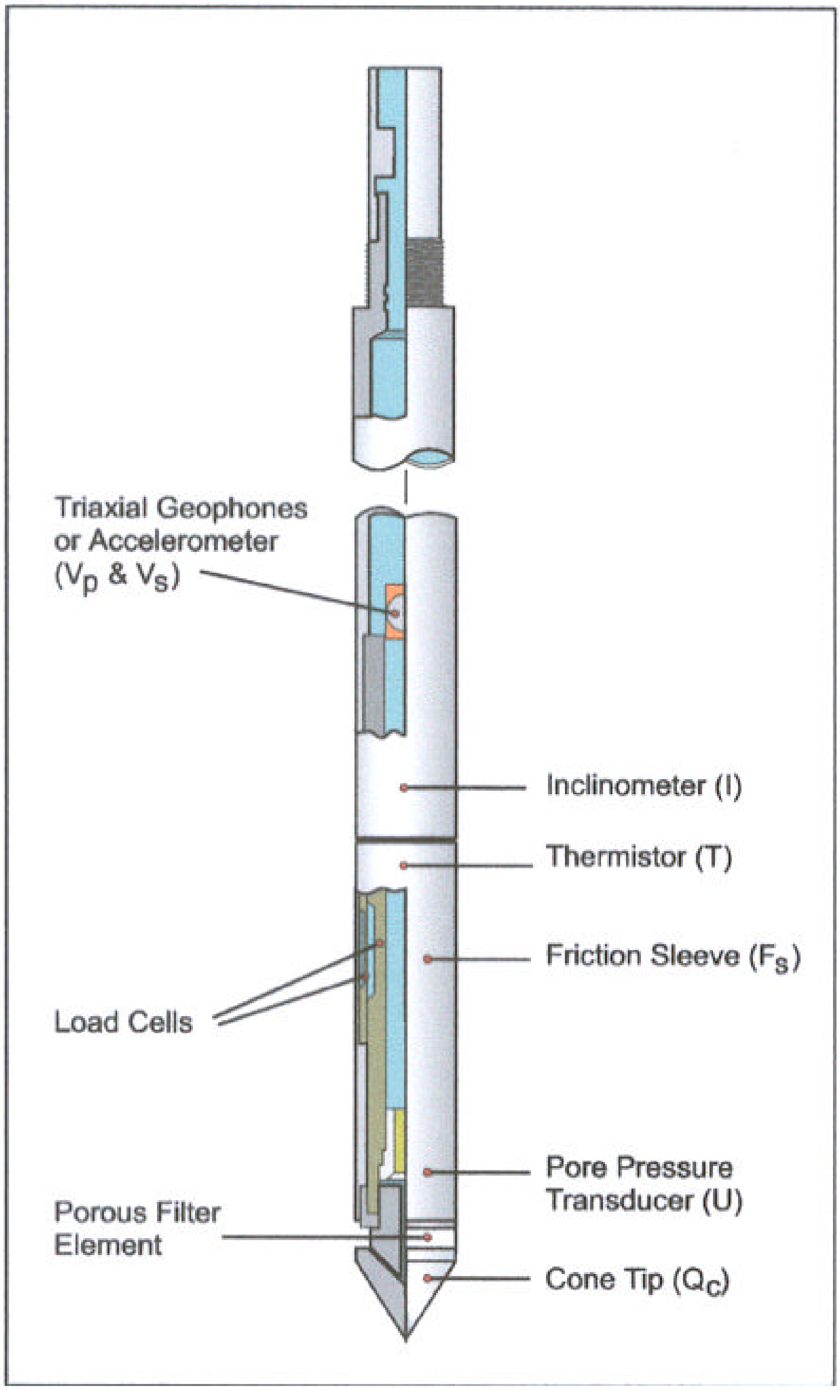
Data Records

The data records immediately follow the header lines. Each data record can occupy several lines in the file and is a complete record of a dissipation test at a particular depth. Each data record starts with a line containing two values separated by spaces; the first value being an index number (not currently used by the Software) and the second being the dissipation test depth in meters. Following this line are the dissipation pore pressure values stored at 5 second intervals with a maximum of 12 entries per line. The last line of the dissipation record may not contain a full 12 entries. The data record is terminated with an ASCII 30 character (appears as a triangle an some editors).

This sequence is repeated for every dissipation test in the sounding. No marker is used to indicate end of file. Units information is not stored in this file. Users would have to check the CPT file for the units that were used.

CPT Interpretations

Basic Geotechnical interpretations are contained in files having the extension TBL. These files are ASCII text files made up of several columns of Geotechnical Interpretations based on CPT data averaged over 20 cm increments. These files can be imported into various applications (e.g. Excel) for further analysis.



Knik Arm Bridge
Anchorage, Alaska

CONE CONFIGURATION

February 2004 | 32-1-01536

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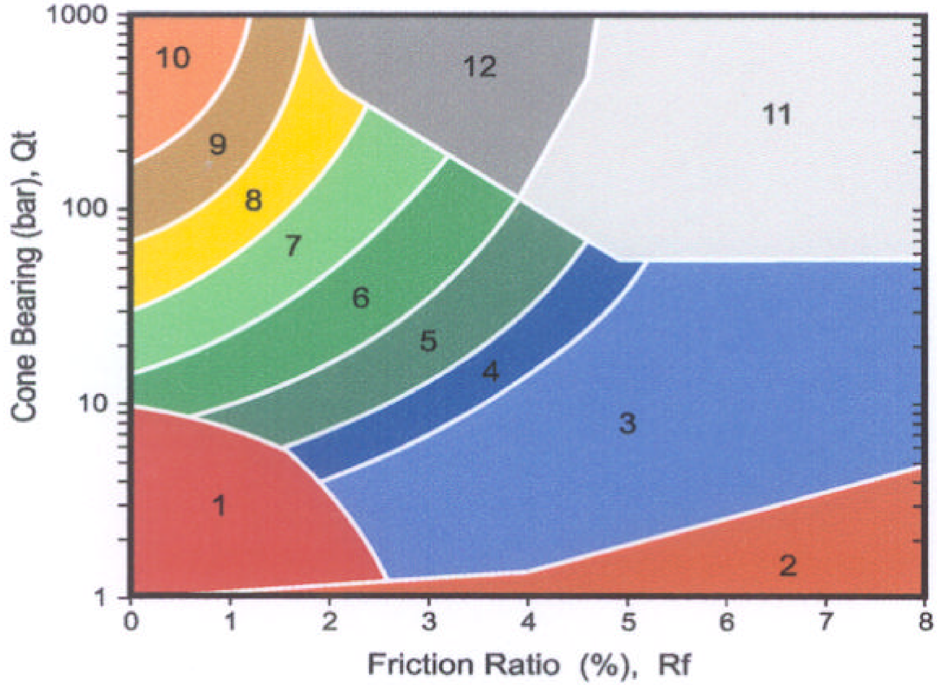
Fig. CPTU

NON-NORMALIZED SOIL BEHAVIOR TYPE CHART



Zone	Qt / N	Soil Behaviour Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

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NON-NORMALIZED SOIL BEHAVIOR TYPE CHART

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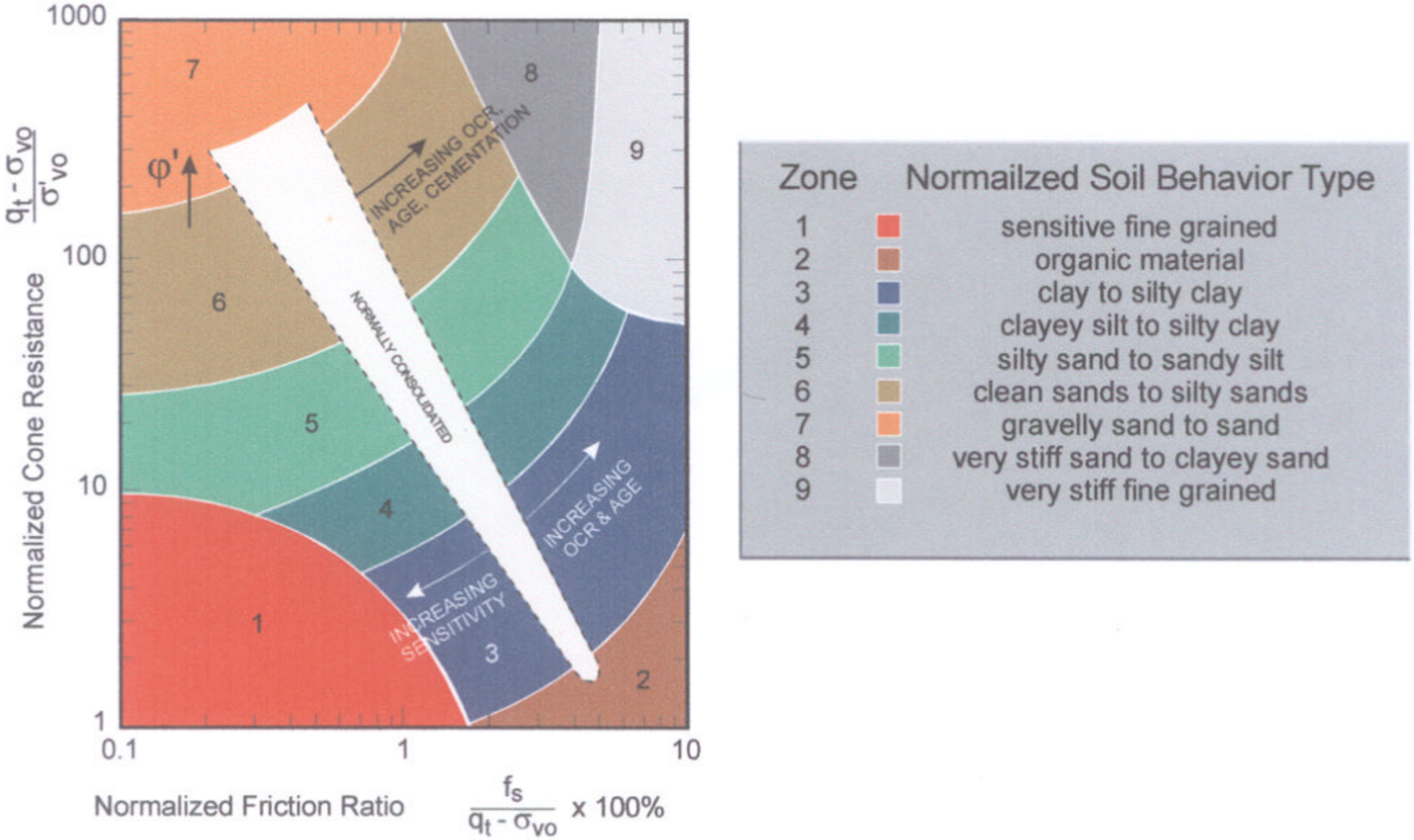
Fig. SBT

NORMALIZED SOIL BEHAVIOR TYPE CHART



Knik Arm Bridge
Anchorage, Alaska

NORMALIZED SOIL BEHAVIOR TYPE CHART

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Fig. SBTn

Pore Pressure

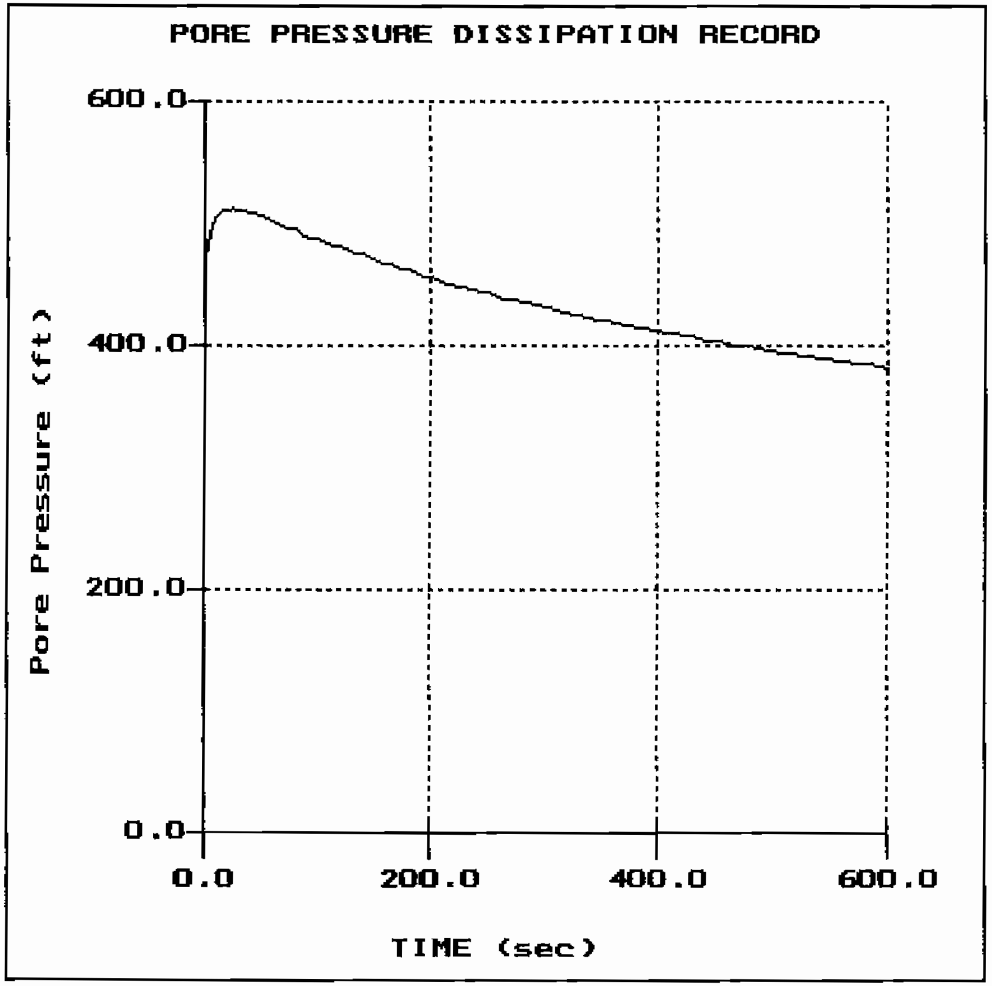
Dissipation Plots

Shannon & Wilson

Hole:KNIK ARM
Location:ADJACENT A-1

Cone: 20 Ton St 113
Date:08:20:03 05:24

File: 212CP01.PPR
Depth (m): 13.21
(ft): 43.34
Duration : 600.0s

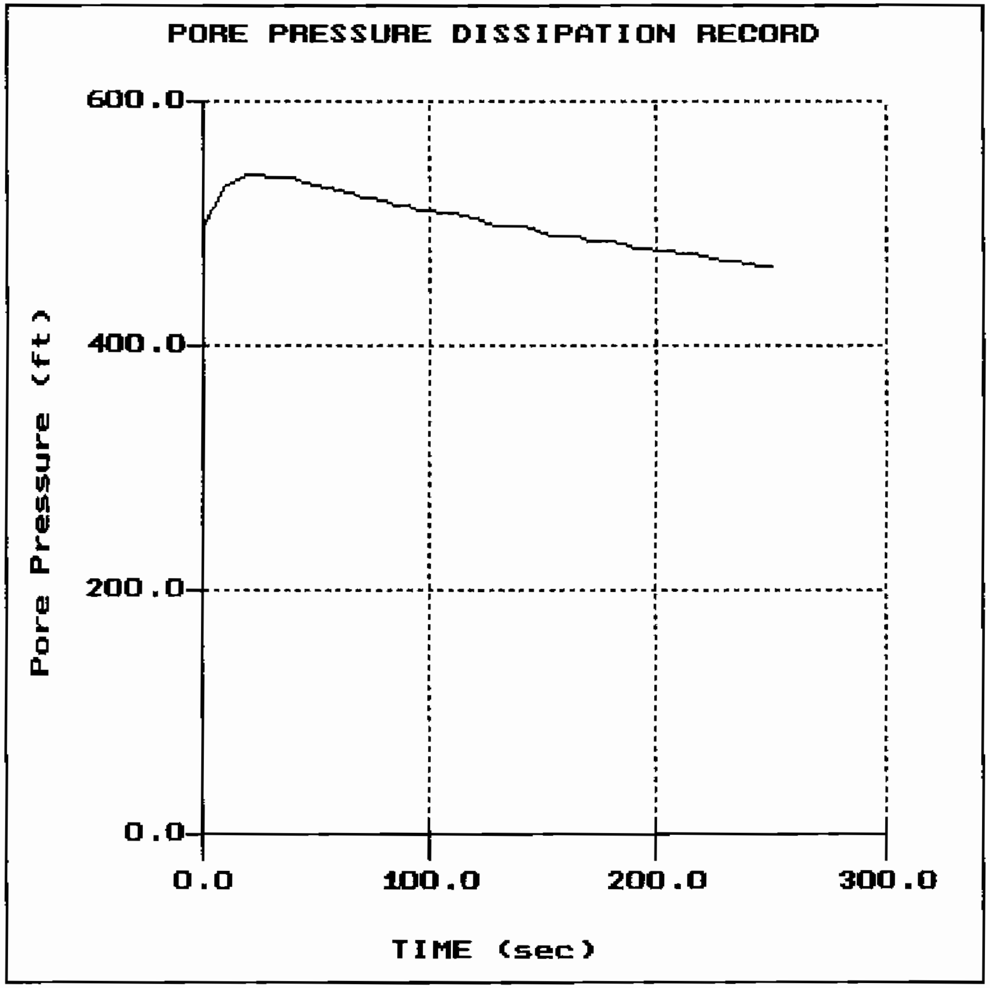


Shannon & Wilson

Hole:KNIK ARM
Location:ADJACENT A-1

Cone: 20 Ton St 113
Date:08:20:03 05:24

File: 212CP01.PPR
Depth (m): 16.50
(ft): 54.13
Duration : 250.0s

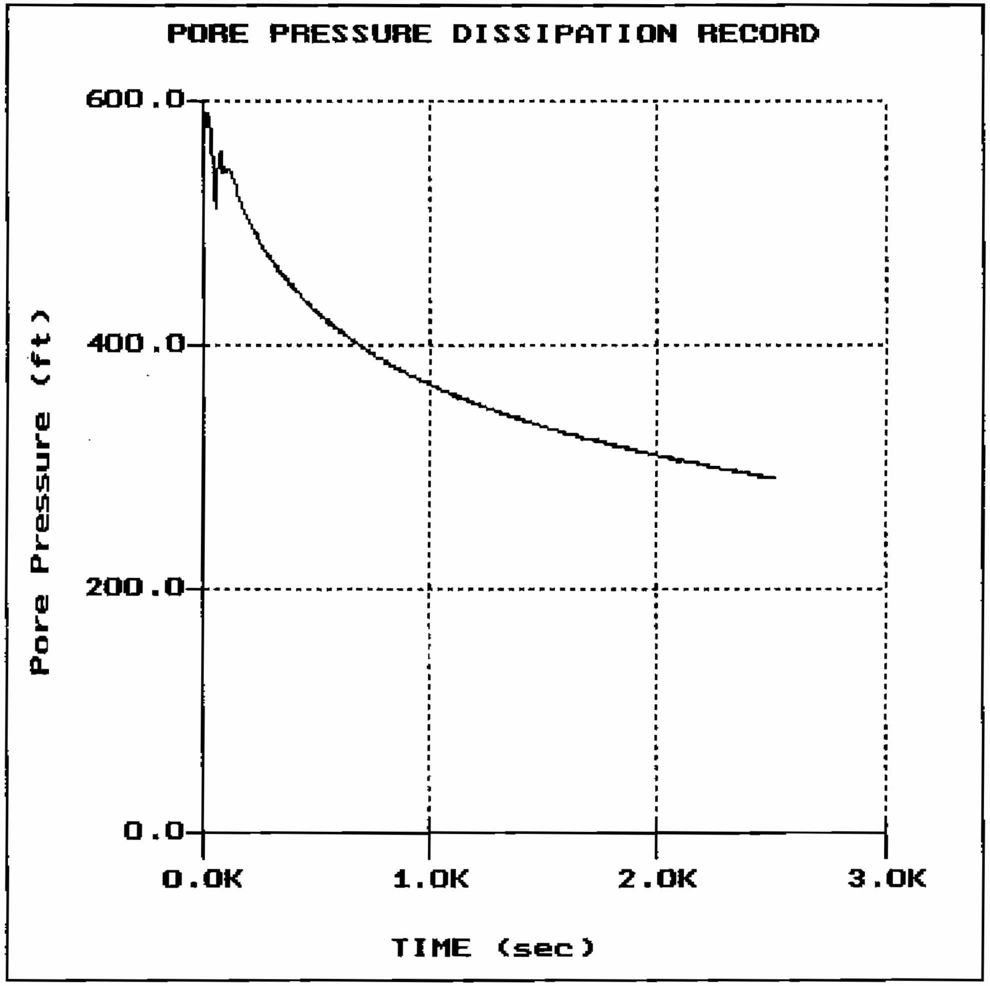


Shannon & Wilson

Hole:KNIK ARM
Location:SCPT-A5A

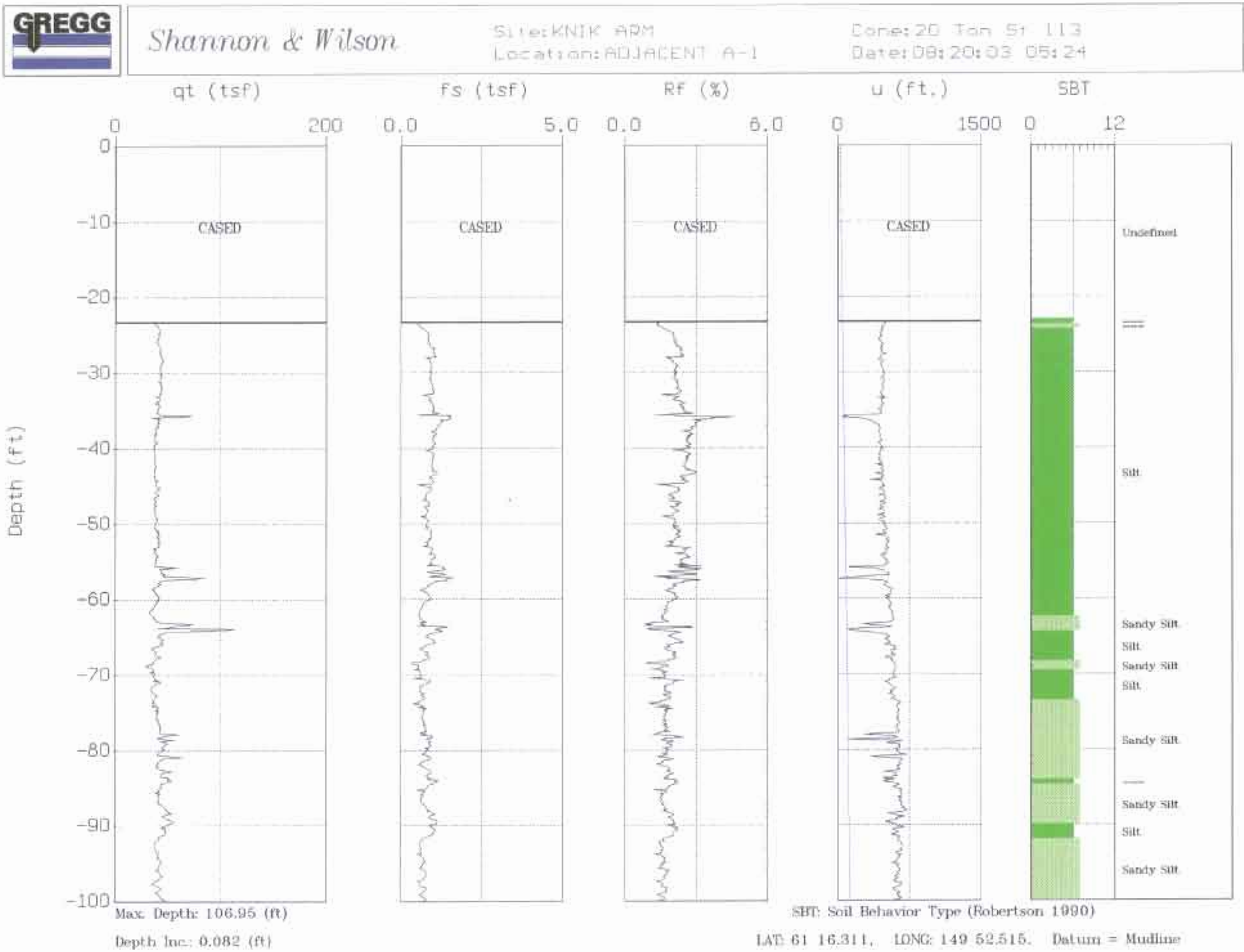
Cone: 20 Ton St 113
Date:09:21:03 10:59

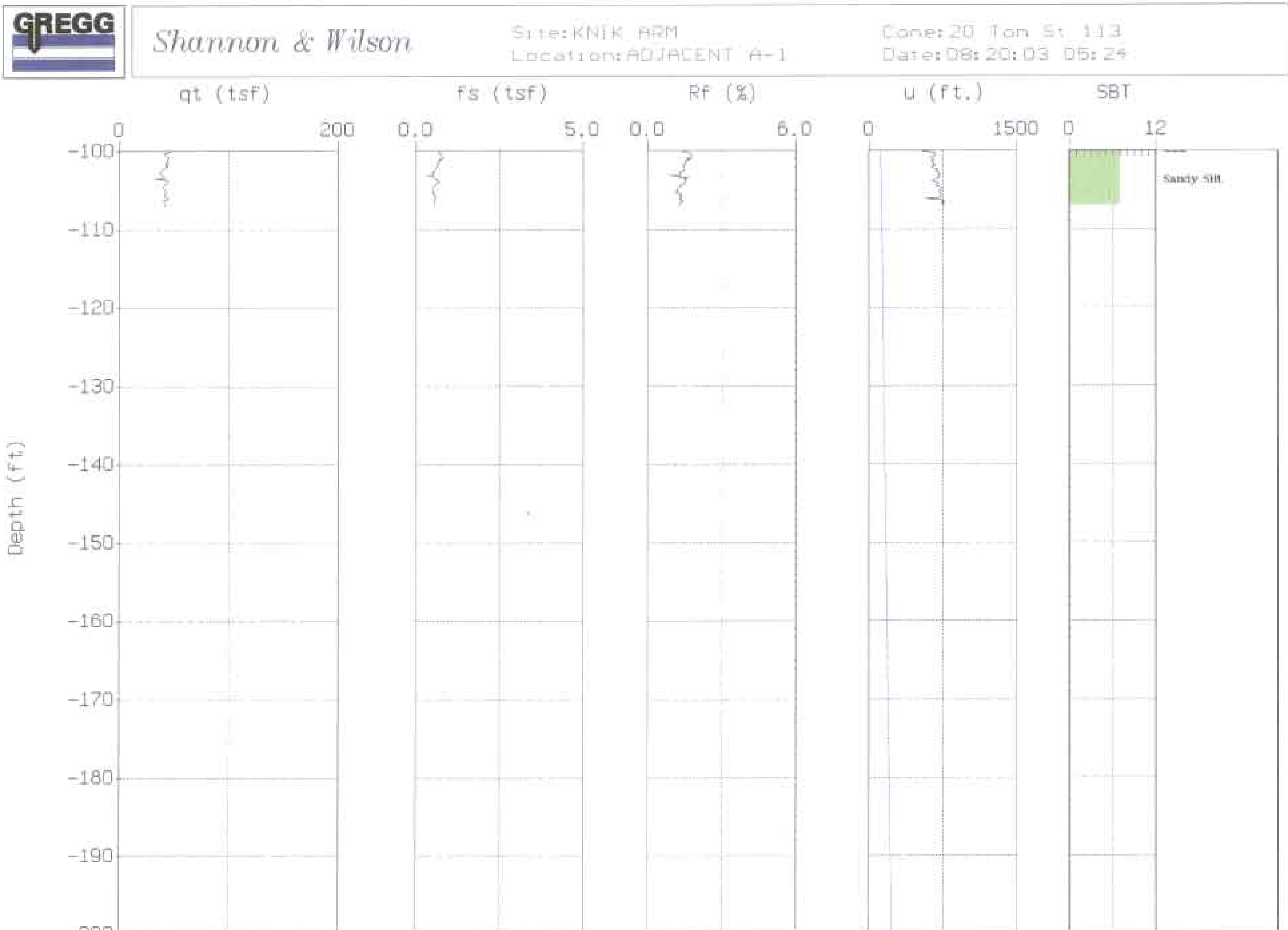
File: 212CP05.PPR
Depth (m): 16.50
(ft): 54.13
Duration : 2510.0s



CPT Plots Based on
Non-Normalized Soil Behavior Type

GREGG


GREGG
Shannon & Wilson
Site:KNIK ARM
Location:ADJACENT A-1
Cone:20 Ton S: 113
Date:08:20:03 05:24
qt (tsf)
fs (tsf)
Rf (%)
u (ft.)
SBT
0
200
0.0
5.0
0.0
6.0
0
1500
0
12
Depth (ft)
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
CASED
CASED
CASED
CASED
Undefined
Silt
Sandy Silt
Silt
Sandy Silt
Silt
Sandy Silt
Sandy Silt
Silt
Sandy Silt
Max. Depth: 106.95 (ft)
Depth Inc.: 0.082 (ft)
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline

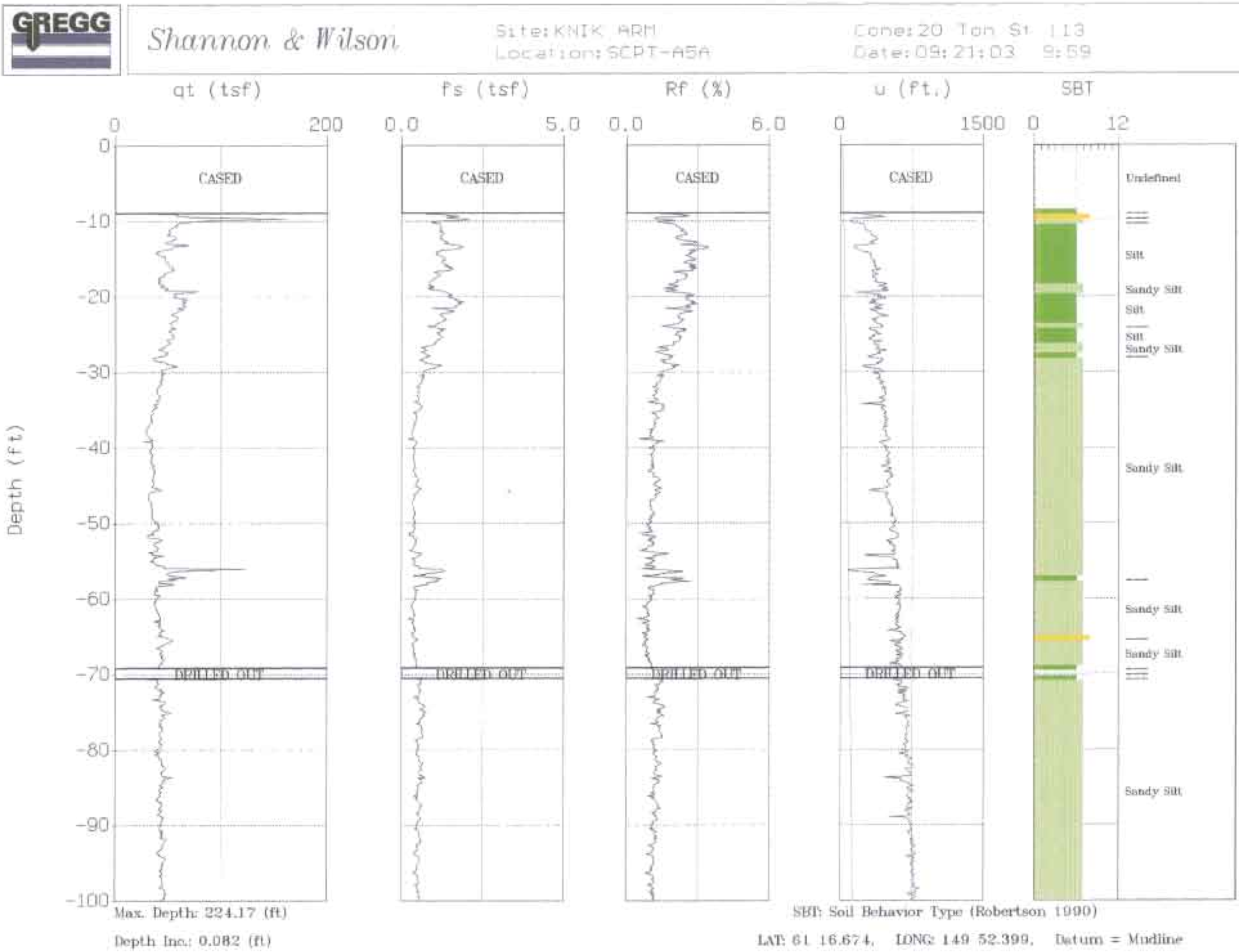


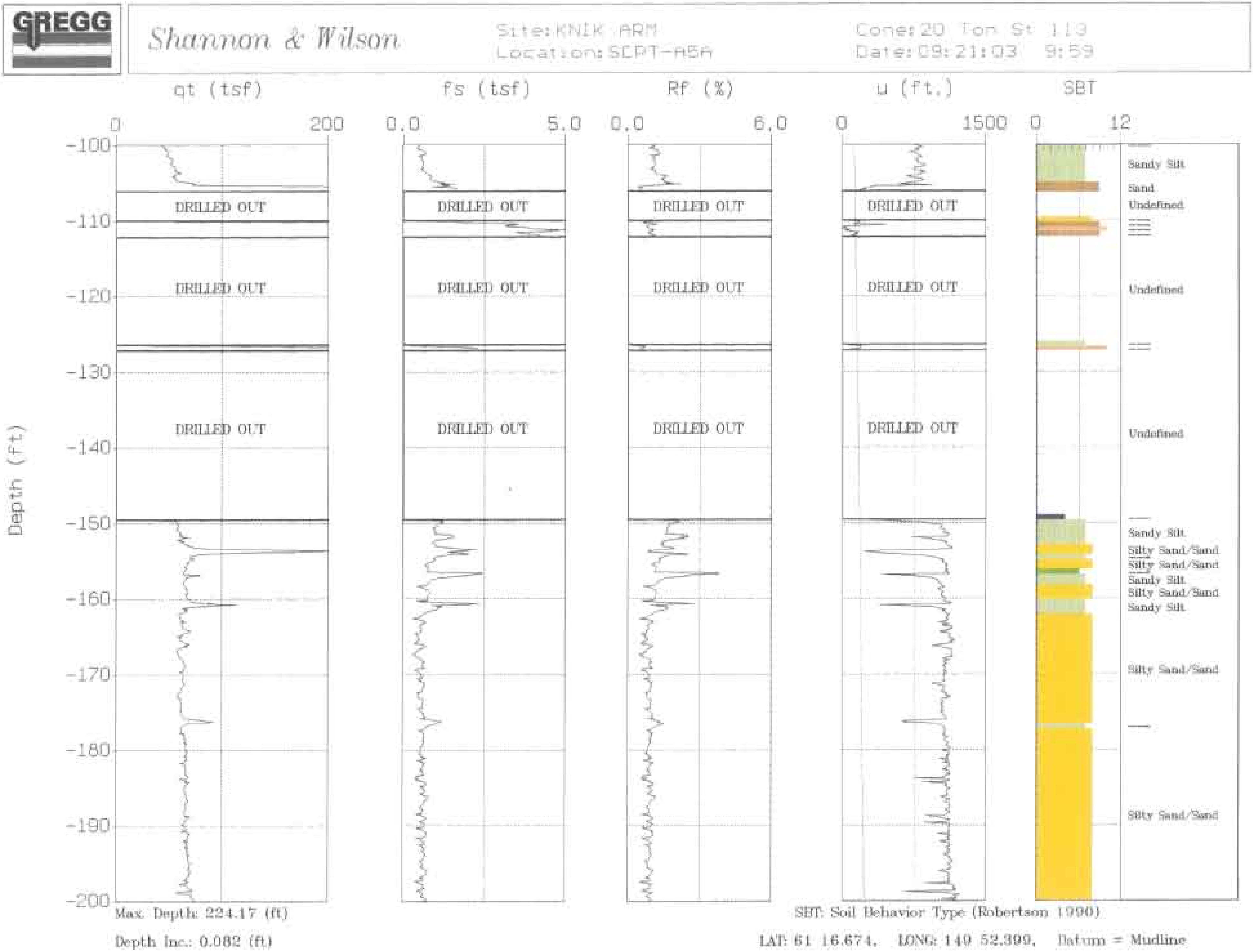
Max. Depth: 106.95 (ft)

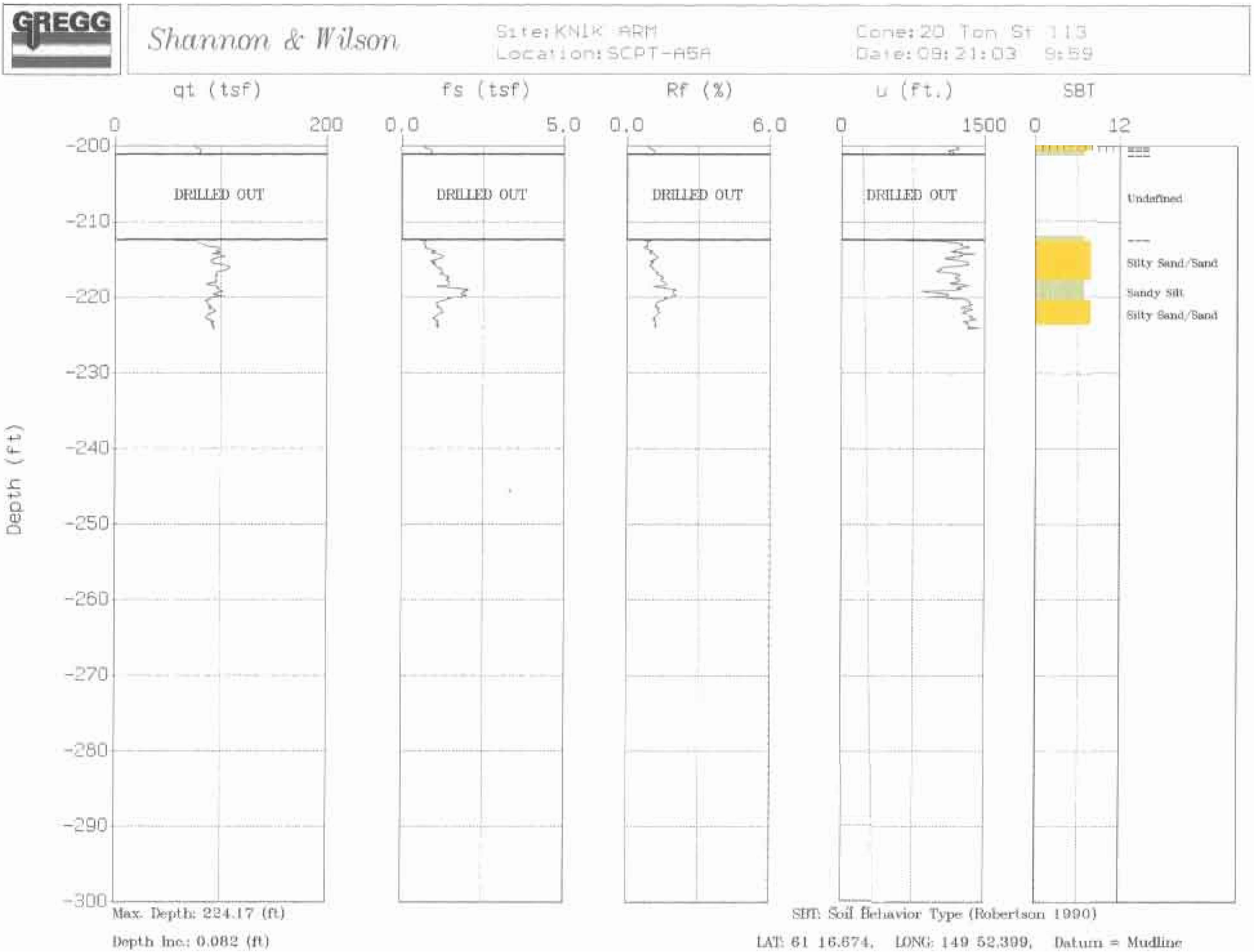
Depth Inc.: 0.082 (ft)

SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline


GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 9:59
qt (tsf)
fs (tsf)
Rf (%)
u (ft.)
SBT
Depth (ft)
CASED
DRILLED OUT
Undefined
Silt
Sandy Silt
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline


GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 9:59
qt (tsf)
fs (tsf)
Rf (%)
u (ft.)
SBT
Depth (ft)
DRILLED OUT
Sandy Silt
Sand
Undefined
Silty Sand/Sand
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

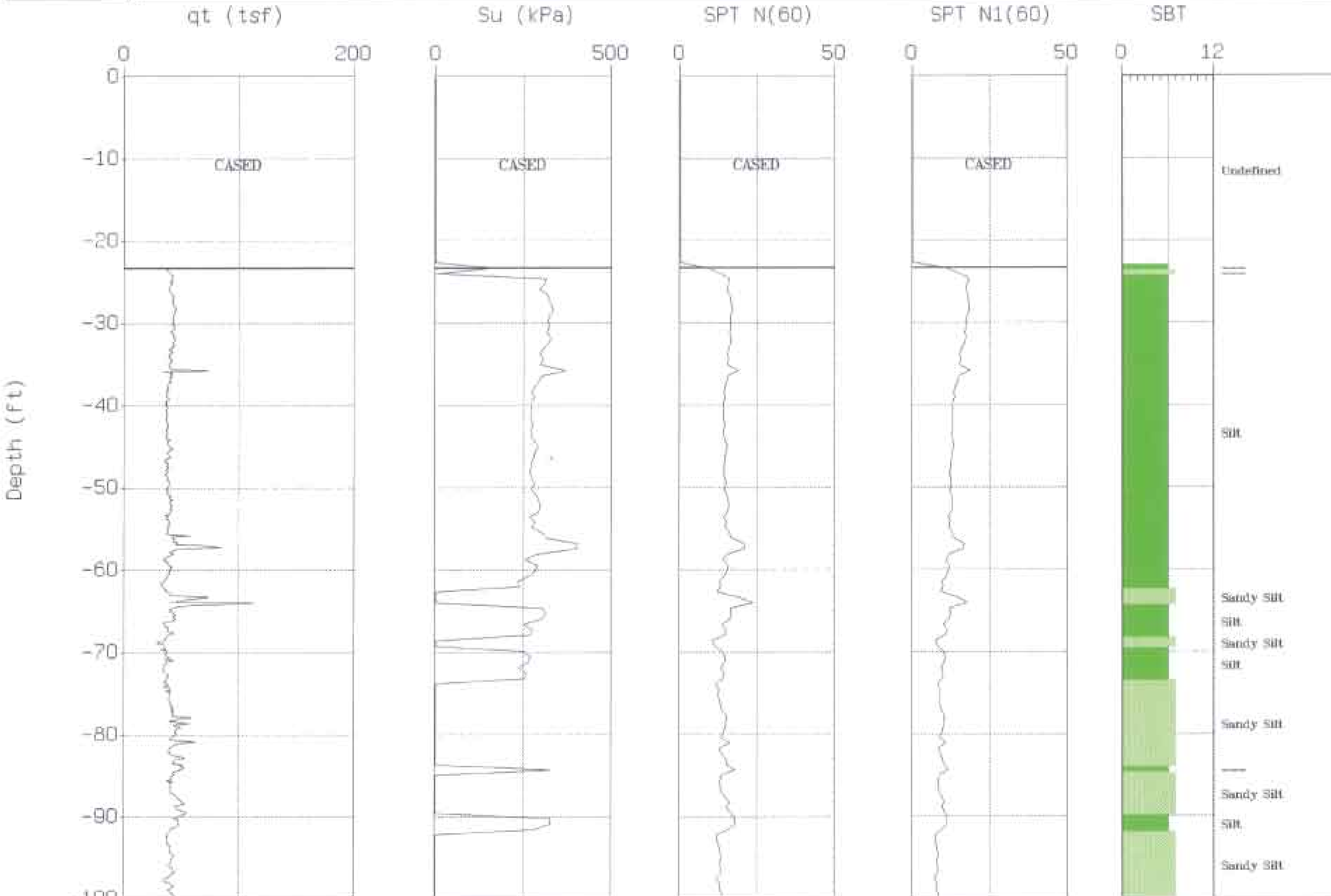

GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 9:59
qt (tsf)
fs (tsf)
Rf (%)
u (ft.)
SBT
Depth (ft)
DRILLED OUT
Undefined
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

CPT Plots of Estimated Values
of SPT N60 and $N_1$60 and Su (Nkt=12)

Shannon & Wilson

Site: KNIK ARM
Location: ADJACENT A-1

Cone: 20 Ton St 113
Date: 08:20:03 05:24

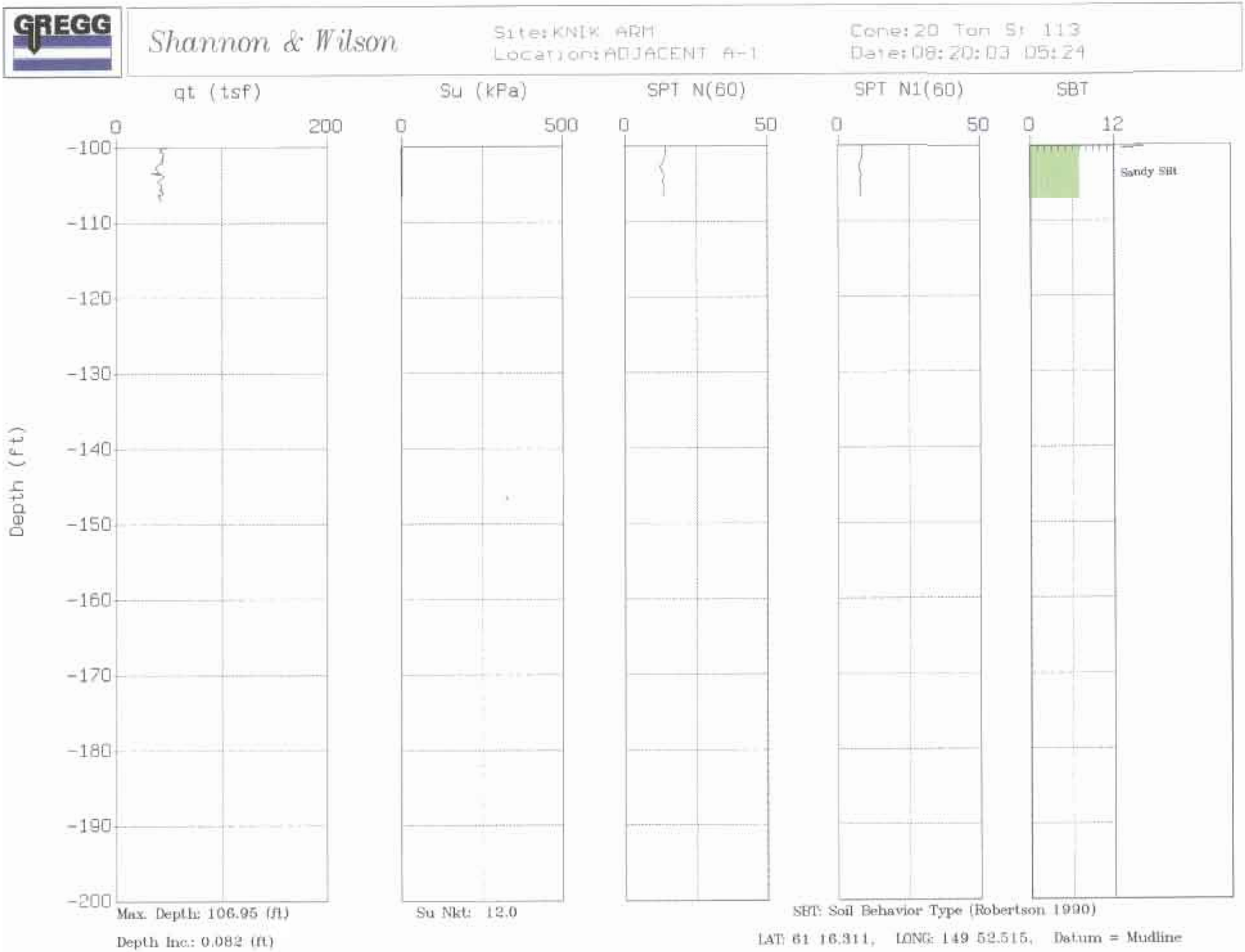


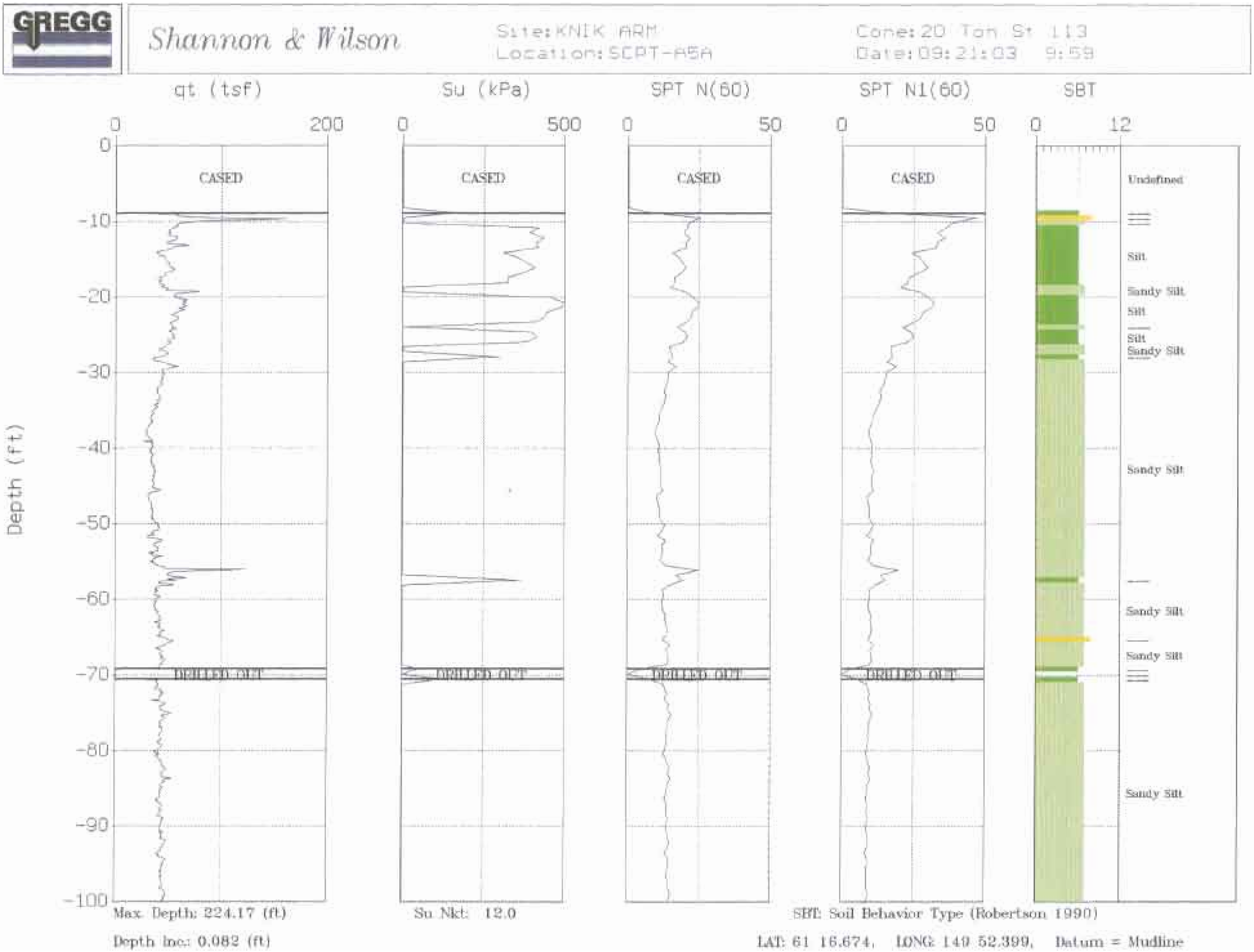
Max. Depth: 106.95 (ft)
Depth Inc.: 0.082 (ft)

Su Nkt: 12.0

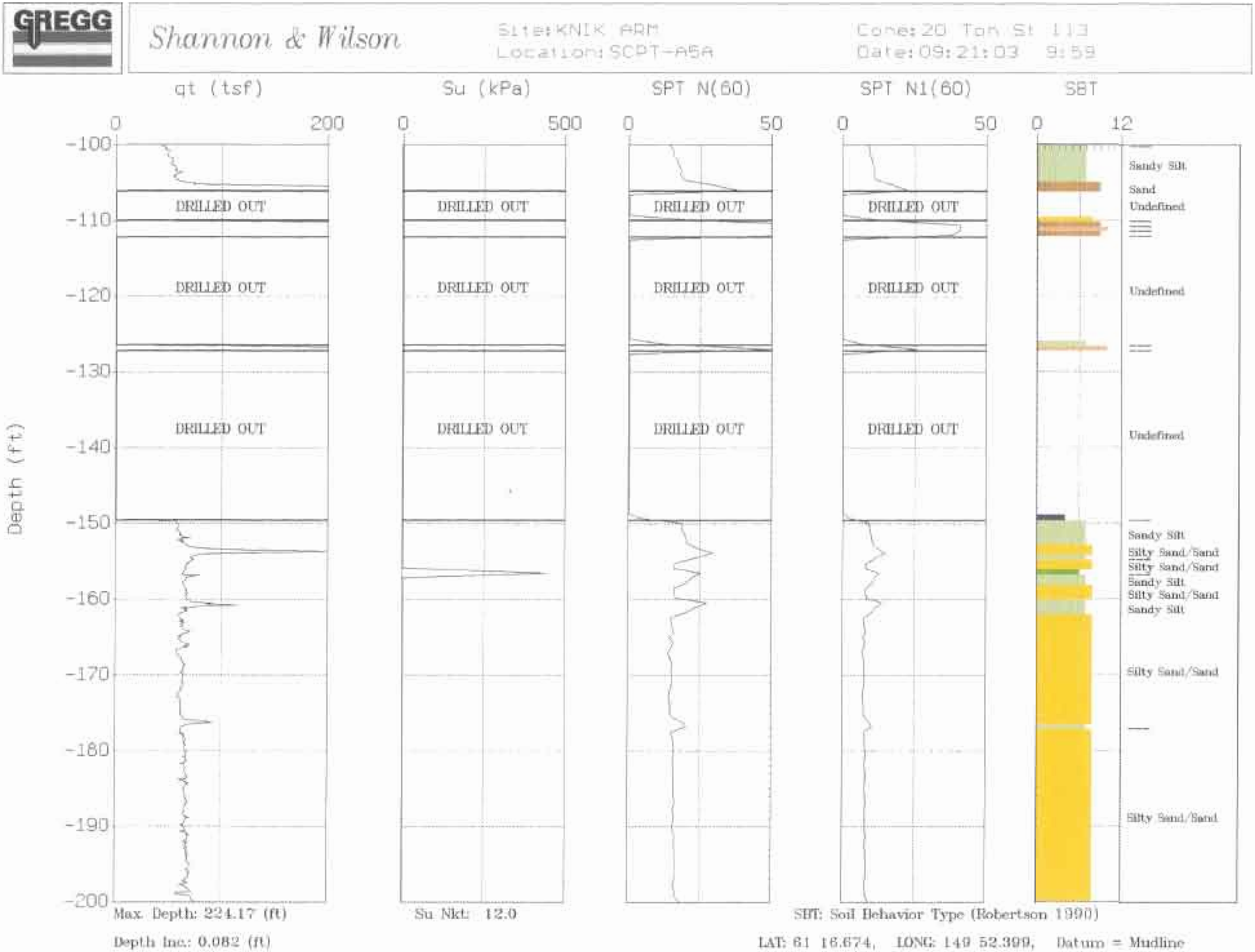
SBT: Soil Behavior Type (Robertson 1990)

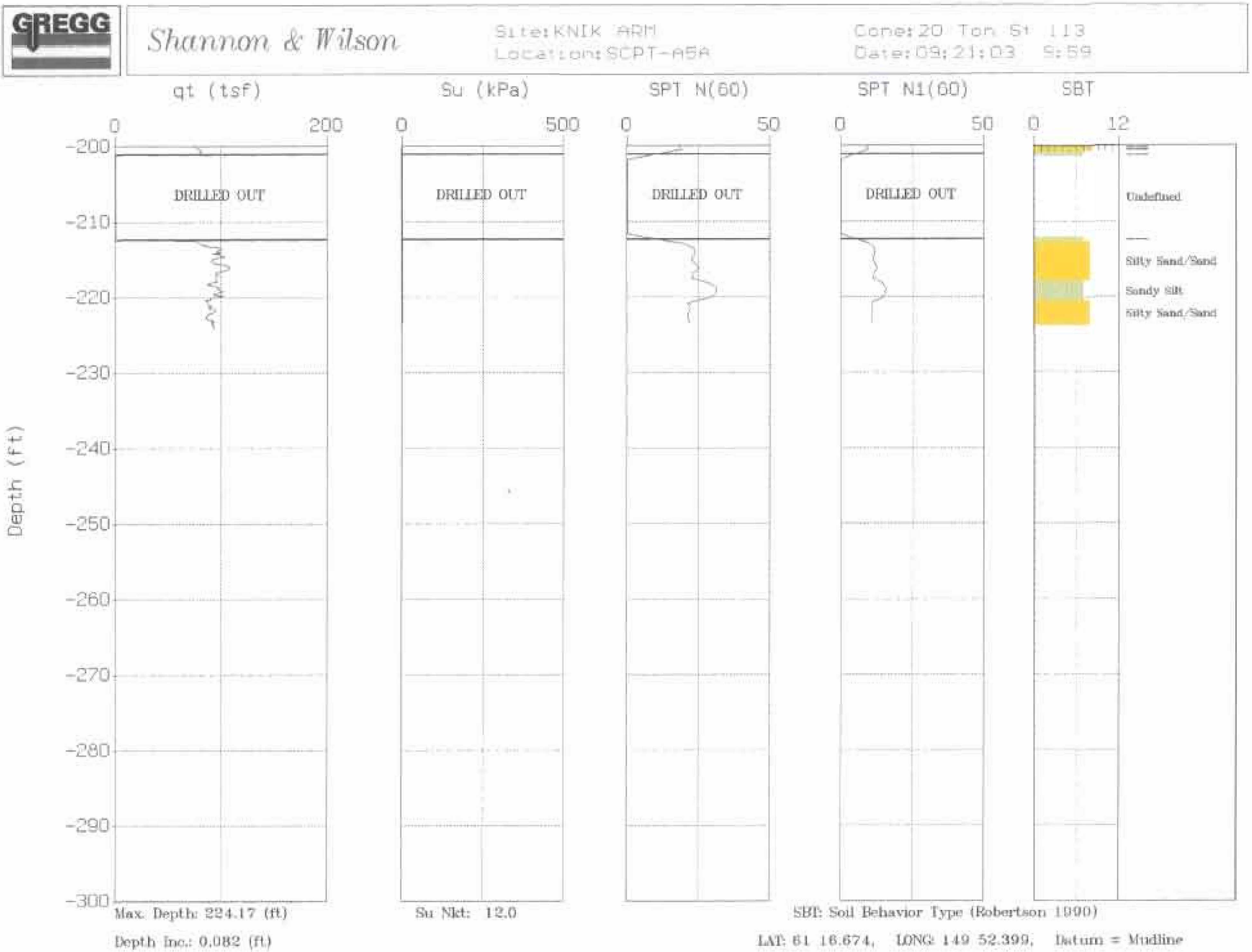
LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline


GREGG
Shannon & Wilson
Site:KNIK ARM
Location:ADJACENT A-1
Cone:20 Ton S: 113
Date:08:20:03 05:24
qt (tsf)
Su (kPa)
SPT N(60)
SPT N1(60)
SBT
Depth (ft)
Sandy Silt
Max. Depth: 106.95 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline



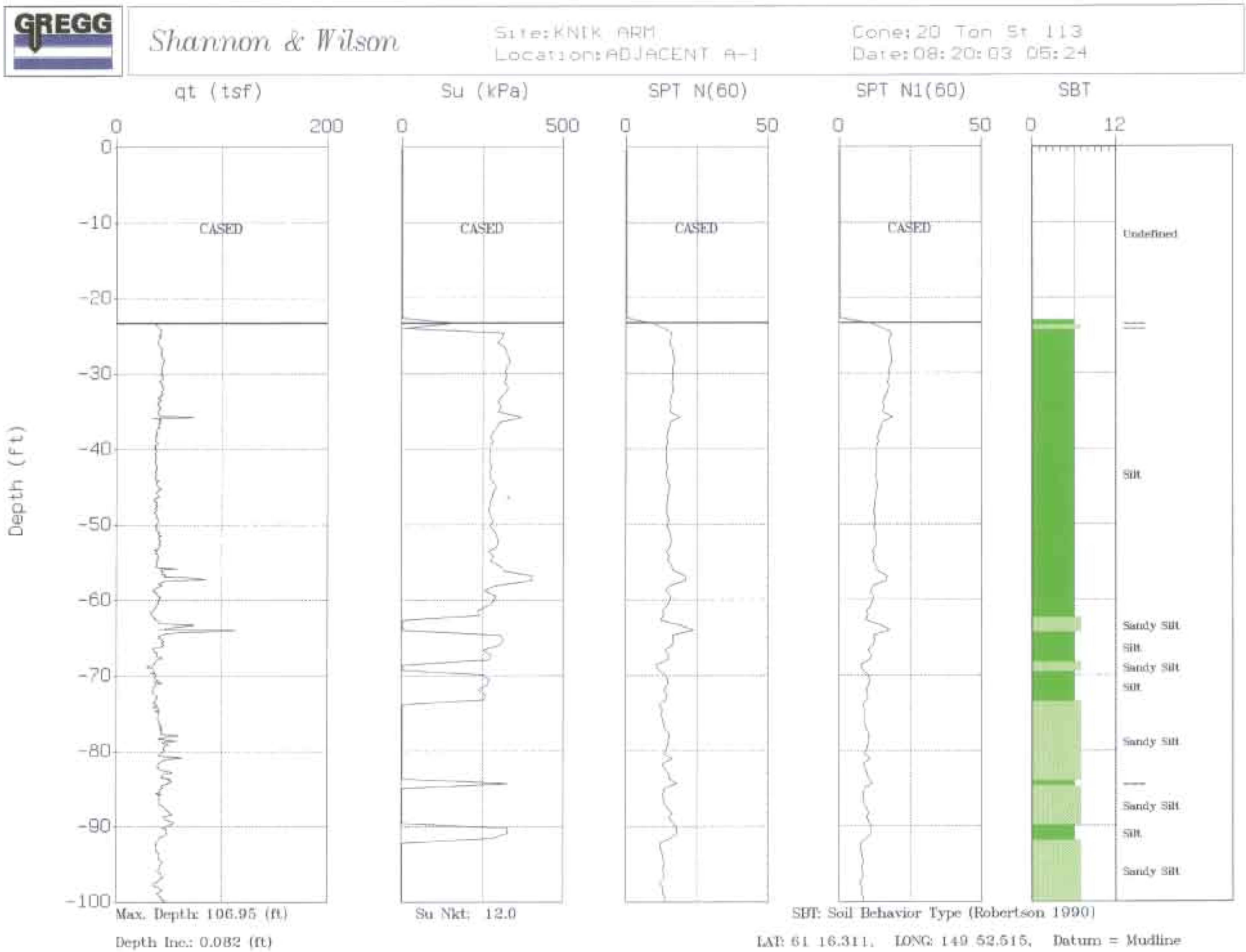
GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 9:59
qt (tsf)
Su (kPa)
SPT N(60)
SPT N1(60)
SBT
Depth (ft)
CASED
DRILLED OUT
Undefined
Silt
Sandy Silt
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

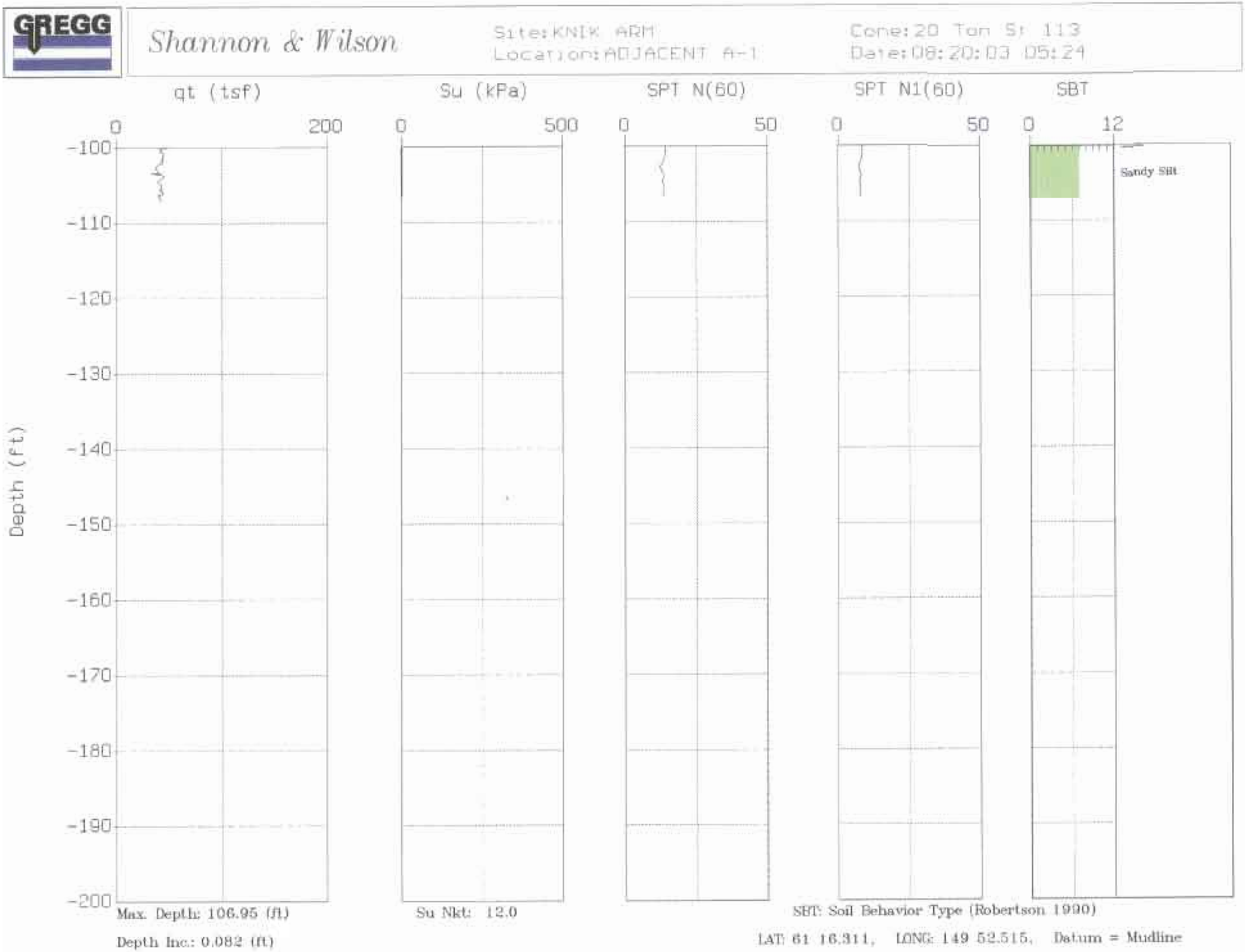

GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 9:59
qt (tsf)
Su (kPa)
SPT N(60)
SPT N1(60)
SBT
0
200
500
50
12
-100
-110
-120
-130
-140
-150
-160
-170
-180
-190
-200
Depth (ft)
DRILLED OUT
Sandy Silt
Sand
Undefined
Silty Sand/Sand
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

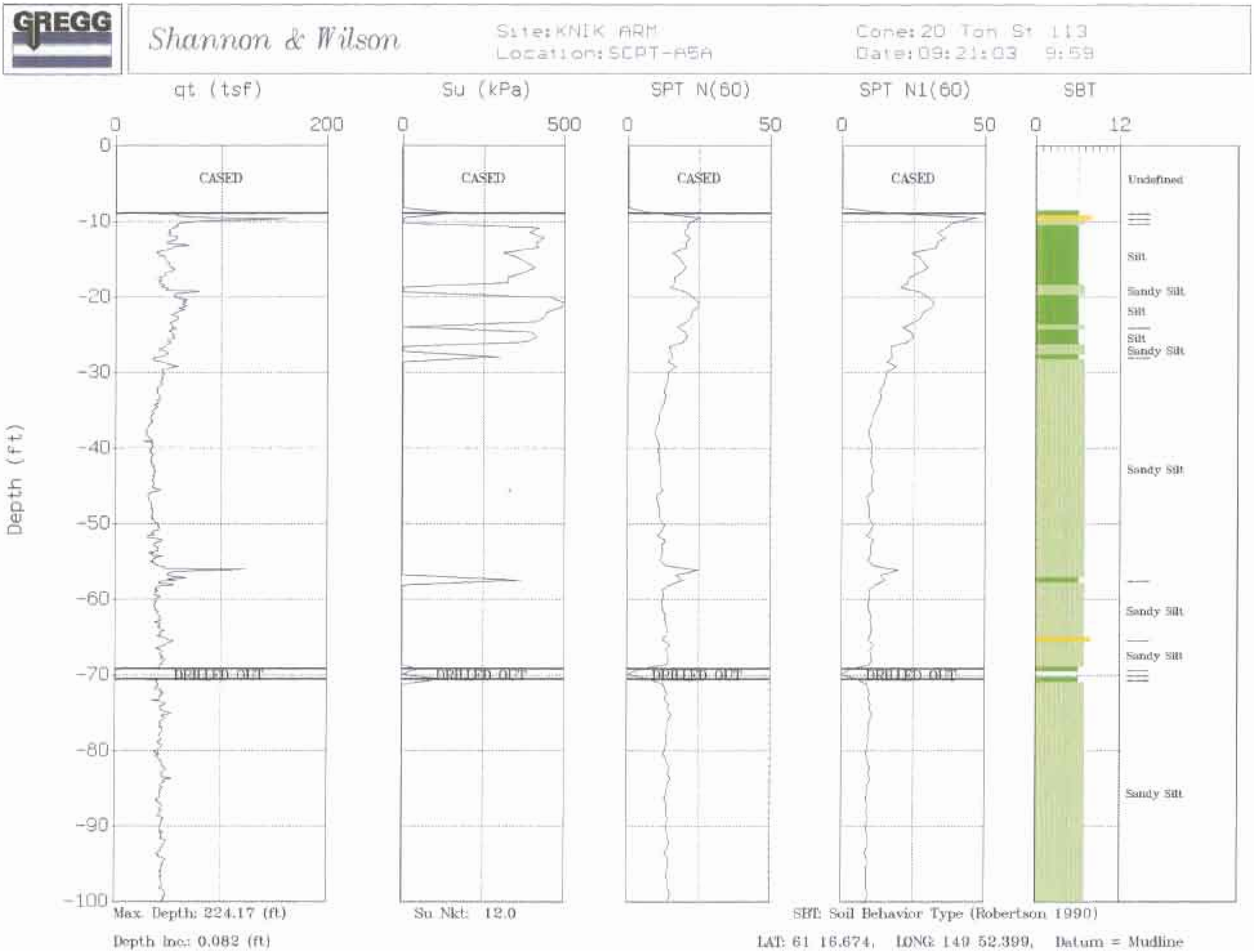

GREGG
Shannon & Wilson
Site:KNIK ARM
Location:SCPT-A5A
Cone:20 Ton S1 113
Date:09:21:03 9:59
qt (tsf)
Su (kPa)
SPT N(60)
SPT N1(60)
SBT
0
200
500
50
12
-200
-210
-220
-230
-240
-250
-260
-270
-280
-290
-300
Depth (ft)
DRILLED OUT
Undefined
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

CPT Plots of Estimated Values

of SPT N60 and $N_1$60 and Su (Nkt=15)


GREGG
Shannon & Wilson
Site: KNIK ARM
Location: ADJACENT A-1
Cone: 20 Ton St 113
Date: 08:20:03 05:24
qt (tsf)
Su (kPa)
SPT N(60)
SPT N1(60)
SBT
Depth (ft)
CASED
Undefined
Silt
Sandy Silt
Max. Depth: 106.95 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline


GREGG
Shannon & Wilson
Site: KNIK ARM
Location: ADJACENT A-1
Cone: 20 Ton S: 113
Date: 08:20:03 05:24
qt (tsf)
Su (kPa)
SPT N(60)
SPT N1(60)
SBT
Depth (ft)
Sandy Silt
Max. Depth: 106.95 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline

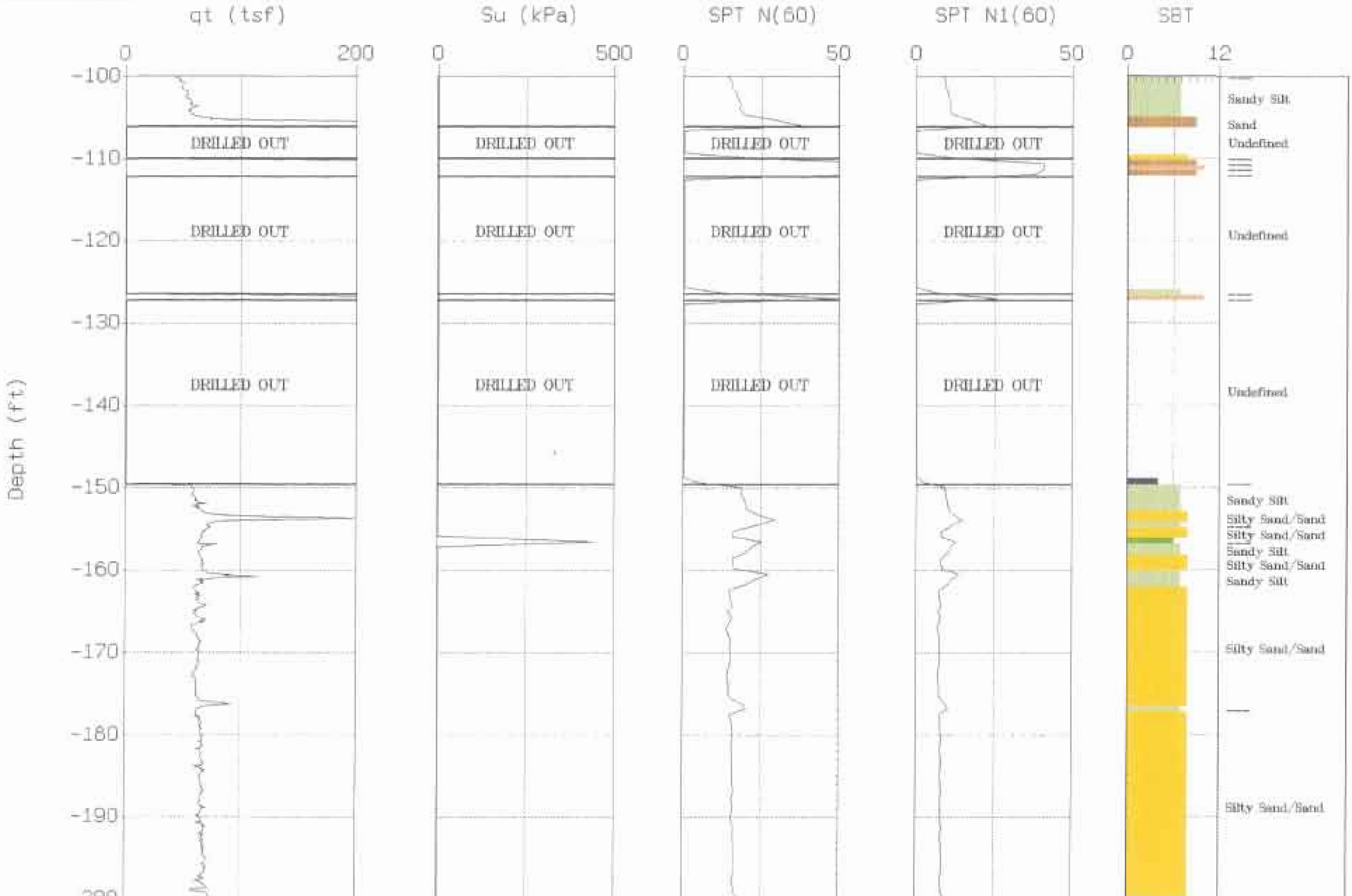

GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 9:59
qt (tsf)
Su (kPa)
SPT N(60)
SPT N1(60)
SBT
Depth (ft)
CASED
DRILLED OUT
Undefined
Silt
Sandy Silt
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

GREGG

Shannon & Wilson

Site:KNIK ARM
Location:SCPT-A5A

Cone:20 Ton S: 113
Date:09:21:03 9:59



Max. Depth: 224.17 (ft)

Depth Inc.: 0.082 (ft)

Su Nkt: 12.0

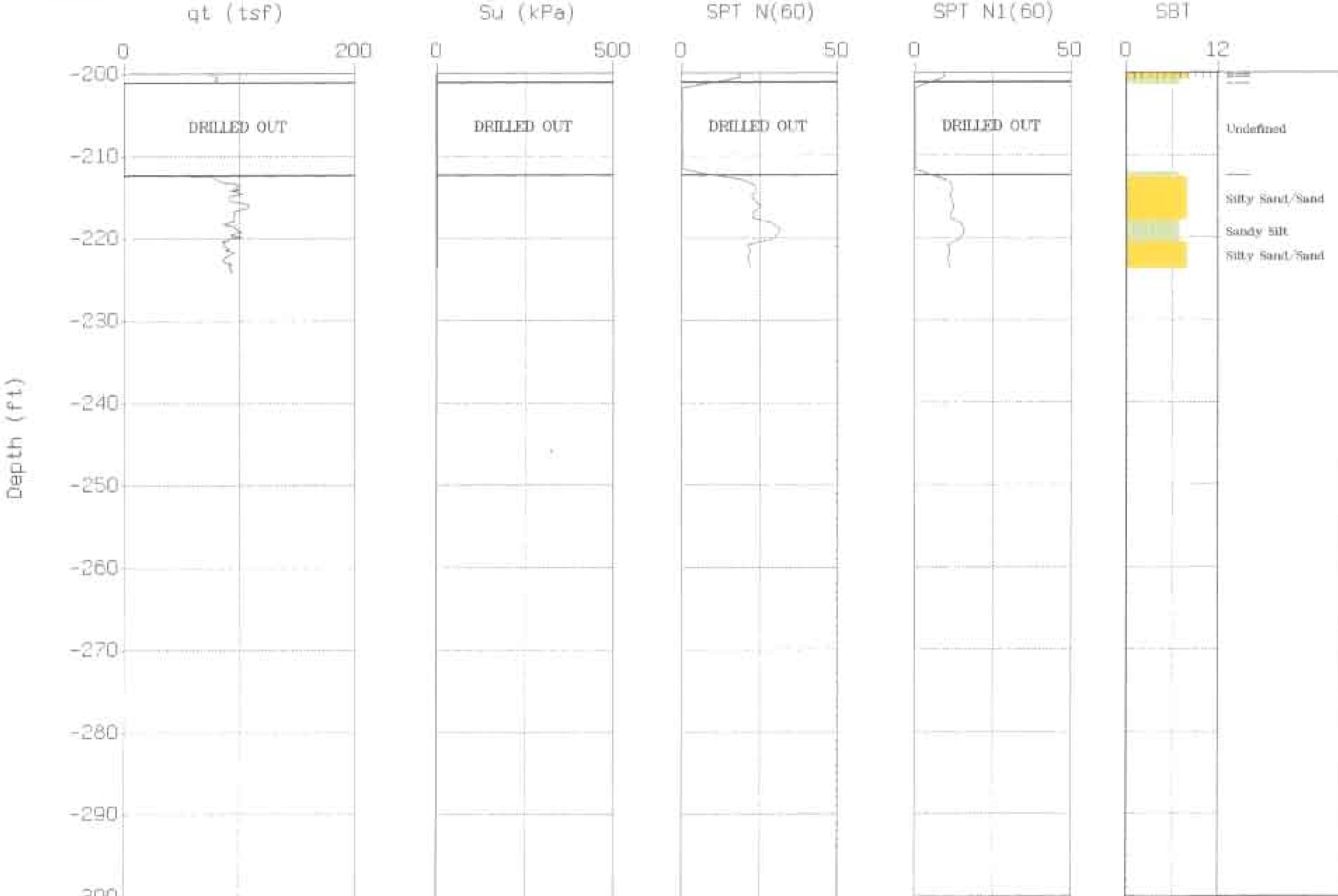
SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

Shannon & Wilson

Site: KNIK ARM
Location: SCPT-A5A

Cone: 20 Ton St 113
Date: 09:21:03 9:59



Max. Depth: 224.17 (ft)

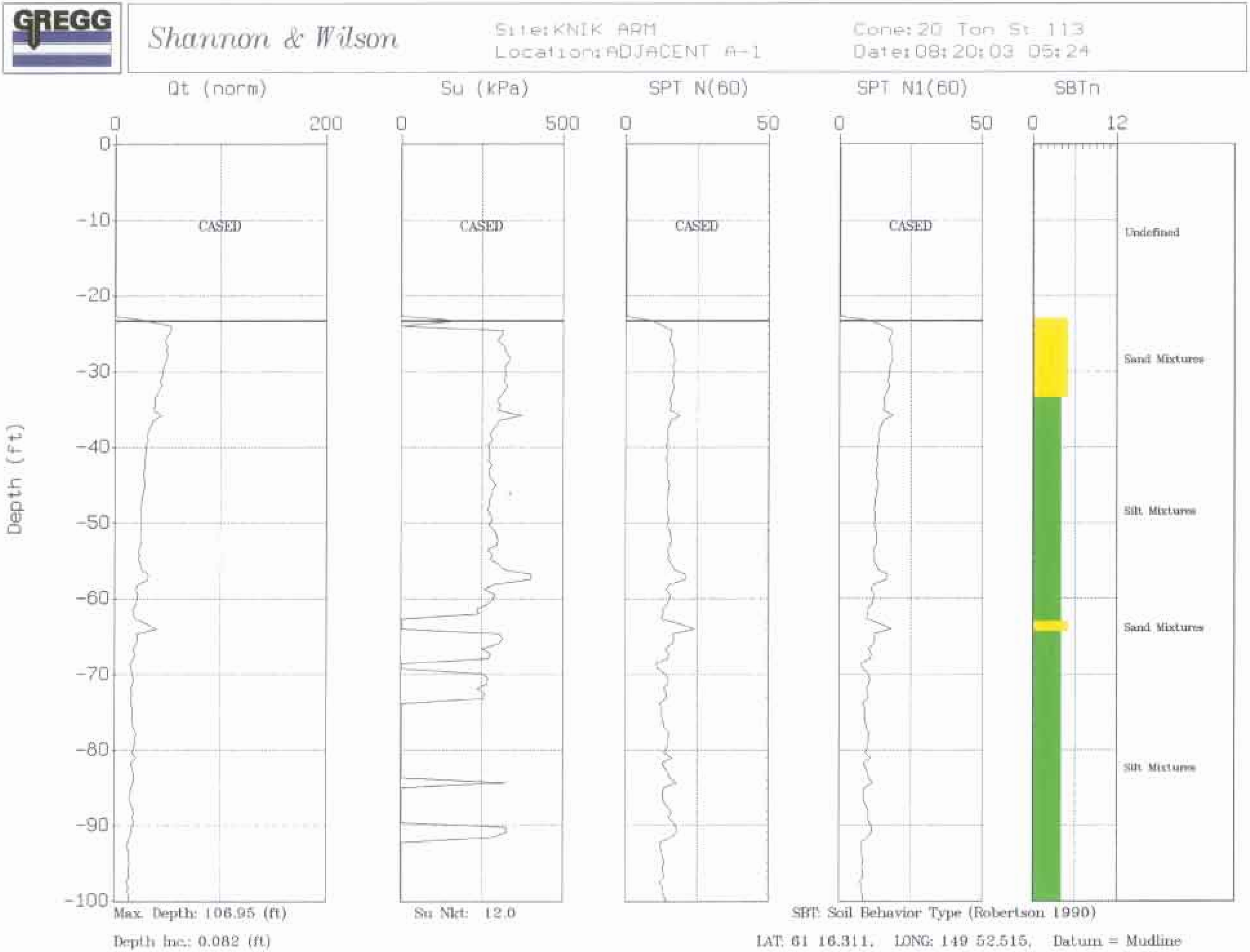
Depth Inc.: 0.082 (ft)

Su Nkt: 15.0

SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

CPT Plots Based on
Normalized Soil Behavior Type

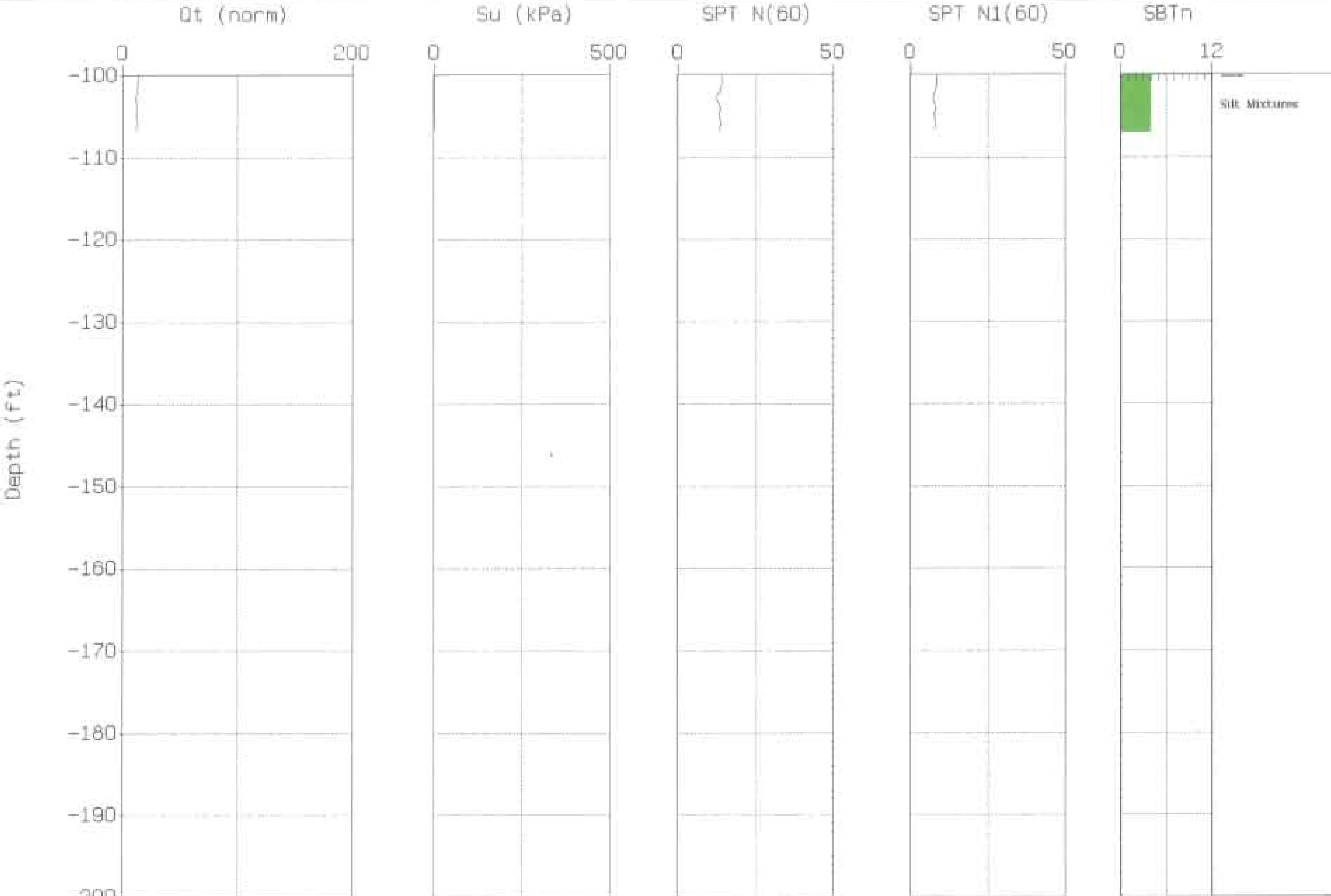

GREGG
Shannon & Wilson
Site:KNIK ARM
Location:ADJACENT A-1
Cone:20 Ton St 113
Date:08:20:03 05:24
Qt (norm)
Su (kPa)
SPT N(60)
SPT N1(60)
SBTn
Depth (ft)
CASED
CASED
CASED
CASED
Undefined
Sand Mixtures
Silt Mixtures
Sand Mixtures
Silt Mixtures
Max. Depth: 106.95 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline

GREGG

Shannon & Wilson

Site:KNIK ARM
Location:ADJACENT A-1

Cone:20 Ton S: 113
Date:08:20:03 05:24

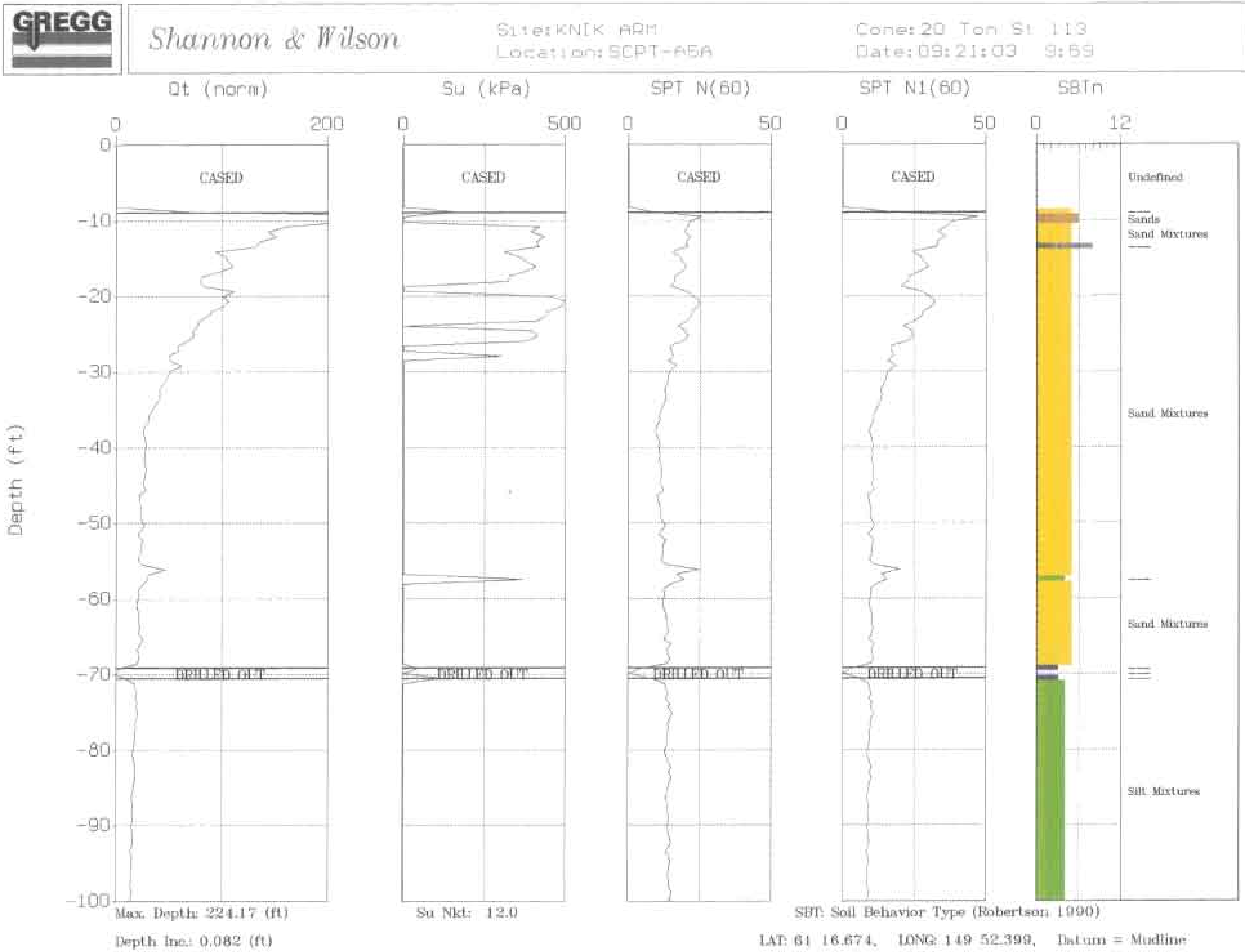


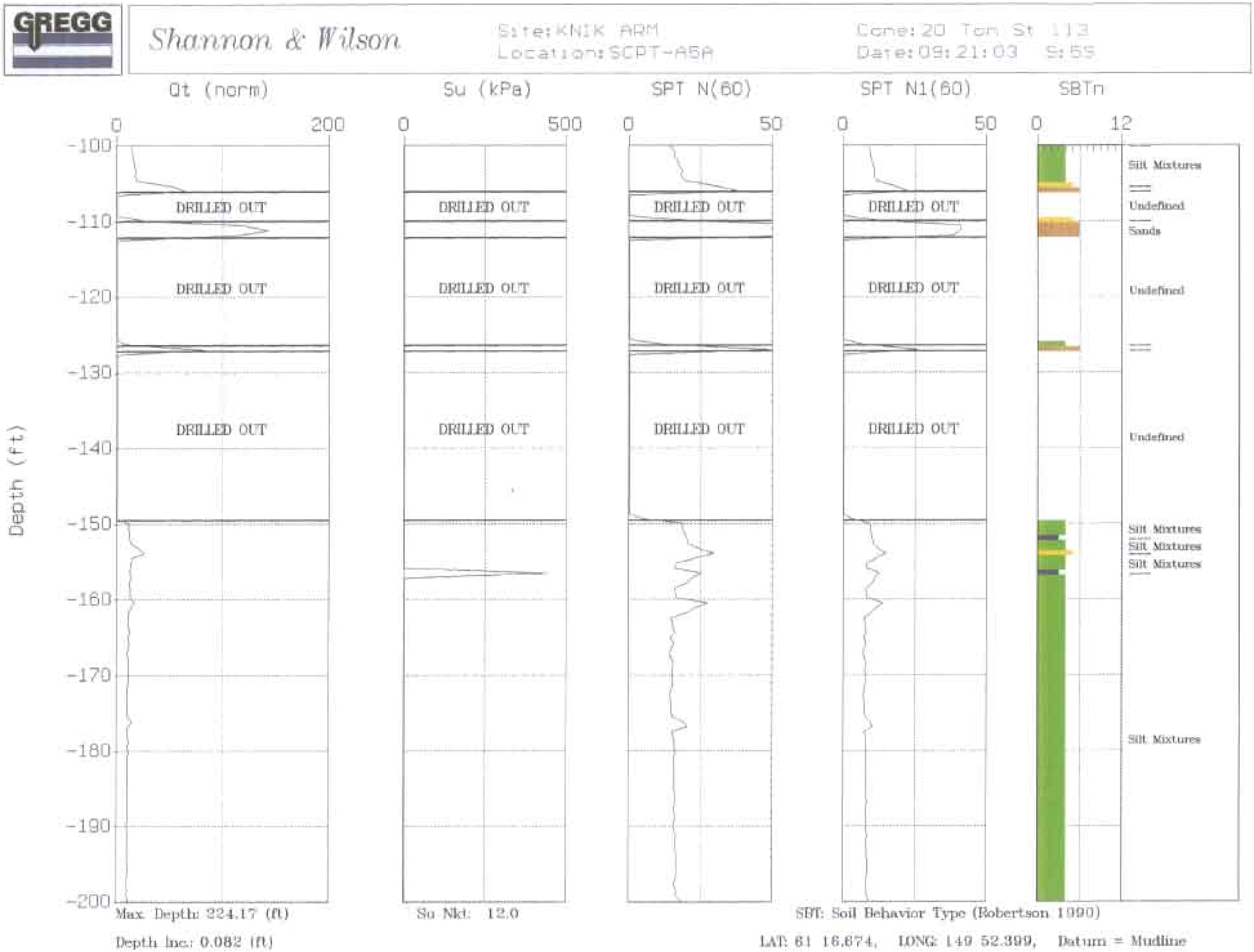
Max. Depth: 106.95 (ft)
Depth Inc.: 0.082 (ft)

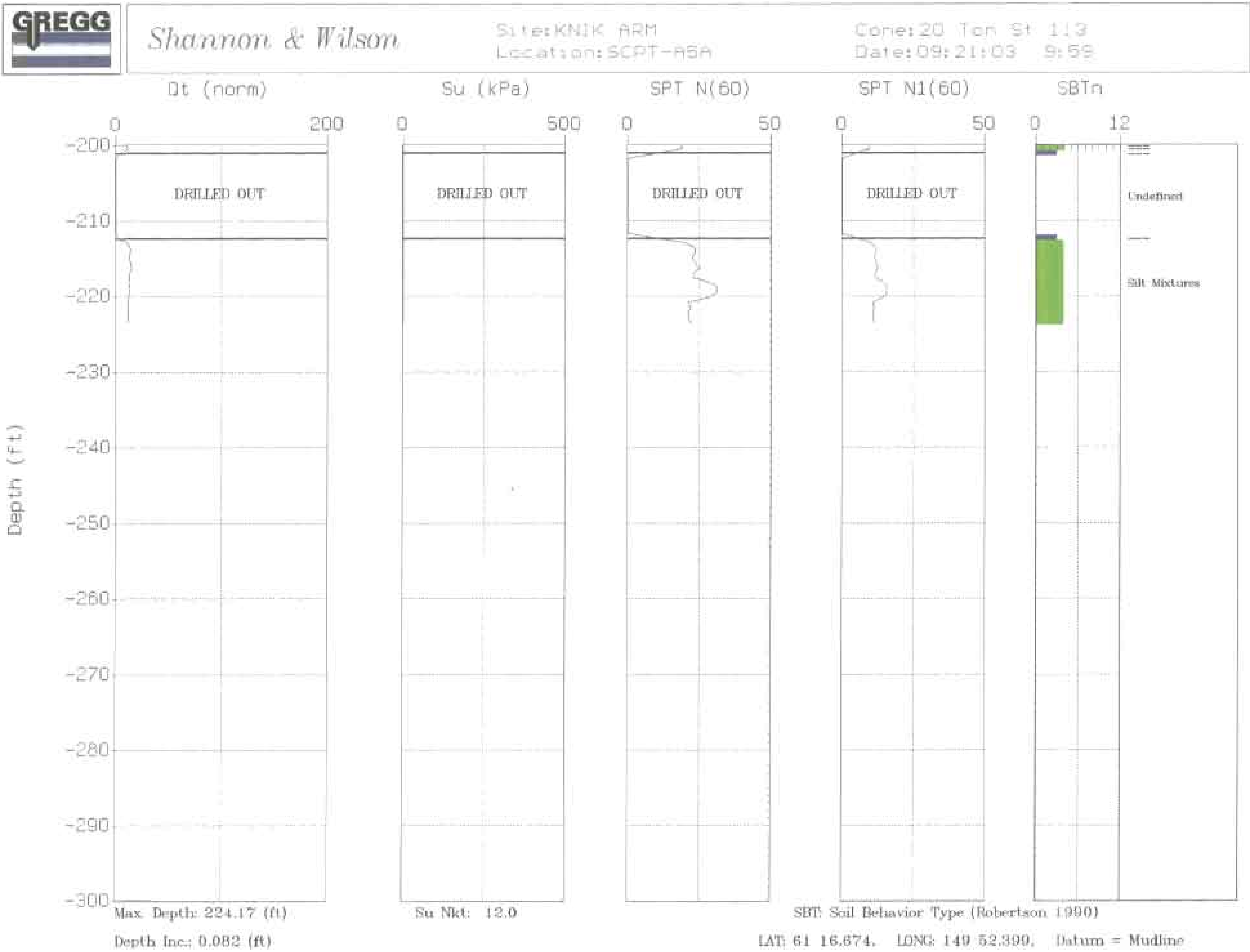
Su Nkt: 12.0

SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline


GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 9:59
Qt (norm)
Su (kPa)
SPT N(60)
SPT N1(60)
SBTn
CASED
DRILLED OUT
Depth (ft)
Undefined
Sands
Sand Mixtures
Sand Mixtures
Sand Mixtures
Silt Mixtures
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

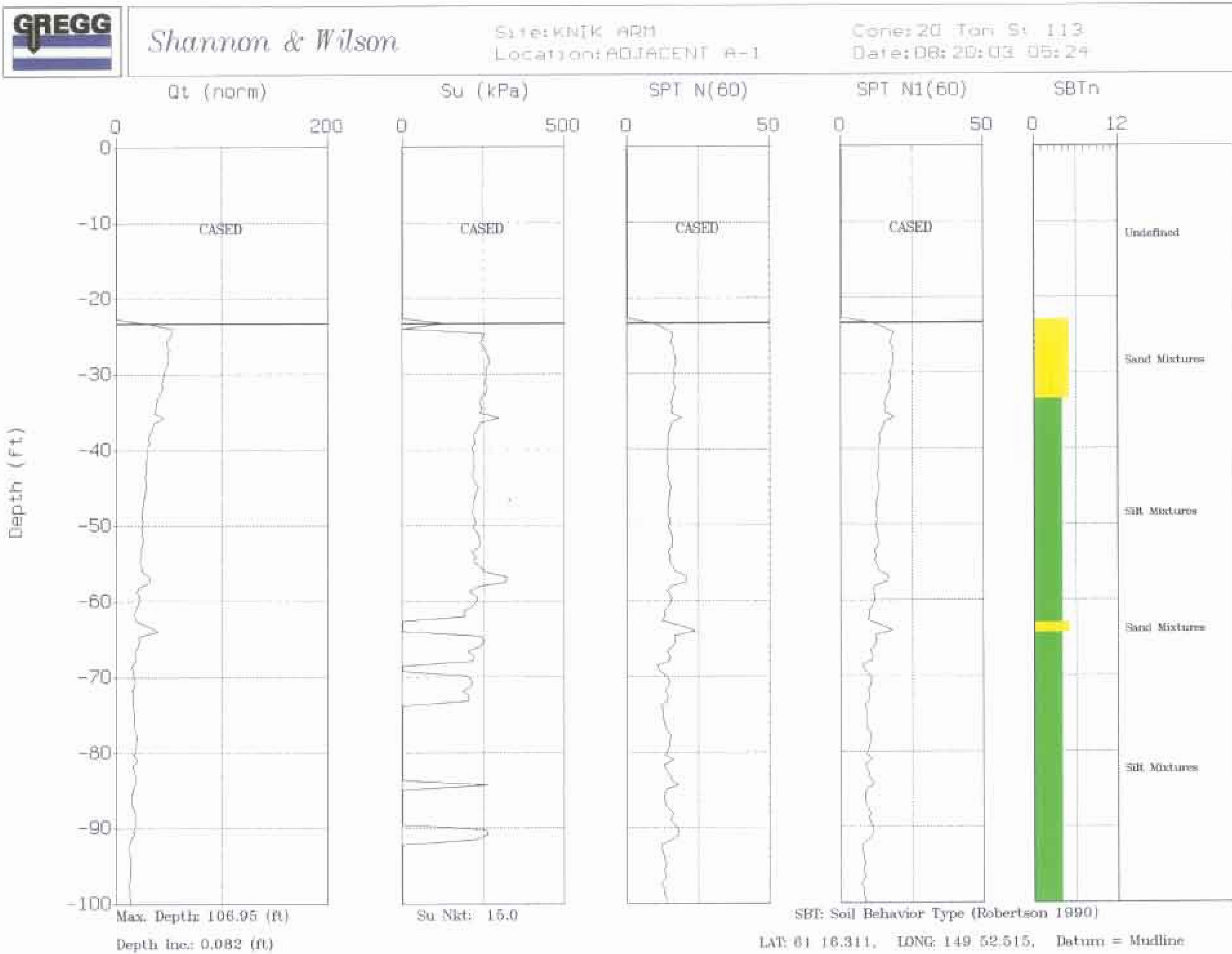

GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 S:59
Qt (norm)
Su (kPa)
SPT N(60)
SPT N1(60)
SBTn
Depth (ft)
DRILLED OUT
Silt Mixtures
Undefined
Sands
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline



GREGG
Shannon & Wilson
Site:KNIK ARM
Location:SCPT-A5A
Cone:20 Ton St 113
Date:09:21:03 9:59
Qt (norm)
Su (kPa)
SPT N(60)
SPT N1(60)
SBTn
0
200
500
50
12
-200
-210
-220
-230
-240
-250
-260
-270
-280
-290
-300
Depth (ft)
DRILLED OUT
Undefined
Silt Mixtures
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 12.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

CPT Plots of Estimated Values

of SPT N60 and $N_1$60 and Su (Nkt=12)

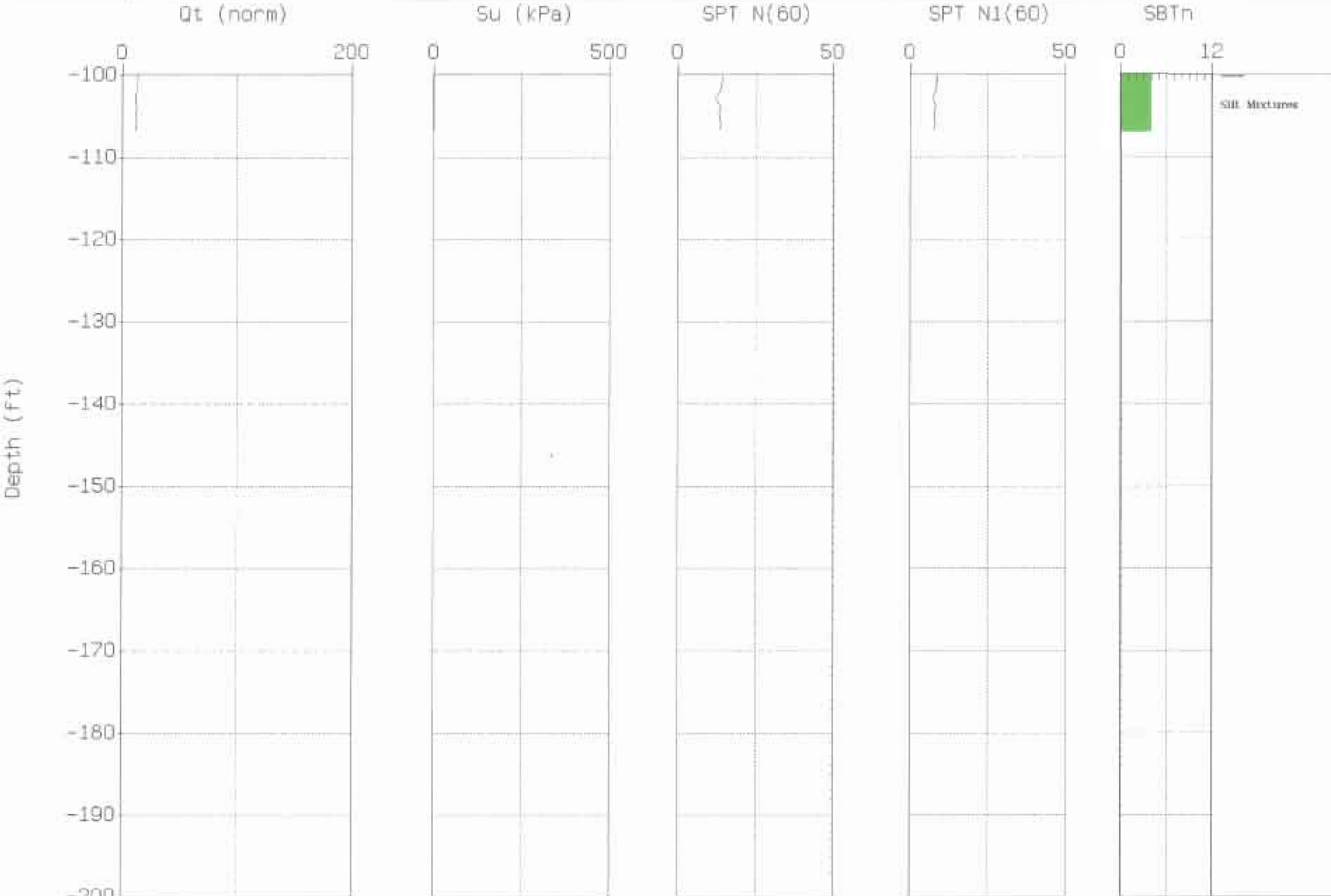

GREGG
Shannon & Wilson
Site:KNIK ARM
Location:ADJACENT A-1
Cone:20 Ton S: 113
Date:08:20:03 05:24
Qt (norm)
Su (kPa)
SPT N(60)
SPT N1(60)
SBTn
Depth (ft)
CASED
CASED
CASED
CASED
Undefined
Sand Mixtures
Silt Mixtures
Sand Mixtures
Silt Mixtures
Max. Depth: 106.95 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 15.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline

GREGG

Shannon & Wilson

Site:KNIK ARM
Location:ADJACENT A-1

Cone:20 Ton S: 113
Date:08:20:03 05:24

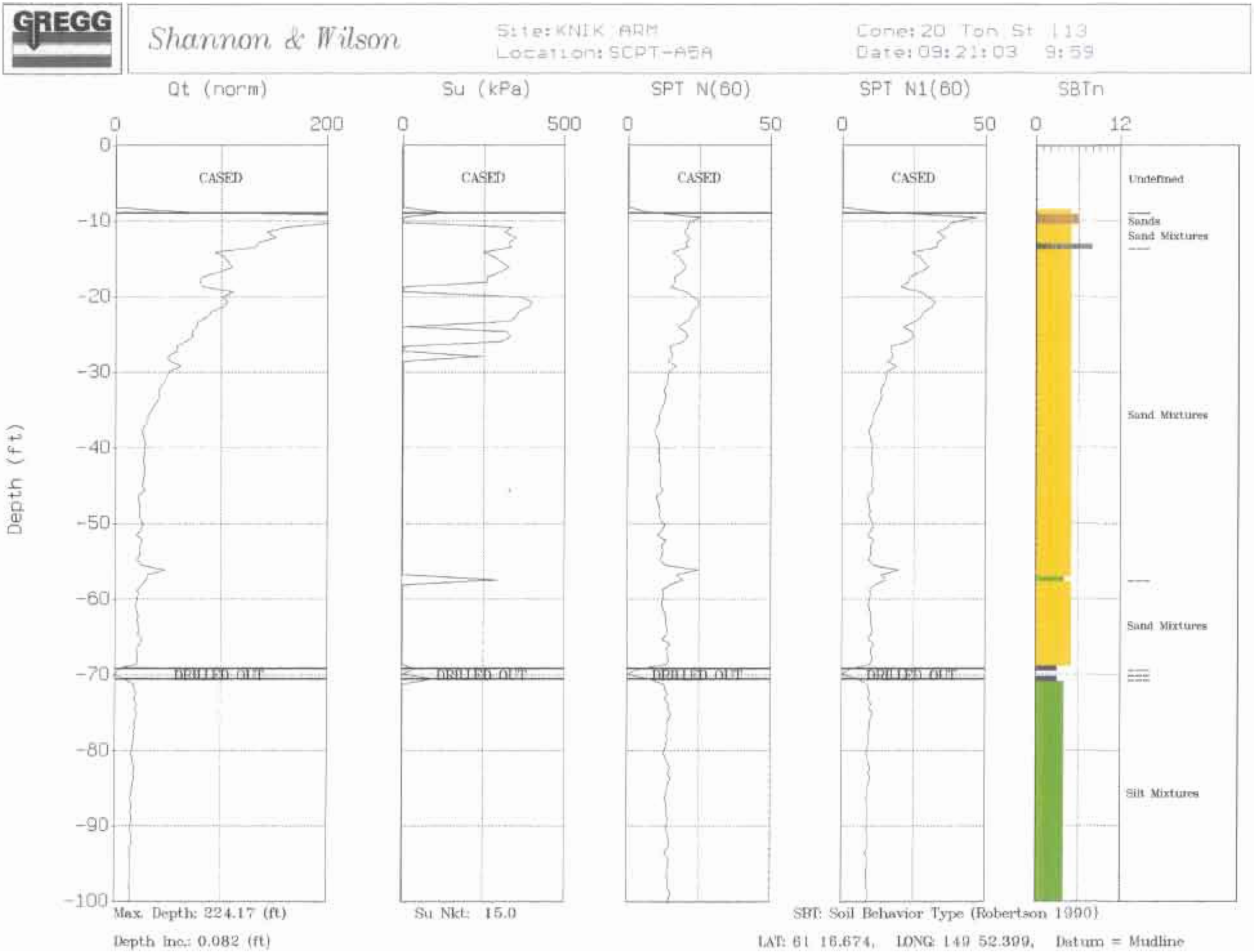


Max. Depth: 106.95 (ft)
Depth Inc.: 0.082 (ft)

Su Nkt: 15.0

SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline

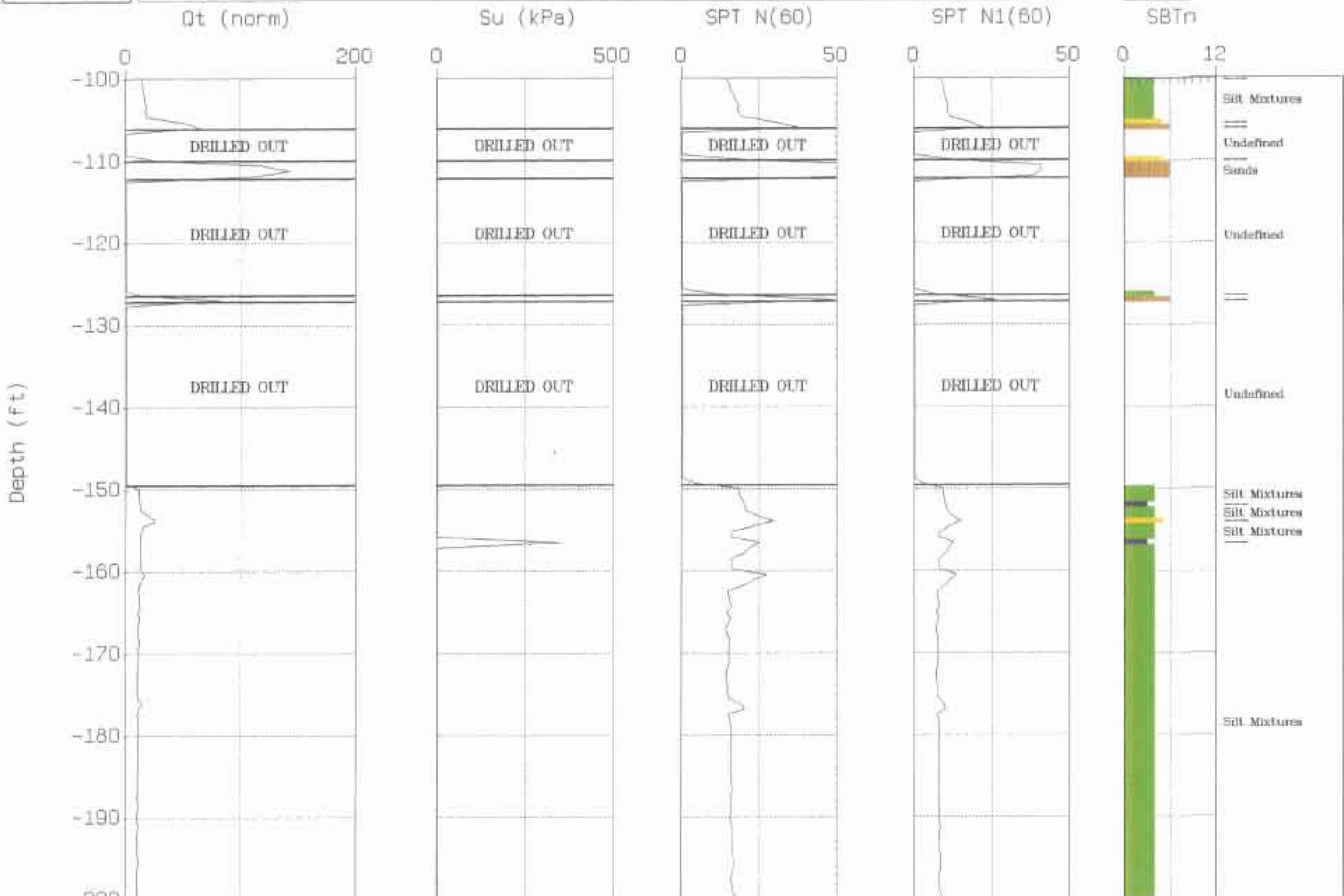

GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 9:59
Qt (norm)
Su (kPa)
SPT N(60)
SPT N1(60)
SBTn
0
200
500
50
12
Depth (ft)
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
CASED
DRILLED OUT
Undefined
Sands
Sand Mixtures
Sand Mixtures
Sand Mixtures
Silt Mixtures
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 15.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

GREGG

Shannon & Wilson

Site: KNIK ARM
Location: SCPT-A5A

Cone: 20 Ton St 113
Date: 09:21:03 9:58

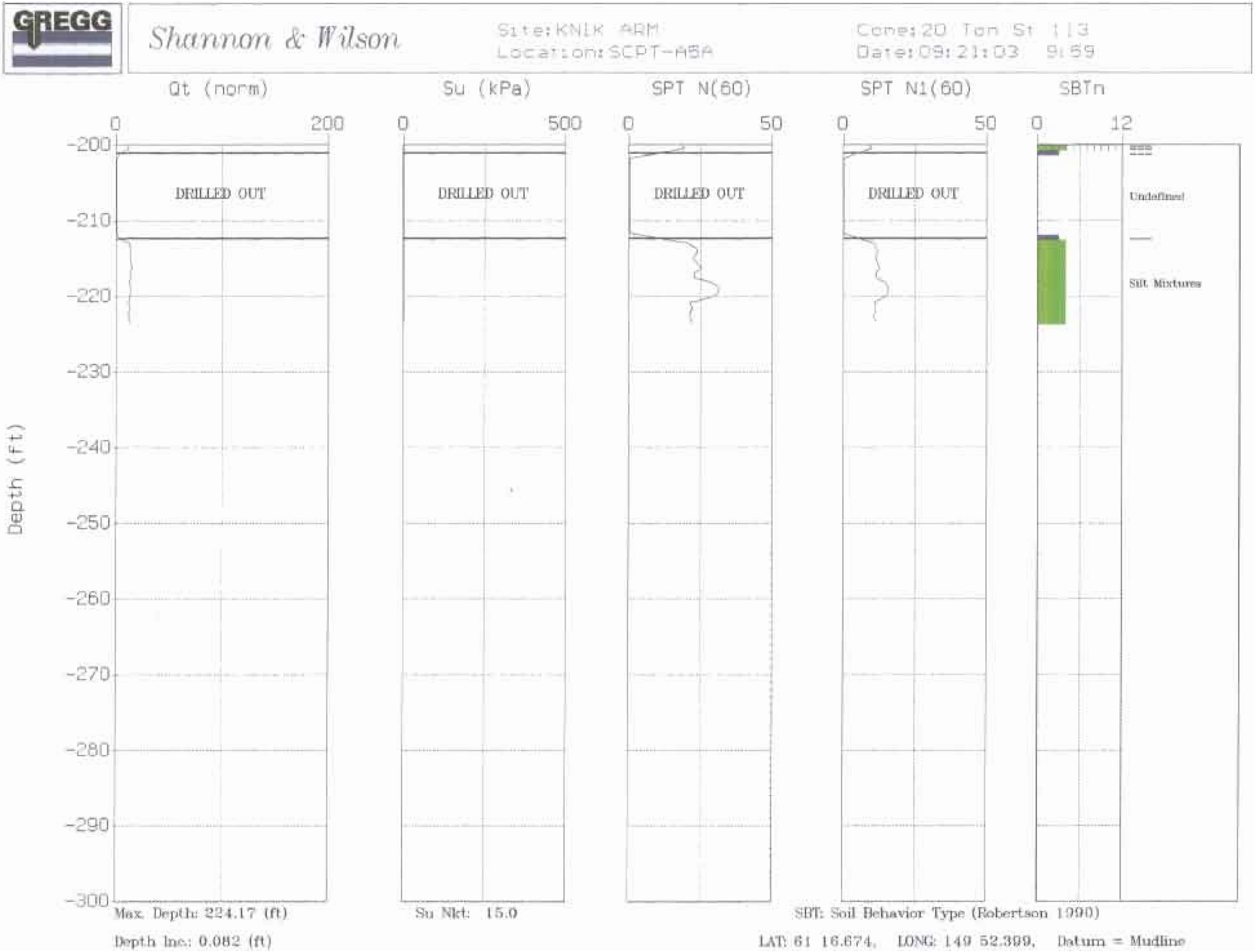


Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)

Su Nkt: 15.0

SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline



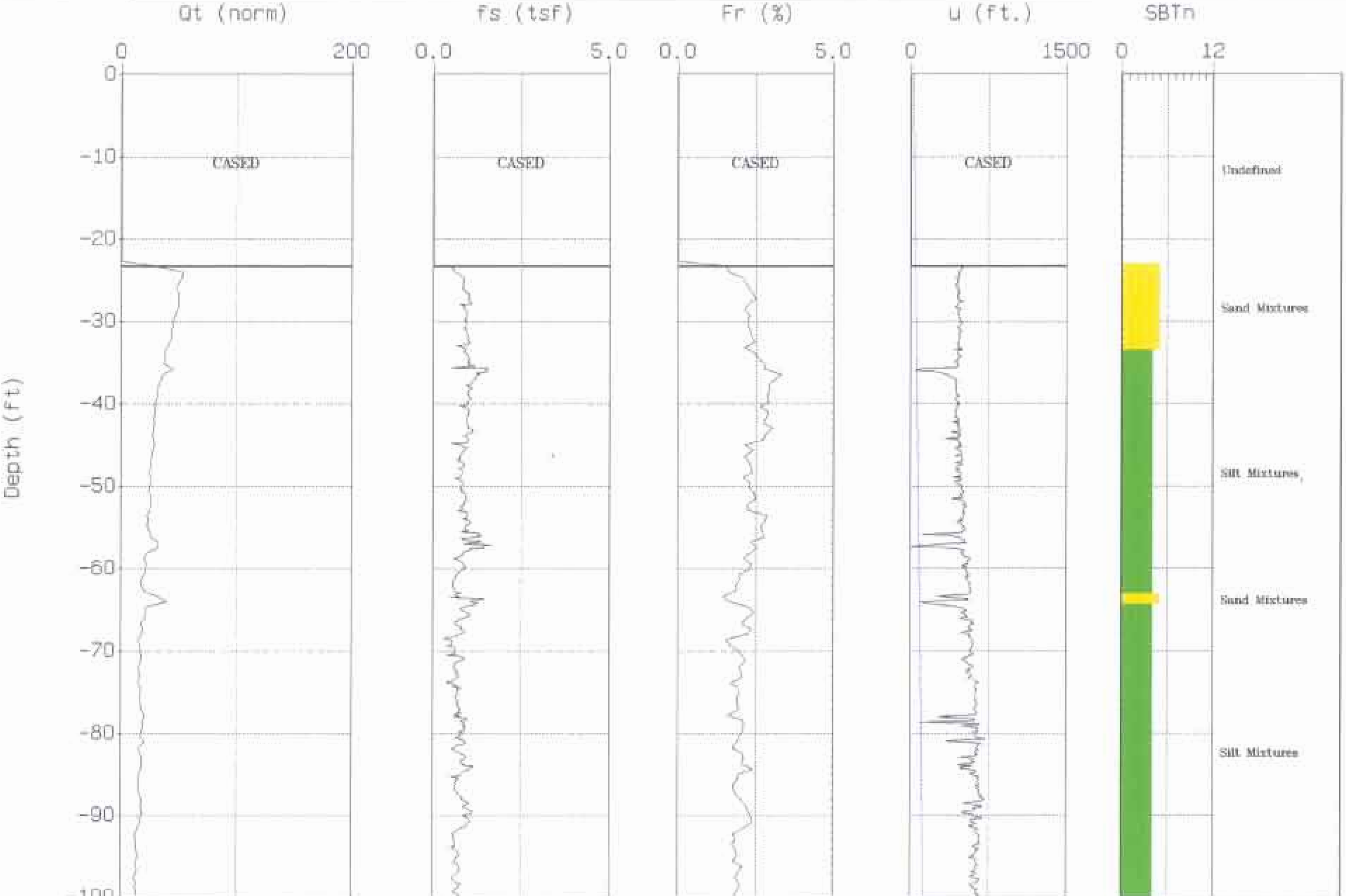
GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 9:59
Qt (norm)
Su (kPa)
SPT N(60)
SPT N1(60)
SBTn
Depth (ft)
DRILLED OUT
Undefined
Silt Mixtures
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
Su Nkt: 15.0
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

CPT Plots of Estimated Values
of SPT N60 and $N_1$60 and Su (Nkt=15)

Shannon & Wilson

Site: KNIK ARM
Location: ADJACENT A-1

Cone: 20 Ton St 113
Date: 08:20:03 05:24



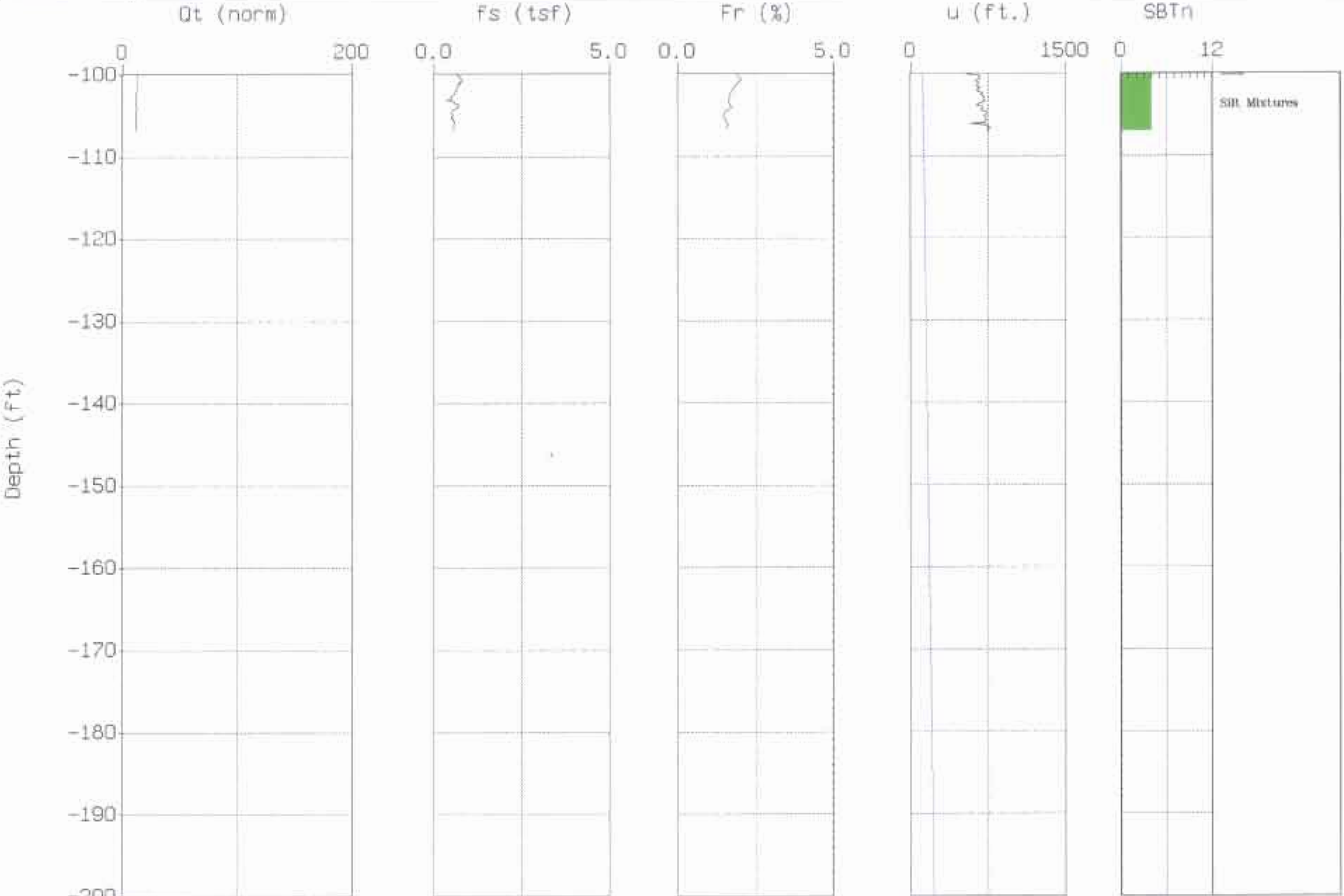
Max. Depth: 106.95 (ft)

Depth Inc.: 0.082 (ft)

SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline

GREGG	Site: KNIK ARM	Cone: 20 Ton St 113
Shannon & Wilson	Location: ADJACENT A-1	Date: 08:20:03 05:24

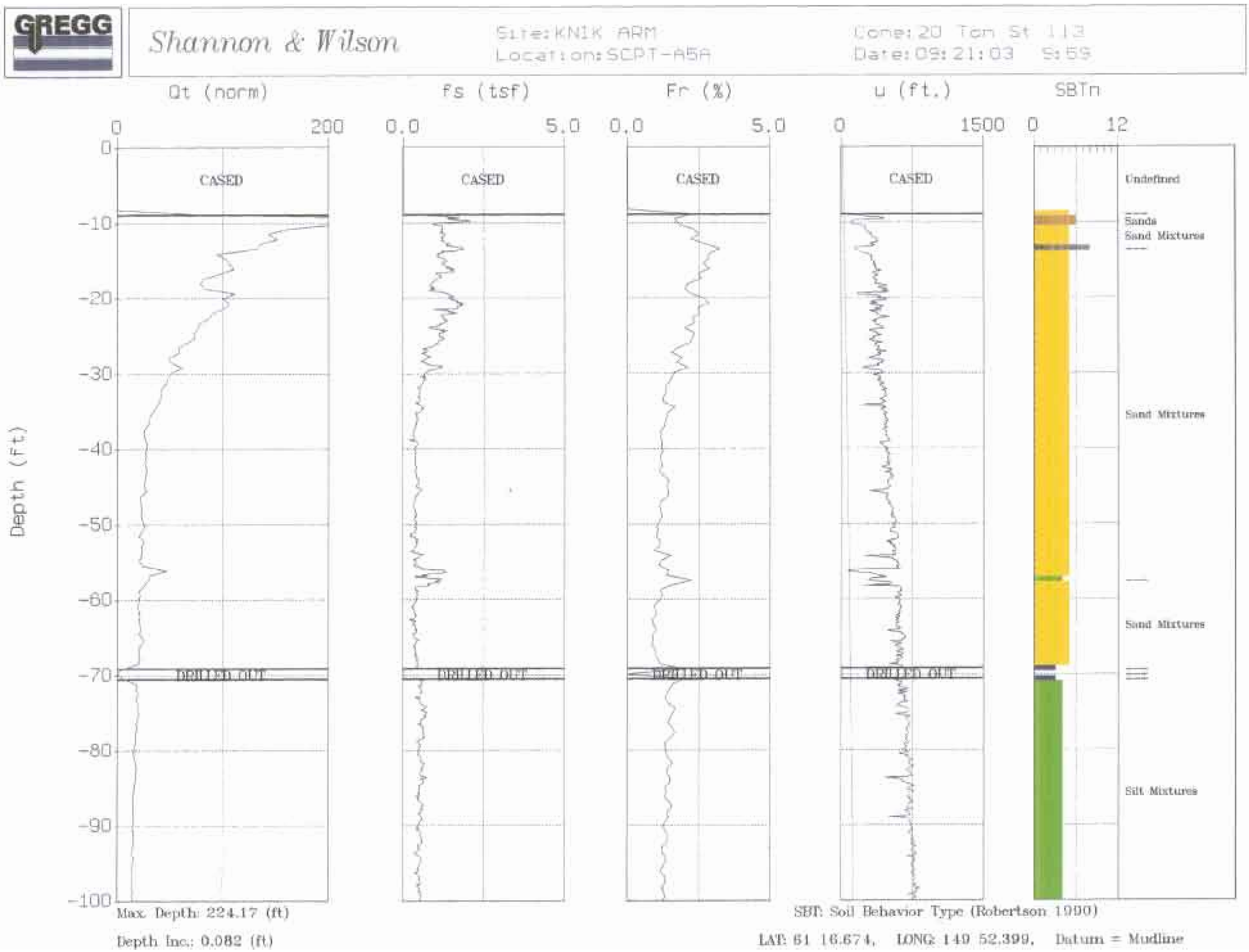


Max. Depth: 106.95 (ft)

Depth Inc.: 0.082 (ft)

SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.311, LONG: 149 52.515, Datum = Mudline

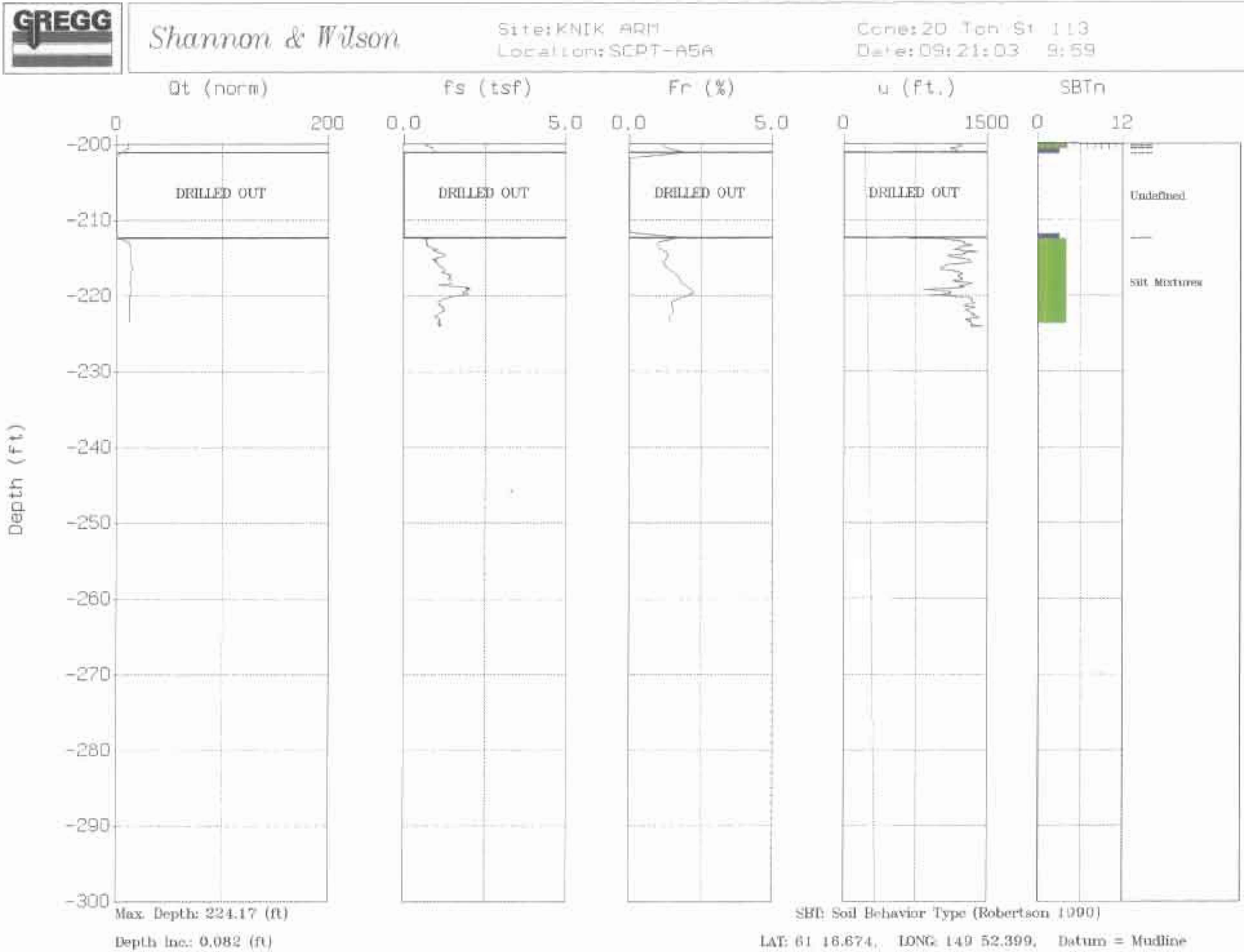

GREGG
Shannon & Wilson
Site:KNIK ARM
Location:SCPT-A5A
Cone:20 Ton St 113
Date:09:21:03 S:59
Qt (norm)
fs (tsf)
Fr (%)
u (ft.)
SBTn
0
200
0.0
5.0
0.0
5.0
0
1500
0
12
Depth (ft)
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
CASED
DRILLED OUT
Undefined
Sands
Sand Mixtures
Sand Mixtures
Sand Mixtures
Silt Mixtures
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

Max. Depth: 224.17 (ft)

Depth Inc.: 0.082 (ft)

SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline


GREGG
Shannon & Wilson
Site:KNIK ARM
Location:SCPT-A5A
Cone:20 Ton St 113
Date:09:21:03 9:59
Qt (norm)
fs (tsf)
Fr (%)
u (ft.)
SBTn
Depth (ft)
DRILLED OUT
Undefined
Silt Mixtures
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

APPENDIX D

SEISMIC CONE PENETRATION TESTING

TABLE OF CONTENTS

Report Prepared by Gregg In Situ, Inc.

LIST OF FIGURES

LIST OF PLOTS

D-1 Seismic Cone Penetration Tests

Seismic wave velocity measurements were conducted at regular intervals during selected cone penetration soundings. Seismic wave velocity measurements were made according to the procedures described by Robertson, et al. (1986). Before taking wave velocity measurements, the rods were decoupled from the CPT rig to avoid transmission of energy down the rods.

The seismic waves were generated using a blasting cap that was mounted on a blast plate lowered to the mudline. The blast box provided a trigger source initiating the recording of the seismic wave traces. The offset of the blast plate and its elevation were taken into account during calculation of the seismic wave velocities.

At each test depth, at least two waves were recorded to check the consistency of the waveforms. The seismic wave receiver used was a horizontally active geophone located in the body of the cone penetrometer. The geophone is located approximately 0.2 meters behind the cone tip. This offset is accounted for in all calculations. Data was sampled at a frequency of 20kHz (i.e., 20,000 samples per second) with a total of at least 5,000 points being recorded per wave trace. To maintain the desired signal resolution, the input sensitivity (gain) of the receiver was increased with depth.

The seismic wave velocity results are presented in both tabular and graphical form in the Seismic CPT Appendix.

Client:	Shannon & Wilson
Project:	Knik Arm Bridge, Anchorage Alaska
Sounding:	SCPT-A5A
Date:	September 21, 2003

Seismic Source:	Blast Plate	
Source Offset:	13.50 ft	
Source Depth:	0.00 ft	
Geophone Offset (m / ft):	0.20 m	0.656 ft

Seismic Test Results

Tip Depth (ft)	Geo. Depth (ft)	Ray Path (ft)	Depth Interval (ft)	Time Interval (ms)	Vs (ft/s)	Mid Layer Depth (ft)
13.93	13.28	18.94				
19.05	18.40	22.82	3.88	4.27	909	15.84
24.04	23.38	27.00	4.18	4.97	841	20.89
29.06	28.40	31.45	4.45	5.01	888	25.89
34.04	33.39	36.01	4.57	4.97	920	30.90
39.06	38.41	40.71	4.70	4.89	962	35.90
44.05	43.40	45.45	4.73	4.53	1045	40.90
49.14	48.48	50.32	4.88	5.41	902	45.94
54.06	53.40	55.08	4.76	5.15	924	50.94
58.98	58.32	59.86	4.78	6.58	727	55.86
63.97	63.31	64.73	4.87	4.62	1054	60.82
68.99	68.33	69.65	4.92	4.56	1078	65.82
74.07	73.41	74.65	5.00	6.57	760	70.87
79.06	78.40	79.55	4.91	5.69	863	75.91
83.98	83.32	84.41	4.85	3.55	1367	80.86
89.00	88.34	89.37	4.96	3.91	1268	85.83
94.02	93.36	94.33	4.97	4.81	1031	90.85
99.00	98.35	99.27	4.94	3.97	1245	95.86
104.02	103.37	104.25	4.98	3.70	1345	100.86
156.71	156.06	156.64				
161.04	160.39	160.96	4.31	4.30	1005	158.22
166.06	165.41	165.96	5.00	3.58	1399	162.90
171.05	170.39	170.93	4.97	4.06	1224	167.90
176.07	175.41	175.93	5.00	4.05	1234	172.90
181.06	180.40	180.91	4.97	3.82	1302	177.91
185.98	185.32	185.81	4.91	3.20	1534	182.86
191.00	190.34	190.82	5.01	2.86	1751	187.83
195.92	195.26	195.73	4.91	4.53	1083	192.80
201.00	200.35	200.80	5.07	3.34	1519	197.81
214.00	213.34	213.77				
218.92	218.26	218.68	4.91	3.66	1344	215.80
224.07	223.41	223.82	5.14	3.66	1405	220.84

All Depths Relative to Mudline

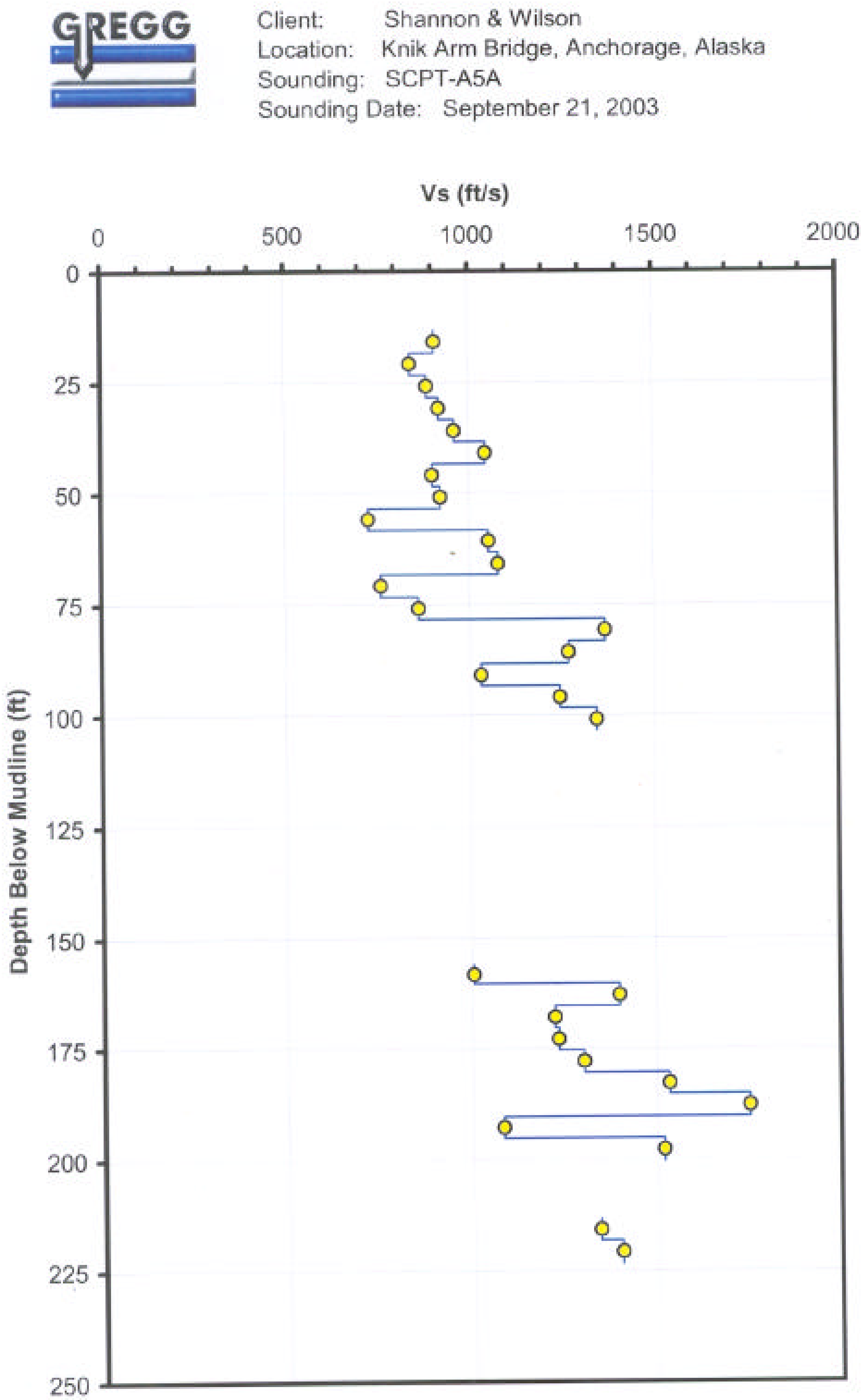
Knik Arm Bridge
Anchorage, Alaska

SEISMIC TEST RESULTS

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. D-1



Knik Arm Bridge
Anchorage, Alaska

SHEAR WAVE VELOCITY vs DEPTH PLOT

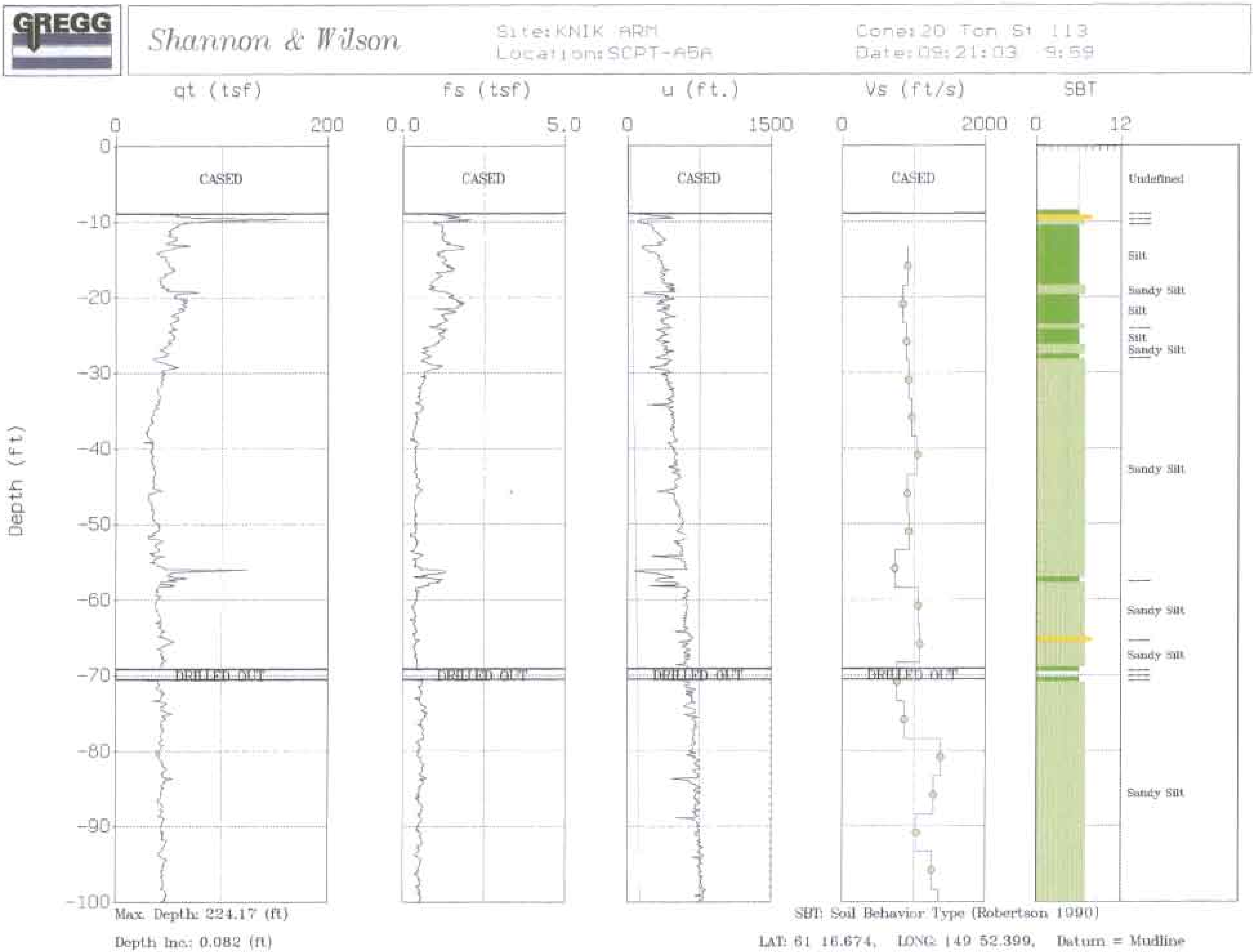
February 2004

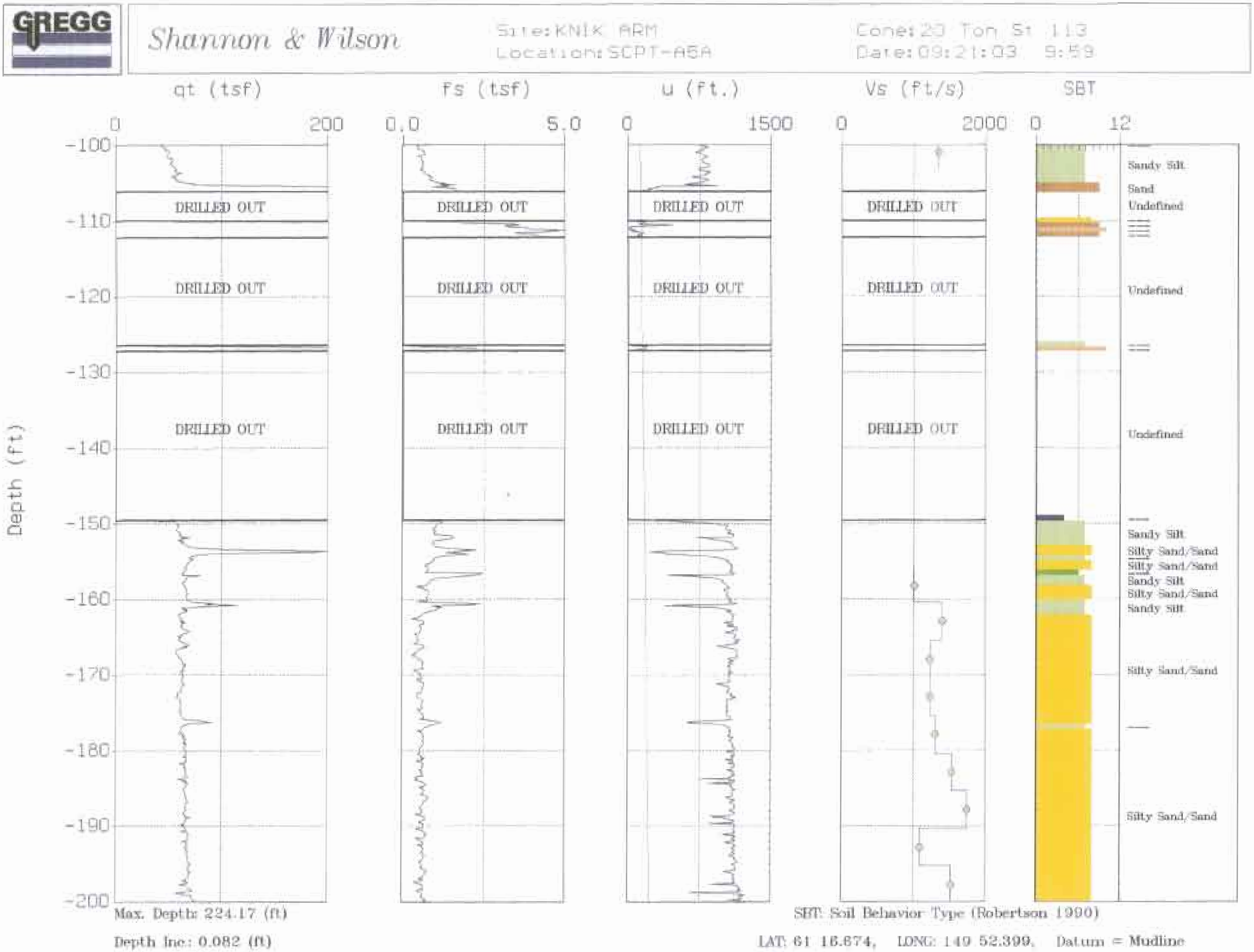
32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. D-2

CPT Plots of Shear Wave Velocities and Non-normalized Parameters


GREGG
Shannon & Wilson
Site:KNIK ARM
Location:SCPT-A5A
Cone:20 Ton S1 113
Date:09:21:03 9:59
qt (tsf)
fs (tsf)
u (ft.)
Vs (ft/s)
SBT
0
200
0.0
5.0
0
1500
0
2000
0
12
Depth (ft)
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
CASED
DRILLED OUT
Undefined
Silt
Sandy Silt
Silt
Silt
Sandy Silt
Sandy Silt
Sandy Silt
Sandy Silt
Sandy Silt
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

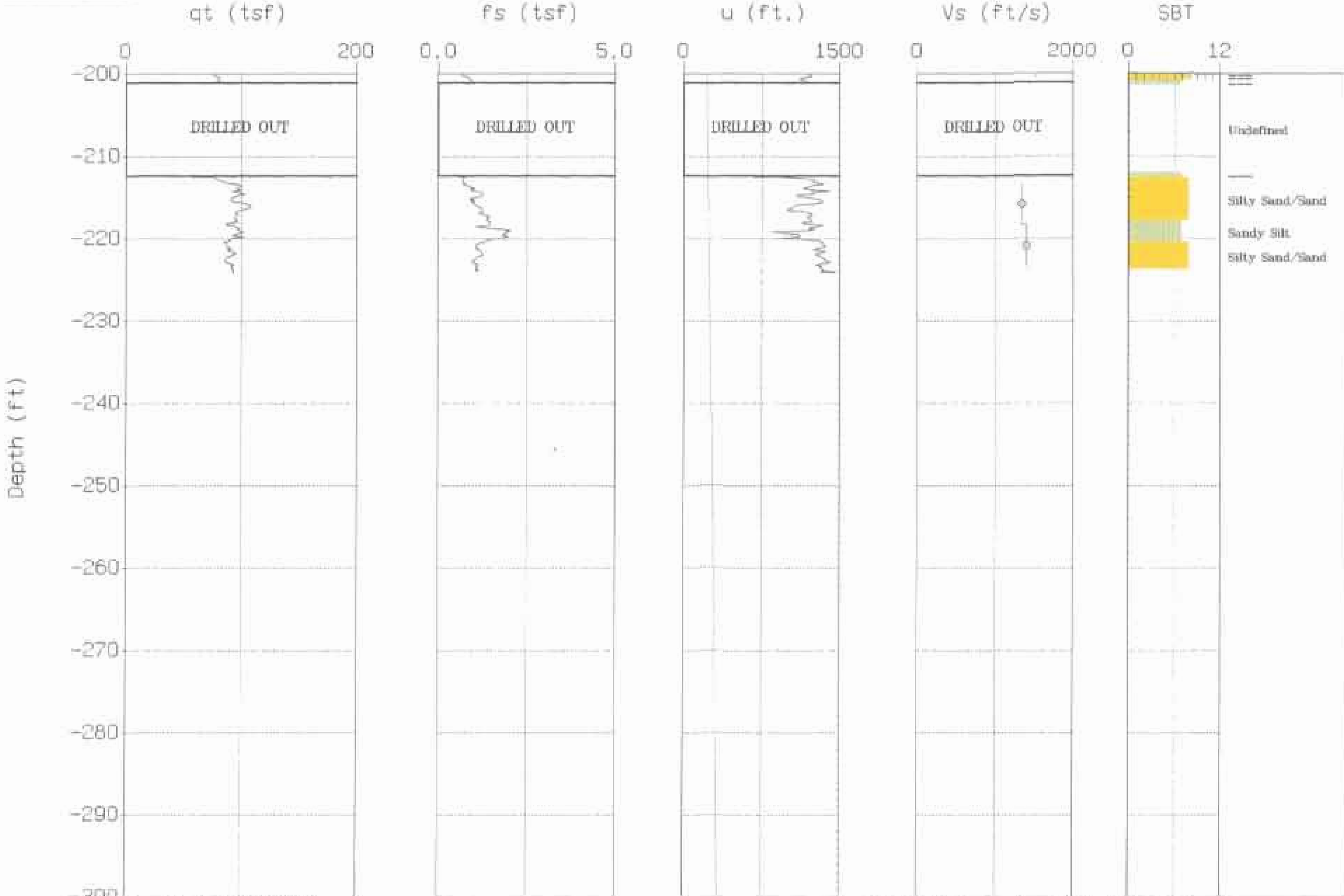

GREGG
Shannon & Wilson
Site:KNIK ARM
Location:SCPT-A5A
Cone:20 Ton St 113
Date:09:21:03 9:59
qt (tsf)
fs (tsf)
u (ft.)
Vs (ft/s)
SBT
0
200
0.0
5.0
0
1500
0
2000
0
12
-100
-110
-120
-130
-140
-150
-160
-170
-180
-190
-200
Depth (ft)
DRILLED OUT
Sandy Silt
Sand
Undefined
Undefined
Undefined
Sandy Silt
Silty Sand/Sand
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Silty Sand/Sand
Max. Depth: 224.17 (ft)
Depth Inc: 0.082 (ft)
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

GREGG

Shannon & Wilson

Site:KNIK ARM
Location:SCPT-A5A

Cone:20 Ton St 113
Date:09:21:03 9:59

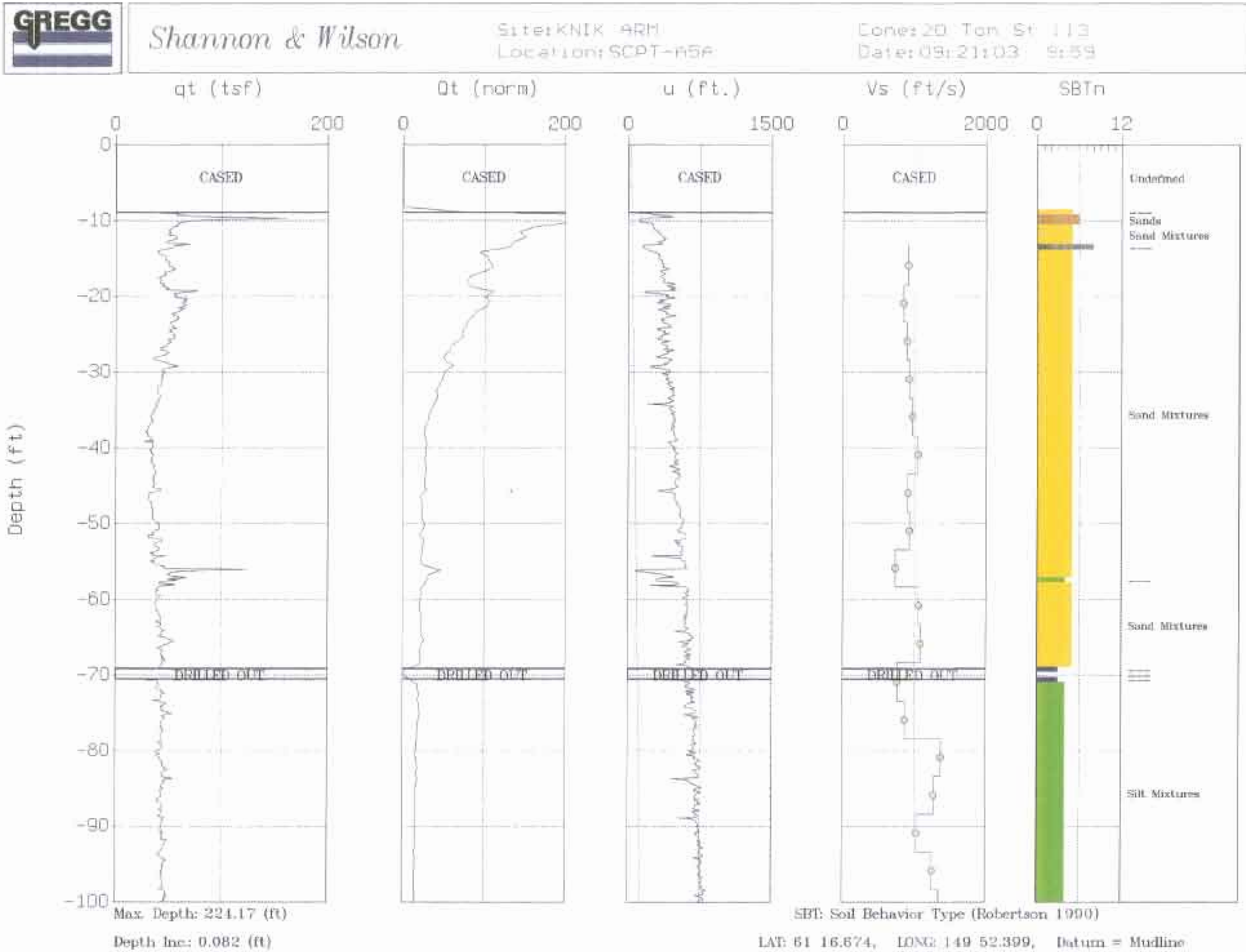


Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)

SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

CPT Plots of Shear Wave Velocities
and Normalized Parameters



GREGG
Shannon & Wilson
Site: KNIK ARM
Location: SCPT-A5A
Cone: 20 Ton St 113
Date: 09:21:03 9:59
qt (tsf)
Qt (norm)
u (ft.)
Vs (ft/s)
SBTn
0
200
0
200
0
1500
0
2000
0
12
Depth (ft)
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
CASED
CASED
CASED
CASED
DRILLED OUT
DRILLED OUT
DRILLED OUT
DRILLED OUT
Undefined
Sands
Sand Mixtures
Sand Mixtures
Sand Mixtures
Silt Mixtures
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)
SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

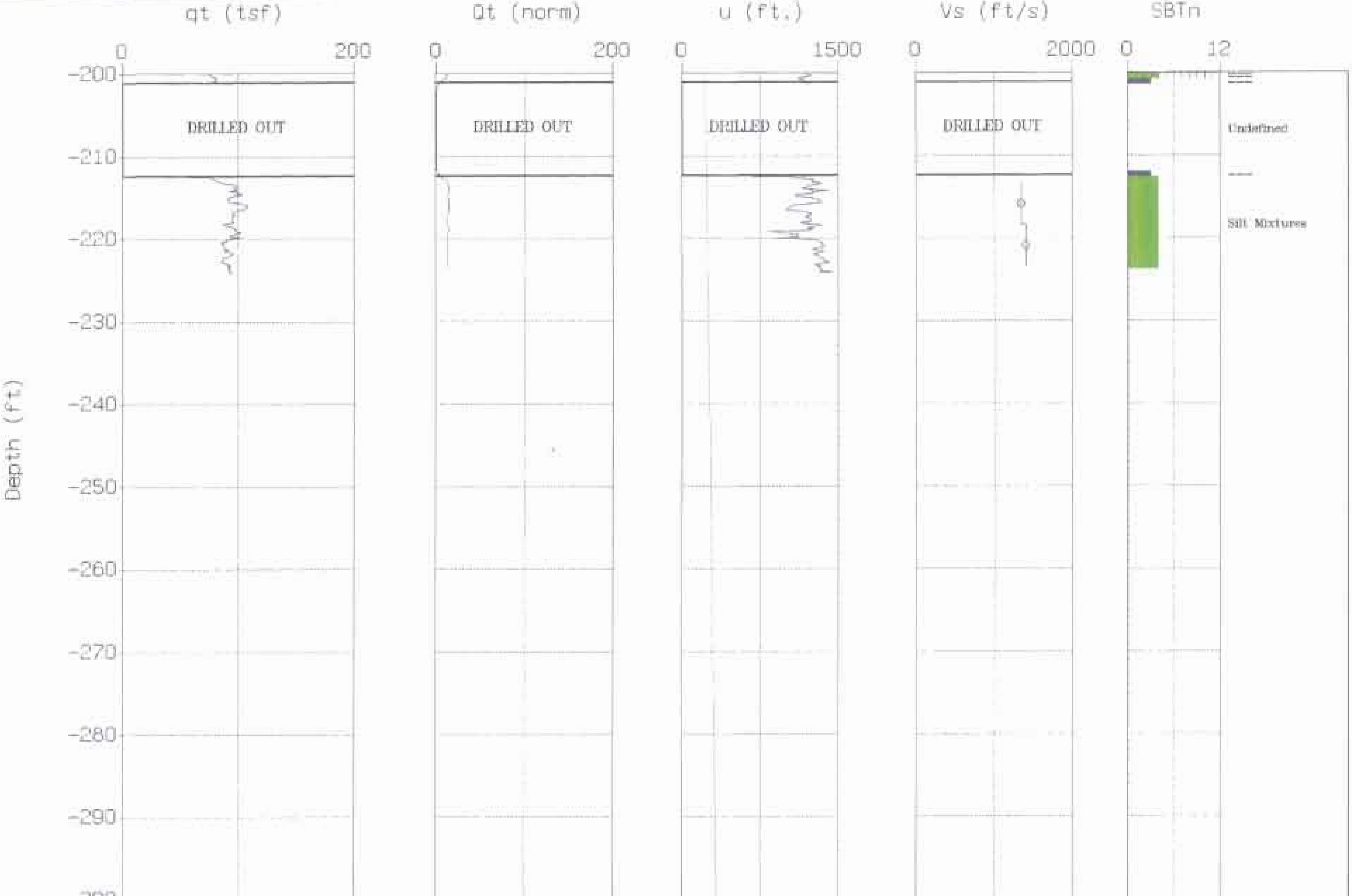
Max. Depth: 224.17 (ft)
Depth Inc.: 0.082 (ft)

SBT: Soil Behavior Type (Robertson 1990)
LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

Shannon & Wilson

Site: KNIK ARM
Location: SCPT-A5A

Cone: 20 Ton St 113
Date: 05:21:03 9:59



Max. Depth: 224.17 (ft)

Depth Inc.: 0.082 (ft)

SBT: Soil Behavior Type (Robertson 1990)

LAT: 61 16.674, LONG: 149 52.399, Datum = Mudline

APPENDIX E

DRILL ROD ENERGY TRANSFER RESULTS

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December 5, 2003

Messrs. Fred Brown, P.E. and Grover Johnson, P.E.
Shannon and Wilson, Inc.
5430 Fairbanks Street, Suite 3
Anchorage, AK 99518

Re: Standard Penetration Test Energy Measurements
Boring A6 September 14 and Boring A10 September 19-20, 2003
Proposed Knik Arm Crossing
Anchorage, Alaska RMDT Job No. 03F55

Dear Sirs,

This letter presents energy transfer measurements made during Standard Penetration Tests for the borings referenced above. ConTec, Ltd. provided and operated the platform, drilling rig and the SPT equipment. Robert Miner Dynamic Testing, Inc. (RMDT) made dynamic measurements with a Pile Driving Analyzer®. Measurements were made for two borings at sample depths ranging from 31 to 166 ft and rod lengths of 72 to 259 ft.

The purpose of RMDT's testing was the measurement of energy transferred to the drill rods. ConTec provided RMDT with a 5 ft long section of their 3.5" OD drill rod; we attached our sensors to the midpoint of that section, and placed the section at the top of the drill string during our monitoring. RMDT's sensors consisted of four strain sensors and four accelerometers, all connected to a Pile Driving Analyzer ® (PDA) which generally processed acceleration and strain measurements from each hammer blow and stored both the measurements and computed results. Measurements and data processing generally followed the ASTM D 4945-89 standard. Energy transfer past the gage location, EFV, was computed by the PDA using force and velocity records as follows:

$$EFV = \int_a^b F(t)\ v(t)\ dt$$

The value "a" corresponds to the start of the record which is when the energy transfer begins and *"b"* is the time at which energy transferred to the rod reaches a maximum value. Appendix A contains more information on our measurement equipment and methods of analysis. (The EFV energy result is essentially identical to the EMX energy result discussed in Appendix A.) The field EFV values apply to the sensor location near the top of the rod; energy transfer to a location near the spoon was evaluated with CAPWAP analysis of selected hammer blows.

TEST DETAILS

Testing occurred on September 14 in Boring A6 and September 19-20 in Boring A10. For both borings the SPT split spoon was advanced with an automatic hammer. RMDT did not observe any marks on the hammer indicating it's manufacture; we understand that the ram weighed 140 lbs and that the ram vertical stroke was 30 inches. The steel rod between the hammer and the sampler was reported to be 3.5" OD with a wall thickness of 0.188", except for a short portion of NW rod just above the spoon.

On September 14 we monitored sampling at 31 ft depth in Boring A6. On September 19-20 we monitored sampling at 66, 88.5, 126, 156 and 166 ft in Boring A10. For additional details regarding each boring, including the boring location, please see the respective soil boring logs and related documents prepared by others.

RESULTS OF TRANSFER ENERGY MEASUREMENTS

Table 1 summarizes RMDT's field results. Appendix B contains additional numerical results and figures with plots of the computed results for each data set. The results in Table 1 includes approximate starting sample depth, reported penetration resistance, number of hammers blows in our data set, measured energy transfer, EFV and the computed Transfer Efficiency, ETR. (The Transfer Efficiency, ETR, is the ratio obtained when the measured transfer energy, EFV, is divided by the ram's theoretical free fall energy. A 140 lb ram raised 30 inches above an impact surface has 350 lb-ft of potential energy. Thus, the transfer energy results for sampling with the 140 lb hammer are divided by 350 lb-ft to yield ETR, which is then given as a percent efficiency for each sample interval.)

For Boring A 6 the single monitored sample interval had an average ETR of 87 percent. For Boring A 10 the sample interval average ETR values ranged from 83 to 89 percent. The combined average of the six sample intervals (A 6 and A 10) was 85 percent.

CAPWAP ANALYSES OF ENERGY LOSS IN THE SAMPLING ROD

CAPWAP analyses were made with one hammer blow selected from each of six monitored sample intervals. These analyses were completed to evaluate energy transfer to a location relatively close to the lower end of the sample rod. Table 3 summarizes the pertinent CAPWAP results and the corresponding results measured with the PDA sensors that were placed near the top of the sample rod. In preparing Table 2 we selected CAPWAP energy results for a location approximately 15 ft above the spoon because computation of the energy at the rod tip is probably subject to greater uncertainty and is probably also less relevant due to end effects. In the discussion and data that follows this CAPWAP energy for a location about 15 ft above the spoon is referred to as the energy "at the spoon" or "near the spoon" or as the "bottom energy".

Figure 1 presents the differences between the CAPWAP computed energy for rod locations near the top and near the spoon. As the rod length increased the difference between energy transfer at the top and bottom increased. Figure 2 presents the ratio of the bottom energy to the top energy, expressed as a percent, and we have called this the Rod Efficiency. The Rod Efficiency decreased as the rod length increased.

Figure 3 presents the difference between Transfer Efficiency, ETR, at the top and bottom of the rod. Transfer Efficiency is defined as the ratio of actual energy to the 350 ft-lbs nominal energy (140 lb ram falling 30 inches). Per Figure 3 the difference between ETR at the top and near the spoon was approximately 22 percent for a 170 ft rod length and approximately 27 percent for 250 ft rod length.

Figure 4 presents the Transfer Efficiency data given in Figure 3 normalized by the rod length. For a rod length of about 170 ft the total difference between the Transfer Efficiency at the top and bottom was 0.13 *percent per ft,* while at greater depth the loss per ft decreased to approximately 0.10 percent per ft. Note that these rates of loss per ft are based on the total rod length, not the incremental changes in the rod length. Given a rod length of 200 ft, ETR near the spoon would be 24 percent (0.12% per ft x200 ft) less than ETR near the top. However, it should be noted that these results are for sampling with an automatic hammer providing top ETR values that were typically close to 85 percent. Operation at substantially different ETR values or with different hammer types may alter the differences between the top and bottom ETR values.

Figure 5 is similar to Figure 4, except that the difference between top and bottom energy is presented as a percent of the top energy rather than as a percentage of the full nominal 350 ft-lb energy. Per Figure 5, for a rod length of depth of 200 ft transfer energy near the spoon would be approximately 26 percent (0.13 % per ft x 200 ft) less than transfer energy near the top.

The pattern exhibited in Figures 4 and 5 suggests relatively constant rates of loss per ft for depths greater than approximately 160 ft, with that loss rate in ETR being approximately 0.10

to 0.12 percent/ft. However, the present data is mostly for one boring and primarily for rod lengths of 160 to 260 ft with a 3.5"OD rod; the data is thus somewhat limited in scope. Prior work by RMDT using similar methods with, NW rods and safety hammers yielded ETR loss rates of approximately 0.7 percent per ft for lengths of approximately 160 to 200 ft.

Extrapolation of the trends of Figure 4 or 5 to rod lengths outside the range of approximately 140 to 260 ft may introduce uncertainty greater than the uncertainty within the available data. Also, use of CAPWAP signal matching for the purpose of evaluating transfer energy patterns is not widely tested, and the results are thus subject to greater uncertainty than are the results for more common CAPWAP applications. Appendix A provides further information about our equipment and methods and information pertaining to the limitations of methods.

It was a pleasure to assist you and all the field staff of this project. Please do not hesitate to contact us if you or your client have any questions about this report.

Sincerely,

Robert Miner Dynamic Testing, Inc.

Robert Miner

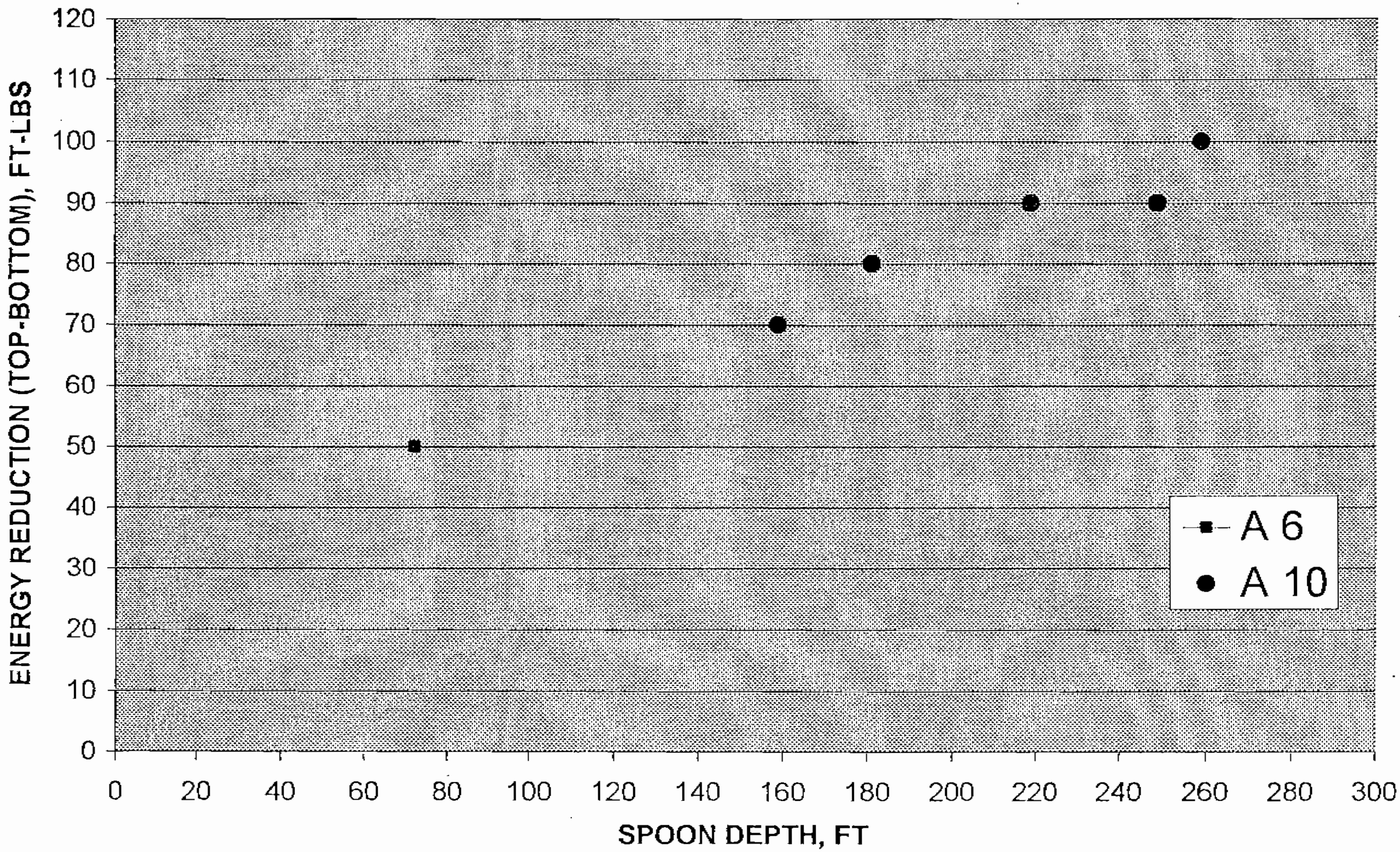
Table 1. Summary of Test Details and Results. September 14, 19, 20, 2003

Boring	Starting Sample Depth (a) ft	Penetration Resistance (a) blows/set	Number of Blows in Data Set	Average Measured Transfer Energy EFV lb-ft	Average Computed Transfer Efficiency ETR percent
A 6	31	28/ft	28	305	87
A 10	66	48/ft	48	293	84
A 10	88.5	38/ft	38	311	89
A 10	126	41/ft	38	299	85
A 10	156	72/ft	72	295	84
A 10	166	84/ft	83	291	83

Table 3. Summary of CAPWAP Results

Boring	Starting Sample Depth ft	Sample Average Measured Transfer Energy EFV lb-ft	CAPWAP Results for One Blow	
			Transfer Energy at Top of Rod lb-ft	Transfer Energy Approx. 15 ft above the Spoon lb-ft
A 6	31	28	300	250
A 10	66	48	290	220
A 10	88.5	38	310	230
A 10	126	41	300	210
A 10	156	72	290	200
A 10	166	83	290	190

Figure 1. Energy Difference (Top-Bottom) vs Rod Length
(CAPWAP Computed Energy in the Rod)



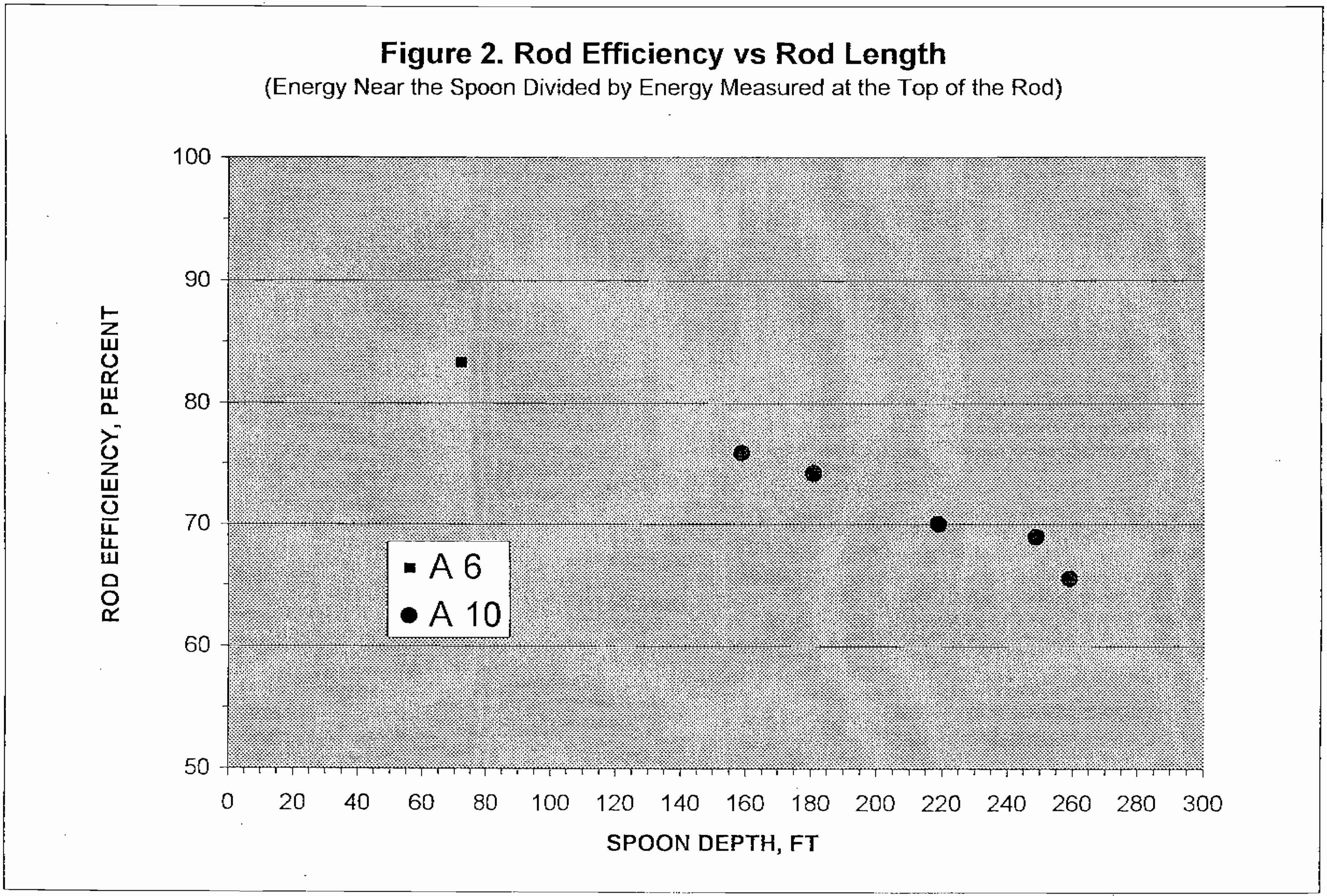


Figure 2. Rod Efficiency vs Rod Length
(Energy Near the Spoon Divided by Energy Measured at the Top of the Rod)
ROD EFFICIENCY, PERCENT
100
90
80
70
60
50
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
SPOON DEPTH, FT
A 6
A 10

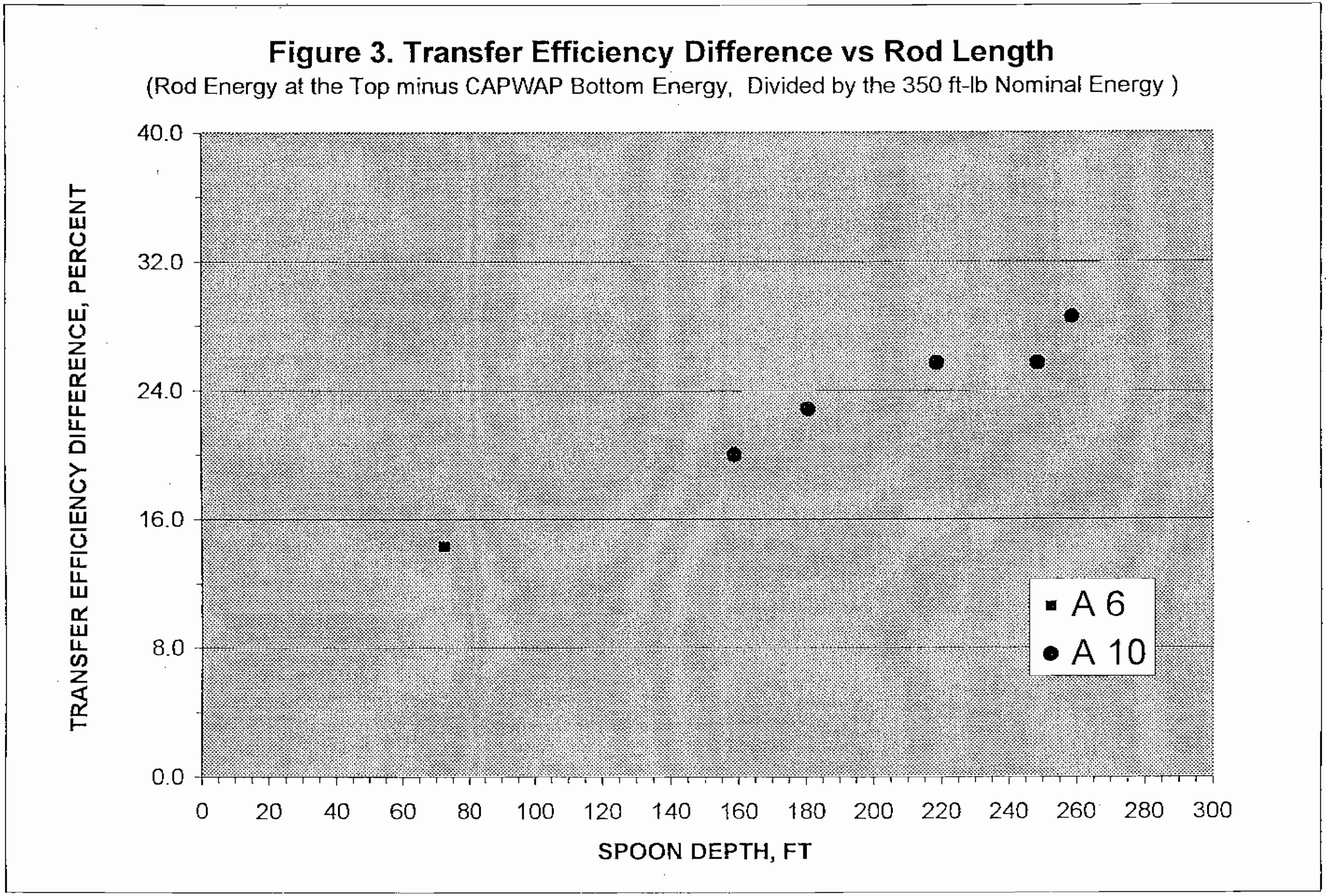
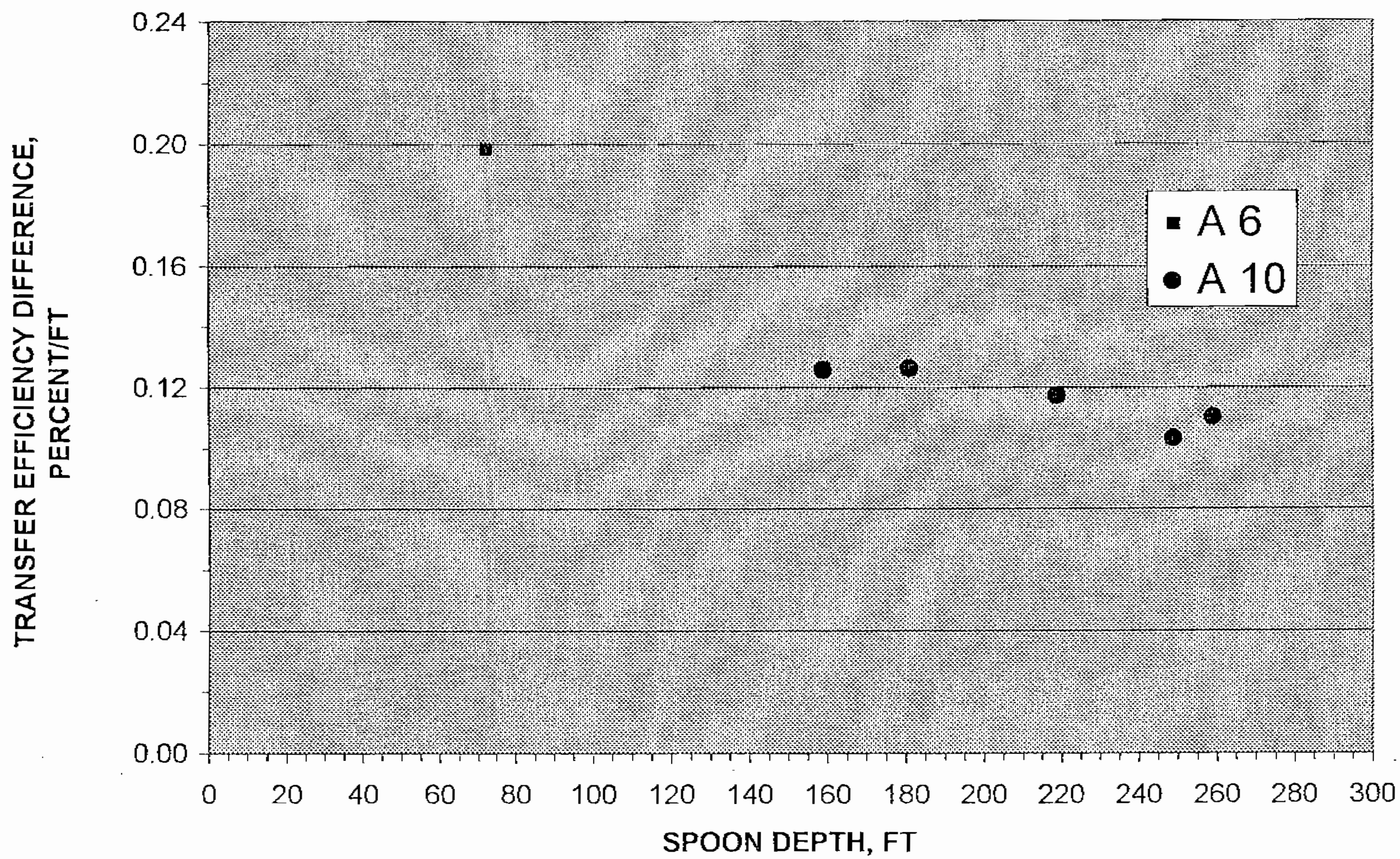


Figure 3. Transfer Efficiency Difference vs Rod Length
(Rod Energy at the Top minus CAPWAP Bottom Energy, Divided by the 350 ft-lb Nominal Energy)
TRANSFER EFFICIENCY DIFFERENCE, PERCENT
40.0
32.0
24.0
16.0
8.0
0.0
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
SPOON DEPTH, FT
A 6
A 10

Figure 4. Length Normalized Transfer Efficiency Diff. vs Rod Length

(Transfer Efficiency Difference Divided by Rod Length)



Robert Miner Dynamic Testing, Inc.

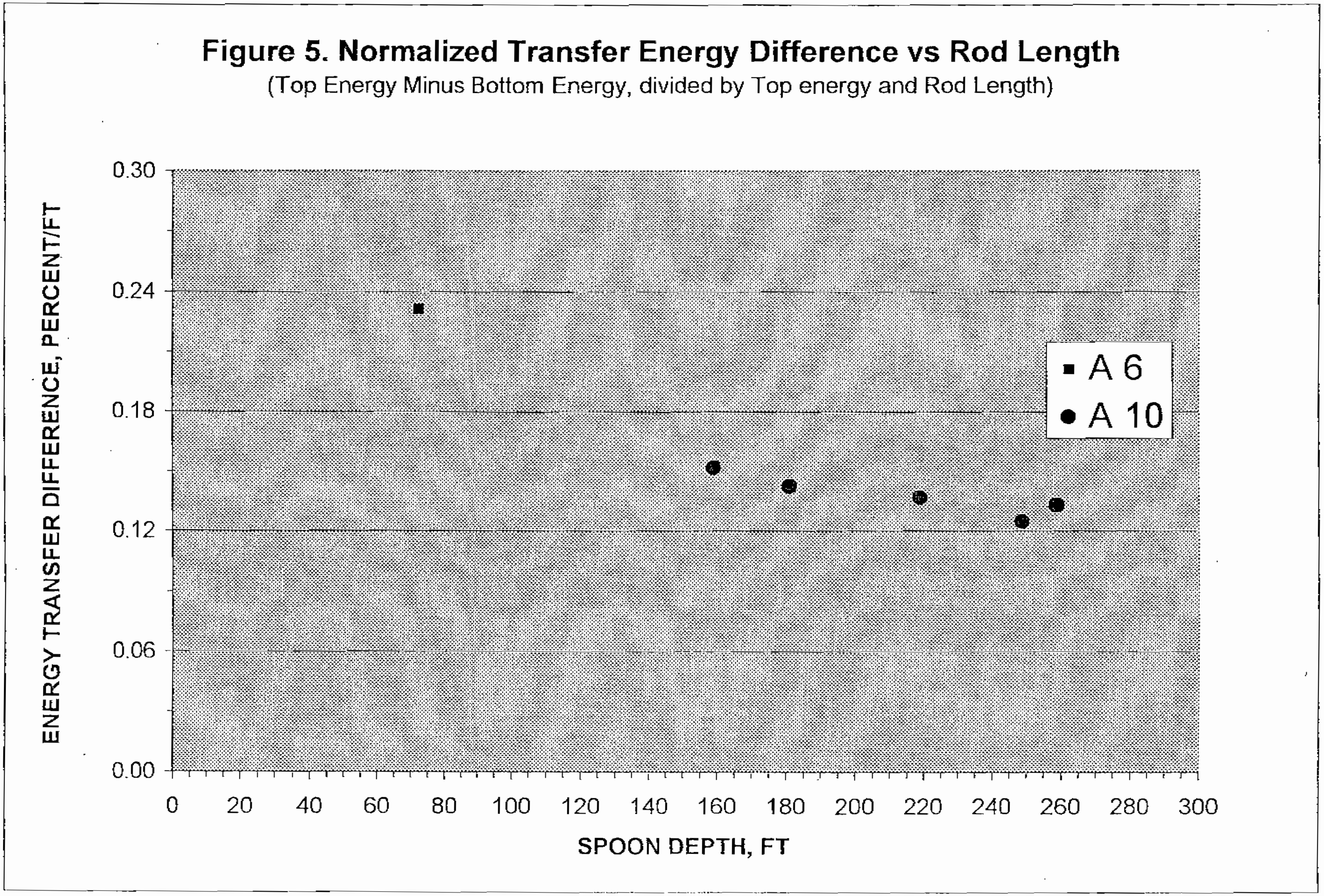


Figure 5. Normalized Transfer Energy Difference vs Rod Length
(Top Energy Minus Bottom Energy, divided by Top energy and Rod Length)
ENERGY TRANSFER DIFFERENCE, PERCENT/FT
0.30
0.24
0.18
0.12
0.06
0.00
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300
SPOON DEPTH, FT
A 6
A 10

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during preconstruction test programs and also production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore, form an important part of a quality assurance program when deep foundations are executed. Several dynamic pile testing methods exist; they have different benefits and limitations and different requirements for proper execution.

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (such as that of a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® (PDA). However, for bearing capacity evaluations an important additional method is CAPWAP® which performs a much more rigorous analysis of the dynamic records than the simpler Case Method.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the Case Method or "High Strain Test" Method of pile testing, however, for the sake of completeness, the "Low Strain Test" performed with the Pile Integrity Test™ (PIT), mainly for pile integrity evaluation, will also be described.

RESULTS FROM DYNAMIC TESTING

There are two main objectives of high strain dynamic pile testing:

- *Dynamic Pile Monitoring* and
- *Dynamic Load Testing.*

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both cast *insitu* piles or drilled shafts and impact driven piles during restrike.

Dynamic Pile Monitoring

During pile installation, the sensors attached to the pile measure pile top force and velocity. A PDA conditions and processes these signals and calculates or evaluates:

- <u>Bearing capacity</u> at the time of testing, including an assessment of shaft resistance development and driving resistance. This information supports formulation of a driving criterion.

- <u>Dynamic pile stresses</u>, axial and averaged over the pile cross section, both tensile and compressive, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- <u>Pile integrity</u> assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early enough, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence.

- <u>Hammer performance</u> parameters including the energy transferred to the pile, the hammer speed in blows per minute and the stroke of open ended diesel hammers.

Dynamic Pile Load Testing

Bearing capacity testing of either driven piles or drilled shafts applies the same basic measurement approach of dynamic pile monitoring. However, the test is done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then its weight should be between 0.8 and 2% of the test load (e.g. between 4 and 10 tons for a 500 ton test load) to assure sufficient soil resistance activation.

For a successful test, it most important that the test is conducted after a <u>sufficient waiting time</u> following pile installation for soil properties approaching their long term condition or concrete to properly set. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain safe stresses and assure sufficient resistance activation. For safe and sufficient testing of drilled shafts, ram energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles may require a warm hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with sufficient energy and safe stresses, the CAPWAP analysis provides the following results:

- <u>Bearing capacity</u> i.e. the mobilized capacity present at the time of testing
- <u>Resistance distribution</u> including shaft resistance and end bearing components
- <u>Stresses in pile or shaft</u> calculated for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or nonuniform contact stresses, e.g. when the pile toe is on uneven rock.
- <u>Shaft impedance</u> vs depth; this is an estimate of the shaft shape if it differs substantially from the planned profile
- <u>Dynamic soil parameters</u> for shaft and toe, i.e. damping factors and quakes (related to the dynamic stiffness of the resistance at the pile/soil interface.)

MEASUREMENTS

PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below.

HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC or recorded on a strip chart.

Saximeter™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

PIT

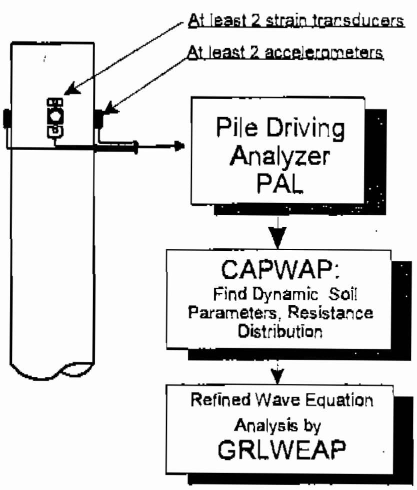
The Pile Integrity Tester™ (PIT) can be used to evaluate defects in concrete piles or shafts which may have occurred during driving or casting. Also timber piles of limited length can be tested in that manner. This so-called "Low Strain Method" or "Pulse-Echo Method" of integrity testing requires only the measurement of acceleration at the pile top. The stress wave producing impact is then generated by a small hand-held hammer and the records interpreted in the time domain. PIT also supports the so-called "Transient Response Method" which requires the additional measurement of the hammer force and an analysis in the frequency domain. This method may also be used to evaluate the unknown length of deep foundations under existing structures.

ANALYTICAL SOLUTIONS
BEARING CAPACITY

Wave Equation

GRL has written the GRLWEAP™ program which calculates a relationship between bearing capacity, pile stress and blow count. This relationship is often called the "bearing graph." Once the blow count is known from pile installation logs, the bearing graph yields the bearing capacity. This approach requires no measurements and therefore can be performed during the design stage of a project, for example for the selection of hammer, cushion and pile size.

After dynamic pile monitoring and/or dynamic load testing has been performed, the "Refined Wave Equation Analysis" or RWEA (see schematic below) is often performed by inputting the PDA and CAPWAP calculated parameters. Then the bearing graph from the RWEA is the basis for a safe and sufficient driving criteria.



Case Method

The Case Method is a closed form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force F(t) and pile top velocity v(t), the total soil resistance is

$$R(t) = \tfrac{1}{2}\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \qquad (1)$$

where

- t = a point in time after impact
- t_2 = time $t + 2L/c$
- L = pile length below gages
- c = $(E/\rho)^{1/2}$ is the speed of the stress wave
- ρ = pile mass density
- Z = EA/c is the pile impedance
- E = elastic modulus of the pile (ρc^2)
- A = pile cross sectional area

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t) \qquad (2)$$

The dynamic component may be computed from a soil damping factor, J, and a pile toe velocity, $v_t(t)$ which is conveniently calculated for the pile toe. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \qquad (3)$$

and finally to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 can be evaluated. Most commonly, t_2 is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used) requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2 therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed.

The static resistance calculated by Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDAPLOT program.

CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors and soil stiffnesses. The method iteratively calculates a number of unknowns by signal matching. While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program works with the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters.

STRESSES

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from individual strain transducers, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance R(t) minus the total shaft resistance, SFT. Again, for this stress estimation uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress can be computed from the pile top measurements by finding the maximum tension wave (either traveling upward, W_U, or downward, W_d) and reducing it by the minimum compressive wave traveling in opposite direction.

$$W_u = \tfrac{1}{2}[F(t) - Zv(t)] \qquad (4)$$

$$W_d = \tfrac{1}{2}[F(t) + Zv(t)] \qquad (5)$$

CAPWAP also calculates tensile and compressive stresses along the pile and, in general, more accurately than the PDA. In fact, for non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA may be in error.

PILE INTEGRITY

High Strain Tests (PDA)

Stress waves in a pile are reflected wherever the pile impedance, $Z = EA/c = \rho cA = A\sqrt{E\rho}$, changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The magnitude of the change of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β_i (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA.

$$\beta_i = (1 - \alpha_i)/(1 + \alpha_i) \quad (6)$$

with

$$\alpha_i = ½(W_{UR} - W_{UD})/(W_{Di} - W_{UR}) \quad (7)$$

where

- W_{UR} is the upward traveling wave at the onset of the reflected wave. It is caused by resistance.
- W_{UD} is the upwards traveling wave due to the damage reflection.
- W_{Di} is the maximum downward traveling wave due to impact.

It can be shown that this formulation is quite accurate as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections.

Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.6.

Low Strain Tests (PIT)

The pile top is struck with a held hand hammer and the resulting pile top velocity is measured, displayed and interpreted for signs of wave reflections. In general, a comparison of the reflected acceleration leads to a relative measure of extent of damage, again the location of the problem is indicated by the arrival time of the reflection. PIT records can also be interpreted by the β-Method. However, low strain tests do not activate much resistance which simplifies Eq. 7 since W_{UR} is then equal to zero.

For drilled shafts and PIT records that clearly show a toe reflection, an approximate shaft profile can be calculated from low strain records using the PITSTOP program's PROFILE routine.

HAMMER PERFORMANCE

The PDA calculates the energy transferred to the pile top from:

$$E(t) = {}_0\int^t F(t)v(t)\, dt \quad (8a)$$

The maximum of the E(t) curve is the most important information for an overall evaluation of the performance of a hammer and driving system. This **EMX** value allows for a classification of the hammer's performance when presented as the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency

$$e_T = EMX/E_R \quad (8b)$$

where

E_R is the manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\, T_B^2 - h_L \quad (9)$$

where

- g is the earth's gravitational acceleration,
- T_B is the time between two hammer blows,
- h_L is a stroke loss value due to gas compression and time losses during impact (usually 0.3 ft or 0.1 m).

DETERMINATION OF WAVE SPEED

An important facet of dynamic pile testing is an assessment of pile material properties. Since in general force is determined from strain by multiplication with elastic modulus, E, and cross sectional area, A, the dynamic elastic modulus has to be determined for pile materials other than steel. In general, the records measured by the PDA clearly indicate a pile toe reflection as long as pile penetration per blow is greater than 1 mm or .04 inches. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T \quad (10)$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho \qquad (11)$$

Since the mass density of the pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is easily found from the wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static one and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from PIT is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is nonuniform then the wave speed c, according to Eq. 10, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c is slower than that at the pile top. It is therefore recommended to determine E in the beginning of pile driving and not adjust it when the average c changes.
- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under "DATA QUALITY CHECKS" to find c as the ratio between the measured velocity and measured strain.

DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing PDA or PIT tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

Proportionality

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\,Z = v\,(EA/c) \qquad (12a)$$

This relationship can also be expressed in terms of stress

$$\sigma = v\,(E/c) \qquad (12b)$$

or strain

$$\varepsilon = v / c \qquad (12c)$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

Measurements are always taken at opposite sides of the pile as a means of calculating the average force and velocity in the pile. The velocity on the two sides of the pile is very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. The averaging of the two strain signals does then not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top.

LIMITATIONS, ADDITIONAL CONSIDERATIONS

Mobilization of capacity

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

Time dependent soil resistance effects

Static pile capacity from dynamic method calculations provide an estimate of the axial pile capacity. Increases and decreases in the pile capacity with time typically occur (soil setup/relaxation). Therefore, **<u>restrike testing</u> usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

(A) Soil setup

Because excess positive pore pressures often develop during pile driving in fine grained soil (clays, silts or even fine sands), the capacity of a pile at the time of driving may often be less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, the soil resistance acting on the pile increases as does the axial pile capacity. This phenomena is routinely called soil setup or soil freeze.

(B) Relaxation

Relaxation (capacity reduction with time) has been observed for piles driven into weathered shale, and may take several days to fully develop. Pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically (with particular emphasis than on the first few blows). Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. Again, restrike tests should be used, with great emphasis on early blows.

Capacity results for open pile profiles

Larger diameter open ended pipe piles (or H-piles which do not bear on rock) may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions.

CAPWAP Analysis Results

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations.

Stresses

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or non-uniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

- 90% of yield strength for steel piles
- 85% of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression
- 100% of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension

70% of the reinforcement strength for regularly reinforced concrete piles in tension

300% of the static design allowable stress for timber

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA for other locations along the pile based on the pile top measurements.

Additional design considerations

Numerous factors have to be considered in pile foundation design. Some of these considerations include

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

Wave equation analysis results

The results calculated by the wave equation analysis program depend on a variety of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities with have often displayed considerable scatter.

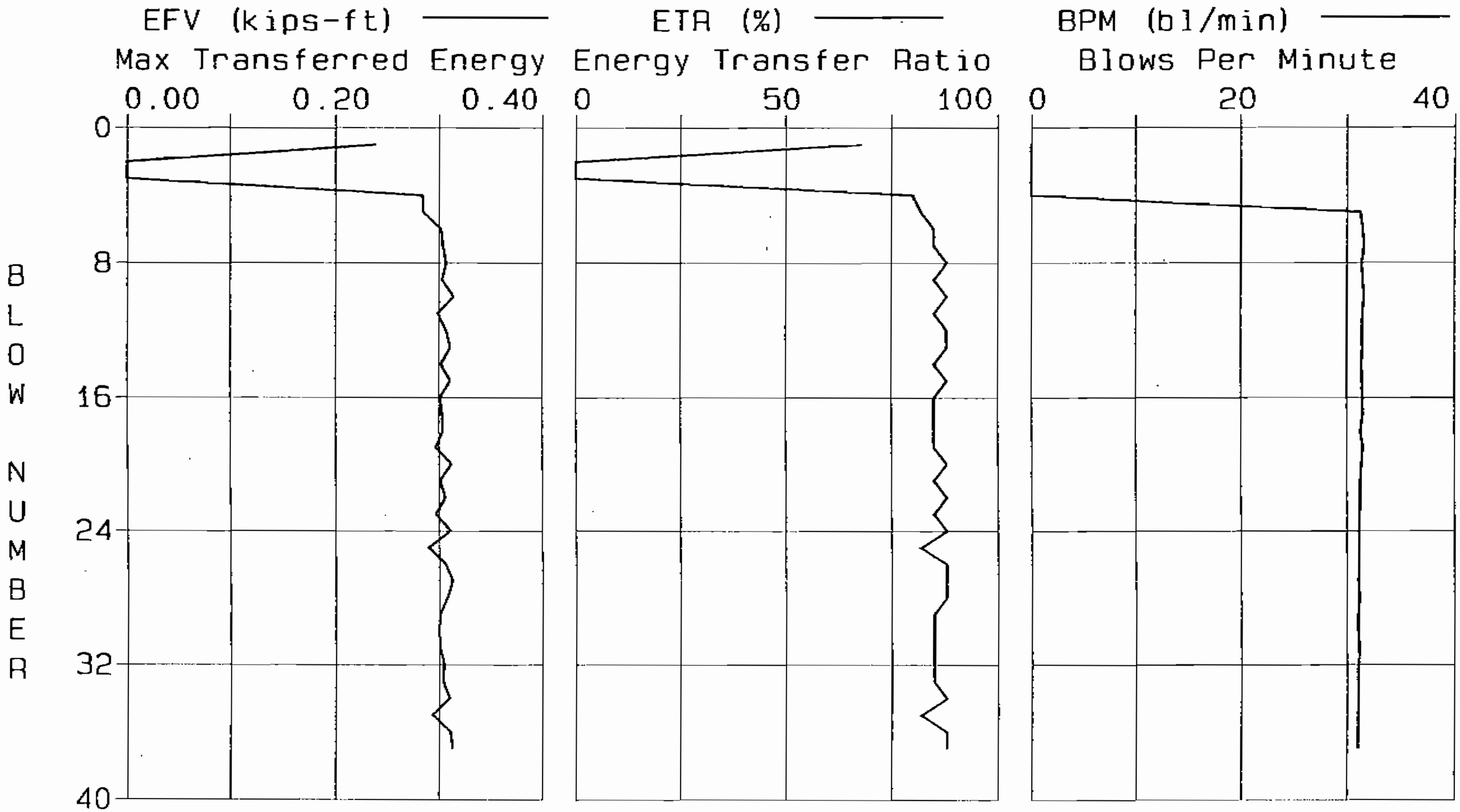
As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

Appendix B

Case Method Results

A Miner Dynamic Test 2003-Sep-14

KNIK ARM, A6 30 ft, 3.5"X3/16", AUTO HAMMER



Robert Miner Dynamic Testing, Inc.

Pile: A6 30 ft
Info: 3.5"X3/16"
AR: 2.0 in^2
LE: 72.3 ft

Proj: KNIK ARM
SP: 0.492 k/ft^3
WS: 16800 ft/s
EM: 29948 KSI

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
VMX: Max Measured Velocity
EFV: Max Transferred Energy
ETR: Energy Transfer Ratio

BPM: Blows Per Minute
EF2: Energy by F^2 Method
RAT: Reflection time Ratio
DMX: Max Meas'd Displacement

BL#	depth ft	CSX ksi	CSI ksi	VMX ft/sec	EFV kips-ft	ETR %	BPM bl/min	EF2 lb-ft	RAT %	DMX inch	BLC bl/ft
10		27.82	31.15	11.2	0.314	88	31.6	403	20	0.69	24
11		25.47	31.46	11.0	0.299	85	31.5	419	31	0.56	24
12		26.59	31.62	10.7	0.307	88	31.5	403	20	0.53	24
13		25.62	32.54	10.9	0.311	88	31.5	439	30	0.63	24
14		26.95	32.54	10.9	0.302	85	31.4	408	22	0.68	24
15		25.98	31.26	10.9	0.311	88	31.5	413	20	0.61	24
16		26.49	31.56	10.9	0.301	85	31.5	406	20	0.54	24
17		26.54	29.98	10.9	0.303	85	31.5	397	20	0.48	24
18		26.75	31.00	10.8	0.304	85	31.3	433	30	0.53	24
19		26.64	30.18	10.8	0.297	85	31.5	391	20	0.52	24
20		27.52	32.23	11.0	0.312	88	31.4	415	22	0.67	24
21	31.00	26.70	31.05	10.8	0.301	85	31.3	396	20	0.61	24
22		26.80	32.90	10.7	0.306	88	31.3	404	20	0.62	32
23		24.70	30.95	10.7	0.297	85	31.2	423	40	0.47	32
24		27.98	31.56	10.9	0.311	88	31.2	409	20	0.49	32
25		26.34	30.33	10.6	0.290	82	31.2	438	40	0.37	32
26		27.46	30.85	11.0	0.306	88	31.2	403	20	0.46	32
27		25.88	30.39	10.7	0.313	88	31.1	436	30	0.47	32
28		26.23	31.97	10.8	0.308	88	31.2	409	20	0.47	32
29		26.54	30.95	10.8	0.301	85	31.1	406	22	0.42	32
30		27.31	30.90	10.9	0.300	85	31.1	399	20	0.41	32
31		25.21	31.77	10.9	0.301	85	31.2	451	70	0.42	32
32		25.16	31.15	10.6	0.305	85	31.1	430	30	0.39	32
33		25.62	29.36	10.9	0.304	85	31.1	447	51	0.41	32
34		25.82	30.33	11.0	0.310	88	31.1	424	31	0.45	32
35		27.62	31.21	11.0	0.294	82	31.1	399	20	0.37	32
36		25.47	30.39	10.6	0.311	88	31.1	464	67	0.40	32
37	31.50	25.93	29.72	10.9	0.313	88	31.1	407	20	0.41	32

STOP: 16:50:43

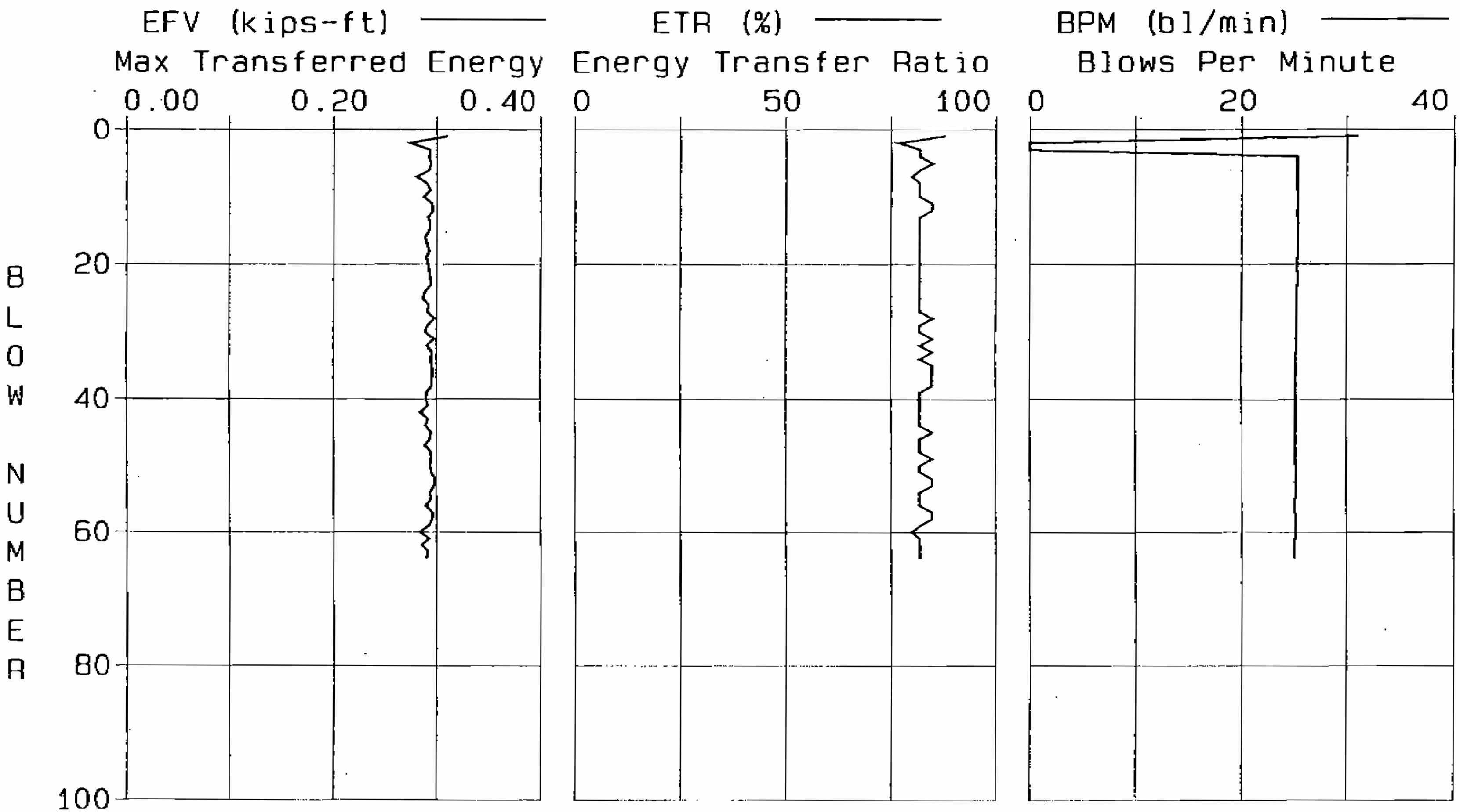
	CSX	CSI	VMX	EFV	ETR	BPM	EF2	RAT	DMX
AVG	26.40	31.12	10.9	0.305	86	31.3	417	28	0.50
STD	0.86	0.86	0.1	0.006	2	0.2	19	14	0.10
MAX	27.98	32.90	11.2	0.314	88	31.6	464	70	0.69
#BLS	28	28	28	28	28	28	28	28	28

DRIVE TIME SUMMARY (2003-Sep-14 : A6031FT.MDF)

	DRIVE minutes	WAIT
BN 4 -> 37, START 16:49:36 -> 16:50:41 STOP,	1.08	
Total Time	1.08 minutes	0.00

A Miner Dynamic Test 2003-Sep-19

KNIK ARM, A10 66 FT, 3.5"X3/16", AUTO HAMMER



Robert Miner Dynamic Testing, Inc.

Pile: A10 66 FT
Info: 3.5"X3/16", AUTO HAMMER
AR: 2.0 in^2
LE: 72.3 ft

Proj: KNIK ARM
SP: 0.492 k/ft^3
WS: 16800 ft/s
EM: 29948 KSI

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
ETR: Energy Transfer Ratio
EFV: Max Transferred Energy
EF2: Energy by F^2 Method

BPM: Blows Per Minute
RAT: Reflection time Ratio
VMX: Max Measured Velocity
FMX: Max Measured Force

BL#	depth ft	CSX ksi	CSI ksi	ETR %	EFV kips-ft	EF2 lb-ft	BPM bl/min	RAT %	VMX ft/sec	FMX kips	BLC bl/ft
17		25.62	29.05	82	0.291	508	25.4	11	11.1	50.0	42
18		25.21	28.80	82	0.293	500	25.4	11	11.1	49.2	42
19		25.52	29.10	82	0.290	507	25.3	11	11.2	49.8	42
20		25.72	29.10	82	0.292	508	25.3	10	11.3	50.2	42
21		25.21	29.26	82	0.293	506	25.3	10	11.2	49.2	42
22		25.11	28.49	82	0.294	493	25.4	11	10.9	49.0	42
23		25.16	29.26	82	0.295	508	25.3	11	11.2	49.1	42
24		25.62	29.16	82	0.289	501	25.3	10	11.1	50.0	42
25		24.70	28.95	82	0.287	495	25.3	11	10.9	48.2	42
26		25.82	28.90	82	0.293	507	25.3	11	11.1	50.4	42
27		25.82	28.75	82	0.291	508	25.2	10	11.4	50.4	42
28		25.16	28.95	85	0.298	506	25.3	10	11.1	49.1	42
29		25.62	29.26	82	0.292	513	25.3	10	11.3	50.0	42
30		25.47	29.10	82	0.289	504	25.3	11	11.1	49.7	42
31		25.06	28.49	85	0.298	498	25.2	11	10.9	48.9	42
32		25.93	29.51	82	0.291	511	25.3	11	11.2	50.6	42
33		25.93	29.51	85	0.296	527	25.2	10	11.4	50.6	42
34		25.57	29.31	82	0.295	508	25.2	11	11.1	49.9	42
35		25.47	29.31	85	0.296	510	25.2	10	11.3	49.7	42
36		25.67	29.36	85	0.297	518	25.2	10	11.3	50.1	42
37	67.00	25.72	29.57	85	0.296	512	25.2	10	11.4	50.2	42
38		25.62	29.72	85	0.296	514	25.2	10	11.4	50.0	54
39		25.36	28.95	82	0.291	500	25.2	11	11.2	49.5	54
40		25.72	29.26	82	0.290	507	25.2	11	11.1	50.2	54
41		25.93	29.31	82	0.292	516	25.2	10	11.4	50.6	54
42		24.85	28.80	82	0.285	480	25.2	11	10.9	48.5	54
43		25.36	29.62	82	0.292	512	25.2	10	11.3	49.5	54
44		25.47	29.41	82	0.290	507	25.2	10	11.2	49.7	54
45		25.57	29.57	85	0.295	507	25.1	10	11.3	49.9	54
46		25.36	29.62	82	0.294	519	25.2	10	11.4	49.5	54
47		25.57	28.95	82	0.289	502	25.1	11	11.1	49.9	54
48		26.18	29.16	82	0.295	511	25.2	10	11.5	51.1	54
49		25.26	28.80	85	0.295	514	25.1	10	11.3	49.3	54
50		25.57	28.69	82	0.294	506	25.2	10	11.2	49.9	54
51		25.06	29.72	82	0.295	518	25.2	10	11.3	48.9	54
52		25.82	28.80	85	0.299	513	25.2	10	11.3	50.4	54
53		25.26	30.18	85	0.298	521	25.2	10	11.2	49.3	54
54		25.98	29.72	82	0.294	513	25.2	10	11.3	50.7	54
55		25.57	29.67	82	0.295	509	25.1	10	11.2	49.9	54
56		25.82	29.51	82	0.290	503	25.1	10	11.1	50.4	54
57		25.47	29.46	85	0.296	506	25.1	10	10.9	49.7	54
58		25.72	29.98	85	0.296	510	25.2	10	11.3	50.2	54
59		25.16	29.46	82	0.293	510	25.2	10	11.1	49.1	54
60		25.26	28.95	80	0.284	491	25.2	11	10.9	49.3	54
61		25.77	29.51	82	0.293	507	25.1	10	11.1	50.3	54
62		25.41	28.95	82	0.286	494	25.1	11	10.9	49.6	54

Info: 3.5"X3/16", AUTO HAMMER

BL#	depth ft	CSX ksi	CSI ksi	ETR %	EFV kips-ft	EF2 lb-ft	BPM bl/min	RAT %	VMX ft/sec	FMX kips	BLC bl/ft
63		25.41	29.98	82	0.292	511	25.1	10	11.2	49.6	54
64	67.50	24.80	30.03	82	0.290	502	25.1	10	11.0	48.4	54

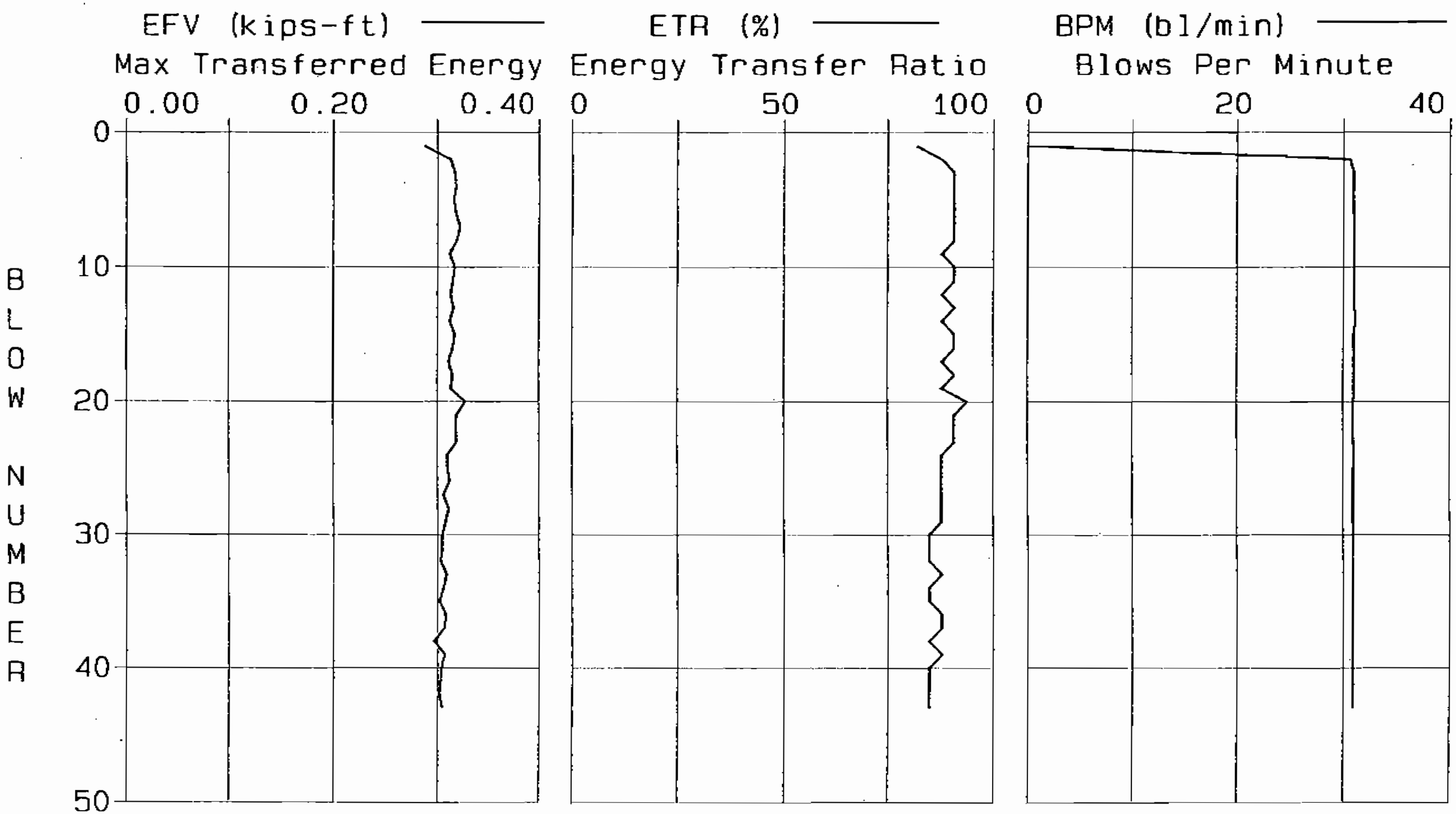
STOP: 12:21:05

	CSX	CSI	ETR	EFV	EF2	BPM	RAT	VMX	FMX
AVG	25.49	29.27	83	0.293	507	25.2	10	11.2	49.7
STD	0.32	0.40	1	0.003	8	0.1	0	0.2	0.6
MAX	26.18	30.18	85	0.299	527	25.4	11	11.5	51.1
#BLS	48	48	48	48	48	48	48	48	48

DRIVE TIME SUMMARY (2003-Sep-19 : A10.MDF)	DRIVE minutes	WAIT
BN 2 -> 64, START 12:17:53 -> 12:21:05 STOP,	3.20	
Total Time	3.20 minutes	0.00

A Miner Dynamic Test 2003-Sep-19

KNIK ARM, A10 88.5 FT, 3.5"X3/16", AUTO HAMMER



Robert Miner Dynamic Testing, Inc.

Pile: A10 88.5 FT
Info: 3.5"X3/16", AUTO HAMMER
AR: 2.0 in^2
LE: 148.8 ft

Proj: KNIK ARM
SP: 0.492 k/ft^3
WS: 16800 ft/s
EM: 29948 KSI

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
ETR: Energy Transfer Ratio
EFV: Max Transferred Energy
EF2: Energy by F^2 Method

BPM: Blows Per Minute
RAT: Reflection time Ratio
VMX: Max Measured Velocity
FMX: Max Measured Force

BL#	depth ft	CSX ksi	CSI ksi	ETR %	EFV kips-ft	EF2 lb-ft	BPM bl/min	RAT %	VMX ft/sec	FMX kips	BLC bl/ft
6		27.00	29.26	91	0.318	692	31.0	9	11.2	52.7	0
7		28.13	29.87	91	0.322	725	31.1	9	11.5	54.9	0
8		28.44	30.18	91	0.319	719	31.1	9	11.5	55.5	0
9		28.08	29.46	88	0.312	695	31.1	9	11.2	54.8	0
10		27.93	31.41	91	0.317	717	31.1	9	11.5	54.5	0
11		28.03	29.67	91	0.315	708	31.1	9	11.3	54.7	0
12		27.87	30.54	88	0.313	693	31.1	9	11.1	54.4	0
13		27.82	29.87	91	0.316	711	31.1	9	11.4	54.3	0
14		27.16	29.57	88	0.312	691	31.2	9	11.2	53.0	0
15		27.57	31.15	91	0.317	708	31.0	9	11.5	53.8	0
16		27.72	29.51	91	0.315	696	31.1	9	11.1	54.1	0
17		27.16	28.75	88	0.311	685	31.1	9	11.1	53.0	0
18	89.50	27.93	30.80	91	0.315	709	31.1	9	11.4	54.5	0
19		27.72	29.87	88	0.313	698	31.1	9	11.1	54.1	0
20		27.77	29.82	94	0.327	727	31.0	9	11.4	54.2	0
21		27.98	29.57	91	0.319	721	31.0	9	11.4	54.6	0
22		28.08	30.08	91	0.319	717	31.0	9	11.3	54.8	0
23		27.77	29.26	91	0.319	702	31.0	9	11.2	54.2	0
24		28.28	31.31	88	0.310	698	31.1	9	11.0	55.2	0
25		27.31	30.08	88	0.310	684	31.0	9	11.0	53.3	0
26		27.21	30.59	88	0.312	684	31.0	9	10.9	53.1	0
27		27.52	30.69	88	0.306	666	31.0	9	10.5	53.7	0
28		26.85	30.13	88	0.311	684	31.0	9	11.1	52.4	0
29		27.52	31.00	88	0.308	677	31.0	9	10.8	53.7	0
30		26.39	30.28	85	0.305	676	31.0	9	10.8	51.5	0
31		26.59	30.13	85	0.305	673	31.0	9	10.6	51.9	0
32		26.75	31.36	85	0.303	669	30.9	9	10.5	52.2	0
33		27.05	31.31	88	0.309	678	31.0	9	10.5	52.8	0
34		26.29	30.69	85	0.306	673	31.0	9	10.6	51.3	0
35		25.88	30.74	85	0.302	667	30.9	9	10.4	50.5	0
36		27.16	31.15	88	0.308	671	31.0	9	10.4	53.0	0
37		26.03	30.80	88	0.307	666	30.9	9	10.5	50.8	0
38		25.47	30.13	85	0.297	632	31.0	9	10.4	49.7	0
39		26.70	31.36	88	0.307	668	31.0	9	10.5	52.1	0
40		26.70	31.15	85	0.304	673	31.0	9	10.4	52.1	0
41		26.34	31.00	85	0.303	670	31.0	9	10.3	51.4	0
42		25.82	30.49	85	0.302	643	31.0	9	10.1	50.4	0
43	90.00	26.59	31.46	85	0.305	679	31.0	9	10.6	51.9	0

STOP: 16:11:18

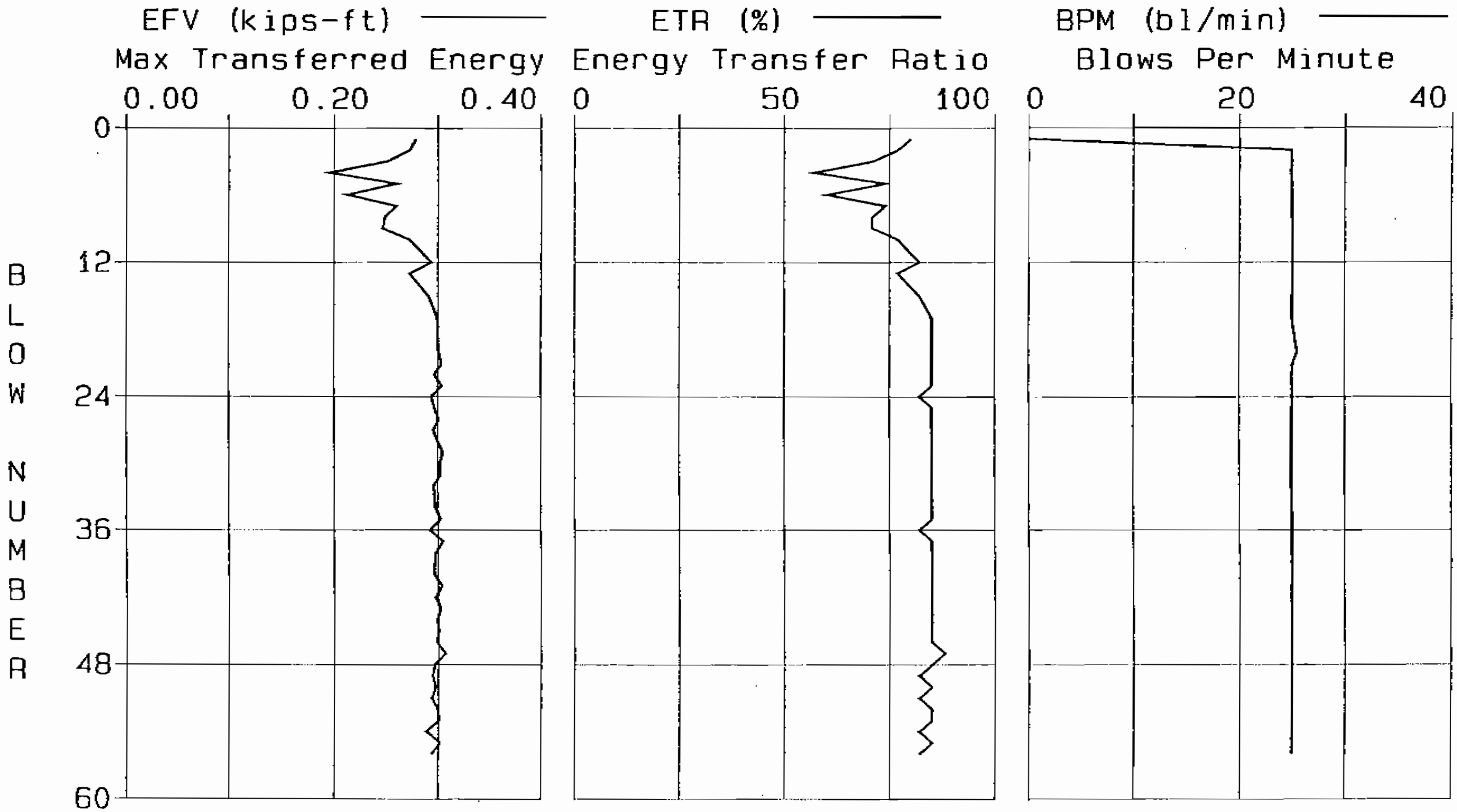
Pile: A10 88.5 FT   Proj: KNIK ARM   Pg2
Info: 3.5"X3/16", AUTO HAMMER

	CSX	CSI	ETR	EFV	EF2	BPM	RAT	VMX	FMX
AVG	27.23	30.38	88	0.311	688	31.0	9	10.9	53.1
STD	0.77	0.72	2	0.007	22	0.1	0	0.4	1.5
MAX	28.44	31.46	94	0.327	727	31.2	9	11.5	55.5
#BLS	38	38	38	38	38	38	38	38	38

DRIVE TIME SUMMARY (2003-Sep-19 : A10885FT.Q00)	DRIVE	WAIT
	----- minutes	----
BN 1 -> 43, START 16:09:57 -> 16:11:18 STOP,	1.35	
Total Time	1.35 minutes	0.00

R Miner Dynamic Test 2003-Sep-19

KNIK ARM, A10 126 FT, 3.5"X3/16", AUTO HAMMER



Robert Miner Dynamic Testing, Inc.

Pile: A10 126 FT
Info: 3.5"X3/16", AUTO HAMMER
AR: 2.0 in^2
LE: 208.8 ft

Proj: KNIK ARM
SP: 0.492 k/ft^3
WS: 16800 ft/s
EM: 29948 KSI

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
ETR: Energy Transfer Ratio
EFV: Max Transferred Energy
EF2: Energy by F^2 Method
BPM: Blows Per Minute
RAT: Reflection time Ratio
VMX: Max Measured Velocity
FMX: Max Measured Force

BL#	depth ft	CSX ksi	CSI ksi	ETR %	EFV kips-ft	EF2 lb-ft	BPM bl/min	RAT %	VMX ft/sec	FMX kips	BLC bl/ft
17		24.54	28.13	85	0.300	491	25.0	12	9.1	47.9	32
20		24.59	27.21	85	0.301	482	25.5	7	11.1	48.0	32
21		25.06	27.82	85	0.304	501	25.1	7	11.3	48.9	32
22		24.85	27.26	85	0.297	476	25.0	7	10.7	48.5	32
23		25.06	27.98	85	0.304	490	25.1	7	11.0	48.9	32
24		24.85	27.36	82	0.294	472	25.1	7	10.7	48.5	32
25		24.80	27.72	85	0.297	481	25.0	7	10.9	48.4	32
26		25.31	28.34	85	0.301	500	25.0	7	11.4	49.4	32
27		25.57	27.16	85	0.296	483	25.0	7	10.8	49.9	32
28		24.95	27.36	85	0.300	480	25.0	7	10.6	48.7	32
29		25.41	28.13	85	0.305	494	25.0	7	11.1	49.6	32
30		25.67	28.34	85	0.302	491	25.0	7	11.1	50.1	32
31	127.00	25.16	27.87	85	0.303	481	25.0	7	10.9	49.1	32
32		25.11	27.87	85	0.296	479	25.0	7	10.8	49.0	50
33		25.67	27.93	85	0.297	490	25.0	7	10.9	50.1	50
34		25.47	28.03	85	0.298	486	25.0	7	10.9	49.7	50
35		26.08	27.16	85	0.303	501	25.1	12	11.0	50.9	50
36		26.23	27.62	82	0.292	469	25.0	7	10.5	51.2	50
37		26.13	27.46	85	0.305	500	25.0	12	10.9	51.0	50
38		25.52	28.90	85	0.298	490	25.0	7	11.0	49.8	50
39		25.82	27.67	85	0.297	490	25.0	12	10.8	50.4	50
40		24.90	28.28	85	0.297	487	25.1	7	10.8	48.6	50
41		26.08	27.52	85	0.304	507	25.0	12	11.0	50.9	50
42		25.77	28.80	85	0.298	497	25.0	12	11.0	50.3	50
43		25.26	28.23	85	0.303	489	25.0	7	11.1	49.3	50
44		25.88	29.00	85	0.299	501	25.0	22	10.9	50.5	50
45		25.72	29.05	85	0.301	503	25.0	12	11.1	50.2	50
46		25.88	27.77	85	0.299	489	25.0	7	10.9	50.5	50
47		25.67	30.23	88	0.307	513	25.0	7	11.4	50.1	50
48		25.98	27.93	85	0.297	497	25.0	12	10.8	50.7	50
49		25.47	28.39	82	0.295	474	25.0	7	10.7	49.7	50
50		25.88	29.05	85	0.298	496	25.0	12	10.9	50.5	50
51		25.52	27.62	82	0.294	479	25.0	7	10.8	49.8	50
52		25.67	28.80	85	0.300	500	25.0	7	11.0	50.1	50
53		25.82	28.34	85	0.301	503	25.0	12	11.0	50.4	50
54		25.16	28.34	82	0.289	465	25.0	7	10.5	49.1	50
55		26.03	27.93	85	0.302	510	25.0	12	11.1	50.8	50
56	127.50	25.41	28.95	82	0.294	485	25.0	7	10.9	49.6	50

STOP: 23:08:28

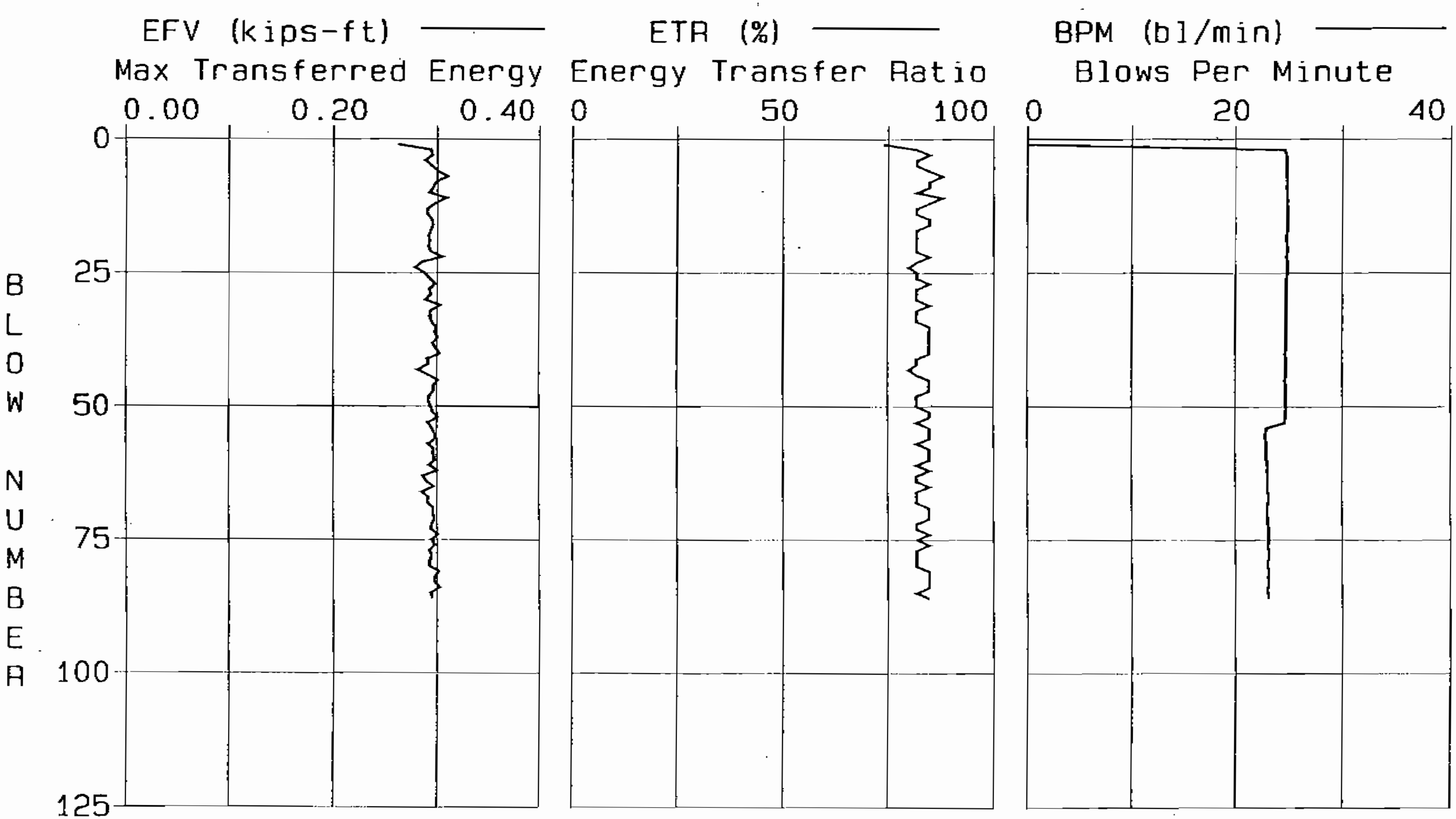
Info: 3.5"X3/16", AUTO HAMMER

	CSX	CSI	ETR	EFV	EF2	BPM	RAT	VMX	FMX
AVG	25.47	28.09	85	0.299	490	25.0	9	10.9	49.7
STD	0.45	0.66	1	0.004	11	0.1	3	0.4	0.9
MAX	26.23	30.23	88	0.307	513	25.5	22	11.4	51.2
#BLS	38	38	38	38	38	38	38	38	38

DRIVE TIME SUMMARY (2003-Sep-19 : A10126FT.Q00)	DRIVE (minutes)	WAIT
BN 1 -> 56, START 23:06:16 -> 23:08:28 STOP,	2.20	
Total Time	2.20 minutes	0.00

A Miner Dynamic Test 2003-Sep-20

KNIK ARM, A10 156 FT, 3.5"X3/16, AUTO HAMMER



Robert Miner Dynamic Testing, Inc.

Pile: A10 156 FT
Info: 3.5"X3/16, AUTO HAMMER
AR: 2.0 in^2
LE: 238.8 ft

Proj: KNIK ARM
SP: 0.492 k/ft^3
WS: 16800 ft/s
EM: 29948 KSI

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
ETR: Energy Transfer Ratio
EFV: Max Transferred Energy
EF2: Energy by F^2 Method

BPM: Blows Per Minute
RAT: Reflection time Ratio
VMX: Max Measured Velocity
FMX: Max Measured Force

BL#	depth	CSX	CSI	ETR	EFV	EF2	BPM	RAT	VMX	FMX	BLC
	ft	ksi	ksi	%	kips-ft	lb-ft	bl/min	%	ft/sec	kips	bl/ft
15		27.82	29.51	85	0.295	539	24.9	7	11.4	54.3	60
16		26.80	30.39	85	0.296	535	24.9	7	11.0	52.3	60
17		27.67	29.77	82	0.294	540	24.9	7	11.2	54.0	60
18		27.11	29.51	82	0.292	523	24.8	6	11.0	52.9	60
19		26.95	29.51	82	0.293	529	24.8	7	11.0	52.6	60
20		27.11	29.46	82	0.292	525	24.9	7	11.2	52.9	60
21		26.95	29.87	82	0.294	531	24.9	6	11.0	52.6	60
22		27.67	30.69	85	0.305	559	24.8	6	11.4	54.0	60
23		26.23	29.16	82	0.286	506	24.9	7	10.5	51.2	60
24		26.64	28.28	80	0.279	503	24.9	7	11.0	52.0	60
25		26.59	29.05	82	0.288	518	24.9	7	10.9	51.9	60
26		27.41	29.05	82	0.293	535	24.8	7	11.4	53.5	60
27		26.80	29.92	85	0.298	539	24.8	6	11.1	52.3	60
28		26.80	29.05	82	0.293	525	24.8	6	10.9	52.3	60
29		26.90	28.95	82	0.294	541	24.8	6	11.2	52.5	60
30		27.21	29.05	82	0.289	519	24.8	7	11.1	53.1	60
31		27.16	30.23	85	0.303	545	24.8	6	11.3	53.0	60
32		27.11	28.80	82	0.294	531	24.8	7	11.2	52.9	60
33		26.80	29.67	82	0.293	532	24.8	6	11.1	52.3	60
34		26.85	29.57	82	0.295	535	24.8	6	11.1	52.4	60
35		26.59	29.57	85	0.299	542	24.8	6	11.2	51.9	60
36		27.21	29.77	85	0.298	542	24.8	7	11.3	53.1	60
37		27.36	30.39	85	0.301	548	24.8	7	11.2	53.4	60
38		27.11	30.44	85	0.296	537	24.8	7	11.2	52.9	60
39		27.11	29.72	85	0.299	546	24.8	7	11.3	52.9	60
40		27.36	29.62	85	0.303	545	24.7	7	11.5	53.4	60
41		26.54	31.00	82	0.291	525	24.8	7	10.8	51.8	60
42		26.49	29.05	82	0.292	536	24.8	6	11.0	51.7	60
43		26.75	28.64	80	0.282	512	24.8	7	11.1	52.2	60
44	157.00	27.11	29.36	82	0.292	531	24.8	6	11.2	52.9	60
45		27.52	30.33	85	0.301	544	24.7	7	11.4	53.7	84
46		27.00	29.46	85	0.297	532	24.8	7	11.2	52.7	84
47		26.49	29.26	85	0.297	533	24.8	7	11.1	51.7	84
48		26.70	29.21	82	0.293	530	24.8	7	11.2	52.1	84
49		26.29	28.90	82	0.292	525	24.8	7	11.1	51.3	84
50		26.70	29.57	82	0.294	527	24.8	7	10.9	52.1	84
51		26.34	29.46	85	0.296	539	24.8	7	10.9	51.4	84
52		26.39	30.54	85	0.301	540	24.7	7	11.2	51.5	84
53		25.88	29.67	82	0.292	524	24.7	7	10.9	50.5	84
54		26.85	30.39	85	0.295	531	23.0	6	10.9	52.4	84
55		27.31	30.59	85	0.298	533	22.9	7	11.0	53.3	84
56		26.75	31.82	85	0.299	541	22.9	7	10.8	52.2	84
57		26.39	30.08	82	0.292	528	23.0	7	11.0	51.5	84
58		27.00	30.74	85	0.297	538	22.9	7	11.2	52.7	84
59		27.16	30.08	85	0.297	542	23.0	6	11.4	53.0	84
60		26.13	30.69	85	0.298	535	23.0	6	11.0	51.0	84

BL#	depth ft	CSX ksi	CSI ksi	ETR %	EFV kips-ft	EF2 lb-ft	BPM bl/min	RAT %	VMX ft/sec	FMX kips	BLC bl/ft
61		26.59	29.67	82	0.293	532	23.1	7	11.1	51.9	84
62		27.46	29.98	85	0.300	550	23.0	7	11.4	53.6	84
63		25.72	30.49	82	0.287	515	23.1	7	10.6	50.2	84
64		26.70	28.80	82	0.290	517	23.1	7	10.8	52.1	84
65		27.05	31.31	85	0.297	541	23.1	7	11.2	52.8	84
66		26.23	29.36	82	0.286	514	23.1	7	10.9	51.2	84
67		26.80	29.92	82	0.292	529	23.2	7	11.1	52.3	84
68		26.18	30.90	82	0.291	524	23.1	6	10.5	51.1	84
69		26.85	30.69	85	0.297	534	23.1	7	11.0	52.4	84
70		25.82	30.69	85	0.296	533	23.1	6	10.8	50.4	84
71		27.67	30.39	85	0.298	540	23.1	7	11.2	54.0	84
72		26.95	30.80	82	0.295	536	23.1	7	10.9	52.6	84
73		26.29	31.31	82	0.294	531	23.2	7	10.7	51.3	84
74		27.57	30.54	85	0.301	545	23.2	7	11.3	53.8	84
75		27.21	30.49	82	0.295	534	23.2	7	11.0	53.1	84
76		26.90	30.90	85	0.298	536	23.1	6	11.1	52.5	84
77		26.34	30.49	82	0.293	524	23.2	6	11.0	51.4	84
78		27.26	30.95	82	0.295	535	23.2	7	11.1	53.2	84
79		26.75	31.36	82	0.293	530	23.1	7	11.1	52.2	84
80		26.54	29.98	82	0.293	524	23.1	7	10.8	51.8	84
81		27.62	30.28	85	0.302	539	23.2	6	11.1	53.9	84
82		26.90	30.80	85	0.298	538	23.1	7	11.0	52.5	84
83		27.57	30.13	85	0.298	538	23.2	7	11.1	53.8	84
84		26.23	29.67	85	0.303	547	23.1	6	11.3	51.2	84
85		27.00	30.23	82	0.294	531	23.1	7	11.1	52.7	84
86	157.50	26.85	31.10	85	0.296	531	23.2	6	10.9	52.4	84

STOP: 03:07:38

	CSX	CSI	ETR	EFV	EF2	BPM	RAT	VMX	FMX
AVG	26.86	29.99	83	0.295	533	24.0	7	11.1	52.4
STD	0.47	0.75	2	0.005	10	0.9	0	0.2	0.9
MAX	27.82	31.82	85	0.305	559	24.9	7	11.5	54.3
#BLS	72	72	72	72	72	72	72	72	72

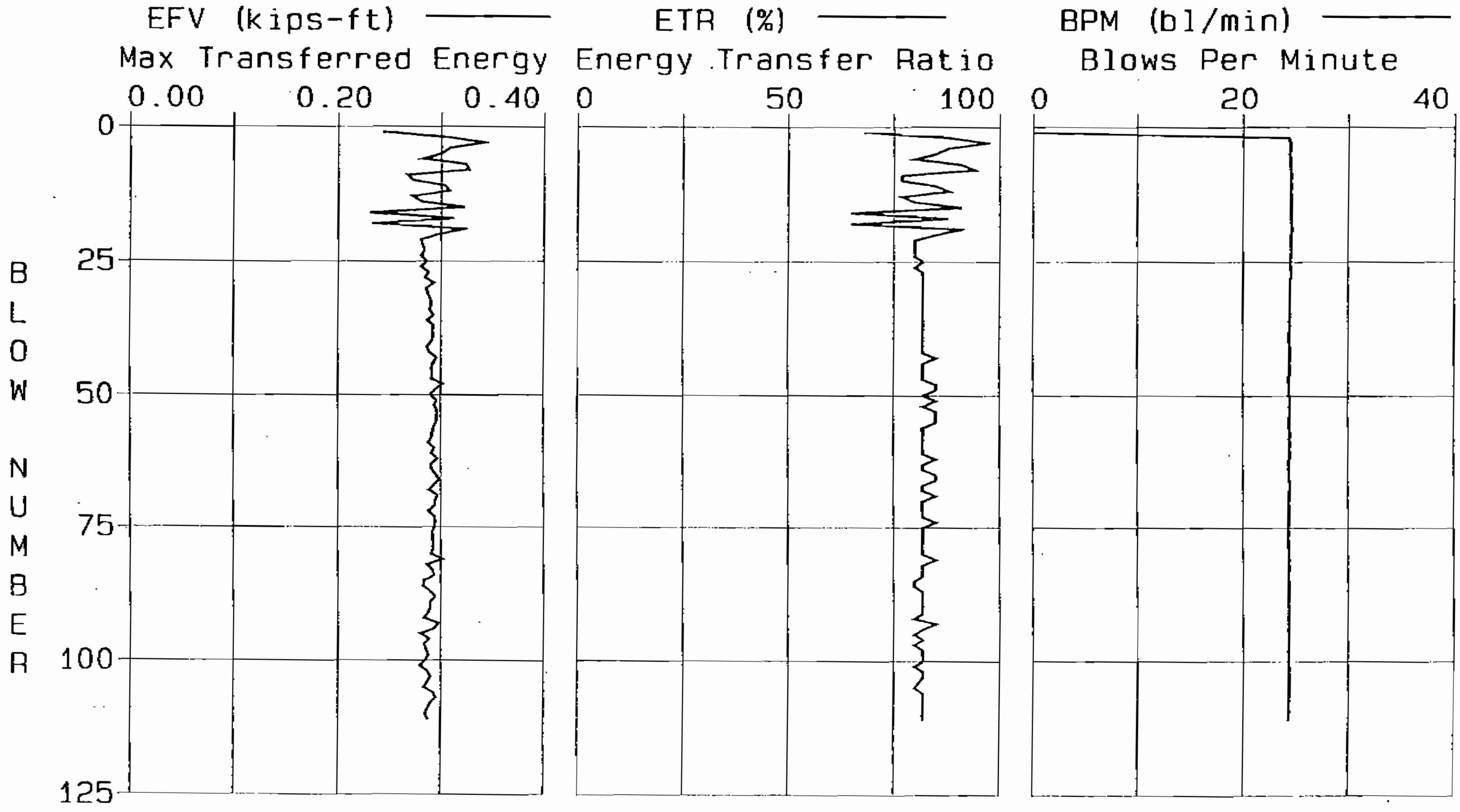
DRIVE TIME SUMMARY (2003-Sep-20 : A10156FT.MDF) DRIVE ----- minutes WAIT ----

BN. 1 -> 86, START 3:04:07 -> 3:07:38 STOP, 3.52

Total Time 3.52 minutes 0.00

A Miner Dynamic Test 2003-Sep-20

KNIK ARM , A10 166 FT, 3.5"X3/16", AUTO HAMMER



Robert Miner Dynamic Testing, Inc.

Pile: A10 166 FT
Info: 3.5"X3/16", AUTO HAMMER
AR: 2.0 in^2
LE: 248.8 ft

Proj: KNIK ARM
SP: 0.492 k/ft^3
WS: 16800 ft/s
EM: 29948 KSI

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
ETR: Energy Transfer Ratio
EFV: Max Transferred Energy
EF2: Energy by F^2 Method

BPM: Blows Per Minute
RAT: Reflection time Ratio
VMX: Max Measured Velocity
FMX: Max Measured Force

BL#	depth ft	CSX ksi	CSI ksi	ETR %	EFV kips-ft	EF2 lb-ft	BPM bl/min	RAT %	VMX ft/sec	FMX kips	BLC bl/ft
28		25.93	28.28	82	0.285	512	24.5	6	10.1	50.6	0
29		26.85	28.28	82	0.293	524	24.5	6	10.3	52.4	0
30		26.03	29.67	82	0.286	522	24.5	6	10.4	50.8	0
31		25.47	27.87	82	0.287	504	24.5	6	9.9	49.7	0
32		26.64	28.80	82	0.290	530	24.5	6	10.2	52.0	0
33		26.13	28.49	82	0.291	527	24.5	6	10.6	51.0	0
34		26.64	28.23	82	0.289	525	24.5	6	10.5	52.0	0
35		25.77	27.93	82	0.293	524	24.5	6	10.5	50.3	0
36		26.64	28.69	82	0.287	510	24.4	6	10.1	52.0	0
37		26.39	28.49	82	0.293	531	24.5	6	10.6	51.5	0
38		25.98	28.34	82	0.292	526	24.4	6	10.6	50.7	0
39		26.44	28.23	82	0.293	534	24.4	6	10.5	51.6	0
40		26.18	28.03	82	0.291	516	24.4	6	10.1	51.1	0
41		26.13	28.39	82	0.287	518	24.5	6	10.2	51.0	0
42		26.18	27.52	82	0.289	523	24.5	6	10.3	51.1	0
43		27.16	29.67	85	0.296	538	24.5	6	10.4	53.0	0
44		26.23	29.05	82	0.293	534	24.4	6	10.6	51.2	0
45		26.03	28.23	82	0.291	527	24.5	6	10.4	50.8	0
46		26.54	28.59	82	0.292	522	24.4	6	9.9	51.8	0
47		26.39	28.59	82	0.291	531	24.4	6	10.6	51.5	0
48		26.80	29.00	85	0.302	542	24.4	6	10.8	52.3	0
49		26.64	27.67	85	0.295	532	24.4	6	10.5	52.0	0
50		27.46	28.49	82	0.290	532	24.4	6	10.1	53.6	0
51		27.00	28.49	85	0.296	545	24.5	6	10.7	52.7	0
52		26.08	28.64	82	0.294	524	24.4	6	10.2	50.9	0
53		26.64	29.16	85	0.297	540	24.5	6	10.7	52.0	0
54		25.98	29.36	85	0.296	524	24.4	6	10.5	50.7	0
55		26.80	29.00	85	0.296	543	24.4	6	10.6	52.3	0
56		27.26	28.13	82	0.293	534	24.4	6	10.4	53.2	0
57		26.59	29.10	82	0.292	532	24.5	6	10.5	51.9	0
58		27.31	28.95	82	0.291	536	24.4	6	10.1	53.3	0
59		26.64	28.18	82	0.288	514	24.5	6	10.1	52.0	0
60	167.00	26.49	28.80	82	0.294	530	24.4	6	10.1	51.7	0
61		26.85	27.77	82	0.291	521	24.5	6	10.1	52.4	0
62		26.59	27.46	85	0.297	522	24.4	6	10.1	51.9	0
63		26.59	28.49	82	0.291	529	24.4	6	10.2	51.9	0
64		26.34	27.87	82	0.291	524	24.4	6	10.3	51.4	0
65		26.49	28.85	85	0.295	539	24.4	6	10.6	51.7	0
66		26.70	28.69	85	0.299	541	24.4	6	10.7	52.1	0
67		27.31	29.46	82	0.294	534	24.5	6	10.3	53.3	0
68		26.13	28.85	82	0.289	525	24.4	6	10.4	51.0	0
69		27.46	29.16	85	0.297	530	24.4	6	10.1	53.6	0
70		26.75	28.85	82	0.295	533	24.4	6	10.3	52.2	0
71		26.23	26.85	82	0.294	518	24.4	6	10.2	51.2	0
72		26.29	28.69	82	0.288	526	24.4	6	10.4	51.3	0
73		27.05	28.03	82	0.294	529	24.4	6	10.3	52.8	0

Info: 3.5"X3/16", AUTO HAMMER

BL#	depth ft	CSX ksi	CSI ksi	ETR %	EFV kips-ft	EF2 lb-ft	BPM bl/min	RAT %	VMX ft/sec	FMX kips	BLC bl/ft
74		26.85	27.67	85	0.295	535	24.4	6	10.6	52.4	0
75		26.49	29.51	82	0.294	536	24.3	6	10.4	51.7	0
76		26.29	28.18	82	0.292	519	24.4	6	10.2	51.3	0
77		27.21	29.26	82	0.293	540	24.3	6	10.1	53.1	0
78		26.44	27.62	82	0.293	527	24.4	6	10.6	51.6	0
79		26.44	29.21	82	0.293	534	24.4	6	10.3	51.6	0
80		26.29	29.57	82	0.291	533	24.4	6	10.2	51.3	0
81		26.39	28.64	85	0.302	544	24.4	6	10.8	51.5	0
82		26.39	29.46	82	0.287	521	24.4	6	10.3	51.5	0
83		26.54	27.77	82	0.292	526	24.4	6	10.4	51.8	0
84		26.39	29.05	82	0.294	536	24.3	6	10.5	51.5	0
85		26.85	28.90	80	0.284	522	24.4	6	10.0	52.4	0
86		25.88	29.00	80	0.283	507	24.4	6	9.9	50.5	0
87		26.39	27.46	82	0.290	516	24.4	6	9.9	51.5	0
88		26.44	28.80	82	0.295	536	24.4	6	10.7	51.6	0
89		25.98	29.16	82	0.290	522	24.3	6	10.3	50.7	0
90		25.77	29.31	82	0.290	525	24.4	6	10.3	50.3	0
91		26.23	28.59	82	0.288	518	24.4	6	10.0	51.2	0
92		26.13	28.95	80	0.284	517	24.4	6	10.1	51.0	0
93		27.46	28.90	85	0.298	534	24.4	6	10.1	53.6	0
94		27.21	28.90	82	0.294	541	24.3	6	10.5	53.1	0
95		25.72	30.39	80	0.281	512	24.4	6	9.9	50.2	0
96		26.39	30.23	82	0.289	526	24.4	6	10.0	51.5	0
97		26.18	29.41	80	0.284	516	24.4	6	9.9	51.1	0
98		26.18	29.62	82	0.286	526	24.3	6	10.2	51.1	0
99		26.75	29.51	82	0.288	531	24.4	6	10.3	52.2	0
100		25.88	29.31	82	0.285	515	24.3	6	10.1	50.5	0
101		26.29	29.77	80	0.280	513	24.4	6	9.6	51.3	0
102		26.39	29.87	82	0.287	529	24.4	6	10.2	51.5	0
103		26.34	29.77	82	0.290	531	24.4	6	10.5	51.4	0
105		26.03	28.75	80	0.284	506	24.4	6	9.7	50.8	0
106		26.03	28.54	82	0.293	511	24.4	6	9.6	50.8	0
107		25.88	28.34	82	0.295	518	24.4	6	10.0	50.5	0
108		26.39	30.74	82	0.290	527	24.3	6	10.4	51.5	0
109		25.57	29.51	82	0.287	517	24.4	6	10.0	49.9	0
110		26.08	30.59	82	0.285	518	24.3	6	10.0	50.9	0
111	167.50	26.49	29.92	82	0.288	525	24.3	6	10.1	51.7	0

STOP: 05:16:02

	CSX	CSI	ETR	EFV	EF2	BPM	RAT	VMX	FMX
AVG	26.44	28.79	82	0.291	526	24.4	6	10.3	51.6
STD	0.44	0.75	1	0.004	9	0.1	0	0.3	0.9
MAX	27.46	30.74	85	0.302	545	24.5	6	10.8	53.6
#BLS	83	83	83	83	83	83	83	83	83

DRIVE TIME SUMMARY (2003-Sep-20 : A10166FT.Q00) DRIVE WAIT

----- minutes ----

BN 1 -> 111, START 5:11:22 -> 5:16:02 STOP, 4.67

Total Time 4.67 minutes 0.00

APPENDIX F

LABORATORY TEST PROCEDURES AND RESULTS

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APPENDIX F
LABORATORY TEST PROCEDURES AND RESULTS

Laboratory tests were performed on selected soil samples from the borings to verify visual classifications and to determine those engineering characteristics pertinent to the design of piles for support of bridge pier loads. The following sections discuss each of the tests performed for the various properties required.

F-1 Classification Tests

All soil samples shipped to our laboratory were carefully examined and classified in the laboratory and their descriptions were checked against those in the field. These descriptions were used in the preparation of our final logs, Figures B-1 through B-16. The Unified Soil Classification System (ASTM D-2488 & 2487-90) was used throughout for all soils and a soil testing summary is presented in Table F-1. Criteria for the above Unified Soil Classification System is included as Table F-2.

Water Content Determinations

Following the visual classification of each soil sample, a portion of the material was taken, weighted and oven dried to determine the natural water content of the soil. The water contents, based on ASTM D-2216, are tabulated in Table F-1 and on the boring logs.

Density Determinations

Since a number of soil specimen were cut flat on the ends for uniaxial compression testing, density determinations are automatically obtained as a by product from these tests. In the preparation procedure, the ends of an approximately 6-inch high cylindrical specimen are square off, the height and diameter are measured, and the volume calculated. The specimen is then weighted to determine the wet unit weight. The results of these determinations are indicated in Table F-3.

Grain Size Analyses

Grain size analyses were conducted on 35 selected samples of the soil. The specimens were primarily granular in nature and were tested to obtain estimates of the material's silt/clay fines. The 35 grain size tests were performed in accordance with the test methods described in ASTM C-136. The results of these measurements are presented in the soil testing summary on Table F-1 and in detail as grain size plots in twelve sheets in Figure F-1.

Atterberg Limits

To aid in classifying and correlating the properties of the cohesive soils, Atterberg limit tests (liquid and plastic limits) were performed on 75 samples, which typically represented the various fine grained materials disclosed in the borings. Liquid limit tests were performed in accordance with ASTM D-423. Plastic limit tests followed ASTM D-424. The results of these tests are summarized on Table F-1, presented on the boring logs and in detail on plasticity charts in Figure F-2. The results in, Figure F-2, indicate that the soils have relatively low plasticity characteristics and mostly correspond to a CL symbol according to the Unified Soil Classification System. The plasticity index generally ranged between 5 and 25 percent.

F-2 Shear Strength Tests

The focus on strength testing was on the various clay units since it was one of the dominant materials encountered in the borings. The procedures used to determine the strength of the silty clay included pocket penetrometer and Torvane tests, unconfined compression tests, and unconsolidated undrained triaxial compression tests. Soil strengths were needed to estimate skin friction and end bearing capacities for the anticipated piles needed to support the bridge piers and for stability evaluations of possible causeways and shoreline embankments in approaches on either end of the bridge.

Pocket Penetrometer and Torvane Tests

These simple tests were performed on most of the more cohesive soil specimen in both disturbed and undisturbed samples. The pocket penetrometer is a small hand-held spring-calibrated ¼-inch cylindrical probe, which is slowly pushed into the clay specimen until ¼-inch penetration is achieved. The maximum reading is then taken and provides a quick reasonably reliable estimate of the unconfined compressive strength; which, if divided by 2, becomes comparable with the undrained shear strength. The results are presented on the boring logs in Appendix B and summarized on the Soils Testing Report on Table F-1. They are also selectively presented in Figure 9. Generally, the tests on the undisturbed samples are more reliable and less affected by disturbance and give higher strengths than tests on disturbed samples. The limit of this test is 4.5 tons per foot. Thus when the limit was exceeded the results are reported as > 4.5 tsf.

The Torvane is likewise a simple hand-held spring calibrated torsional device with about six small steel vanes on the end. In this test the vanes are pushed into the specimen and then torqued until failure by shearing results. The highest reading is then read and recorded as a direct estimate of the materials undrained shear strength. Similar to the pocket penetrometer, the higher readings usually occur on undisturbed samples and often low bound strengths are recorded if the sample is being tested is clay which is silty or sandy. Torvane testing in stiff to hard silts and clays has typically been found to provide lower readings than actual soil strengths, and are misleading. Therefore, although testing was performed on samples for this project, the results have not been shown on any tables or figures. The results will be kept on file in our office for future reference should they be needed.

Unconfined Compression Tests

Unconfined compression tests were performed on 31 of the more clayey specimens to generally estimate its intact compressive or undrained shear strength. The tests were performed in accordance with ASTM D-2166. In this test, the approximately 6-inch long by 2.8 inch diameter cylindrical specimen are squared off at the ends, placed in a compression machine, and loaded axially to failure. The results of these tests are summarized on Table F-1 and selectively on Figure 9. The actual stress strain curves for each test and a sketch depicting the mode of failure for each test are presented as Figures F-3.

Triaxial Compression Tests

Unconsolidated undrained triaxial compression tests were performed on 13 of the more silty and/or clayey specimens to generally estimate its intact strength. After preparation, each cylindrical specimen was encased in a rubber membrane and placed in a triaxial chamber. With the drain valve closed, each specimen was subjected to a predetermined confining pressure, generally a value estimated as the effective overburden pressure. With the pressure kept constant, the specimen was then loaded axially to failure with no drainage from the specimen allowed. The results of these tests are summarized in Table F-1. Plots of deviator stress (total stresses) vs. axial strain, and all pertinent specimen and test data are included as Figure F-4. In a number of cases, two specimens from a given sample were often prepared and then tested, one as an unconfined compression test specimen and one for triaxial testing. Mohr circles for the 13 triaxial specimens and their matching unconfined compression test, where performed, are summarized in Figure 10.

F-3 Consolidation Tests

One dimensional consolidation tests were performed on five undisturbed samples of the silty clays or gravelly, silty clays to attempt to estimate preconsolidation pressures on these mostly heavily overconsolidated soils. In this test, performed in a consolidometer, relatively undisturbed samples were first trimmed and fitted into a rigid ring. Porous stones were then placed on the top and bottom of the specimen to allow drainage and a vertical seating load of 0.25 tsf applied. The specimen was then loaded in doubling increments with each increment being held for about 2 hours to allow for consolidation to take place. The deflection time deflection curve under each load was plotted and the deflection at 100 minutes was used to reflect the amount of consolidation for each load. Figure F-5 presents the results of these tests for each specimen. These figures are in the form of deflection vs. log pressure plots with specimen details provided to produce percent settlement or void ratio verses log plots. Compression vs. log time curves for each load increment are available but not presented.

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			8	10	13	14.5	18	23
Test Hole No.			A-1	A-1	A-1	A-1	A-1	A-1
Field Sample No.			S1	S2	S3	S4	S5	S6
Date Sampled			August 16, 2003	August 16, 2003	August 16, 2003	August 16, 2003	August 16, 2003	August 16, 2003
Lab No.			A-1 S1	A-1 S2	A-1 S3	A-1 S4	A-1 S5	A-1 S6
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm	100.0%					
	1"	25mm	97.0%					
	0.75"	19mm	97.0%					
	0.5"	12.5mm	97.0%					
	0.375"	9.5mm	97.0%					
	0.25"	6.3mm	97.0%					
	#4	4.75mm	97.0%					
	#8	2.36mm	97.0%					
	#10	2mm						
	#16	1.18mm	97.0%					
	#30	0.6mm	97.0%					
	#40	0.425mm						
	#50	0.3mm	96.0%					
	#100	0.15mm	37.0%					
	#200	0.075mm	10.9%					
DOTTSD								
Liquid Limit								37
Plastic Index								18
Moisture Content %			26.0%	22.3%	28.0%	17.8%	20.0%	23.0%
Organic Content %								
% Gravel			3%					
% Sand			77%					
% Silt & Clay			20%					
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								2tsf
Pocket Pen Value				3	2.3	>4.5	3.7	3.3

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			28	30	35	40	45	50
Test Hole No.			A-1	A-1	A-1	A-1	A-1	A-1
Field Sample No.			S7	S8	S9	S10	S11	S12
Date Sampled			August 16, 2003	August 16, 2003	August 16, 2003	August 16, 2003	August 16, 2003	August 16, 2003
Lab No.			A1 S7	A1 S8	A1 S9	A1 S10	A1 S11	A1 S12
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm	NO RECOVERY					
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit					38	37		
Plastic Index					19	18		
Moisture Content %				23.0%	22.0%	24.0%	26.0%	23.7%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u					3 tsf			
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u					2.6 tsf	4.2 tsf		
Pocket Pen Value				3.5	3.5	3.2	3	3.5

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			55	60	65	70	75	80
Test Hole No.			A-1	A-1	A-1	A-1	A-1	A-1
Field Sample No.			S13	S14	S15	S16	S17	S18
Date Sampled			August 16, 2003	August 16, 2003	August 16, 2003	August 16, 2003	August 17, 2003	August 17, 2003
Lab No.			A1 S13	A1 S14	A1 S15	A1 S16	A1 S17	A1 S18
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			37	35				
Plastic Index			18	16				
Moisture Content %			24.3%	19.0%	25.3%			26.9%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u			3.9 tsf	1.4 tsf				
Pocket Pen Value			3.5	>4.5	4	3		2.75

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			85	90	95	100	105	110
Test Hole No.			A-1	A-1	A-1	A-1	A-1	A-1
Field Sample No.			S19	S20	S21	S22	S23	S24
Date Sampled			August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003
Lab No.			A1 S19	A1 S20	A1 S21	A1 S22	A1 S23	A1 S24
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			44			41		
Plastic Index			20			21		
Moisture Content %			29.0%		28.3%	33.0%		29.3%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u			1.8 tsf					
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u						1.5 tsf		
Pocket Pen Value			1.7	2.5	2.5	2.5	2.25	2

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			115	120	125	130	135	140
Test Hole No.			A-1	A-1	A-1	A-1	A-1	A-1
Field Sample No.			S25	S26	S27	S28	S29	S30
Date Sampled			August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003
Lab No.			A1 S25	A1 S26	A1 S27	A1 S28	A1 S29	A1 S30
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			39	44		39	46	
Plastic Index			19	21		19	23	
Moisture Content %			25.0%	25.0%	28.2%	26.0%	27.0%	28.1%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u				3.9 tsf			2.6 tsf	
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u			2.8 tsf			2.8 tsf	2.4 tsf	
Pocket Pen Value			2.25	3.7	2.5	2.3	2.5	2

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			145	150	157.5	165	172.5	180
Test Hole No.			A-1	A-1	A-1	A-1	A-1	A-1
Field Sample No.			S31	S32	S33	S34	S35	S36
Date Sampled			August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003
Lab No.			A1 S31	A1 S32	A1 S33	A1 S34	A1 S35	A1 S36
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit				43			43	39
Plastic Index				23			23	19
Moisture Content %				26.0%	26.0%		32.0%	26.9%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u				4.4 tsf			1.6 tsf	
Pocket Pen Value			2.5	2.75	2.5	2.75	1.75	2.5

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			18.5	195	202.5	210	217.5	225
Test Hole No.			A-1	A-1	A-1	A-1	A-1	A-1
Field Sample No.			S37	S38	S39	S40	S41	S42
Date Sampled			August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003
Lab No.			A1 S37	A1 S38	A1 S39	A1 S40	A1 S41	A1 S42
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			37			32		
Plastic Index			17			15		
Moisture Content %			22.6%		22.0%	21.0%		24.0%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u						2.2 tsf		
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u			2.4 tsf					
Pocket Pen Value			2.75	2.75	2.25	2		2.5

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			235	245	255	265	275	285
Test Hole No.			A-1	A-1	A-1	A-1	A-1	A-1
Field Sample No.			S43	S44	S45	S46	S47	S48
Date Sampled			August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003	August 17, 2003	August 18, 2003
Lab No.			A1 S43	A1 S44	A1 S45	A1 S46	A1 S47	A1 S48
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			37	34		24		22
Plastic Index			17	14		10		4
Moisture Content %			26.0%	25.0%	24.0%	16.8%	24.0%	24.4%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u			.57 tsf			4.4 tsf	1.2 tsf	
Pocket Pen Value			1.2	2.25	3.5	3.5	1.7	2.5

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			295	305	315	325	335	
Test Hole No.			A-1	A-1	A-1	A-1	A-1	
Field Sample No.			S49	S50	S51	S52	S53	
Date Sampled			August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003	
Lab No.			A1 S49	A1 S50	A1 S51	A1 S52	A1 S53	
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			30	30	27	36	27	
Plastic Index			12	14	12	15	11	
Moisture Content %			27.0%	16.0%	14.4%	21.7%	22.0%	
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u				3.3 tsf				
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u				3.5 tsf		1 tsf	4.7 tsf	
Pocket Pen Value			>4.5	3.75		>4.5	3.3	

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			6	11	16	21	26	31
Test Hole No.			A-2	A-2	A-2	A-2	A-2	A-2
Field Sample No.			S1	S2	S3	S4a	S4b	S5
Date Sampled			August 20, 2003	August 20, 2003	August 20, 2003	August 20, 2003	August 20, 2003	August 20, 2003
Lab No.			A2 S1	A2 S2	A2 S3	A2 S4a	A2 S4b	A2 S5
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm				100.0%		
	#40	0.425mm						
	#50	0.3mm				85.0%		
	#100	0.15mm				24.0%		
	#200	0.075mm				10.8%		
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			27.4%	28.7%	26.1%	27.4%	30.0%	31.2%
Organic Content %								
% Gravel								
% Sand						89.0%		
% Silt & Clay						11.0%	10.8%	
Coeff. Of Consolidation Cv								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			36	41	46	51	56	61
Test Hole No.			A-2	A-2	A-2	A-2	A-2	A-2
Field Sample No.			S6	S7	S8	S9	S10	S11
Date Sampled			August 20, 2003	August 20, 2003	August 20, 2003	August 20, 2003	August 20, 2003	August 20, 2003
Lab No.			A2 S6	A2 S7	A2 S8	A2 S9	A2 S10	A2 S11
Percent Passing Sieve Size	3"	75mm						
	2"	50mm	NO RECOVERY			NO RECOVERY		
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm		100.0%				
	#40	0.425mm						
	#50	0.3mm		90.0%				
	#100	0.15mm		20.0%				
	#200	0.075mm		8.3%				
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %				25.7%	24.8%		13.2%	29.8%
Organic Content %								
% Gravel								
% Sand				92%				
% Silt & Clay				8%				
Coeff. Of Consolidation Cv								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			61	71	78	85.5	95.5	105.5
Test Hole No.			A-2	A-2	A-2	A-2	A-2	A-2
Field Sample No.			S12	S13	S14	S15	S16	S17
Date Sampled			August 20, 2003	August 21, 2003	August 21, 2003	August 21, 2003	August 21, 2003	August 21, 2003
Lab No.			A2 S12	A2 S13	A2 S14	A2 S15	A2 S16	A2 S17
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm	100.0%	NO RECOVERY	100.0%		NO RECOVERY	
	#8	2.36mm	100.0%		100.0%			
	#10	2mm						
	#16	1.18mm	100.0%		100.0%			
	#30	0.6mm	100.0%		100.0%			
	#40	0.425mm						
	#50	0.3mm	95.0%		95.0%			
	#100	0.15mm	28.0%		44.0%			
	#200	0.075mm	15.5%		15.8%			
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			23.5%		28.2%	28.0%		27.9%
Organic Content %								
% Gravel								
% Sand			84.0%		84.0%			
% Silt & Clay			16.0%		16.0%	0.3%		
Coeff. Of Consolidation Cv								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			115.5	125	135	145	155	160
Test Hole No.			A-2	A-2	A-2	A-2	A-2	A-2
Field Sample No.			S18	S19	S20	S21	S22	S23
Date Sampled			August 21, 2003	August 21, 2003	August 21, 2003	August 21, 2003	August 21, 2003	August 21, 2003
Lab No.			A2 S18	A2 S19	A2 S20	A2 S21	A2 S22	A2 S23
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm	100.0%					
	0.5"	12.5mm	99.0%					
	0.375"	9.5mm	99.0%					
	0.25"	6.3mm	99.0%					
	#4	4.75mm	99.0%		100.0%			
	#8	2.36mm	97.0%		100.0%			
	#10	2mm						
	#16	1.18mm	90.0%		100.0%			
	#30	0.6mm	67.0%		99.0%			
	#40	0.425mm						
	#50	0.3mm	37.0%		92.0%			
	#100	0.15mm	15.0%		76.0%			
	#200	0.075mm	7.2%		42.3%			
DOTTSD								
Liquid Limit								32
Plastic Index								14
Moisture Content %			20.3%	29.3%	35.8%		25.0%	20.0%
Organic Content %								
% Gravel			1.0%					
% Sand			92.0%		58.0%			
% Silt & Clay			7.0%	0.8%	42.0%			
Coeff. Of Consolidation Cv								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								3.5

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			164	167	176	186	196	
Test Hole No.			A-2	A-2	A-2	A-2	A-2	
Field Sample No.			S24	S25	S26	S27	S28	
Date Sampled			August 21, 2003	August 21, 2003	August 21, 2003	August 21, 2003	August 21, 2003	
Lab No.			A2 S24	A2 S25	A2 S26	A2 S27	A2 S28	
Percent Passing Sieve Size	3"	75mm						
	2"	50mm	NO RECOVERY					
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit				23	29	28		
Plastic Index				8	12	9		
Moisture Content %					27.0%	22.0%		
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay						94.4%		
Coeff. Of Consolidation Cv								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u					2.5 tsf			
Pocket Pen Value				3.5	4		2.5	

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			5.5	6	11	16	21	26
Test Hole No.			A-4	A-4	A-4	A-4	A-4	A-4
Field Sample No.			S1	S2	S3	S4	S5	S6
Date Sampled			August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003
Lab No.			A4 S1	A4 S2	A4 S3	A4 S4	A4 S5	A4 S6
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm			100.0%			
	1"	25mm			90.0%			100.0%
	0.75"	19mm			87.0%			100.0%
	0.5"	12.5mm			80.0%			95.0%
	0.375"	9.5mm			76.0%			83.0%
	0.25"	6.3mm			71.0%			70.0%
	#4	4.75mm			67.0%			64.0%
	#8	2.36mm			61.0%			54.0%
	#10	2mm						
	#16	1.18mm			57.0%			46.0%
	#30	0.6mm			53.0%			41.0%
	#40	0.425mm						
	#50	0.3mm			46.0%			35.0%
	#100	0.15mm			39.0%			29.0%
	#200	0.075mm			30.6%			24.1%
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			5.2%	8.4%	8.2%	6.9%	7.9%	11.9%
Organic Content %								
% Gravel					33.0%			36.0%
% Sand					36.0%			40.0%
% Silt & Clay					31.0%			24.0%
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			31	36	37.5	41	42.5	51
Test Hole No.			A-4	A-4	A-4	A-4	A-4	A-4
Field Sample No.			S7	S8	S9	S10	S11	S12
Date Sampled			August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003
Lab No.			A4 S7	A4 S8	A4 S9	A4 S10	A4 S11	A4 S12
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm		100.0%				
	0.75"	19mm		83.0%				
	0.5"	12.5mm		46.0%				
	0.375"	9.5mm		25.0%				
	0.25"	6.3mm		17.0%				
	#4	4.75mm		11.0%				
	#8	2.36mm		8.4%				
	#10	2mm		5.0%				
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit					24	25	12	23
Plastic Index					8	11	1	9
Moisture Content %			13.6%	2.9%	1.1%	10.8%	11.0%	11.4%
Organic Content %								
% Gravel				92.0%				
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			56	66	73.5	81	87.5	96
Test Hole No.			A-4	A-4	A-4	A-4	A-4	A-4
Field Sample No.			S13	S14	S15	S16	S17	S18
Date Sampled			August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003	August 18, 2003
Lab No.			A4 S13	A4 S14	A4 S15	A4 S16	A4 S17	A4 S18
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			18		16			15
Plastic Index			6		5			0
Moisture Content %					13.0%	19.0%		11.4%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u						.84 tsf		
Pocket Pen Value			>4.5			4.5		

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			101	108.5	116	123.5	131	138.5
Test Hole No.			A-4	A-4	A-4	A-4	A-4	A-4
Field Sample No.			S19	S20	S21	S22	S23	S24
Date Sampled			August 19, 2003	August 19, 2003	August 19, 2003	August 19, 2003	August 19, 2003	August 19, 2003
Lab No.			A4 S19	A4 S20	A4 S21	A4 S22	A4 S23	A4 S24
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit					19		20	
Plastic Index					6		4	
Moisture Content %			17.0%	16.0%	18.4%	18.2%	17.2%	
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value			>4.5	1.25	>4.5		>4.5	>4.5

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			146	156	166	181		
Test Hole No.			A-4	A-4	A-4	A-4		
Field Sample No.			S25	S26	S27	S28		
Date Sampled			August 19, 2003	August 19, 2003	August 19, 2003	August 19, 2003		
Lab No.			A4 S25	A4 S26	A4 S27	A4 S28		
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			28	28	28	25		
Plastic Index			11	11	11	7		
Moisture Content %			17.2%	16.0%	17.8%	18.1%		
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value			>4.5	>4.5	>4.5	>4.5		

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			6	11	16	21	26	31
Test Hole No.			A-5	A-5	A-5	A-5	A-5	A-5
Field Sample No.			S1	S2	S3	S4	S5	S6
Date Sampled			September 16, 2003	September 16, 2003	September 16, 2003	September 16, 2003	September 16, 2003	September 16, 2003
Lab No.			A5 S1	A5 S2	A5 S3	A5 S4	A5 S5	A5 S6
Percent Passing Sieve Size	3"	75mm						
	2"	50mm				NO RECOVERY		
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit				39			38	
Plastic Index				23			18	
Moisture Content %			27.0%	23.4%	25.7%		21.1%	20.7%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u							.54 tsf	
Pocket Pen Value			2.75	4.5	3.5		2.5	>4.5

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			36	44	46	51	53	60.5
Test Hole No.			A-5	A-5	A-5	A-5	A-5	A-5
Field Sample No.			S7	S8	S9	S10	S11	S12
Date Sampled			September 16, 2003	September 16, 2003	September 16, 2003	September 17, 2003	September 17, 2003	September 17, 2003
Lab No.			A5 S7	A5 S8	A5 S9	A5 S10	A5 S11	A5 S12
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit				43				41
Plastic Index				22				20
Moisture Content %			20.9%	27.0%	26.0%		23.1%	
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u				2.3 tsf				2.4 tsf
Pocket Pen Value			3.5	2.5	2	3.5	4	2.5

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			62.5	68	75.5	77.5	83	85
Test Hole No.			A-5	A-5	A-5	A-5	A-5	A-5
Field Sample No.			S13	S14	S15	S16	S17	S18
Date Sampled			September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003
Lab No.			A5 S13	A5 S14	A5 S15	A5 S16	A5 S17	A5 S18
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit				38			40	
Plastic Index				18			20	
Moisture Content %			25.5%	24.0%		24.6%	25.1%	26.3%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u							2.3 tsf	
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u				1.4 tsf			1.4 tsf	
Pocket Pen Value				3	3.5	2.75	1	3

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			90.5	92.5	98	108	118	128
Test Hole No.			A-5	A-5	A-5	A-5	A-5	A-5
Field Sample No.			S19	S20	S21	S22	S23	S24
Date Sampled			September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003
Lab No.			A5 S19	A5 S20	A5 S21	A5 S22	A5 S23	A5 S24
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						100.0%
	#10	2mm						
	#16	1.18mm						99.8%
	#30	0.6mm						98.3%
	#40	0.425mm						
	#50	0.3mm						64.3%
	#100	0.15mm						18.8%
	#200	0.075mm						11.8%
DOTTSD								
Liquid Limit						39		
Plastic Index						19		
Moisture Content %				29.0%	33.1%	25.0%	25.8%	21.0%
Organic Content %								
% Gravel								
% Sand								88%
% Silt & Clay								12%
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u						2.0 tsf		
Pocket Pen Value			2.5			2	3.5	

SOILS TESTING REPORT

Table F-1

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			138	143	151	163	173	183
Test Hole No.			A-5	A-5	A-5	A-5	A-5	A-5
Field Sample No.			S25	S26	S27	S28	S29	S30
Date Sampled			September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003
Lab No.			A5 S25	A5 S26	A5 S27	A5 S28	A5 S29	A5 S30
Percent Passing Sieve Size	3"	75mm						
	2"	50mm	NO RECOVERY					
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm				100.0%		
	#8	2.36mm				100.0%		
	#10	2mm						
	#16	1.18mm				99.8%		
	#30	0.6mm				99.8%		
	#40	0.425mm						
	#50	0.3mm				99.2%		
	#100	0.15mm				97.5%		
	#200	0.075mm				96.5%		
DOTTSD								
Liquid Limit					39			40
Plastic Index					20			18
Moisture Content %				22.5%	20.1%	21.4%	22.6%	20.3%
Organic Content %								
% Gravel								
% Sand						4%		
% Silt & Clay						96%		
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u					6.5 tsf			4.2 tsf
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u					3.2 tsf			3.0 tsf
Pocket Pen Value				2.25	4.25	3.5	3	<1

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			193	203	213	223	233	240
Test Hole No.			A-5	A-5	A-5	A-5	A-5	A-5
Field Sample No.			S31	S32	S33	S34	S35	S36
Date Sampled			September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003	September 17, 2003
Lab No.			A5 S31	A5 S32	A5 S33	A5 S34	A5 S35	A5 S36
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm	100.0%					
	0.5"	12.5mm	98.6%					
	0.375"	9.5mm	98.4%					
	0.25"	6.3mm	98.3%		100.0%			
	#4	4.75mm	98.2%		99.9%			
	#8	2.36mm	97.9%		99.9%			
	#10	2mm						
	#16	1.18mm	97.6%		99.9%			
	#30	0.6mm	97.3%		99.5%			
	#40	0.425mm						
	#50	0.3mm	96.3%		76.4%			
	#100	0.15mm	94.5%		15.3%			
	#200	0.075mm	93.1%		3.1%			
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			22.4%	20.3%	22.2%	23.3%	18.3%	17.8%
Organic Content %								
% Gravel			2%					
% Sand			5%		97%			
% Silt & Clay			93%		3%			
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value			2.5	3	4.5	3.5		

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			242					
Test Hole No.			A-5					
Field Sample No.			S37					
Date Sampled			September 18, 2003					
Lab No.			A5 S37					
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			15.9%					
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			11.5	16	21	26	31	36
Test Hole No.			A-6	A-6	A-6	A-6	A-6	A-6
Field Sample No.			S1	S2	S3	S4	S5	S6
Date Sampled			September 14, 2003	September 14, 2003	September 14, 2003	September 14, 2003	September 14, 2003	September 14, 2003
Lab No.			A6 S1	A6 S2	A6 S3	A6 S4	A6 S5	A6 S6
Percent Passing Sieve Size	3"	75mm						
	2"	50mm	NO RECOVERY					
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit					24			28
Plastic Index					10			9
Moisture Content %				27.6%	17.4%	22.7%	15.7%	19.7%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value				3	4.25	2		

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			42	46	51	58	63	73
Test Hole No.			A-6	A-6	A-6	A-6	A-6	A-6
Field Sample No.			S7	S8	S9	S10	S11	S12
Date Sampled			September 14, 2003	September 14, 2003	September 14, 2003	September 15, 2003	September 15, 2003	September 15, 2003
Lab No.			A6 S7	A6 S8	A6 S9	A6 S10	A6 S11	A6 S12
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						100.0%
	1"	25mm						96.2%
	0.75"	19mm						96.2%
	0.5"	12.5mm						96.2%
	0.375"	9.5mm						93.2%
	0.25"	6.3mm						86.0%
	#4	4.75mm						82.8%
	#8	2.36mm						75.3%
	#10	2mm						
	#16	1.18mm						67.5%
	#30	0.6mm						58.9%
	#40	0.425mm						
	#50	0.3mm						35.7%
	#100	0.15mm						22.4%
	#200	0.075mm						19.5%
DOTTSD								
Liquid Limit			32					
Plastic Index			15					
Moisture Content %			21.0%	5.8%		7.8%	6.2%	14.5%
Organic Content %								
% Gravel								17%
% Sand								63%
% Silt & Clay								20%
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			85	86.5	89.5	92.5	94.5	97
Test Hole No.			A-6	A-6	A-6	A-6	A-6	A-6
Field Sample No.			S13	S14	S15	S16	S17	S18
Date Sampled			September 15, 2003	September 15, 2003	September 15, 2003	September 15, 2003	September 15, 2003	September 15, 2003
Lab No.			A6 S13	A6 S14	A6 S15	A6 S16	A6 S17	A6 S18
Percent Passing Sieve Size	3"	75mm						
	2"	50mm				NO RECOVERY		
	1.5"	37.5mm						
	1"	25mm			100.0%			
	0.75"	19mm			97.6%			
	0.5"	12.5mm			96.8%			
	0.375"	9.5mm			96.1%			
	0.25"	6.3mm			93.8%			
	#4	4.75mm			89.3%			
	#8	2.36mm			65.0%			
	#10	2mm						
	#16	1.18mm			44.5%			
	#30	0.6mm			43.5%			
	#40	0.425mm						
	#50	0.3mm			43.3%			
	#100	0.15mm			43.0%			
	#200	0.075mm			42.8%			
DOTTSD								
Liquid Limit								43
Plastic Index								24
Moisture Content %			21.3%		16.3%			22.5%
Organic Content %								
% Gravel					11%			
% Sand					46%			
% Silt & Clay					43%			
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								9 tsf
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								10 tsf
Pocket Pen Value				>4.5				1.75

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			99	105	16.5	115	125	135
Test Hole No.			A-6	A-6	A-6	A-6	A-6	A-6
Field Sample No.			S19	S20	S21	S22	S23	S24
Date Sampled			September 15, 2003	September 15, 2003	September 15, 2003	September 15, 2003	September 15, 2003	September 15, 2003
Lab No.			A6 S19	A6 S20	A6 S21	A6 S22	A6 S23	A6 S24
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						100.0%
	#4	4.75mm						99.8%
	#8	2.36mm						99.7%
	#10	2mm						
	#16	1.18mm						99.5%
	#30	0.6mm						99.2%
	#40	0.425mm						
	#50	0.3mm						98.6%
	#100	0.15mm						97.0%
	#200	0.075mm						90.1%
DOTTSD								
Liquid Limit				45				
Plastic Index				24				
Moisture Content %				26.2%		26.2%		27.3%
Organic Content %								
% Gravel								
% Sand								10%
% Silt & Clay								90%
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value			>4.5	>4.5	>4.5	>4.5	2	2.25

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			145	146.5	155	165	166.6	169
Test Hole No.			A-6	A-6	A-6	A-6	A-6	A-6
Field Sample No.			S25	S26	S27	S28	S29	S30
Date Sampled			September 15, 2003	September 15, 2003	September 15, 2003	September 15, 2003	September 15, 2003	September 15, 2003
Lab No.			A6 S25	A6 S26	A6 S27	A6 S28	A6 S29	A6 S30
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			27.5%	26.2%	20.1%		26.6%	
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value			2.75		3.5	>4		2.75

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			175	176.6	178	188	208	
Test Hole No.			A-6	A-6	A-6	A-6	A-6	
Field Sample No.			S31	S32	S33	S34	S35	
Date Sampled			September 16, 2003	September 16, 2003	September 16, 2003	September 16, 2003	September 16, 2003	
Lab No.			A6 S31	A6 S32	A6 S33	A6 S34	A6 S35	
Percent Passing Sieve Size	3"	75mm						
	2"	50mm					NO RECOVERY	
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			38					
Plastic Index			15					
Moisture Content %			16.3%	24.8%		15.3%		
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u			3 tsf					
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u			3 tsf					
Pocket Pen Value			>4.5					

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			5	10	15	20	25	30
Test Hole No.			A-7	A-7	A-7	A-7	A-7	A-7
Field Sample No.			S1	S2	S3	S4	S5	S6
Date Sampled			October 16, 2003	October 16, 2003	October 16, 2003	October 16, 2003	October 16, 2003	October 16, 2003
Lab No.			A7 S1	A7 S2	A7 S3	A7 S4	A7 S5	A7 S6
Percent Passing Sieve Size	3"	75mm						
	2"	50mm	NO RECOVERY					
	1.5"	37.5mm						
	1"	25mm					100.0%	
	0.75"	19mm					97.0%	
	0.5"	12.5mm					93.9%	
	0.375"	9.5mm					91.1%	
	0.25"	6.3mm					86.0%	
	#4	4.75mm					82.7%	
	#8	2.36mm					76.8%	
	#10	2mm						
	#16	1.18mm					72.7%	
	#30	0.6mm					69.5%	
	#40	0.425mm						
	#50	0.3mm					65.7%	
	#100	0.15mm					61.5%	
	#200	0.075mm					57.8%	
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %				5.9%	5.7%	6.0%	6.2%	11.8%
Organic Content %								
% Gravel							17%	
% Sand							25%	
% Silt & Clay							58%	
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Table F-1

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			35	40	45	50	55	60
Test Hole No.			A-7	A-7	A-7	A-7	A-7	A-7
Field Sample No.			S7	S8	S9	S10	S11	S12
Date Sampled			October 16, 2003	October 16, 2003	October 16, 2003	October 16, 2003	October 16, 2003	October 16, 2003
Lab No.			A7 S7	A7 S8	A7 S9	A7 S10	A7 S11	A7 S12
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			8.7%	8.2%	8.8%	22.2%	21.7%	8.6%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay						29%		
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			65	70	75	80	85	90
Test Hole No.			A-7	A-7	A-7	A-7	A-7	A-7
Field Sample No.			S13	S14	S15	S16	S17	S18
Date Sampled			October 16, 2003	October 16, 2003	October 16, 2003	October 17, 2003	October 17, 2003	October 17, 2003
Lab No.			A7 S13	A7 S14	A7 S15	A7 S16	A7 S17	A7 S18
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						NO RECOVERY
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			8.0%	11.1%	15.1%	14.6%	11.9%	
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			100	120	125	135	145	155
Test Hole No.			A-7	A-7	A-7	A-7	A-7	A-7
Field Sample No.			S19	S20	S21	S22	S23	S24
Date Sampled			October 17, 2003	October 18, 2003	October 19, 2003	October 19, 2003	October 19, 2003	October 19, 2003
Lab No.			A7 S19	A7 S20	A7 S21	A7 S22	A7 S23	A7 S24
Percent Passing Sieve Size	3"	75mm						
	2"	50mm	NO RECOVERY					
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit							42	
Plastic Index							19	
Moisture Content %					22.6%	22.0%	25.3%	25.0%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value					>4.5	>4.5		>4.5

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			165	175	185	195		
Test Hole No.			A-7	A-7	A-7	A-7		
Field Sample No.			S25	S26	S27	S28		
Date Sampled			October 19, 2003	October 19, 2003	October 19, 2003	October 20, 2003		
Lab No.			A7 S25	A7 S26	A7 S27	A7 S28		
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			41		23			
Plastic Index			19		4			
Moisture Content %			19.5%	26.2%	27.5%	24.2%		
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay				93%	89%			
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u			9.5 tsf					
Pocket Pen Value			>4.5					

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			5	10	15	20	25	30
Test Hole No.			A-8	A-8	A-8	A-8	A-8	A-8
Field Sample No.			S1	S2	S3	S4	S5	S6
Date Sampled			September 10, 2003	September 10, 2003	September 10, 2003	September 10, 2003	September 10, 2003	September 10, 2003
Lab No.			A8 S1	A8 S2	A8 S3	A8 S4	A8 S5	A8 S6
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			13.7%		5.6%	5.0%	7.7%	8.6%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			35	40	40	50	60	67.5
Test Hole No.			A-8	A-8	A-8	A-8	A-8	A-8
Field Sample No.			S7	S8	S9	S10	S11	S12
Date Sampled			September 10, 2003	September 10, 2003	September 10, 2003	September 10, 2003	September 10, 2003	September 10, 2003
Lab No.			A8 S7	A8 S8	A8 S9	A8 S10	A8 S11	A8 S12
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm			100.0%			
	0.5"	12.5mm			98.5%			
	0.375"	9.5mm			96.5%			
	0.25"	6.3mm			93.3%			
	#4	4.75mm			91.5%			
	#8	2.36mm			87.3%			
	#10	2mm						
	#16	1.18mm			80.1%			
	#30	0.6mm			53.8%			
	#40	0.425mm						
	#50	0.3mm			18.6%			
	#100	0.15mm			11.9%			
	#200	0.075mm			9.1%			
DOTTSD								
Liquid Limit							28	
Plastic Index							14	
Moisture Content %			8.1%	17.7%	13.5%	18.6%	10.8%	9.6%
Organic Content %								
% Gravel					8%			
% Sand					82%			
% Silt & Clay					9%			
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			75	82.5	90	97.5	110	120
Test Hole No.			A-8	A-8	A-8	A-8	A-8	A-8
Field Sample No.			S13	S14	S15	S16	S17	S18
Date Sampled			September 11, 2003	September 11, 2003	September 11, 2003	September 11, 2003	September 11, 2003	September 11, 2003
Lab No.			A8 S13	A8 S14	A8 S15	A8 S16	A8 S17	A8 S18
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm		100.0%				
	1"	25mm		97.3%				
	0.75"	19mm		95.9%				
	0.5"	12.5mm		92.9%				
	0.375"	9.5mm		91.7%				
	0.25"	6.3mm		88.5%				
	#4	4.75mm		87.3%				
	#8	2.36mm		84.2%				
	#10	2mm						
	#16	1.18mm		81.7%				
	#30	0.6mm		78.8%				
	#40	0.425mm						
	#50	0.3mm		73.1%				
	#100	0.15mm		64.8%				
	#200	0.075mm		55.2%				
DOTTSD								
Liquid Limit				19				
Plastic Index				7				
Moisture Content %			11.2%	9.9%	10.5%	14.0%	11.4%	12.7%
Organic Content %								
% Gravel				13%				
% Sand				32%				
% Silt & Clay				55%				
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			130	140	150	160	170	180
Test Hole No.			A-8	A-8	A-8	A-8	A-8	A-8
Field Sample No.			S19	S20	S21	S22	S23	S24
Date Sampled			September 11, 2003	September 11, 2003	September 11, 2003	September 12, 2003	September 12, 2003	September 12, 2003
Lab No.			A8 S19	A8 S20	A8 S21	A8 S22	A8 S23	A8 S24
Percent Passing Sieve Size	3"	75mm						
	2"	50mm				NO RECOVERY		
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit			22				29	
Plastic Index			9				11	
Moisture Content %			19.6%	14.4%	12.9%		17.8%	11.6%
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u			1.5 tsf					
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			185						
Test Hole No.			A-8						
Field Sample No.			S25						
Date Sampled			September 12, 2003						
Lab No.			A8 S25						
Percent Passing Sieve Size	3"	75mm							
	2"	50mm							
	1.5"	37.5mm							
	1"	25mm							
	0.75"	19mm							
	0.5"	12.5mm							
	0.375"	9.5mm							
	0.25"	6.3mm							
	#4	4.75mm							
	#8	2.36mm							
	#10	2mm							
	#16	1.18mm							
	#30	0.6mm							
	#40	0.425mm							
	#50	0.3mm							
	#100	0.15mm							
	#200	0.075mm							
DOTTSD									
Liquid Limit			28						
Plastic Index			11						
Moisture Content %			14.6%						
Organic Content %									
% Gravel									
% Sand									
% Silt & Clay									
Max. Dry Density									
Opt. Moisture %									
Unconsol. Unconfined Triaxial U_u									
Coeff. Of Consolidation C_v									
Unc. Comp. Strength Q_u									
Pocket Pen Value									

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			6	11	16	23	32	42.5
Test Hole No.			A-9	A-9	A-9	A-9	A-9	A-9
Field Sample No.			S1	S2	S3	S4	S5	S6
Date Sampled			September 12, 2003	September 12, 2003	September 12, 2003	September 12, 2003	September 12, 2003	September 12, 2003
Lab No.			A9 S1	A9 S2	A9 S3	A9 S4	A9 S5	A9 S6
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm		100.0%			100.0%	
	1"	25mm		95.0%			90.0%	
	0.75"	19mm		90.0%			84.0%	
	0.5"	12.5mm		84.0%			64.0%	
	0.375"	9.5mm		80.0%			57.0%	
	0.25"	6.3mm		77.0%			46.0%	
	#4	4.75mm		74.0%			42.0%	
	#8	2.36mm		70.0%			33.0%	
	#10	2mm						
	#16	1.18mm		64.0%			26.0%	
	#30	0.6mm		54.0%			17.0%	
	#40	0.425mm						
	#50	0.3mm		26.0%			9.0%	
	#100	0.15mm		15.0%			6.0%	
	#200	0.075mm		7.0%			4.7%	
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			10.1%	11.5%	7.8%	3.9%	4.1%	6.2%
Organic Content %								
% Gravel				26%			58%	
% Sand				67%			38%	
% Silt & Clay				7%		4%	5%	
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			46.5	51	56	63	73.5	83
Test Hole No.			A-9	A-9	A-9	A-9	A-9	A-9
Field Sample No.			S7	S8	S9	S10	S11	S12
Date Sampled			September 13, 2003	September 13, 2003	September 13, 2003	September 13, 2003	September 13, 2003	September 13, 2003
Lab No.			A9 S7	A9 S8	A9 S9	A9 S10	A9 S11	A9 S12
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm			100.0%			
	1"	25mm			89.7%			
	0.75"	19mm			79.3%			
	0.5"	12.5mm			73.4%			
	0.375"	9.5mm			68.3%			
	0.25"	6.3mm			45.7%			
	#4	4.75mm			31.1%			
	#8	2.36mm			9.1%			
	#10	2mm						
	#16	1.18mm			2.2%			
	#30	0.6mm			1.6%			
	#40	0.425mm						
	#50	0.3mm			0.4%			
	#100	0.15mm			0.3%			
	#200	0.075mm			0.3%			
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %					1.7%	8.2%	7.7%	8.1%
Organic Content %								
% Gravel					69%			
% Sand					31%			
% Silt & Clay					0%		5%	
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			94	95	104	104.9	109	114
Test Hole No.			A-9	A-9	A-9	A-9	A-9	A-9
Field Sample No.			S13a	S13b	S14	S15	S16	S17
Date Sampled			September 13, 2003	September 13, 2003	September 13, 2003	September 13, 2003	September 13, 2003	September 14, 2003
Lab No.			A9 S13a	A9 S13b	A9 S14	A9 S15	A9 S16	A9 S17
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						100.0%
	0.5"	12.5mm						96.1%
	0.375"	9.5mm						94.7%
	0.25"	6.3mm						91.5%
	#4	4.75mm						91.5%
	#8	2.36mm						87.9%
	#10	2mm						
	#16	1.18mm						84.9%
	#30	0.6mm						83.6%
	#40	0.425mm						
	#50	0.3mm						81.8%
	#100	0.15mm						78.8%
	#200	0.075mm						69.9%
DOTTSD								
Liquid Limit				27			26	
Plastic Index				4			7	
Moisture Content %			11.0%	9.5%	34.0%		28.9%	
Organic Content %								
% Gravel								9%
% Sand								22%
% Silt & Clay								70%
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value						2.25	2.3	

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153€ Sampled By: Elizabeth Karcheski

Depth			7	12	14	20	30	35
Test Hole No.			A-10	A-10	A-10	A-10	A-10	A-10
Field Sample No.			S1	S2	S3	S4	S5	S6
Date Sampled			September 18, 2003	September 18, 2003	September 18, 2003	September 19, 2003	September 19, 2003	September 19, 2003
Lab No.			A10 S1	A10 S2	A10 S3	A10 S4	A10 S5	A10 S6
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						100.0%
	#10	2mm						
	#16	1.18mm						99.9%
	#30	0.6mm		100.0%				99.6%
	#40	0.425mm						
	#50	0.3mm		98.6%				95.8%
	#100	0.15mm		33.3%				12.3%
	#200	0.075mm		9.5%				4.4%
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			21.6%	21.2%		22.8%	21.1%	22.5%
Organic Content %								
% Gravel								
% Sand				91%				96%
% Silt & Clay				9%				4%
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153€ Sampled By: Elizabeth Karcheski

Depth			40	45	50	52	59	66
Test Hole No.			A-10	A-10	A-10	A-10	A-10	A-10
Field Sample No.			S7	S8	S9	S10	S11	S12
Date Sampled			September 19, 2003	September 19, 2003	September 19, 2003	September 19, 2003	September 19, 2003	September 19, 2003
Lab No.			A10 S7	A10 S8	A10 S9	A10 S10	A10 S11	A10 S12
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm		100.0%				
	0.25"	6.3mm		99.5%				
	#4	4.75mm		99.5%				
	#8	2.36mm		98.4%				
	#10	2mm						
	#16	1.18mm		96.8%				
	#30	0.6mm		95.4%				
	#40	0.425mm						
	#50	0.3mm		72.3%				
	#100	0.15mm		21.0%				
	#200	0.075mm		9.3%				
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			22.4%	24.3%	22.1%	23.5%	10.7%	23.1%
Organic Content %								
% Gravel				1%				
% Sand				90%				
% Silt & Clay				9%				6%
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			73.5	81	88.5	96	106	116
Test Hole No.			A-10	A-10	A-10	A-10	A-10	A-10
Field Sample No.			S13	S14	S15	S16	S17	S18
Date Sampled			September 19, 2003	September 19, 2003	September 19, 2003	September 19, 2003	September 19, 2003	September 19, 2003
Lab No.			A10 S13	A10 S14	A10 S15	A10 S16	A10 S17	A10 S18
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm	100.0%		100.0%			
	0.25"	6.3mm	98.2%		98.8%			
	#4	4.75mm	93.9%		95.0%			
	#8	2.36mm	93.0%		92.9%		100.0%	
	#10	2mm						
	#16	1.18mm	93.0%		91.8%		99.9%	
	#30	0.6mm	92.9%		91.3%		98.9%	
	#40	0.425mm						
	#50	0.3mm	87.8%		88.5%		78.1%	
	#100	0.15mm	29.4%		24.0%		22.2%	
	#200	0.075mm	8.9%		9.1%		9.2%	
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			22.5%	22.7%	21.7%	24.0%	22.6%	21.1%
Organic Content %								
% Gravel			6%		4%			
% Sand			85%		87%		91%	
% Silt & Clay			9%		9%		9%	10%
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			126	146	156	166	175.5	186.5
Test Hole No.			A-10	A-10	A-10	A-10	A-10	A-10
Field Sample No.			S19	S20	S21	S22	S23	S24
Date Sampled			September 19, 2003	September 20, 2003	September 20, 2003	September 20, 2003	September 20, 2003	September 20, 2003
Lab No.			A10 S19	A10 S20	A10 S21	A10 S22	A10 S23	A10 S24
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm	100.0%					
	0.25"	6.3mm	99.9%					
	#4	4.75mm	99.8%					
	#8	2.36mm	99.6%					
	#10	2mm						
	#16	1.18mm	98.5%					
	#30	0.6mm	80.4%					
	#40	0.425mm						
	#50	0.3mm	51.5%					
	#100	0.15mm	35.6%					
	#200	0.075mm	13.9%					
DOTTSD								
Liquid Limit							22	24
Plastic Index							5	7
Moisture Content %			25.9%		18.4%	16.7%	16.0%	15.2%
Organic Content %								
% Gravel								
% Sand			86%					
% Silt & Clay			14%		67%			65%
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u							3.5 tsf	
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u							3 tsf	
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			196.5	216.5				
Test Hole No.			A-10	A-10				
Field Sample No.			S25	S26				
Date Sampled			September 20, 2003	September 20, 2003				
Lab No.			A10 S25	A10 S26				
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			6.7%					
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay								
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153€ Sampled By: Elizabeth Karcheski

Depth			5	10	15	20	25	30
Test Hole No.			A-11	A-11	A-11	A-11	A-11	A-11
Field Sample No.			S1	S2	S3	S4	S5	S6
Date Sampled			October 21, 2003	October 21, 2003	October 21, 2003	October 21, 2003	October 21, 2003	October 21, 2003
Lab No.			A11 S1	A11 S2	A11 S3	A11 S4	A11 S5	A11 S6
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm		100.0%				
	1"	25mm		95.2%				
	0.75"	19mm		95.2%				
	0.5"	12.5mm		94.4%				
	0.375"	9.5mm		92.8%				
	0.25"	6.3mm		91.4%				
	#4	4.75mm		91.0%				
	#8	2.36mm		89.8%				
	#10	2mm						
	#16	1.18mm		88.8%				
	#30	0.6mm		87.4%				
	#40	0.425mm						
	#50	0.3mm		84.1%				
	#100	0.15mm		78.7%				
	#200	0.075mm		73.4%				
DOTTSD								
Liquid Limit								26
Plastic Index								13
Moisture Content %			8.3%	10.5%	17.9%	14.4%	8.4%	8.2%
Organic Content %								
% Gravel				9%				
% Sand				18%				
% Silt & Clay				73%				
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			5	10	15	20	25	
Test Hole No.			A-12	A-12	A-12	A-12	A-12	
Field Sample No.			S1	S2	S3	S4	S5	
Date Sampled			October 21, 2003	October 21, 2003	October 21, 2003	October 21, 2003	October 21, 2003	
Lab No.			A12 S1	A12 S2	A12 S3	A12 S4	A12 S5	
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit				25			30	
Plastic Index				10			14	
Moisture Content %			13.5%	11.6%	19.8%	21.4%	16.5%	
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay				90%				
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153€ Sampled By: Elizabeth Karcheski

Depth			5	10	15	20	25	
Test Hole No.			A-13	A-13	A-13	A-13	A-13	
Field Sample No.			S1	S2	S3	S4	S5	
Date Sampled			October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003	
Lab No.			A13 S1	A13 S2	A13 S3	A13 S4	A13 S5	
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm	100.0%					
	1"	25mm	97.8%					
	0.75"	19mm	97.1%					
	0.5"	12.5mm	92.9%					
	0.375"	9.5mm	89.8%					
	0.25"	6.3mm	86.9%					
	#4	4.75mm	84.4%					
	#8	2.36mm	79.9%					
	#10	2mm						
	#16	1.18mm	76.6%					
	#30	0.6mm	72.6%					
	#40	0.425mm						
	#50	0.3mm	61.7%					
	#100	0.15mm	54.3%					
	#200	0.075mm	52.1%					
DOTTSD								
Liquid Limit							23	
Plastic Index							8	
Moisture Content %			10.6%	20.7%	14.8%	14.4%	14.5%	
Organic Content %								
% Gravel			16%					
% Sand			32%					
% Silt & Clay			52%					
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-0153(Sampled By: Elizabeth Karcheski

Depth			5	10	15	20	25	
Test Hole No.			A-14	A-14	A-14	A-14	A-14	
Field Sample No.			S1	S2	S3	S4	S5	
Date Sampled			October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003	
Lab No.			A14 S1	A14 S2	A14 S3	A14 S4	A14 S5	
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			15.4%	21.4%	20.4%	23.3%	13.0%	
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay					24%		79%	
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			5	10	15	20	25	
Test Hole No.			A-15	A-15	A-15	A-15	A-15	
Field Sample No.			S1	S2	S3	S4	S5	
Date Sampled			October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003	
Lab No.			A15 S1	A15 S2	A15 S3	A15 S4	A15 S5	
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm						
	0.75"	19mm						
	0.5"	12.5mm						
	0.375"	9.5mm						
	0.25"	6.3mm						
	#4	4.75mm						
	#8	2.36mm						
	#10	2mm						
	#16	1.18mm						
	#30	0.6mm						
	#40	0.425mm						
	#50	0.3mm						
	#100	0.15mm						
	#200	0.075mm						
DOTTSD								
Liquid Limit								
Plastic Index								
Moisture Content %			23.5%	22.3%	12.1%	10.9%	15.5%	
Organic Content %								
% Gravel								
% Sand								
% Silt & Clay			99%		45%			
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			5	10	15	20	30	
Test Hole No.			A-16	A-16	A-16	A-16	A-16	
Field Sample No.			S1	S2	S3	S4	S5	
Date Sampled			October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003	
Lab No.			A16 S1	A16 S2	A16 S3	A16 S4	A16 S5	
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm	100.0%					
	1"	25mm	95.8%					
	0.75"	19mm	95.8%					
	0.5"	12.5mm	93.8%					
	0.375"	9.5mm	92.5%					
	0.25"	6.3mm	90.9%					
	#4	4.75mm	89.8%					
	#8	2.36mm	87.4%					
	#10	2mm						
	#16	1.18mm	85.6%					
	#30	0.6mm	83.3%					
	#40	0.425mm						
	#50	0.3mm	74.0%					
	#100	0.15mm	66.6%					
	#200	0.075mm	62.8%					
DOTTSD								
Liquid Limit							34	
Plastic Index							15	
Moisture Content %			12.6%	21.1%	21.9%	21.0%	22.4%	
Organic Content %								
% Gravel			10%					
% Sand			27%					
% Silt & Clay			63%		23%			
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

SOILS TESTING REPORT

Project Name: Knik Arm Bridge

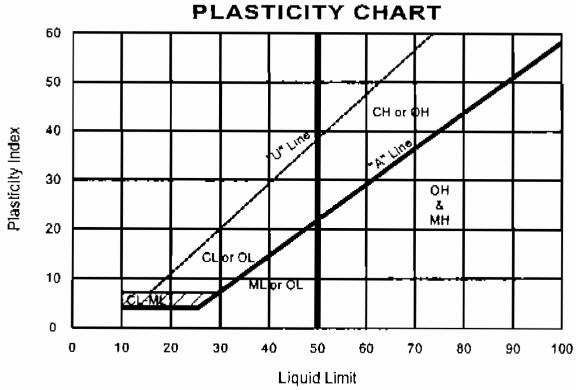
Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth			5	10	15	20	25	30
Test Hole No.			A-17	A-17	A-17	A-17	A-17	A-17
Field Sample No.			S1	S2	S3	S4	S5	S6
Date Sampled			October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003	October 22, 2003
Lab No.			A17 S1	A17 S2	A17 S3	A17 S4	A17 S5	A17 S6
Percent Passing Sieve Size	3"	75mm						
	2"	50mm						
	1.5"	37.5mm						
	1"	25mm		100.0%				
	0.75"	19mm		97.2%				
	0.5"	12.5mm		89.8%				
	0.375"	9.5mm		87.5%				
	0.25"	6.3mm		86.0%				
	#4	4.75mm		84.0%				
	#8	2.36mm		80.2%				
	#10	2mm						
	#16	1.18mm		76.2%				
	#30	0.6mm		72.4%				
	#40	0.425mm						
	#50	0.3mm		68.0%				
	#100	0.15mm		64.1%				
	#200	0.075mm		62.8%				
DOTTSD								
Liquid Limit					32			33
Plastic Index					13			16
Moisture Content %			32.2%	14.4%	22.0%	20.9%	20.0%	17.6%
Organic Content %								
% Gravel				20%				
% Sand				17%				
% Silt & Clay				63%				
Max. Dry Density								
Opt. Moisture %								
Unconsol. Unconfined Triaxial U_u								
Coeff. Of Consolidation C_v								
Unc. Comp. Strength Q_u								
Pocket Pen Value								

Unified Soil Classification System

GROUP NAME Criteria for Assigning Group Names and Group Symbols			Soil Classification Group Symbol with Generalized Group Descriptions	
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVELS 50% or more of coarse fraction retained on No. 4 sieve	Clean GRAVELS Less than 5% fines	GW	Well-graded Gravels
			GP	Poorly-graded Gravels
		GRAVELS with fines More than 12% fines	GM	Gravel & Silt Mixtures
			GC	Gravel & Clay Mixtures
	SANDS More than 50% of coarse fraction passes No. 4 sieve	Clean SANDS Less than 5% fines	SW	Well-graded Sands
			SP	Poorly-graded Sands
		SANDS with fines More than 12% fines	SM	Sand & Silt Mixtures
			SC	Sand & Clay Mixtures
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid limit 50% or less	INORGANIC	ML	Non-plastic & Low-plasticity Silts
			CL	Low-plasticity Clays
		ORGANIC	OL	Non-plastic and Low-plasticity Organic Clays Non-plastic and Low-plasticity Organic Silts
	SILTS AND CLAYS Liquid limit greater than 50%	INORGANIC	CH	High-plasticity Clays
			MH	High-plasticity Silts
		ORGANIC	OH	High-plasticity Organic Clays High-plasticity Organic Silts
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat

PLASTICITY CHART



Descriptive Terminology Denoting Component Proportions

Description	Range of Proportion
Add the adjective "slightly"	5 - 12%
Add soil adjective[a]	12 - 50%
Major proportion in upper case, (e.g., SAND)	>50%

(a) Use gravelly, sandy, or silty as appropriate

NOTE: The soil descriptions used in the boring logs lists constituents from smallest percentage to largest percentage.

Knik Arm Bridge
Anchorage, Alaska

SOIL CLASSIFICATION LEGEND

February 2004 | 32-1-01536

SHANNON & WILSON, INC.
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Table F-2

Table F-3
Summary of Unit Weight Measurements

Boring No.	Sample No.	Depth	Weight Wet Unit, pcf	Water Content %
A-1	S6	23	131.8	22
	S9	35	131	24
	S10	40	131.2	22
	S13	55	129.8	24
	S14	60	133.8	19
	S19	85	127.2	29
	S25	100	120.5	33
	S25	115	127.2	29
	S26	120	125.1	25
	S28	130	122.4	26-31
	S29	135	129	27
	S32	150	126.3	26-27
	S35	172.5	128.3	32
	S37	188	132.8	23
	S40	210	142.3	21
	S43	235	129	25.9-28.2
	S46	265	139.1	15.5-16.8
	S47	275	130	23-24.5
	S50	305	131.4	16
	S52	325	162.7	22
	S53	335	136.5	22
A-2	S26	176	125.8	23
A-4	S16	81	148.5	20
A-5	S5	26	128.3	24
	S8	44	125	27
	S12	60.5	129.9	23
	S14a	69	130.2	24
	S17	84	128	25
	S22	109	131	23
	S27	151	125	20
	S30	184	129.5	21
A-6	S18	97	162.2	22.5
	S31	175	128	16.6
A-8	S19	130	136.6	19.6
A-10	S23	175.5	135	16

SUMMARY OF UNIT WEIGHT MEASUREMENTS

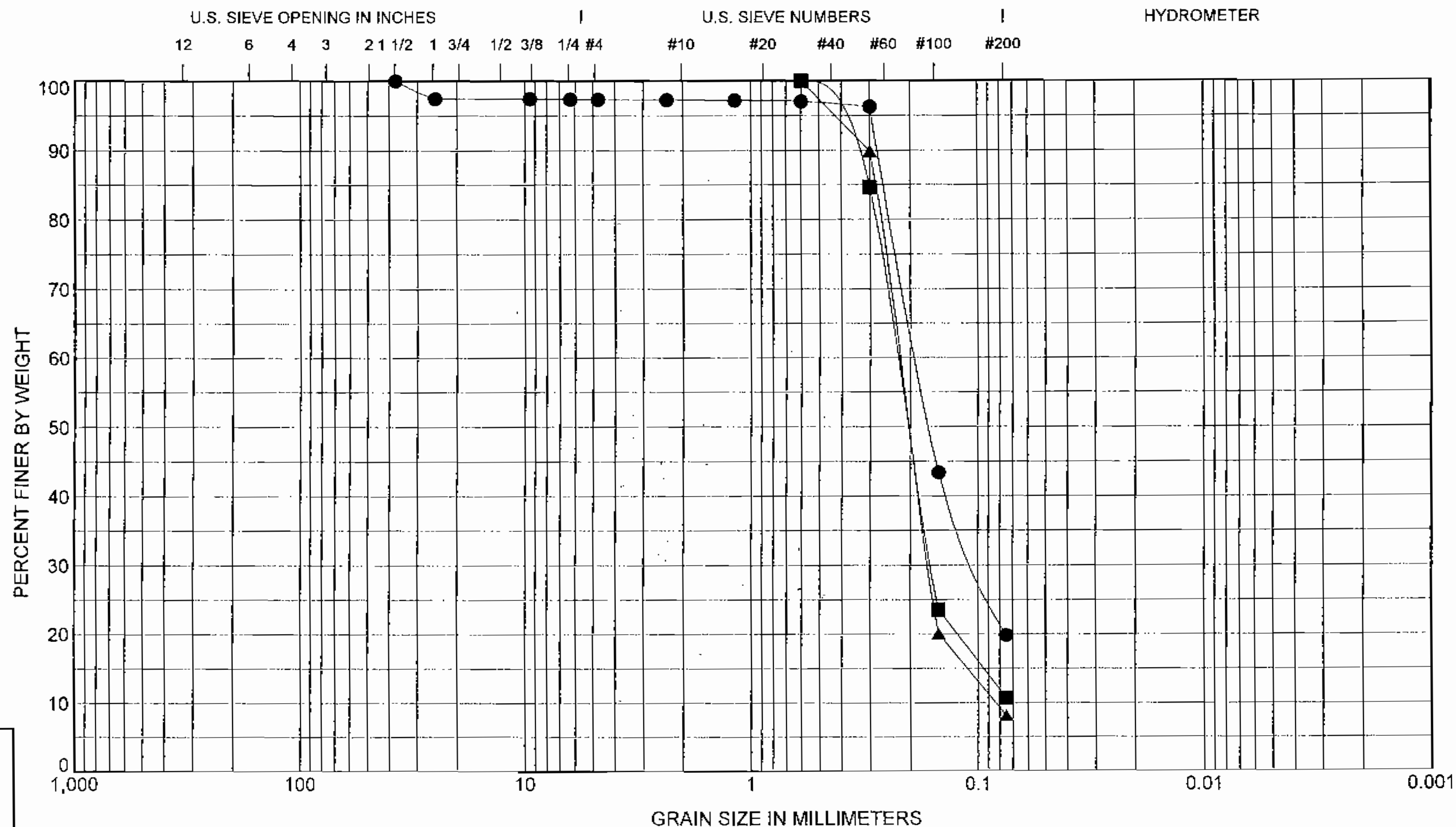
Project Name: Knik Arm Bridge

Project No.: 32-1-01536 Sampled By: Elizabeth Karcheski

Depth	176		81		26	44
Test Hole No.	**A-2**		**A-4**		**A-5**	**A-5**
Sample No.	S26		S16		S5	S8
Wet Unit Weight, pcf	125.8		148.5		128.3	125
Water Content %	23		20		24	27

Depth	60.5	69	84	109	151	184
Test Hole No.	**A-5**	**A-5**	**A-5**	**A-5**	**A-5**	**A-5**
Sample No.	S12	S14a	S17	S22	S27	S30
Wet Unit Weight, pcf	129.9	130.2	128	131	125	129.5
Water Content %	23	24	25	23	20	21

Depth	97	175		130		175.5
Test Hole No.	**A-6**	**A-6**		**A-8**		**A-10**
Sample No.	S18	S31		S19		S23
Wet Unit Weight, pcf	162.2	128		136.6		135
Water Content %	22.5	16.6		19.6		16



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample		Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-1 S1	8.0 - 10.0	**Silty SAND, trace of gravel (SM)**					
■	A-2 S4a	21.0 - 22.5	**Slightly silty SAND (SP-SM)**				**1.6**	**3.2**
▲	A-2 S7	41.0 - 42.5	**Slightly silty SAND (SP-SM)**				**1.5**	**2.7**

Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-1 S1	8.0 - 10.0	**37.5**	**0.19**	**0.1**		**3**	**77**	**20**	
■	A-2 S4a	21.0 - 22.5	**0.6**	**0.23**	**0.16**		**0**	**89**	**11**	
▲	A-2 S7	41.0 - 42.5	**0.6**	**0.22**	**0.17**	**0.08**	**0**	**92**	**8**	

Knik Arm Bridge
Anchorage, Alaska

GRAIN SIZE CLASSIFICATION

February 2004 32-1-01536

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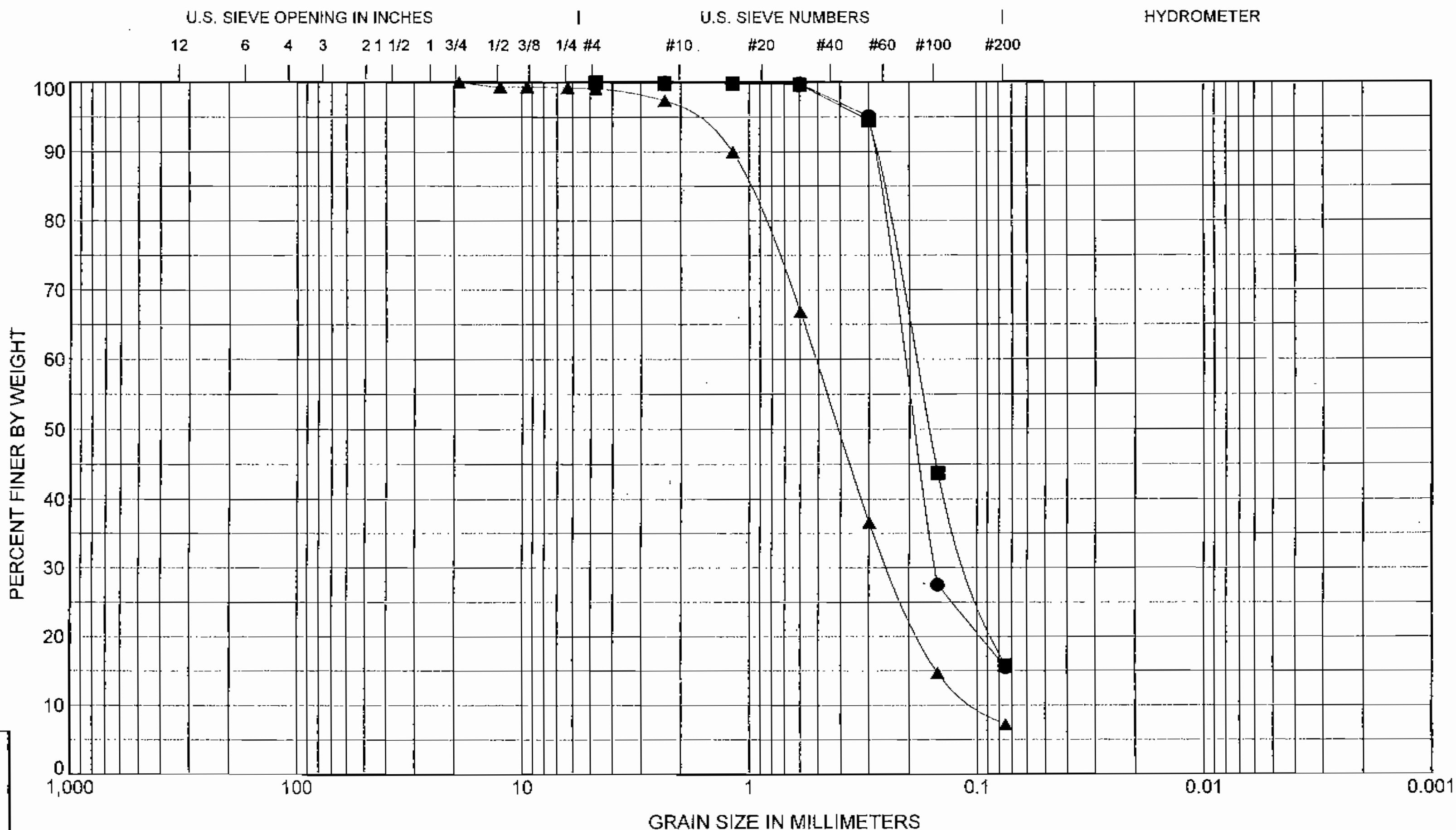
Fig. F-1
Sheet 1 of 12



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-2 S12	61.1 - 66.1	**Silty SAND (SM)**					
■	A-2 S14	78.0 - 79.5	**Silty SAND (SM)**					
▲	A-2 S18	115.5 - 117.0	**Slightly silty SAND, trace of gravel (SP-SM)**				**1.2**	**5.3**

	Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-2 S12	61.1 - 66.1	**4.75**	**0.21**	**0.15**		**0**	**84**	**16**	
■	A-2 S14	78.0 - 79.5	**4.75**	**0.19**	**0.11**		**0**	**84**	**16**	
▲	A-2 S18	115.5 - 117.0	**19**	**0.51**	**0.24**	**0.1**	**1**	**92**	**7**	

Knik Arm Bridge
Anchorage, Alaska

GRAIN SIZE CLASSIFICATION

February 2004

32-1-01536

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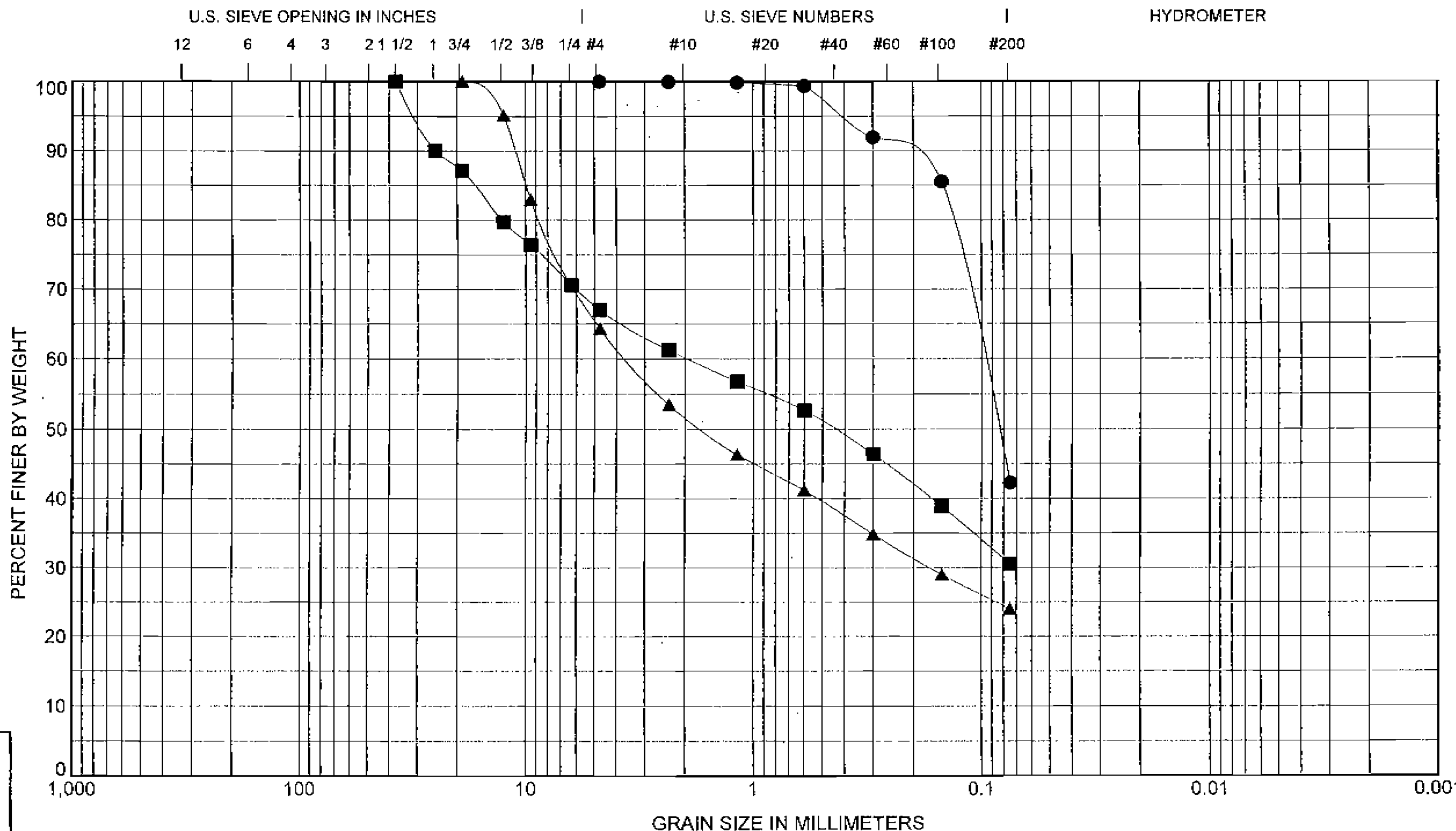
Fig. F-1
Sheet 2 of 12



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-2 S20	135.0 - 136.5	**Silty SAND (SM)**					
■	A-4 S3	11.0 - 12.5	**Silty, gravelly SAND (SWG)**	**NP**	**NP**	**NP**		
▲	A-4 S6	26.0 - 26.8	**Silty, gravelly SAND (SWG)**					

	Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-2 S20	135.0 - 136.5	4.75	0.1			0	58	42	
■	A-4 S3	11.0 - 12.5	37.5	1.95			33	36	31	
▲	A-4 S6	26.0 - 26.8	19	3.59	0.17		36	40	24	

Knik Arm Bridge
Anchorage, Alaska

GRAIN SIZE CLASSIFICATION

February 2004 32-1-01536

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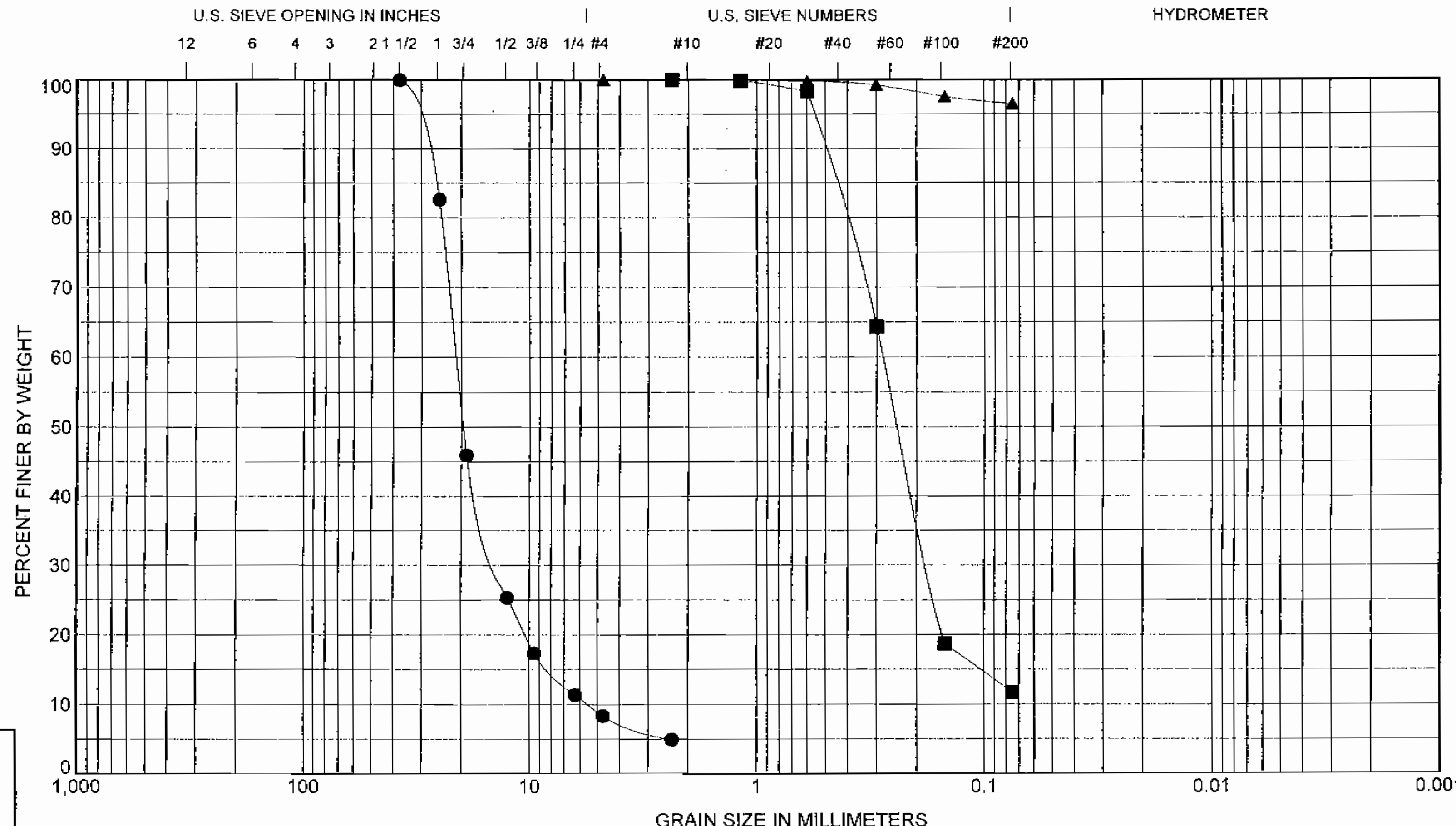
Fig. F-1
Sheet 3 of 12



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample		Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-4 S8	36.0 - 36.8	**GRAVEL (GW)**	38	20	18	1.6	3.8
■	A-5 S24	128.0 - 130.0	**Slightly silty SAND, (SP-SM)**				1.8	4.5
▲	A-5 S28	163.0 - 164.5	**CLAY with trace of sand, (CL)**					

Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-4 S8	36.0 - 36.8	37.5	21.11	13.73	5.53	92			
■	A-5 S24	128.0 - 130.0	2.36	0.28	0.18		0	88	12	
▲	A-5 S28	163.0 - 164.5	4.75				0	4	96	

Knik Arm Bridge
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GRAIN SIZE CLASSIFICATION

February 2004 32-1-01536

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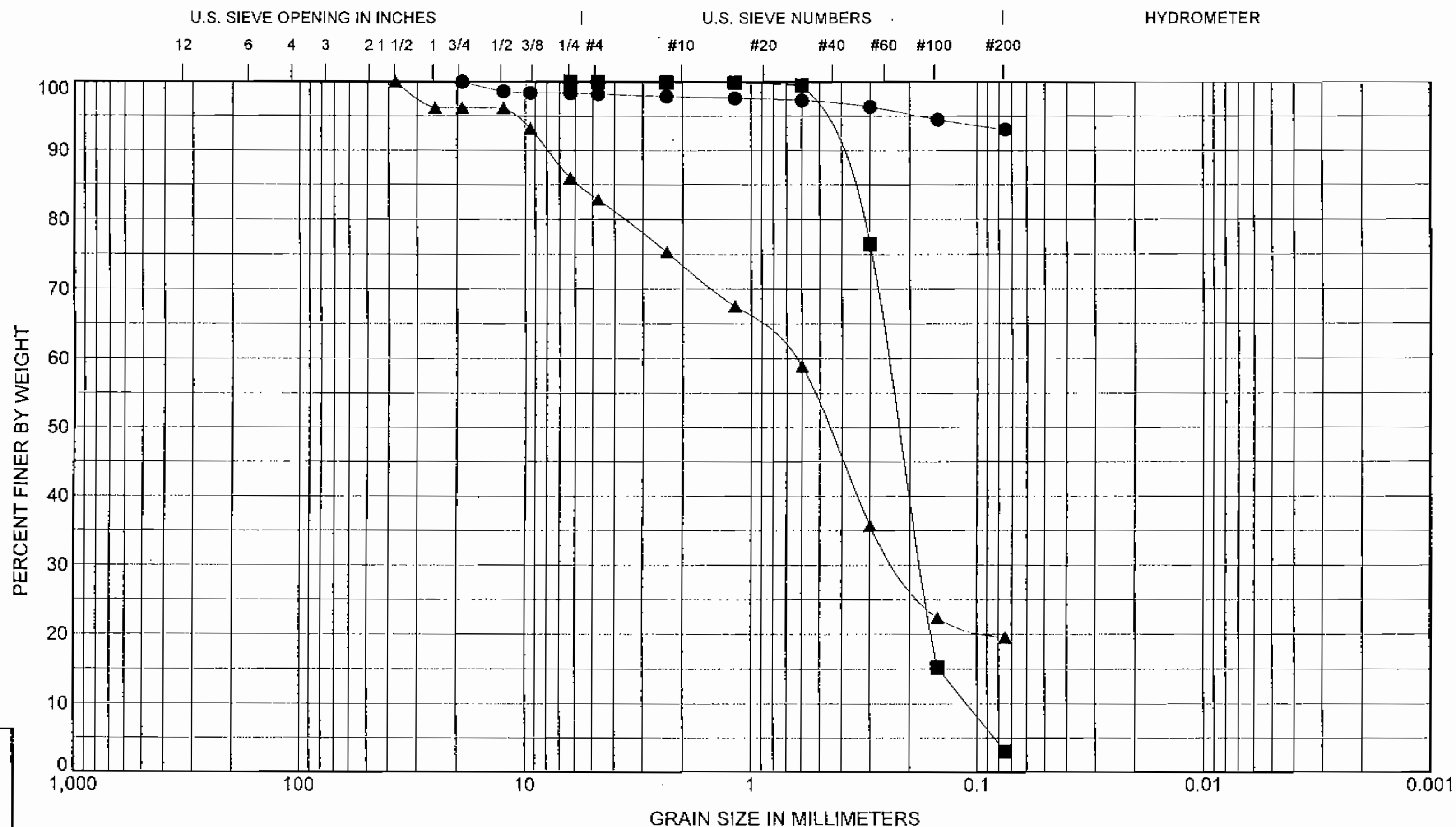
Fig. F-1
Sheet 4 of 12



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample		Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-5 S31	193.0 - 194.5	**Slightly sandy CLAY, (CL)**					
■	A-5 S33	213.0 - 215.0	**SAND with trace clay, (SP)**				**1.1**	**2.2**
▲	A-6 S12	73.0 - 73.6	**Gravelly, silty SAND, (SM)**					

Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-5 S31	193.0 - 194.5	**19**				**2**	**5**	**93**	
■	A-5 S33	213.0 - 215.0	**6.3**	**0.25**	**0.18**	**0.11**	**0**	**97**	**3**	
▲	A-6 S12	73.0 - 73.6	**37.5**	**0.66**	**0.22**		**17**	**63**	**20**	

Knik Arm Bridge
Anchorage, Alaska

GRAIN SIZE CLASSIFICATION

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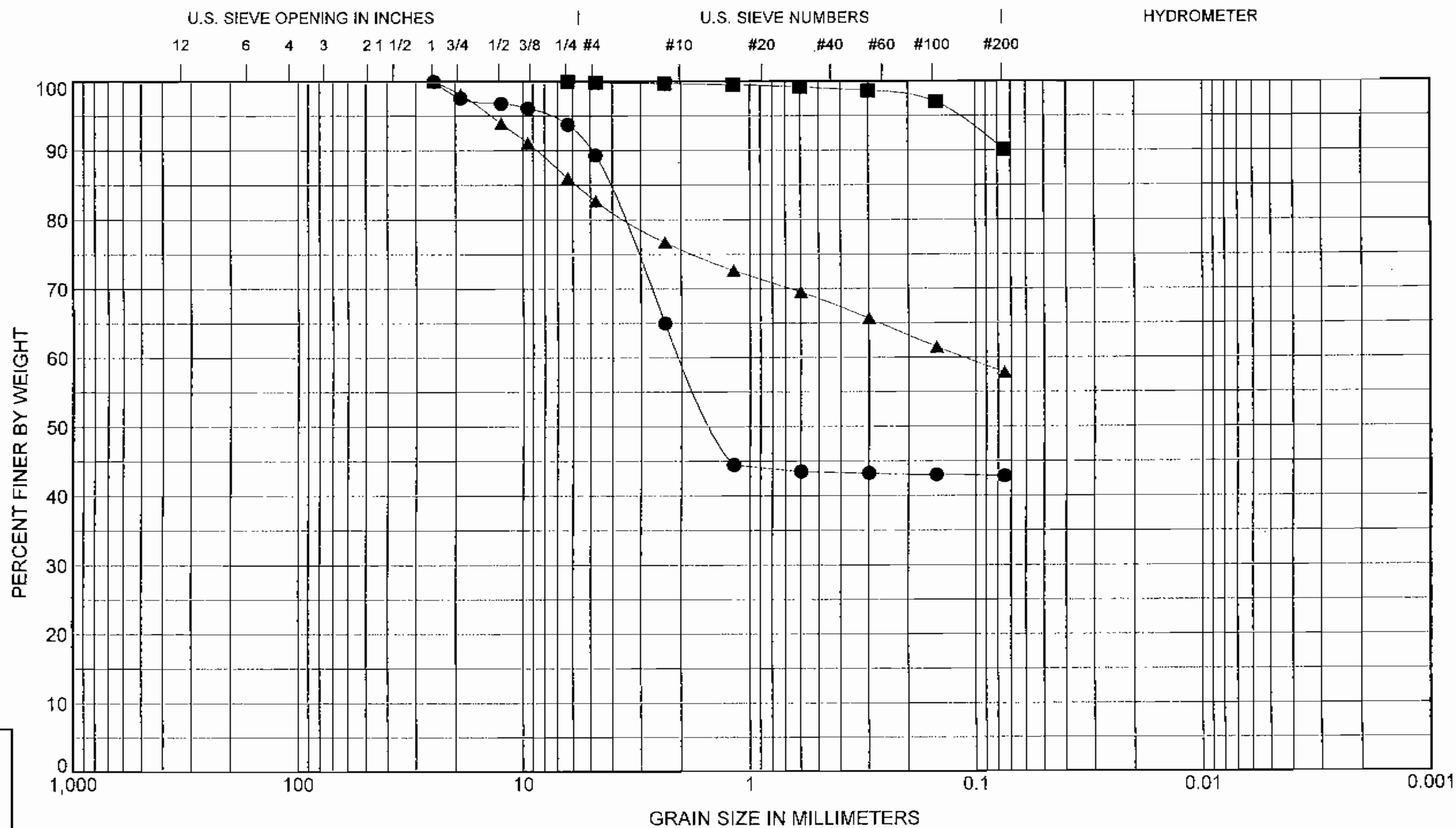
Fig. F-1
Sheet 5 of 12



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-6 S15	89.5 - 92.5	**Slightly gravelly, silty SAND, (SM)**					
■	A-6 S24	135.0 - 136.5	**Slightly sandy CLAY, (CL)**					
▲	A-7 S5	25.0 - 27.0	**Gravelly, sandy CLAY (CL)**					

	Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-6 S15	89.5 - 92.5	**25**	**2**			**11**	**46**	**43**	
■	A-6 S24	135.0 - 136.5	**6.3**				**0**	**10**	**90**	
▲	A-7 S5	25.0 - 27.0	**25**	**0.11**			**17**	**25**	**58**	

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Fig. F-1
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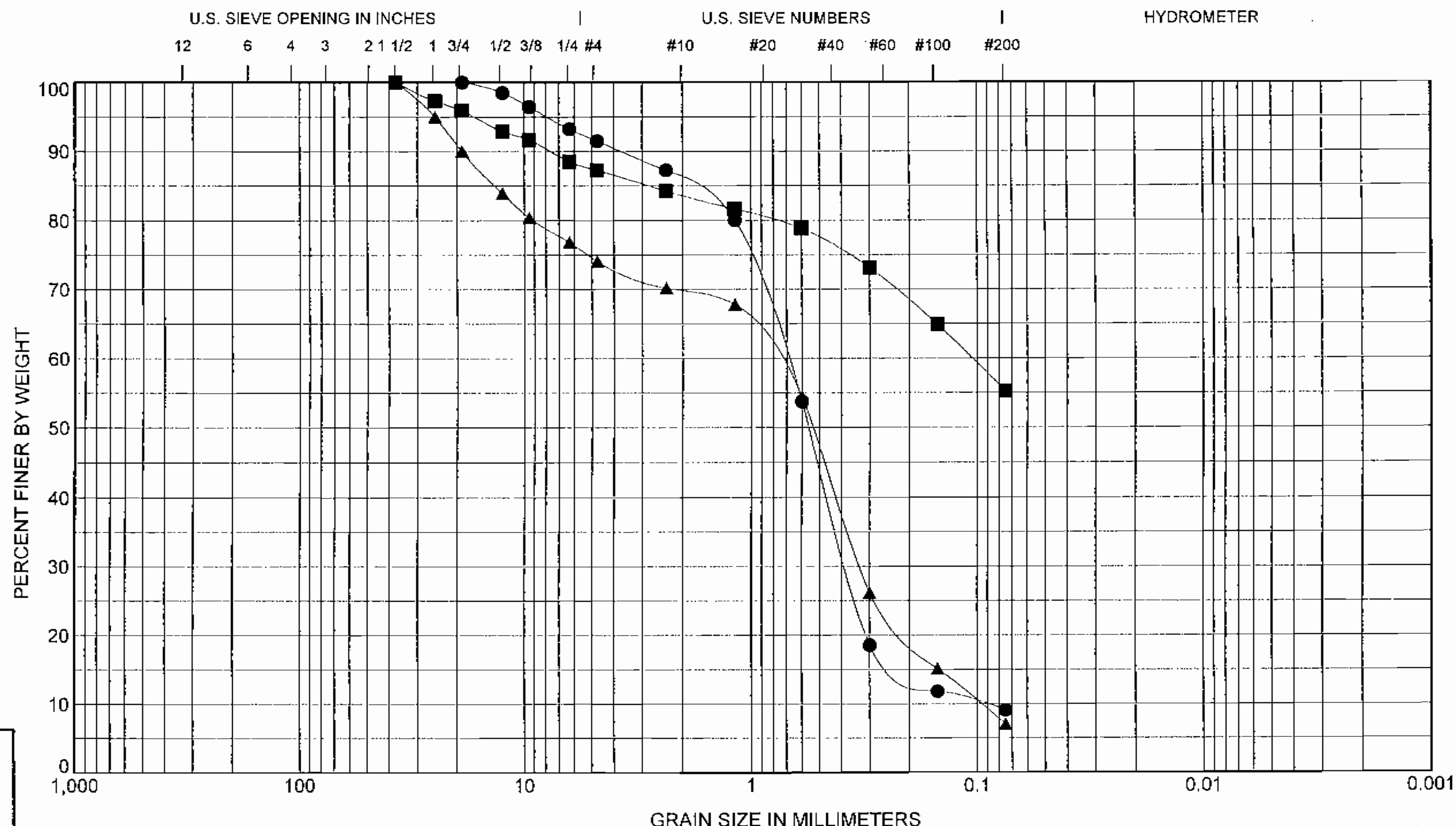

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample		Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-8 S9	40.5 - 41.5	**Slightly gravelly, slightly silty SAND, (SW-SM)**				2.1	7.5
■	A-8 S14	82.5 - 84.0	**Gravelly, sandy CLAY, (CL)**	19	12	7		
▲	A-9 S2	11.0 - 12.5	**Slightly silty, gravelly SAND, (SW-SM)**				1.4	8.2

Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-8 S9	40.5 - 41.5	19	0.7	0.38	0.09	8	82	9	
■	A-8 S14	82.5 - 84.0	37.5	0.11			13	32	55	
▲	A-9 S2	11.0 - 12.5	37.5	0.8	0.33	0.1	26	67	7	

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Fig. F-1
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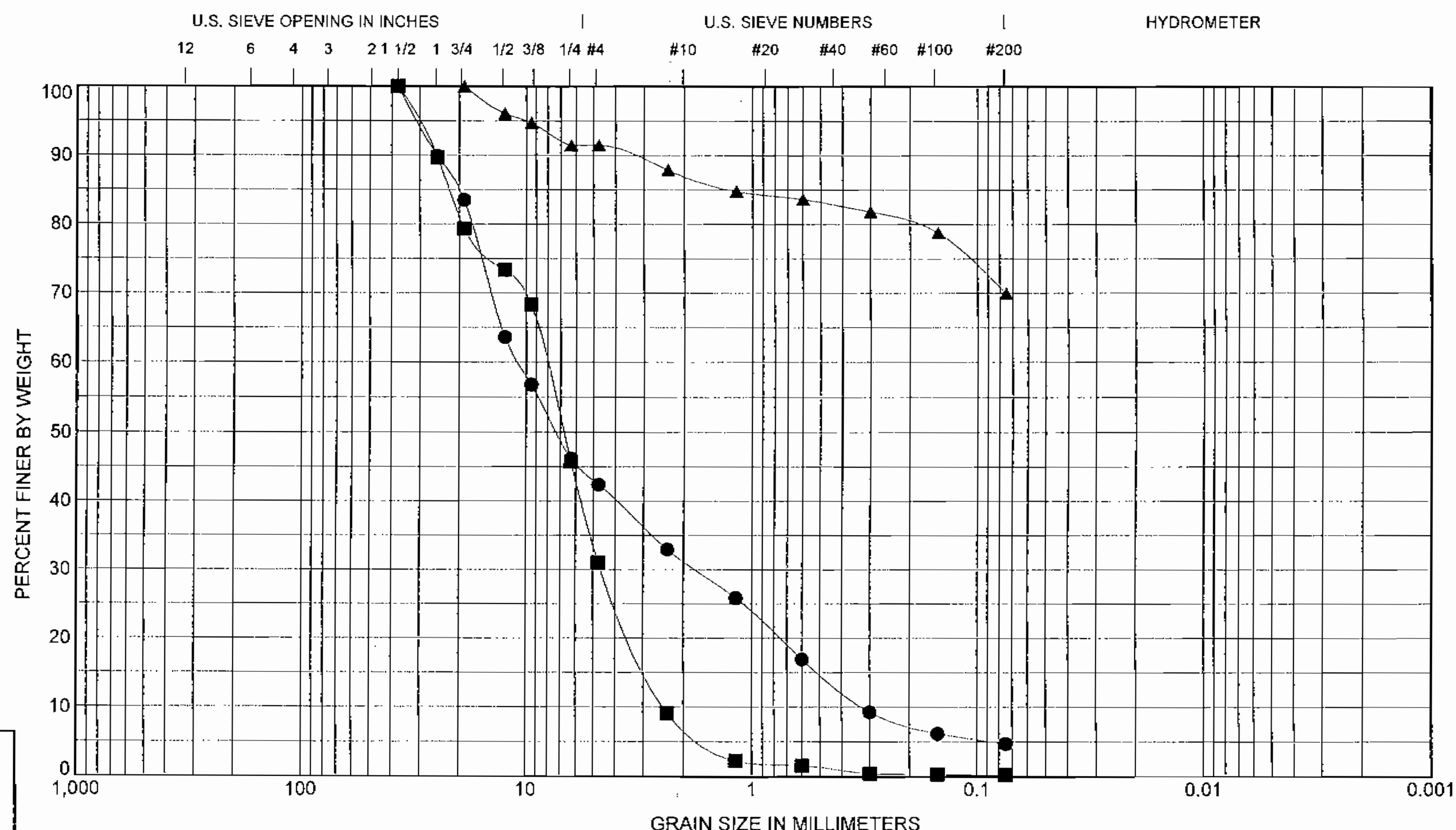

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample		Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-9 S5	32.0 - 33.0	**Slightly silty, sandy GRAVEL, (GP-GM)**				**0.9**	**33.8**
■	A-9 S9	56.0 - 56.5	**Sandy GRAVEL, (GP)**				**1.1**	**3.4**
▲	A-9 S17	114.0 - 114.9	**Slightly gravelly, sandy CLAY, (CL)**					

Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-9 S5	32.0 - 33.0	**37.5**	**10.81**	**1.76**	**0.32**	**58**	**38**	**5**	
■	A-9 S9	56.0 - 56.5	**37.5**	**8.17**	**4.59**	**2.42**	**69**	**31**	**0**	
▲	A-9 S17	114.0 - 114.9	**19**				**9**	**22**	**70**	

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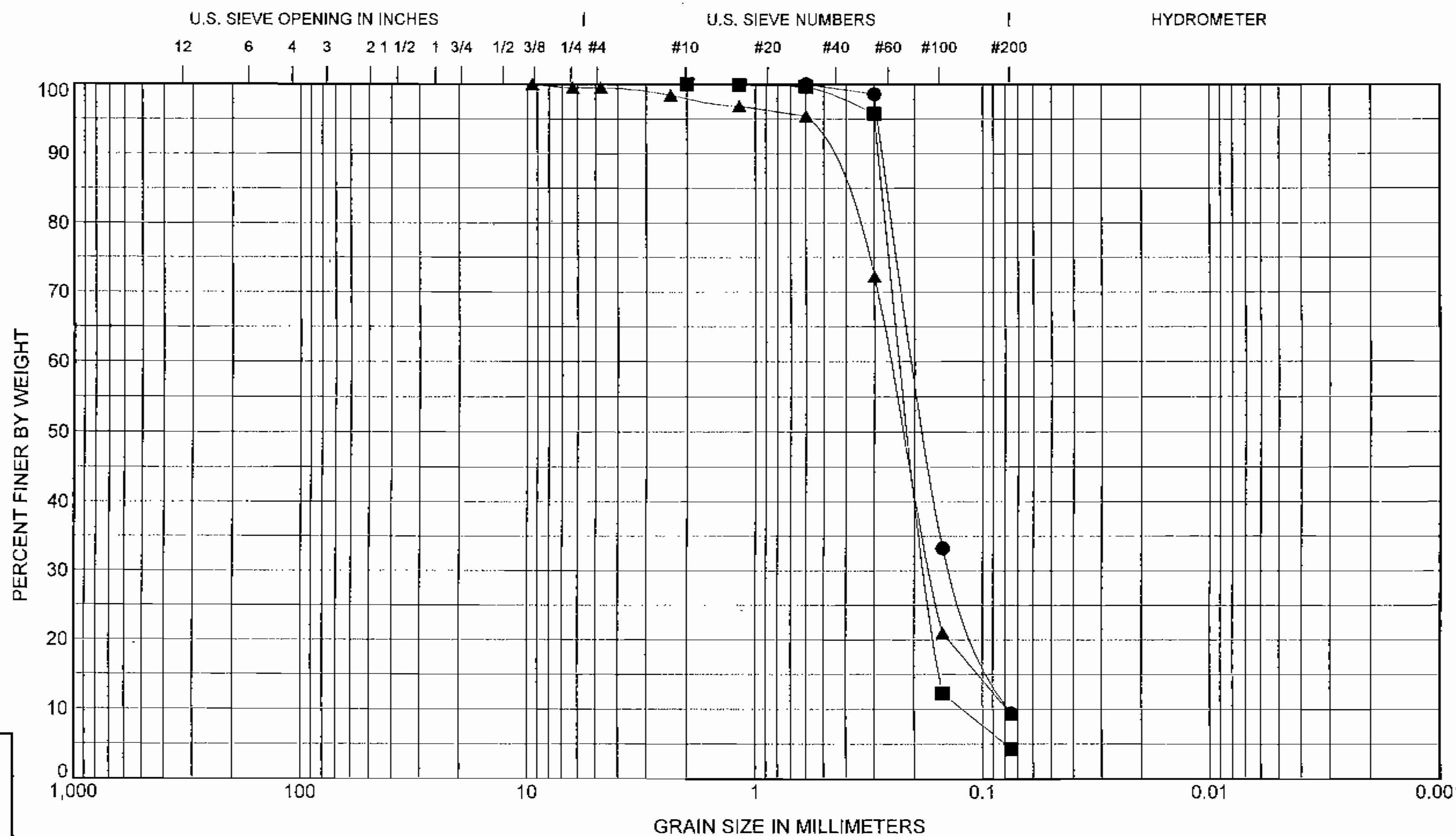

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample		Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-10 S2	12.0 - 13.5	**Slightly silty SAND, (SP-SM)**				1.2	2.6
■	A-10 S6	35.0 - 36.5	**SAND, trace silt, (SP)**				1.1	1.8
▲	A-10 S10	52.0 - 53.5	**Slightly silty SAND, (SP-SM)**				1.4	3.3

Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt / %Clay
●	A-10 S2	12.0 - 13.5	0.6	0.2	0.14	0.08	0	91	9
■	A-10 S6	35.0 - 36.5	2	0.22	0.17	0.12	0	96	4
▲	A-10 S10	52.0 - 53.5	9.5	0.25	0.17	0.08	1	90	9

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Fig. F-1
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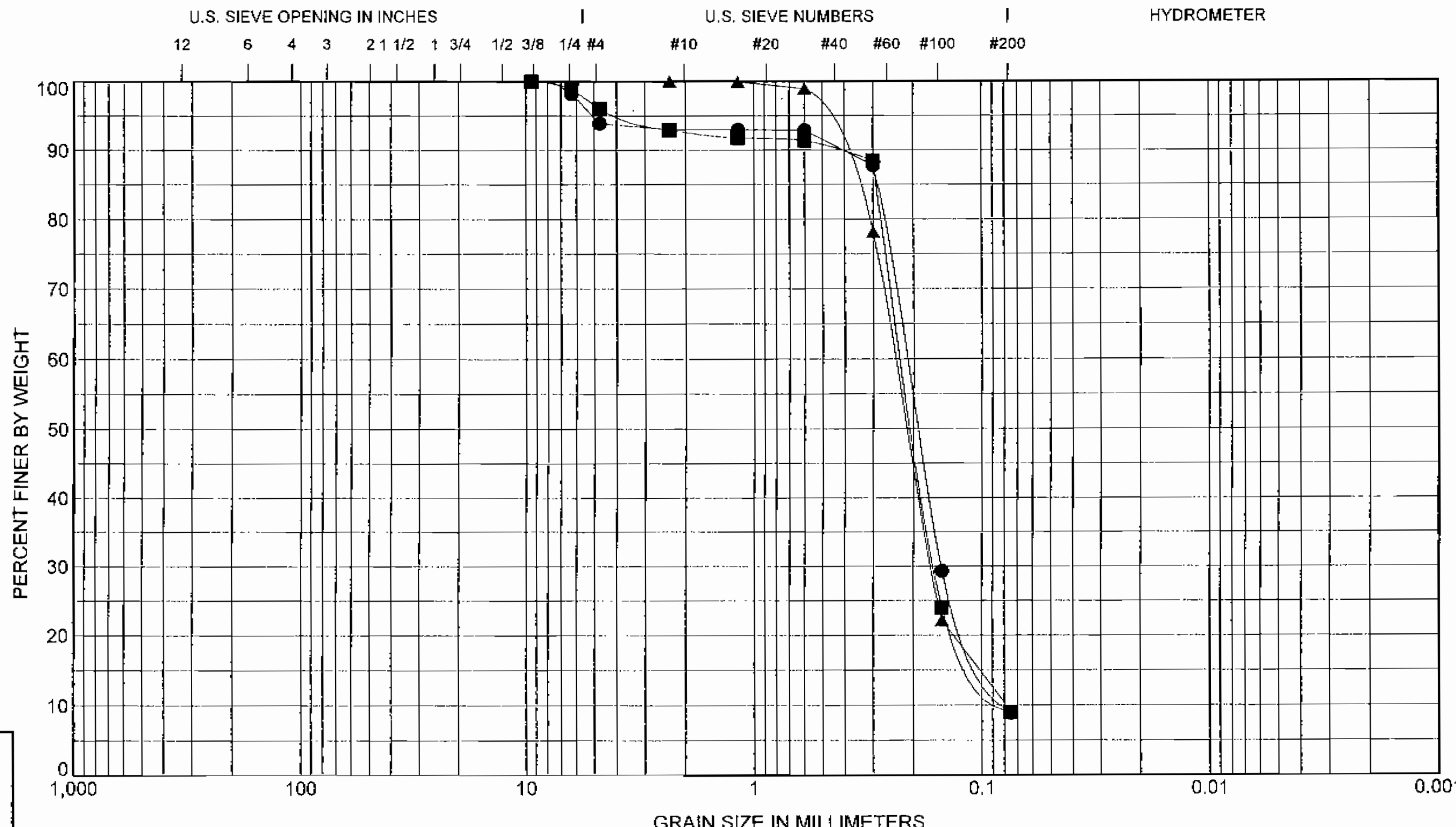

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-10 S13	73.5 - 75.0	**Slightly gravelly, slightly silty SAND, (SP-SM)**				1.4	2.8
■	A-10 S15	88.5 - 90.0	**Slightly silty SAND, trace gravel, (SP-SM)**				1.5	2.8
▲	A-10 S17	106.0 - 107.3	**Slightly silty SAND, (SP-SM)**				1.5	3.1

	Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-10 S13	73.5 - 75.0	9.5	0.22	0.15	0.08	6	85	9	
■	A-10 S15	88.5 - 90.0	9.5	0.22	0.16	0.08	4	87	9	
▲	A-10 S17	106.0 - 107.3	2.36	0.24	0.17	0.08	0	91	9	

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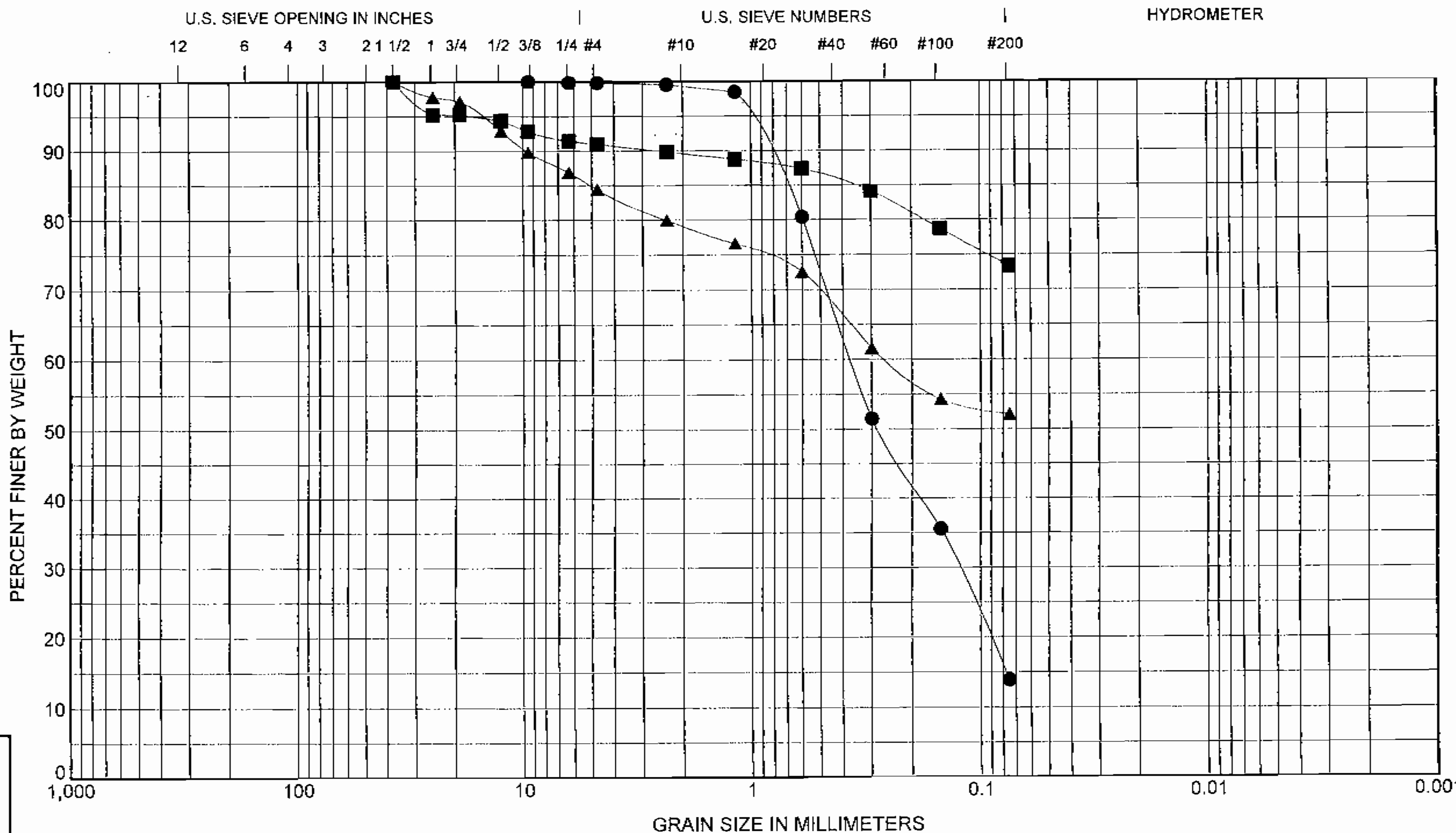
Fig. F-1
Sheet 10 of 12



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-10 S19	126.0 - 127.5	**Silty SAND, (SM)**					
■	A-11 S2	10.0 - 11.5	**Slightly gravelly, sandy CLAY (CL)**					
▲	A-13 S1	5.0 - 6.5	**Gravelly, sandy CLAY (CL)**					

	Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt / %Clay
●	A-10 S19	126.0 - 127.5	9.5	0.37	0.13		0	86	14
■	A-11 S2	10.0 - 11.5	37.5				9	18	73
▲	A-13 S1	5.0 - 6.5	37.5	0.26			16	32	52

Knik Arm Bridge
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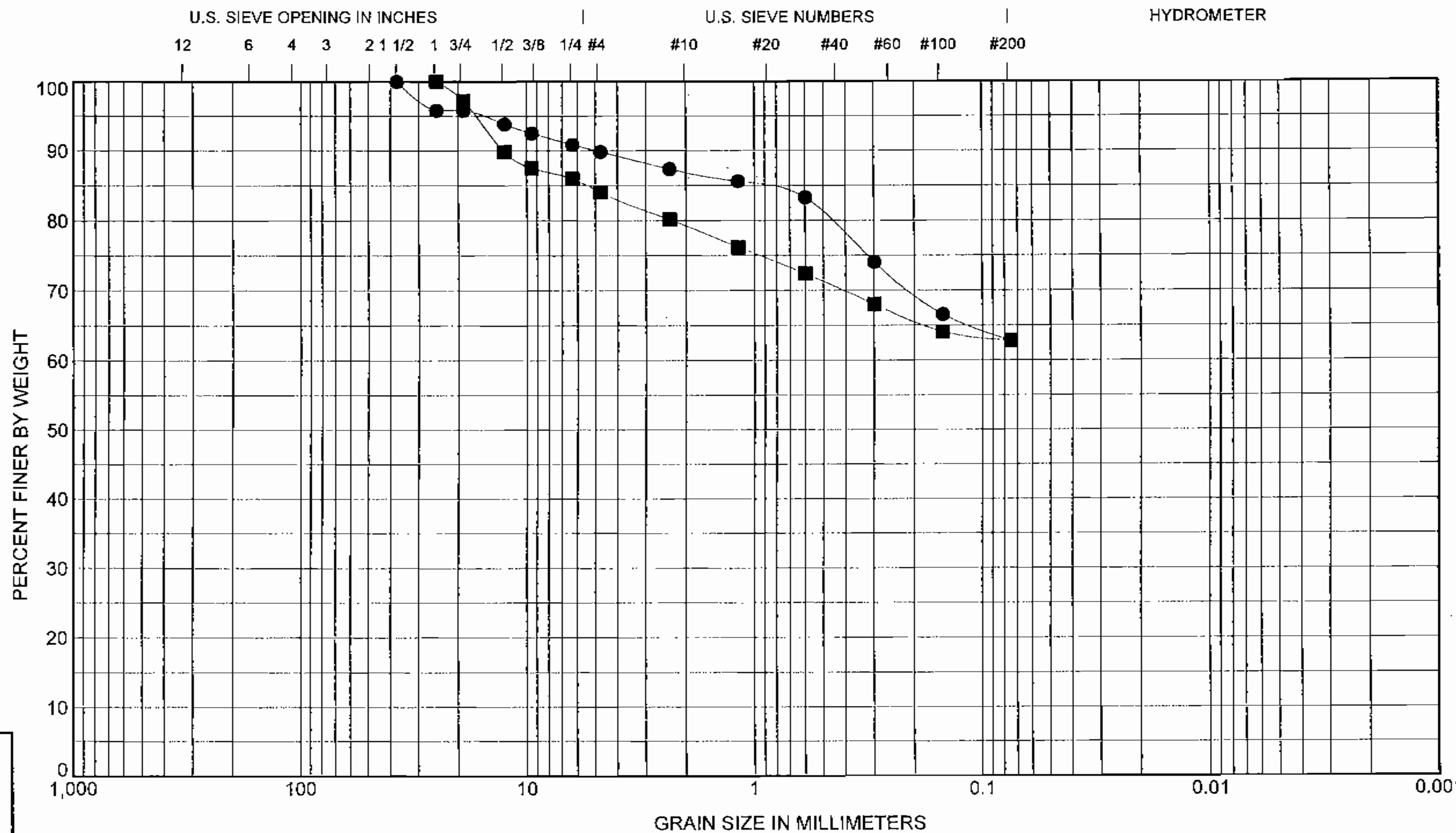
Fig. F-1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	A-16 S1	5.0 - 6.5	**Slightly gravelly, sandy CLAY (CL)**					
■	A-17 S2	10.0 - 11.5	**Gravelly, sandy CLAY (CL)**					

	Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-16 S1	5.0 - 6.5	37.5				10	27	63	
■	A-17 S2	10.0 - 11.5	25				16	21	63	

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Fig. F-1
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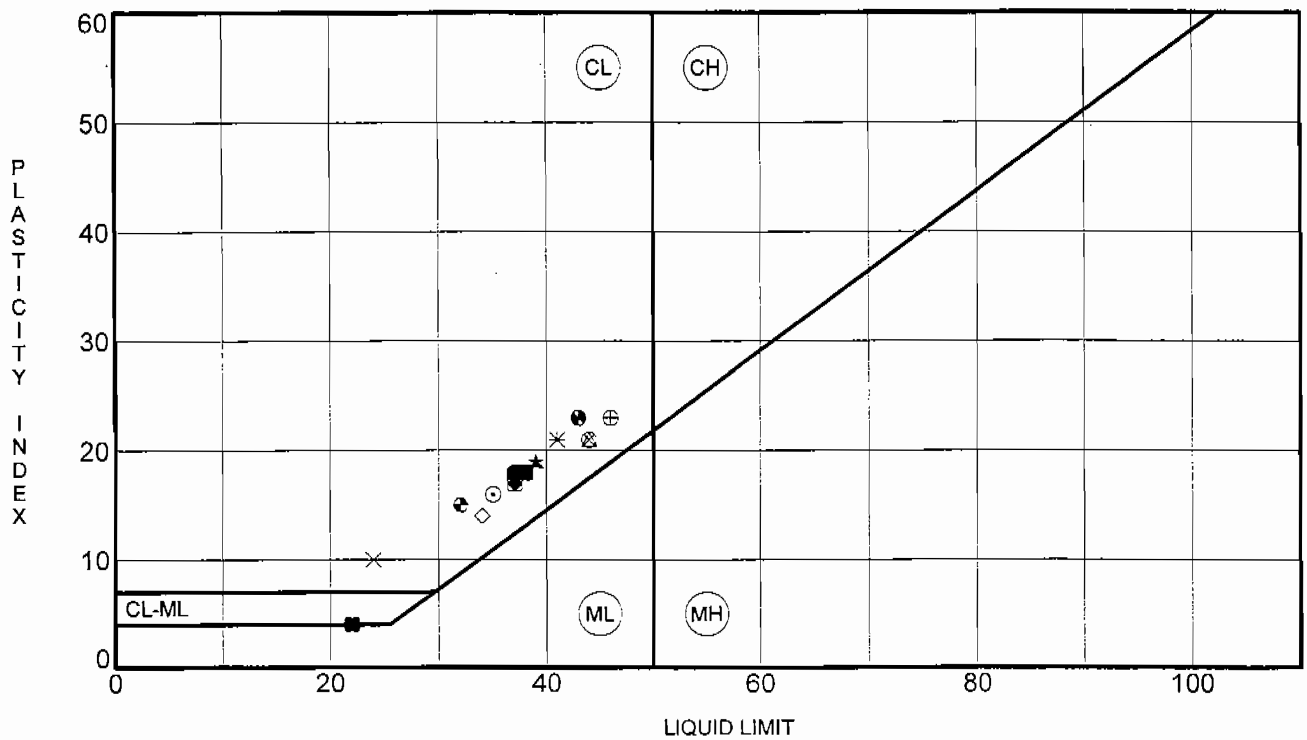

	Boring	Depth, Ft	LL	PL	PI	Fines	Classification
●	TP-A-1	23.0 - 24.5	37	19	18		CL
■	TP-A-1	35.0 - 36.5	38	20	18		CL
▲	TP-A-1	40.0 - 41.5	37	19	18		CL
○	TP-A-1	55.0 - 56.5	37	19	18		CL
⊙	TP-A-1	60.0 - 61.5	35	19	16		CL
△	TP-A-1	85.0 - 86.5	44	23	21		CL
✳	TP-A-1	100.0 - 101.5	41	20	21		CL
⅄	TP-A-1	115.0 - 116.5	39	20	19		CL
⊗	TP-A-1	120.0 - 121.5	44	23	21		CL
★	TP-A-1	130.0 - 131.5	39	20	19		CL
⊕	TP-A-1	135.0 - 136.5	46	23	23		CL
◒	TP-A-1	150.0 - 151.5	43	20	23		CL
◐	TP-A-1	172.5 - 174.0	43	20	23		CL
☆	TP-A-1	180.0 - 181.5	39	20	19		CL
☋	TP-A-1	187.5 - 189.0	37	20	17		CL
◑	TP-A-1	210.0 - 211.5	32	17	15		CL
◆	TP-A-1	235.0 - 236.5	37	20	17		CL

Knik Arm Bridge
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ATTERBERG LIMITS RESULTS

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Fig. F-2
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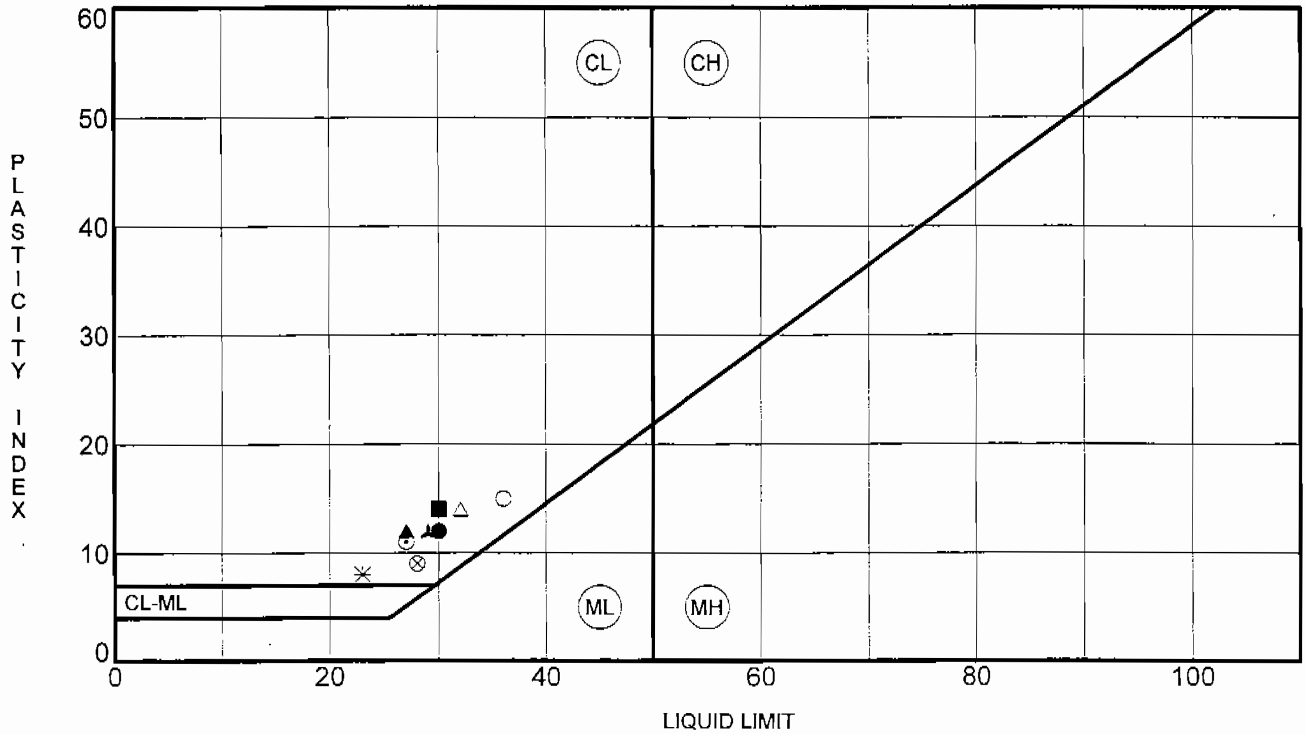

Boring		Depth, Ft	LL	PL	PI	Fines	Classification
●	**TP-A-1**	**295.0 - 296.5**	**30**	**18**	**12**		**CL**
■	**TP-A-1**	**305.0 - 306.5**	**30**	**16**	**14**		**CL**
▲	**TP-A-1**	**315.0 - 316.5**	**27**	**15**	**12**		**CL**
○	**TP-A-1**	**325.0 - 326.5**	**36**	**21**	**15**		**CL**
⊙	**TP-A-1**	**335.0 - 336.5**	**27**	**16**	**11**		**CL**
△	**TP-A-2**	**160.0 - 161.5**	**32**	**18**	**14**		**CL**
✳	**TP-A-2**	**167.0 - 168.5**	**23**	**15**	**8**		**CL**
⅄	**TP-A-2**	**176.0 - 177.5**	**29**	**17**	**12**		**CL**
⊗	**TP-A-2**	**196.0 - 197.5**	**28**	**19**	**9**		**CL**

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ATTERBERG LIMITS RESULTS

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SHANNON & WILSON, INC.
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Fig. F-2
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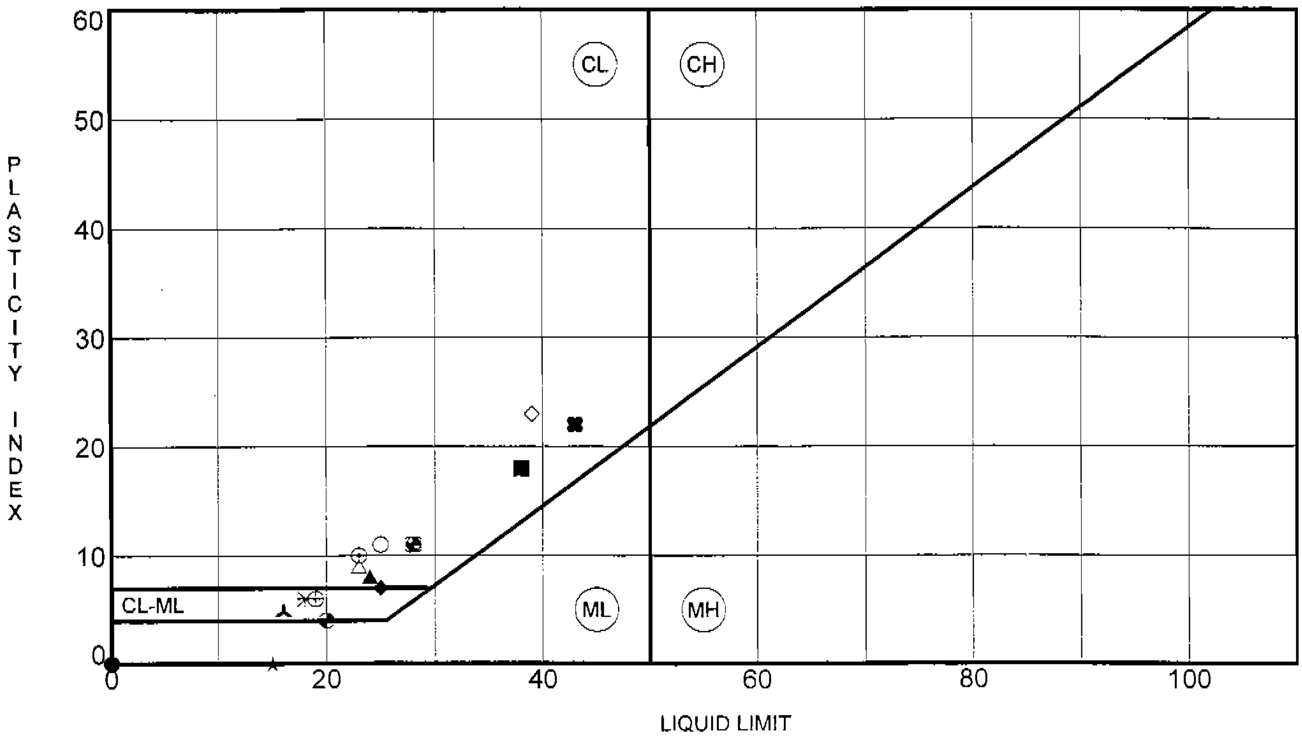

Boring		Depth, Ft	LL	PL	PI	Fines	Classification
●	TP-A-4	11.0 - 12.5	NP	NP	NP	31	NP
■	TP-A-4	36.0 - 37.5	38	20	18		
▲	TP-A-4	37.5 - 39.0	24	16	8		CL
○	TP-A-4	41.0 - 42.5	25	14	11		CL
⊙	TP-A-4	42.5 - 44.0	23	13	10		CL
△	TP-A-4	51.0 - 52.5	23	14	9		CL
✳	TP-A-4	56.0 - 57.5	18	12	6		CL-ML
⅄	TP-A-4	73.5 - 75.0	16	11	5		CL-ML
⊗	TP-A-4	81.0 - 82.5	NP	NP	NP		NP
★	TP-A-4	96.0 - 97.5	15	15	NP		ML
⊕	TP-A-4	116.0 - 117.5	19	13	6		CL-ML
⊖	TP-A-4	123.5 - 125.0	NP	NP	NP		NP
◕	TP-A-4	131.0 - 132.5	20	16	4		CL-ML
☆	TP-A-4	146.0 - 147.5	28	17	11		CL
⌘	TP-A-4	156.0 - 157.5	28	17	11		CL
◔	TP-A-4	166.0 - 167.5	28	17	11		CL
◆	TP-A-4	181.0 - 182.5	25	18	7		CL-ML

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ATTERBERG LIMITS RESULTS

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Fig. F-2
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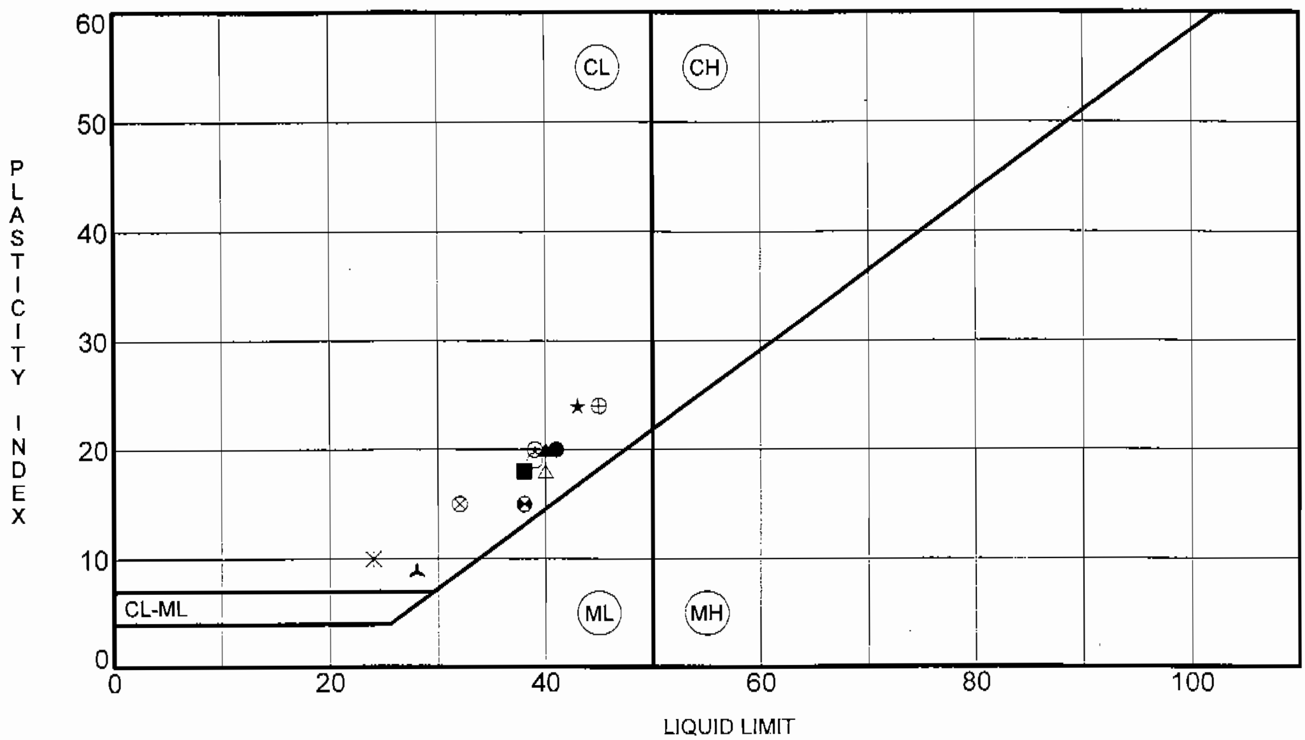

Boring		Depth, Ft	LL	PL	PI	Fines	Classification
●	TP-A-5	60.5 - 62.0	41	21	20		CL
■	TP-A-5	68.0 - 69.5	38	20	18		CL
▲	TP-A-5	83.0 - 84.5	40	20	20		CL
○	TP-A-5	108.0 - 109.5	39	20	19		CL
⊙	TP-A-5	151.0 - 152.5	39	19	20		CL
△	TP-A-5	183.0 - 184.5	40	22	18		CL
✳	TP-A-6	21.0 - 22.5	24	14	10		CL
⅄	TP-A-6	36.0 - 37.5	28	19	9		CL
⊗	TP-A-6	42.0 - 43.5	32	17	15		CL
★	TP-A-6	97.0 - 98.5	43	19	24		CL
⊕	TP-A-6	105.0 - 106.5	45	21	24		CL
⊖	TP-A-6	175.0 - 176.5	38	23	15		CL

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Fig. F-2
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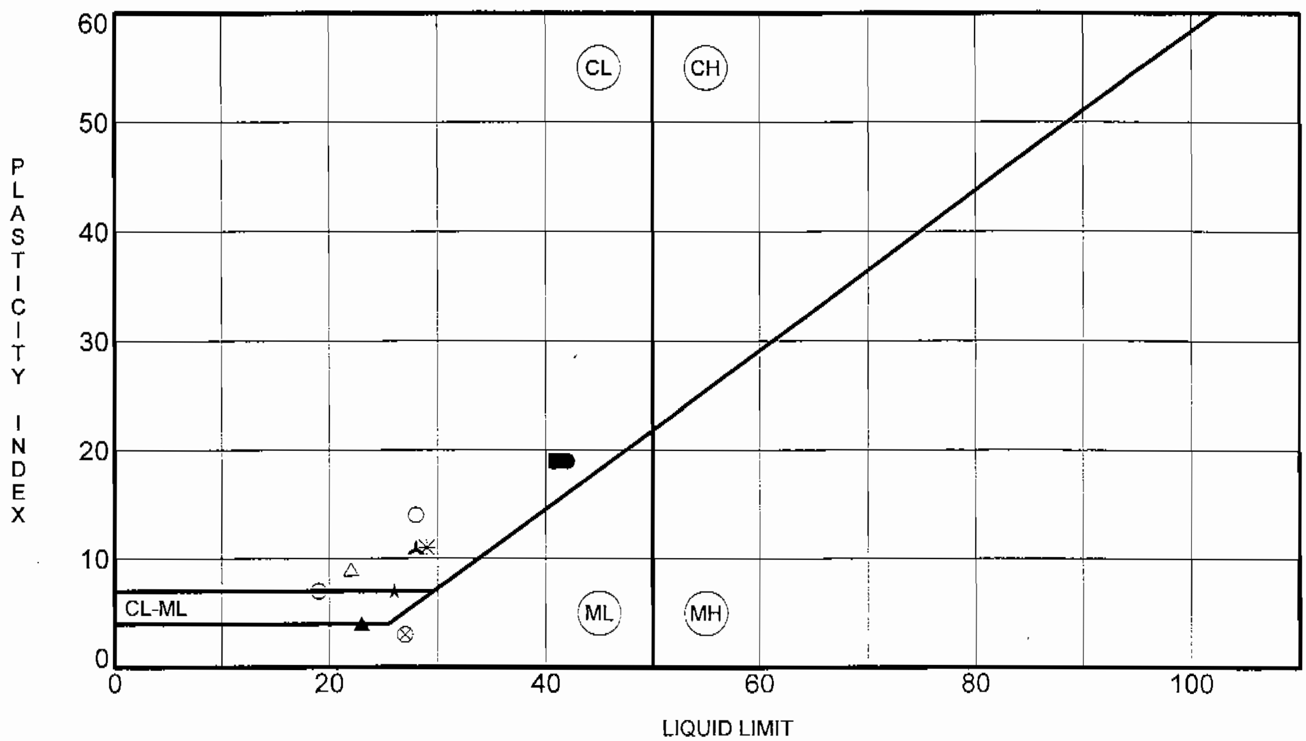

Boring	Depth, Ft	LL	PL	PI	Fines	Classification
● TP-A-7	145.0 - 146.5	42	23	19		CL
■ TP-A-7	165.0 - 166.5	41	22	19		CL
▲ TP-A-7	185.0 - 186.5	23	19	4		CL-ML
○ TP-A-8	60.5 - 62.0	28	14	14		CL
⊙ TP-A-8	82.5 - 84.0	19	12	7	55	CL
△ TP-A-8	130.0 - 131.5	22	13	9		CL
✳ TP-A-8	170.0 - 171.5	29	18	11		CL
⅄ TP-A-8	185.0 - 186.5	28	17	11		CL
⊗ TP-A-9	104.0 - 105.5	27	24	3		ML
★ TP-A-9	109.0 - 110.5	26	19	7		CL

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ATTERBERG LIMITS RESULTS

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Fig. F-2
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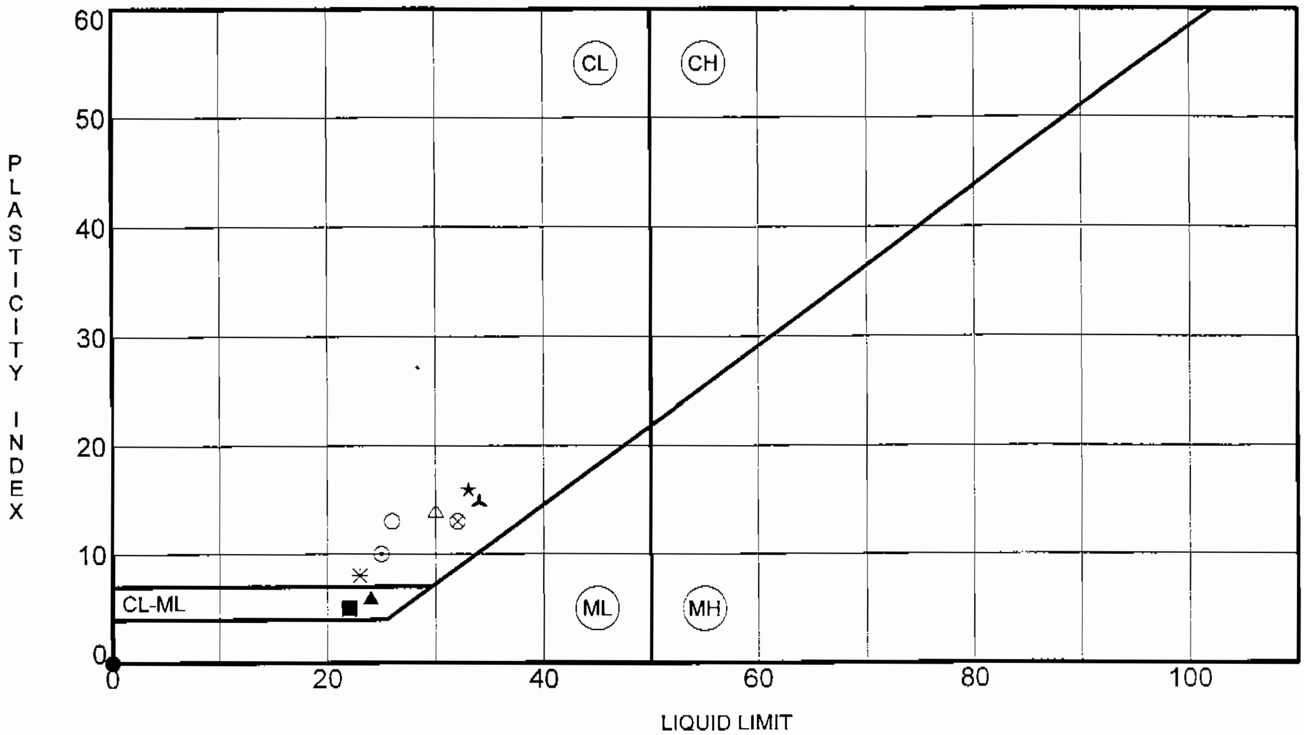

	Boring	Depth, Ft	LL	PL	PI	Fines	Classification
●	**TP-A-10**	**166.0 - 167.5**	**NP**	**NP**	**NP**		**NP**
■	**TP-A-10**	**175.5 - 177.0**	**22**	**17**	**5**		**CL-ML**
▲	**TP-A-10**	**186.5 - 188.0**	**24**	**18**	**6**		**CL-ML**
○	**TP-A-11**	**30.0 - 31.5**	**26**	**13**	**13**		**CL**
⊙	**TP-A-12**	**10.0 - 11.5**	**25**	**15**	**10**		**CL**
△	**TP-A-12**	**25.0 - 26.5**	**30**	**16**	**14**		**CL**
✳	**TP-A-13**	**25.0 - 26.5**	**23**	**15**	**8**		**CL**
⅄	**TP-A-16**	**30.0 - 31.5**	**34**	**19**	**15**		**CL**
⊗	**TP-A-17**	**15.0 - 16.5**	**32**	**19**	**13**		**CL**
★	**TP-A-17**	**30.0 - 31.5**	**33**	**17**	**16**		**CL**

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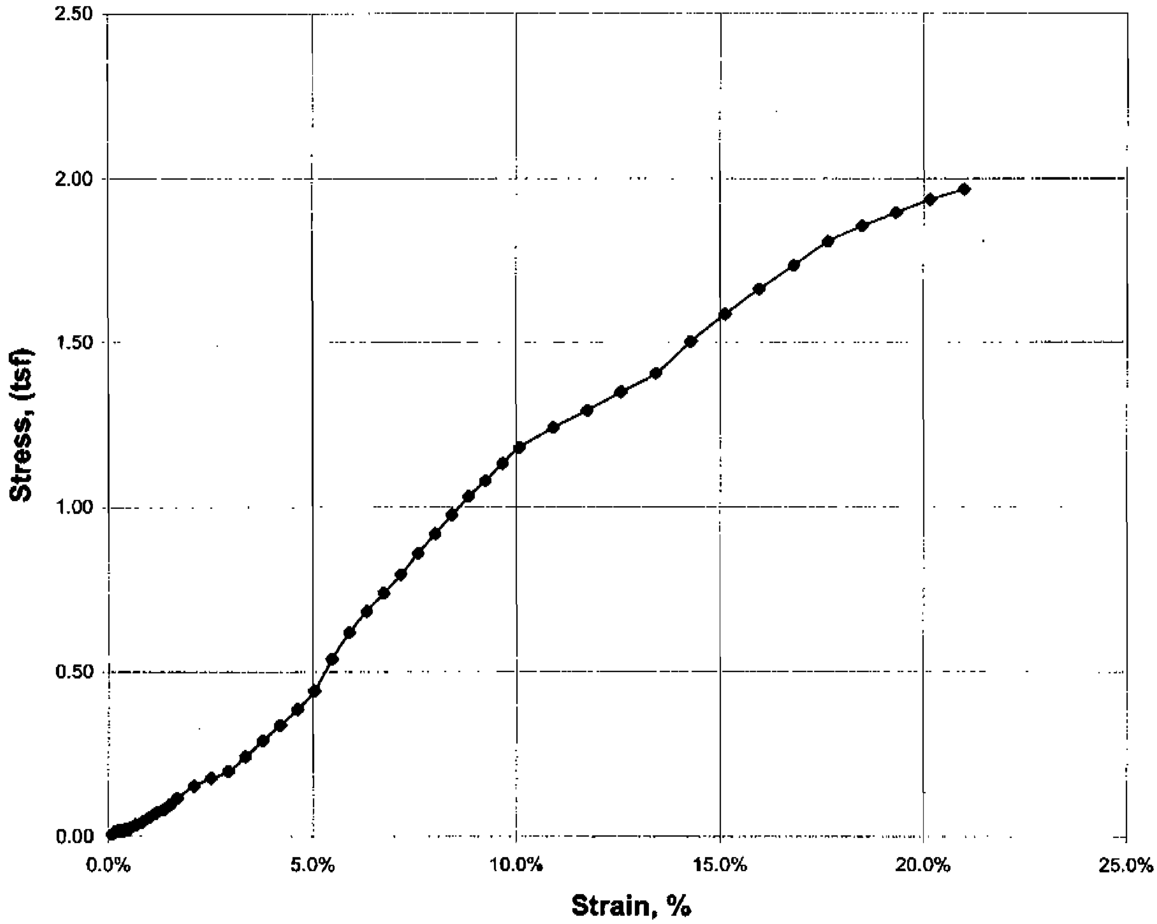


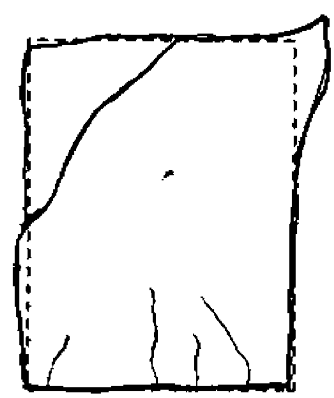
Sample Data

Sample Depth: 23-25ft
Initial Diameter: 2.729 in.
Initial Height: 5.952 In.
Initial Moisture Content: 23%
Liquid Limit: 37
Plastic Limit: 19
Wet Density: 131.8lb/ft3

Maximum Stress: 2tons/ft2
Strain at Maximum Stress: 21%

Classification: Gray, silty CLAY (CL)

Failure Sketch



Knik Arm Bridge Anchorage, Alaska	
UNCONFINED COMPRESSION TEST RESULTS	
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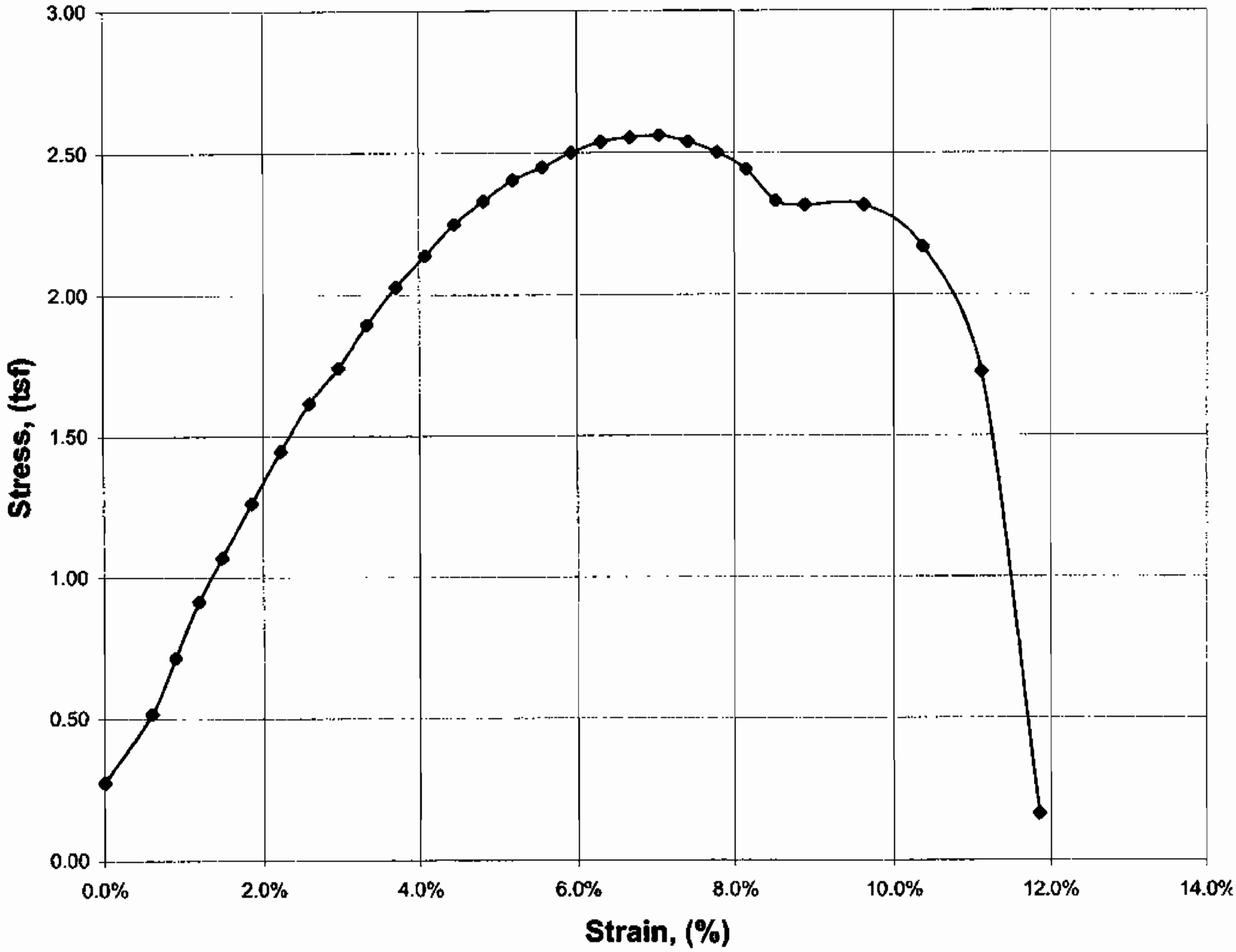
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	**Fig. F-3** **Sheet 1 of 31**

A1 S9



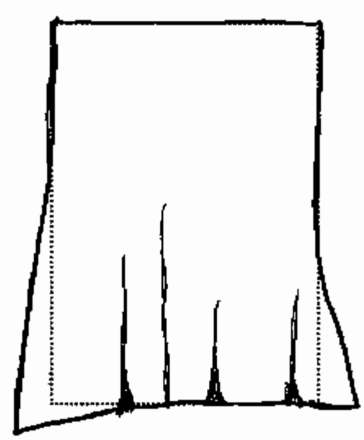
Sample Data

Sample Depth: 35-37ft
Initial Diameter: 2.847in.
Initial Height: 6.75In.
Initial Moisture Content:24 %
Liquid Limit: 38
Plastic Limit: 20
Wet Density: 131lb/ft3

Maximum Stress: 2.6tons/ft2
Strain at Maximum Stress: 7%

Classification: Gray, silty CLAY (CL)

Failure Sketch



Knik Arm Bridge
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UNCONFINED COMPRESSION TEST RESULTS

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Fig. F-3
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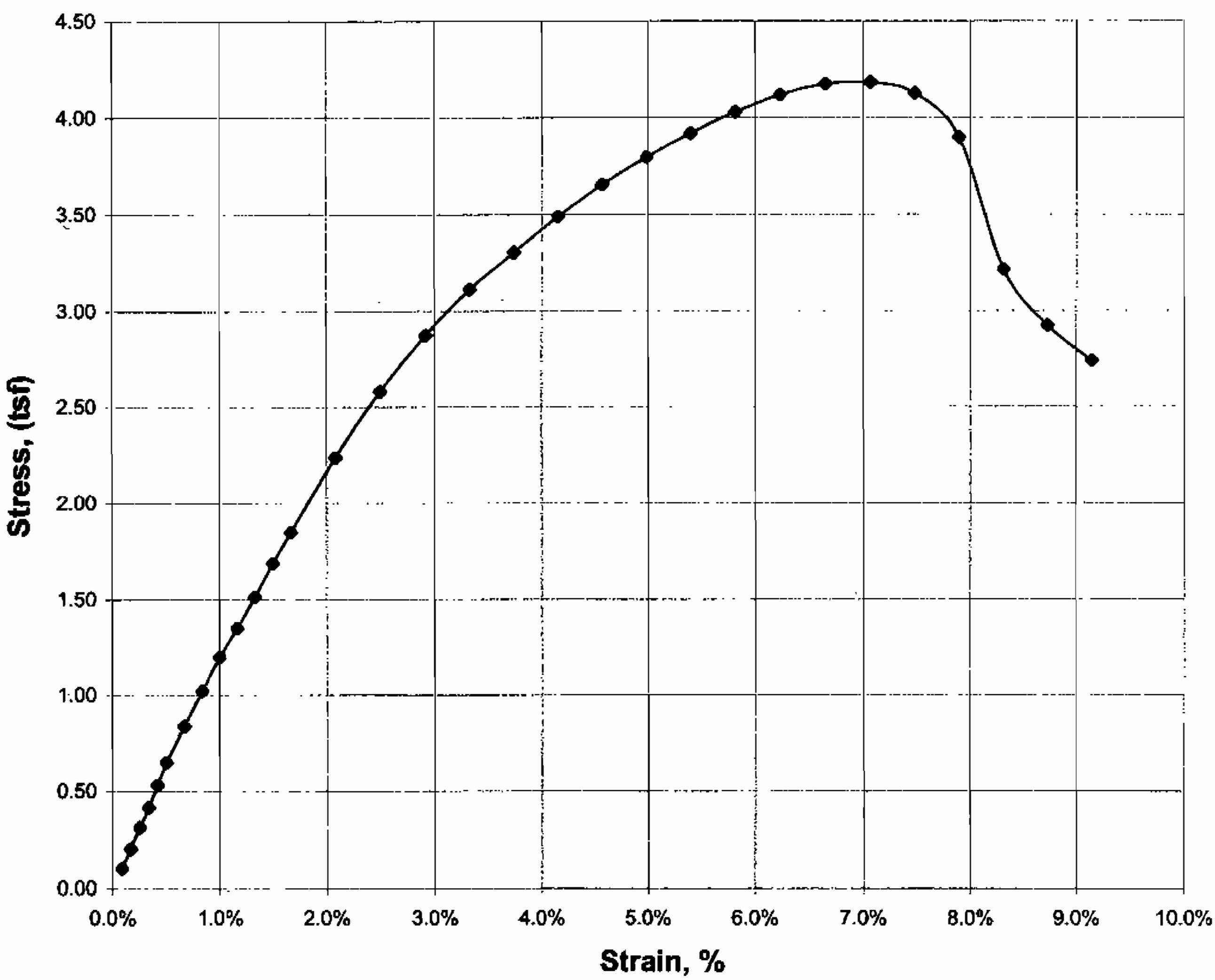


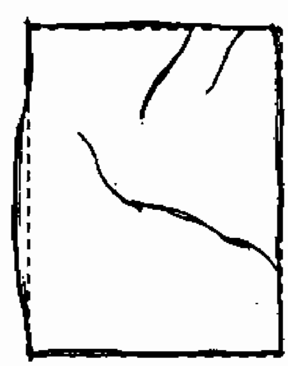
Sample Data

Sample Depth: 40-42ft
Initial Diameter: 2.860 in.
Initial Height: 6.061 In.
Initial Moisture Content: 24%
Liquid Limit: 37
Plastic Limit: 19
Wet Density: 131.2lb/ft3

Maximum Stress: 4.2 tons/ft2
Strain at Maximum Stress: 7%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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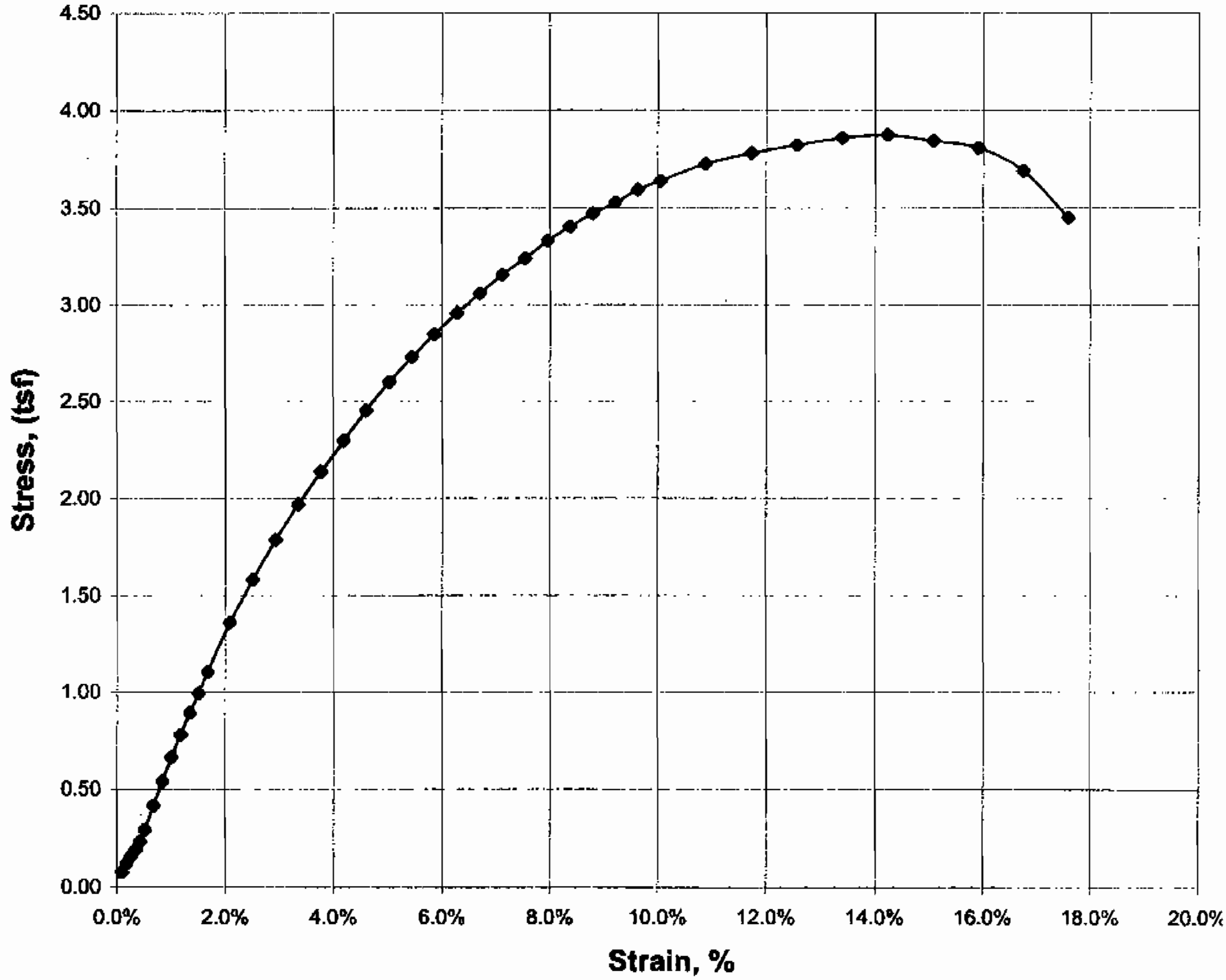


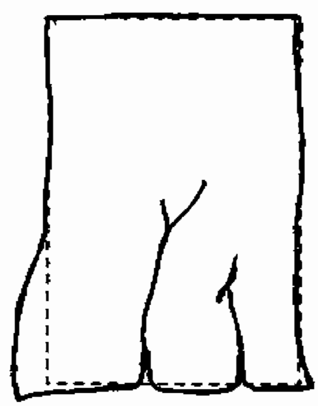
Sample Data

Sample Depth: 55-57ft
Initial Diameter: 2.858 in.
Initial Height: 5.972 In.
Initial Moisture Content: 24%
Liquid Limit: 37
Plastic Limit: 19
Wet Density: 129.8 lb/ft3

Maximum Stress: 3.9 tons/ft2
Strain at Maximum Stress: 14%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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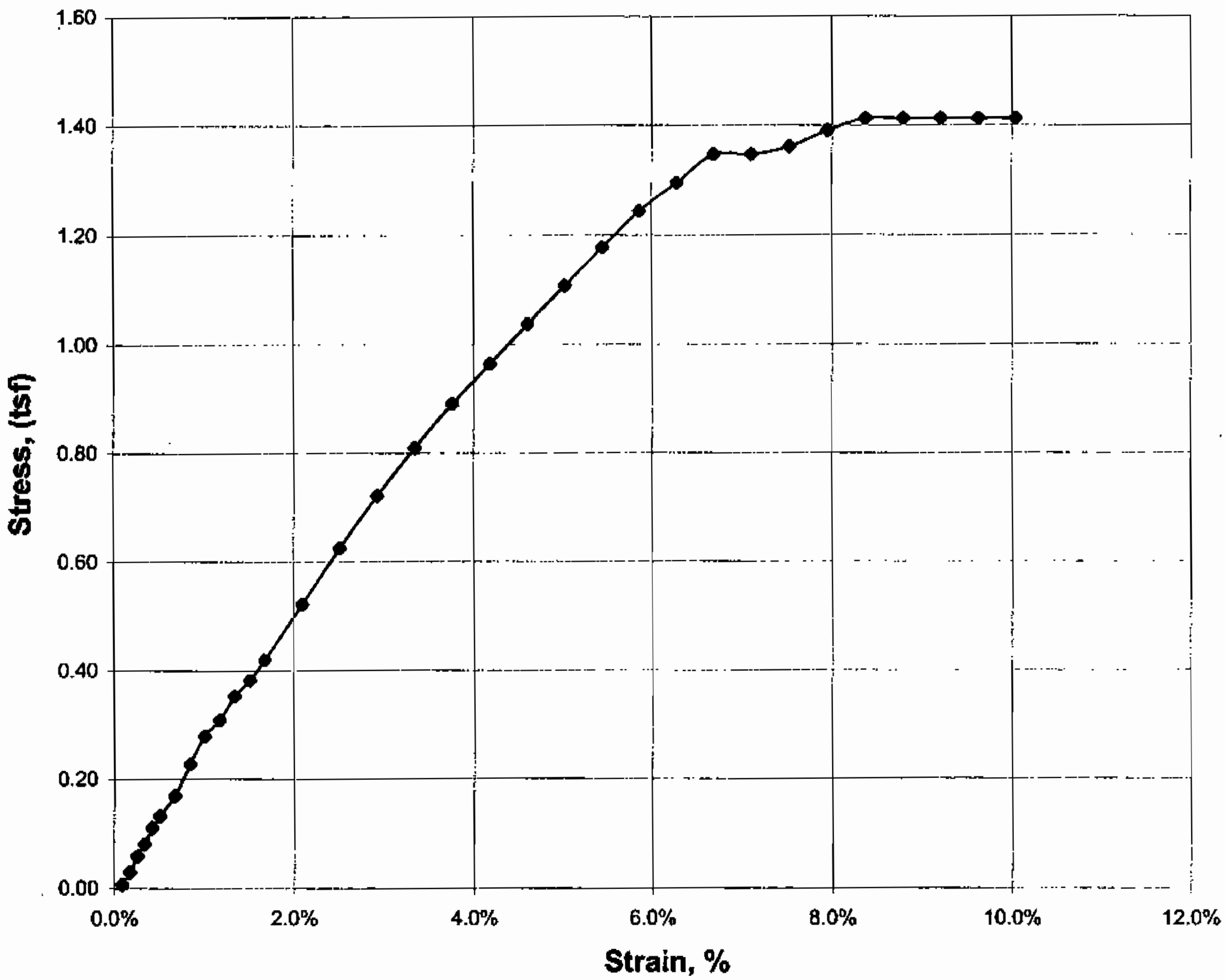


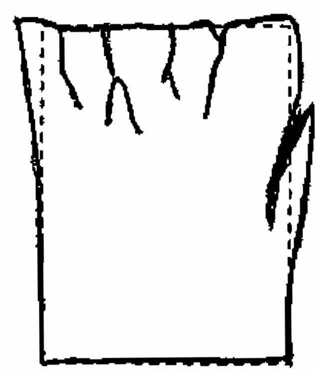
Sample Data

Sample Depth: 60-62ft
Initial Diameter: 2.846 in.
Initial Height: 5.975 In.
Initial Moisture Content: 19%
Liquid Limit: 35
Plastic Limit: 19
Wet Density: 133.8lb/ft3

Maximum Stress: 1.4 tons/ft2
Strain at Maximum Stress: 9%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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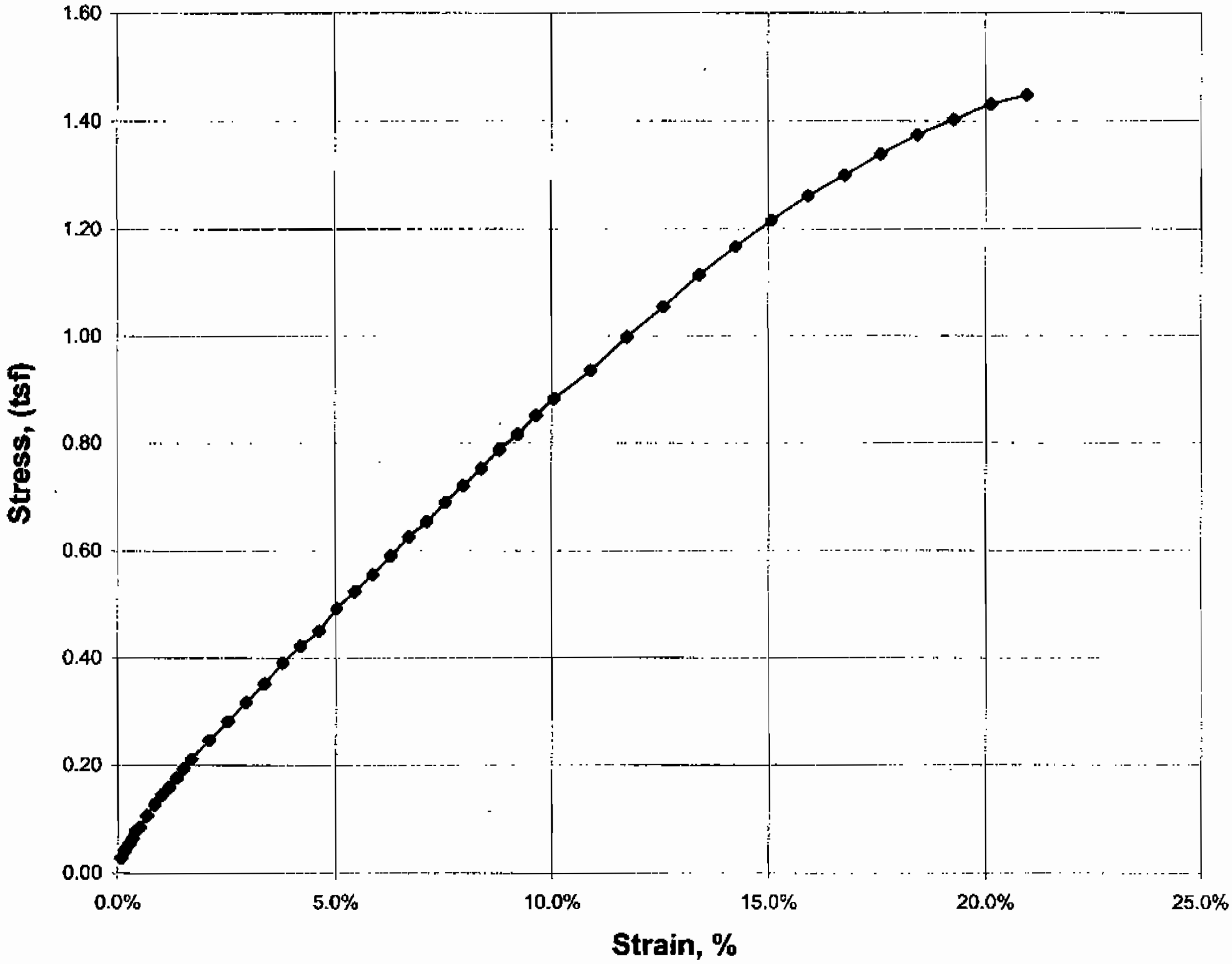
A1 S22



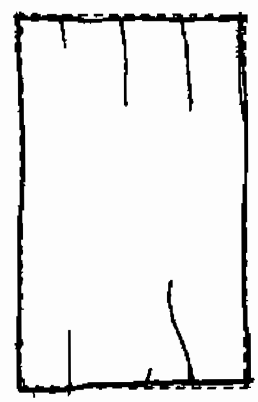
Sample Data

Sample Depth: 100-102ft
Initial Diameter: 2.917 in.
Initial Height: 5.967 In.
Initial Moisture Content: 33%
Liquid Limit: 41
Plastic Limit: 20
Wet Density: 120.5 lb/ft3

Maximum Stress: 1.5 tons/ft2
Strain at Maximum Stress: 21%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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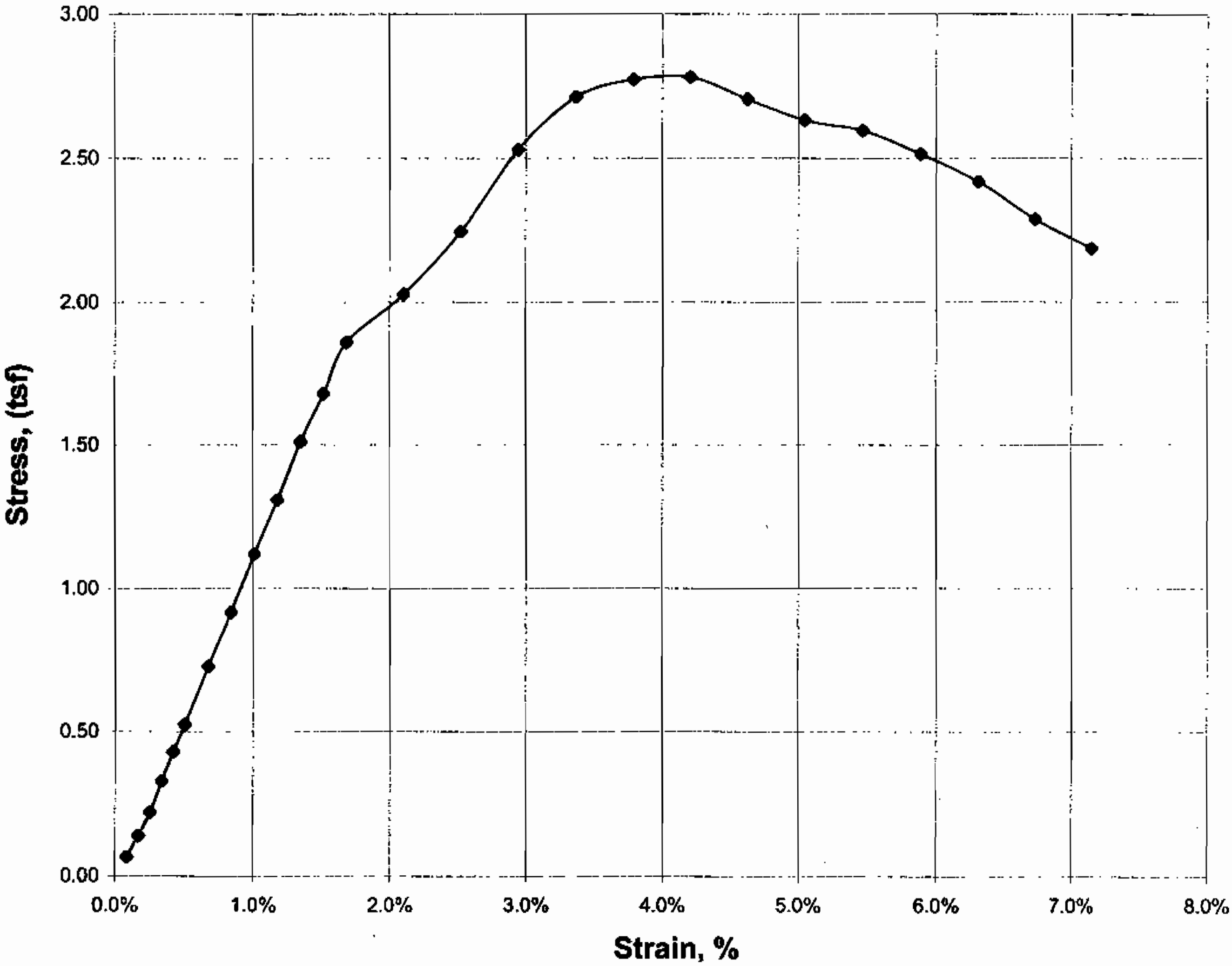


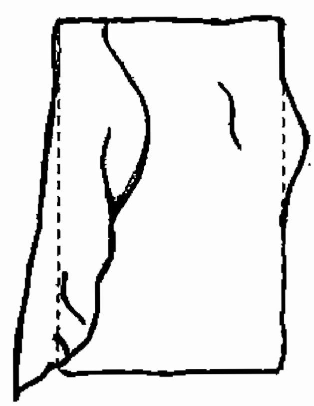
Sample Data

Sample Depth: 115-117ft
Initial Diameter: 2.87 in.
Initial Height: 5.946 In.
Initial Moisture Content: 29%
Liquid Limit: 39
Plastic Limit: 20
Wet Density: 127.2 lb/ft3

Maximum Stress: 2.8 tons/ft2
Strain at Maximum Stress: 4.2%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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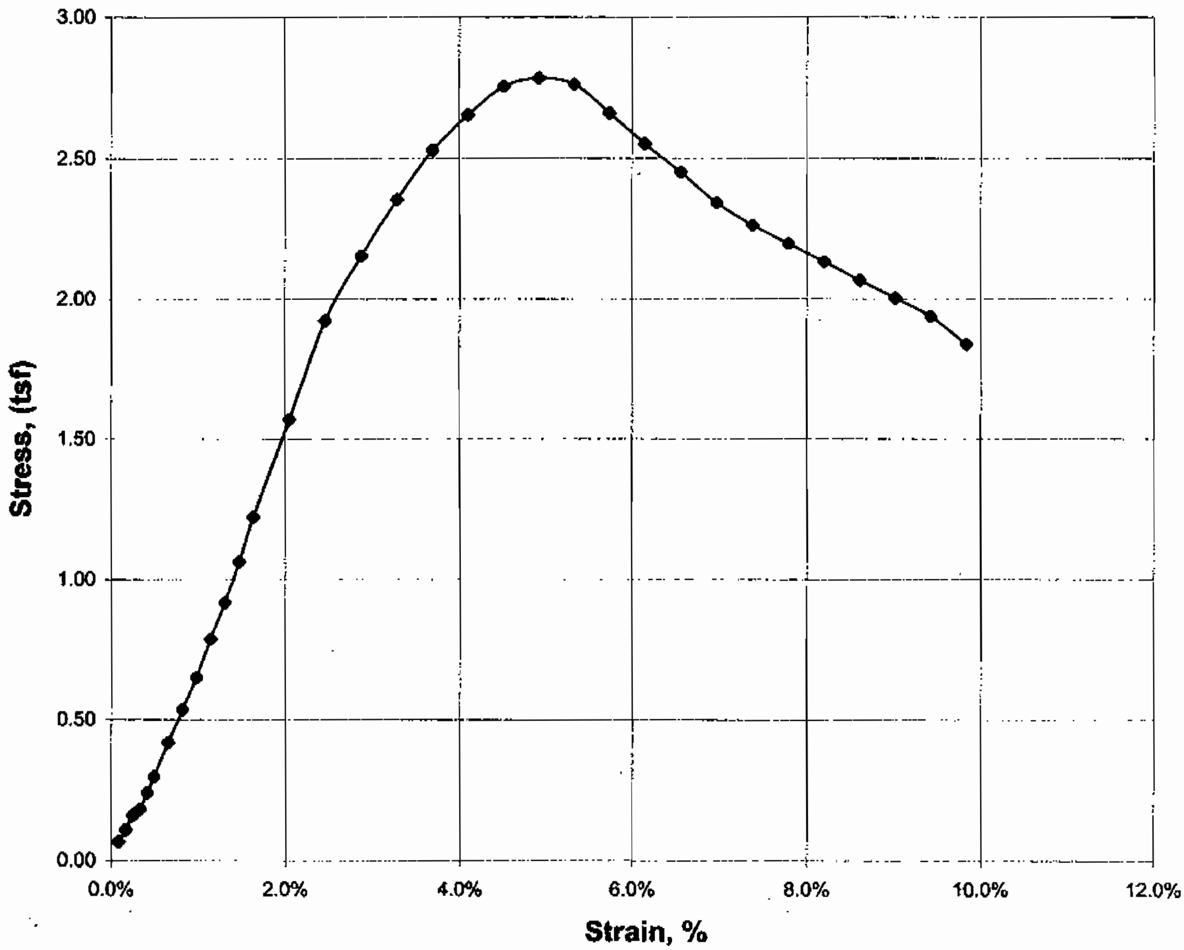
A1 S28



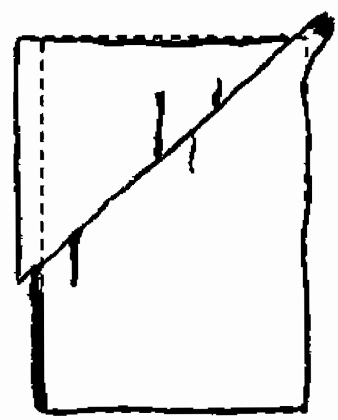
Sample Data

Sample Depth: 130-132ft
Initial Diameter: 2.876 in.
Initial Height: 6.10 In.
Initial Moisture Content: 28%
Liquid Limit: 39
Plastic Limit: 20
Wet Density: 122.4 lb/ft3

Maximum Stress: 2.8 tons/ft2
Strain at Maximum Stress: 5%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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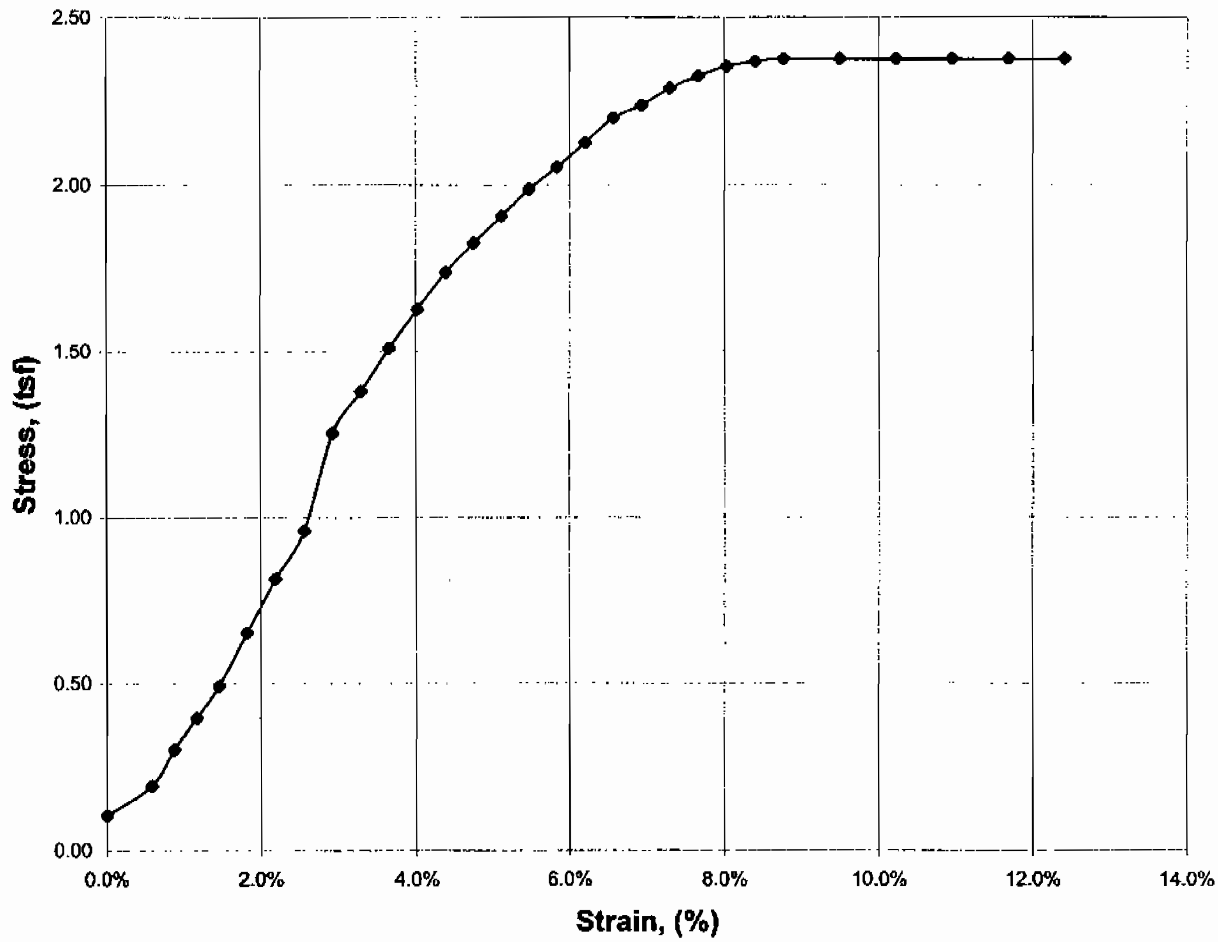


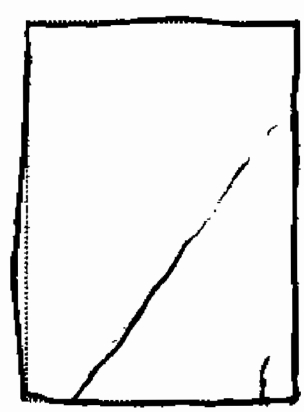
Sample Data

Sample Depth: 135-137ft
Initial Diameter: 2.856in.
Initial Height: 6.85In.
Initial Moisture Content:27 %
Liquid Limit: 46
Plastic Limit: 23
Wet Density: 129 lb/ft3

Maximum Stress: 2.4 tons/ft2
Strain at Maximum Stress: 8%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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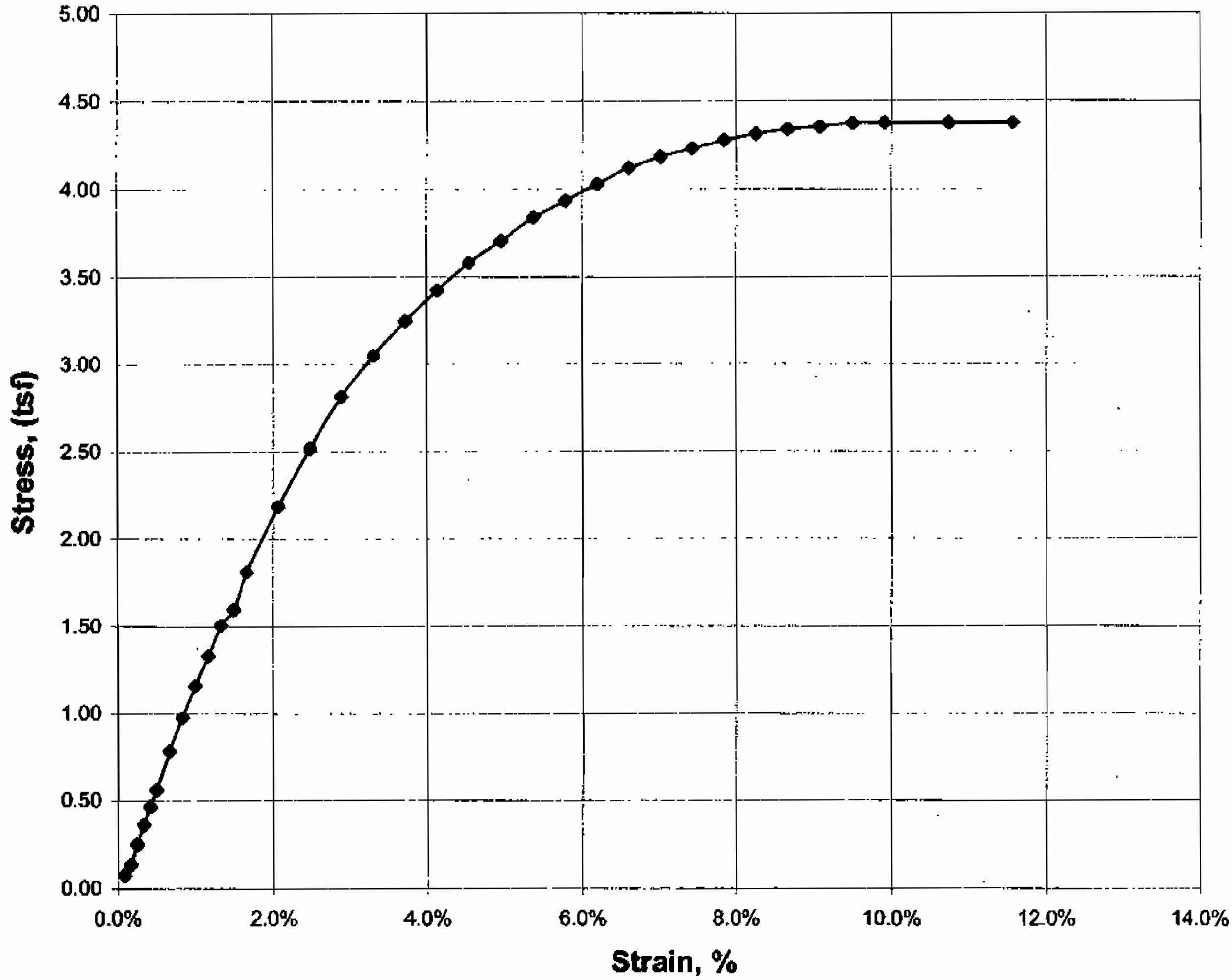


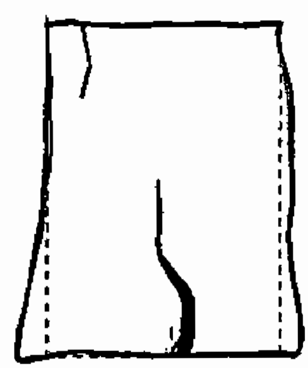
Sample Data

Sample Depth: 150-152ft
Initial Diameter: 2.846 in.
Initial Height: 6.055 In.
Initial Moisture Content: 27%
Liquid Limit: 43
Plastic Limit: 20
Wet Density: 126.3 lb/ft3

Maximum Stress: 4.4 tons/ft2
Strain at Maximum Stress: 10%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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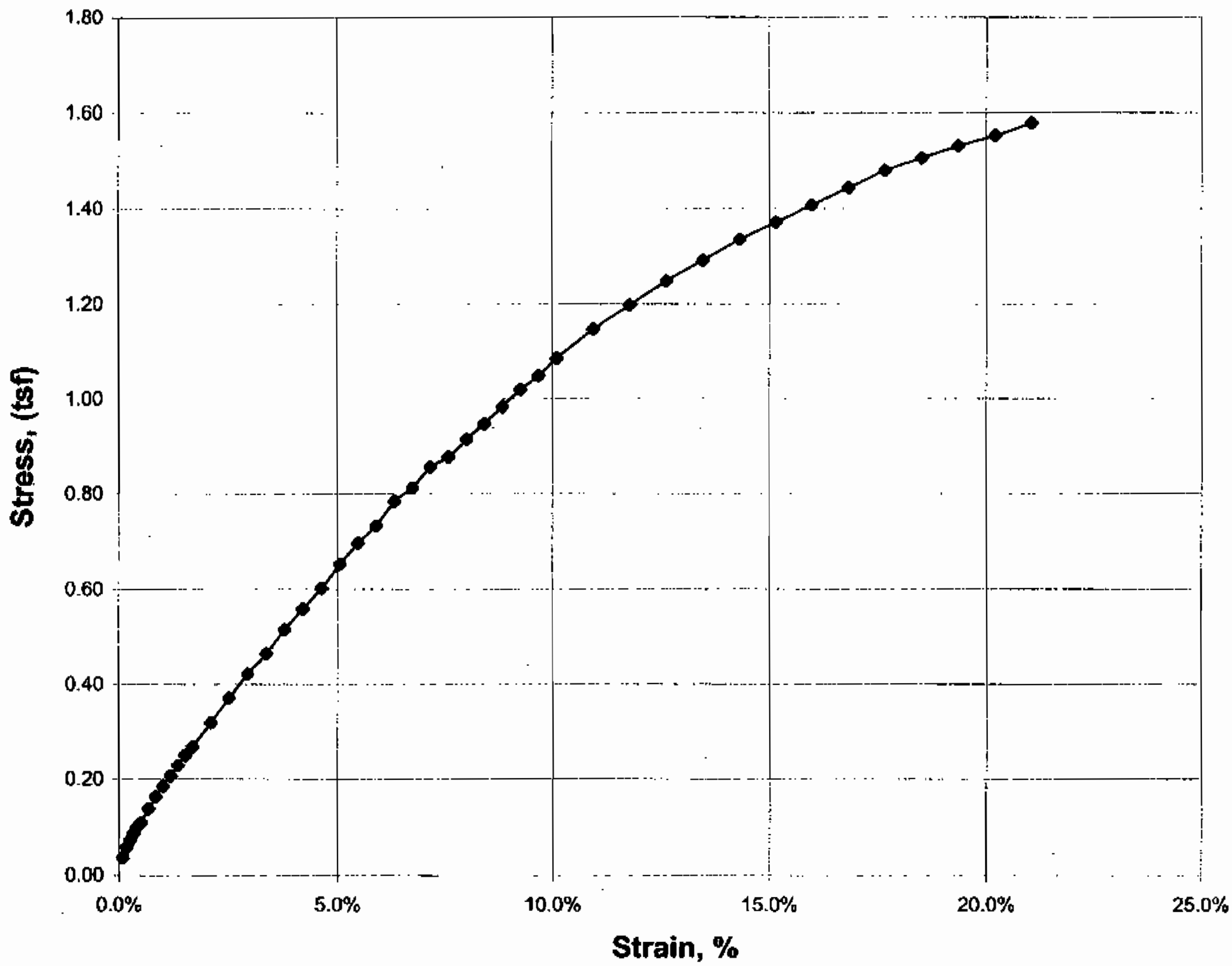


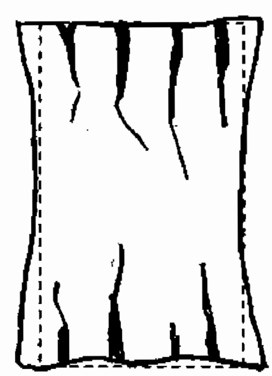
Sample Data

Sample Depth: 172.5-174 ft
Initial Diameter: 2.87 in.
Initial Height: 5.94in.
Initial Moisture Content: 32%
Liquid Limit: 43
Plastic Limit: 20
Wet Density: 128.3 lb/ft3

Maximum Stress: 1.6 tons/ft2
Strain at Maximum Stress: 17.2%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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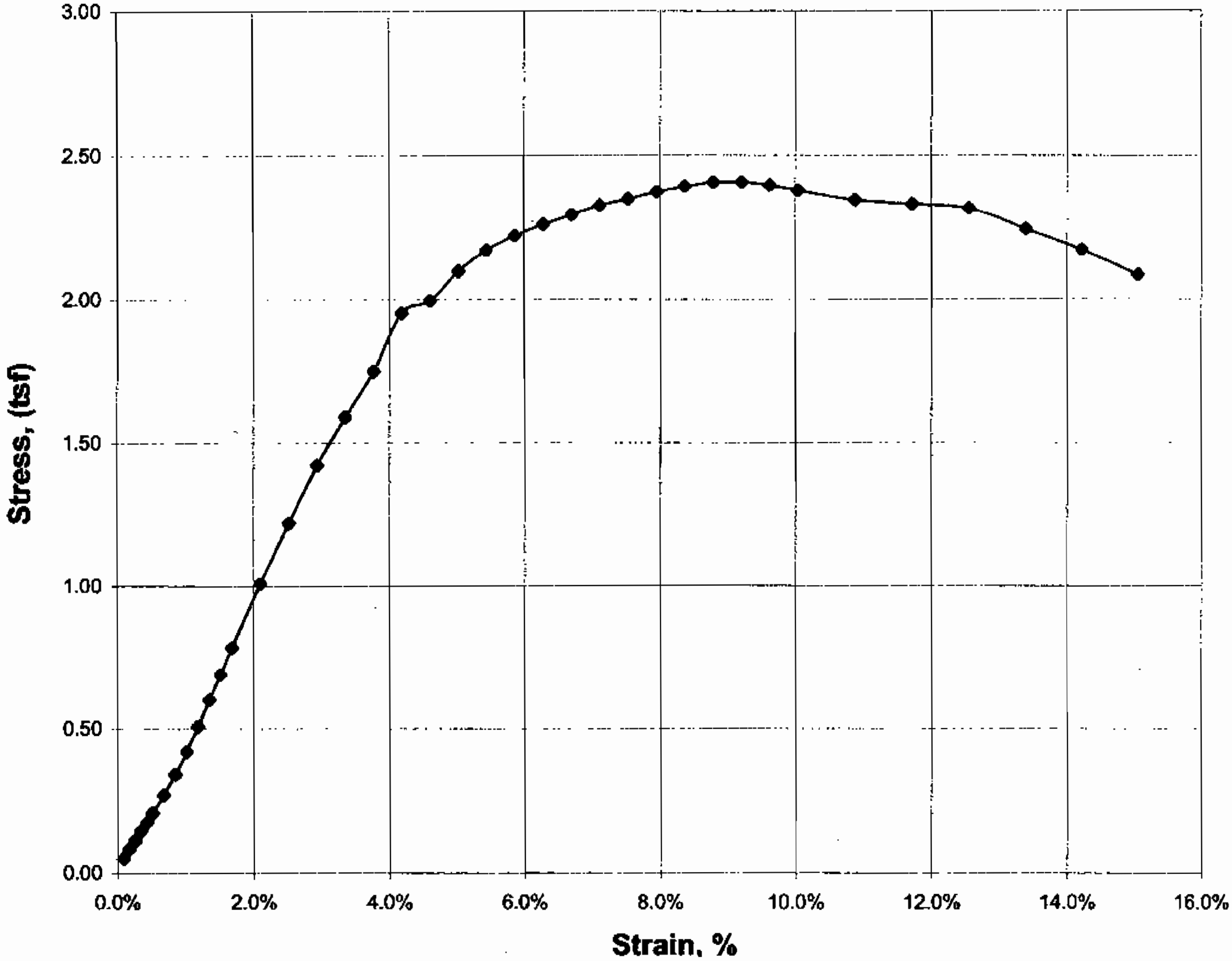


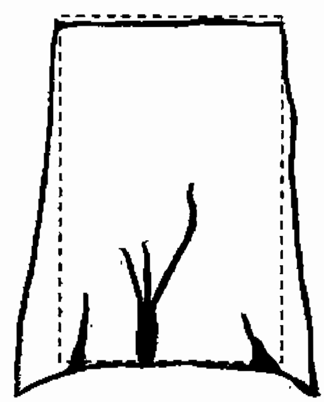
Sample Data

Sample Depth: 188-190ft
Initial Diameter: 2.87 in.
Initial Height: 5.978 In.
Initial Moisture Content: 23%
Liquid Limit: 37
Plastic Limit: 20
Wet Density: 132.8 lb/ft3

Maximum Stress: 2.4 tons/ft2
Strain at Maximum Stress: 9%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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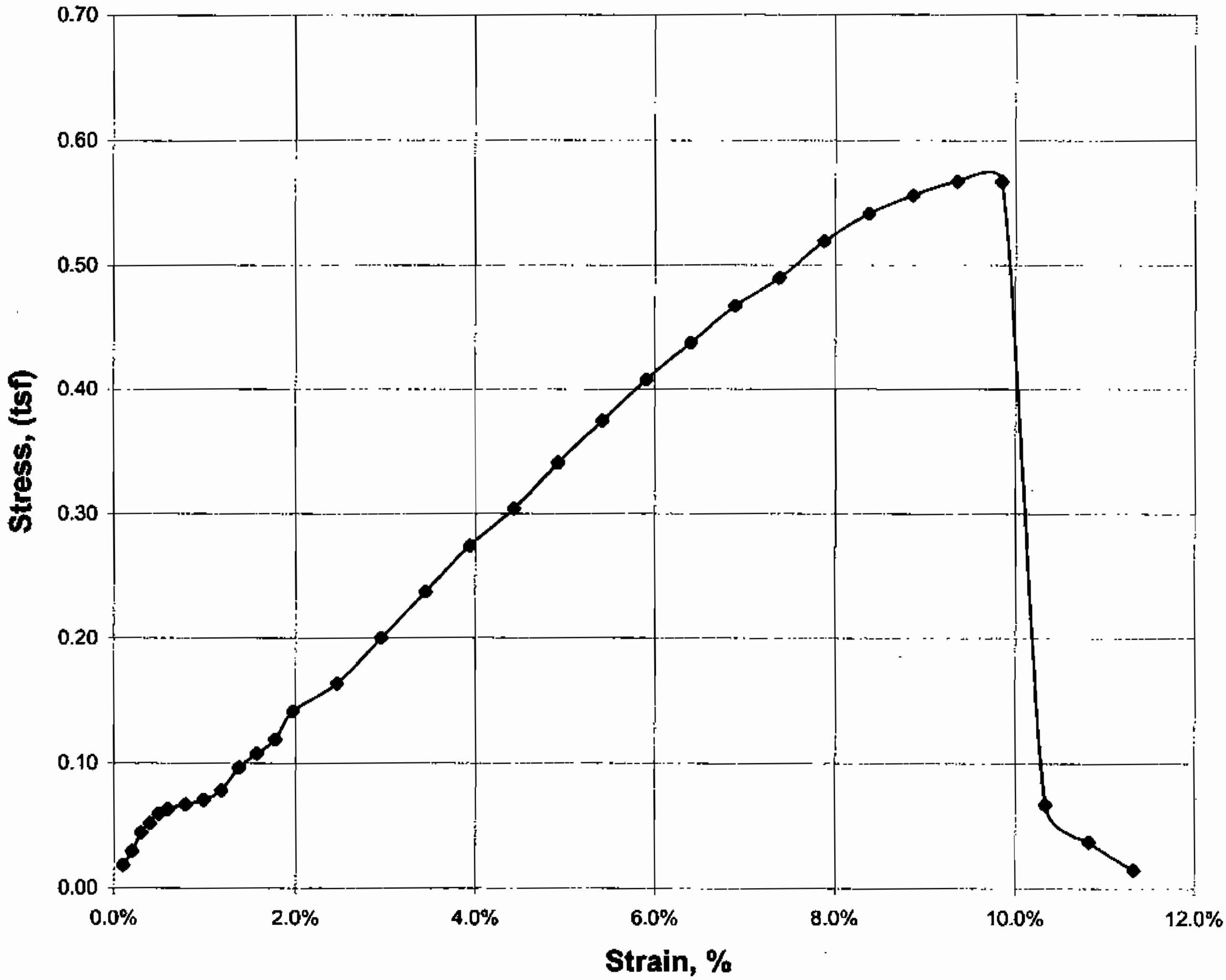


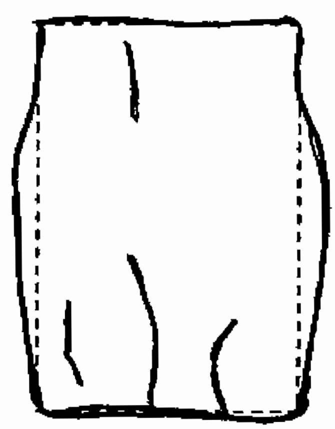
Sample Data

Sample Depth: 235-237ft
Initial Diameter: 2.84 in.
Initial Height: 5.080 In.
Initial Moisture Content: 27%
Liquid Limit: 37
Plastic Limit: 20
Wet Density: 129 lb/ft3

Maximum Stress: .57 tons/ft2
Strain at Maximum Stress: 9.5%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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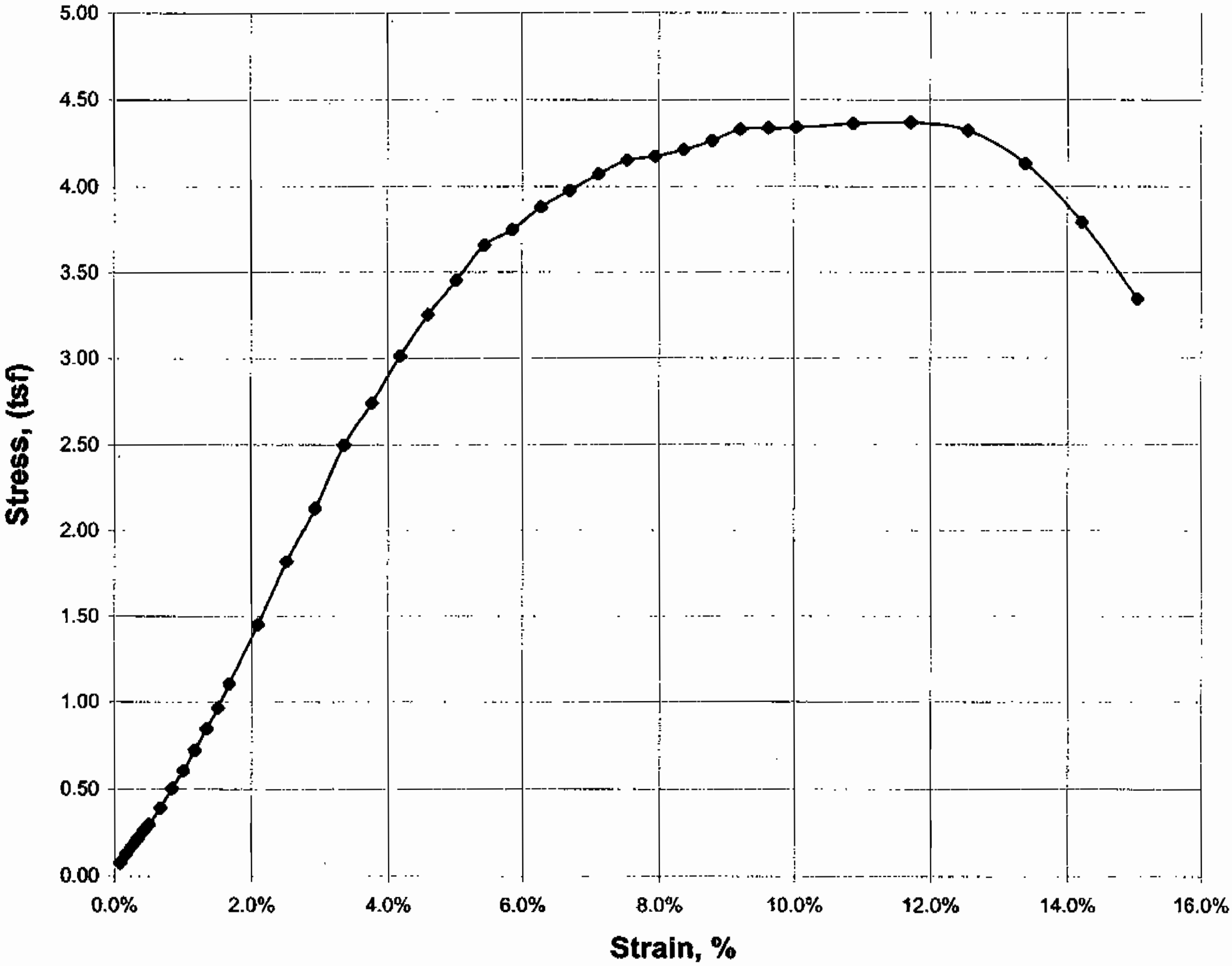
A1 S46



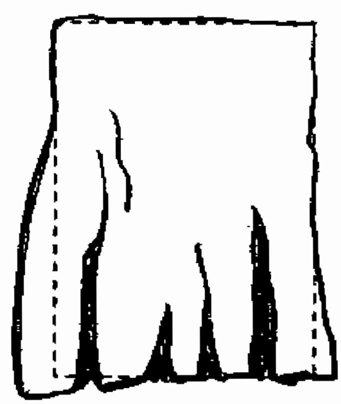
Sample Data

Sample Depth: 265-267ft
Initial Diameter: 2.85 in.
Initial Height: 5.984In.
Initial Moisture Content: 16%
Liquid Limit: 24
Plastic Limit: 14
Wet Density: 139.1 lb/ft3

Maximum Stress: 4.4 tons/ft2
Strain at Maximum Stress: 11%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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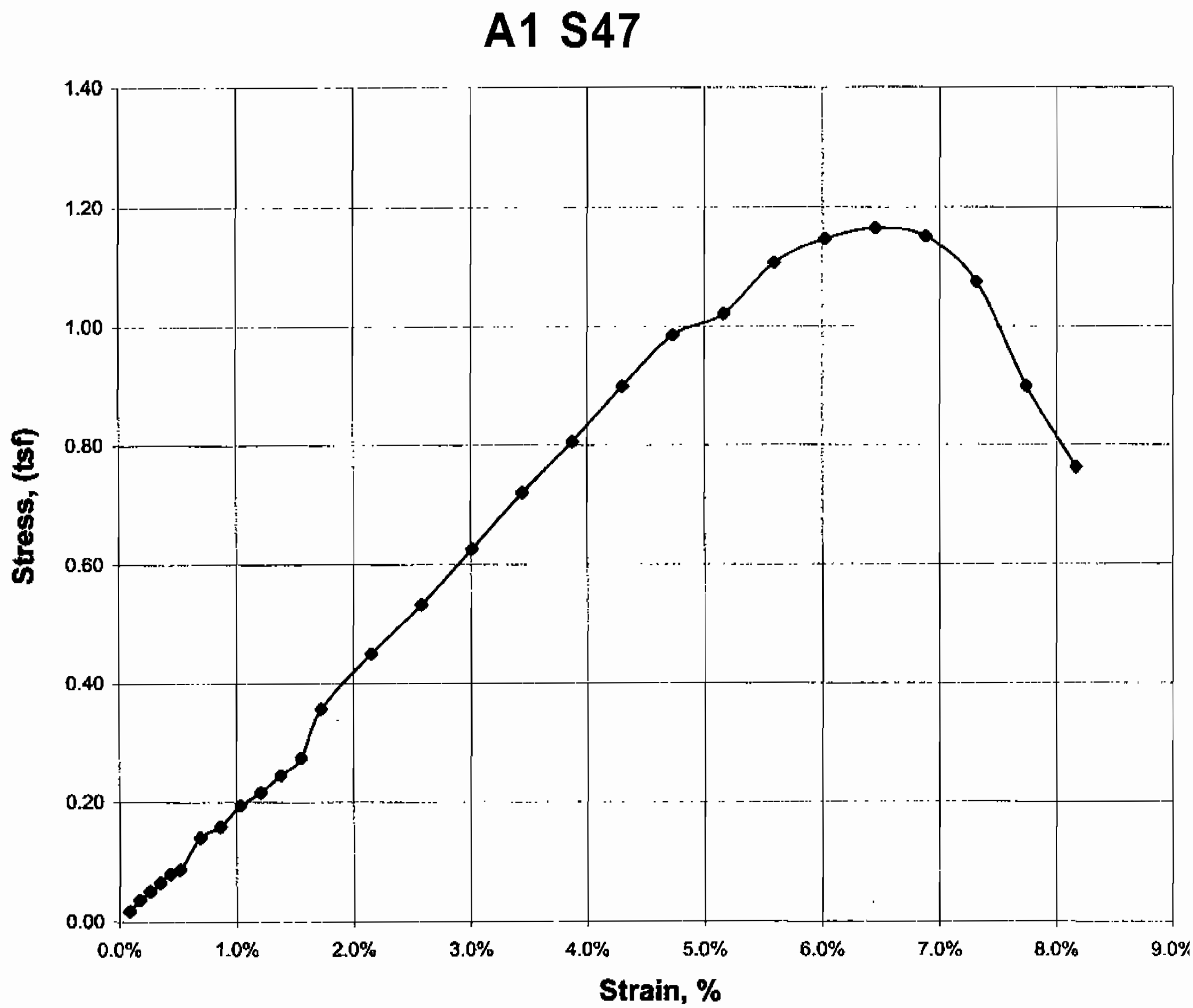


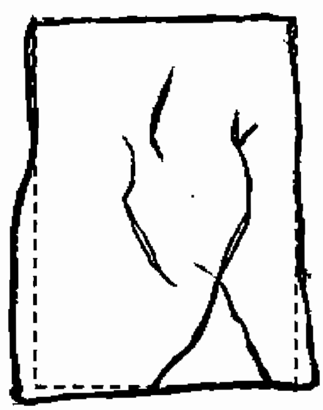
Sample Data

Sample Depth: 275-277ft
Initial Diameter: 2.88 in.
Initial Height: 5.815In.
Initial Moisture Content: 24%
Liquid Limit:
Plastic Limit
Wet Density: 130 lb/ft3

Maximum Stress: 1.2 tons/ft2
Strain at Maximum Stress: 6.4%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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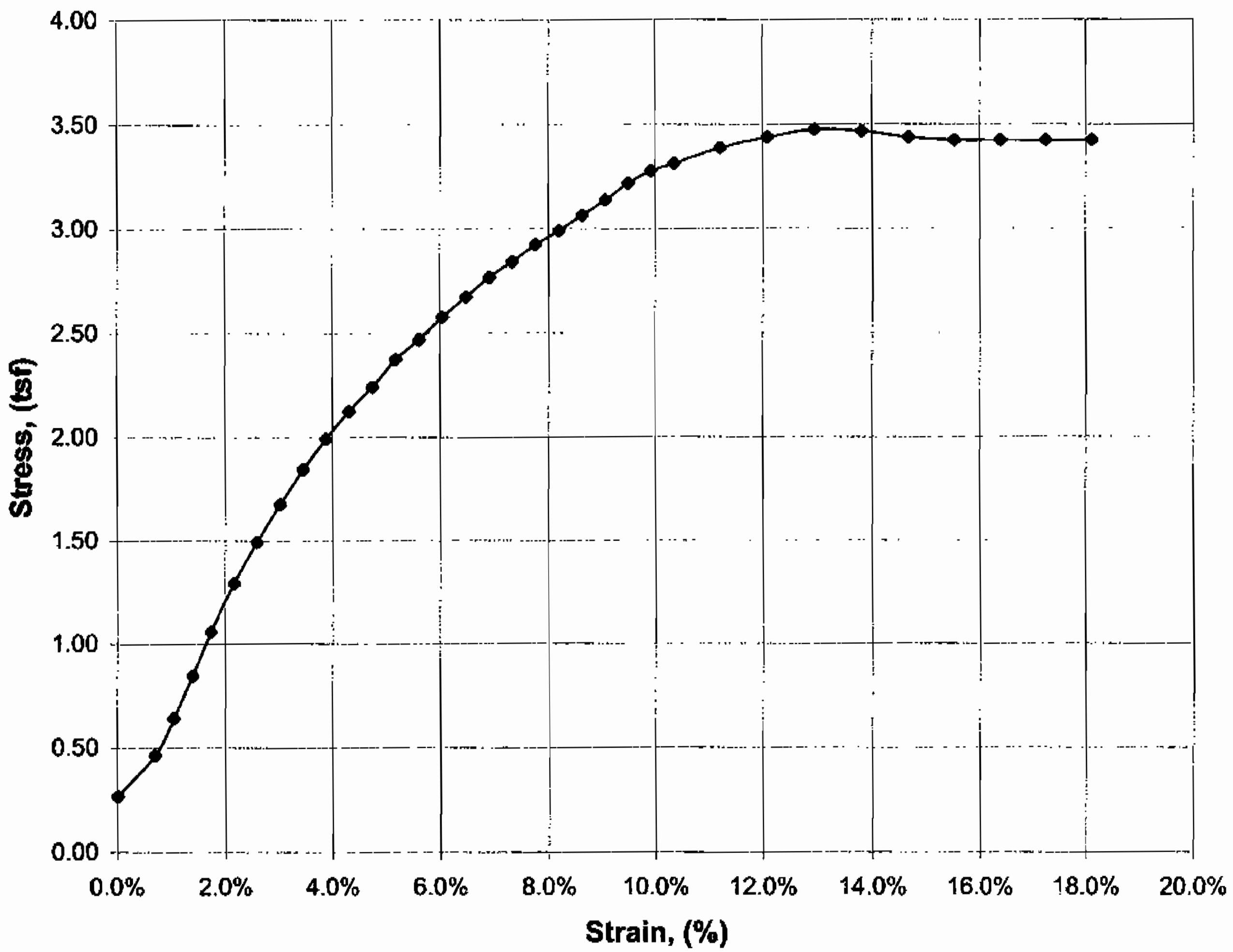


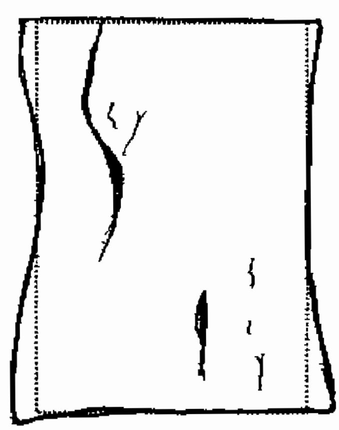
Sample Data

Sample Depth:305-307 ft
Initial Diameter: 2.853in.
Initial Height: 5.796In.
Initial Moisture Content:16 %
Liquid Limit: 30
Plastic Limit: 16
Wet Density: 131.4 lb/ft3

Maximum Stress: 3.5tons/ft2
Strain at Maximum Stress: 13%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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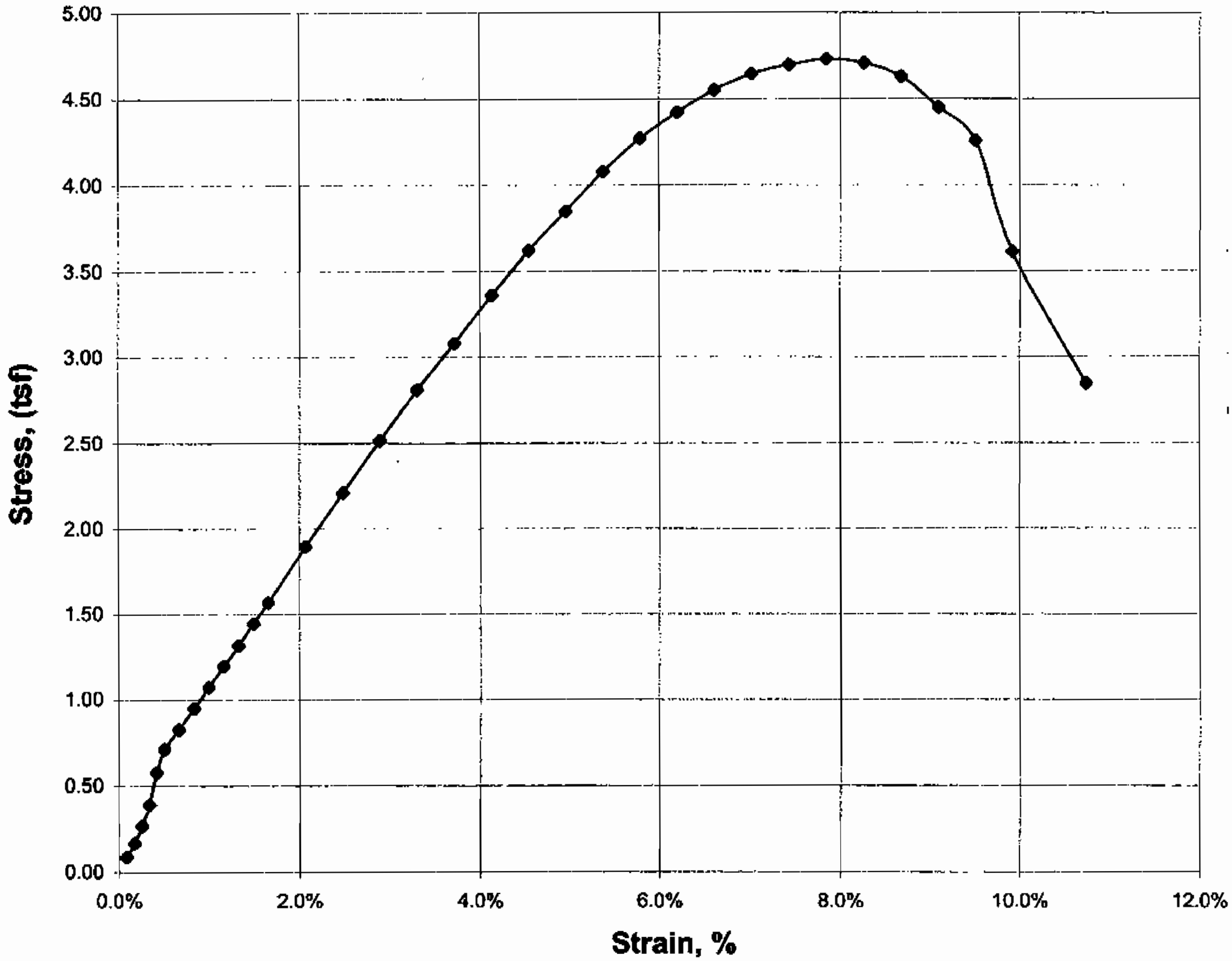


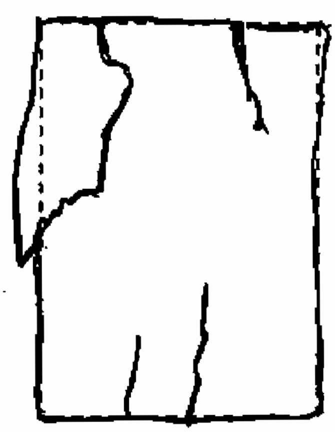
Sample Data

Sample Depth: 335-337ft
Initial Diameter: 2.89 in.
Initial Height: 6.051In.
Initial Moisture Content: 22%
Liquid Limit: 27
Plastic Limit: 16
Wet Density: 136.5 lb/ft3

Maximum Stress: 4.7 tons/ft2
Strain at Maximum Stress: 8%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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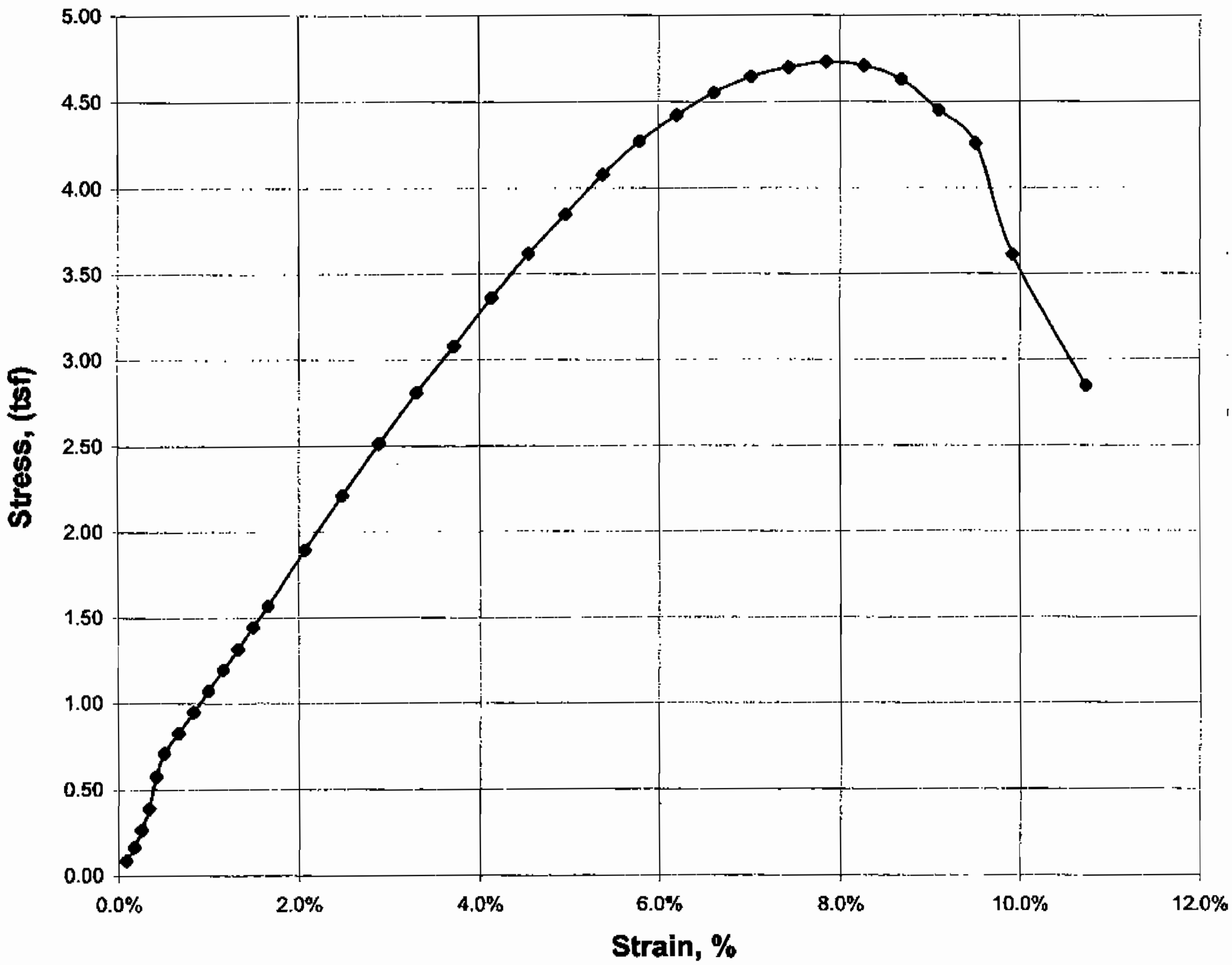


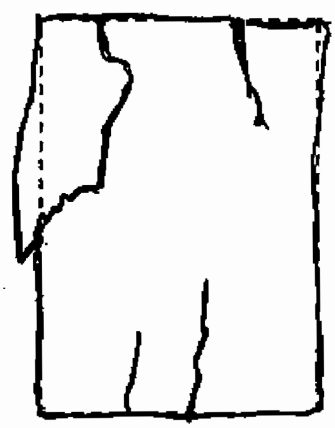
Sample Data

Sample Depth: 335-337ft
Initial Diameter: 2.89 in.
Initial Height: 6.051In.
Initial Moisture Content: 22%
Liquid Limit: 27
Plastic Limit: 16
Wet Density: 136.5 lb/ft3

Maximum Stress: 4.7 tons/ft2
Strain at Maximum Stress: 8%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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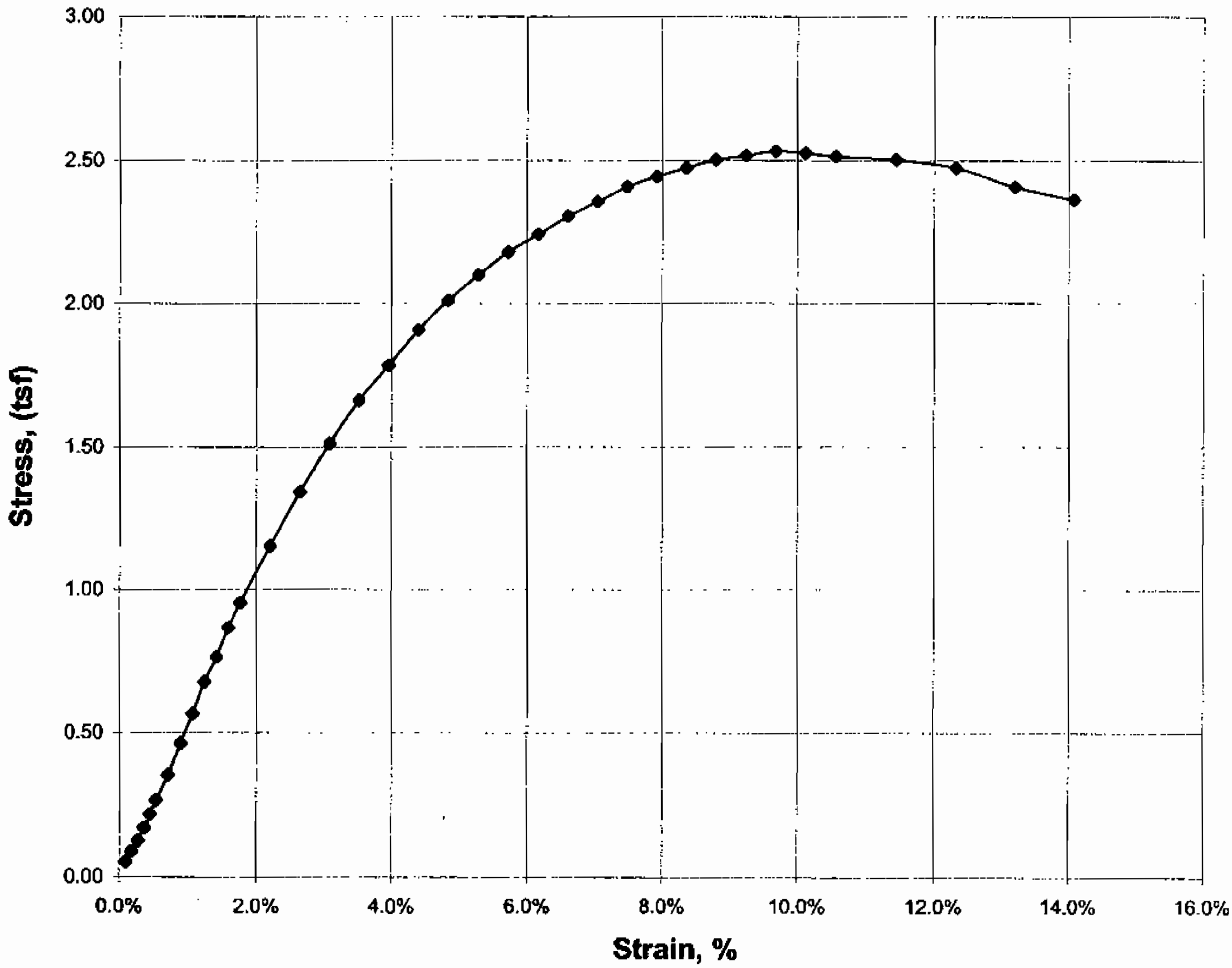


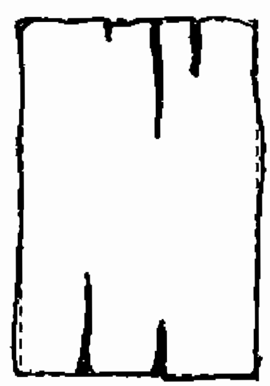
Sample Data

Sample Depth: 176-178ft
Initial Diameter: 2.85 in.
Initial Height: 5.682In.
Initial Moisture Content: 23%
Liquid Limit: 29
Plastic Limit: 17
Wet Density: 125.8 lb/ft3

Maximum Stress: 2.5 tons/ft2
Strain at Maximum Stress: 10%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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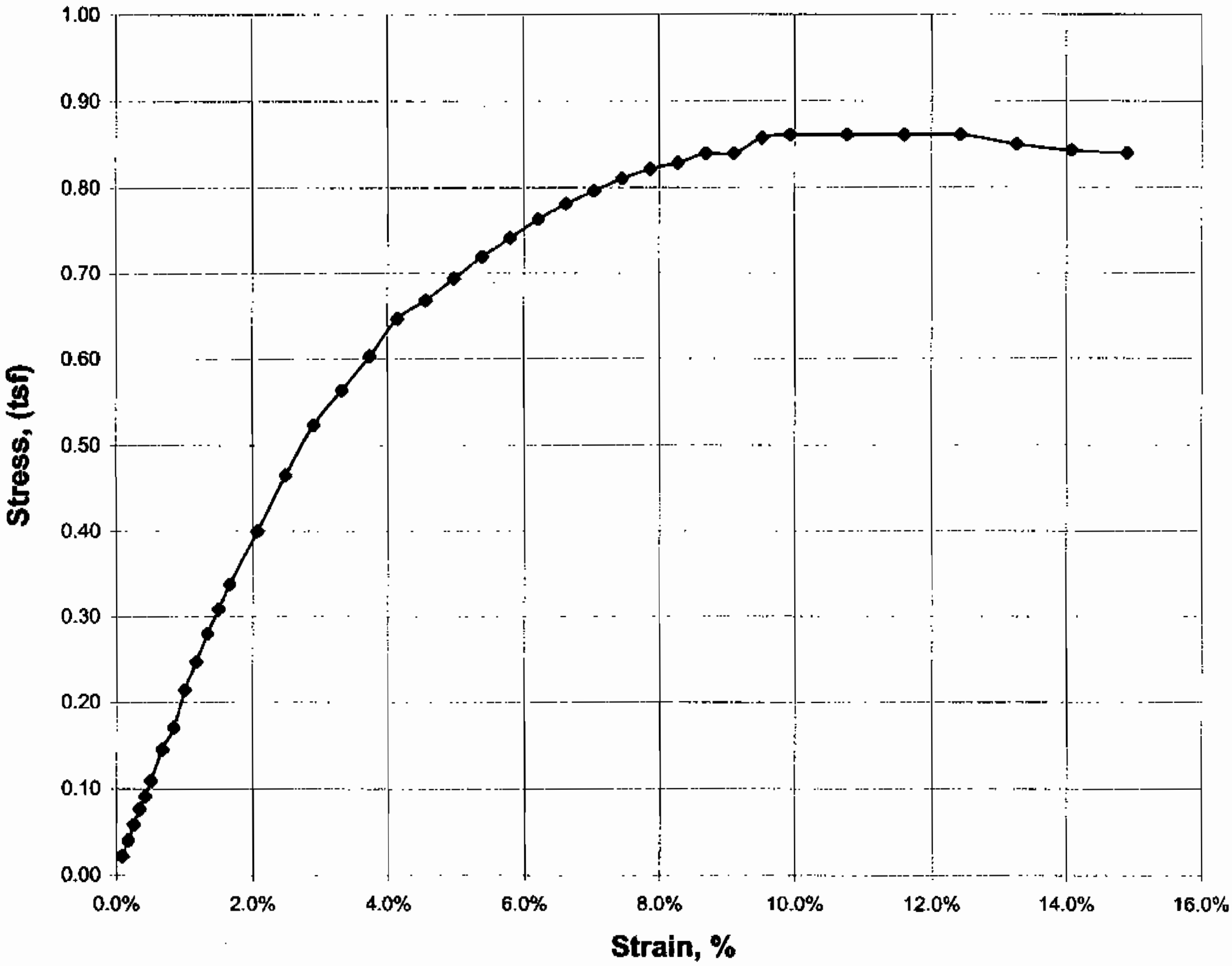


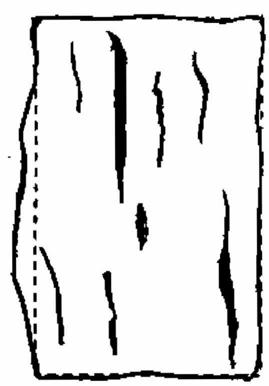
Sample Data

Sample Depth: 81-83ft
Initial Diameter: 2.87 in.
Initial Height: 6.038 In.
Initial Moisture Content: 20%
Liquid Limit: 0
Plastic Limit: 0
Wet Density: 148.5 lb/ft3

Maximum Stress: .84 tons/ft2
Strain at Maximum Stress: 9%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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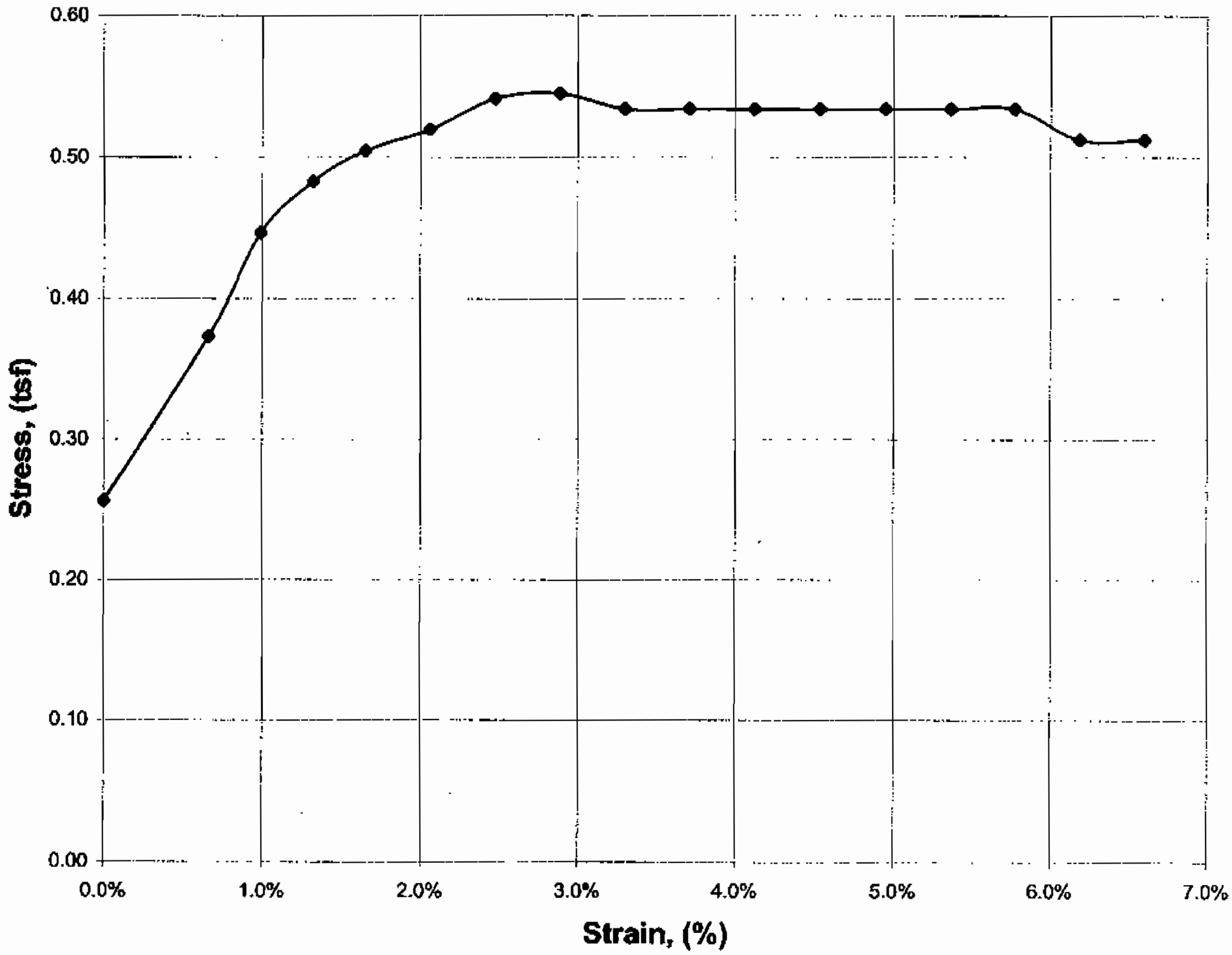
A5 S5



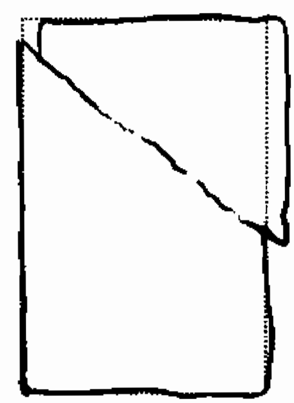
Sample Data

Sample Depth: 26-28ft
Initial Diameter: 2.86 in.
Initial Height: 6.062 In.
Initial Moisture Content: 24 %
Liquid Limit: 38
Plastic Limit: 20
Wet Density: 128.3 lb/ft3

Maximum Stress: 0.54 tons/ft2
Strain at Maximum Stress:
2.5%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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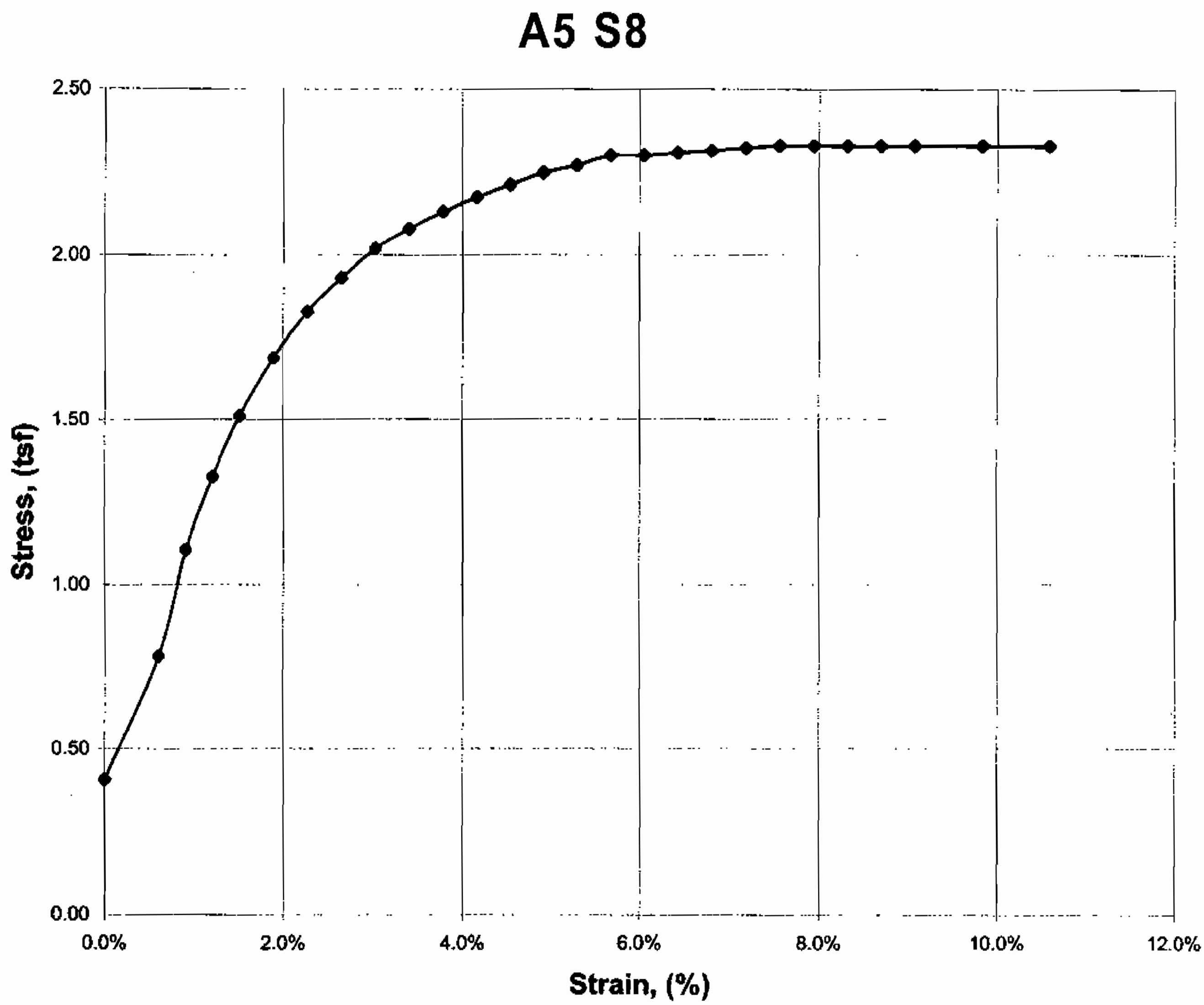


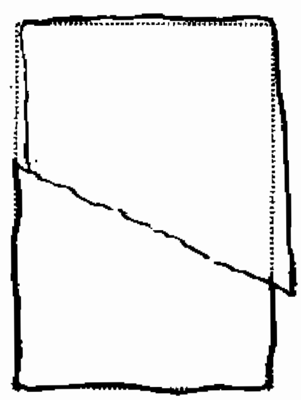
Sample Data

Sample Depth:44-46 ft
Initial Diameter:2.85 in.
Initial Height:6.612 In.
Initial Moisture Content:27 %
Liquid Limit: 43
Plastic Limit: 21
Wet Density:125 lb/ft3

Maximum Stress:2.3 tons/ft2
Strain at Maximum Stress: 6%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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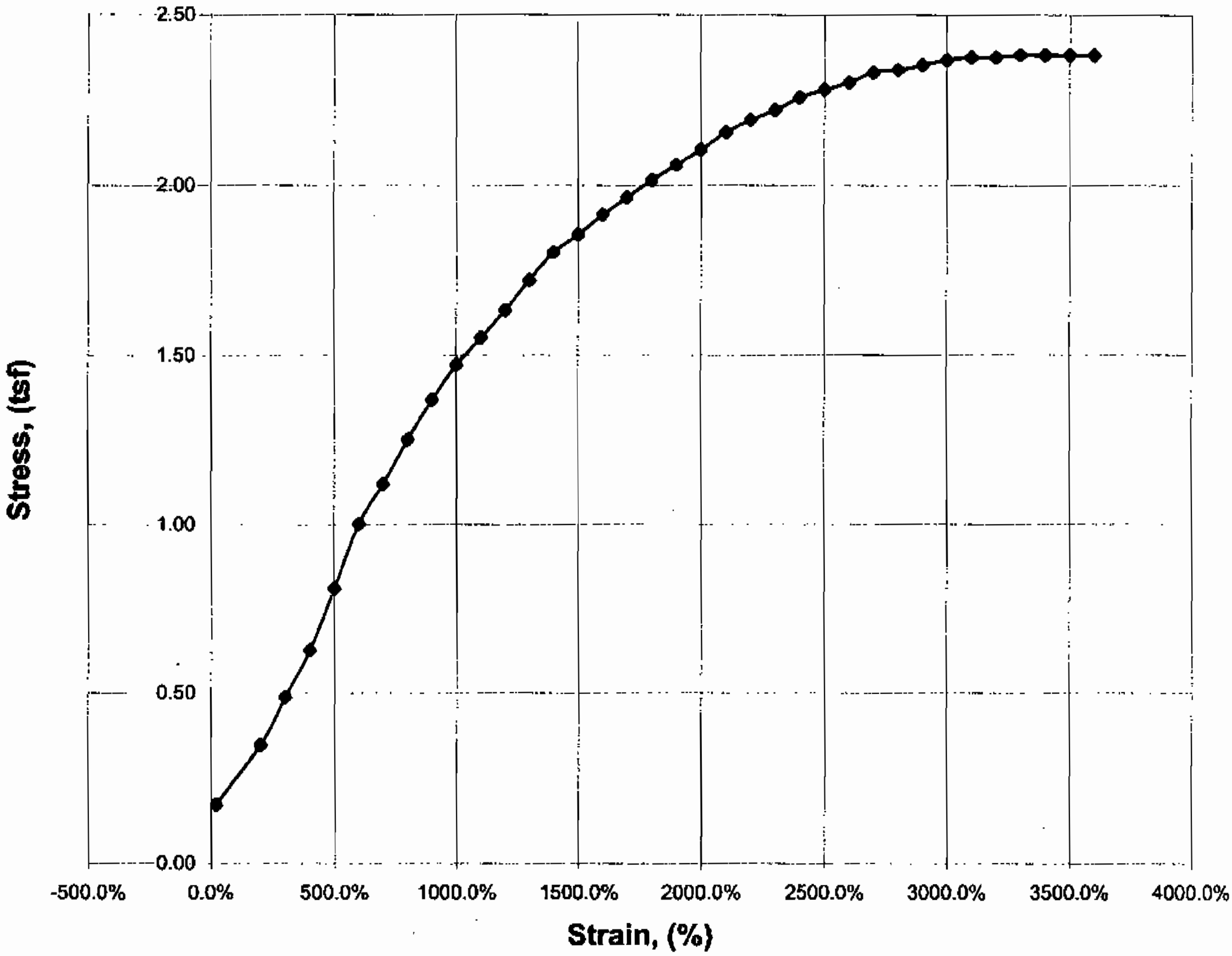


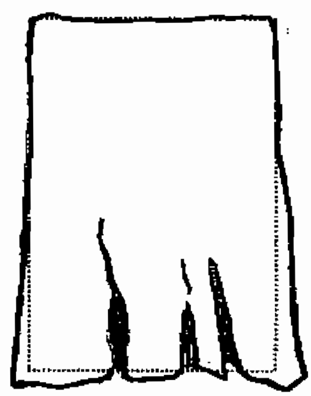
Sample Data

Sample Depth: 60.5-62 ft
Initial Diameter:2.853 in.
Initial Height:6.14 In.
Initial Moisture Content:23 %
Liquid Limit: 41
Plastic Limit: 21
Wet Density:129.9 lb/ft3

Maximum Stress: 2.4 tons/ft2
Strain at Maximum Stress: 13%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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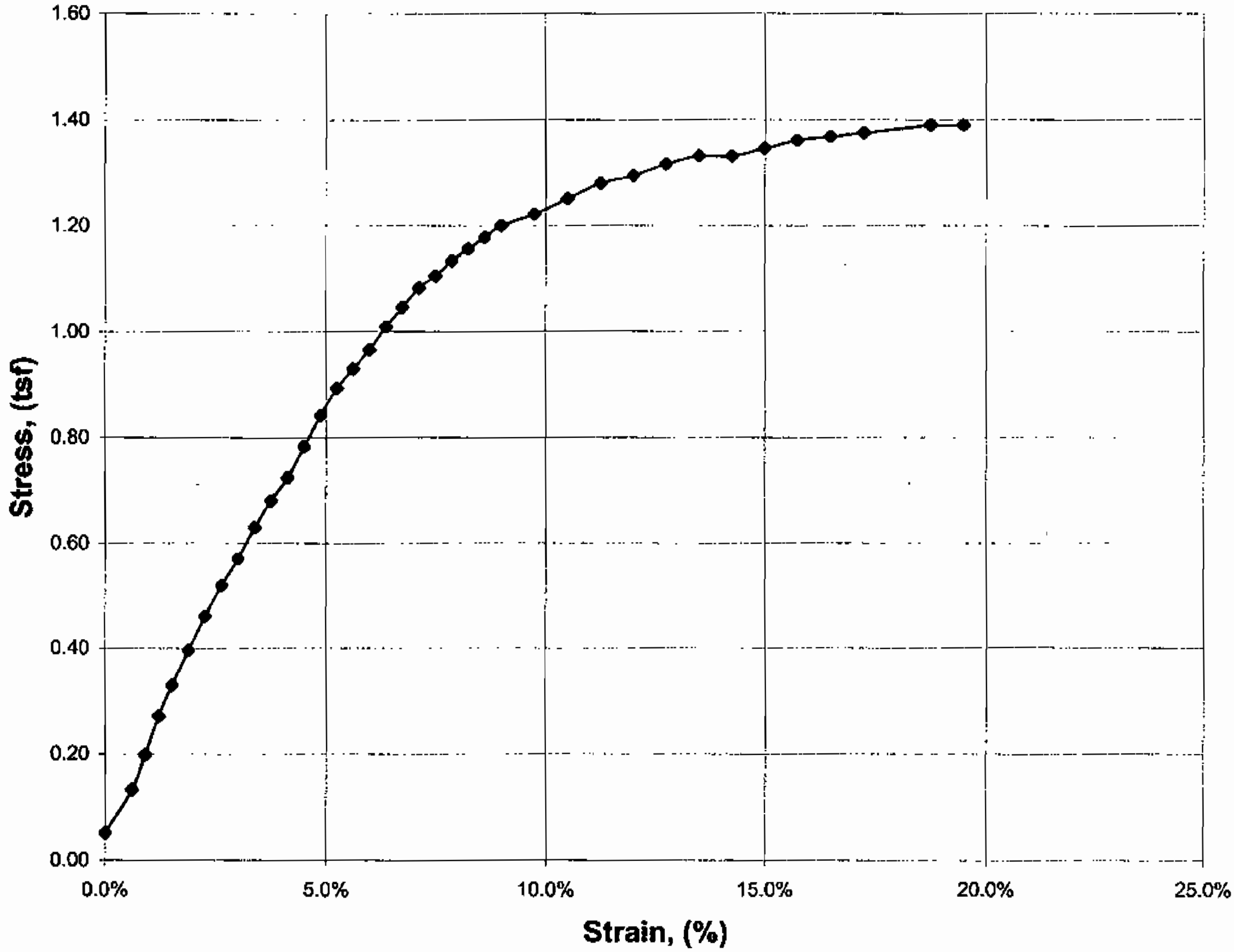
A5 S14a



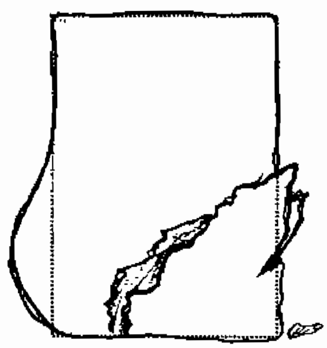
Sample Data

Sample Depth: 69-71ft
Initial Diameter: 2.859in.
Initial Height: 6.672In.
Initial Moisture Content: 24 %
Liquid Limit: 38
Plastic Limit: 20
Wet Density: 130.2lb/ft3

Maximum Stress: 1.4 tons/ft2
Strain at Maximum Stress: 19%

Classification: Gray, silty CLAY (CL)

Failure Sketch



Knik Arm Bridge
Anchorage, Alaska

UNCONFINED COMPRESSION TEST RESULTS

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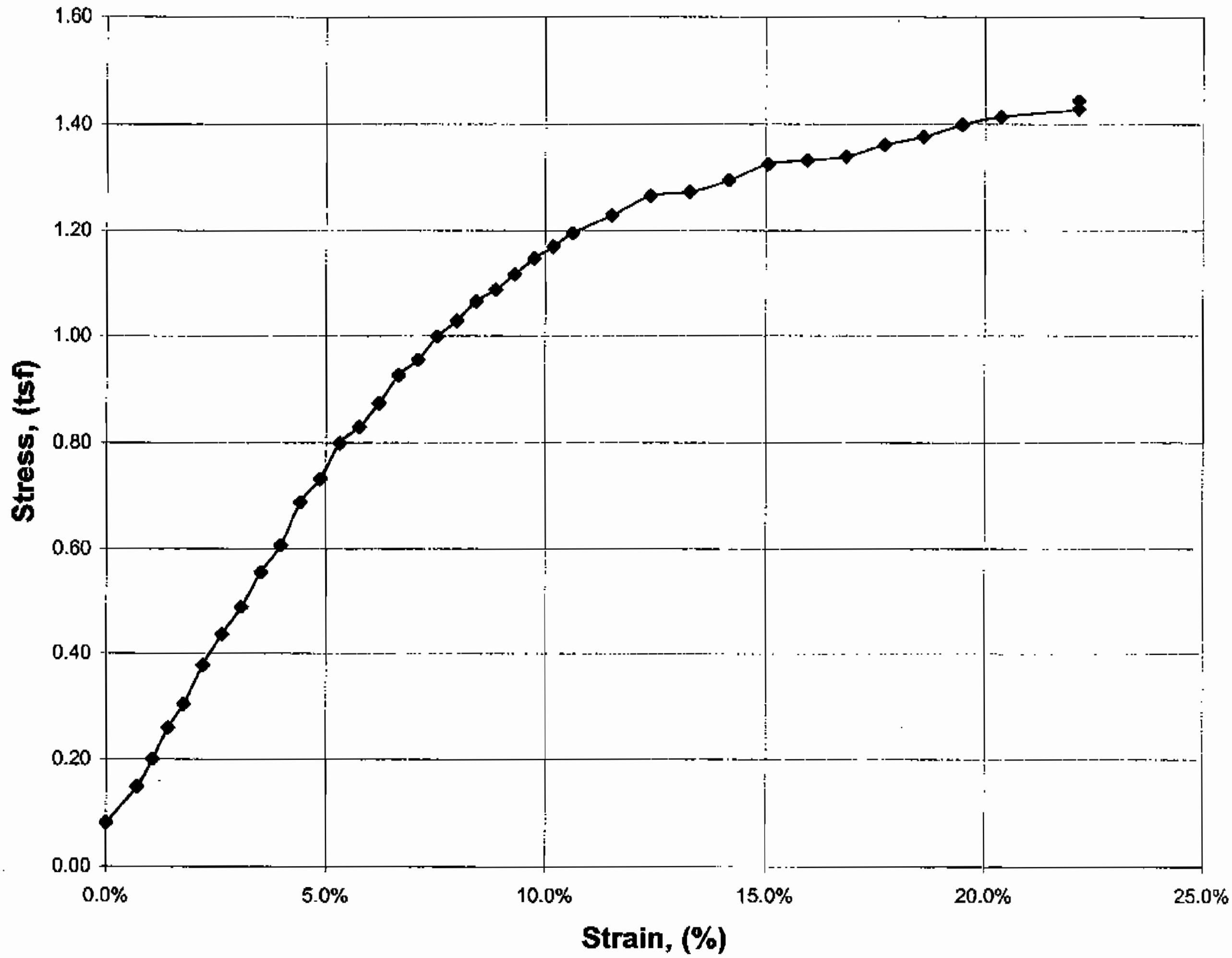
A5 S17



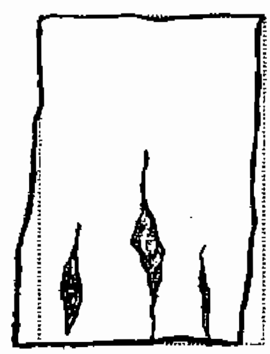
Sample Data

Sample Depth: 83-85ft
Initial Diameter: 2.842in.
Initial Height: 5.645In.
Initial Moisture Content: 25 %
Liquid Limit: 40
Plastic Limit: 20
Wet Density: 128 lb/ft3

Maximum Stress: 1.4 tons/ft2
Strain at Maximum Stress: 20%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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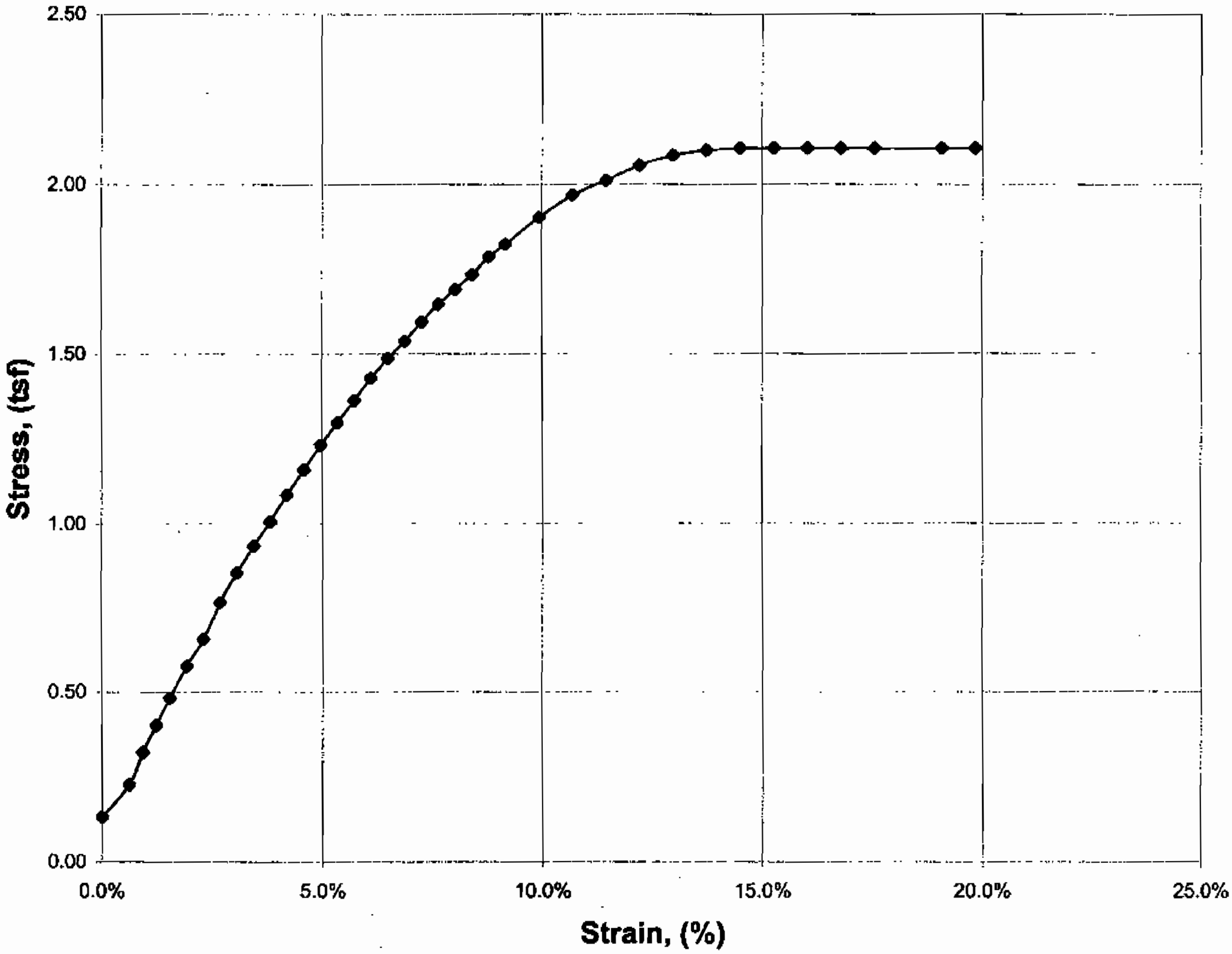
A5 S22



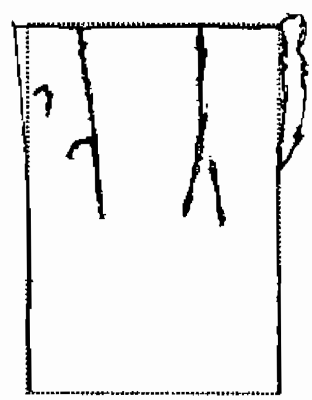
Sample Data

Sample Depth: 109 -111ft
Initial Diameter: 2.864 in.
Initial Height: 6.552 In.
Initial Moisture Content: 23 %
Liquid Limit: 39
Plastic Limit: 20
Wet Density: 131 lb/ft3

Maximum Stress: 2 tons/ft2
Strain at Maximum Stress: 11%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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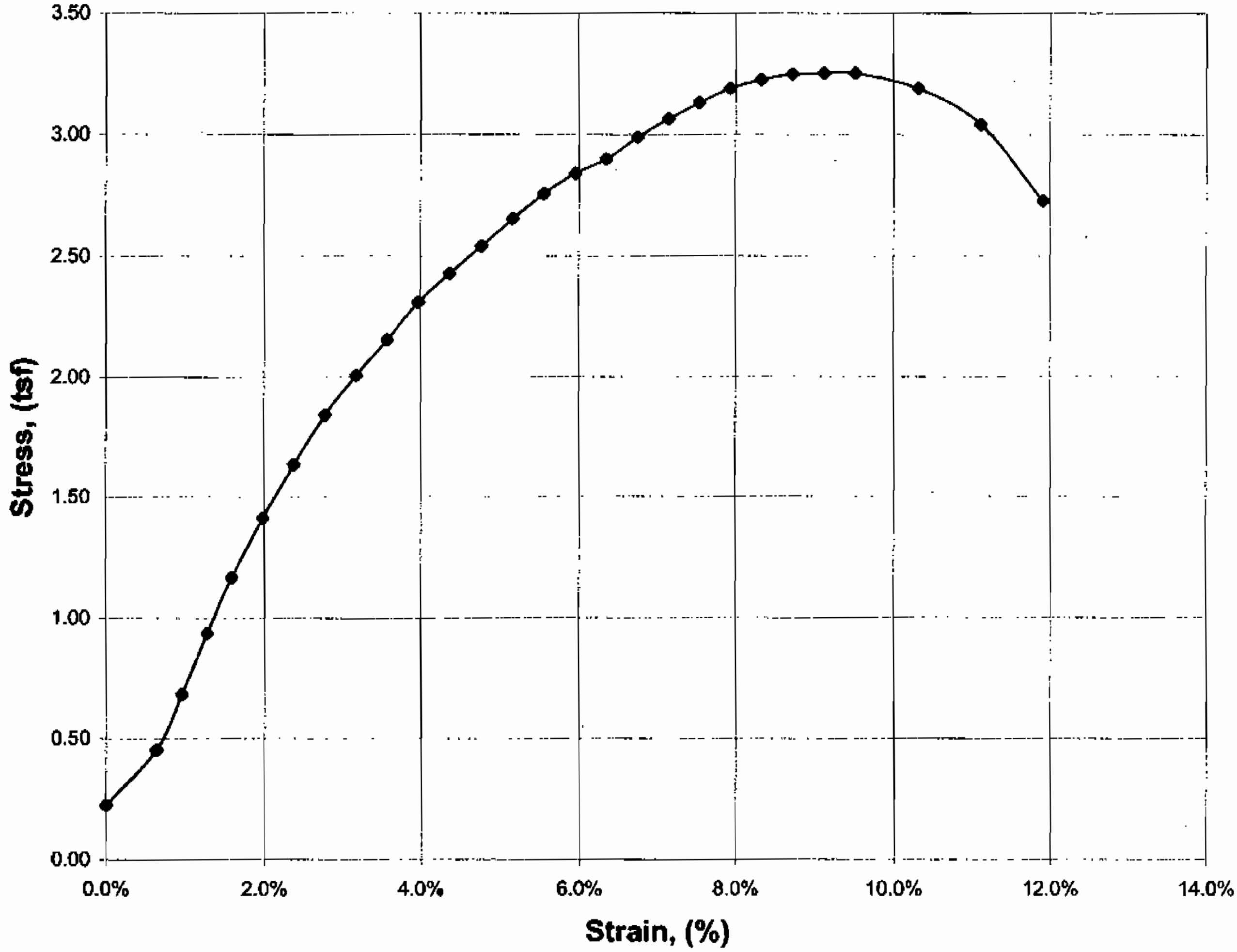
A5 S27



Sample Data

Sample Depth: 151-153ft
Initial Diameter:2.837 in.
Initial Height: 6.302In.
Initial Moisture Content: 20%
Liquid Limit: 39
Plastic Limit: 19
Wet Density: 125lb/ft3

Maximum Stress: 3.2tons/ft2
Strain at Maximum Stress: 8%

Classification: Gray, silty CLAY (CL)

Failure Sketch

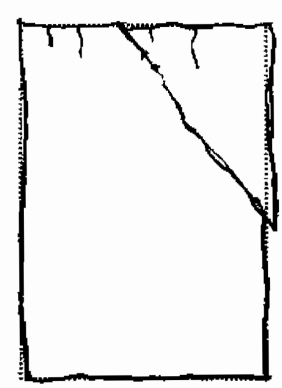


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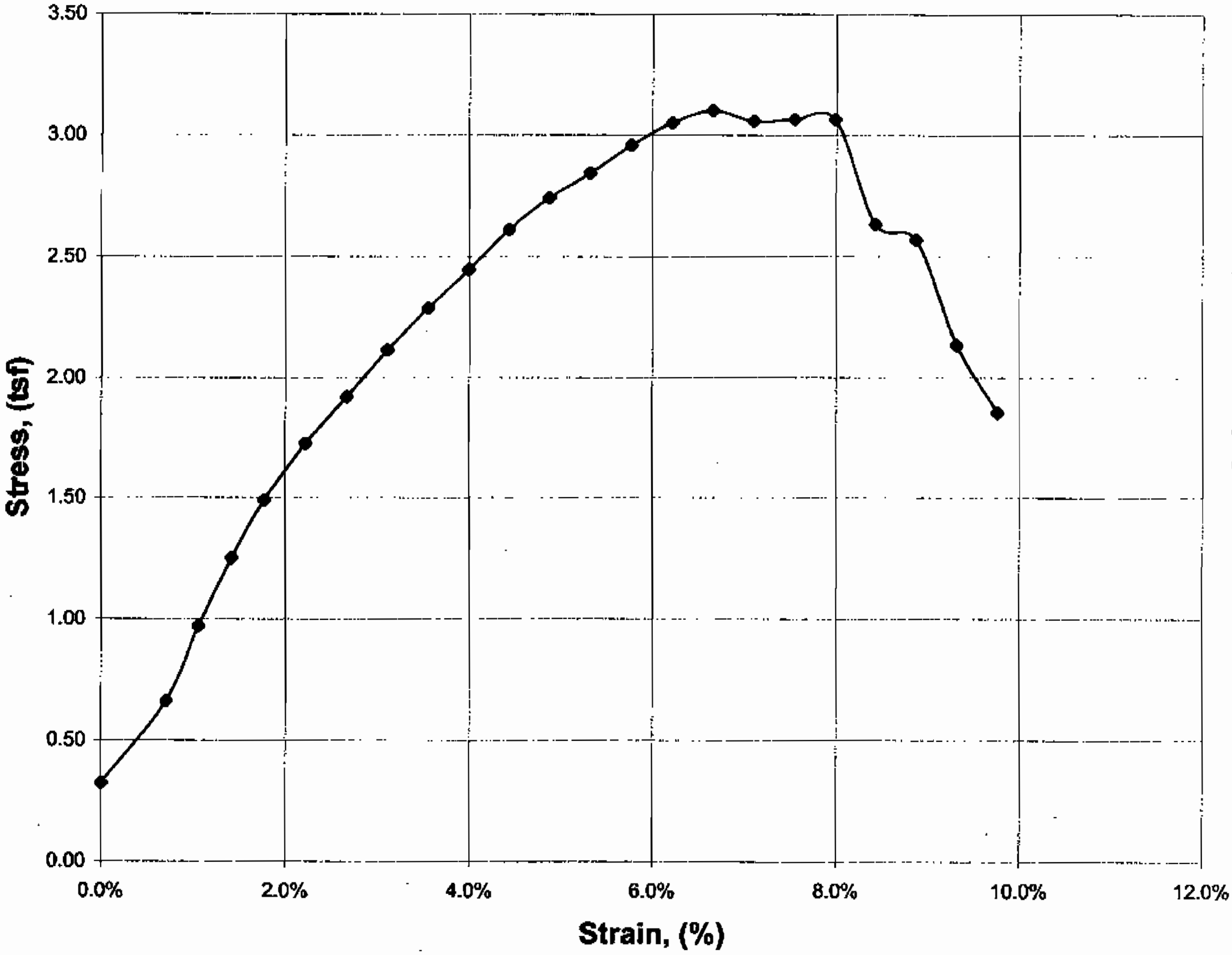
Fig. F-3

A5 S30



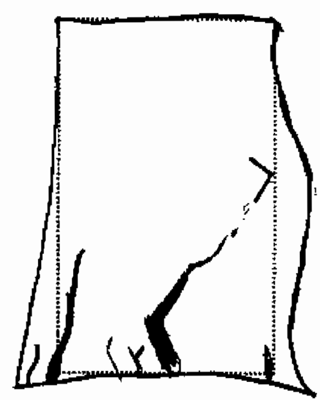
Sample Data

Sample Depth: 183-185ft
Initial Diameter: 2.883in.
Initial Height: 5.635In.
Initial Moisture Content:21%
Liquid Limit: 40
Plastic Limit: 22
Wet Density: 129.2 lb/ft3

Maximum Stress: 3 tons/ft2
Strain at Maximum Stress: 7%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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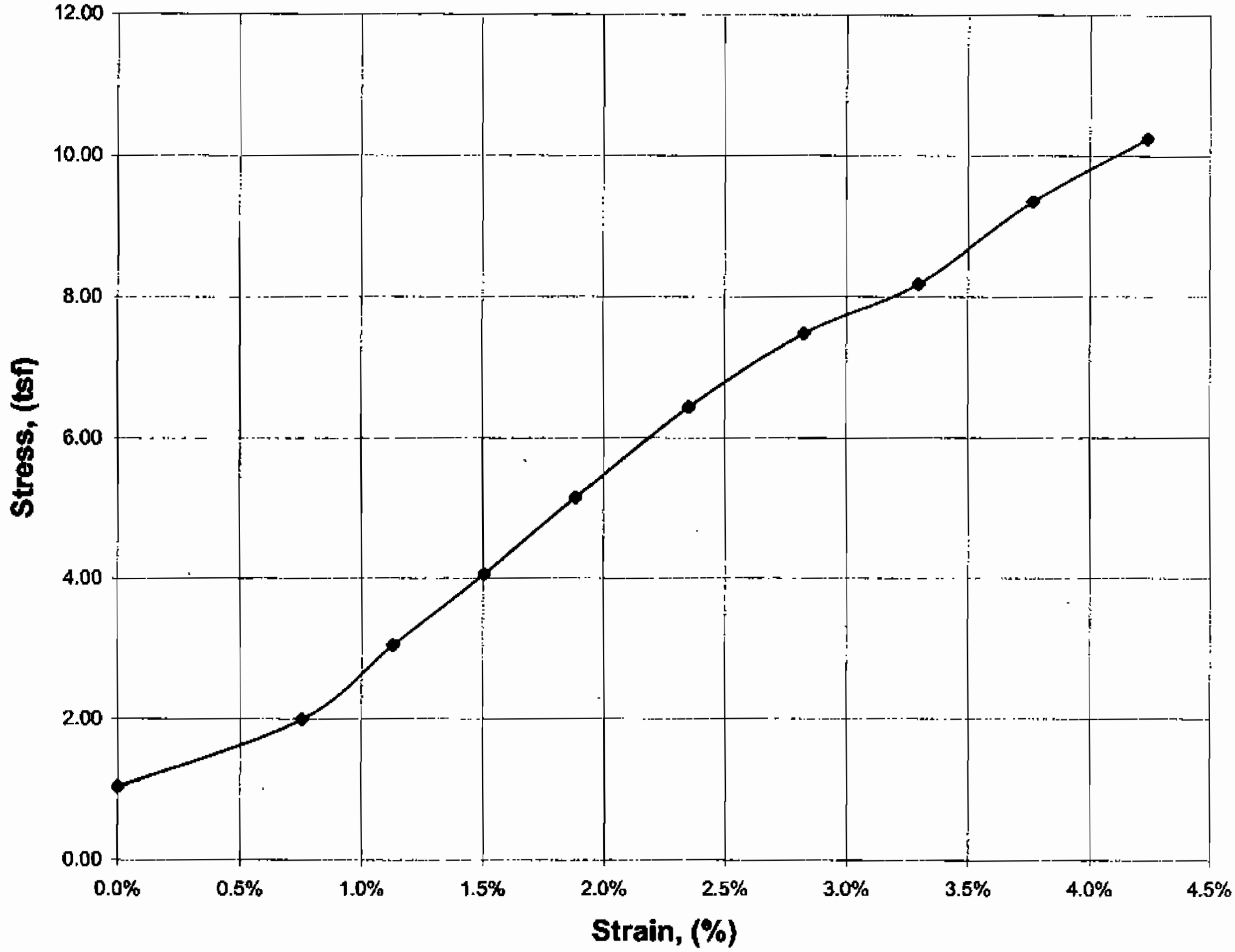
A6 S18



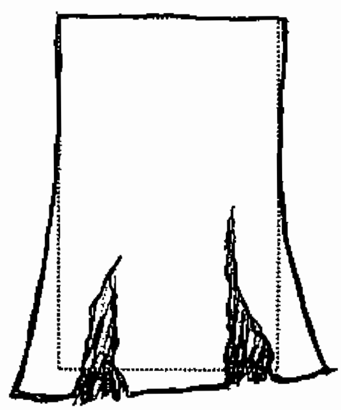
Sample Data

Sample Depth: 97-99ft
Initial Diameter: 2.849in.
Initial Height: 5.31In.
Initial Moisture Content: 22.5%
Liquid Limit: 43
Plastic Limit: 19
Wet Density: 162.2lb/ft3

Maximum Stress: 10tons/ft2
Strain at Maximum Stress: 4%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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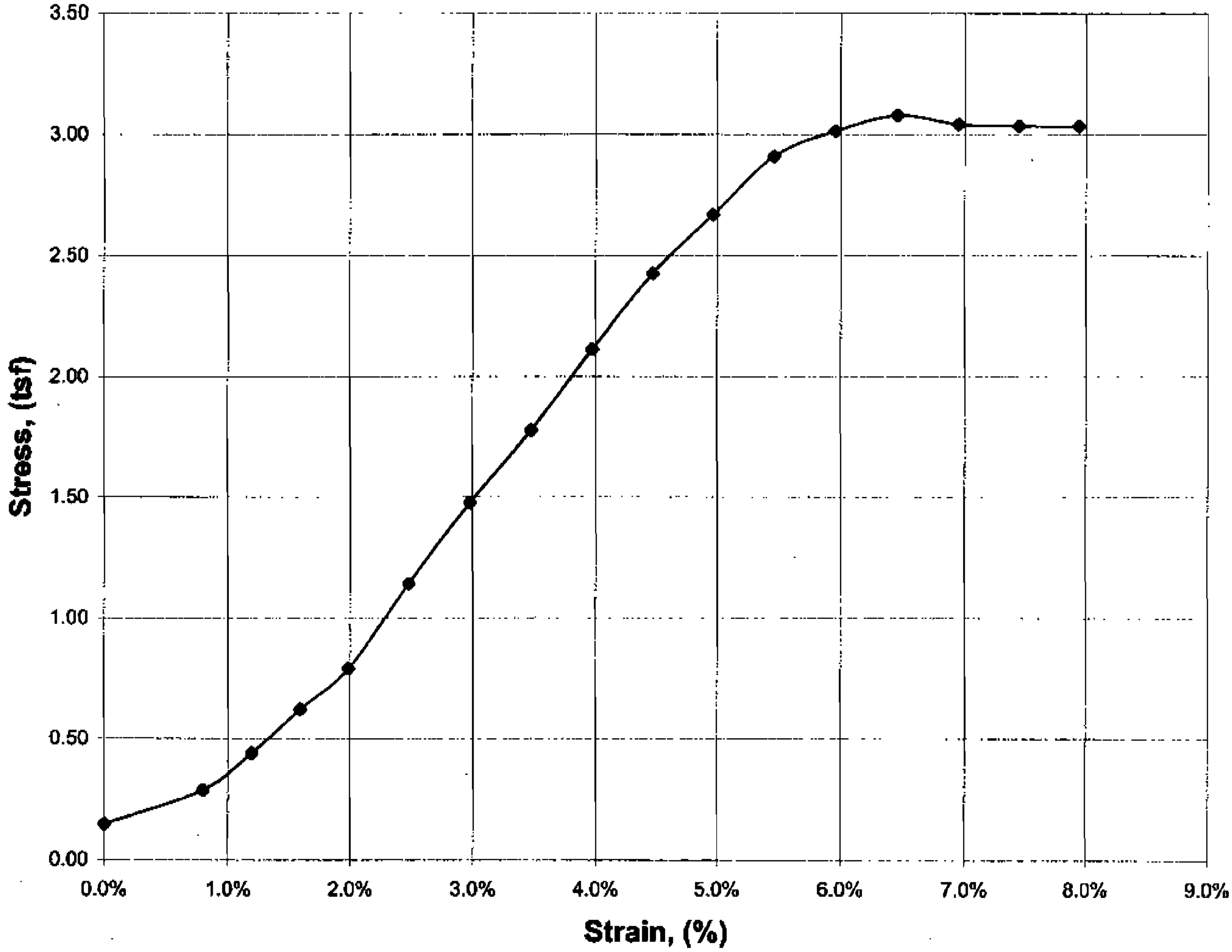
A6 S31



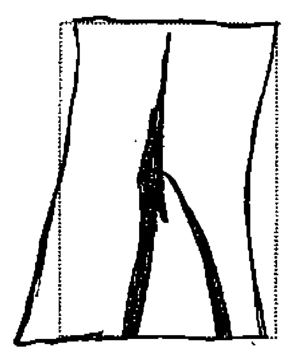
Sample Data

Sample Depth: 175-177ft
Initial Diameter: 2.86in.
Initial Height: 5.037In.
Initial Moisture Content:16.6 %
Liquid Limit: 38
Plastic Limit: 23
Wet Density: 128 lb/ft3

Maximum Stress: 3tons/ft2
Strain at Maximum Stress: 6%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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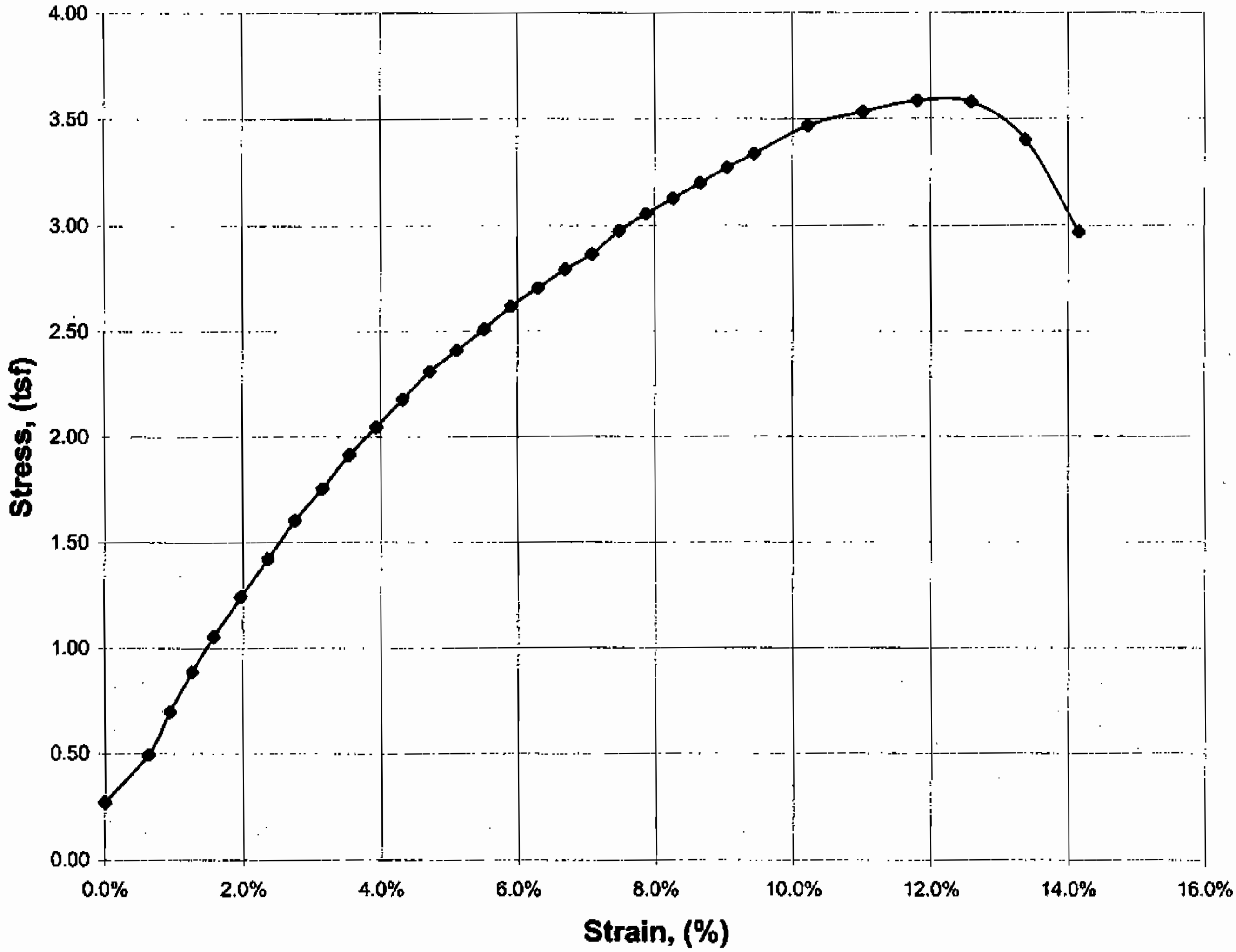
A10 S23



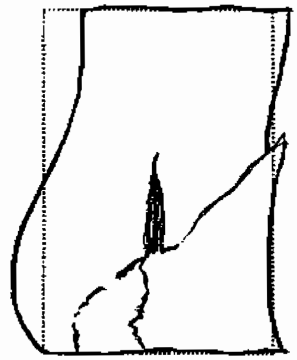
Sample Data

Sample Depth: 175.5-177ft
Initial Diameter: 2.872in.
Initial Height: 6.355 In.
Initial Moisture Content: 16%
Liquid Limit: 22
Plastic Limit: 17
Wet Density: 135lb/ft3

Maximum Stress: 3.5tons/ft2
Strain at Maximum Stress:11 %

Classification: Gray, silty CLAY (CL)

Failure Sketch



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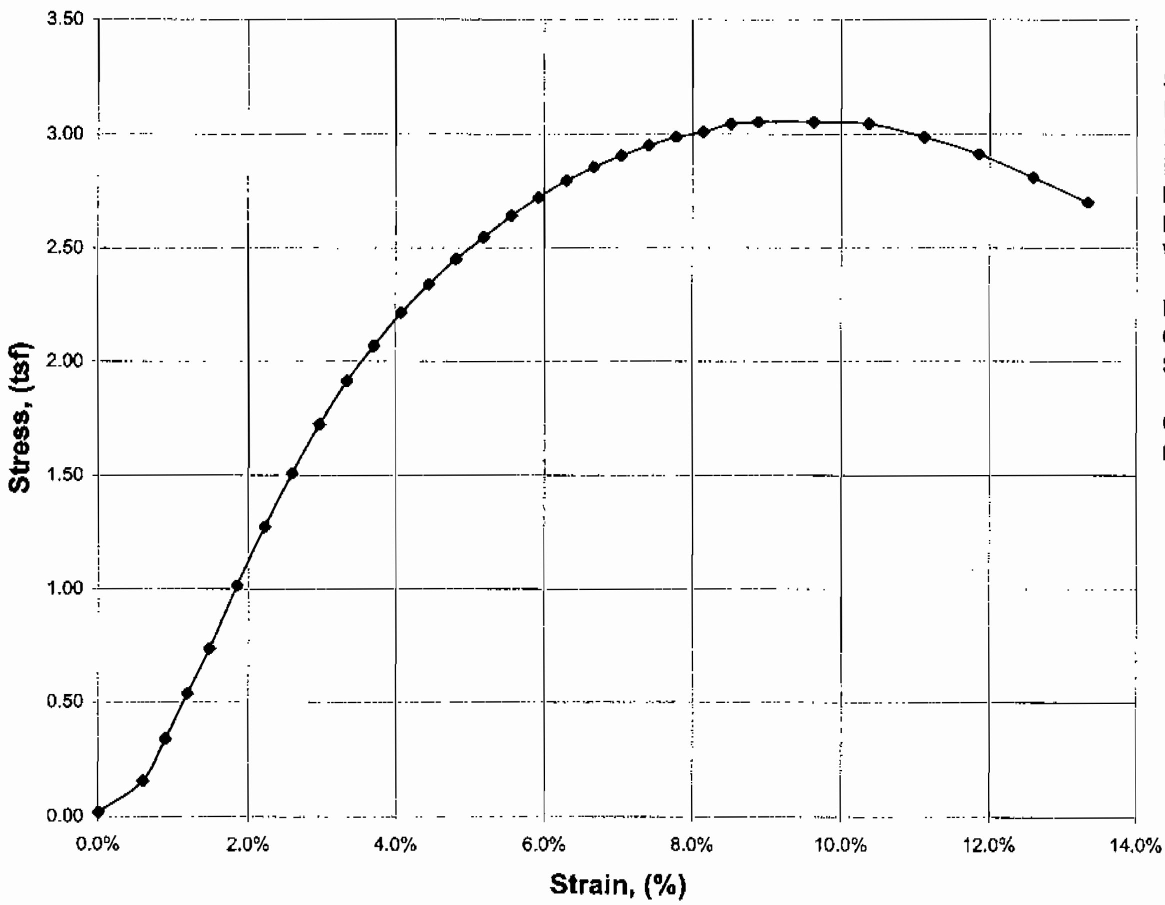
A1 S9



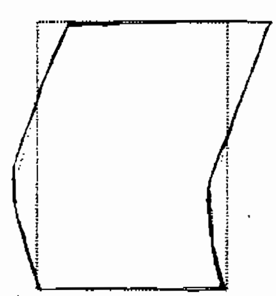
Sample Data

Sample Depth: 35-37ft
Initial Diameter: 2.851in.
Initial Height: 6.767In.
Initial Moisture Content:24 %
Liquid Limit: 38
Plastic Limit: 20
Wet Density: 130.4lb/ft3

Maximum Stress: 3tons/ft2
Confining Pressure: 3.3 tons/ft2
Strain at Maximum Stress: 9%

Classification: Gray, silty CLAY (CL)

Failure Sketch



% *Triaxial test results

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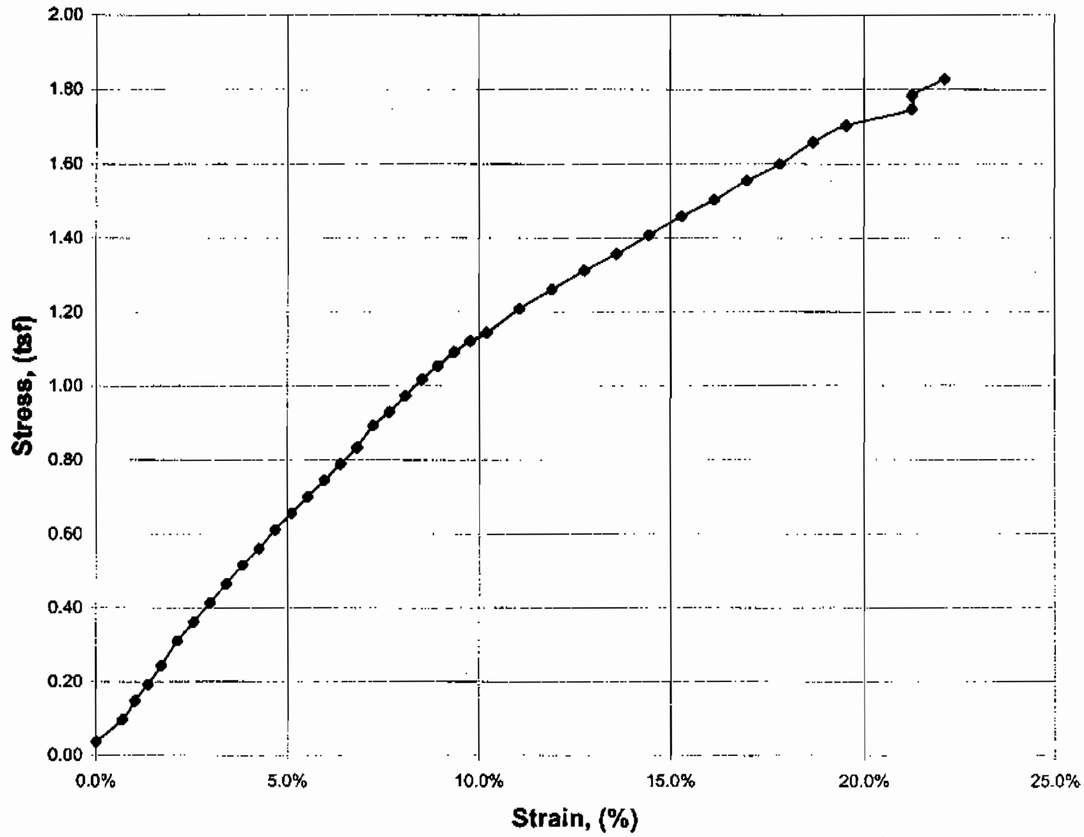
A1 S19



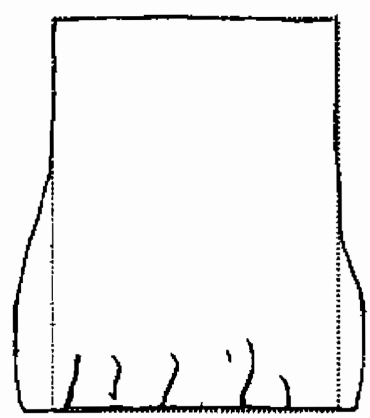
Sample Data

Sample Depth: 85-87ft
Initial Diameter: 2.848 in.
Initial Height: 5.885 In.
Initial Moisture Content: 29%
Liquid Limit: 4
Plastic Limit: 23
Wet Density: 127.2 lb/ft3

Maximum Stress: 1.8 tons/ft2
Confing Pressure: 1.8 tons/ft2
Strain at Maximum Stress: 22%

Classification: Gray, silty CLAY (CL)

Failure Sketch



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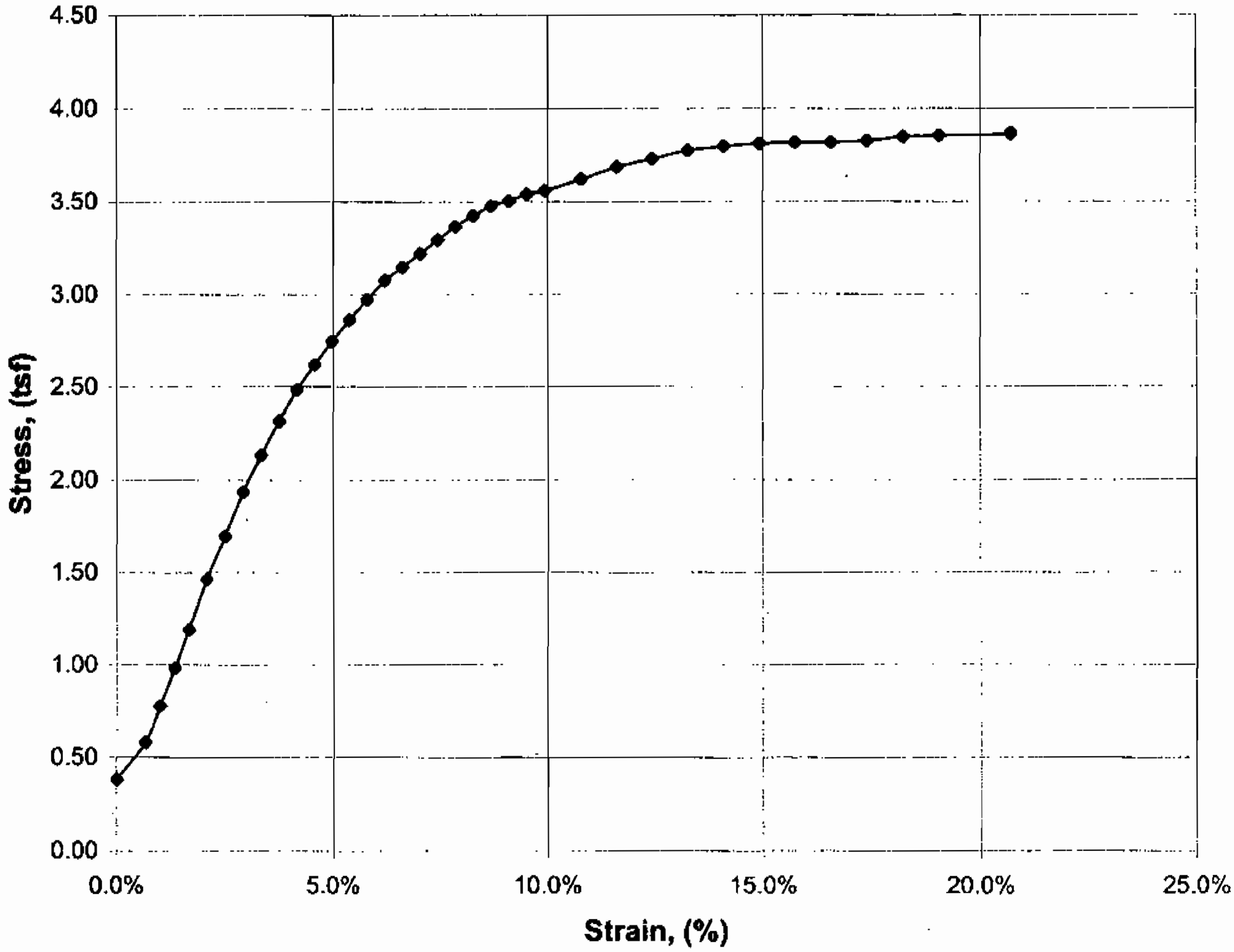
A1 S26



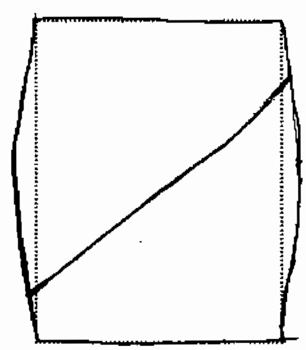
Sample Data

Sample Depth:120-122 ft
Initial Diameter: 2.863in.
Initial Height: 6.036In.
Initial Moisture Content:25 %
Liquid Limit: 44
Plastic Limit: 23
Wet Density: 125.1lb/ft3

Maximum Stress: 3.9 tons/ft2
Confining Pressure: 2.88 tons/ft2
Strain at Maximum Stress: 21%

Classification: Gray, silty CLAY (CL)

Failure Sketch



*Triaxial test results

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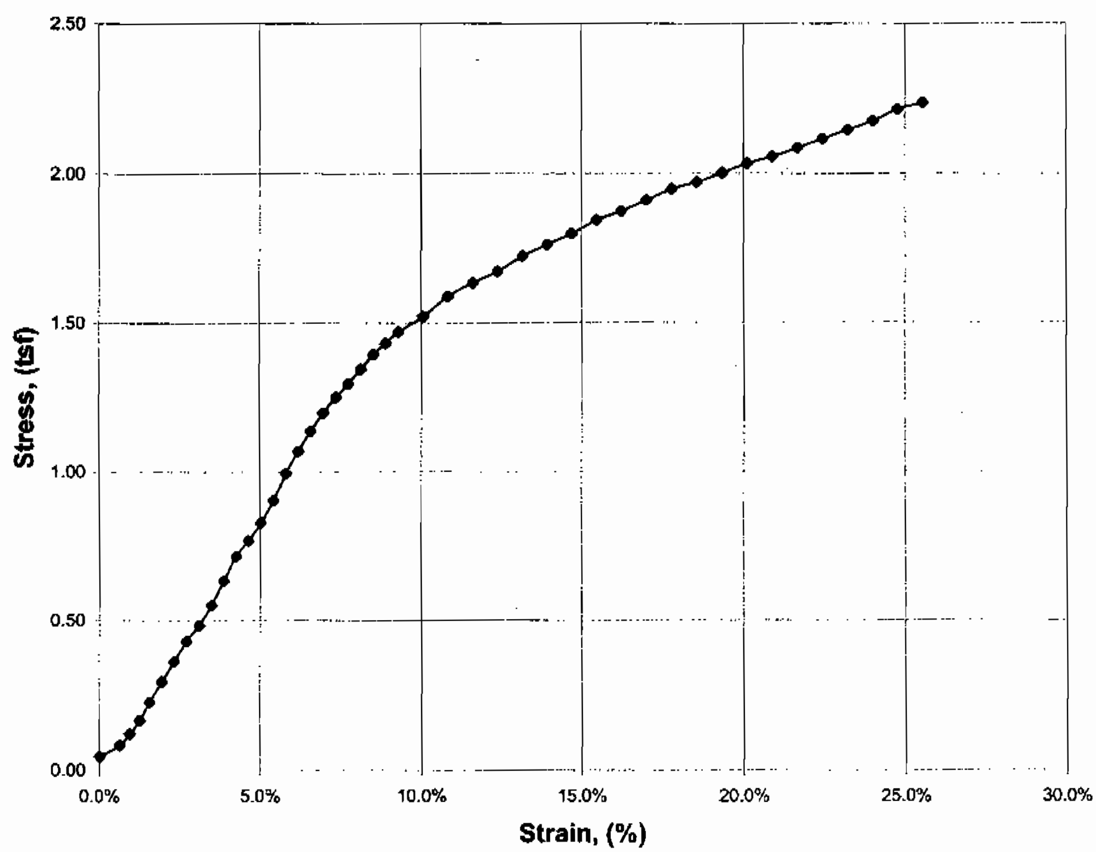
A1 S40



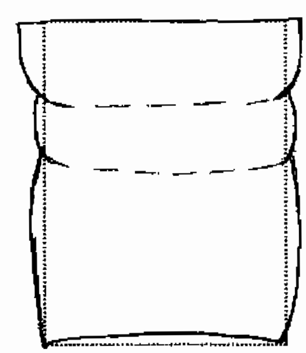
Sample Data

Sample Depth: 210-212ft
Initial Diameter: 2.819in.
Initial Height: 6.460In.
Initial Moisture Content:21 %
Liquid Limit: 32
Plastic Limit: 17
Wet Density: 142.3 lb/ft3

Maximum Stress: 2.2 tons/ft2
Confining Pressure: 2.88 tons/ft2
Strain at Maximum Stress: 26%

Classification: Gray, silty CLAY (CL)

Failure Sketch



*Triaxial test results

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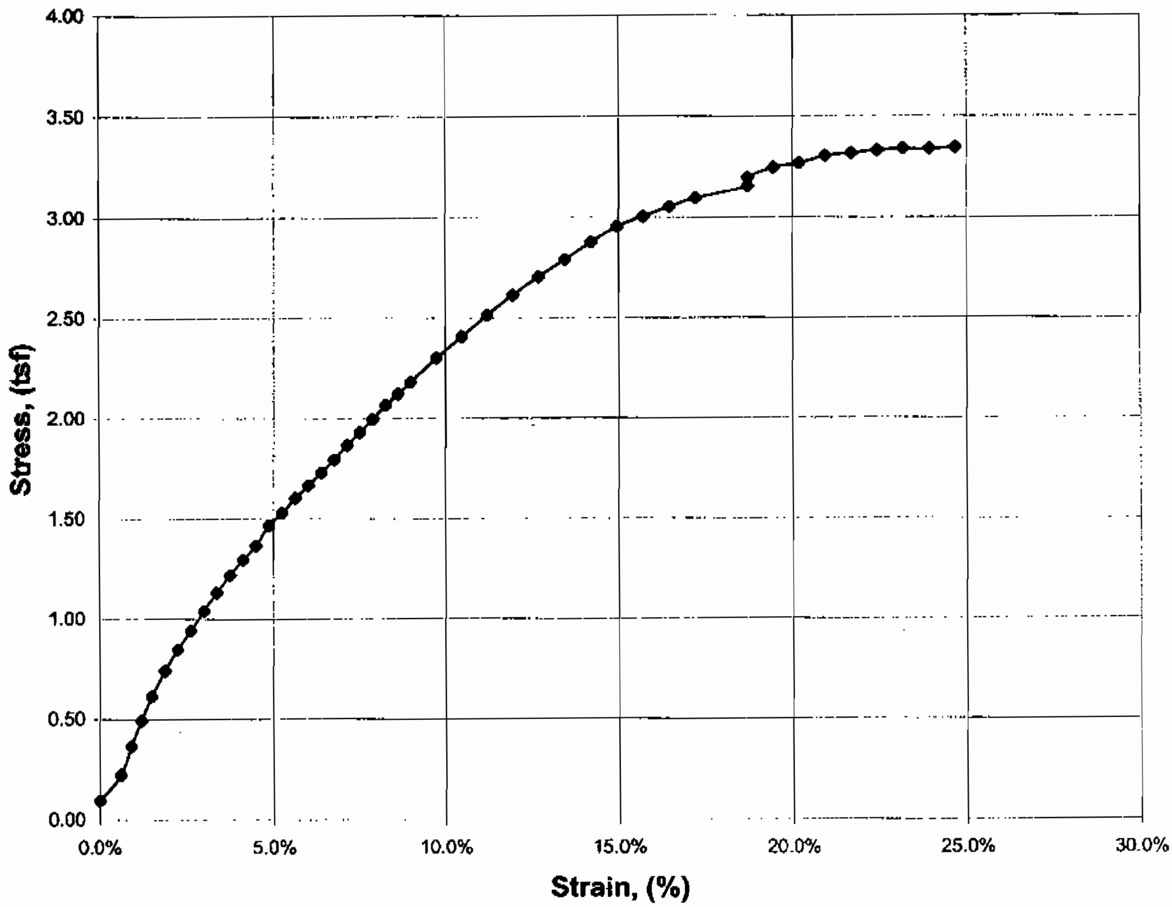
Fig. F-4

A1 S50



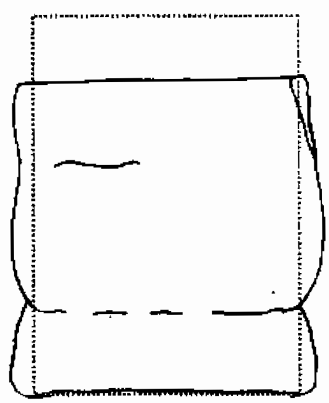
Sample Data

Sample Depth: 305-307ft
Initial Diameter: 2.853in.
Initial Height: 5.796In.
Initial Moisture Content:16%
Liquid Limit: 30
Plastic Limit: 16
Wet Density: 131.4lb/ft3

Maximum Stress: 3.3 tons/ft2
Confining Pressure: 8.1 tons/ft2
Strain at Maximum Stress: 25%

Classification: Gray, silty CLAY (CL)

Failure Sketch



*Triaxial test results

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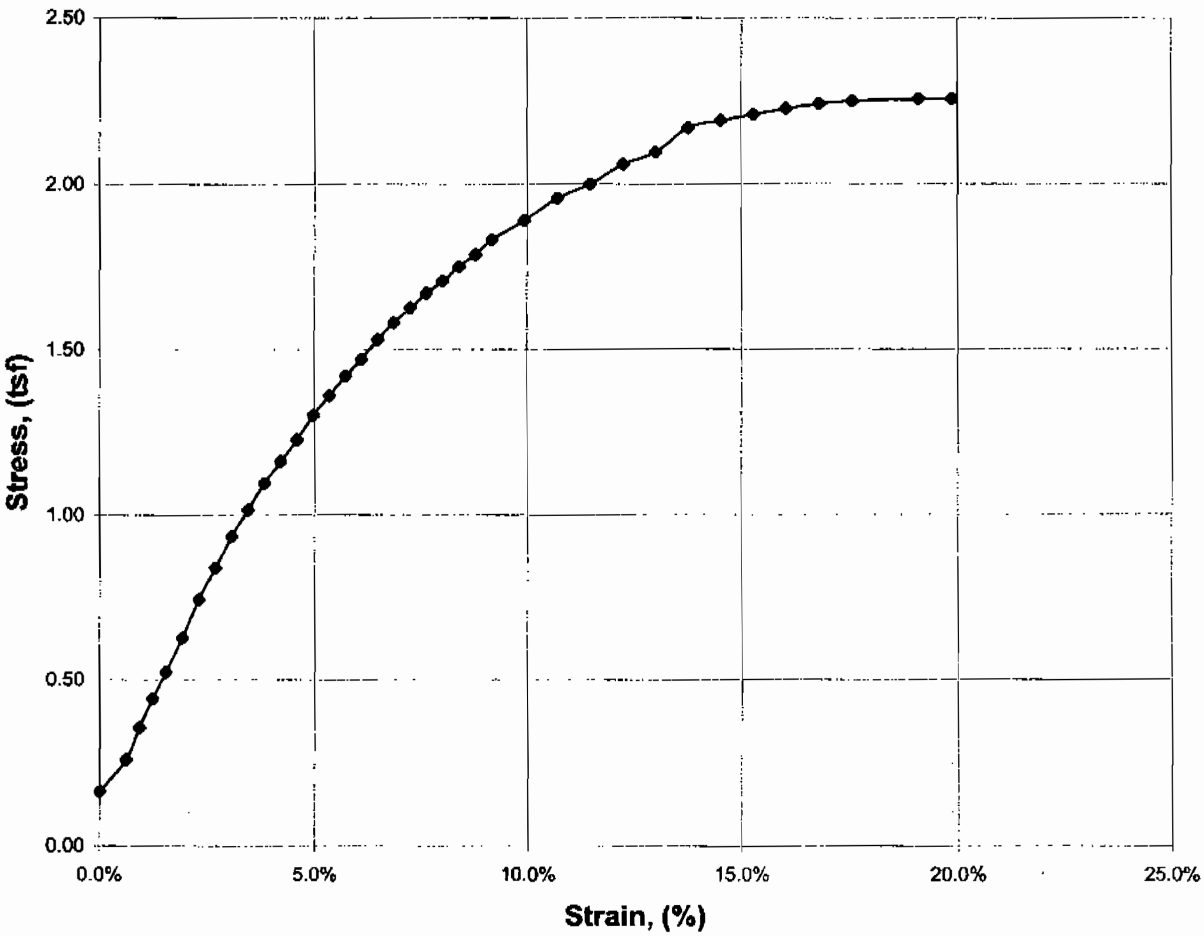


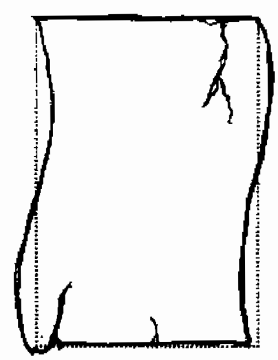
Sample Data

Sample Depth: 83-85ft
Initial Diameter: 2.852in.
Initial Height: 6.545In.
Initial Moisture Content:25 %
Liquid Limit: 40
Plastic Limit: 20
Wet Density: 128 lb/ft3

Maximum Stress:2.3 tons/ft2
Confining Pressure: 1.7 tons/ft2
Strain at Maximum Stress: 17%

Classification: Gray, silty CLAY (CL)

Failure Sketch



*Triaxial test results

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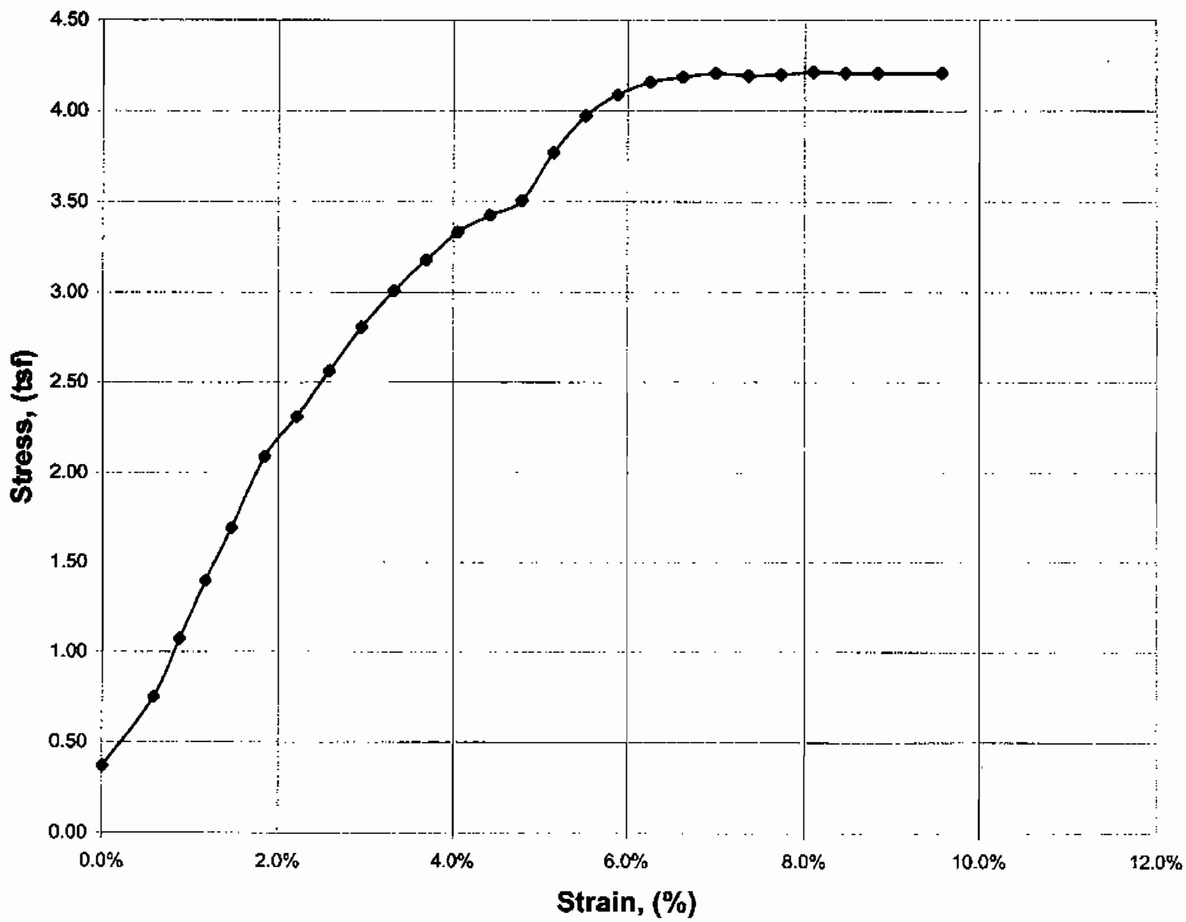
A5 S30



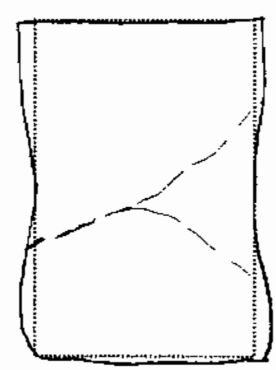
Sample Data

Sample Depth: 183-185ft
Initial Diameter: 2.883in.
Initial Height: 6.793In.
Initial Moisture Content:21%
Liquid Limit: 40
Plastic Limit: 22
Wet Density: 129.2lb/ft3

Maximum Stress: 4.2 tons/ft2
Confining Pressure: 4.8 tons/ft2
Strain at Maximum Stress: 7%

Classification: Gray, silty CLAY (CL)

Failure Sketch



*Triaxial test results

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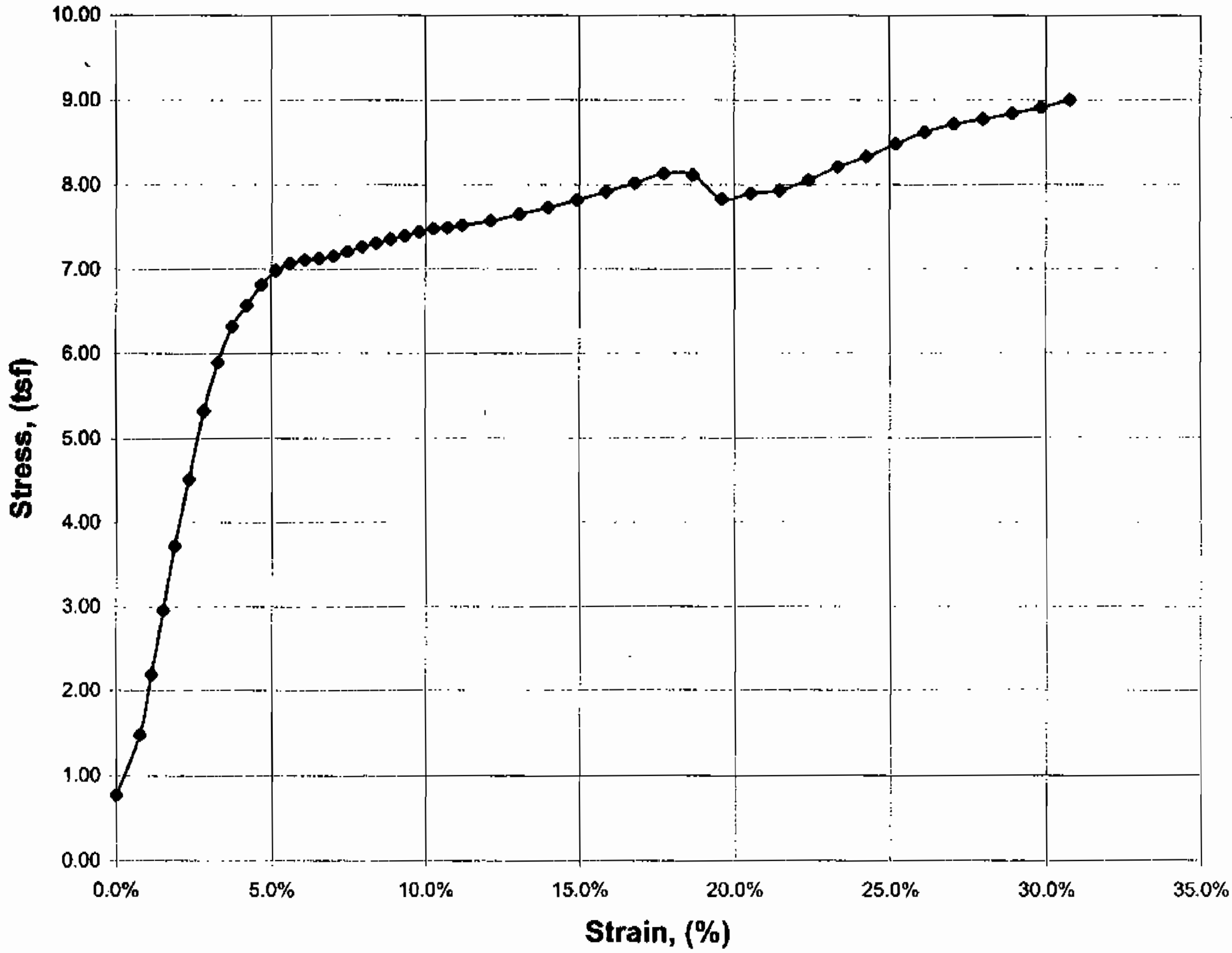
A6 S18



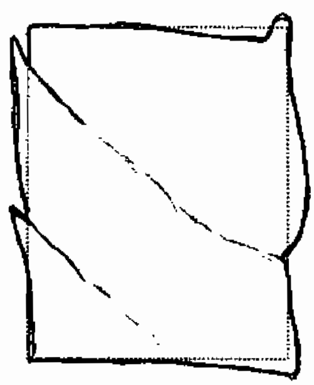
Sample Data

Sample Depth: 97-99ft
Initial Diameter: 2.847in.
Initial Height: 5.36In.
Initial Moisture Content: 22.5%
Liquid Limit: 43
Plastic Limit: 19
Wet Density: 162.2 lb/ft3

Maximum Stress: 9 tons/ft2
Confining Pressure: 1.2 tons/ft2
Strain at Maximum Stress: 30%

Classification: Gray, silty CLAY (CL)

Failure Sketch



*Triaxial test results

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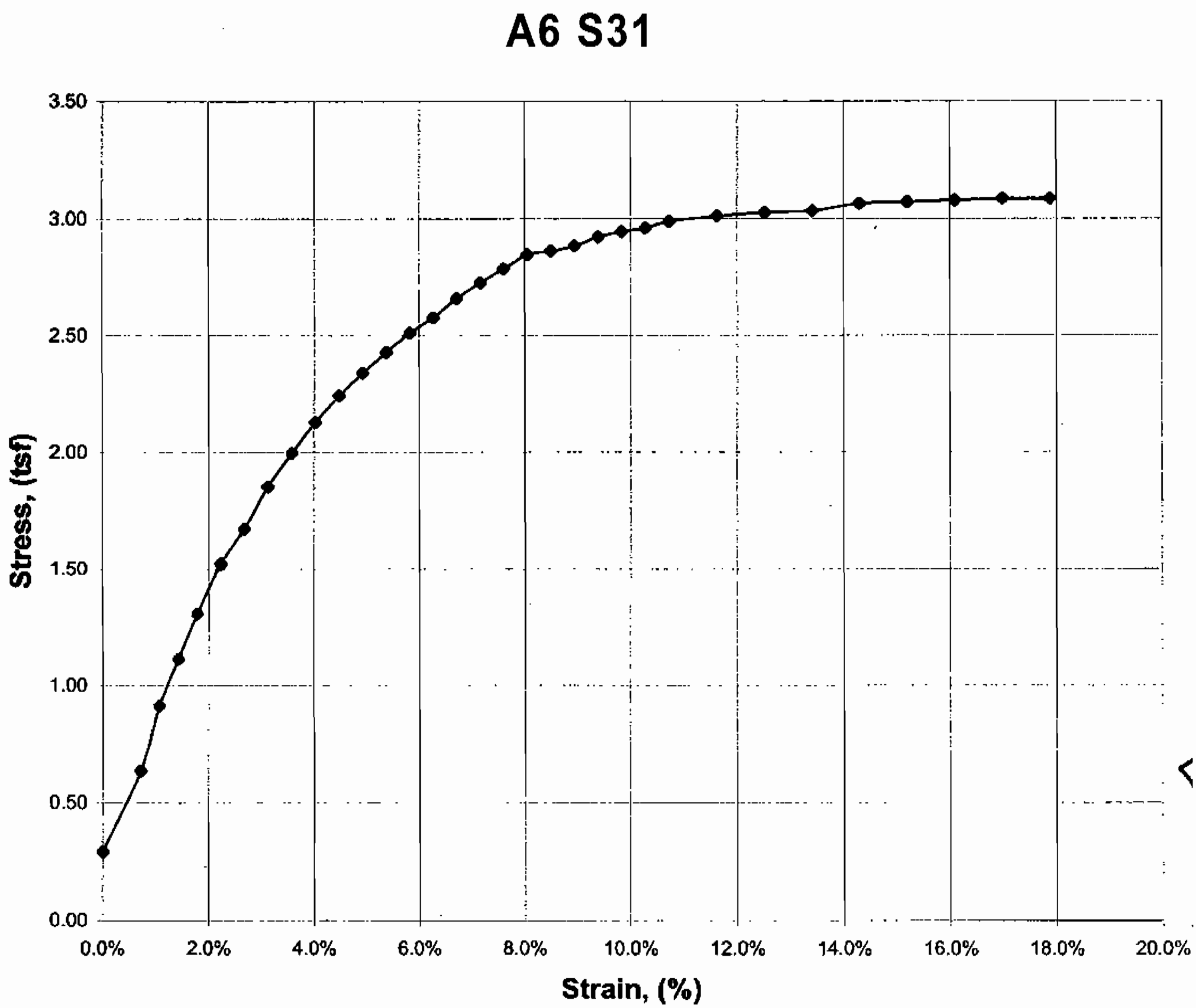


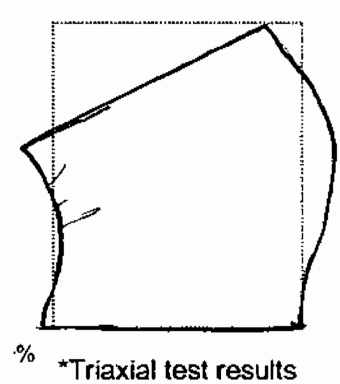
Sample Data

Sample Depth: 175-177ft
Initial Diameter: 2.829in.
Initial Height: 5.594In.
Initial Moisture Content:16.6 %
Liquid Limit: 38
Plastic Limit: 23
Wet Density: 152.8lb/ft3

Maximum Stress: 3 tons/ft2
Confining Pressure: 3.7 tons/ft2
Strain at Maximum Stress: 11%

Classification: Gray, silty CLAY (CL)

Failure Sketch



*Triaxial test results

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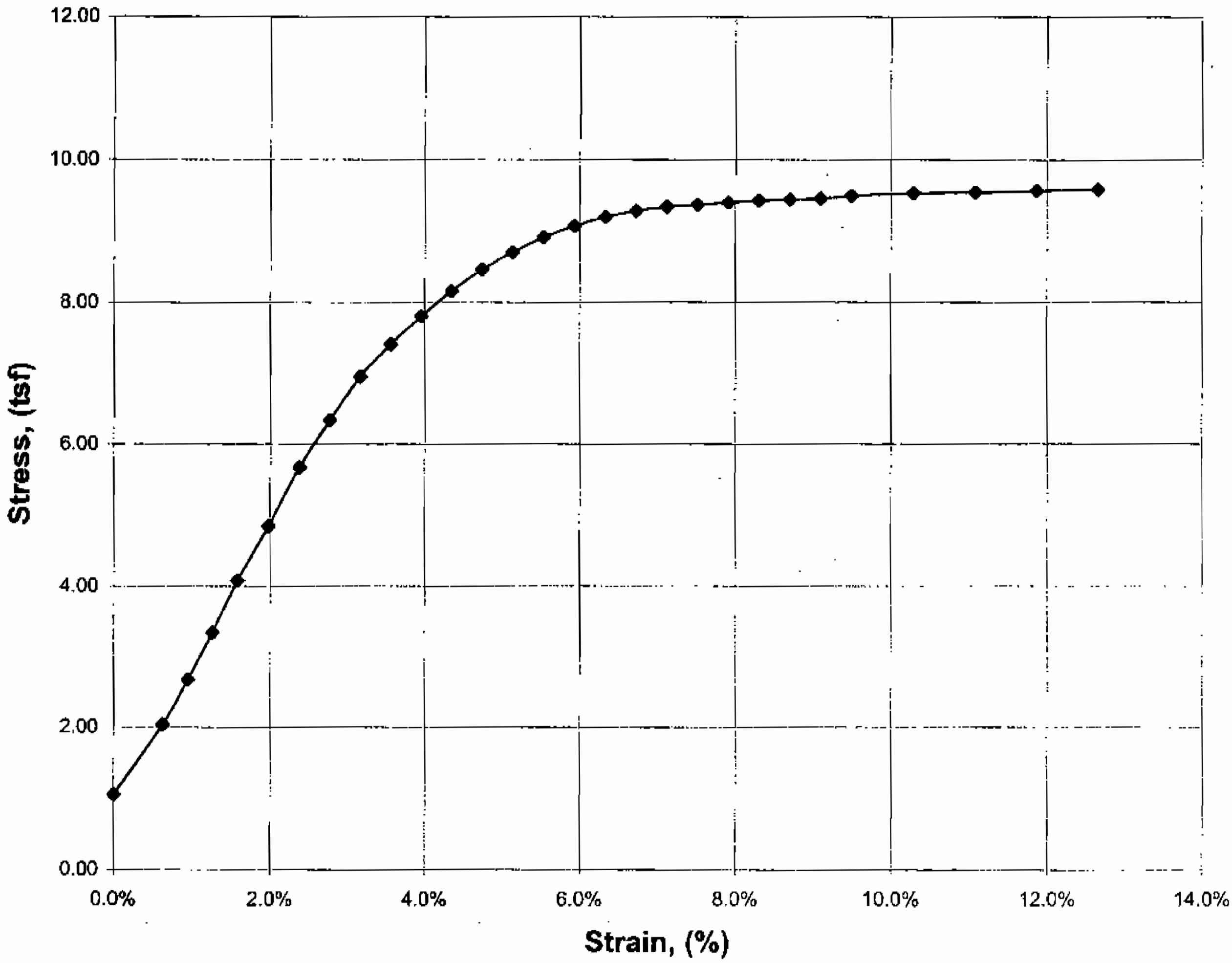
A7 S25



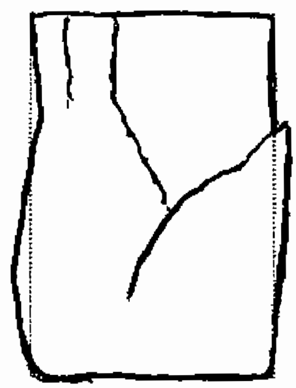
Sample Data

Sample Depth:165-167ft
Initial Diameter: 2.858 in.
Initial Height: 6.32 In.
Initial Moisture Content: 25%
Liquid Limit: 41
Plastic Limit: 22
Wet Density: 130.5 lb/ft3

Maximum Stress: 9.5 tons/ft2
Confining Pressure: 10.4 tons/ft2
Strain at Maximum Stress:10 %

Classification: Gray, silty CLAY (CL)

Failure Sketch



*Triaxial test results

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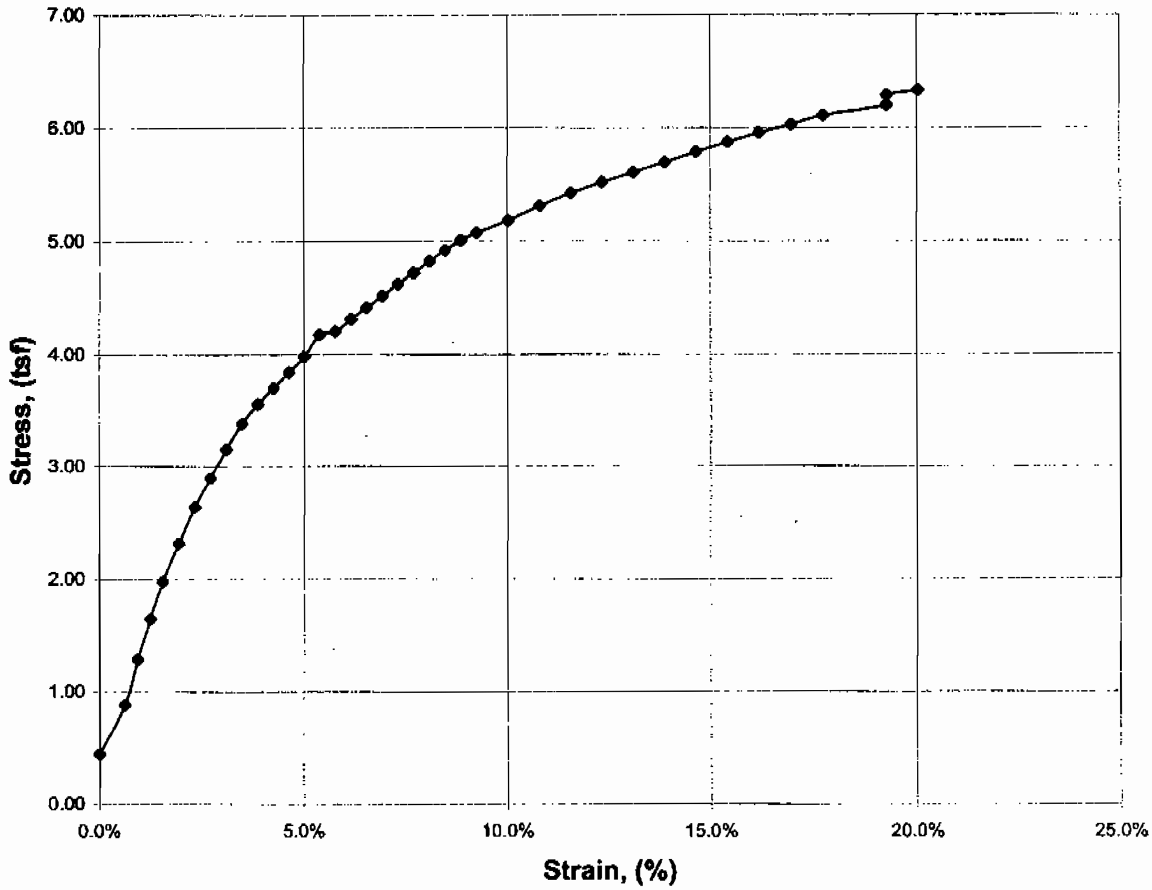
A7 S27



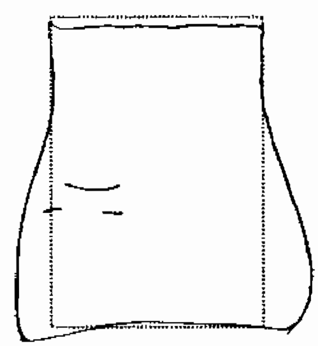
Sample Data

Sample Depth: 151-153ft
Initial Diameter:2.855 in.
Initial Height: 6.486In.
Initial Moisture Content: 20%
Liquid Limit: 39
Plastic Limit: 19
Wet Density: 125 lb/ft3

Maximum Stress: 6.5 tons/ft2
Confining Pressure: 3.8 tons/ft2
Strain at Maximum Stress: 25%

Classification: Gray, silty CLAY (CL)

Failure Sketch



*Triaxial test results

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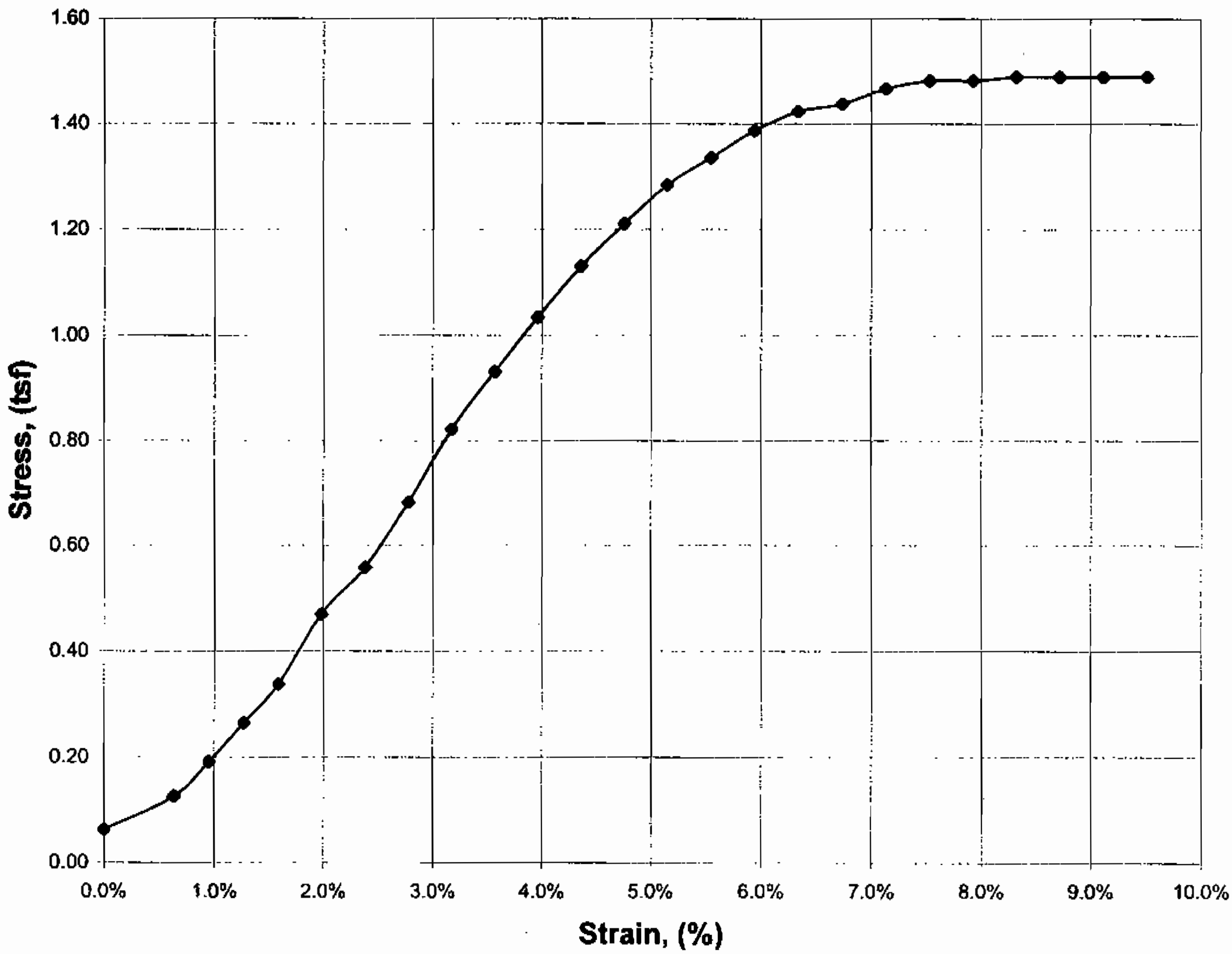
A8 S19



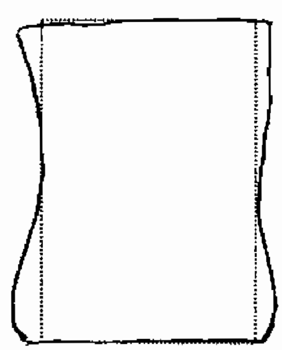
Sample Data

Sample Depth: 130-132ft
Initial Diameter: 2.855in.
Initial Height:6.31 In.
Initial Moisture Content: 19.6%
Liquid Limit: 22
Plastic Limit: 13
Wet Density: 136.6lb/ft3

Maximum Stress: 1.5 tons/ft2
Confining Pressure: 3.2 tons/ft2
Strain at Maximum Stress: 7%

Classification: Gray, silty CLAY (CL)

Failure Sketch



*Triaxial test results

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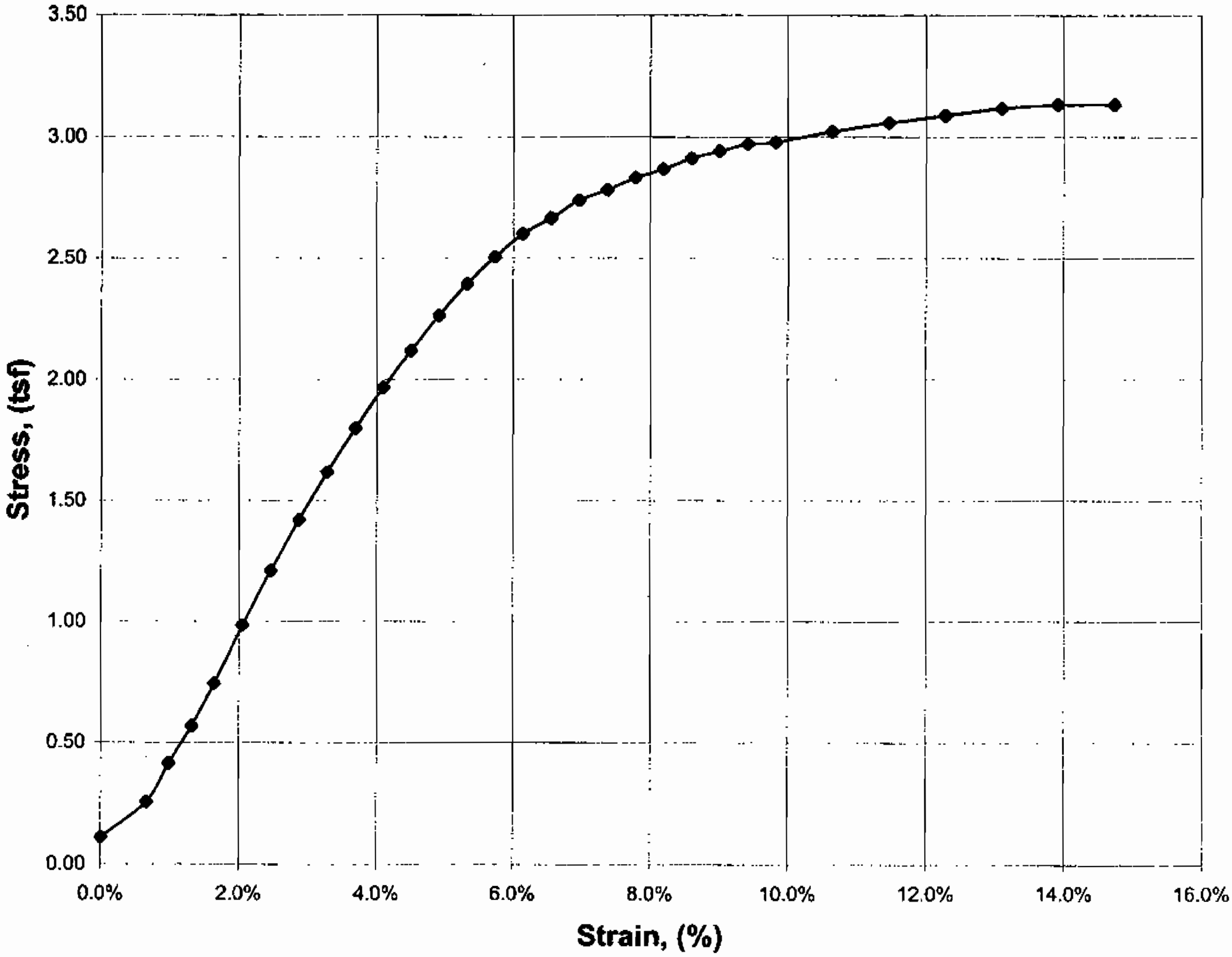
A10 S23



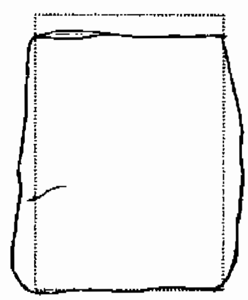
Sample Data

Sample Depth: 175.5-177ft
Initial Diameter: 2.866in.
Initial Height: 6.109 In.
Initial Moisture Content: 16%
Liquid Limit: 22
Plastic Limit: 17
Wet Density: 136lb/ft3

Maximum Stress: 3 tons/ft2
Confining Pressure: 3.4 tons/ft2
Strain at Maximum Stress:10 %

Classification: Gray, silty CLAY (CL)

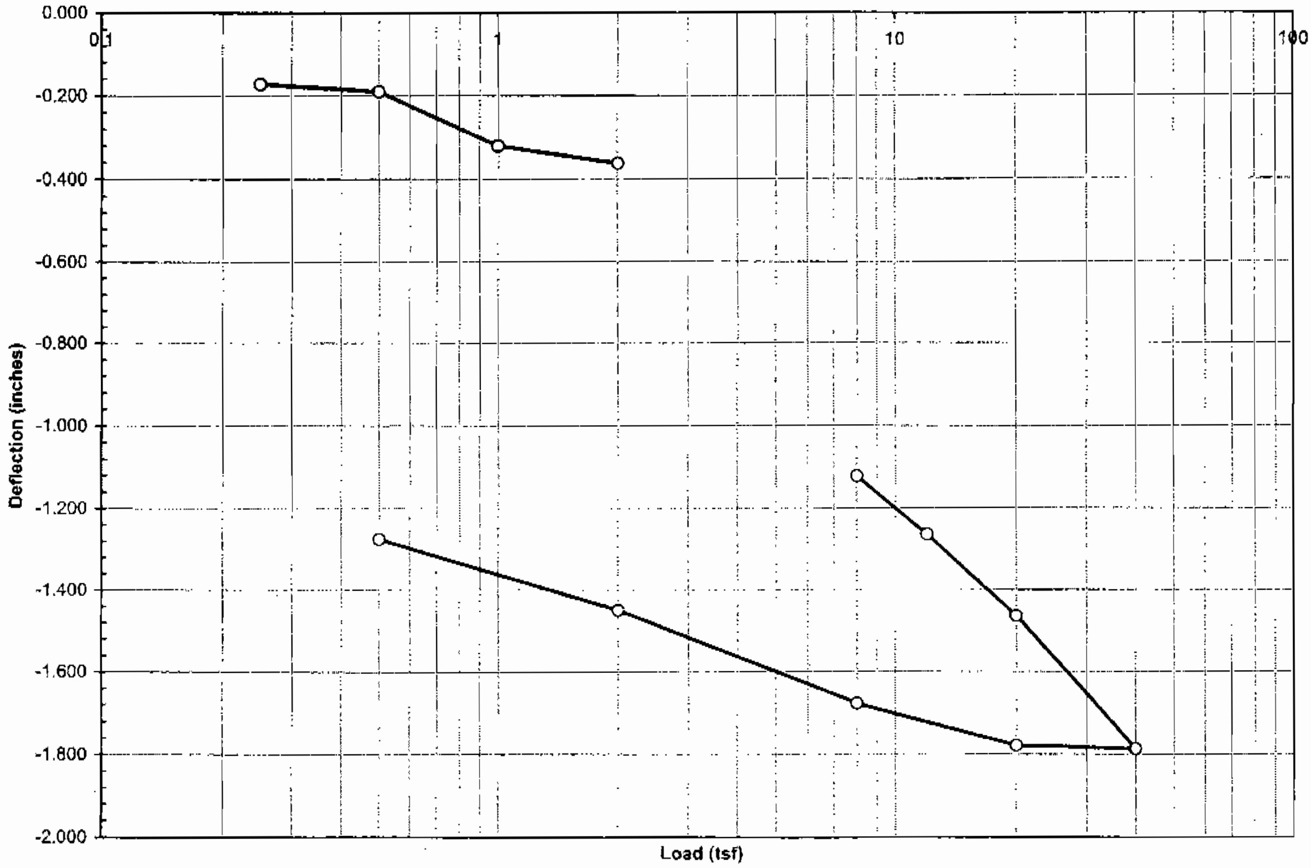
Failure Sketch



*Triaxial test results

Knik Arm Bridge Anchorage, Alaska	
TRIAXIAL COMPRESSION TEST RESULTS	
February 2004	32-1-01536
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. F-4 Sheet 13 of 13

Deflection verses Load Graph
A1 S44



Sample Descriptions:

Initial Weight:	170.9 grams
Initial Moisture:	25.1%
Initial Sample Height:	0.986 inches
Sample Diameter:	2.54 inches

Sample Classification:

Gray, silty CLAY (CL)

Knik Arm Bridge
Anchorage, Alaska

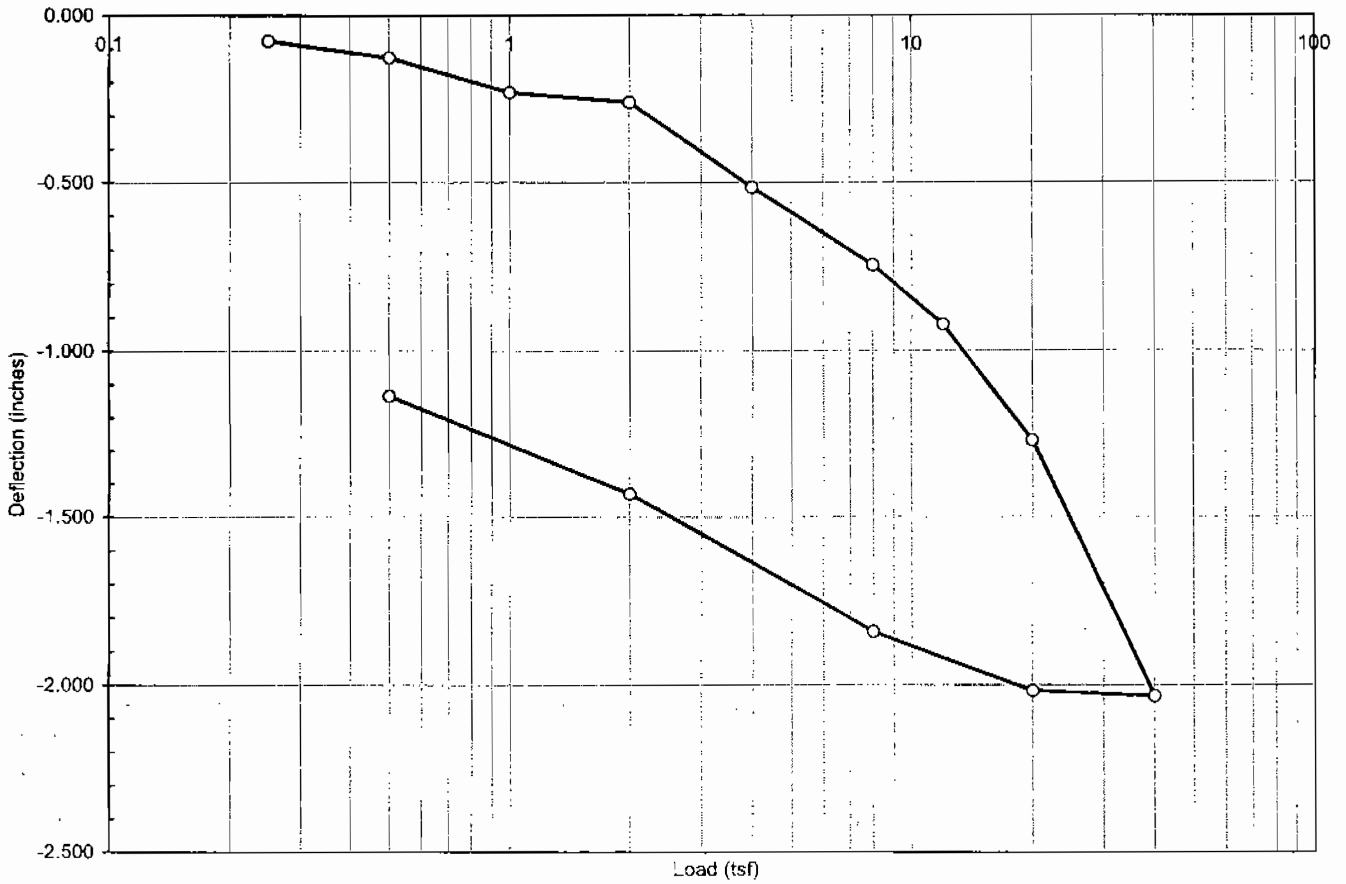
CONSOLIDATION TEST RESULTS

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. F-5
Sheet 1 of 5

Deflection verses Load Graph
A1 S49



Sample Descriptions:

Initial Weight:	167.35 grams
Initial Moisture:	28%
Initial Sample Height:	0.986 inches
Sample Diameter:	2.54 inches

Sample Classification:

Gray, silty CLAY (CL)

Knik Arm Bridge
Anchorage, Alaska

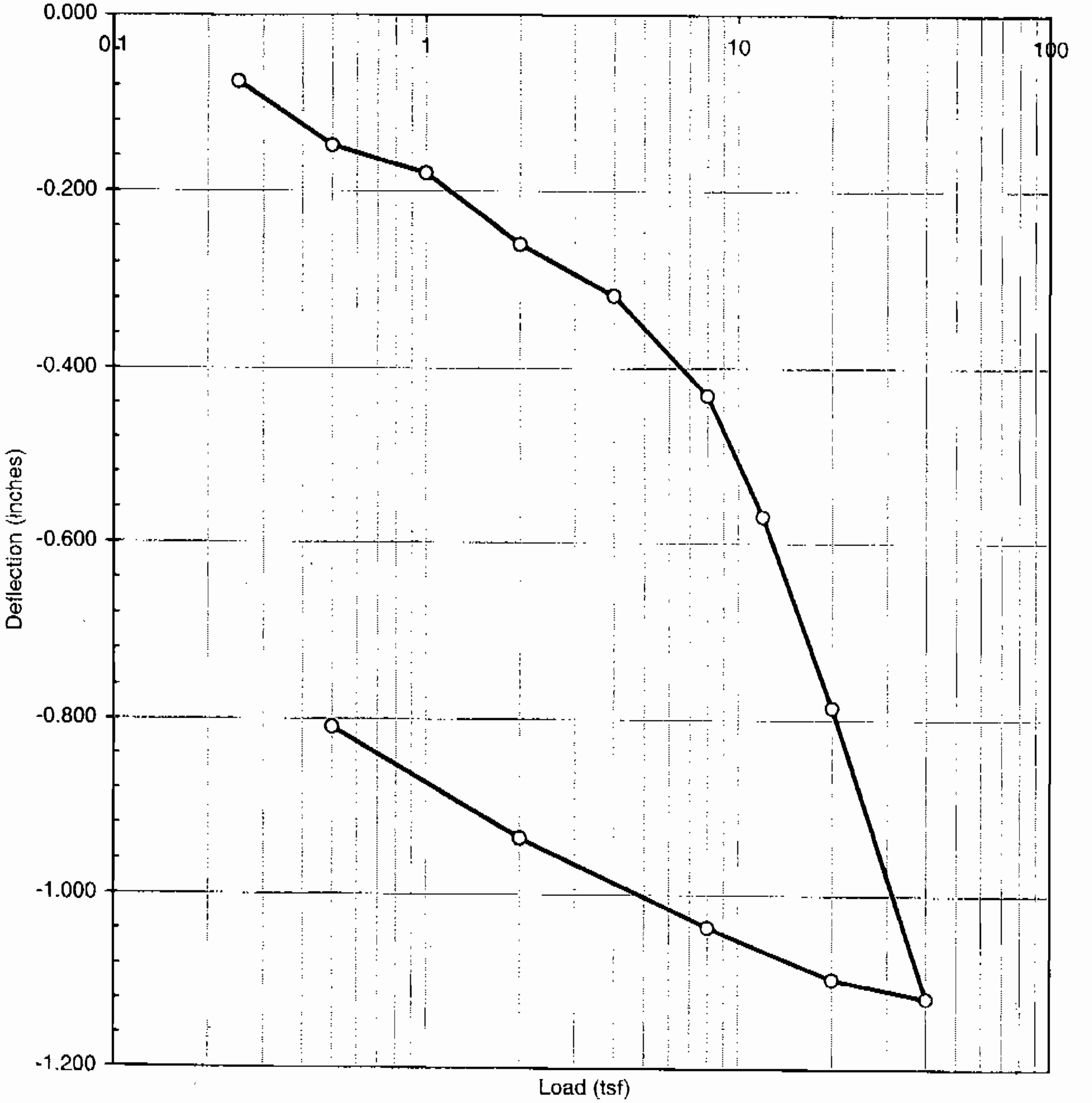
CONSOLIDATION TEST RESULTS

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. F-5
Sheet 2 of 5

Deflection verses Load Graph
A4 S19



Sample Descriptions:

Initial Weight:	183.65 grams
Initial Moisture:	16.7%
Initial Sample Height:	0.986 inches
Sample Diameter:	2.54 inches

Sample Classification:

Gray, slightly sandy, gravelly, silty CLAY (CL)

Knik Arm Bridge
Anchorage, Alaska

CONSOLIDATION TEST RESULTS

February 2004 32-1-01536

SHANNON & WILSON, INC.
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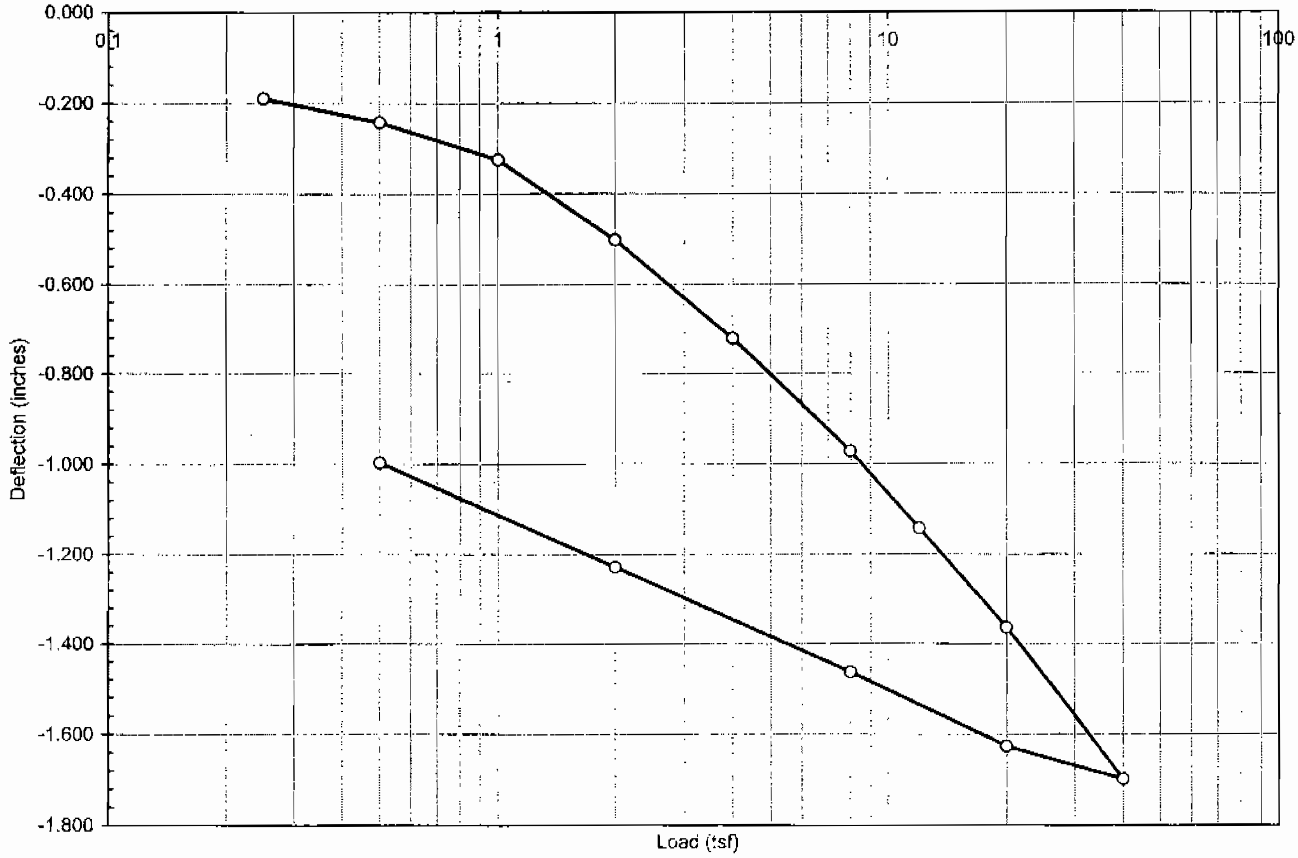
Fig. F-5
Sheet 3 of 5

Deflection verses Load Graph
A5 S14



Sample Descriptions:

Initial Weight:	174.86 grams
Initial Moisture:	25.9%
Initial Sample Height:	0.986 inches
Sample Diameter:	2.54 inches

Sample Classification:

Gray, sandy silty CLAY to silty clayey SAND (CL-SC)

Knik Arm Bridge
Anchorage, Alaska

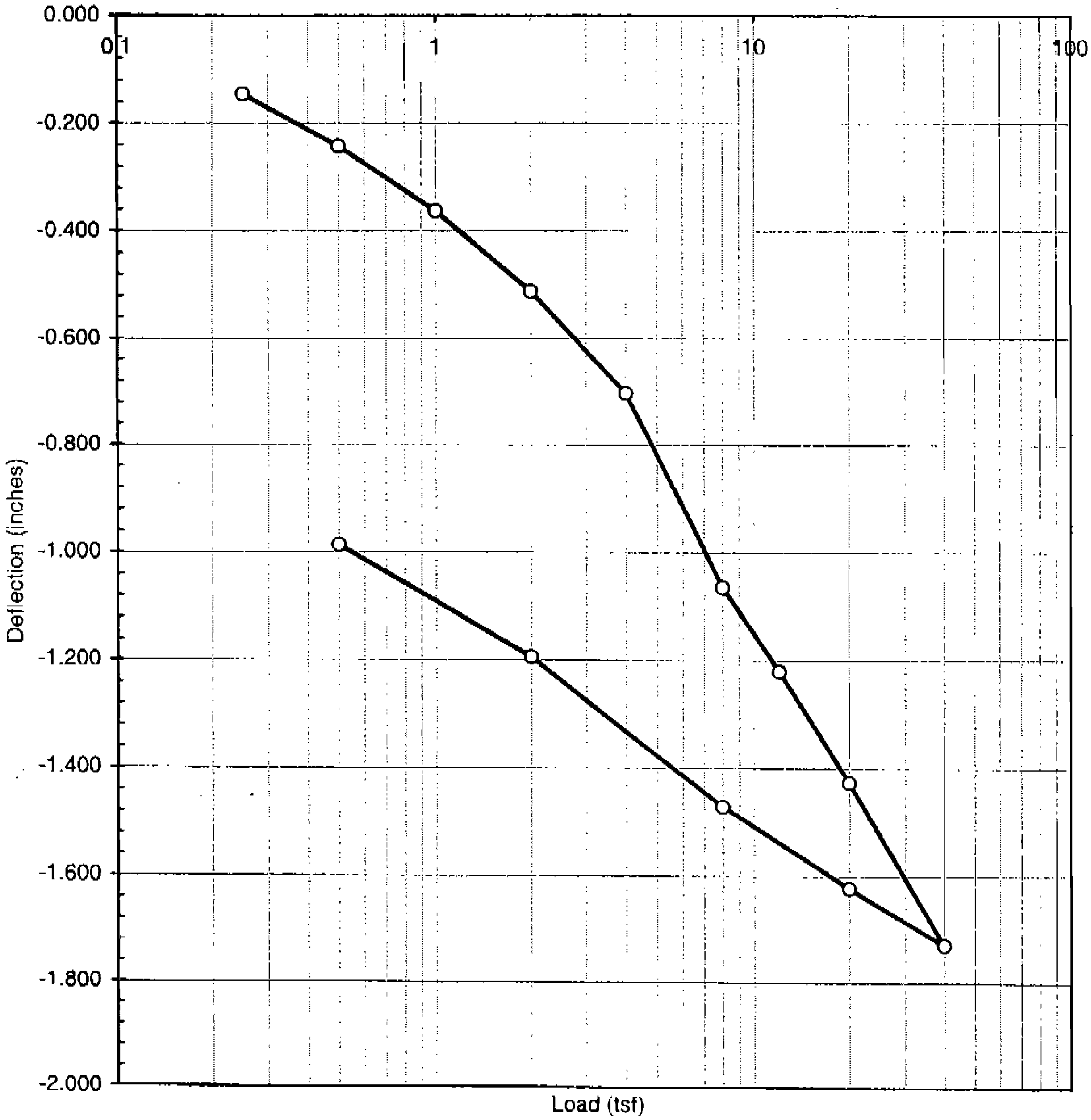
CONSOLIDATION TEST RESULTS

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. F-5
Sheet 4 of 5

Deflection verses Load Graph
A6 S7



Sample Descriptions:

Initial Weight: 185.01 grams
Initial Moisture: 20.6%
Initial Sample Height: 0.986 inches
Sample Diameter: 2.54 inches

Sample Classification:

Gray, gravelly, silty CLAY (CL)

Knik Arm Bridge
Anchorage, Alaska

CONSOLIDATION TEST RESULTS

February 2004 32-1-01536

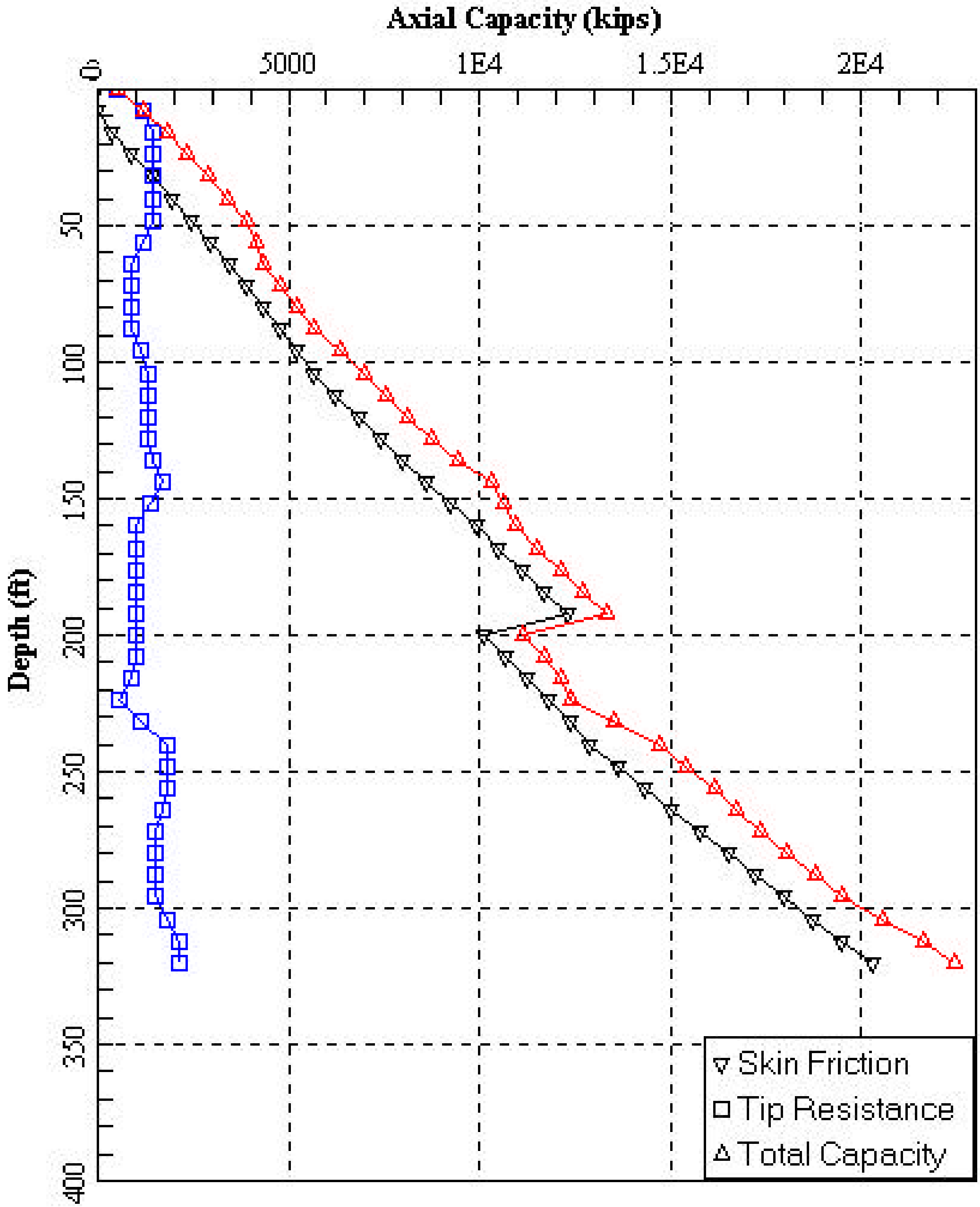
SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. F-5
Sheet 5 of 5

APPENDIX G

ULTIMATE CAPACITY AND EMBEDMENT DEPTHS FOR 8- AND 4 FT DIAMETER PIPE PILES

LIST OF FIGURES

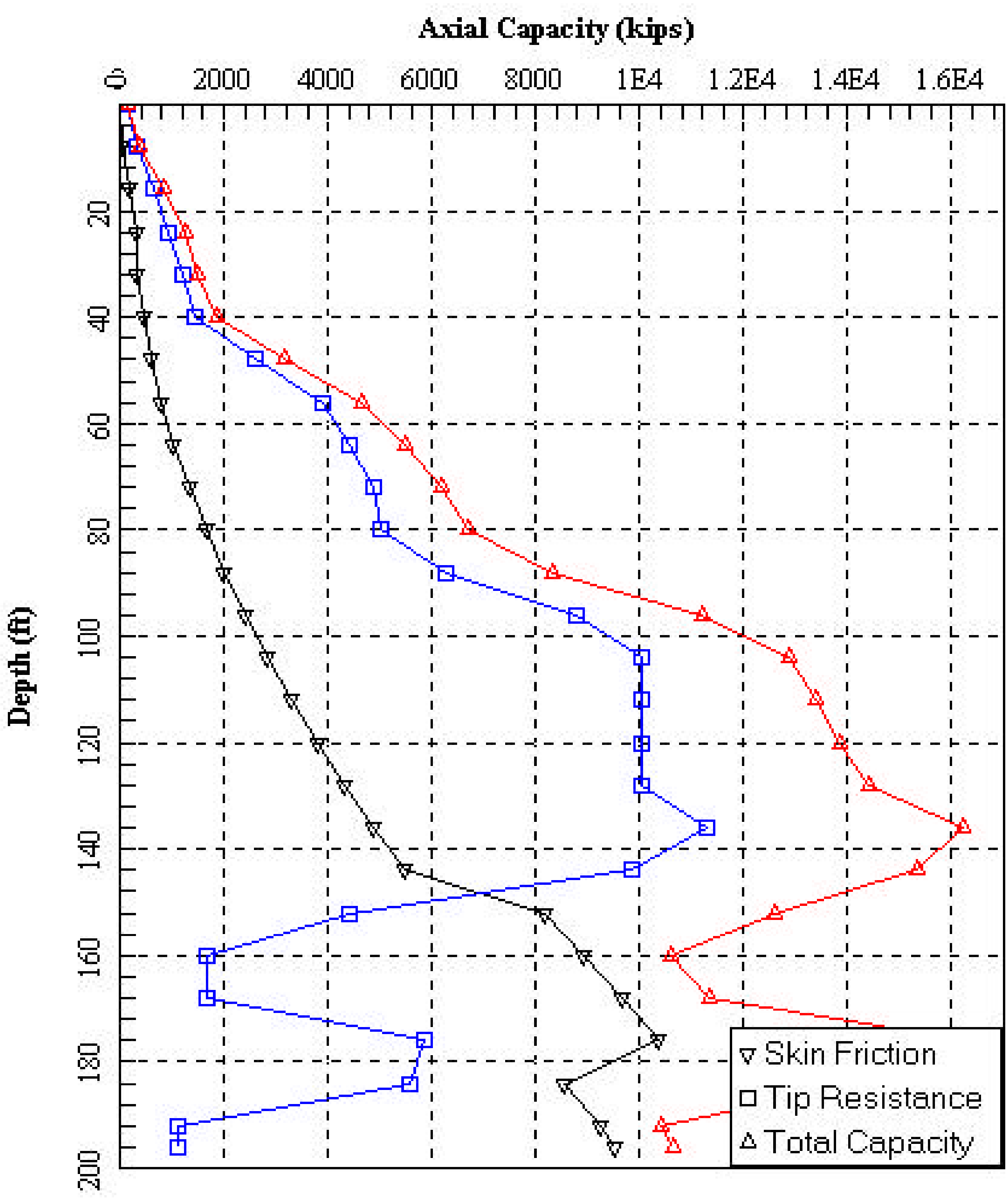


ULTIMATE CAPACITY VS DEPTH

BORING A-1

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge Anchorage, Alaska	
ULTIMATE CAPACITY 8 FT. DIAMETER PIPE PILE February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. G-1

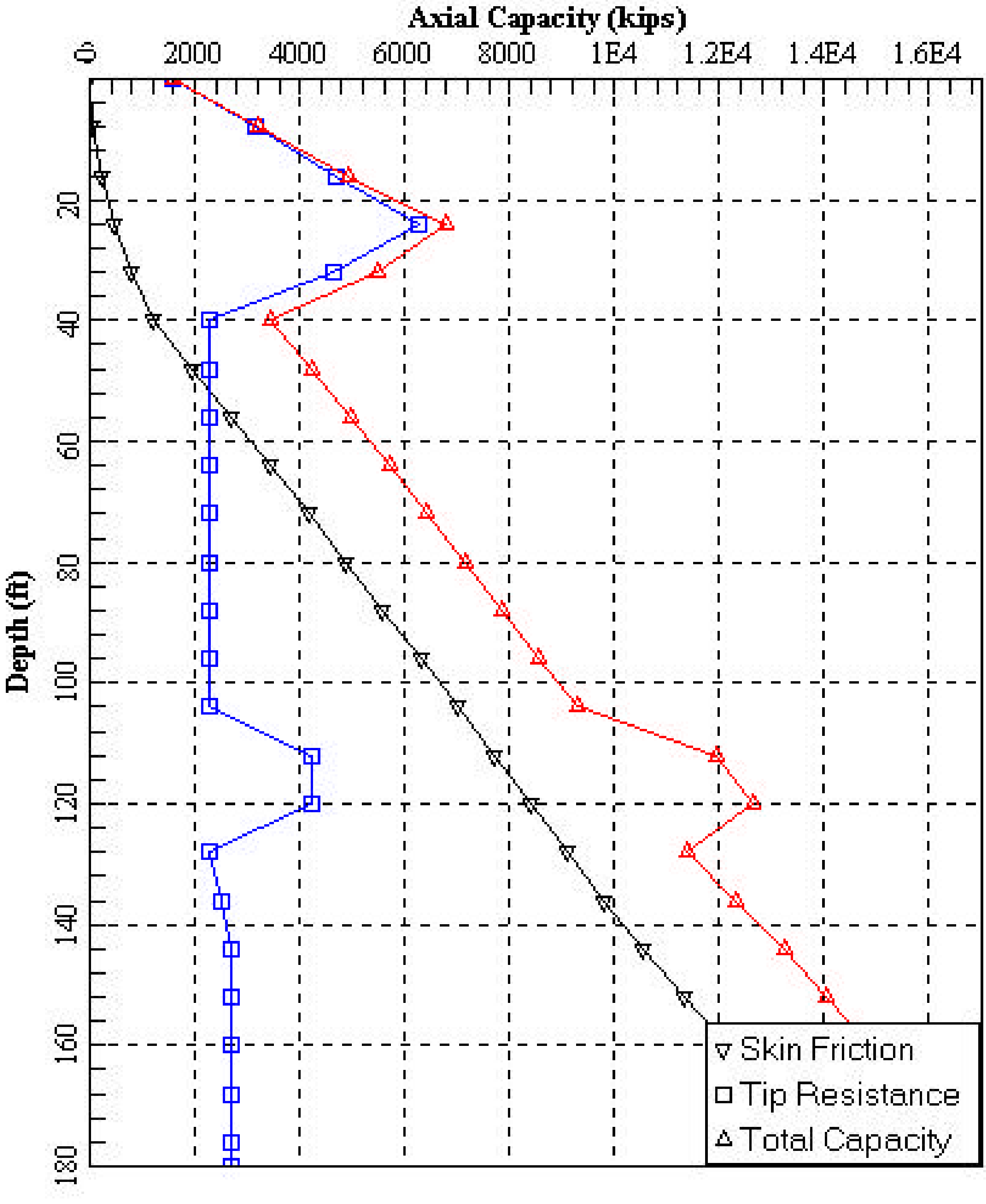


ULTIMATE CAPACITY VS DEPTH

BORING A-2

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge Anchorage, Alaska	
ULTIMATE CAPACITY 8 FT. DIAMETER PIPE PILE February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. G-2



ULTIMATE CAPACITY VS DEPTH

BORING A-4

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge
Anchorage, Alaska

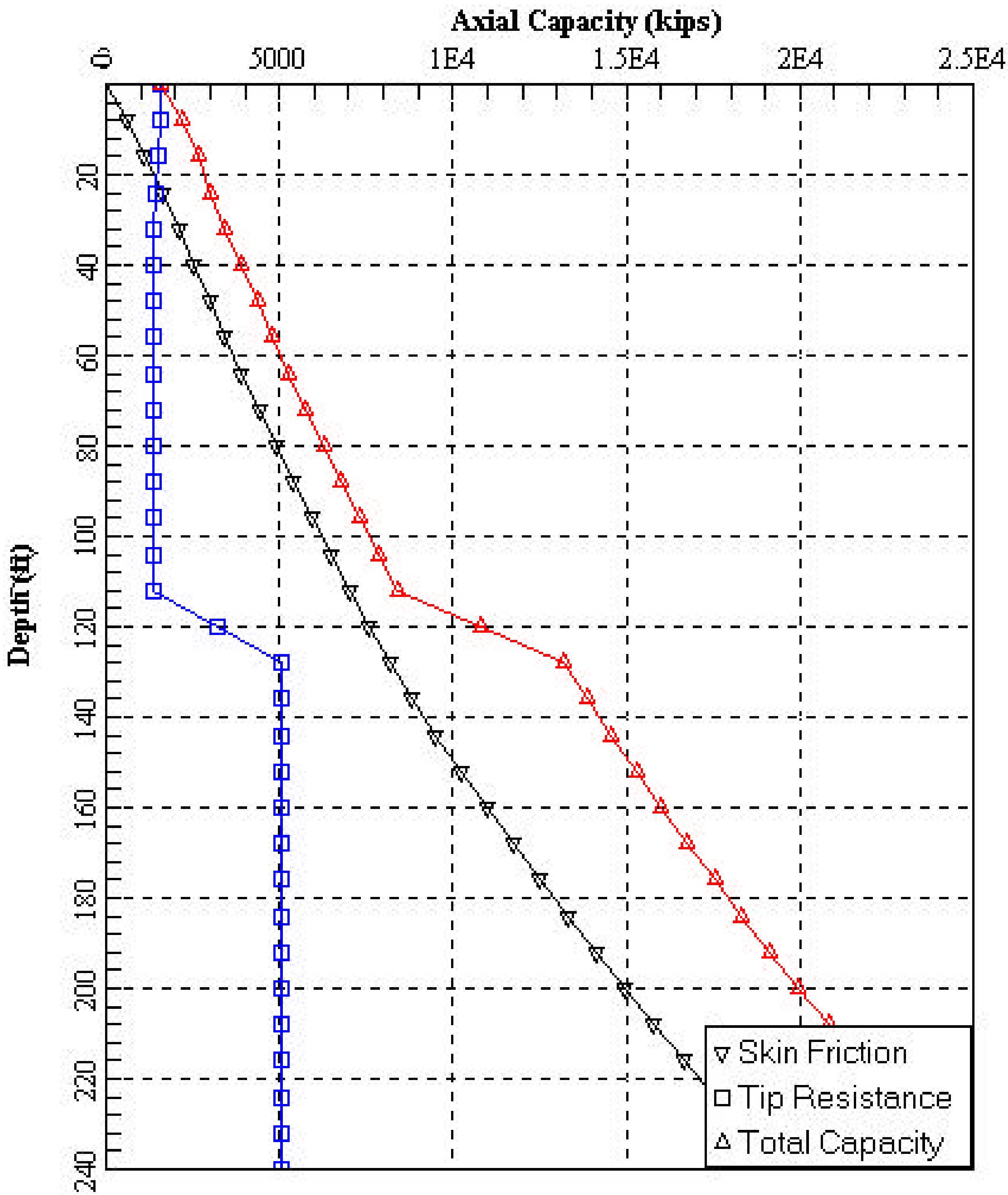
ULTIMATE CAPACITY
8 FT. DIAMETER PIPE PILE

February 2004

32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. G-3

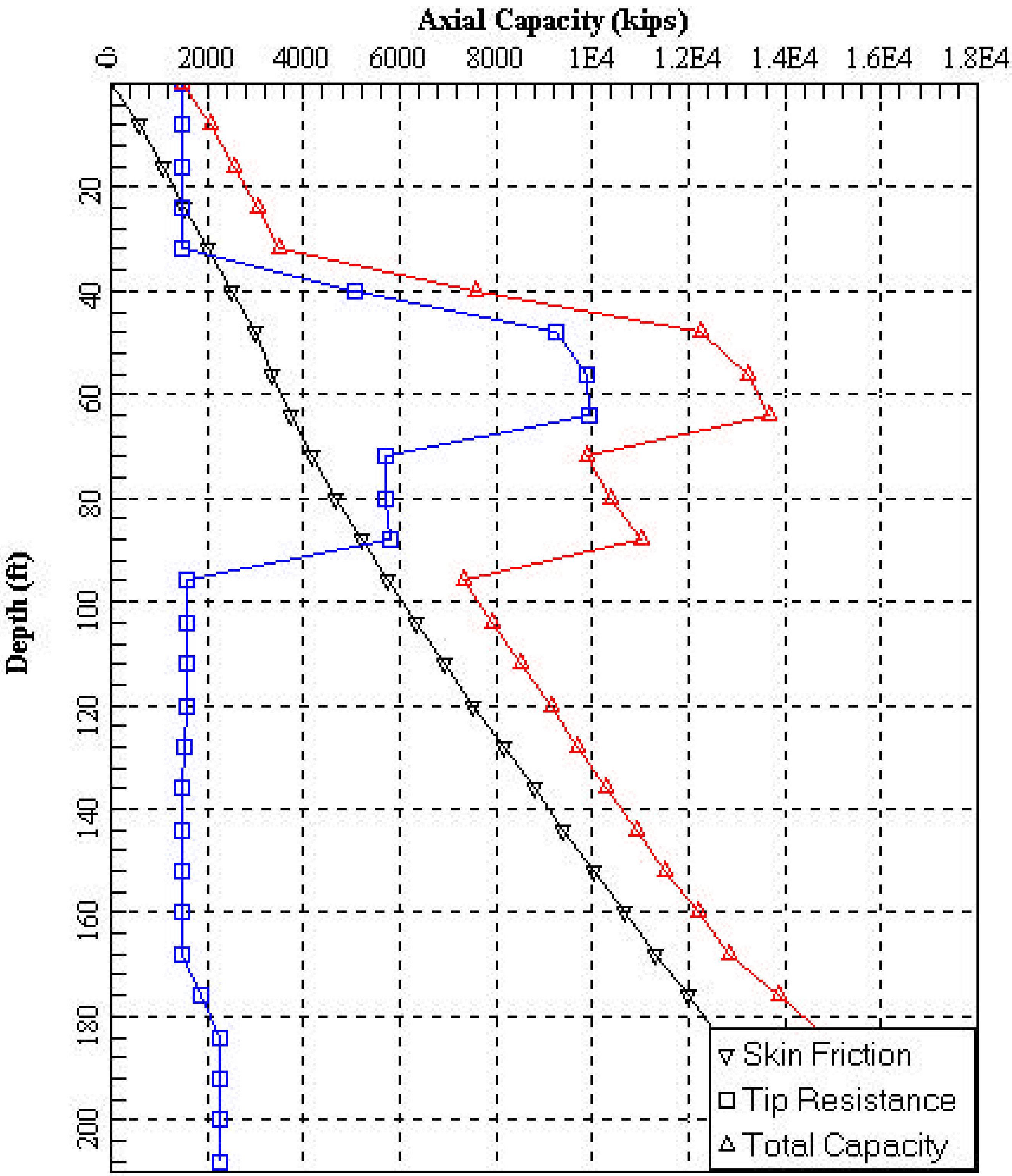


ULTIMATE CAPACITY VS DEPTH

BORING A-5

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge Anchorage, Alaska	
ULTIMATE CAPACITY 8 FT. DIAMETER PIPE PILE February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. G-4

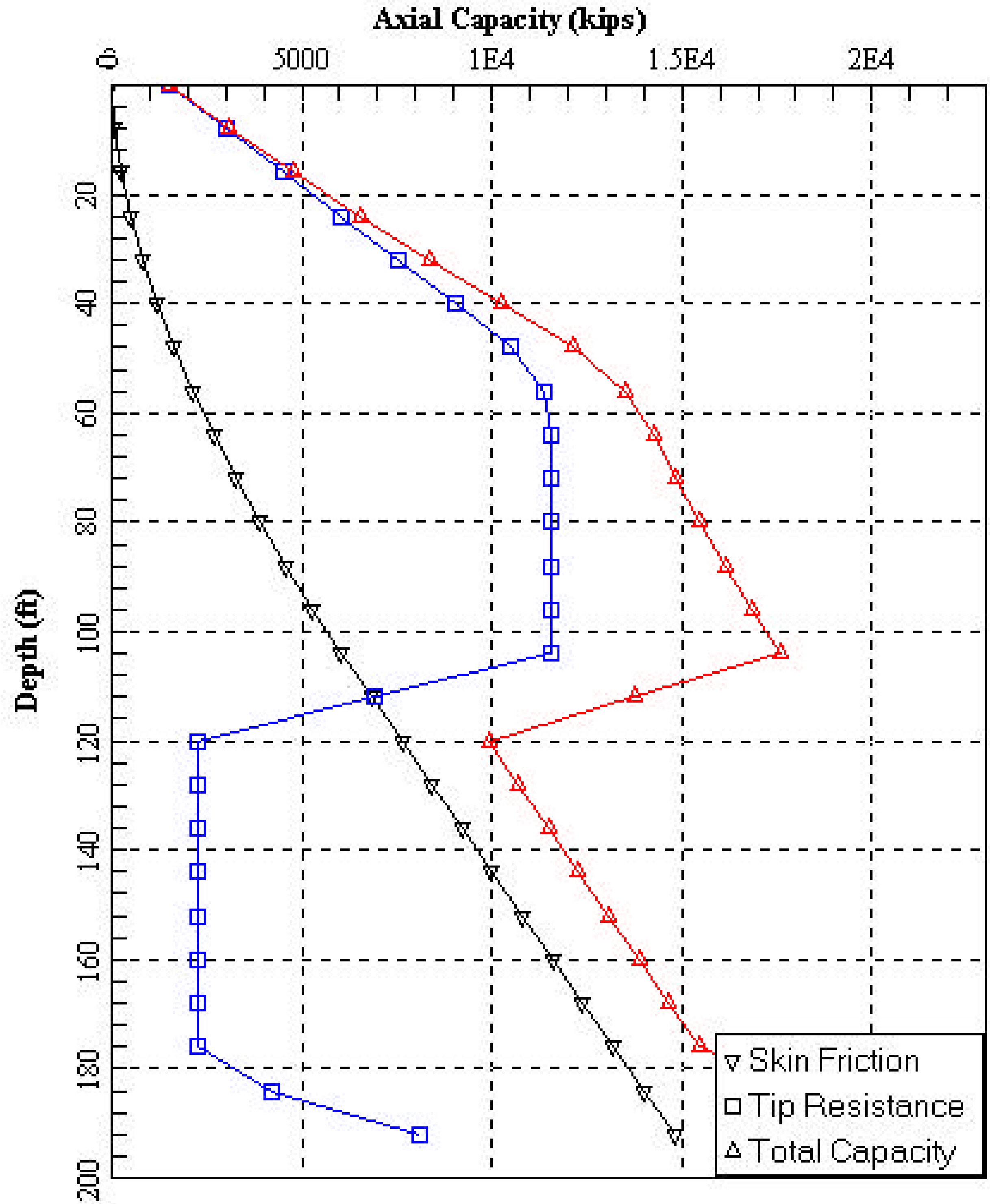


ULTIMATE CAPACITY VS DEPTH

BORING A-6

ASSUMES LOOSE SOILS DRILLED OUT AND CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge Anchorage, Alaska	
ULTIMATE CAPACITY **8 FT. DIAMETER PIPE PILE** February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	**Fig. G-5**

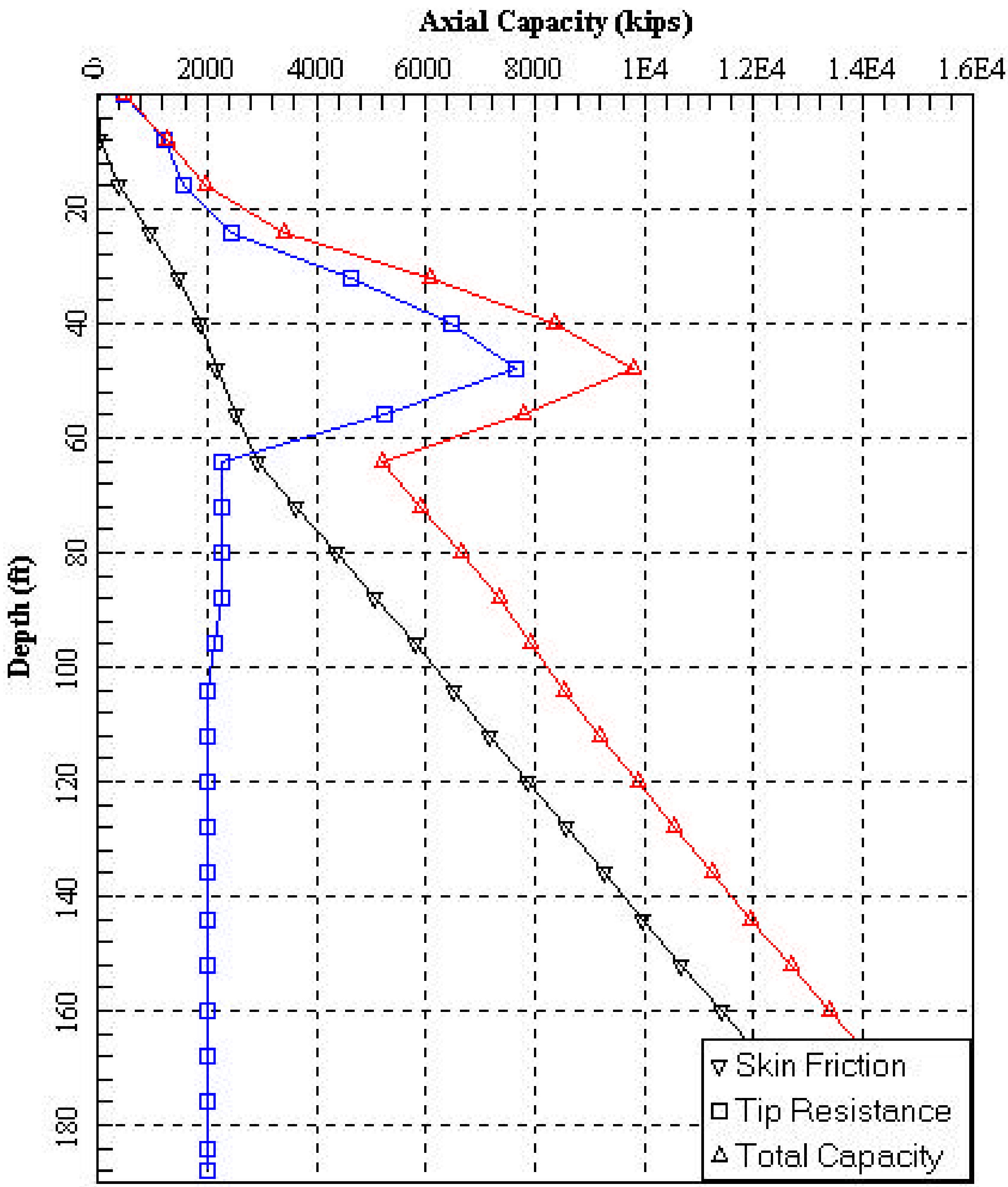


ULTIMATE CAPACITY VS DEPTH

BORING A-7

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge Anchorage, Alaska	
ULTIMATE CAPACITY 8 FT. DIAMETER PIPE PILE February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. G-6

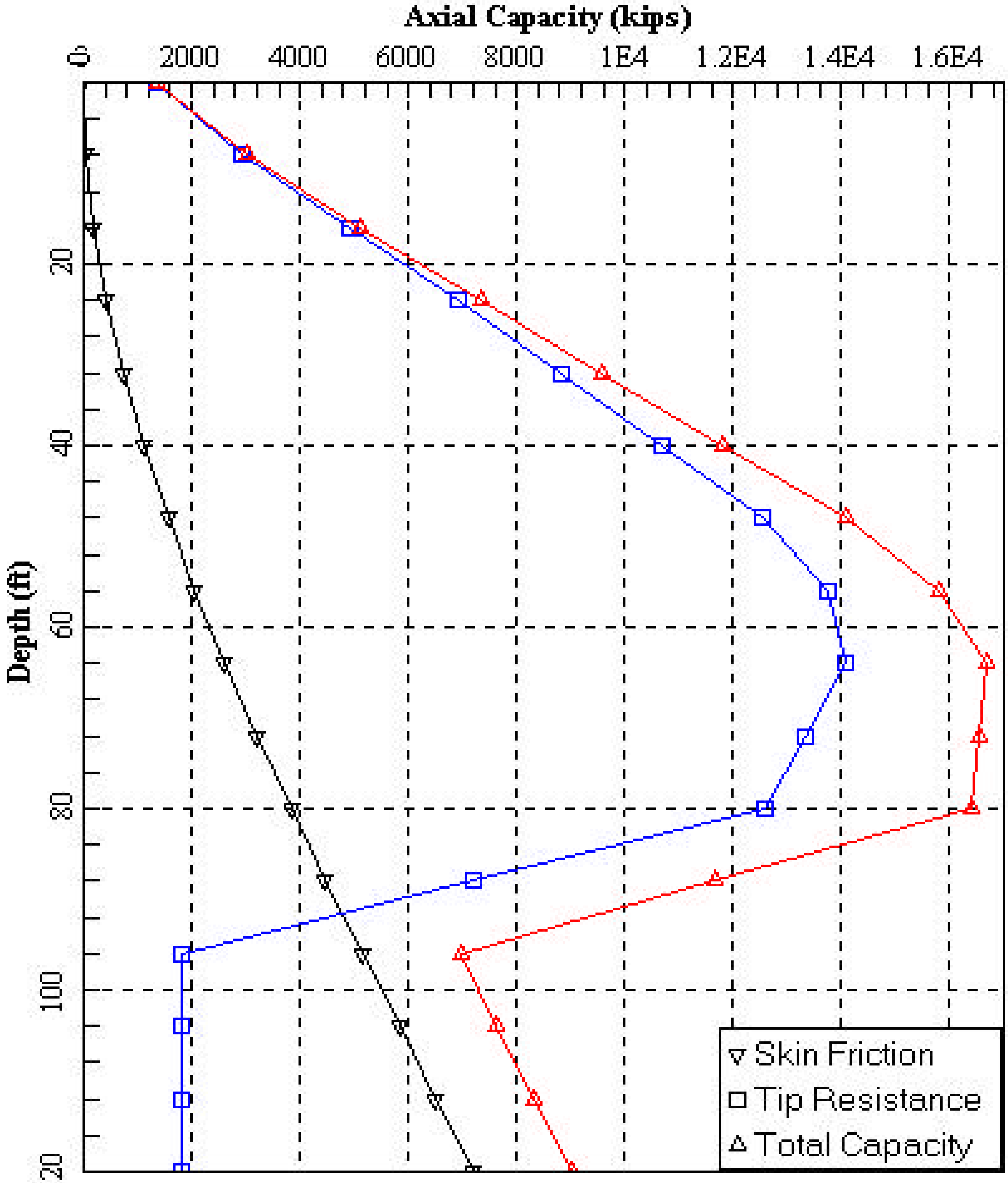


ULTIMATE CAPACITY VS DEPTH

BORING A-8

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge Anchorage, Alaska	
ULTIMATE CAPACITY 8 FT. DIAMETER PIPE PILE February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. G-7



ULTIMATE CAPACITY VS DEPTH

BORING A-9

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

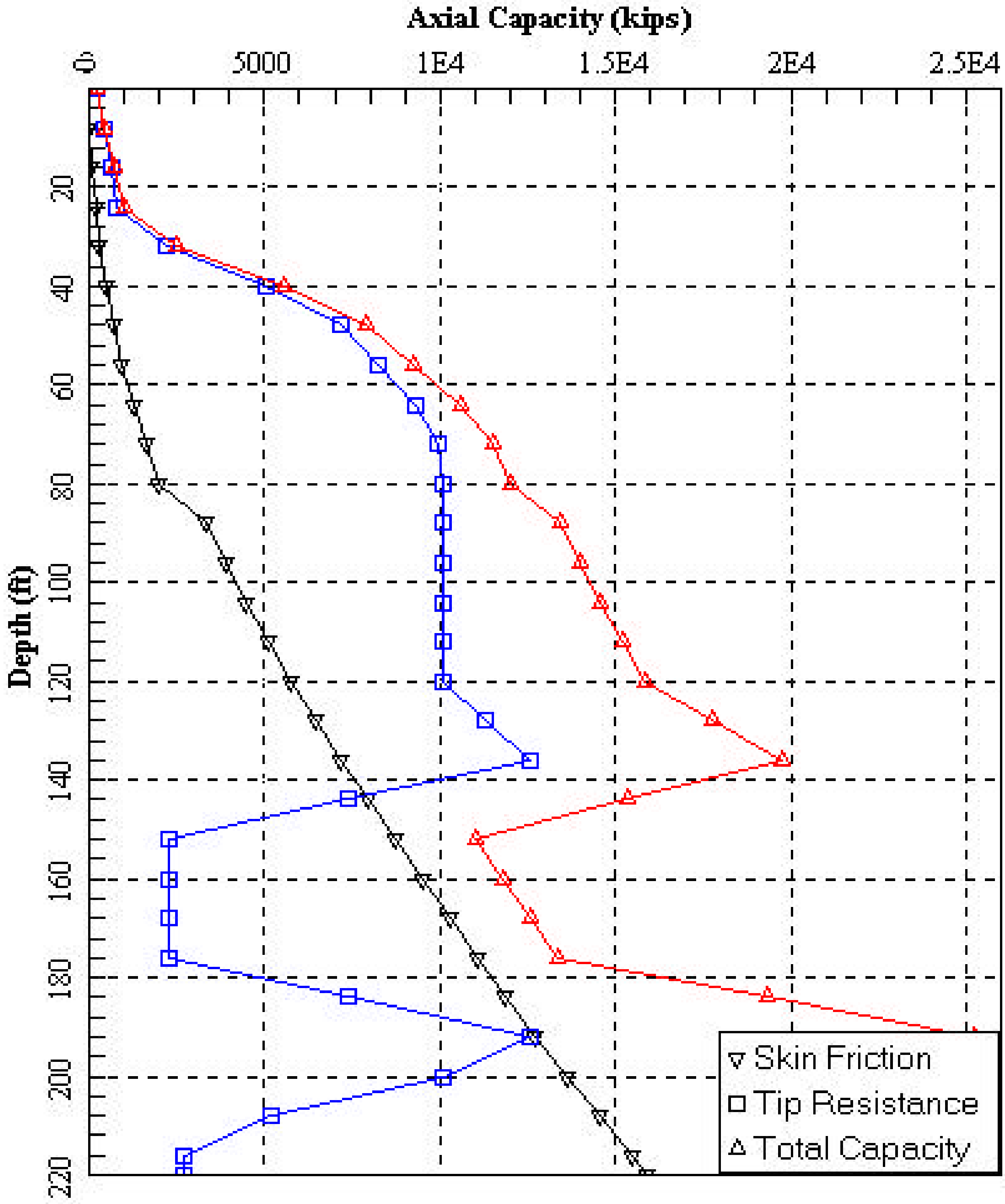
Knik Arm Bridge
Anchorage, Alaska

ULTIMATE CAPACITY
8 FT. DIAMETER PIPE PILE

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. G-8



ULTIMATE CAPACITY VS DEPTH

BORING A-10

ASSUMES LOOSE SOILS DRILLED OUT AND CONCRETE ADDED TO FORM PLUG AT TIP

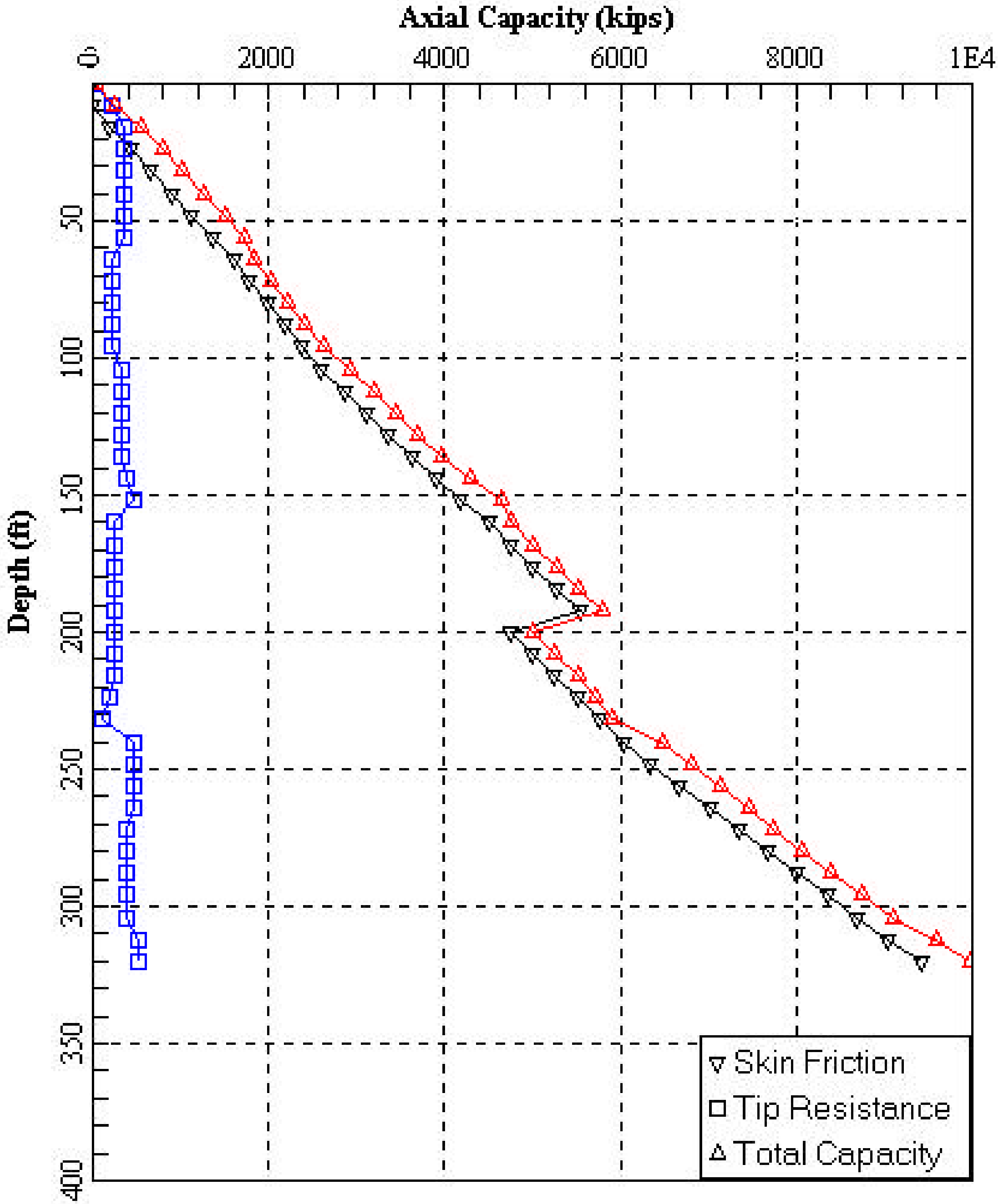
Knik Arm Bridge
Anchorage, Alaska

ULTIMATE CAPACITY
8 FT. DIAMETER PIPE PILE

February 2004 32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. G-9



ULTIMATE CAPACITY VS DEPTH

Knik Arm Bridge
Anchorage, Alaska

ULTIMATE CAPACITY
4 FT. DIAMETER PIPE PILE

February 2004

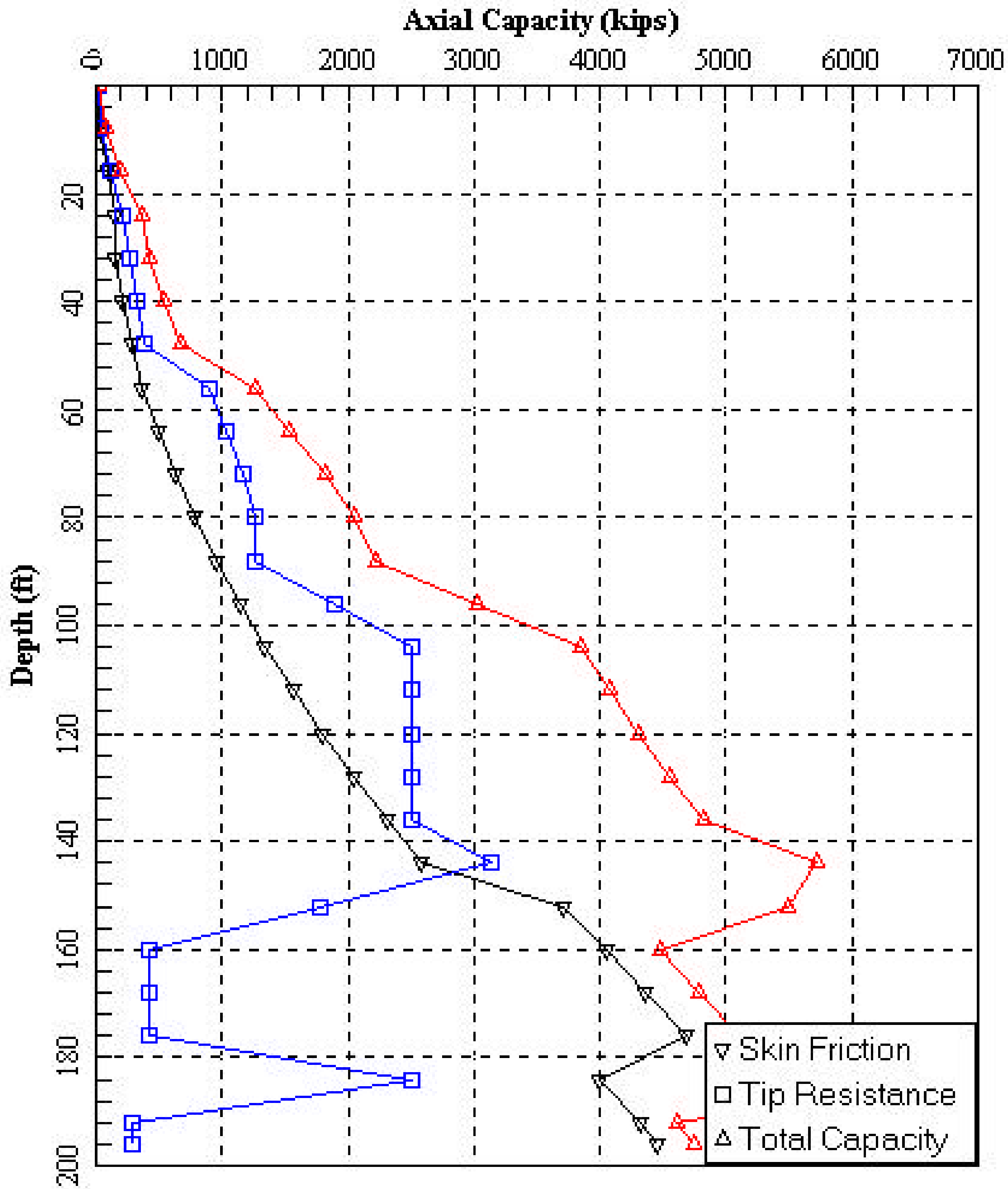
32-1-01536

BORING A-1

ASSUMES LOOSE SOILS DRILLED OUT AND CONCRETE ADDED TO FORM PLUG AT TIP

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. G-10



ULTIMATE CAPACITY VS DEPTH

BORING A-2

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge
Anchorage, Alaska

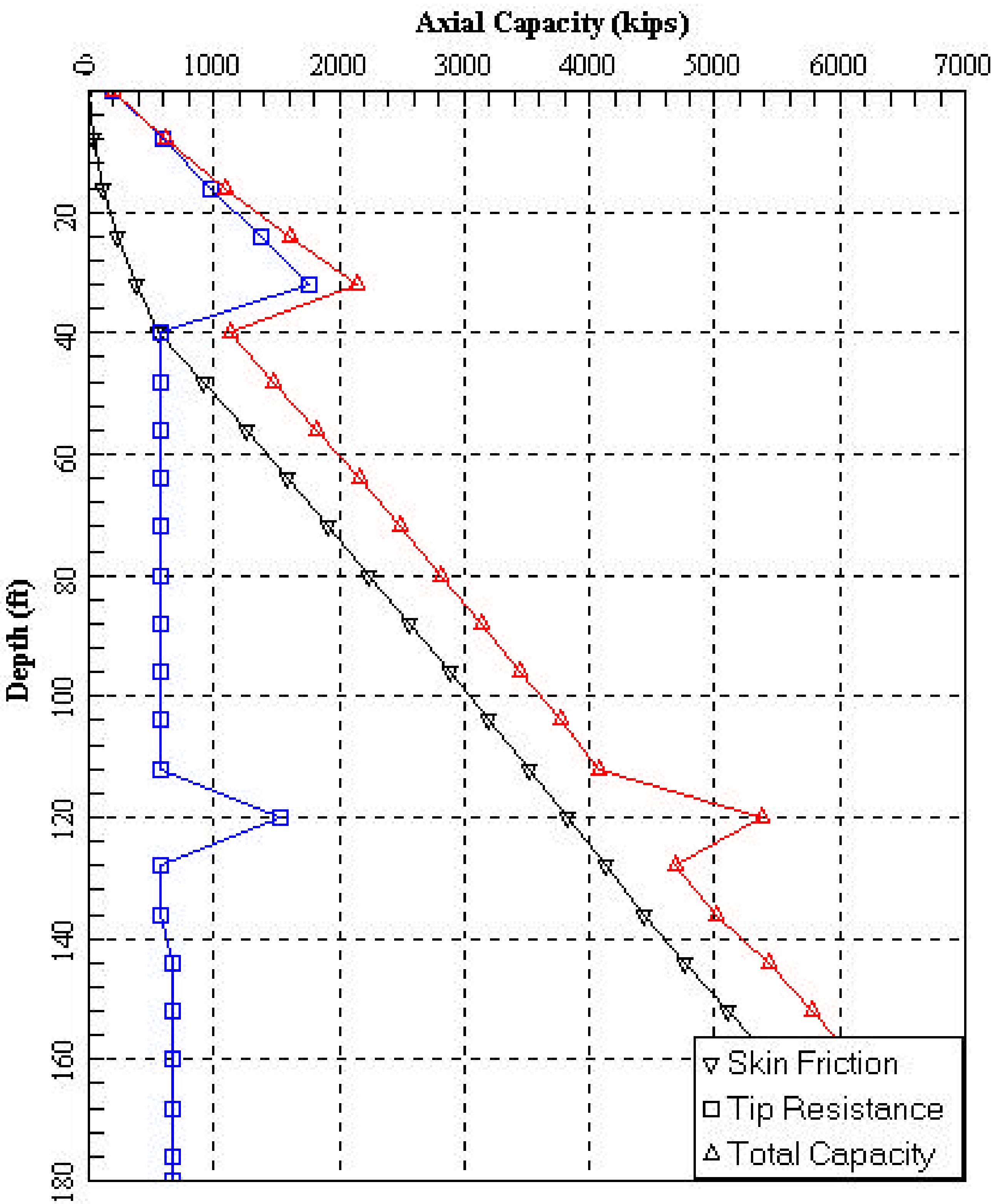
ULTIMATE CAPACITY
4 FT. DIAMETER PIPE PILE

February 2004

32-1-01536

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. G-11



ULTIMATE CAPACITY VS DEPTH

BORING A-4

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge
Anchorage, Alaska

ULTIMATE CAPACITY
4 FT. DIAMETER PIPE PILE

February 2004 32-1-01536

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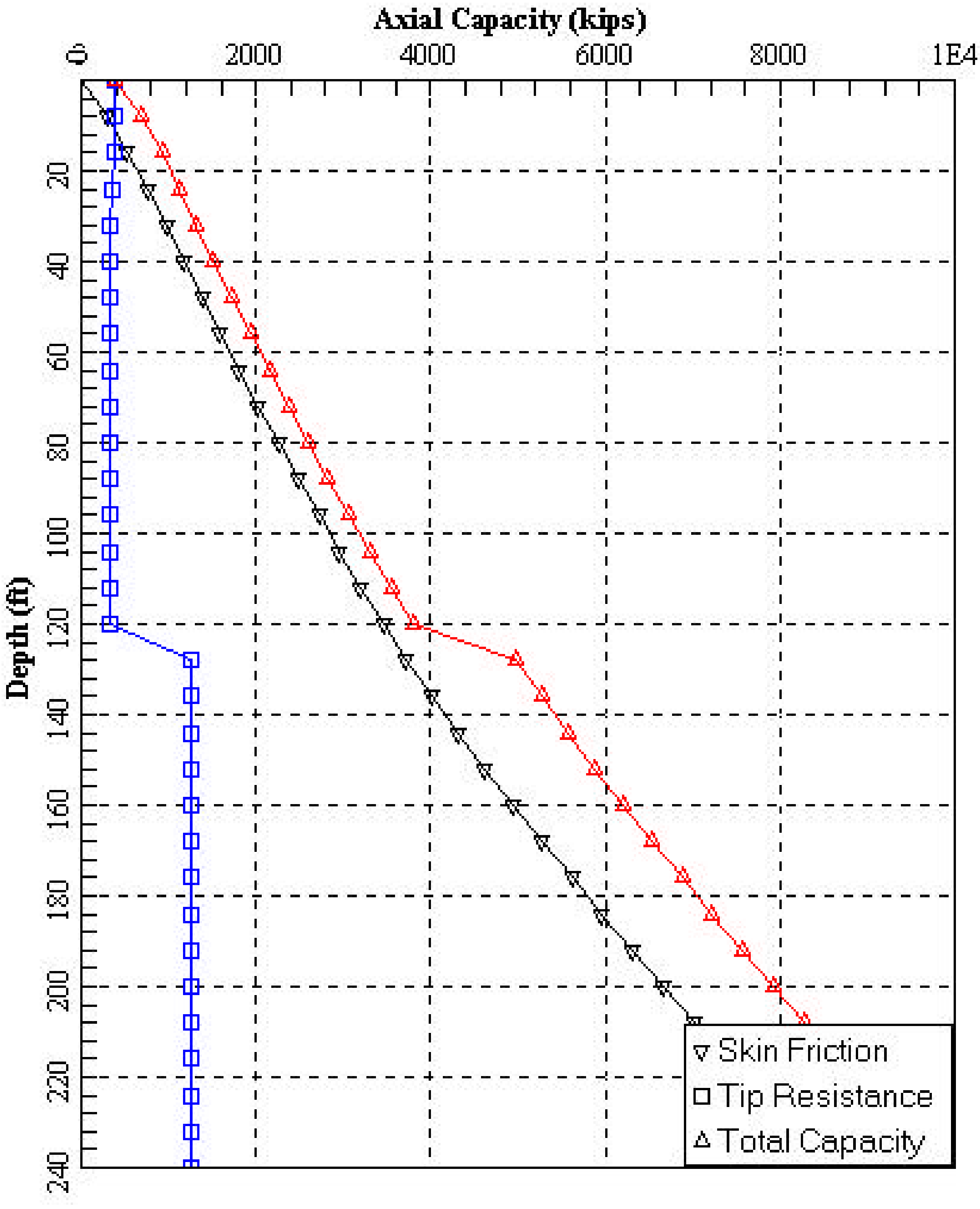
Geotechnical & Environmental Consultants

Fig. G-12



ULTIMATE CAPACITY VS DEPTH

BORING A-5

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge
Anchorage, Alaska

ULTIMATE CAPACITY
4 FT. DIAMETER PIPE PILE

February 2004 32-1-01536

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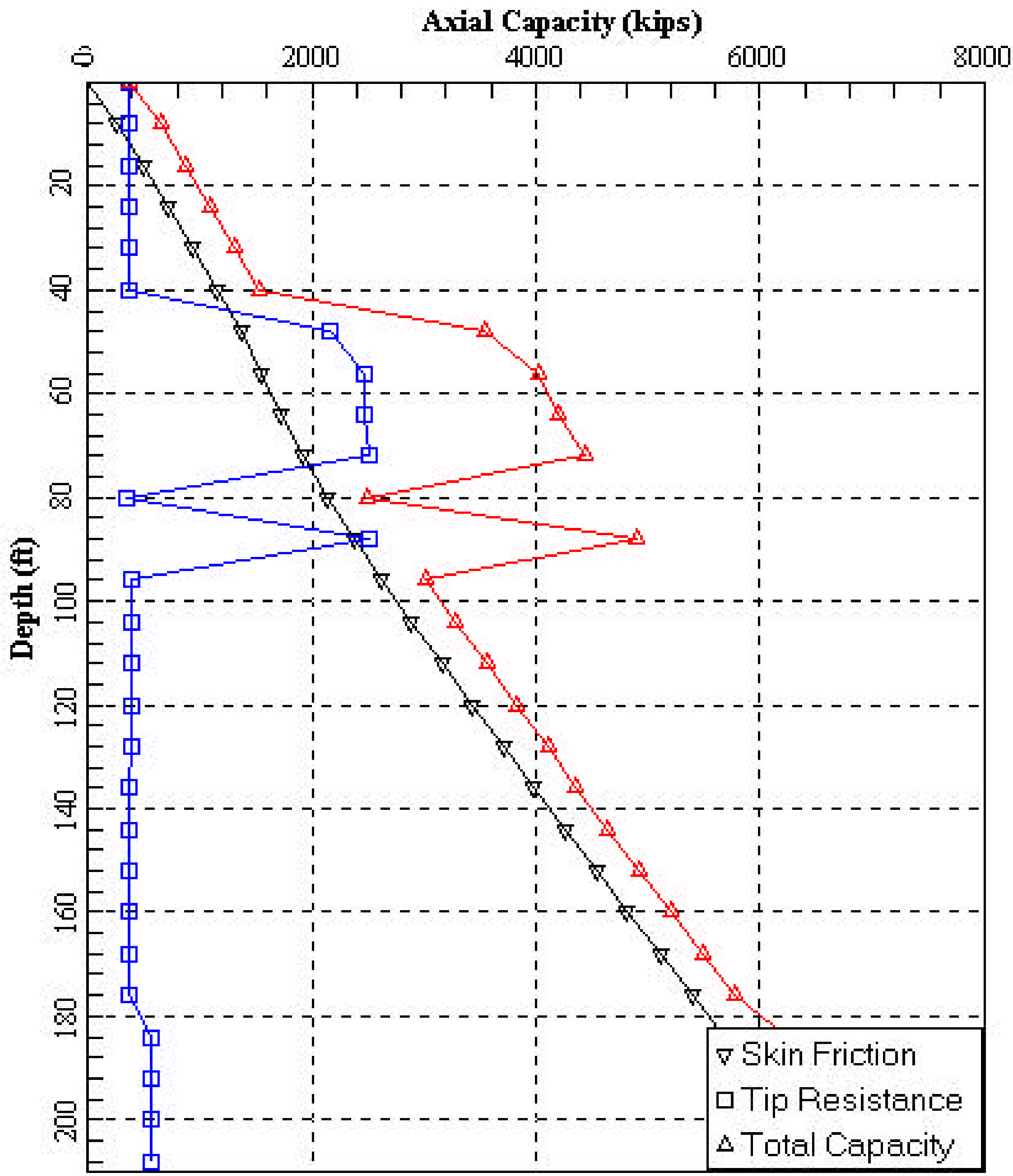
Geotechnical & Environmental Consultants

Fig. G-13



ULTIMATE CAPACITY VS DEPTH

BORING A-6

ASSUMES LOOSE SOILS DRILLED OUT AND CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge
Anchorage, Alaska

ULTIMATE CAPACITY
4 FT. DIAMETER PIPE PILE

February 2004 | 32-1-01536

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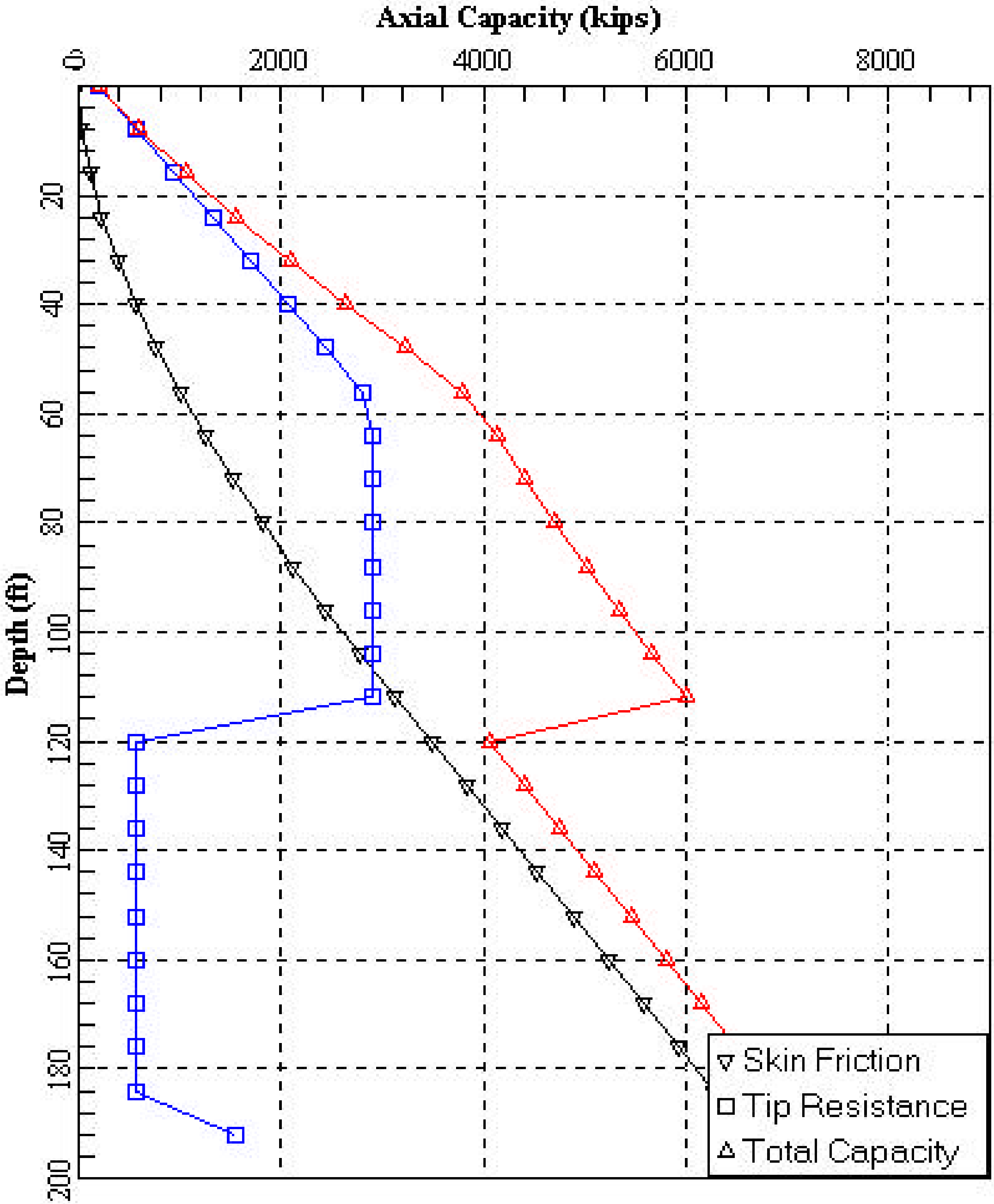
Geotechnical & Environmental Consultants

Fig. G-14



ULTIMATE CAPACITY VS DEPTH

BORING A-7

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge
Anchorage, Alaska

ULTIMATE CAPACITY
4 FT. DIAMETER PIPE PILE

February 2004 32-1-01536

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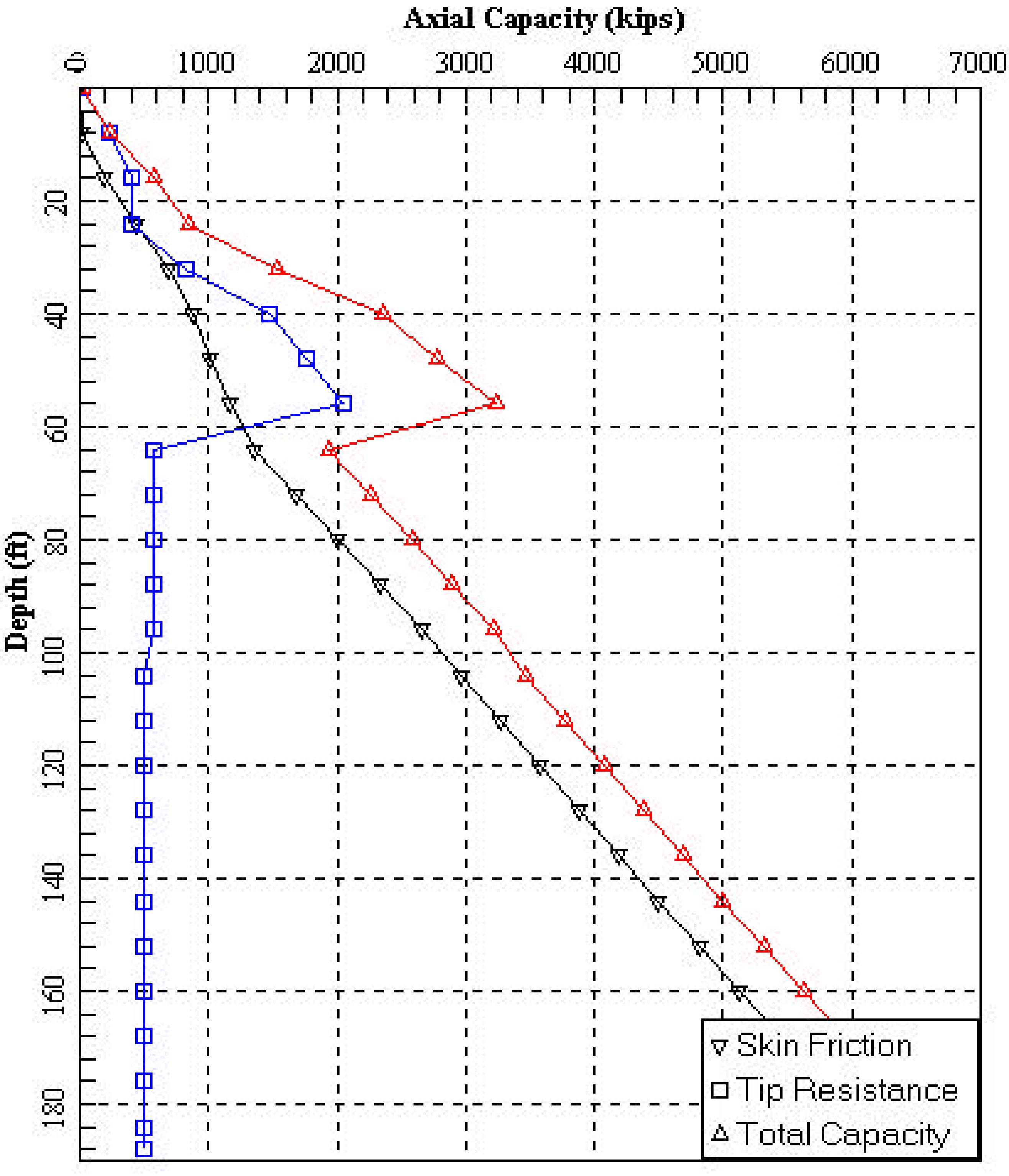
Geotechnical & Environmental Consultants

Fig. G-15



ULTIMATE CAPACITY VS DEPTH

Knik Arm Bridge
Anchorage, Alaska

BORING A-8

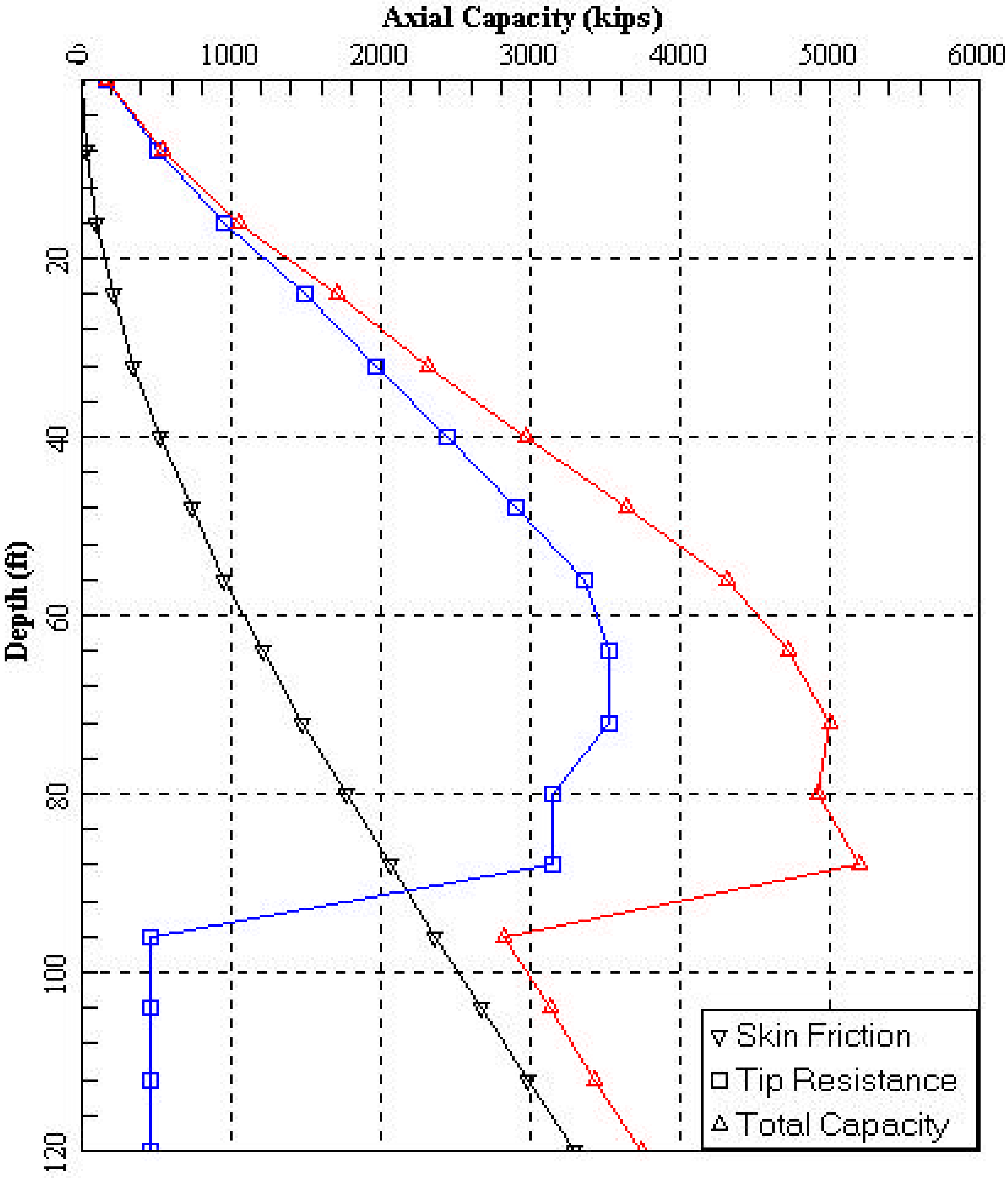
ULTIMATE CAPACITY
4 FT. DIAMETER PIPE PILE

February 2004 32-1-01536

ASSUMES LOOSE SOILS DRILLED OUT AND CONCRETE ADDED TO FORM PLUG AT TIP

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. G-16



ULTIMATE CAPACITY VS DEPTH

Knik Arm Bridge
Anchorage, Alaska

ULTIMATE CAPACITY
4 FT. DIAMETER PIPE PILE

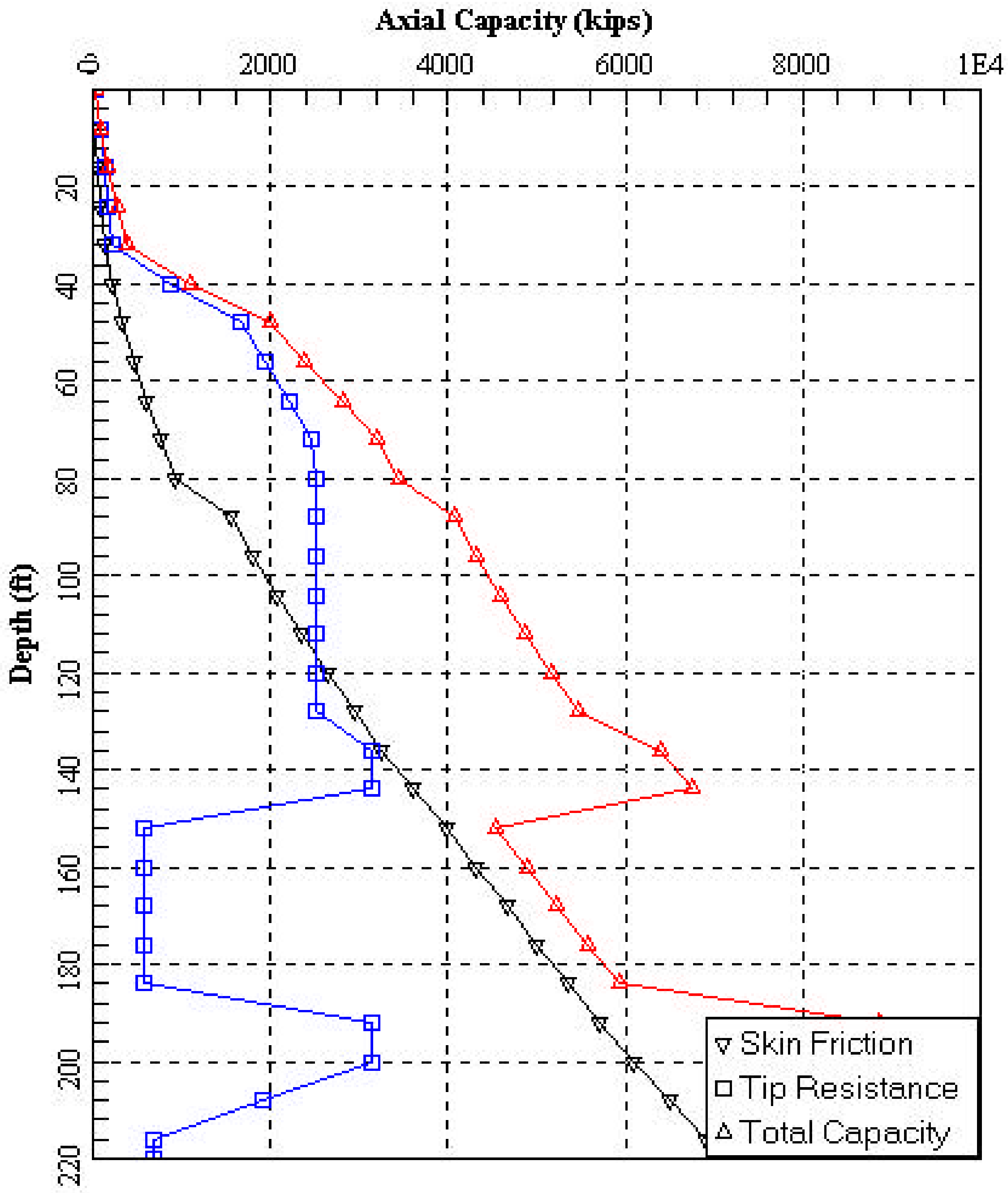
February 2004 32-1-01536

BORING A-9

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. G-17



ULTIMATE CAPACITY VS DEPTH

BORING A-10

ASSUMES LOOSE SOILS DRILLED OUT AND
CONCRETE ADDED TO FORM PLUG AT TIP

Knik Arm Bridge
Anchorage, Alaska

ULTIMATE CAPACITY
4 FT. DIAMETER PIPE PILE

February 2004 32-1-01536

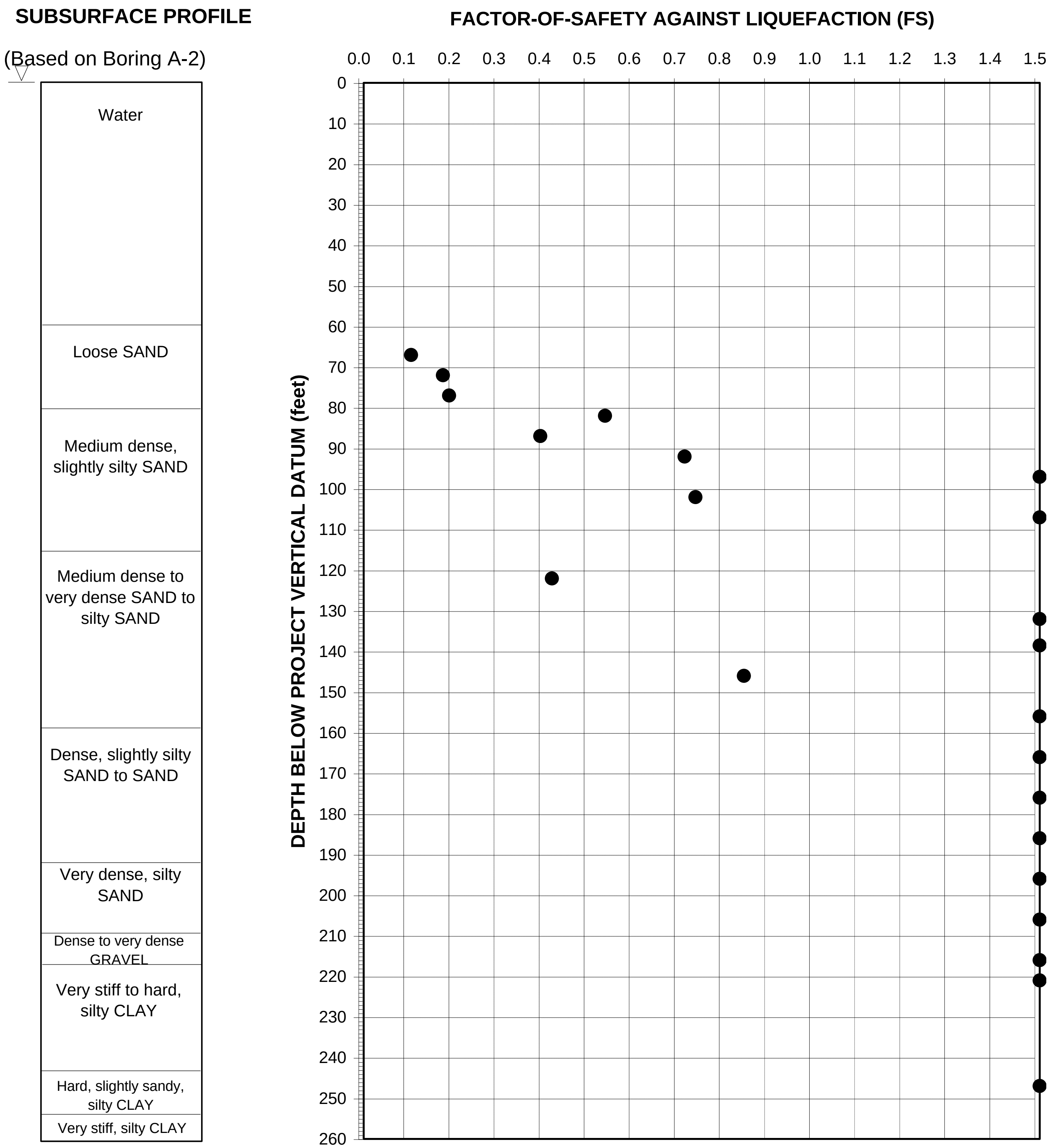
SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. G-18

APPENDIX H

LIQUEFACTION ANALYSES RESULTS

LIST OF FIGURES



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed for an earthquake with a magnitude of 7.5 and a peak ground acceleration of 0.36g.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

Knik Arm Bridge
Anchorage, Alaska

RESULTS OF
LIQUEFACTION ANALYSES
BORING A-2

February 2004 32-1-01536

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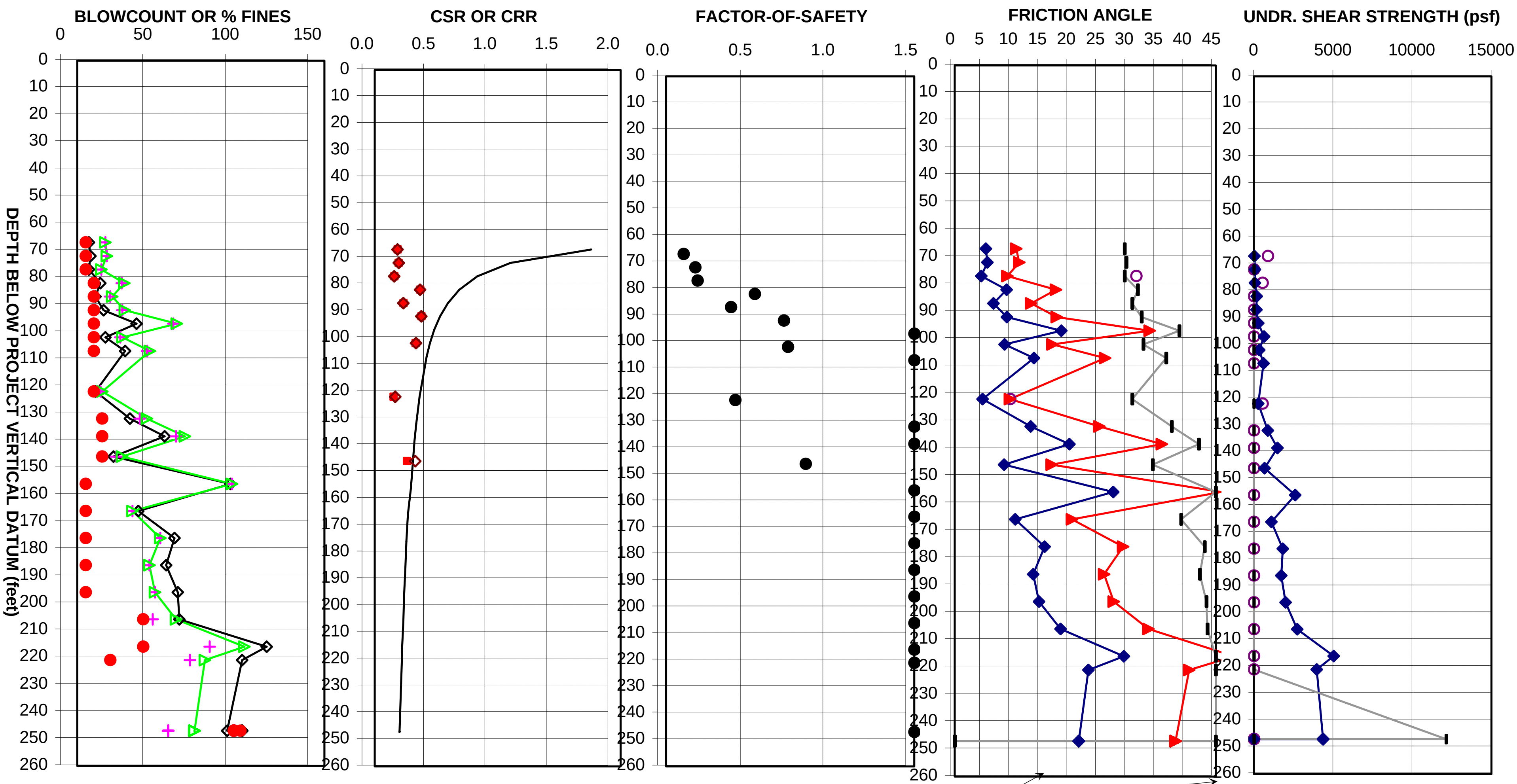
FIG. H-1



LEGEND:

- Estimated % Fines
- Uncorrected Blowcount, N
- Corrected Blowcount, $(N_1)_{60}$
- Clean Sand Equivalent Blowcount, $(N_1)_{60\text{-cs}}$

LEGEND:

- Cyclic Resistance Ratio
- Uncorrected Cyclic Stress Ratio
- Corrected Cyclic Stress Ratio

LEGEND:

- Residual Value (Upper Bound)
- Critical Value (Lower Bound)
- Yield Value
- Static Value

FIG. H-2

Knik Arm Bridge
Anchorage, Alaska

DETAILED RESULTS OF LIQUEFACTION ANALYSES BORING A-2

February 2004 32-1-01536

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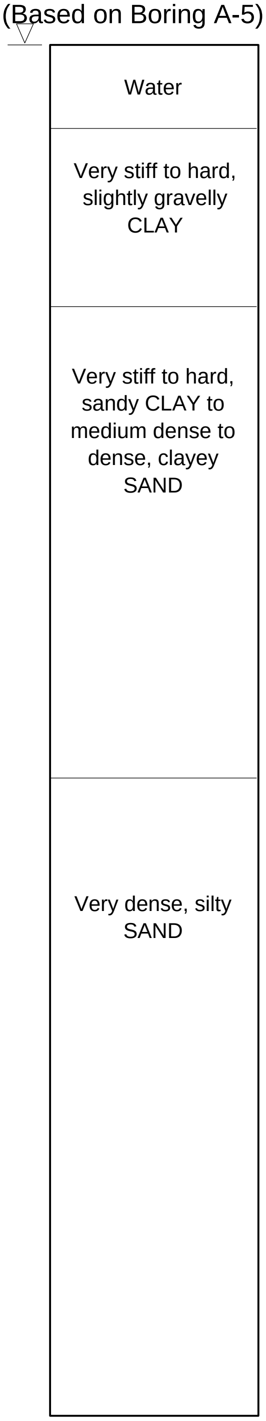
Geotechnical and Environmental Consultants

FIG. H-2

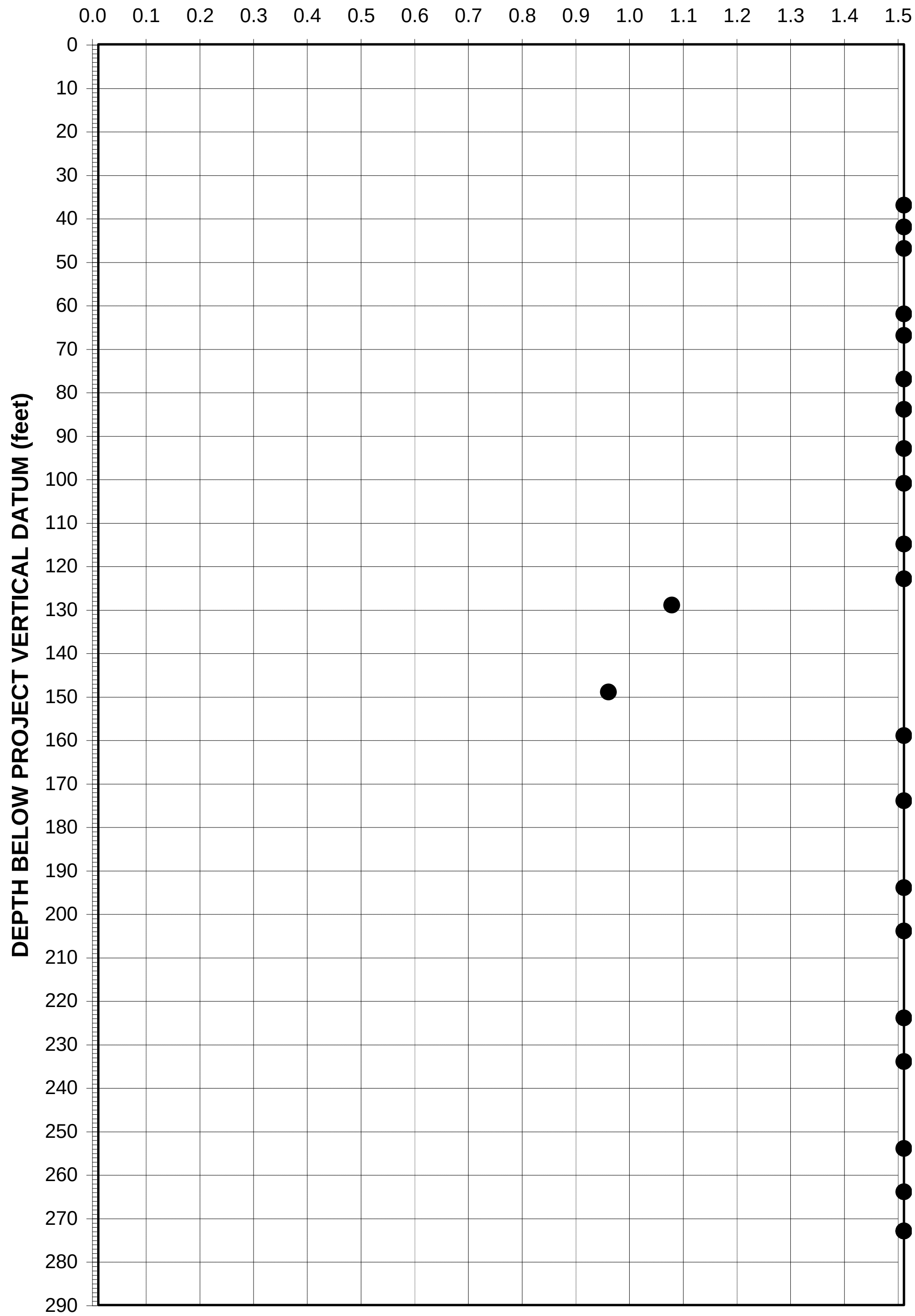
2/12/2004-Figure H-1 & H-2.xls-author

SUBSURFACE PROFILE

(Based on Boring A-5)



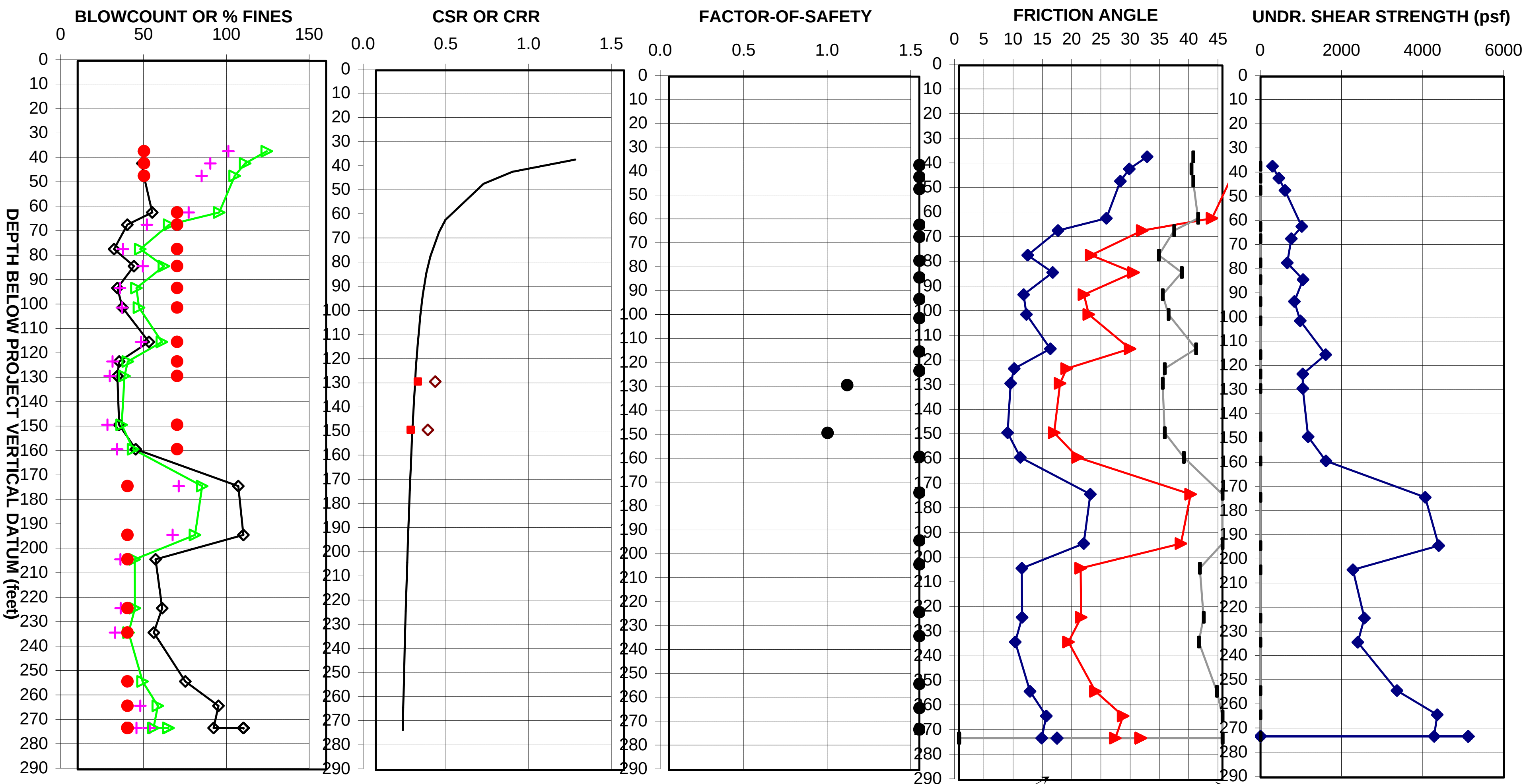
FACTOR-OF-SAFETY AGAINST LIQUEFACTION (FS)



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed for an earthquake with a magnitude of 7.5 and a peak ground acceleration of 0.36g.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

Knik Arm Bridge Anchorage, Alaska	
RESULTS OF LIQUEFACTION ANALYSES BORING A-5 February 2004 32-1-01536	
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. H-3**



LEGEND:

- Estimated % Fines
- Uncorrected Blowcount, N
- Corrected Blowcount, $(N_1)_{60}$
- Clean Sand Equivalent Blowcount, $(N_1)_{60\text{-cs}}$

LEGEND:

- Cyclic Resistance Ratio
- Uncorrected Cyclic Stress Ratio
- Corrected Cyclic Stress Ratio

LEGEND:

- Residual Value (Upper Bound)
- Critical Value (Lower Bound)
- Yield Value
- Static Value

FIG. H-4

Knik Arm Bridge
Anchorage, Alaska

DETAILED RESULTS OF LIQUEFACTION ANALYSES BORING A-5

February 2004 | 32-1-01536

SHANNON & WILSON, INC.
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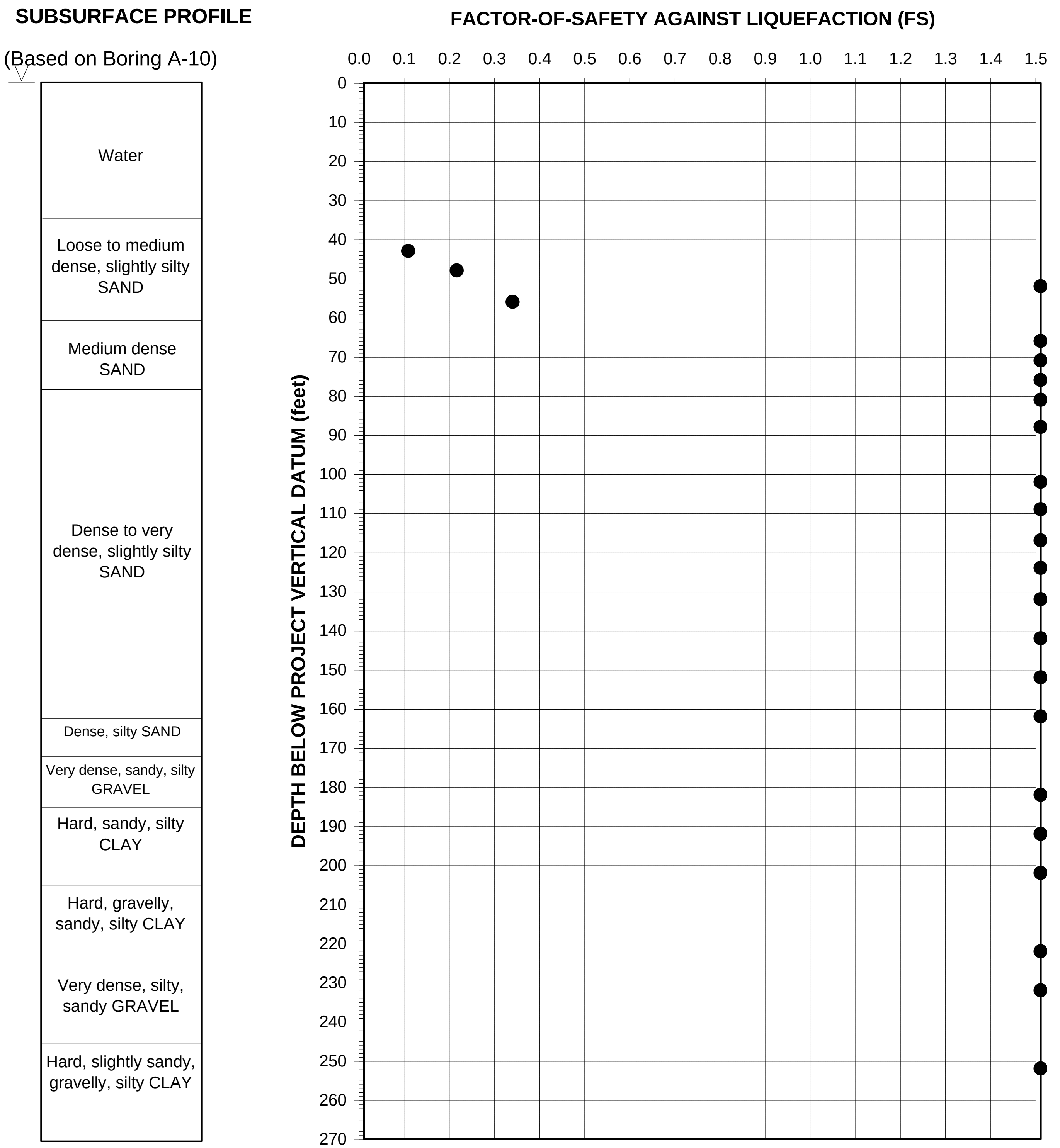
FIG. H-4

2/12/2004-Fig H-3&H-4.xls-author



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/ NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed for an earthquake with a magnitude of 7.5 and a peak ground acceleration of 0.36g.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

Knik Arm Bridge
Anchorage, Alaska

RESULTS OF LIQUEFACTION ANALYSES BORING A-10

February 2004 | 32-1-01536

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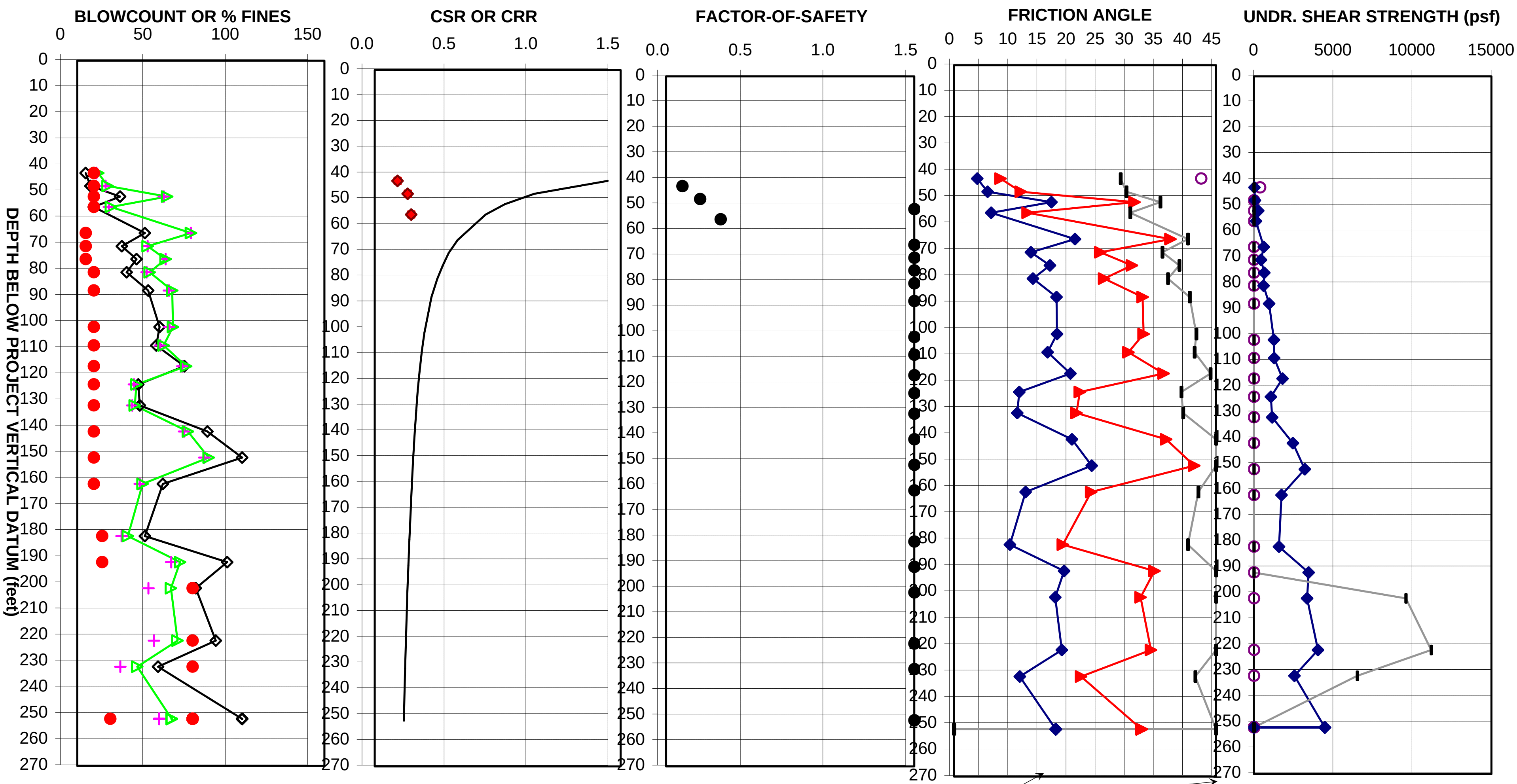
Geotechnical and Environmental Consultants

FIG. H-5



LEGEND:

Estimated % Fines
Uncorrected Blowcount, N
Corrected Blowcount, $(N_1)_{60}$
Clean Sand Equivalent Blowcount, $(N_1)_{60\text{-cs}}$

LEGEND:

Cyclic Resistance Ratio
Uncorrected Cyclic Stress Ratio
Corrected Cyclic Stress Ratio

LEGEND:

Residual Value (Upper Bound)
Critical Value (Lower Bound)
Yield Value
Static Value

Knik Arm Bridge
Anchorage, Alaska

DETAILED RESULTS OF LIQUEFACTION ANALYSES BORING A-10

February 2004 | 32-1-01536

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-6

FIG. H-6

APPENDIX I

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland