GEOTECHNICAL STUDY
MKT SPUR CONNECTOR AND BIKEWAY BRIDGE
WHITE OAK BAYOU
CSJ NO.: 0912-72-299
WBS NO.: N-320420-030A-3
HOUSTON, TEXAS

Prepared for

Klotz Associates, Inc.
Houston, Texas

November 2015

Prepared by

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November 23, 2015
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Attn: Mr. Brad Brown, P.E.

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Dear Mr. Brown:

We are pleased to submit our revised report covering the geotechnical study for the above referenced project. This report incorporates the comments made by City of Houston in a memo dated October 20, 2015. Should you have any questions or comments concerning the information contained herein, please contact us.

We appreciate the opportunity to serve you during this phase of the project, and look forward to continue our services during the construction phase and on future projects.

11/23/2015

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Sincerely,

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SUMMARY

The principal findings and conclusions of the geotechnical study for the proposed Missouri Kansas Texas (MKT) Spur Connector and Bikeway Bridge located along the northern bank of White Oak Bayou between Studemont Street and the MKT pedestrian bridge are summarized as follows:

The subsurface soil stratigraphy at the high bank of *hike and bike trail and retaining wall*, based on boring B-1, consists of cohesive fill to 10-ft depth, followed by soft to stiff consistency, medium plasticity cohesive sandy lean clays (CL) and to a lesser extent silt with sand (ML), silty clay (CL-ML) to the 28-ft explored depth, where boring was terminated due to refusal on an unknown object. At the maintenance shelf, based on boring B-1A, the subsurface soil stratigraphy consists of cohesive fill to 16-ft depth, underlain by soft to stiff, but mostly stiff consistency, high plasticity cohesive fat clays (CH) to 45-ft depth, followed by compact semi-cohesive clayey sands (SC) to the 50-ft boring termination depth.

The subsurface soil stratigraphy at the *bridge over ditch* (Borings B-2 and B-3), consists of intermixed layers of cohesive/semi-cohesionless/cohesionless fill to 10-ft to 12-ft depth, underlain by very loose to slightly compact, cohesionless/semi-cohesionless/semi-cohesive silty sands (SM), silty clayey sands (SC-SM), silt (ML) to 21-ft and 31-ft depths, followed by very soft to very stiff, but mostly stiff consistency, medium to high` plasticity cohesive lean clays (CL), lean clays with sand (CL), sandy lean clays (CL), fat clays (CH), fat clays with sand (CH) to the 46-ft depth, and slightly compact to compact, cohesionless/semi-cohesive silty sands (SM), silty clayey sands (SC-SM) from 46-ft to the 50-ft termination depth.

The subsurface soil stratigraphy within the *pavement area at Studemont Street connector*, inferred from boring B-4, consists of slightly compact, cohesionless silty sand “FILL” to 11-ft depth, followed by very soft consistency, low plasticity cohesive sandy lean clay “FILL” to the 15-ft termination depth.

Groundwater conditions observed in the project borings and in the two piezometers are tabulated within Section *6.4 Groundwater Conditions* of this report.

An analysis of historic topographic maps and a geologic map did not identify any surface faulting within the project site.

The proposed bridge structure over drainage ditch may be supported on drilled, cast-in-place concrete shafts. The allowable skin friction curves for drilled shafts are presented in Appendix C. The allowable end bearing values for drilled shafts with diameters greater than 24-in. are presented in the report text.

We also evaluated the foundation design for the proposed bridge structure using driven piles. The allowable skin friction curves for driven piles are presented in Appendix D.

We performed the stability analyses to evaluate the slope embankment under the bridge of tributary with side slope declivity of 3H:1V. The results of the tributary channel slope stability analyses for short term, intermediate term (rapid drawdown), and long term conditions are presented in the report text and on Figures 4 through 6.

We understand that embankments may be constructed at bridge abutment areas to connect the hike and bike trail. If cut and fill is required for grade adjustment, the fill may be TxDOT Type "C" soils. Stripping and proof-rolling should be performed according to TxDOT specifications. Type "C" fill should be free of organics, deleterious materials, and excessive silt. It is Houston District practice to limit the liquid limit to 65 for Type "C" materials.

Recommended subgrade preparation and design thickness for the walkway rigid pavement are included in the report text.

1. INTRODUCTION

1.1 General

Klotz Associates, Inc. (KAI) retained Tolunay-Wong Engineers, Inc. (TWEI) to perform a geotechnical study for the proposed MKT Spur Connector and Bikeway Bridge located along the northern bank of White Oak Bayou between Studemont Street and the MKT pedestrian bridge in Houston, Texas (Key Map 493 E). The study was performed in general accordance with TWEI Proposal No. P14-G121.rev-2, dated September 3, 2014. A subcontract for consultant service agreement between Klotz Associates, Inc. and Tolunay-Wong Engineers, Inc. was signed on May 8, 2015.

1.2 Project Information

The project consists of the construction of a bikeway bridge over ditch, new walkway trail and concrete retaining wall along the northern bank of White Oak Bayou in Houston, Texas. The bikeway bridge will span an outfall ditch approximately midway through the project alignment. The tributary channel depth is about 20-ft from top-of-safety shelf near abutment and channel bottom, and the bridge foundation is expected to be drilled piers. The adjoining White Oak Bayou Channel is about 35-ft deep from top-of-high bank and channel flow line, and the proposed trail beyond the bridge structure will be a rigid pavement. Our review of plan and profile of MKT Trail Spur connector indicates that the concrete block retaining wall will be about 2-ft to 10-ft high and will be placed along walkway route near MKT bridge. The bayou near MKT bridge is about 43-ft to 45-ft deep. Ms. Jennifer A. Black, P.E. of KAI provided project details including site drawings and profiles via e-mail transmittals on May 9 and 22, 2014. We received updated drawings for retaining walls via e-mail transmittal from Ms. Black on September 2, 2014. Mr. Brad Brown, P.E. of KAI indicated that this report will be reviewed by US Army Corps of Engineers (USACE) via e-mail transmittal on the same day. The general layout of the project site and boring locations are shown on Figure 1.

This report provides recommendations for the bridge structure and walkway trail pavement. The retaining wall design and detailed slope stability analysis of the White Oak Bayou embankments and the embankment under the new bridge segment is provided in an addendum report.

2. SCOPE OF STUDY

The objectives of this study were to explore soil and groundwater conditions within the project site to formulate geotechnical design recommendations and construction guidelines for the referenced project in accordance with the Texas Department of Transportation (TxDOT) Geotechnical Manual. The scope of the geotechnical study included the following work tasks:

- Drilling of five (5) soil borings within the project site to evaluate the subsurface conditions. Boring B-1 was planned to be drilled to 75-ft for the trail, but due to refusal at the 28-ft depth, an additional boring (B-1A) was drilled to 50-ft depth, southeast of B-1. Borings B-2 and B-3 were drilled to 50-ft depth for the bridge at the abutment area, on alternate sides of the bridge structure. Borings B-5 and B-6 were planned to be drilled for the bridge but due to inaccessibility, they were cancelled. Boring B-4 was drilled to 15-ft depth for the pavement at Studemont connector.

- Installing two (2) piezometers in borings B-1A and B-3 upon completion of sampling to evaluate the groundwater condition at the project site.

- Performing laboratory tests on selected soil samples recovered from the borings to evaluate physical and engineering properties of the subsoil.

- Preparing project soil boring logs by the TxDOT WinCore program and gINT.

- Conducting engineering analyses to develop geotechnical design recommendations and construction guidelines for the pavement, foundations for the bikeway bridge, concrete block retaining wall, global stability analyses for the side slopes of White Oak Bayou along the trail, and the embankment under the new bridge segment.

Environmental assessments, recommendations for areas not covered by the boring layout, and a site-specific fault study were outside the scope of work for this study.

3. FIELD EXPLORATION

3.1 Test Borings

The field exploration was performed on June 26 and 30, and July 8 through 10, 2015. We drilled five (5) soil borings to depths ranging from 15-ft to 50-ft to evaluate subsurface soil conditions. Drilling and sampling were performed by Triangle Resources, Inc. TWEI representatives coordinated the field activities, logged the boreholes, and obtained water level readings during drilling, at the end of the day after drilling completion and the next day. Borings B-3 and B-1A were converted into respective piezometers PZ-1 and PZ-2 after completion of drilling. The approximate boring and piezometer locations are shown on Figure 1.

Boring B-1 was offset to B-1A due to refusal at the 28-ft depth on an unknown object. Borings B-5 and B-6 could not be drilled due to access problems.

3.2 Drilling Methods

Soil samples from the project borings were typically obtained at 2-ft continual intervals to 10-ft depth, from 11-ft to 15-ft explored depth in boring B-4, and to the 20-ft depth in the remaining borings, continuing at 5-ft intervals thereafter to the termination depths of 28-ft and 50-ft in accordance with either ASTM D 1587 or D 1586. During sampling, Texas Cone Penetrometer (TCP) Tests (Tex-132-E) were performed at 5-ft intervals to completion depths of the borings.

3.3 Soil Sampling

Cohesive soil and soil interpreted to be cohesive during drilling were sampled in the test borings by hydraulically pushing a 3-inch-diameter, thin-walled tube a distance of about 24 inches. The field sampling procedure was conducted in accordance with the *Standard Practice for Thin-Walled Tube Sampling of Soils* (ASTM D 1587). The field technician extruded the soil samples in the field, visually classified the recovered soils, and obtained penetration resistance measurement of the cohesive soils using a calibrated pocket penetrometer. The penetrometer readings are presented on the boring logs in Appendix A. Based on experience with local soils, a factor of 0.67 was applied to penetrometer reading to estimate soil consistency. Representative portions of the soil samples extruded in the field were placed into secure containers, protected from disturbance, and transported to the laboratory.

Cohesionless/semi-cohesionless to semi-cohesive soil and soil interpreted to be granular during drilling were sampled by driving a 2-inch diameter, split barrel sampler. The sampler was driven 18 inches by a 140-pound hammer falling about 30 inches in general accordance with the *Standard Method for Penetration Test and Split-Barrel Sampling of Soils* (ASTM D 1586). The field technician recorded the number of blows required to drive the sampler through three consecutive 6-inch sampling intervals. The sum of the blows required to penetrate the final 12 inches is the Standard Penetration Test (SPT) “N” value. Each sample obtained from the split-

barrel was visually classified and placed in a plastic bag for transport to the laboratory. The compactness of the cohesionless/semi-cohesive sands and the consistency of semi-cohesive to cohesive samples recovered with the SPT sampler were inferred from the "N_{60}" value. N_{60} is the SPT "N" value, corrected for field procedure to an average energy ratio of 60% (Terzaghi, Peck, and Mesri, 1996). The SPT data are presented on the boring logs in Appendix A.

Cohesive and cohesionless/semi-cohesionless to semi-cohesive soil in the project borings were tested at 5-ft intervals by the Texas Cone Penetrometer Test (Tex-132-E). The TxDOT cone test is used to determine the relative density or consistency of a soil material. The test consists of driving a 3-inch diameter cone with a 170-pound hammer, which is dropped from a height of 2 feet. The cone is seated and driven to 12 blows or 6 inches whichever comes first. Then it is driven for two consecutive 6-inch increments, and the blow counts for each increment are recorded. In hard materials, the cone is driven with the resulting penetration in inches recorded for 50 blows. If the first penetration is less than or equal to six inches, then the cone is driven again for either another 50 blows or six inches whichever comes first. The number of blows for each 6-inch increment, and/or the amount of penetration for each 50 blows is presented on the boring logs included in Appendix A.

3.4 Boring Logs

Our interpretations of general soil and water-level conditions at the boring locations are included on the boring logs presented in Appendix A. The soil classifications are based on the Unified Soil Classification System (ASTM D 2487). A key to the terms and symbols used on the boring logs is included in Appendix A. The open boreholes were backfilled with soil cuttings after the last groundwater level observation.

3.5 Water-Level Measurements

The project borings were initially dry augered to free water depth, to depth where the borehole cannot be maintained open, or to where the soil is resistant to further drilling using the dry auger method, then continued with the wet rotary method to boring termination depths, to evaluate the groundwater condition. The groundwater was observed during drilling, 10-minutes after encountering free water, and the next day after completion of drilling. Borings B-3 and B-1A were converted into respective piezometers PZ-1 and PZ-2, after completion of drilling. The piezometer installation reports are presented in Figures 2 and 3.

4. LABORATORY TESTING

Selected soil mechanics laboratory tests were performed on soil samples recovered from the borings to measure their physical and engineering properties. The laboratory program consisted of moisture content, plasticity characteristics, amount of material in soils finer than the No. 200 sieve, particle size analysis, unconfined compression and unconsolidated-undrained triaxial compression tests. A brief description of each test is given below.

- ***Laboratory Determination of Water (Moisture) Content of Soil and Rock – Tex-103-E (similar to ASTM D 2216).*** The water content of a material, expressed as a percentage, is defined as the ratio of the mass of pore or free water in a given mass of material to the mass of the solid material particles. Moisture content can provide an indication of cohesive soil shear strength and compressibility when compared to Atterberg limits.

- ***Liquid Limit, Plastic Limit, and Plasticity Index of Soils – Tex-104-E and Tex-105-E (similar to ASTM D 4318).*** These soil properties are used to distinguish different soil types and provide an evaluation of volume change potential when considered in conjunction with natural moisture content. The liquid limit and plastic limit of soils, referred to as the Atterberg limits, distinguish the boundaries of the several consistency states of plastic soils. The plasticity index of the soil is the difference between the liquid limit and the plastic limit.

- ***Amount of Material in Soils Finer than No. 200 (75-µm) Sieve – Tex-111-E (similar to ASTM D 1140).*** This test determines the total amount of material in soils finer than the No. 200 sieve. The test result is presented as the percent of silt and clay sizes by weight in the sample and can provide an indication of the soil permeability.

- ***Particle Size Analysis of Soils – Tex-110-E (similar to ASTM D 422).*** This method quantitavely determines the distribution of particle sizes larger than 75-µm (retained on the No.200 sieve) by sieving. The distribution of particle sizes smaller than 75-µm can be determined by a sedimentation process, using a hydrometer.

- ***Unconfined Compressive Strength of Cohesive Soil - ASTM D 2166.*** This test determines the unconfined compressive strength of cohesive soil in the undisturbed, remolded or compacted condition, using strain-controlled application of the axial load. The unconfined compressive strength of a cohesive soil sample is twice its undrained shear strength.

- ***Unconsolidated-Undrained Triaxial Compression – Tex-118-E (similar to ASTM D 2850).*** This compression test consists of placing a relatively undisturbed cylindrical specimen in a pressurized triaxial chamber and then loading it to failure, without allowing drainage, using strain-controlled application of the axial load. The compressive strength of a cohesive soil sample is twice its undrained shear strength.

- ***Density of Soil - ASTM D 7263, Modified B -*** Total unit weight of the soil aggregate is defined as the weight of the aggregate (soil plus water) per unit volume. Knowing the total unit weight and moisture content, dry unit weight can be computed. Dry unit weight can be used to roughly estimate the compactness of the semi-cohesive [clayey sand (SC)] and cohesionless [silty sand (SM] soils.

The test results are presented on the boring logs in Appendix A. Selected soil particle size analysis test results are presented in Appendix B.

5. GENERAL GEOLOGY AND CURSORY FAULT REVIEW

5.1 General Geology

The site is situated in the Gulf Coastal Plain Physiographic region of Texas. The Gulf Coastal Plain region contains sediments that have been deposited by fluvial and deltaic processes. The sediments are represented as a series of wedges thickening towards the coast. The sediments of the Brazoria County area were deposited during the Quaternary Period of the Cenozoic Era. The Quaternary Period is divided into two geologic epochs, the Holocene and the Pleistocene.

The Pleistocene Epoch includes the sedimentary deposits of the Beaumont, Lissie, and Willis Formations. The Beaumont is the youngest, followed by the Lissie and the Willis. In the past, the Lissie was divided into the Montgomery and the Bentley Formations. In modern nomenclature, the Lissie is considered to be one formation. The sediments that compose each of these geologic formations are similar in depositional composition in that streams of coalescing fluvial and deltaic systems deposited them. The soils, which developed from these systems, include clay, silt, sand, and their intermixtures.

According to the Geologic Atlas of Texas, Houston Sheet, 1982, the site is located entirely on a surface outcrop of the Beaumont Formation. The Beaumont Formation was deposited during the Pleistocene Epoch. The formation is mostly stream channel; point bar, natural levee, back swamp, and to a lesser extent coastal marsh and mud-flat deposits. The geologic units, which formed these deposits, include interdistributary muds, abandoned channel fill muds, and overbank fluvial muds. Surface soils that comprise the Beaumont Formation at the site consist of, dominantly clay, silt, sand and their intermixtures.

5.2 Geologic Fault Review

A cursory study of geologic faulting in the general site area was performed using available in-house and public records to identify any evidence of surface faults within the project site. Our records indicate that the Pecore Fault which is an up the coast surface fault trending in a northeast to southwest direction and dipping to the northwest is located approximately 3/8 of a mile north of the site. The Pierce Junction Salt Dome is approximately 7 miles south of the site. Aerial photographs can reveal photo linear or geomorphic features, which can be indicative of surface fault activity. A photo linear feature could represent the surface expression of a surface fault line. A photo linear feature can be formed as a result of moisture differences from one side of the surface fault to the other. We reviewed aerial photographs from 1943, 1952, 1977, 1988, 1995, 2001, 2003, 2004, 2005, 2006, 2008, 2010, 2011, 2012, 2013, 2014, and 2015 and did not identify photographic linear features in the site area. A detailed Phase I Fault Study was beyond the scope of this study.

6. SUBSURFACE CONDITIONS

6.1 General

Our interpretations of soil and groundwater conditions at the site are based on information obtained at the soil boring locations only. The project boring logs are presented in Appendix A. This information has been used as the basis for our conclusions and recommendations. Subsurface conditions may vary between soil boring locations. Significant variations at areas not explored by the project borings will require re-evaluation of our recommendations.

6.2 Soil Stratigraphy

This section presents generalized observed soil stratigraphy at the project site as characterized by the project borings.

6.2.1 Hike and Bike Trail Retaining Wall (B-1 and B-1A)

The subsurface soil stratigraphy at the high bank of *hike and bike trail retaining wall,* based on boring B-1, consists of cohesive fill to 10-ft depth, followed by soft to stiff consistency, medium plasticity cohesive sandy lean clays (CL) and to a lesser extent silt with sand (ML), silty clays (CL-ML) to the 28-ft explored depth, where boring was terminated due to refusal on an unknown object. At the maintenance shelf, based on boring B-1A, the subsurface soil stratigraphy consists of cohesive fill to 16-ft depth, underlain by soft to stiff, but mostly stiff consistency, high plasticity cohesive fat clays (CH) to 45-ft depth, followed by compact semi-cohesive clayey sands (SC) to the 50-ft boring termination depth.

6.2.2 Bridge over Ditch (B-2 and B-3)

The subsurface soil stratigraphy at the *bridge over ditch* (Borings B-2 and B-3), consists of intermixed layers of cohesive/semi-cohesionless/cohesionless fill to 10-ft to 12-ft depth, underlain by very loose to slightly compact, cohesionless/semi-cohesionless/semi-cohesive silty sands (SM), silty clayey sands (SC-SM), silt (ML) to 21-ft and 31-ft depths, followed by very soft to very stiff, but mostly stiff consistency, medium to high` plasticity cohesive lean clays (CL), lean clays with sand (CL), sandy lean clays (CL), fat clays (CH), fat clays with sand (CH) to the 46-ft depth, and slightly compact to compact, cohesionless/semi-cohesive silty sands (SM), silty clayey sands (SC-SM) from 46-ft to the 50-ft termination depth.

6.2.3 Pavement at Studemont Connector (B-4)

The subsurface soil stratigraphy within the *pavement area at Studemont Street connector*, inferred from boring B-4, consists of slightly compact, cohesionless silty sand “FILL” to 11-ft depth, followed by very soft consistency, low plasticity cohesive sandy lean clay “FILL” to the 15-ft termination depth.

The fill comprised of cohesive sandy lean clays, lean clays with sand, and cohesionless silty sands. It should be noted that it is relatively difficult to accurately delineate fill from similar natural soils. Fill classifications are made based upon visual observations and require considerable judgment. The interpreted fill depths may vary somewhat from the actual conditions.

Ferrous and calcareous nodules, sand and silt seams/pockets/partings, organics, wood pieces, roots, claystones, and slickensided substructures were observed within the matrix of cohesive soils. Detailed descriptions of the soils are presented on the boring logs in Appendix A.

6.3 Soil Properties

6.3.1 Hike and Trail Retaining Wall (B-1 and B-1A)

Lean/Fat Clay (CL/CH) We measured liquid limits ranging from 28% to 45%, with corresponding plasticity indices ranging from 12 to 28, on six (6) selected lean clay samples recovered from various depths in borings B-1 and B-1A. The median values of liquid limit and plasticity index are 38% and 21, respectively. In situ moisture contents of the samples were one percentage point dry to seven percentage points wet of their corresponding plastic limit. The tested samples and one additional sample yielded fines contents ranging from 53% to 75%.

Liquid limits ranging from 54% to 67%, with corresponding plasticity indices ranging from 33 to 45, were measured on four (4) selected fat clay samples recovered from various depths in boring B-1A. In situ moisture contents of the samples were one percentage point dry to five percentage points wet of their corresponding plastic limit. The samples yielded fines contents ranging from 97% to 100%.

We measured deviator stresses ranging from 11.6 psi to 37.5 psi on selected five (5) cohesive samples recovered from various depths in the project borings during unconsolidated-undrained triaxial (UU) and unconfined compression (UC) testing. Total unit weights of the tested samples and additional four (4) samples ranged from 120 pcf to 135 pcf. TCP values ranging from 4 to 34 blows per foot were registered within the cohesive strata. Based on the measured undrained strengths and the TxDOT cone data, the cohesive soils are inferred to have very soft to very stiff, but mostly stiff to very stiff consistencies.

Sand/Silt (SC/ML) TCP value of 50 blows per foot was measured within semi-cohesive clayey sands at the 48.5-ft depth in boring B-1A. The TCP blow counts indicate a compact consistency. Liquid limit of 36%, plasticity index of 20, and fines content of 47%, were measured on the semi-cohesive clayey sand sample recovered from the 46-ft to 48-ft depth range in boring B-1A. Total unit weight of the sample was 131 pcf.

Measured fines content of the semi-cohesionless sample recovered from the 10.5-ft to 12-ft depth range in boring B-1 was 74%.

6.3.2 Bridge over Ditch (B-2 and B-3)

Lean/Fat Clay (CL/CH) We measured liquid limits ranging from 27% to 45%, with corresponding plasticity indices ranging from 12 to 28, on five (5) selected lean clay samples recovered from various depths in borings B-2 and B-3. In situ moisture contents of the samples were between one percentage point less and ten percentage points wet of their corresponding plastic limit. The samples yielded fines contents ranging from 61% to 98%.

Liquid limits ranging from 60% to 70%, with corresponding plasticity indices ranging from 38 to 46, were measured on five (5) selected fat clay samples recovered from various depths in borings B-2 and B-3. Four of the samples had in situ moisture contents between two and eight percentage points wet of their corresponding plastic limit. The five tested samples yielded fines contents ranging from 81% to 100%.

We measured deviator stresses ranging from 36.0 psi to 48.6 psi on selected four (4) cohesive samples recovered from various depths in borings B-2 and B-3 during unconsolidated-undrained triaxial (UU) and unconfined compression (UC) testing. Two (2) fat clay samples, recovered from 26-ft to 28-ft and 36-ft to 38-ft depths in boring B-2, failed along slickensided surfaces during testing. Total unit weights of the tested samples and three (3) additional samples ranged from 120 pcf to 135 pcf. TCP values ranging from 5 to 40 blows per foot were registered within the cohesive strata in the project borings. Based on the measured undrained shear strengths and the TxDOT cone data, the cohesive soils are inferred to have very soft to very stiff, but mostly stiff to very stiff consistencies.

Sand/Silt (SC-SM/SM/ML) TCP values ranging from 2 to 47 blows per foot were measured within cohesionless/semi-cohesionless strata at various depths in borings B-2 and B-3. The TCP blow counts indicate very loose to compact consistency. Ten (10) selected cohesionless/semi-cohesionless silty sand, silty clayey sand, sandy silt, and silt samples yielded fines contents ranging from 32% to 86%.

Total unit weight of the silty sand sample recovered from the 46-ft to 48-ft depth range in boring B-2 was 126 pcf.

6.3.3 Pavement at Studemont Connector (B-4)

Lean Clay "FILL" Liquid limit of 27%, plasticity index of 11, and fines content of 55%, were measured on the lean clay sample recovered from the 11.5-ft to 13-ft depth range in boring B-4. In situ moisture content of the sample was four percentage points wet of its plastic limit. TCP value of 3 blows per foot was registered within this stratum. Based on the TxDOT cone data, the cohesive soil is inferred to have a very soft consistency.

Sand "FILL" TCP values of 21 and 37 blows per foot were measured within the cohesionless stratum at respective depths of 9-ft and 5-ft in boring B-4. The TCP blow counts indicate a slightly compact consistency. Three (3) selected cohesionless samples yielded fines contents of 32% to 49%.

6.4 Groundwater Conditions

Groundwater level measurements taken during drilling and the groundwater observations in the piezometers are summarized in the following Table 6-1.

Table 6-1
Groundwater Level Measurements

Boring/ Piezometer No.	Boring/ Piezometer Depth (ft)	Depth of Dry Augering (ft)	Depth to Free Water During Drilling (ft)	Groundwater Observations		
				Depth to Groundwater (ft)	Caved Depth (ft)	Time Lapse
B-1	28	16	Cave @ 14	-	-	-
B-1A/PZ-2	50/50	8	Dry @ 8	*	-	-
				30.0 27.8 28.2 28.3	-	7/10/2015 7/16/2015 7/27/2015 8/14/2015
B-2	50	18	Cave @ 16	-	-	-
B-3/PZ-1	50/48.5	12	Dry @ 8	Dry**	8	10 minutes
				Dry Dry 42.6 42.6	-	7/6/2015 7/16/2015 7/27/2015 8/14/2015
B-4	15	15	13	4.0	8.1	17 minutes

(*) Boring B-1A was converted to PVC standpipe piezometer PZ-2 after completion of sampling.
(**) Boring B-3 was converted to PVC standpipe piezometer PZ-1 three days after completion of sampling.

Water level measured in an open borehole may not accurately reflect the true (static) groundwater condition since it may be influenced by drill water, wall instability, etc., and may be time dependent. A more accurate determination of the static groundwater level is made in open standpipe piezometers. Piezometer installation was included in the work scope of this project. Borings B-1A and B-3 were converted into respective piezometers PZ-2 and PZ-1 after completion of drilling. The piezometer installation reports are presented in Figures 2 and 3.

6.5 Potentially Petroleum Contaminated Area

We did not encounter any unusual staining or hydrocarbon odors in the project borings during drilling.

7. BRIDGE FOUNDATION DESIGN RECOMMENDATIONS

It is planned to construct a bridge structure over a ditch along hike and bike trail at the north bank of White Oak Bayou in Houston, Texas. We understand that the bridge over drainage ditch will be about 120-ft long. The proposed bridge structure may be supported on drilled, cast-in-place concrete shafts or pre-cast, pre-stressed driven piles. Borings B-2 and B-3 were drilled within the bridge area at the drainage ditch. Recommendations regarding drilled shaft and driven pile design and construction based on borings B-2 and B-3 are given in the following sections.

7.1 Analysis Criteria

Drilled shaft and driven pile capacities were calculated using the procedures described in the TxDOT Geotechnical Manual dated August 2006. Drilled shafts may be designed for both skin friction and end bearing. Driven piles end bearing should be neglected and only skin friction is considered.

The design capacities are based on having a complete soil cover around the full length of the shaft/pile. We recommend discounting the frictional resistance of the soils in the upper 10-ft depth for the bridge at the drainage ditch for drilled shafts/piles to account for possible disturbance during construction, scour, and possible shrinkage of the cohesive soils with variations in moisture.

For the bridge over ditch, the frictional resistance of the soils should be discounted in the upper 10-ft depth for drilled shafts/piles at the abutment areas to account for possible disturbance during construction, scour, and possible shrinkage of the cohesive soils with variations in moisture. For interior bents, if used, the frictional resistance should be discounted deeper to account for possible changes of modified channel cross-sections, disturbance during construction, scour, and shrinkage of the cohesive soils with variations in moisture. We will be glad to provide additional scour depth information for interior bents, if drawings are provided.

It is Houston District practice to disregard end bearing for drilled shafts less than or equal to 24 inches in diameter. Allowable unit end bearing for drilled shafts is assumed to be a maximum of 2 tsf for shaft diameters between 24-in. and 48-in. End bearing for drilled shaft diameter sizes greater than 48 inches may be computed as described in the TxDOT Geotechnical Manual. Allowable end bearing for drilled shaft diameter sizes greater than 48 inches is limited to 5.3 tsf due to restriction placed on TCP values when computing allowable end bearing capacity. End bearing capacity should not be considered in driven pile design. The following Table 7-1 summarizes the recommended allowable end bearing values for drilled shaft foundations for the proposed bridge structure over the drainage ditch:

Table 7-1
Allowable Drilled Shaft End Bearing Values for Design of Bridge over Ditch

Boring No.	Depth (ft)	Drilled Shaft Diameter Sizes Between 24 and 48 inches (*)	Drilled Shaft Diameter Sizes Greater than 48 inches (**)
B-2	0 - 10	Ignored	Ignored
	10 - 15	0.2 tsf	0.2 tsf
	15 - 20	0.4 tsf	0.4 tsf
	20 - 25	0.25 tsf	0.25 tsf
	25 - 30	0.85 tsf	0.85 tsf
	30 - 35	1.15 tsf	1.15 tsf
	35 - 40	1.3 tsf	1.3 tsf
	40 - 45	2.0 tsf	2.45 tsf
	45 - 50	1.65 tsf	1.65 tsf
B-3	0 - 10	Ignored	Ignored
	10 - 15	0.1 tsf	0.1 tsf
	15 - 20	0.2 tsf	0.2 tsf
	20 - 25	0.4 tsf	0.4 tsf
	25 - 30	1.2 tsf	1.2 tsf
	30 - 35	1.45 tsf	1.45 tsf
	35 - 40	1.0 tsf	1.0 tsf
	40 - 45	2.0 tsf	2.0 tsf
	45 - 50	2.0 tsf	2.5 tsf

(*) End bearing is limited to 2 tsf for drilled shaft diameter sizes between 24 and 48 inches.
(**) End bearing is limited to 5.3 tsf for drilled shaft diameter greater than 48 inches.

The accumulative allowable static frictional resistance capacity plots for the proposed bridge, presented in the TxDOT developed WinCore version 3.1 format similar to TxDOT Form 1190 for drilled shaft and driven pile foundation design, are presented in Appendices C and D. The capacities are for gravity loads on individual foundation units with a minimum center-to-center spacing of two and one half (2.5) shaft diameters. The soil design parameters were developed based on laboratory undrained triaxial shear strength test data in clay soils and the TxDOT cone penetrometer blow count data in sandy soils. A soil reduction factor of 0.7 was used to obtain the skin friction curves for the drilled shafts. For driven piles, unit skin friction value was limited to 1.25 tsf in accordance with TxDOT Houston District practice. For drilled shafts, unit skin friction value was limited to 0.70 x 1.25 = 0.88 tsf in accordance with TxDOT Houston District practice.

The aspect ratio of a drilled shaft, or its length divided by its diameter (L/D), should not exceed about 30 (O'Neill and Reese, 1999, pg 11). If allowable end bearing values are added to the allowable friction values to size a drilled shaft then the maximum drilled shaft embedment depth should be limited to within three drilled shaft diameters above the maximum depth of the borings.

Uplift capacity for drilled shafts and driven piles may be estimated by applying a 0.67 reduction factor to the cumulative axial compressive friction capacity curves presented in Appendices C and D, and adding the buoyant weight of the embedded portion of the drilled shaft/driven pile.

7.2 Lateral Load Analyses

It is necessary to design the foundation unit to resist both vertical and lateral loads. The foundations will be subjected to lateral loading from wind forces and other sources. The lateral forces generated from those sources will be taken by mobilization of resistance in the surrounding soils as the unit deflects, and by the structural capacity of the foundation section. We will provide lateral load capacities in an addendum report to aid the design, if the lateral loads for the proposed bridge structure is of concern.

7.3 Axial Group Effects

The overall allowable axial compression load carrying capacity of a group of drilled shafts/piles, in some cases, could be less than the sum of the individual allowable capacities. A reduction in the individual shaft/pier capacity, to allow for group effects, is usually not necessary for shafts/piers having a center-to-center spacing of 2.5 or more shaft diameters. The reduction in individual capacity depends on several factors including number of shafts in the group, shaft sizes, shaft penetration, shaft spacing, etc. We will be glad to provide axial group effect recommendation to aid the design, when the number of shafts in the group, shaft sizes, shaft penetration, and shaft spacing are selected.

7.4 Foundation Settlement

Post-construction vertical movements due to design loads are anticipated to be less than 0.5 in. for drilled shafts/piles installed using proper construction techniques. Movements consist of elastic shortening of the foundation unit and deformation at the foundation tip.

7.5 Drilled Shaft Construction Recommendations

Drilled shaft construction and installation should follow TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, June 2004 Edition, Item 416, *Drilled Shaft Foundations*. The ACI 336.1-89 *Standard Specifications for End Bearing Drilled Piers* document can be used in the absence of TxDOT Specifications. The TxDOT Specifications shall always supercede any other specification. Presented below are a few specific recommendations.

1) Drilled shaft excavations should be inspected for verticality and side sloughing. Verticality is specified at one inch in ten feet of the shaft length, and should be checked to the full depth of dry augering prior to introducing drilling mud.

2) Slurry should contain four to eight percent by weight of bentonite additive and should satisfy the slurry specification set forth in TxDOT Standard Specification Item 416.3 or ACI 336.1-89 Section 2.3.5.2.e. Note that the ACI requirements are more stringent than TxDOT Standard Specification Item 416.3. Stricter slurry specifications are required to assure suspension of detritus from drilling operations, and to assure adequate cleaning of the slurry prior to concreting. Cleaning of the slurry is important to prevent deposition of detritus on reinforcement cages and ensure that inclusions of detritus will not be formed within the concrete mass.

3) Before placing concrete, the shaft bottoms should be cleaned out with a drilling bucket in order to remove any sediments which may not be displaced by the concrete. The shaft bottoms should be cleaned with a "clean-out" bucket until rotation on the bottom without crowd (i.e. penetration under force) produces little spoil. Probing after clean-out is essential to verify the condition of the base of the shaft.

4) Concrete should conform to the requirements of TxDOT Standard Specification Item 421, (*Portland Cement Concrete*), or ACI 336.1-89 Section 2.3.5.5.

5) Concrete placement should be accomplished as directed in TxDOT Standard Specification Item 416.3.F. The tremie pipe diameter should be at least eight times as large as the largest concrete aggregate size.

6) A computation of the final concrete volume for each shaft should be made. Shafts taking an unreasonably high or low volume of concrete should be cored to check their integrity.

7) If casing is used it should be extracted slowly and smoothly with a vibratory hammer. The casing should always remain at least one foot below the level of the concrete during placement. Our analyses assume no casing will be left in place. We should be informed if casing would be left in place so we may provide revised shaft capacities.

Shaft excavations should not be made within two and one half (2.5) shaft diameters (edge to edge) of the shafts which have been concreted within the last 24 hours.

If it is deemed necessary to verify the design charts provided in Appendix C a pile load test may be performed on the selected drilled shaft type prior to the start of construction. The shaft subjected to the load test should be drilled to the design tip elevation. The pile load test should be performed in accordance with TxDOT Standard Specification Item 404.3.G and ASTM D 1143.

7.6 Driven Pile Construction Recommendations

Precast pre-stressed driven pile construction and installation should follow TxDOT Standard Specification Items 404 and 409, TxDOT Standard Specifications for Construction of Highways,

Streets and Bridges, dated June 1, 2004 and the AASHTO "Standard Specifications for Highway Bridges", 17th edition, 2002.

If it is deemed necessary to verify the design charts provided in Appendix D, then prior to the start of the construction a pile load test may be performed on the selected pile type. The pile subjected to the load test should be driven to the design pile tip elevation using construction equipment which will be employed during pile installation operations. The pile load test should be performed in accordance with TxDOT Standard Specification Item 404.3.G and ASTM D 1143.

The following Table 7-2 summarizes AASHTO ("Standard Specifications for Highway Bridges", 17th edition, 2002) recommended design and driving stresses for axially loaded reinforced concrete piles in terms of the concrete compressive strength, f'_c and the yield strength of the reinforcing steel, f_y. The recommended maximum allowable design stress is limited to $0.33f'_c$ on the gross cross sectional area of the concrete. The concrete must have a minimum 28 day compressive strength of 5,000 psi.

AASHTO specifications limit the maximum allowable compression driving stress to $0.85f'_c$ and the maximum tension driving stress to $0.70f_y$. Control of driving stresses is particularly important when driving reinforced concrete piles at high driving stress levels while penetrating through dense soil layers because of possible development of tensile stresses.

Table 7-2
Maximum Allowable Stresses for Conventionally Reinforced Concrete Piles

Type of Stress	AASHTO (2002) Recommendations [(1)]
Design Stresses	0.33 f'_c (on gross concrete area) f'_c minimum 5,000 psi
Driving Stresses	Compression Limit < 0.85 f'_c Tension Limit < 0.70 f_y (of steel reinforcement)

(1) Reference from the AASHTO 2002 "Standard Specifications for Highway Bridges" 17th edition, page 74, Section 4.5.11 Entitled "Maximum Allowable Driving Stresses".

Piles driven to below about El. -25.6 ft to -26.6 ft at the bridge over ditch could encounter dense sands. The piles can be stopped short of the penetrations shown on the pile capacity charts if driving refusal occurs. Typically refusal could occur after penetration of 2 to 3 ft into the thicker dense sand layers. The driving refusal criteria should be established based on the pile type and size, the design loads, and the pile hammer type, efficiency and cushion materials. The lateral and tension load capacities of the piles should also be checked.

Piles driven with impact hammers require an adequate helmet or drive head to distribute the hammer blow to the pile head. The helmet shall be axially aligned with the hammer and the pile. The helmet shall be guided by the leads and not be free-swinging. The helmet shall fit around the pile head in such a manner so as to prevent transfer of torsional forces during driving, while maintaining proper alignment of hammer and pile.

The heads of concrete piles shall be protected by pile cushion. Pile cushions shall be made of plywood, hardwood, or composite plywood and hardwood materials. The minimum pile cushion thickness placed on the pile head prior to driving shall not be less than 4-inch for short pile (50 feet or less) and at least 6-inch thick for longer piles. A new pile cushion shall be provided for each pile. In addition, the pile cushion shall be replaced if during the driving of any pile the cushion is compressed more than one-half the original thickness, or it begins to burn. The pile cushion dimensions shall match the cross sectional area of the pile. The use of manufactured pile cushion materials in lieu of a woodpile cushion shall be evaluated on a case-by-case basis.

Piles shall be supported in line and positioned with leads while being driven. Pile driver leads shall be constructed in a manner that affords freedom of movement of the hammer while maintaining alignment of the hammer and the pile to assure a concentric impact for each blow. Leads may be either fixed or swinging type. Swinging leads, when used, shall be fitted with a pile gate at the bottom of the leads and, in the case of batter piles, a horizontal brace may be required between the crane and the leads. The pile section being driven shall not extend above the leads. The leads shall be adequately embedded in the ground or the pile constrained in a structural frame such as a template to maintain alignment. The leads shall be of sufficient length to make the use of a follower unnecessary, and shall be so designed as to permit proper alignment of batter piles.

A drivability analysis should be conducted for the driven piles based on the final design loads, pile details, the actual hammer/cap block/cushion system and hammer efficiency.

Important concerns include monitoring the location and alignment of driven piles, confirming the penetration below cut-off elevation, and verifying that pile hammers are operating at expected efficiencies. Several practical considerations of importance for driven piles are presented below.

The driving penetration resistance of each pile should be recorded. At the contractor's option, controlled pre-drilling could be considered. The maximum diameter of the pilot hole permitted will be approximately four (4) inches less than the diagonal of square piles, and one (1) inch less than the diameter of round piles. The piles shall be driven below the depth of the pilot hole a minimum of one (1) foot or 100 blows, but not less than the bearing resistance shown on the plan. In any case, piles shall not be driven beyond the point where the penetration per blow is less than 0.1 inch as determined by an average of 10 blows. Piles that are stopped short of the design penetration should be checked for tension and lateral capacities. Refusal criteria should be based on the hammer, cap and cushion being used, the design load, and possible damage to piles during driving. Driving refusal criteria should be confirmed prior to construction based on specific hammers and driving methods proposed by the contractor.

Pre-cast concrete piles installed in closely spaced groups should be checked for lateral and vertical heave. If the piles have been displaced, they should be re-tapped to their original tip elevation. Piles driven in groups should begin with the center piles and proceed outward to minimize driving difficulties.

7.7 Scour

A detailed scour analysis was not within the scope of work for this project. Scour is due to three mechanisms. The first is contraction, which is a general scour and considers the total area of the bridge, the second is local scour and considers scour around the drilled shafts/piles, and the third is scour due to debris floating in the channel.

A hydrologist expert in scour analysis should evaluate the three types of scour. All three types of scour should be considered in the foundation design of the bridge.

Local scour for cohesionless soils may be estimated using the HEC 18 computer program which requires input such as the soil grain size, D_{50}, for the soils located along the channel bottom and banks, channel cross-section, channel flow rates, etc. and requires a hydrologist experienced in processing and interpreting data produced from the HEC 18 program which is beyond our expertise and work scope. The following Table 7-3 summarizes the estimated D_{50} from selected Particle Size Analysis test results provided in Appendix B.

Table 7-3
Summary of D_{50} Test Results

Boring	Depth (ft)	Approx. Elevation (ft)	Material Description	USCS	D_{50} (mm)
B-1	2-4	44.5 to 42.5	Lean Clay w/ Sand	CL	D_{50} = 0.0279
B-2	2-4	17.4 to 15.4	Sandy Lean Clay	CL	D_{50} = 0.0469
B-2	26-28	-6.6 to (-8.6)	Fat Clay	CH	D_{50} = 0.0
B-3	41-43	-20.6 to (-22.6)	Lean Clay w/ Sand	CL	D_{50} = 0.0171

The Particle Size Distribution Reports are provided in Appendix B.

7.8 Embankment Slope

Slope stability analyses were performed for the embankment slope under the bridge to evaluate the slope of the tributary flowing into White Oak Bayou from north to south. During the analyses, a 5-in. thick concrete liner/paving was assumed along the slope face.

The stability analysis was performed using the computer program Slide 6 *(Ref. Slide version 6.029 by Rocscience Inc.)* and the Modified Bishop slope stability analysis procedures. Soil parameters used in the stability analysis was based on experience with local soils and laboratory test results of the soil samples obtained during this study. The Slide 6 computer code searches for the critical slope failure plane and computes the minimum safety factor for a given slope geometry and subsurface soil and groundwater conditions.

Total stress cohesion values for clay soils for the short term (end-of-construction) condition were based upon unconfined compression (UC) test results, unconsolidated-undrained (UU) triaxial

test results, and TWEI's experience with similar soils. Total stress cohesion and angle of internal friction for clay soils for rapid drawdown (intermediate) condition were based upon the consolidated-undrained (CU) triaxial compression with pore pressure measurement test, and TWEI's experience with similar soils. Effective stress cohesion and angle of internal friction for clay soils for long-term condition were based upon the consolidated-undrained (CU) triaxial compression with pore pressure measurement tests, and TWEI's experience with similar soils.

The rapid drawdown condition was analyzed based on the three stage method *(Ref. EM 1110-2-1902, Appendix G, Improved Method for Rapid Drawdown by Duncan, Wright & Wong, 1990).* In the three-stage method, the slope stability analysis is performed in three separate stability calculations for each trial slip surface. The first computation is used to calculate the effective stresses to which the soil is consolidated before drawdown. The second set of computation is performed using undrained shear strengths corresponding to effective stresses calculated in the first stage. If the drained shear strength is less than the undrained shear strength for any slice, a third set of calculations is performed, using drained shear strengths for those slices. The factor of safety from the last stage is the factor of safety after rapid drawdown.

The cohesion for the rapid drawdown and long-term condition (using drained parameters) were reduced to accommodate weathering effect on the exposed cohesive soils [i.e., using a correction factor for peak strength recommended by Mesri and Abdel-Ghaffar (1993)]. Calculations for soil parameters using weathering effects are presented in Appendix G.

The slope stability analyses for the embankment slope under the bridge were performed based on the soil profiles from borings B-1 (upper section of slope) and B-2 (lower section of slope) for the following three design conditions:

- Short Term (End of Construction) Condition. The end of construction condition models the initial undrained condition of the soil. Total stress shear strength parameters were used for analysis. A partially saturated slope was assumed for the short-term condition. For the tributary embankment slope analysis, the piezometric level was assumed at El. +8 ft.

- Intermediate Term (Rapid Drawdown) Condition – Total and effective stress shear strength parameters from the CU tests and TWEI's experience with similar soils were used for analyses. The rapid drawdown case models the condition where high flood waters saturate and piezometrically "load" a slope, and then quickly recede, leaving a large unbalanced piezometric head in the slope. The unbalanced head significantly increases shear stresses in the slope. The stresses relax as the unbalanced piezometric head drains out of the slope with time. For our analyses, the high water was placed near the top-of-bank at El. 39.1 ft (based on 100 year flood plain elevation), along the slope face, and down to the toe of the slope (El. 9.7 ft).

- Long Term Condition – Effective stress shear strength parameters, a partially saturated channel side-slope, and a saturated bottom were used in the analyses. The water level for the long term condition was assumed at the same elevation of short term condition. The long-term case represents steady state piezometric and stress conditions. The long-term

models a case in which the slope soils rely on their available drained shear strength for long-term stability.

Soil parameters used in the slope stability analyses for tributary embankment under the bridge with side slope declivity of 3H:1V are summarized in the table 7-4. Soil parameters at high bank were based on boring B-1, and soil parameters from safety bench to the 50-ft depth were based on boring B-2.

During our analyses, we assumed construction equipment load of 720 psf plus abutment fill height of 5-ft, total of 1345 psf on top of the embankment for short term condition. For long term, the load should be 865 psf (5-ft high fill plus 240 psf from maintenance vehicle) and for rapid drawdown condition the loading would be the fill height (625 psf).

Table 7-4
Soil Parameters for Borings B-1 and B-2

Stra -tum	Elevation (ft)	Description	Unit Weight		ST		RDD		LT	
			γ_t (pcf)	γ_{sat} (pcf)	c_{uu} (psf)	ϕ_{uu} (deg)	c_{cu} (psf)	ϕ_{cu} (deg)	c' (psf)	ϕ' (deg)
1	-	Concrete Paving	150	150	0	0	0	0	0	0
2	34 to 21 High Bank B-1	Sandy Lean Clay (CL)*	132	134	1750	0	260	20	118**	25
3	21 to 19	Silty Clay (CL-ML)*	130	130	1500	0	100	25	60	30
4	19 to 13 Safety Shelf B-2	Sandy Lean Clay "FILL"	127	134	800	0	60	20	50	25
5	-	Sandy Lean Clay "FILL"	127	134	800	0	60	20	50	25
6	13 to 7	Sandy Silt "FILL"	115	115	0	29	0	29	0	29
7		Sandy Silt (ML)	115	115	0	29	0	29	0	29
8	7 to (-2)	Silty Sand (SM)	115	115	0	29	0	29	0	29
9	-2 to (-12)	Fat Clay w. Sand/Fat Clay (CH)	124	126	880	0	194	17	126	25
10	-12 to (-22)	Fat Clay (CH)	123	124	1960	0	562	15	472	21
11	-22 to (-27)	Sandy Lean Clay (CL)	130	132	2640	0	260	20	240	25
12	-27 to (-31)	Silty Sand (SM)	126	128	0	33	0	33	0	33

* Soil layers from high bank boring B-1 were incorporated into the soil layers from boring B-2 which was drilled from the safety shelf.
**Soil properties were adjusted to account for the effect of weathered conditions (due to shrink-swell) in clays.

Where:

ST: Short Term Condition
RDD: Rapid Drawdown Condition
LT: Long Term Condition (Drained Condition)
γ_t: Moist Unit Weight of Soil
γ_{sat}: Saturated Unit Weight of Soil
c_{uu}: Unconsolidated-Undrained Cohesion

ϕ_{uu} Unconsolidated-Undrained Friction Angle
c_{cu}: Consolidated-Undrained Cohesion
ϕ_{cu} Consolidated-Undrained Friction Angle
c': Consolidated-Drained Cohesion
ϕ': Consolidated-Drained Friction Angle

The computed minimum factors of safety against slope failure for the section under the proposed bridge using concrete liner along slope face are presented in Figures 4 through 6 and are summarized in Table 7-5:

Table 7-5
Computed Minimum Factors of Safety

Condition	**Calculated Factor of Safety**
	B-1 and B-2
	3H:1V Slope w/ Concrete Liner
Short Term	1.88
Rapid Drawdown (Duncan, Wright & Wong 1990)	1.64
Long Term	1.82

Table 7-6 summarizes the recommended minimum factors of safety for slope stability analysis.

Table 7-6
Recommended Minimum Factors of Safety

Condition	**Minimum Recommended Factor of Safety**
Short Term	1.30
Rapid Drawdown	1.25
Long Term	1.50

Results of the slope stability analyses indicate that the proposed embankment of tributary under the bridge with side slope declivity of 3H:1V will provide stable slopes for the short term, intermediate term (rapid drawdown) and long term conditions. During the slope stability analyses, concrete liner was assumed along the slope face.

8. WALKWAY PAVEMENT DESIGN

The walkway pavement subgrade preparation including stripping, proof-rolling, subgrade stabilization, and fill placement may be required prior to the construction of rigid pavement. These considerations are addressed in the following paragraphs.

8.1 Site Stripping

After preparation of the alignment along north bank of the bayou, the areas within the walkway should be stripped of topsoil and organic matter, such as major root systems, to a depth of at least 4-in., if required. Any concrete debris, or wood stumps, if present, within the pavement areas should be grubbed to a minimum depth of 3-ft.

8.2 Proof Rolling

After stripping, the project alignment should be proof-rolled to detect zones of soft or wet material for removal. If encountered, weak/soft soils should be undercut and replaced with material of similar physical and moisture characteristics. The ground surface should be appropriately graded throughout construction to prevent ponding of rainfall runoff and provide positive drainage. The stripping and proof-rolling should be witnessed by the Geotechnical Engineer or a representative.

8.3 Fill Placement for Pavement

Fill, if required, for grading at pavement areas may be on-site cohesive soil, free of organic matter and excessive silt. All cohesive fill should be placed in lifts not exceeding 8-in. loose measure, compacted to at least 95% of the standard Proctor test maximum dry density at moisture content within two percentage points of the optimum moisture content. The laboratory-measured maximum dry density and optimum moisture content should be determined in accordance with ASTM D 698.

8.4 Subgrade Stabilization

The surficial soils within upper 2-ft to 3-ft depths in the project borings are cohesive low to high plasticity sandy lean clays, lean clays with sand and cohesionless silty sands.

Subgrade stabilization may be necessary if the cohesionless, and low plasticity cohesive subgrade soils become saturated and cannot be adequately compacted. Subgrade stabilization will facilitate the contractor's site preparation activities. Stabilization will also increase the modulus of subgrade reaction and thus the pavement life.

For planning purposes, a lime-fly ash content of 3% lime and 8% fly ash, by dry weight, may be specified in the project documents for stabilization. We recommend establishing a separate line item for stabilization since the actual requirements should be verified in the field after stripping and grading. The stabilized soil should have a minimum thickness of 6-in. and compacted to at least 95% of the standard Proctor maximum dry density within two percentage points of the

optimum moisture content (ASTM D 698). Lime-fly ash stabilization should adhere to City of Houston Standard Specification Section 02337, *Lime Fly Ash Stabilized Subgrade*.

The subgrade stabilization should extend a minimum of 2-ft beyond the pavement width to reduce potential edge influences. Compaction should begin immediately after final mixing and the stabilized subgrade should cure at least two days before placing pavement.

8.5 Pavement Design

We understand that the proposed trail will be used for pedestrian as well as maintenance vehicle traffic. Concrete pavement is planned for the walkway. The subgrade soils should be stabilized with lime-fly ash prior to pavement construction as detailed in Section 8.4 *Subgrade Stabilization*. **The minimum thickness of the walkway can be considered as 6-in.**

Maintaining the pavement to prevent infiltration of water into the subgrade soils is essential. Allowing water in the subgrade will result in high maintenance costs and premature pavement failure. Periodic maintenance should be performed on the pavement sections to seal any surface cracks and prevent infiltration of water into the subgrade.

9. CLOSURE

9.1 Limitations

This report has been prepared for the exclusive use of Klotz Associates, Inc. and their design team for specific application to the construction of the referenced project at the aforementioned location in Houston, Texas. Our report has been prepared in accordance with generally accepted geotechnical engineering practice common to the local area. No other warranty, express or implied, is made.

The analyses and recommendations contained in this report are based on the data obtained from the referenced subsurface exploration. The borings indicated subsurface conditions only at the specific locations, at the time borings were drilled, and only to the depths penetrated. The borings do not necessarily reflect strata variations that may exist between boring locations. The validity of the recommendations is based in part on assumptions about the stratigraphy made by the Geotechnical Engineer. Such assumptions may be confirmed only during earthwork and construction. If subsurface conditions different from those described are noted during construction, recommendations in this report must be re-evaluated.

If any changes in the nature, design, or location of the proposed project are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report are modified or verified in writing by TWEI. TWEI is not responsible for any claims, damages, or liability associated with interpretation of subsurface data by others or reuse of the subsurface data by others or engineering analyses by others without the expressed written authorization of TWEI.

9.2 Design Review

Review of the design and construction plans as well as the specifications should be performed by TWEI before release. The review is aimed at determining if the geotechnical design recommendations and construction criteria presented in this report have been properly interpreted. Design review is not within the scope of work authorized in this study.

9.3 Construction Surveillance

Surveillance of the walkway subgrade preparation, pavement and bridge structure construction is recommended, and has been assumed in preparing our recommendations. This field service is required to check for changed conditions that may result in modifications to our recommendations. The quality of the construction practices will affect trail performance and should be monitored.

10. REFERENCES

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Terzaghi, K., Peck, R.B., and Mesri, G. (1996) "*Soil Mechanics in Engineering Practice,*" 3rd Edition, John Wiley and Sons, N.Y., pp. 44-45

Texas Department of Transportation (TxDOT) (2014) "*Standard Specifications for The Construction and Maintenance of Highways, Streets, and Bridges.*"

TxDOT (2012) "*Geotechnical Manual.*"

FIGURES

NOTES:

1. B-5 AND B-6 NOT DRILLED DUE TO ACCESS PROBLEM
2. B-1 WAS OFFSET TO B-1A DUE TO REFUSAL AT 28' DEPTH

MKT SPUR CONNECTOR AND BIKEWAY BRIDGE WHITE OAK BAYOU HOUSTON, TEXAS	**Tolunay-Wong Engineers, Inc.** Houston, Texas	PLAN OF BORINGS
KLOTZ ASSOCIATES, INC. HOUSTON, TEXAS		PROJECT NO.: 15.13.077 / FIGURE 1

PIEZOMETER COMPLETION

Date: 06/29/15

Dry Augered: 0.0 ft to 12.0 ft

Wash Bored: 12.0 ft to 50.0 ft

Drilling Fluid:

PIEZOMETER DEVELOPMENT

Date: 6/29/2015

Method: Air Lift

WATER LEVEL READINGS

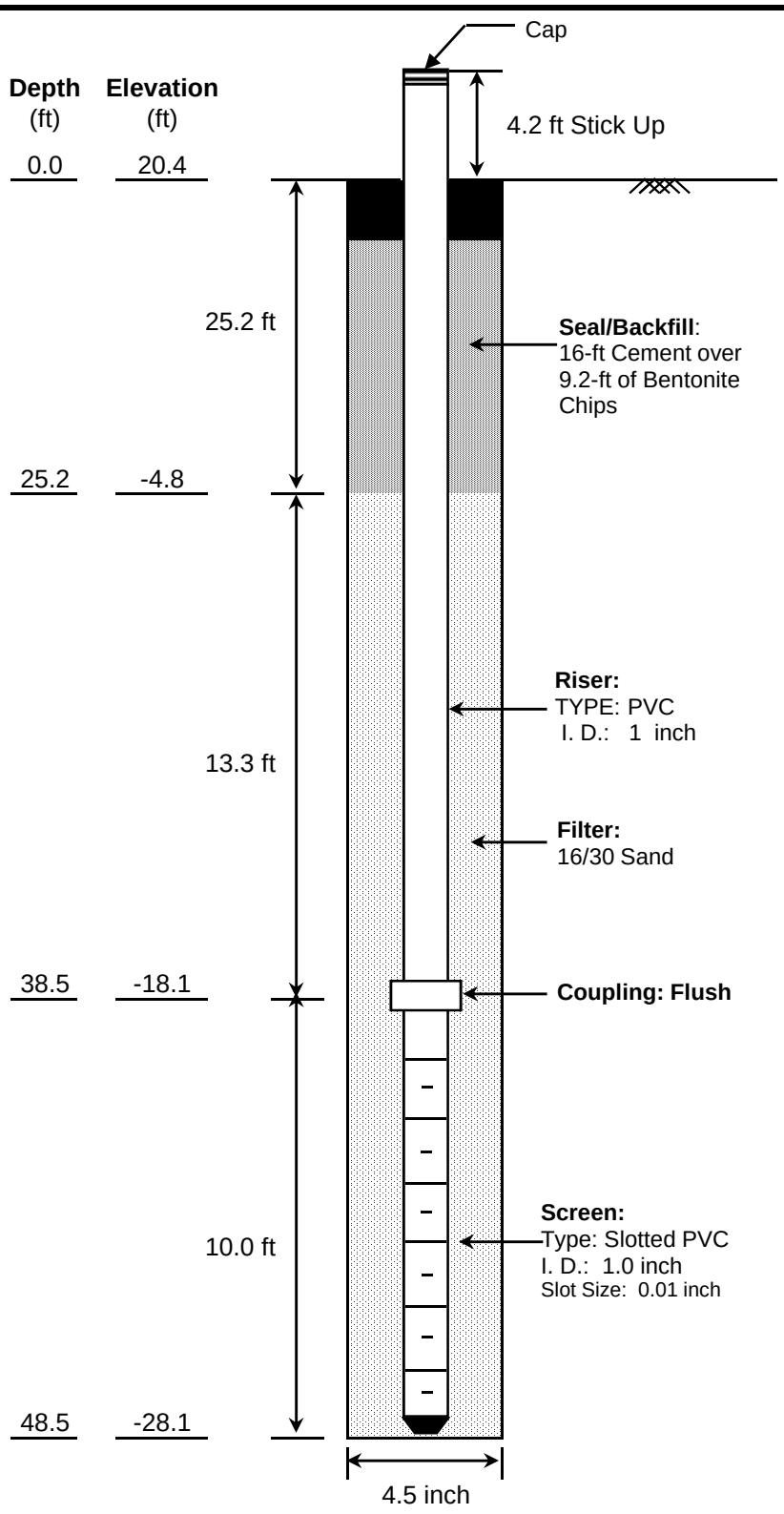
Free Water at: 12.0 ft

Date	Depth, ft*	Elevation, ft
07/06/15	Dry	
07/16/15	Dry	
07/27/15	42.55	-22.18
08/14/15	42.60	-22.23

*Depths are measured below existing grade

REMARKS

1. Boring B-3 converted to standpipe piezometer PZ-1 three days after completion of sampling.



Project:
MKT Spur Connector and Bikeway Bridge
White Oak Bayou
Houston, Texas

Tolunay-Wong Engineers, Inc.
Houston, Texas

Project Number:
15.13.077

Client:
Klotz Associates, Inc.
Houston, Texas

Piezometer Installation Report:
PZ-1

Figure 2

PIEZOMETER COMPLETION

Date: 07/10/15

Dry Augered: 0.0 ft to 8.0 ft

Wash Bored: 8.0 ft to 50.0 ft

Drilling Fluid:

PIEZOMETER DEVELOPMENT

Date: 7/10/2015

Method: Air Lift

WATER LEVEL READINGS

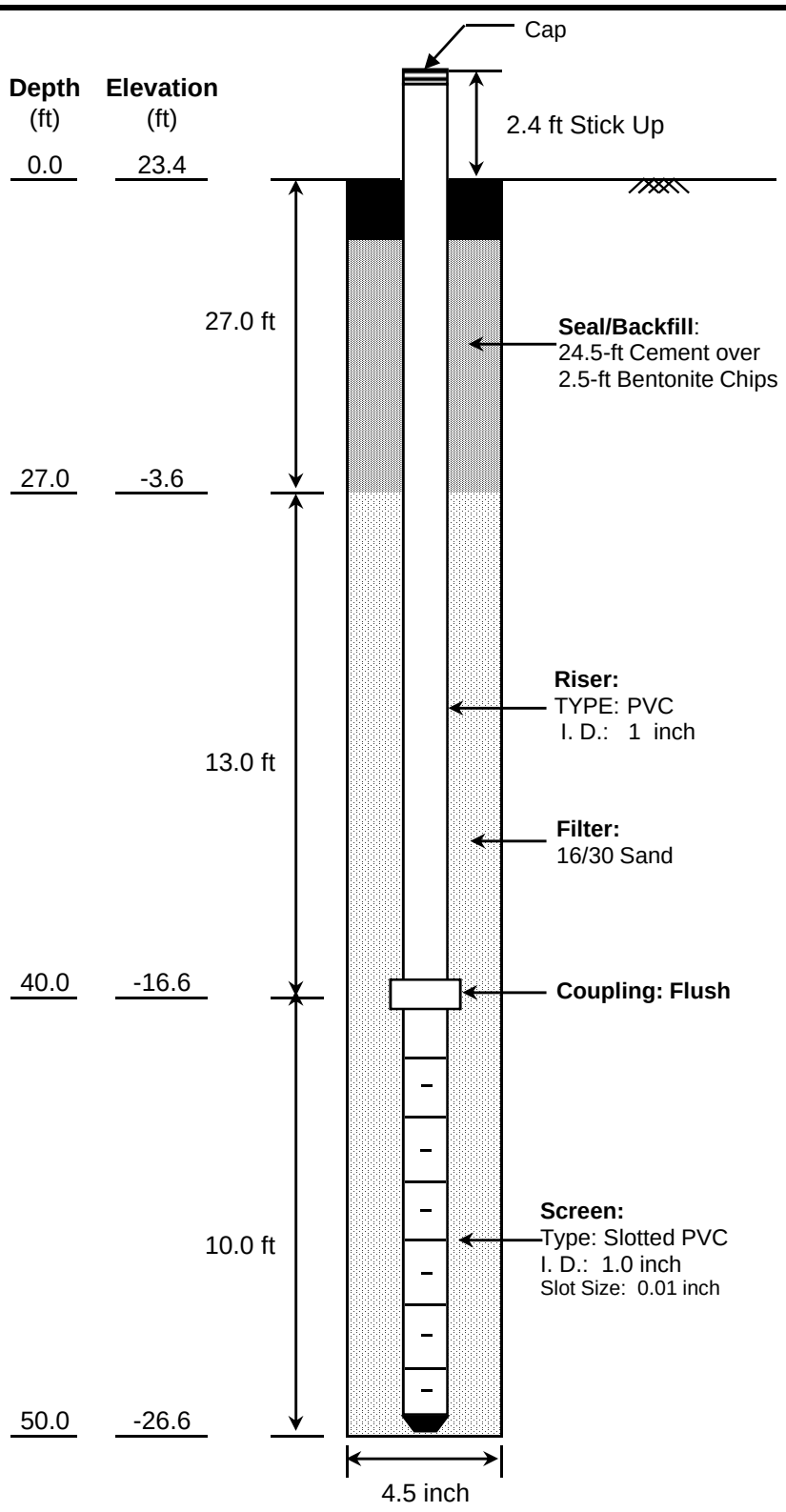
Free Water at:

Date	Depth, ft*	Elevation, ft
07/10/15	29.97	-6.60
07/16/15	27.80	-4.43
07/27/15	28.15	-4.78
08/14/15	28.34	-4.97

*Depths are measured below existing grade

REMARKS

1. Boring B-1A converted to standpipe piezometer PZ-2 upon completion of sampling.



Project:
MKT Spur Connector and Bikeway Bridge
White Oak Bayou
Houston, Texas

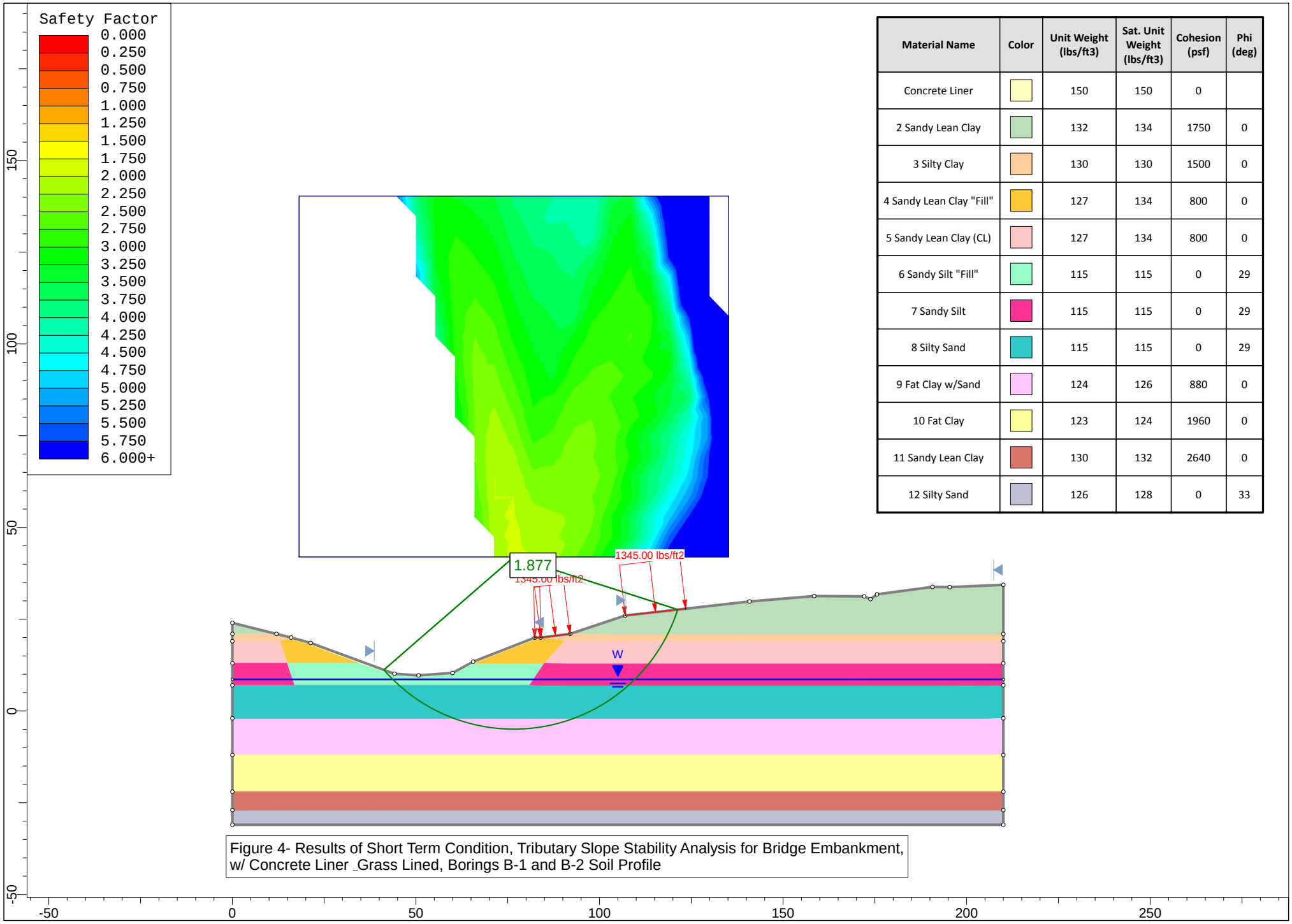
Tolunay-Wong Engineers, Inc.
Houston, Texas

Project Number:
15.13.077

Client:
Klotz Associates, Inc.
Houston, Texas

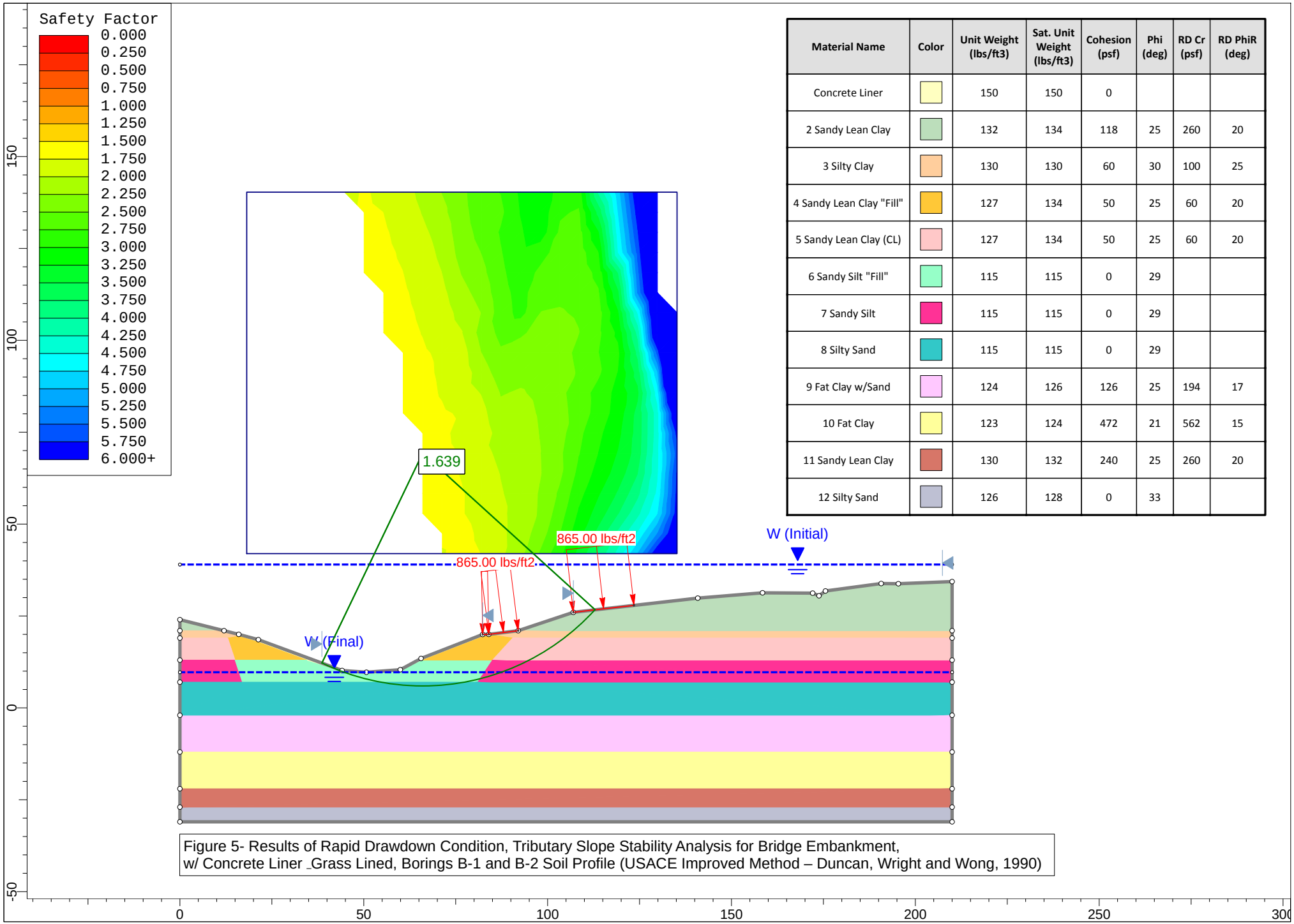
Piezometer Installation Report:
PZ-2

Figure 3



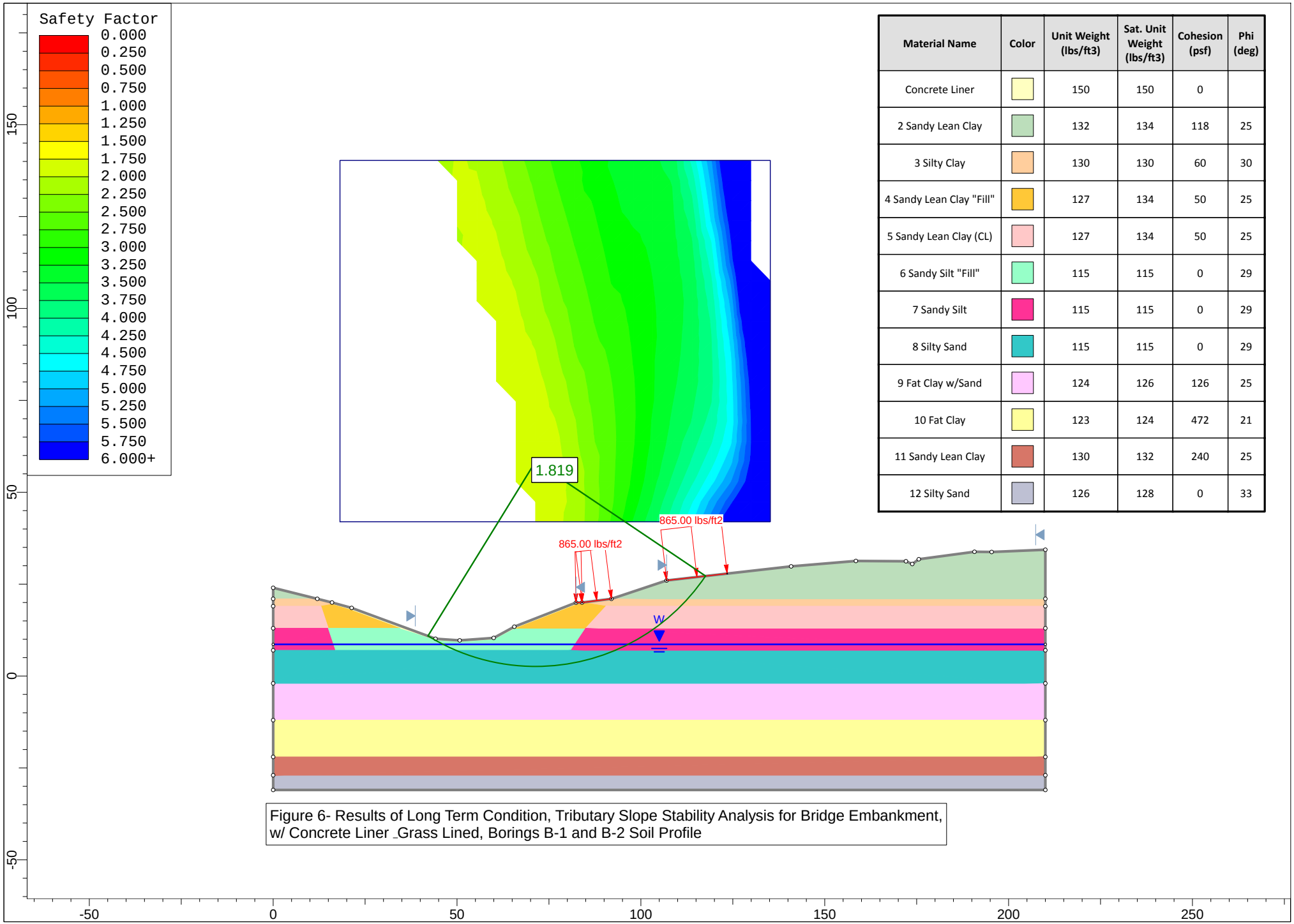
Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Concrete Liner		150	150	0	
2 Sandy Lean Clay		132	134	1750	0
3 Silty Clay		130	130	1500	0
4 Sandy Lean Clay "Fill"		127	134	800	0
5 Sandy Lean Clay (CL)		127	134	800	0
6 Sandy Silt "Fill"		115	115	0	29
7 Sandy Silt		115	115	0	29
8 Silty Sand		115	115	0	29
9 Fat Clay w/Sand		124	126	880	0
10 Fat Clay		123	124	1960	0
11 Sandy Lean Clay		130	132	2640	0
12 Silty Sand		126	128	0	33

Figure 4- Results of Short Term Condition, Tributary Slope Stability Analysis for Bridge Embankment, w/ Concrete Liner _Grass Lined, Borings B-1 and B-2 Soil Profile



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	RD Cr (psf)	RD PhiR (deg)
Concrete Liner		150	150	0			
2 Sandy Lean Clay		132	134	118	25	260	20
3 Silty Clay		130	130	60	30	100	25
4 Sandy Lean Clay "Fill"		127	134	50	25	60	20
5 Sandy Lean Clay (CL)		127	134	50	25	60	20
6 Sandy Silt "Fill"		115	115	0	29		
7 Sandy Silt		115	115	0	29		
8 Silty Sand		115	115	0	29		
9 Fat Clay w/Sand		124	126	126	25	194	17
10 Fat Clay		123	124	472	21	562	15
11 Sandy Lean Clay		130	132	240	25	260	20
12 Silty Sand		126	128	0	33		

Figure 5- Results of Rapid Drawdown Condition, Tributary Slope Stability Analysis for Bridge Embankment, w/ Concrete Liner _Grass Lined, Borings B-1 and B-2 Soil Profile (USACE Improved Method – Duncan, Wright and Wong, 1990)



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Concrete Liner		150	150	0	
2 Sandy Lean Clay		132	134	118	25
3 Silty Clay		130	130	60	30
4 Sandy Lean Clay "Fill"		127	134	50	25
5 Sandy Lean Clay (CL)		127	134	50	25
6 Sandy Silt "Fill"		115	115	0	29
7 Sandy Silt		115	115	0	29
8 Silty Sand		115	115	0	29
9 Fat Clay w/Sand		124	126	126	25
10 Fat Clay		123	124	472	21
11 Sandy Lean Clay		130	132	240	25
12 Silty Sand		126	128	0	33

Figure 6- Results of Long Term Condition, Tributary Slope Stability Analysis for Bridge Embankment, w/ Concrete Liner _Grass Lined, Borings B-1 and B-2 Soil Profile

APPENDIX A

WINCORE LOGS (BORINGS B-1, B-1A, B-2, B-3 AND B-4)

DRILLING LOG

WinCore	County	Harris	Hole	B-1	District	Houston
	Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	06.30.2015
Version 3.1	CSJ	0912-72-299	Station	-0+29.08	Grnd. Elev.	46.51 ft
			Offset	48.41' L	GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
			CLAY, lean with sand, very soft, tan & gray, w/ sand pockets, ferrous nodules "FILL"			14.4			128.4	PP = 3.25 tsf
				0	37.5	17.8	45	28	135.4	Strain @ failure = 4.9% Pass # 200 (%) = 74.6 D50 = 0.0279 mm PP = 3.00 tsf
5		2 (6) 2 (6)								
40.5			CLAY, sandy lean, soft, gray, w/ sand pockets "FILL"			18.2	28	12	124.7	Pass # 200 (%) = 54.1 PP = 2.75 tsf
		7 (6) 11 (6)								
36.5 10			SILT, with sand, medium dense, light gray & reddish brown (ML)			21.4				Pass # 200 (%) = 73.5 SPT N = 12
34.5			CLAY, sandy lean, soft to stiff, reddish brown & light gray, w/ calcareous nodules, sand pockets, sand partings, sand seams (CL)			14.2				SPT N = 24
15		7 (6) 9 (6)								
						16.0	40	23		Pass # 200 (%) = 61.8 SPT N = 6
		17 (6) 16 (6)								
20										
						16.2	38	21	131.5	Pass # 200 (%) = 67.0 PP = 4.50 tsf
		12 (6) 19 (6)								
25										
20.5			CLAY, silty, light gray, w/ sand seams & partings (CL-ML)			19.4			130.4	PP = 4.50 tsf -refusal @ 28' due to unknown object
18.5										
30										

Remarks: Due to caving at 14-ft depth, the borehole was continued with the wet rotary technique below the 16-ft depth.Boring B-1 was offset to B-1A due to refusal at 28 ft depth.

The ground water elevation was not determined during the course of this boring.

Driller: Triangle Resources, Inc. **Logger:** T. Horton **Organization:** Tolunay-Wong Engineers, Inc.

DRILLING LOG

WinCore	County	Harris	Hole	B-1A	District	Houston
	Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	07.9 &10.2015
Version 3.1	CSJ	0912-72-299	Station	1+34.39	Grnd. Elev.	23.37 ft
			Offset	41.51' R	GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
			CLAY, sandy lean, very soft to soft, reddish brown & gray, fine roots, w/ organic material @ 8' -10', "FILL"			13.3				SPT N = 11 Pass # 200 (%) = 53.2
						13.0				SPT N = 8
5		6 (6) 6 (6)								
						22.8	38	21		PP = 1.00 tsf Pass # 200 (%) = 69.8
										-organic material @ 8' -10'
10		3 (6) 2 (6)								
						23.6	37	20		PP = 1.00 tsf Pass # 200 (%) = 69.3
15		2 (6) 2 (6)								
7.4			CLAY, fat, soft to stiff, reddish brown & gray, w/ ferrous stains, sand pockets & seams, claystones, silt pockets & partings (CH)	0	11.6	25.8	54	33	126.6	PP = 4.00 tsf Pass # 200 (%) = 98.8 Strain @ failure = 0.6%
		5 (6) 5 (6)								
20						31.2	58	36	122.8	PP = 4.50 tsf Pass # 200 (%) = 96.6
		7 (6) 9 (6)								
25				0	31.8	26.6	67	45	123.9	PP = 4.50 tsf Pass # 200 (%) = 99.7 Strain @ failure = 4.2%
		15 (6) 14 (6)								
30										

Remarks: Due to resistance from organic materials at the 8-ft depth, the borehole was continued with the wet rotary technique below that depth. B-1A was converted to piezometer PZ-2 after the completion of drilling.

The ground water elevation was not determined during the course of this boring.

Driller: Triangle Resources, Inc. **Logger:** L. Cantin **Organization:** Tolunay-Wong Engineers, Inc.

DRILLING LOG

WinCore	County	Harris	Hole	B-1A	District	Houston
	Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	07.9 &10.2015
Version 3.1	CSJ	0912-72-299	Station	1+34.39	Grnd. Elev.	23.37 ft
			Offset	41.51' R	GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
			CLAY, fat, soft to stiff, reddish brown & gray, w/ ferrous stains, sand pockets & seams, claystones, silt pockets & partings (CH)			16.7			126.3	PP = 4.50 tsf
		18 (6) 16 (6)								
35				33	30.2	21.4	58	36	124.7	PP = 4.50 tsf Pass # 200 (%) = 98.8 Strain @ failure = 14.8%
		14 (6) 13 (6)								
40						32.7			120.1	PP = 4.00 tsf
		13 (6) 12 (6)								
-21.6 45			SAND, clayey, compact, reddish brown & gray, w/ calcareous nodules (SC)			14.7	36	20	130.5	Pass # 200 (%) = 47.0
		26 (6) 24 (6)								
-26.6 50										
55										
60										

Remarks: Due to resistance from organic materials at the 8-ft depth, the borehole was continued with the wet rotary technique below that depth. B-1A was converted to piezometer PZ-2 after the completion of drilling.

The ground water elevation was not determined during the course of this boring.

Driller: Triangle Resources, Inc. **Logger:** L. Cantin **Organization:** Tolunay-Wong Engineers, Inc.

DRILLING LOG

WinCore	County	Harris	Hole	B-2	District	Houston
Version 3.1	Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	07.8&9.2015
	CSJ	0912-72-299	Station	4+88.28	Grnd. Elev.	19.35 ft
			Offset	52.58' R	GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
			CLAY, sandy lean, soft, reddish brown, tan & gray, w/ fine roots "FILL"			10.8			126.5	PP = 4.00 tsf
						14.2	27	12	128.1	PP = 3.25 tsf Pass # 200 (%) = 66.1 D50 = 0.0469 mm
5		5 (6) 7 (6)								
13.4			SILT, sandy, very loose, tan, w/ sand seams & pockets, clay seams, "FILL"			15.1				Pass # 200 (%) = 54.2 SPT N = 3
		1 (6) 3 (6)								
10						22.5				SPT N = 2
7.4			SAND, silty, very loose, tan & gray, w/ clay pockets & seams (SM)			23.2				Pass # 200 (%) = 48.3 SPT N = WOH
		2 (6) 1 (6)								
15						19.9				SPT N = 4
		3 (6) 4 (6)								
20										
-1.6			CLAY, fat w/ sand, very soft, reddish brown, w/ ferrous & calcareous nodules, sand seams (CH)			27.5	63	42		Pass # 200 (%) = 80.6 SPT N = 8
		1 (6) 4 (6)								
25										
-6.6			CLAY, fat, soft to stiff, reddish brown & gray, slickensided, w/ calcareous & ferrous nodules, sand seams, claystones, silt seams (CH)	0	40.2	26.2	70	46	124.4	PP = 4.50 tsf Pass # 200 (%) = 99.6 Strain @ failure = 3.3%
		8 (6) 9 (6)								
30										

Remarks: Due to caving at 16-ft depth, the borehole was continued with the wet rotary technique below the 16-ft depth.

The ground water elevation was not determined during the course of this boring.

Driller: Triangle Resources, Inc. **Logger: L. Cantin** **Organization: Tolunay-Wong Engineers, Inc.**

DRILLING LOG

	County	Harris	Hole	B-2	District	Houston
WinCore	Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	07.8&9.2015
Version 3.1	CSJ	0912-72-299	Station	4+88.28	Grnd. Elev.	19.35 ft
			Offset	52.58' R	GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
			CLAY, fat, soft to stiff, reddish brown & gray, slickensided, w/ calcareous & ferrous nodules, sand seams, claystones, silt seams (CH)			26.7	65	42	125.7	Pass # 200 (%) = 99.8 PP = 4.50 tsf
		11 (6) 11 (6)								
35				33	36	28.4	68	46	119.8	PP = 4.50 tsf Pass # 200 (%) = 99.7 Strain @ failure = 8.1%
		13 (6) 13 (6)								
40										
-21.6			CLAY, sandy lean, very stiff, reddish brown & gray, w/ sand seams & pockets (CL)	0	48.6	17	45	28	130	PP = 4.50 tsf Pass # 200 (%) = 63.8 Strain @ failure = 7.5%
		18 (6) 22 (6)								
45										
-26.6			SAND, silty, slightly compact, gray, w/ calcareous nodules, clay pockets (SM)			19.5			126.4	Pass # 200 (%) = 40.3
		15 (6) 16 (6)								
-30.6 50										
55										
60										

Remarks: Due to caving at 16-ft depth, the borehole was continued with the wet rotary technique below the 16-ft depth.

The ground water elevation was not determined during the course of this boring.

Driller: Triangle Resources, Inc. **Logger: L. Cantin** **Organization: Tolunay-Wong Engineers, Inc.**

DRILLING LOG

WinCore	County	Harris	Hole	B-3	District	Houston
Version 3.1	Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	06.26.2015
	CSJ	0912-72-299	Station	6+19.28	Grnd. Elev.	20.37 ft
			Offset	38.19' R	GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	Properties: MC	Properties: LL	Properties: PI	Properties: Wet Den. (pcf)	Additional Remarks
			SAND, silty, loose, tan, w/ fine roots "FILL"			14.6				Pass # 200 (%) = 39.1 SPT N = 9
18.4			CLAY, sandy lean, soft, reddish brown "FILL"			15.4	28	14		Pass # 200 (%) = 60.6 SPT N = 10
5		7 (6) 7 (6)								
14.4			SAND, silty, slightly compact, gray "FILL"			12.6				Pass # 200 (%) = 40.2
		15 (6) 14 (6)								
10.4 10			SAND, silty, very loose, tan (SM)			20.8				SPT N = 3
						20.8				Pass # 200 (%) = 41.3 SPT N = 5
15		1 (6) 1 (6)								
4.4			SAND, silty clayey, very loose to loose, gray & tan, w/ clay pockets (SC-SM)			25.1				Pass # 200 (%) = 42.7 SPT N = WOH
		1 (6) 3 (6)								
20						23.2				Pass # 200 (%) = 32.1 SPT N = WOH
		4 (6) 3 (6)								
25										
-5.6			SILT, slightly compact, reddish brown (ML)			27.1				Pass # 200 (%) = 85.9 SPT N = 22
		12 (6) 11 (6)								
30										

Remarks: Free water was encountered at the 12-ft depth. The borehole was dry with caving depth of 8-ft 10 minutes after encountering free water. Boring B-3 was converted to piezometer PZ-1 three days after drilling completion.

The ground water elevation was not determined during the course of this boring.

Driller: Triangle Resources, Inc. **Logger:** G. Singleton **Organization:** Tolunay-Wong Engineers, Inc.

DRILLING LOG

WinCore	County	Harris	Hole	B-3	District	Houston
Version 3.1	Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	06.26.2015
	CSJ	0912-72-299	Station	6+19.28	Grnd. Elev.	20.37 ft
			Offset	38.19' R	GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
-10.6			SILT, slightly compact, reddish brown (ML)							
			CLAY, lean, stiff, reddish brown (CL)			29.1	44	25		Pass # 200 (%) = 98.4 SPT N = 21
		12 (6) 12 (6)								
35										
-15.6			CLAY, fat, stiff, reddish brown, slickensided,w/ silt pockets (CH)			29.8	60	38	122.9	Pass # 200 (%) = 99.5 PP = 4.25 tsf
		10 (6) 10 (6)								
40										
-20.6			CLAY, lean with sand, stiff, tan & gray, w/ ferrous & calcareous nodules (CL)	0	39.3	18.2	42	25	134.6	Strain @ failure = 5.5% Pass # 200 (%) = 73.4 D50 = 0.0171 mm PP = 4.50 tsf
		17 (6) 17 (6)								
45										
-25.6			SAND, silty clayey, compact, tan & reddish brown (SC-SM)			20.4				Pass # 200 (%) = 38.4 SPT N = 33
		20 (6) 27 (6)								
-29.6 50										
55										
60										

Remarks: Free water was encountered at the 12-ft depth. The borehole was dry with caving depth of 8-ft 10 minutes after encountering free water. Boring B-3 was converted to piezometer PZ-1 three days after drilling completion.

The ground water elevation was not determined during the course of this boring.

Driller: Triangle Resources, Inc. **Logger:** G. Singleton **Organization:** Tolunay-Wong Engineers, Inc.

DRILLING LOG

WinCore	County	Harris	Hole	B-4	District	Houston
Version 3.1	Highway	MKT Spur Con. & Bikeway	Structure	Pavement	Date	06.26.2015
	CSJ	0912-72-299	Station	8+28.32	Grnd. Elev.	21.62 ft
			Offset	11.50' L	GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	Properties: MC	Properties: LL	Properties: PI	Properties: Wet Den. (pcf)	Additional Remarks
			SAND, silty, slightly compact, tan & gray, w/ roots, clay seams & pockets "FILL"			14.4				Pass # 200 (%) = 49.4
						14.1				Pass # 200 (%) = 31.5 SPT N = 15
5		15 (6) 22 (6)								
						11.7				Pass # 200 (%) = 32.6 SPT N = 35
		9 (6) 12 (6)								
10										
10.6			CLAY, sandy lean, very soft, gray "FILL"			20.4	27	11		Pass # 200 (%) = 54.6 SPT N = 9
		1 (6) 2 (6)								
6.6 15										
20										
25										
30										

Remarks: Free water was encountered at the 13-ft depth. Water level in the open borehole rose to 4.0-ft depth, 17 minutes after completion of sampling. Borehole was open to 8.1 ft depth.

The ground water elevation was not determined during the course of this boring.

Driller: Triangle Resources, Inc. **Logger:** G. Singleton **Organization:** Tolunay-Wong Engineers, Inc.

SYMBOLS AND TERMS USED ON BORING LOGS

Unified Soil Classifications System Symbols

- Lean Clay (CL)
- Lean Clay w/ Sand (CL)
- Sandy Lean Clay (CL)
- Fat Clay (CH)
- Fat Clay w/ Sand (CH)
- Sandy Fat Clay (CH)
- Silty Clay (CL-ML)
- Sandy Silty Clay (CL-ML)
- Silty Clayey Sand (SC-SM)
- Well Graded Sand (SW)
- Well Graded Sand w/ Gravel (SW-GM)
- Poorly Graded Sand (SP)
- Poorly Graded Sand w/ Silt (SP-SM)
- Silt (ML)
- Clayey Sand (SC)
- Sandy Silt (ML)
- Silty Sand (SM)
- Silt w/ Sand (ML)

Miscellaneous Materials

- Fill
- Concrete
- Asphalt and/or Base

Sampler Symbols	Meaning
	Depth of thin-walled tube sample
	Depth of Standard Penetration Test (SPT)
	Depth of auger sample
	Depth of sampling attempt with no recovery
	TxDOT Cone Penetrometer Test

Field Test Data

27(6)	Blow count per 6-in. interval of the TxDOT Cone Penetrometer Test
2.50	Pocket penetrometer reading in tons per square foot
8/6	Blow count per 6-in. interval of the Standard Penetration Test
	Observed free water during drilling
	Observed static water level

Laboratory Test Data

W_c (%)	Moisture content in percent
Dens. (pcf)	Dry unit weight in pounds per cubic foot
Q_u (tsf)	Unconfined compressive strength in tons per square foot
UU (tsf)	Compressive strength under confining pressure in tons per square foot
Str. (%)	Strain at failure in percent
LL	Liquid limit in percent
PI	Plasticity index
#200 (%)	Percent passing the No. 200 mesh sieve
( )	Confining pressure in pounds per square inch
*	Slickensided failure
**	Did not fail

RELATIVE DENSITY OF COHESIONLESS & SEMI-COHESIVE SOILS

The following descriptive terms for relative density apply to cohesionless soils such as gravels, silty fine sands, and fine sands as well as semi-cohesive and semi-cohesionless soils such as sandy silts, clayey silts, and clayey sands.

Relative Density	Typical Texas Cone Pen. Value Range*	Typical SPT "N" Value Range**
Very Loose	0 to 8	0-4
Loose	8 to 20	5-10
Slightly Compact	20 to 40	11-30
Compact	40 to 80	---
Dense	80 to 5"/100	31-50
Very Dense	5"/100 to 0"/100	Over 50

* Texas Cone Penetrometer (TCP) is seated in the soil or rock by driving the TCP 12 blows or 6 inches whichever comes first. The blows are from a 170-lb weight having a free fall of 24-in. required to penetrate the final 12-in. in relatively soft materials or 100 blows into relatively hard materials. The cone is driven with the hammer in two 6-inch increments, a total of 12 inches into relatively soft materials. Penetration in inches is recorded for the 1st and 2nd 50 blows for a total of 100 blows in hard materials

CONSISTENCY OF COHESIVE SOILS

The following descriptive terms for consistency apply to cohesive soils such as clays, sandy clays, and silty clays.

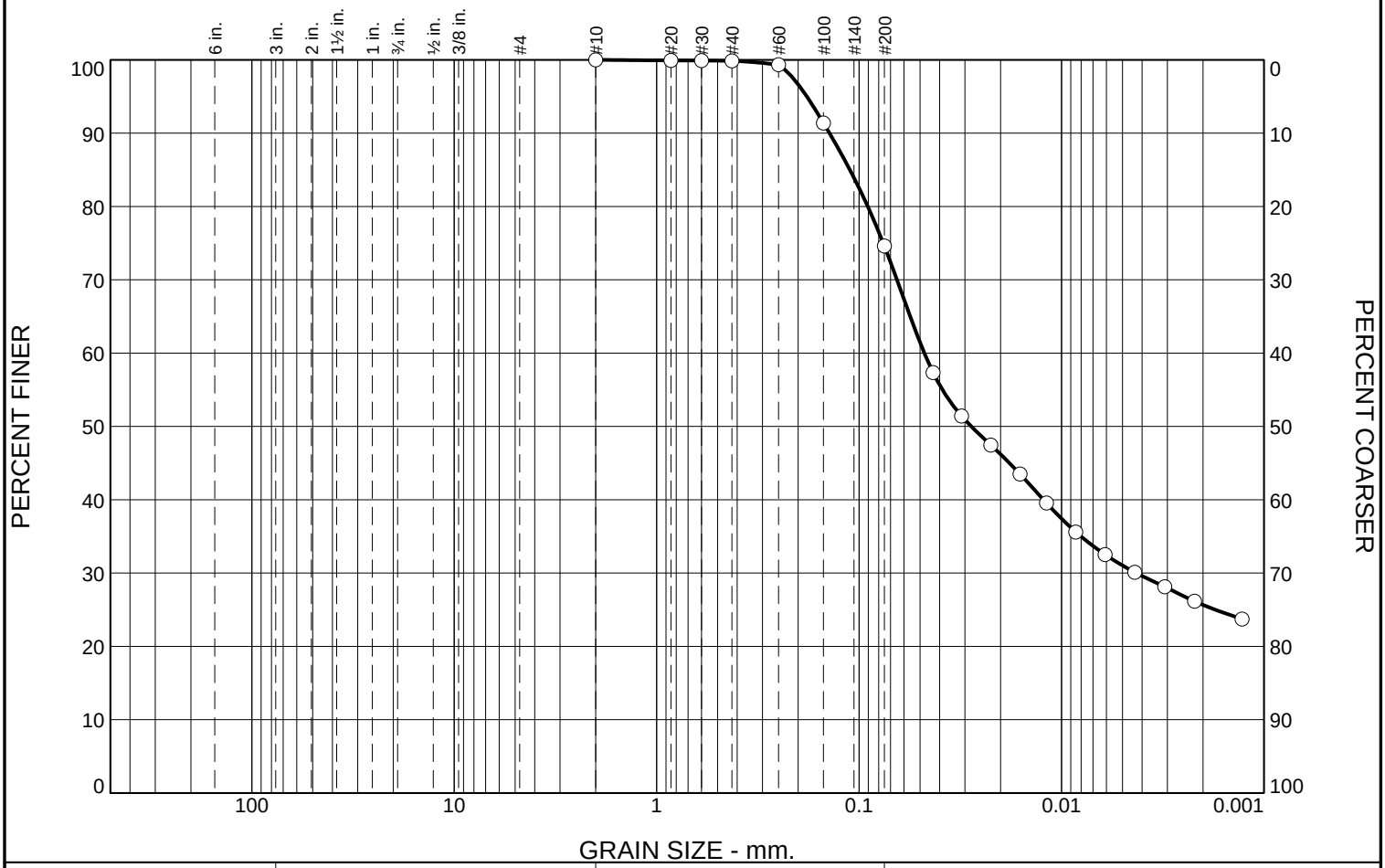
Typical Unconfined Compressive Strength (tsf)	Consistency	Typical Texas Cone Pen. Value Range*
$q_u < 0.25$	Very Soft	0 to 8
$0.25 < q_u \leqq 1.00$	Soft	8 to 20
$1.00 < q_u \leqq 2.00$	Stiff	20 to 40
$2.00 < q_u \leqq 4.00$	Very Stiff	40 to 80
$q_u > 4.00$	Hard	80 to 5"/100
	Very Hard	5"/100 to 0"/100

** "N" is the number of blows from a 140-lb weight having a free fall of 30-in. required to penetrate the final 12-in. of an 18-in. sample interval. The density designations correspond to a SPT "N" value range based on an effective overburden pressure of 1 tsf.

APPENDIX B

SELECTED PARTICLE SIZE ANALYSIS OF SOILS TEST RESULTS

Particle Size Analysis (ASTM D422)



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0.0	0.0	0.1	25.3	48.9	25.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.9		
#30	99.9		
#40	99.9		
#60	99.3		
#100	91.4		
#200	74.6		

* (no specification provided)

Material Description

Gray & tan LEAN CLAY w/ SAND; ferrous nodules

Atterberg Limits

PL= 17 LL= 45 PI= 28

Coefficients

D_{90}= 0.1401 D_{85}= 0.1111 D_{60}= 0.0476
D_{50}= 0.0279 D_{30}= 0.0043 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(20)

Remarks

F.M.=0.09

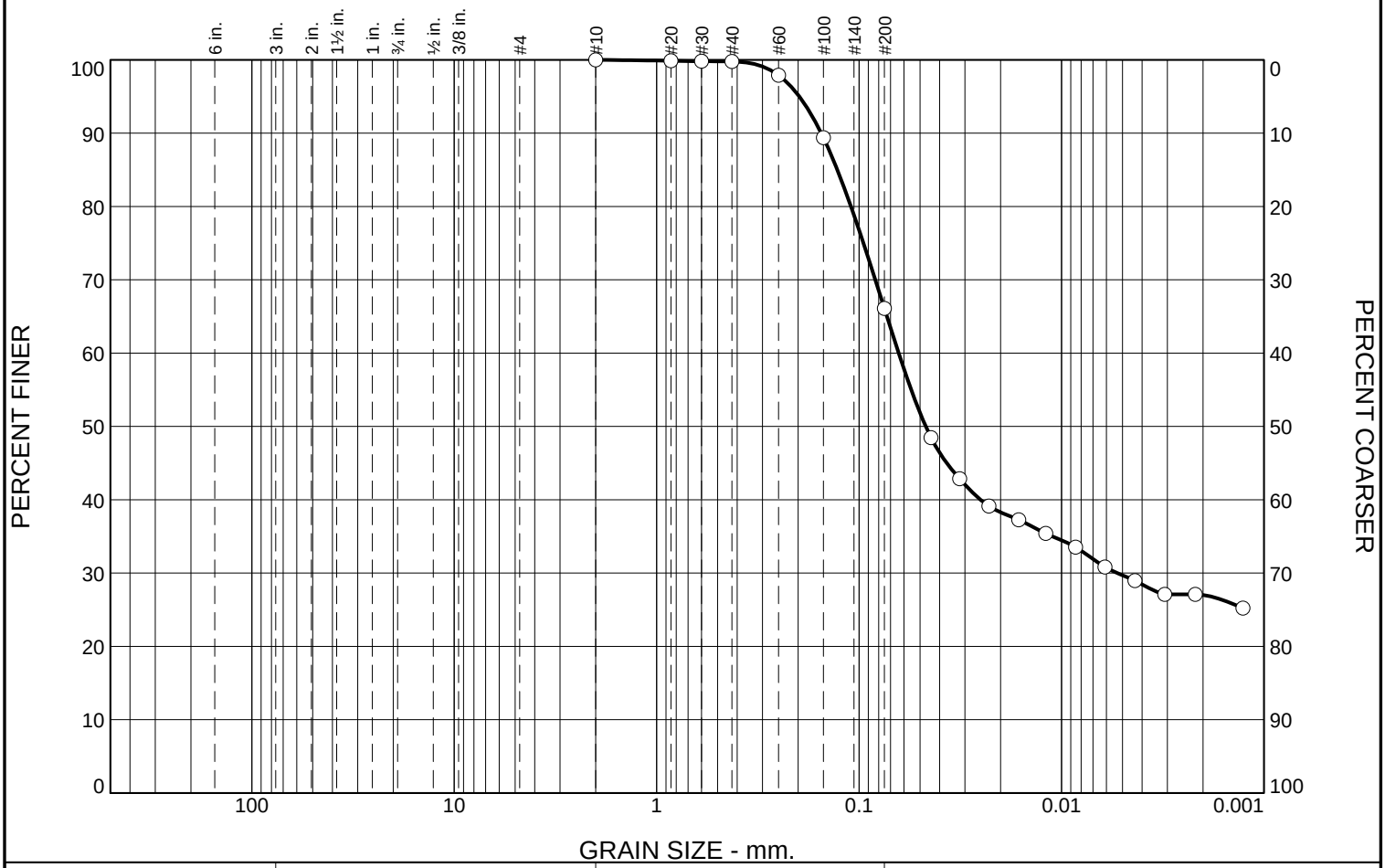
Source of Sample: B-1 **Depth:** 2'-4'

Date: 6/30/15

Tolunay-Wong Engineers, Inc. Houston, Texas

Client: Klotz Associates, Inc., Houston, Texas
Project: MKT Spur Connector and Bikeway Bridge, White Oak Bayou Houston, Texas
Project No: 15.13.077 **Figure**

Particle Size Analysis (ASTM D422)



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0.0	0.0	0.2	33.7	39.1	27.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.9		
#30	99.8		
#40	99.8		
#60	97.9		
#100	89.4		
#200	66.1		

* (no specification provided)

Material Description

Reddish brown, tan & gray SANDY LEAN CLAY

Atterberg Limits

PL= 15 LL= 27 PI= 12

Coefficients

D_{90}= 0.1538 D_{85}= 0.1282 D_{60}= 0.0638
D_{50}= 0.0469 D_{30}= 0.0053 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(5)

Remarks

F.M.=0.12

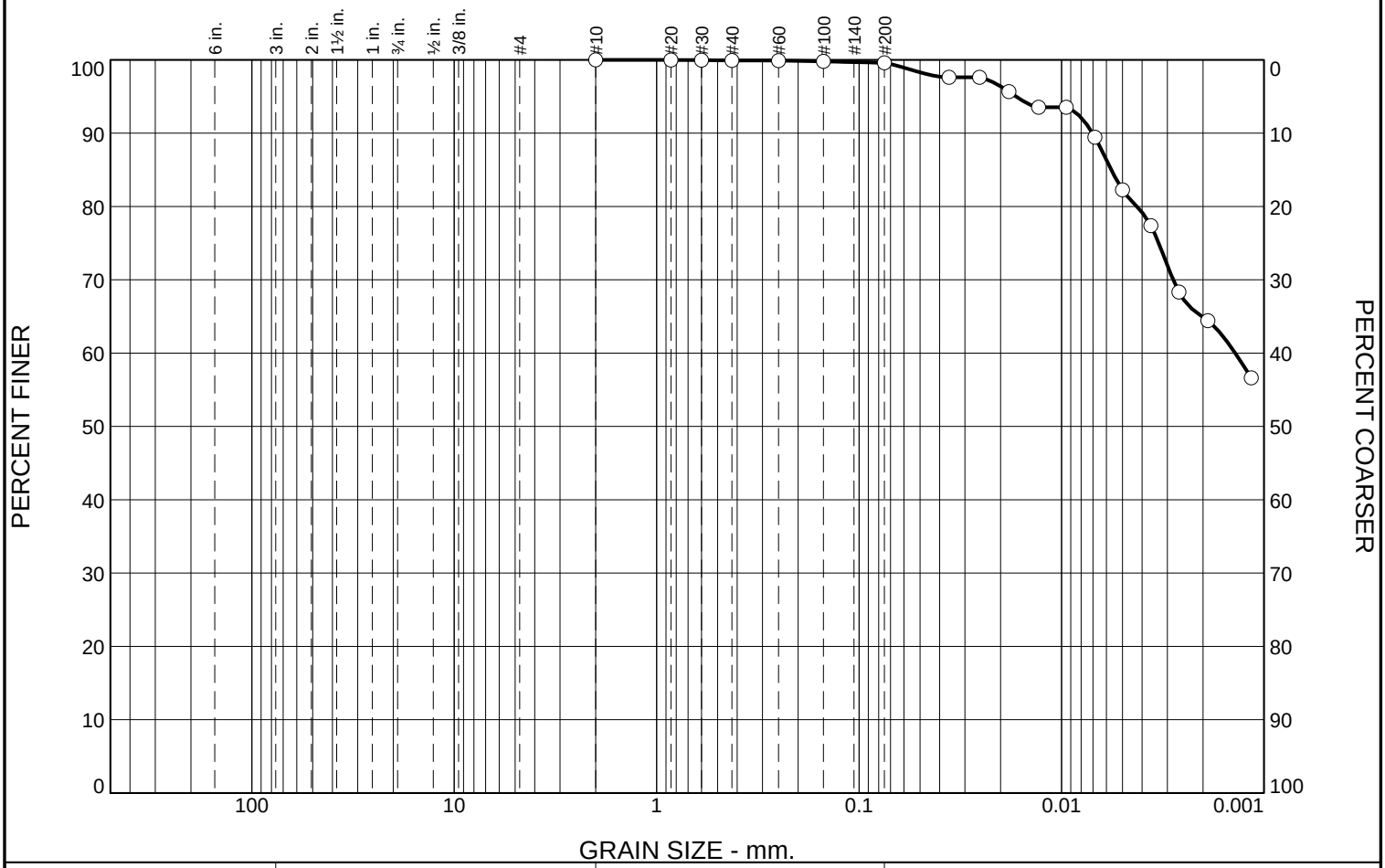
Source of Sample: B-2 **Depth:** 2'-4'

Date: 7/8&9/15

Tolunay-Wong Engineers, Inc. Houston, Texas

Client: Klotz Associates, Inc., Houston, Texas
Project: MKT Spur Connector and Bikeway Bridge, White Oak Bayou Houston, Texas
Project No: 15.13.077 **Figure**

Particle Size Analysis (ASTM D422)



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0.0	0.0	0.1	0.3	34.7	64.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	100.0		
#30	100.0		
#40	99.9		
#60	99.9		
#100	99.8		
#200	99.6		

* (no specification provided)

Material Description

Reddish brown & gray FAT CLAY; claystones

Atterberg Limits

PL= 24 LL= 70 PI= 46

Coefficients

D_{90}= 0.0070 D_{85}= 0.0057 D_{60}= 0.0014
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(53)

Remarks

F.M.=0.00

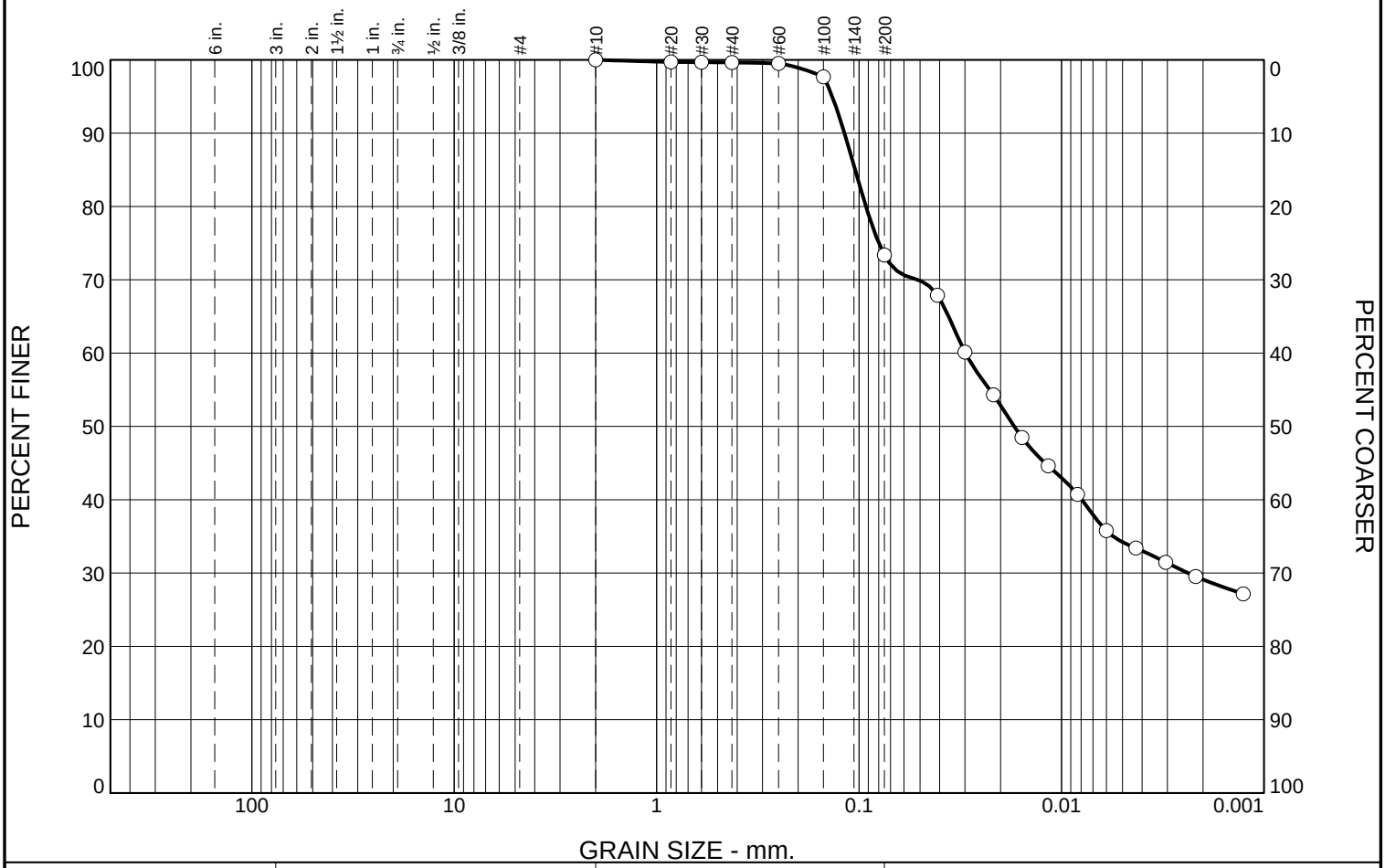
Source of Sample: B-2 **Depth:** 26'-28'

Date: 7/8&9/15

Tolunay-Wong Engineers, Inc. Houston, Texas

Client: Klotz Associates, Inc., Houston, Texas
Project: MKT Spur Connector and Bikeway Bridge, White Oak Bayou Houston, Texas
Project No: 15.13.077 **Figure**

Particle Size Analysis (ASTM D422)



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0.0	0.0	0.4	26.2	44.3	29.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.7		
#30	99.7		
#40	99.6		
#60	99.5		
#100	97.7		
#200	73.4		

* (no specification provided)

Material Description

Tan & gray LEAN CLAY w/ SAND; calcareous nodules

Atterberg Limits

PL= 17 LL= 42 PI= 25

Coefficients

D_{90}= 0.1182 D_{85}= 0.1046 D_{60}= 0.0298
D_{50}= 0.0171 D_{30}= 0.0024 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(17)

Remarks

F.M.=0.03

Source of Sample: B-3 **Depth:** 41'-43'

Date: 6/26/15

Tolunay-Wong Engineers, Inc.
Houston, Texas

Client: Klotz Associates, Inc., Houston, Texas
Project: MKT Spur Connector and Bikeway Bridge, White Oak Bayou Houston, Texas
Project No: 15.13.077 **Figure**

APPENDIX C

WINCORE
DRILLED SHAFT SKIN FRICTION TABLES AND DESIGN CURVES
(BORINGS B-2 AND B-3)

SOIL STRENGTH ANALYSIS

WinCore
Version 3.1

County	Harris	Hole	B-2	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	07.8&9.2015
Control		Station	4+88.28	Grnd. Elev.	19.35 ft
		Offset	52.58' R	GW Elev.	N/A

TAT Values Preferentially Used

Soil reduction factor of 0.7 applied

Strata No.	Elev. (Feet) From	To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	19.4	13.4	0.00	0	0.0	0.00	0.00
2	13.4	9.4	0.00	0	0.0	0.00	0.00
3	9.4	3.4	0.03	0	0.0	0.03	0.16
4	3.4	-1.7	0.06	0	0.0	0.06	0.46
5	-1.7	-6.7	0.07	0	0.0	0.07	0.81
6	-6.7	-11.7	0.24	0	0.0	0.24	2.00
7	-11.7	-16.7	0.31	0	0.0	0.31	3.54
8	-16.7	-21.7	0.36	2592	0.0	0.45	5.81
9	-21.7	-26.7	0.47	3499	0.0	0.61	8.87
10	-26.7	-31.7	0.27	0	0.0	0.27	10.37

Note: Disregard TCP blow count within the upper 10-ft depth

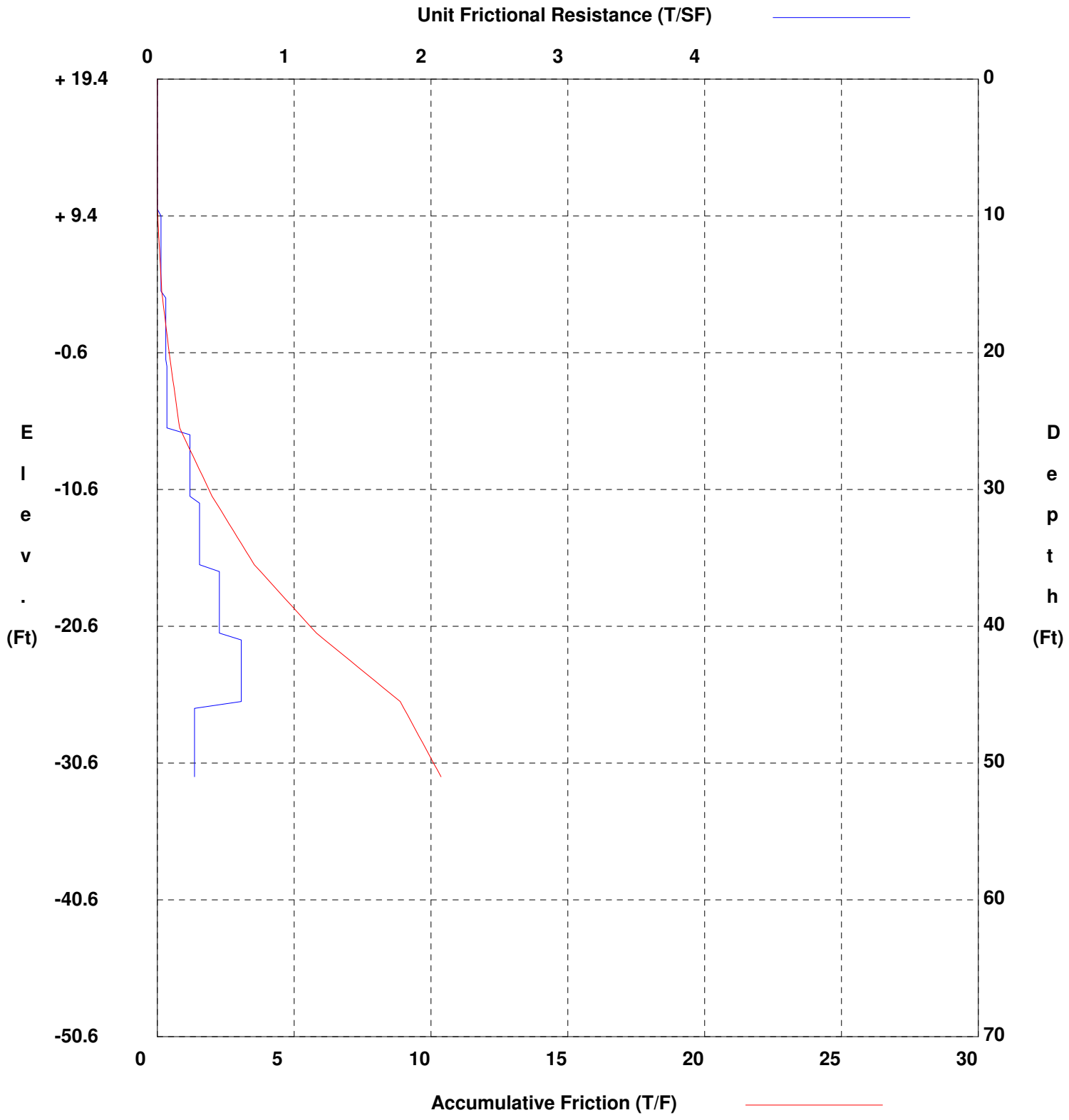
SKIN FRICTION DESIGN

WinCore
Version 3.1

County	Harris	Hole	B-2	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	07.8&9.2015
Control		Station	4+88.28	Grnd. Elev.	19.35 ft
		Offset	52.58' R	GW Elev.	N/A

Drilled Shaft Design: Soil Reduction Factor = 0.7

TAT Friction Values Used



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Note: Disregard TCP blow count within the upper 10-ft depth

SOIL STRENGTH ANALYSIS

WinCore
Version 3.1

County	Harris	Hole	B-3	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	06.26.2015
Control		Station	6+19.28	Grnd. Elev.	20.37 ft
		Offset	38.19' R	GW Elev.	N/A

TAT Values Preferentially Used

Soil reduction factor of 0.7 applied

Strata No.	Elev. (Feet) From	To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	20.4	18.4	0.00	0	0.0	0.00	0.00
2	18.4	14.4	0.00	0	0.0	0.00	0.00
3	14.4	10.4	0.00	0	0.0	0.00	0.00
4	10.4	4.4	0.02	0	0.0	0.02	0.11
5	4.4	-0.6	0.04	0	0.0	0.04	0.28
6	-0.6	-5.6	0.06	0	0.0	0.06	0.59
7	-5.6	-10.6	0.20	0	0.0	0.20	1.59
8	-10.6	-15.6	0.28	0	0.0	0.28	2.99
9	-15.6	-20.6	0.28	0	0.0	0.28	4.39
10	-20.6	-25.6	0.40	2830	0.0	0.50	6.87
11	-25.6	-30.6	0.41	0	0.0	0.41	9.13

Note: Disregard TCP blow count within the upper 10-ft depth

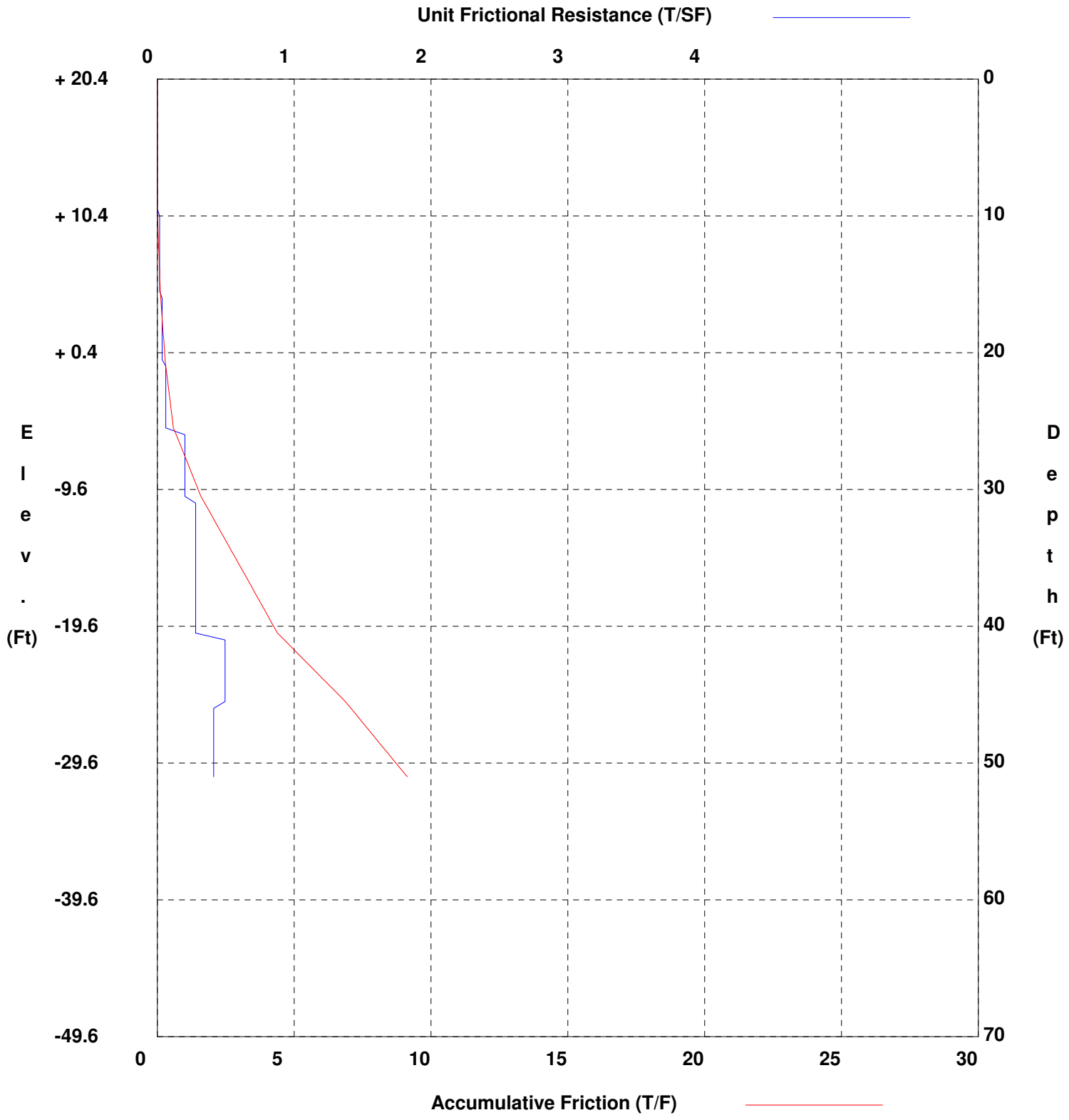
SKIN FRICTION DESIGN

WinCore
Version 3.1

County	Harris	Hole	B-3	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	06.26.2015
Control		Station	6+19.28	Grnd. Elev.	20.37 ft
		Offset	38.19' R	GW Elev.	N/A

Drilled Shaft Design: Soil Reduction Factor = 0.7

TAT Friction Values Used



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Note: Disregard TCP blow count within the upper 10-ft depth

APPENDIX D

WINCORE
DRIVEN PILE SKIN FRICTION TABLES AND DESIGN CURVES
(BORINGS B-2 AND B-3)

SOIL STRENGTH ANALYSIS

WinCore
Version 3.1

County	Harris	Hole	B-2	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	07.8&9.2015
Control		Station	4+88.28	Grnd. Elev.	19.35 ft
		Offset	52.58' R	GW Elev.	N/A

TAT Values Preferentially Used Skin Friction Limit = 1.25 tsf No soil reduction factor applied

Strata No.	Elev. (Feet) From	To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	19.4	13.4	0.00	0	0.0	0.00	0.00
2	13.4	9.4	0.00	0	0.0	0.00	0.00
3	9.4	3.4	0.04	0	0.0	0.04	0.23
4	3.4	-1.7	0.09	0	0.0	0.09	0.66
5	-1.7	-6.7	0.10	0	0.0	0.10	1.16
6	-6.7	-11.7	0.34	0	0.0	0.34	2.86
7	-11.7	-16.7	0.44	0	0.0	0.44	5.06
8	-16.7	-21.7	0.52	2592	0.0	0.65	8.30
9	-21.7	-26.7	0.67	3499	0.0	0.87	12.68
10	-26.7	-31.7	0.39	0	0.0	0.39	14.81

Note: Disregard TCP blow count within the upper 10-ft depth

SKIN FRICTION DESIGN

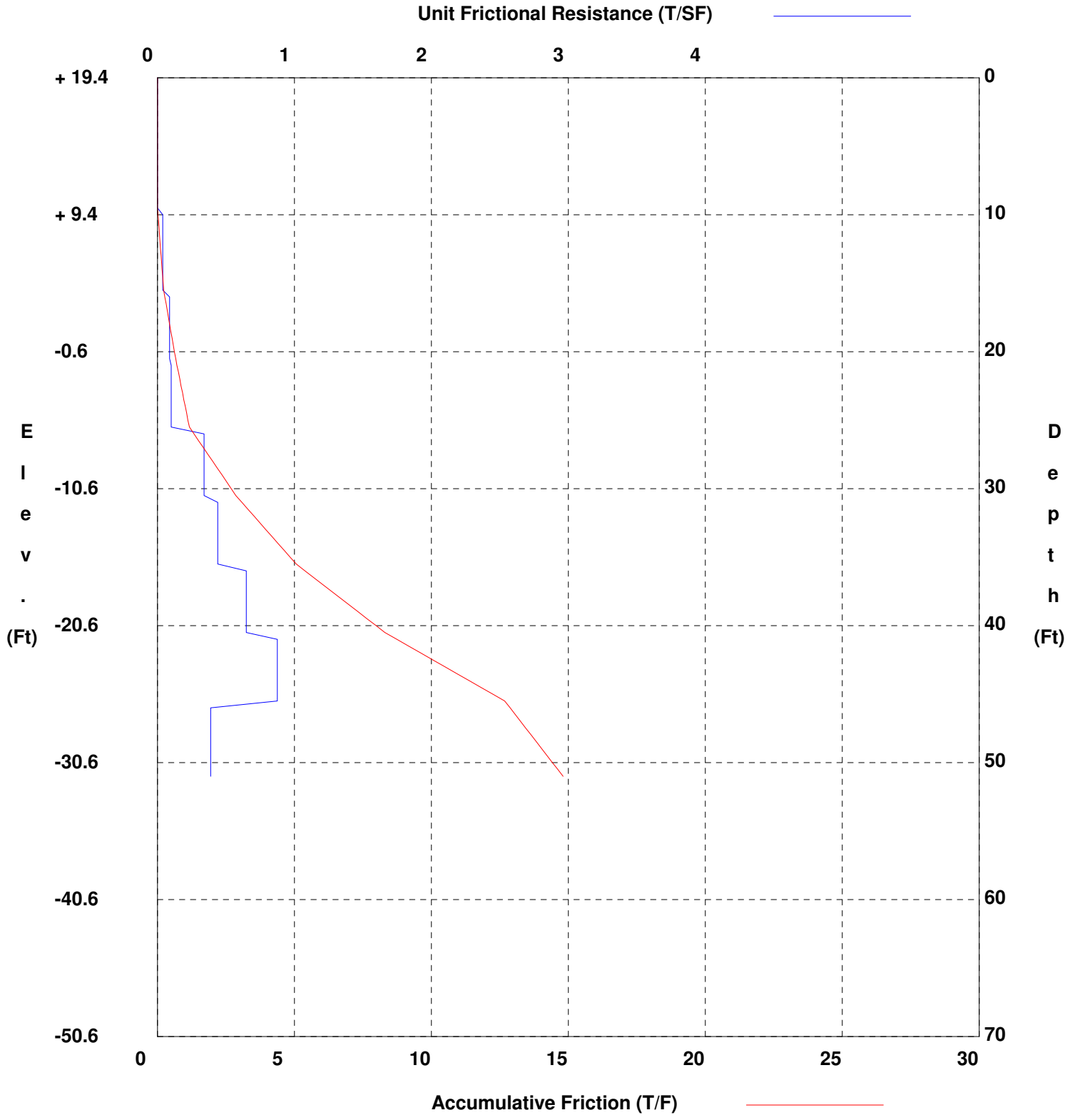
WinCore
Version 3.1

County	Harris	Hole	B-2	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	07.8&9.2015
Control		Station	4+88.28	Grnd. Elev.	19.35 ft
		Offset	52.58' R	GW Elev.	N/A

Piling Design: No Soil Reduction Factor

TAT Friction Values Used

Skin Friction Limit = 1.3 tsf



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Note: Disregard TCP blow count within the upper 10-ft depth

SOIL STRENGTH ANALYSIS

WinCore
Version 3.1

County	Harris	Hole	B-3	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	06.26.2015
Control		Station	6+19.28	Grnd. Elev.	20.37 ft
		Offset	38.19' R	GW Elev.	N/A

TAT Values Preferentially Used **Skin Friction Limit = 1.25 tsf** **No soil reduction factor applied**

Strata No.	Elev. (Feet) From	To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	20.4	18.4	0.00	0	0.0	0.00	0.00
2	18.4	14.4	0.00	0	0.0	0.00	0.00
3	14.4	10.4	0.00	0	0.0	0.00	0.00
4	10.4	4.4	0.03	0	0.0	0.03	0.15
5	4.4	-0.6	0.05	0	0.0	0.05	0.40
6	-0.6	-5.6	0.09	0	0.0	0.09	0.84
7	-5.6	-10.6	0.29	0	0.0	0.29	2.28
8	-10.6	-15.6	0.40	0	0.0	0.40	4.28
9	-15.6	-20.6	0.40	0	0.0	0.40	6.27
10	-20.6	-25.6	0.57	2830	0.0	0.71	9.81
11	-25.6	-30.6	0.59	0	0.0	0.59	13.04

Note: Disregard TCP blow count within the upper 10-ft depth

SKIN FRICTION DESIGN

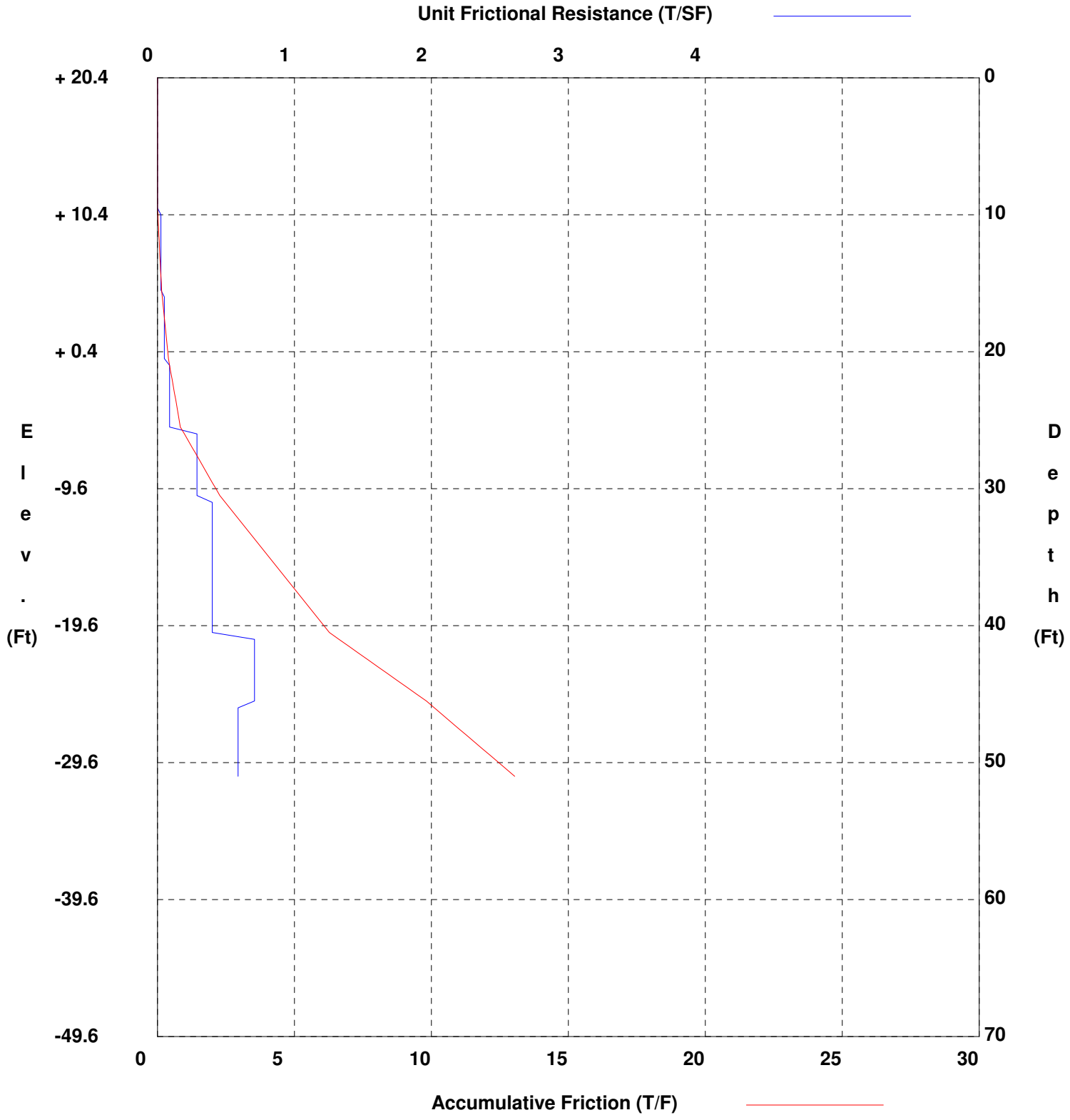
WinCore
Version 3.1

County	Harris	Hole	B-3	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Bridge	Date	06.26.2015
Control		Station	6+19.28	Grnd. Elev.	20.37 ft
		Offset	38.19' R	GW Elev.	N/A

Piling Design: No Soil Reduction Factor

TAT Friction Values Used

Skin Friction Limit = 1.3 tsf



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Note: Disregard TCP blow count within the upper 10-ft depth

APPENDIX E

WINCORE
DRILLED SHAFT SKIN FRICTION TABLES AND DESIGN CURVES
(BORINGS B-1, B-1A AND B-4)

SOIL STRENGTH ANALYSIS

WinCore
Version 3.1

County	Harris	Hole	B-1	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	06.30.2015
Control		Station	-0+29.08	Grnd. Elev.	46.51 ft
		Offset	48.41' L	GW Elev.	N/A

TAT Values Preferentially Used

Soil reduction factor of 0.7 applied

Strata No.	Elev. (Feet) From	To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	46.5	40.5	0.00	0	0.0	0.00	0.00
2	40.5	36.5	0.00	0	0.0	0.00	0.00
3	36.5	30.5	0.19	0	0.0	0.19	1.12
4	30.5	25.5	0.39	0	0.0	0.39	3.05
5	25.5	18.5	0.36	0	0.0	0.36	5.76

Note: Disregard TCP blow count within the upper 10-ft depth

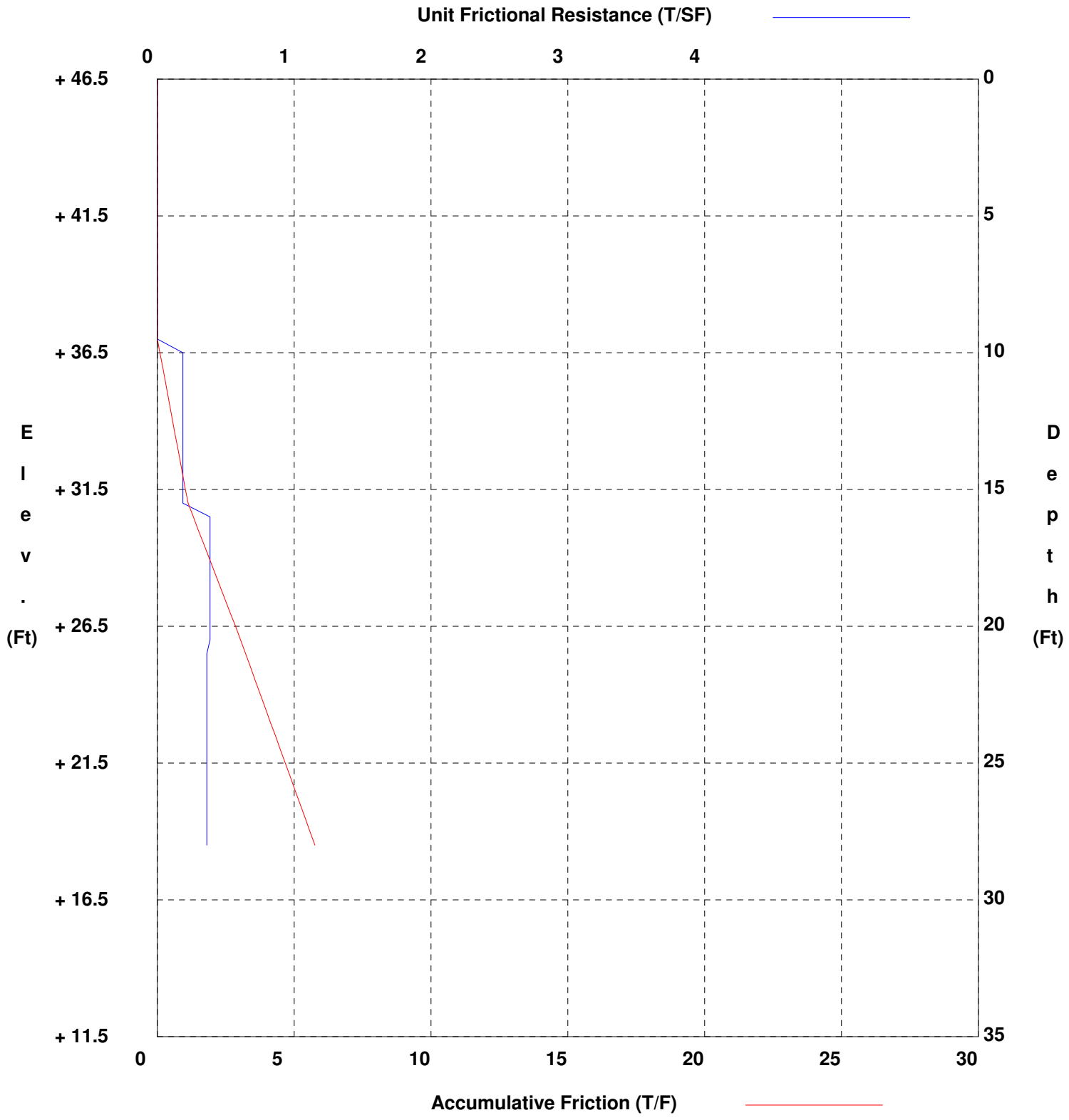
SKIN FRICTION DESIGN

WinCore
Version 3.1

County	Harris	Hole	B-1	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	06.30.2015
Control		Station	-0+29.08	Grnd. Elev.	46.51 ft
		Offset	48.41' L	GW Elev.	N/A

Drilled Shaft Design: Soil Reduction Factor = 0.7

TAT Friction Values Used



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Note: Disregard TCP blow count within the upper 10-ft depth

SOIL STRENGTH ANALYSIS

WinCore
Version 3.1

County	Harris	Hole	B-1A	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	07.9 &10.2015
Control		Station	1+34.39	Grnd. Elev.	23.37 ft
		Offset	41.51' R	GW Elev.	N/A

TAT Values Preferentially Used

Soil reduction factor of 0.7 applied

Strata No.	Elev. (Feet) From	To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	23.4	17.4	0.00	0	0.0	0.00	0.00
2	17.4	13.4	0.00	0	0.0	0.00	0.00
3	13.4	7.4	0.04	0	0.0	0.04	0.24
4	7.4	2.4	0.14	835	0.0	0.15	0.97
5	2.4	-2.6	0.22	0	0.0	0.22	2.09
6	-2.6	-7.6	0.41	2290	0.0	0.40	4.09
7	-7.6	-12.6	0.48	0	0.0	0.48	6.47
8	-12.6	-17.6	0.38	2174	0.0	0.38	8.37
9	-17.6	-22.6	0.35	0	0.0	0.35	10.12
10	-22.6	-27.6	0.50	0	0.0	0.50	12.87

Note: Disregard TCP blow count within the upper 10-ft depth

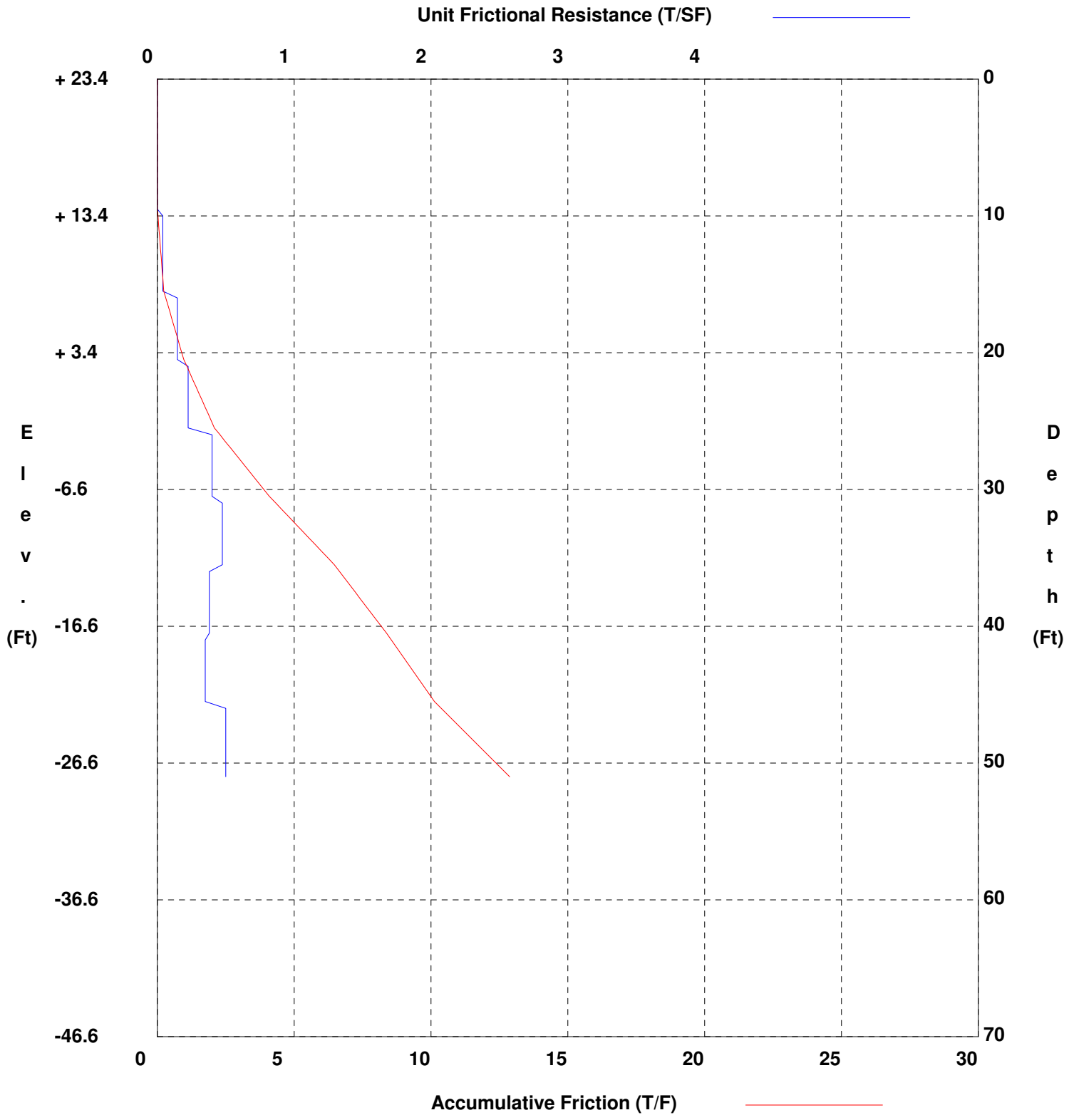
SKIN FRICTION DESIGN

WinCore
Version 3.1

County	Harris	Hole	B-1A	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	07.9 &10.2015
Control		Station	1+34.39	Grnd. Elev.	23.37 ft
		Offset	41.51' R	GW Elev.	N/A

Drilled Shaft Design: Soil Reduction Factor = 0.7

TAT Friction Values Used



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Note: Disregard TCP blow count within the upper 10-ft depth

SOIL STRENGTH ANALYSIS

WinCore
Version 3.1

County	Harris	Hole	B-4	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Pavement	Date	06.26.2015
Control		Station	8+28.32	Grnd. Elev.	21.62 ft
		Offset	11.50' L	GW Elev.	N/A

TAT Values Preferentially Used

Soil reduction factor of 0.7 applied

Strata No.	Elev. (Feet) From	To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	21.6	16.6	0.00	0	0.0	0.00	0.00
2	16.6	10.6	0.09	0	0.0	0.09	0.55
3	10.6	5.6	0.03	0	0.0	0.03	0.70

Note: Disregard TCP blow count within the upper 10-ft depth

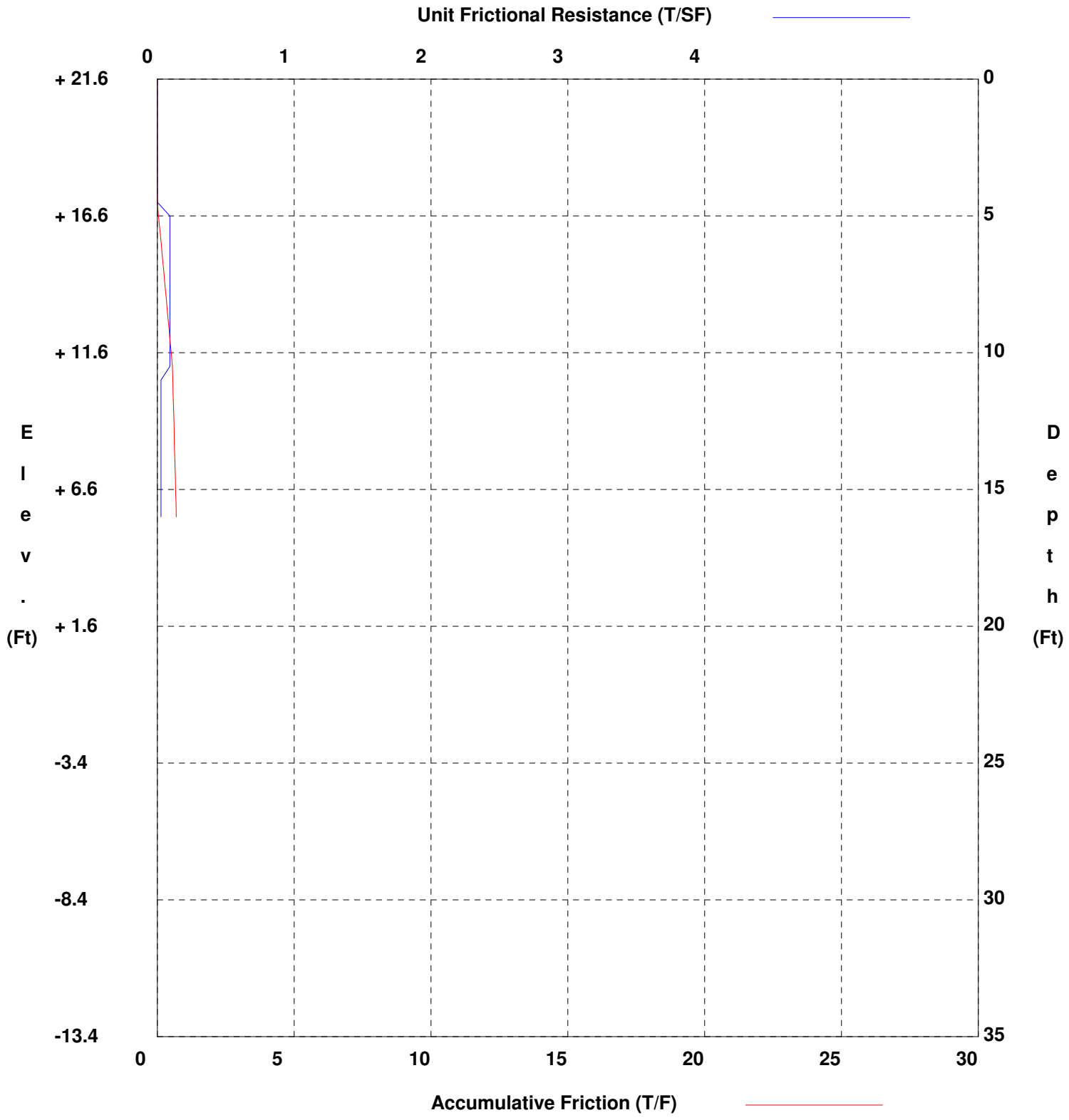
SKIN FRICTION DESIGN

WinCore
Version 3.1

County	Harris	Hole	B-4	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Pavement	Date	06.26.2015
Control		Station	8+28.32	Grnd. Elev.	21.62 ft
		Offset	11.50' L	GW Elev.	N/A

Drilled Shaft Design: Soil Reduction Factor = 0.7

TAT Friction Values Used



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Note: Disregard TCP blow count within the upper 10-ft depth

APPENDIX F

WINCORE
DRIVEN PILE SKIN FRICTION TABLES AND DESIGN CURVES
(BORINGS B-1, B-1A AND B-4)

SOIL STRENGTH ANALYSIS

WinCore
Version 3.1

County	Harris	Hole	B-1	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	06.30.2015
Control		Station	-0+29.08	Grnd. Elev.	46.51 ft
		Offset	48.41' L	GW Elev.	N/A

TAT Values Preferentially Used

Soil reduction factor of 0.7 applied

Strata No.	Elev. (Feet) From	To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	46.5	40.5	0.00	0	0.0	0.00	0.00
2	40.5	36.5	0.00	0	0.0	0.00	0.00
3	36.5	30.5	0.19	0	0.0	0.19	1.12
4	30.5	25.5	0.39	0	0.0	0.39	3.05
5	25.5	18.5	0.36	0	0.0	0.36	5.76

Note: Disregard TCP blow count within the upper 10-ft depth

SKIN FRICTION DESIGN

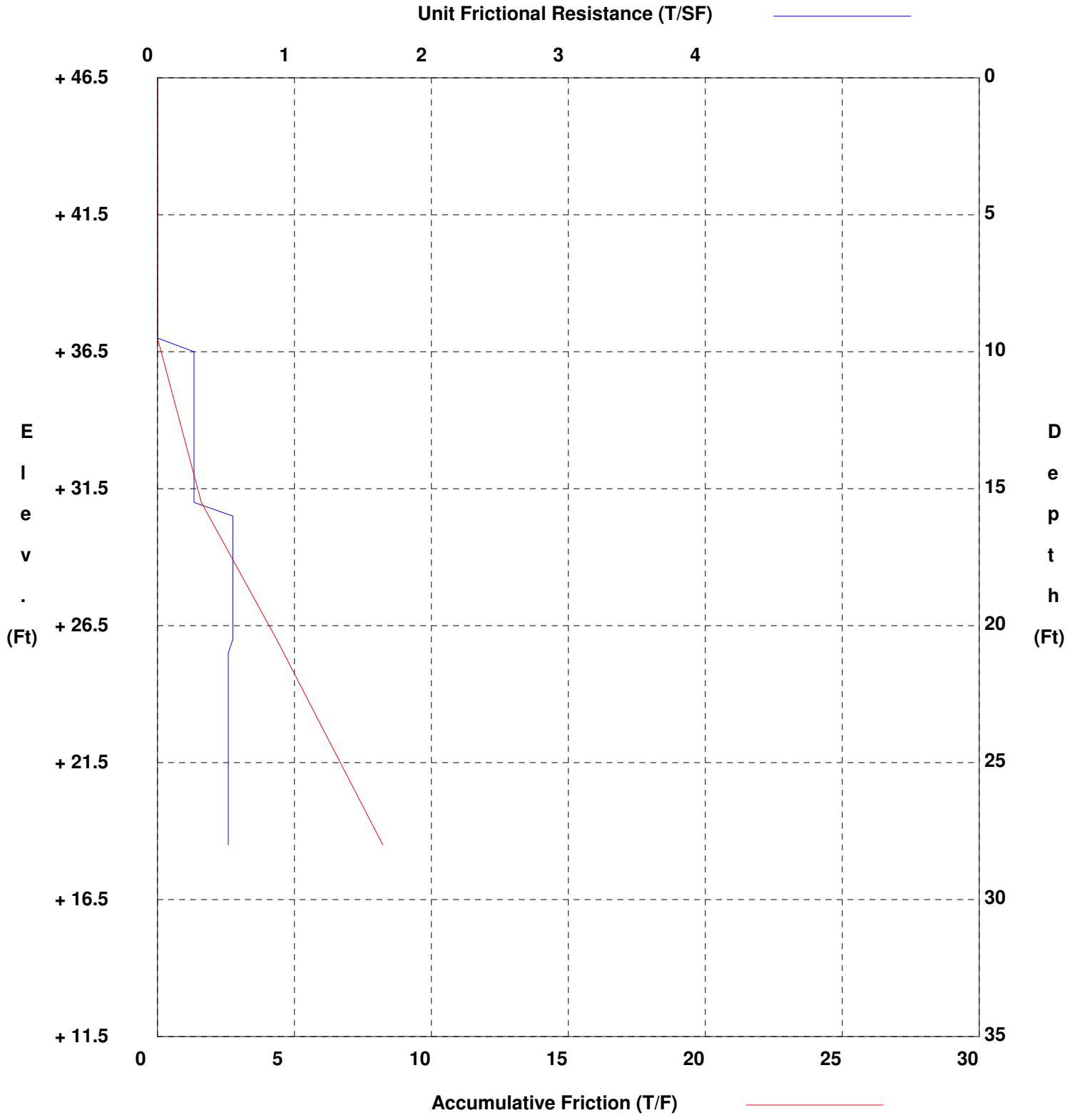
WinCore
Version 3.1

County	Harris	Hole	B-1	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	06.30.2015
Control		Station	-0+29.08	Grnd. Elev.	46.51 ft
		Offset	48.41' L	GW Elev.	N/A

Piling Design: No Soil Reduction Factor

TAT Friction Values Used

Skin Friction Limit = 1.3 tsf



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Note: Disregard TCP blow count within the upper 10-ft depth

SOIL STRENGTH ANALYSIS

WinCore
Version 3.1

County	Harris	Hole	B-1A	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	07.9 &10.2015
Control		Station	1+34.39	Grnd. Elev.	23.37 ft
		Offset	41.51' R	GW Elev.	N/A

TAT Values Preferentially Used Skin Friction Limit = 1.25 tsf No soil reduction factor applied

Strata No.	Elev. (Feet) From	To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	23.4	17.4	0.00	0	0.0	0.00	0.00
2	17.4	13.4	0.00	0	0.0	0.00	0.00
3	13.4	7.4	0.06	0	0.0	0.06	0.34
4	7.4	2.4	0.20	835	0.0	0.21	1.38
5	2.4	-2.6	0.32	0	0.0	0.32	2.98
6	-2.6	-7.6	0.58	2290	0.0	0.57	5.84
7	-7.6	-12.6	0.68	0	0.0	0.68	9.24
8	-12.6	-17.6	0.54	2174	0.0	0.54	11.96
9	-17.6	-22.6	0.50	0	0.0	0.50	14.46
10	-22.6	-27.6	0.71	0	0.0	0.71	18.39

Note: Disregard TCP blow count within the upper 10-ft depth

SKIN FRICTION DESIGN

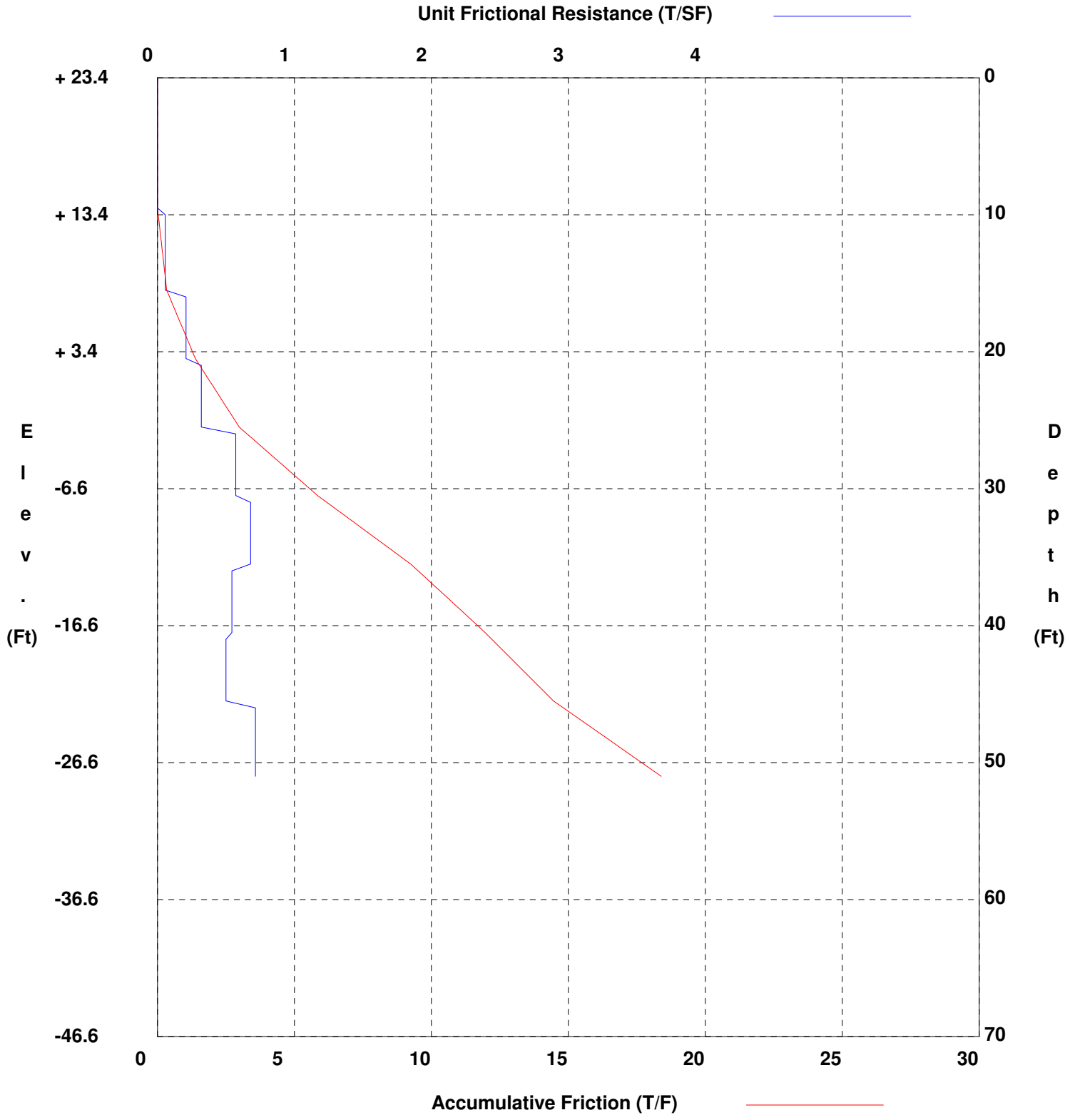
WinCore
Version 3.1

County	Harris	Hole	B-1A	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Retaining Wall	Date	07.9 &10.2015
Control		Station	1+34.39	Grnd. Elev.	23.37 ft
		Offset	41.51' R	GW Elev.	N/A

Piling Design: No Soil Reduction Factor

TAT Friction Values Used

Skin Friction Limit = 1.3 tsf



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Note: Disregard TCP blow count within the upper 10-ft depth

SOIL STRENGTH ANALYSIS

WinCore
Version 3.1

County	Harris	Hole	B-4	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Pavement	Date	06.26.2015
Control		Station	8+28.32	Grnd. Elev.	21.62 ft
		Offset	11.50' L	GW Elev.	N/A

TAT Values Preferentially Used **Skin Friction Limit = 1.25 tsf** **No soil reduction factor applied**

Strata No.	Elev. (Feet) From	To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	21.6	16.6	0.00	0	0.0	0.00	0.00
2	16.6	10.6	0.13	0	0.0	0.13	0.79
3	10.6	5.6	0.04	0	0.0	0.04	0.99

Note: Disregard TCP blow count within the upper 10-ft depth

SKIN FRICTION DESIGN

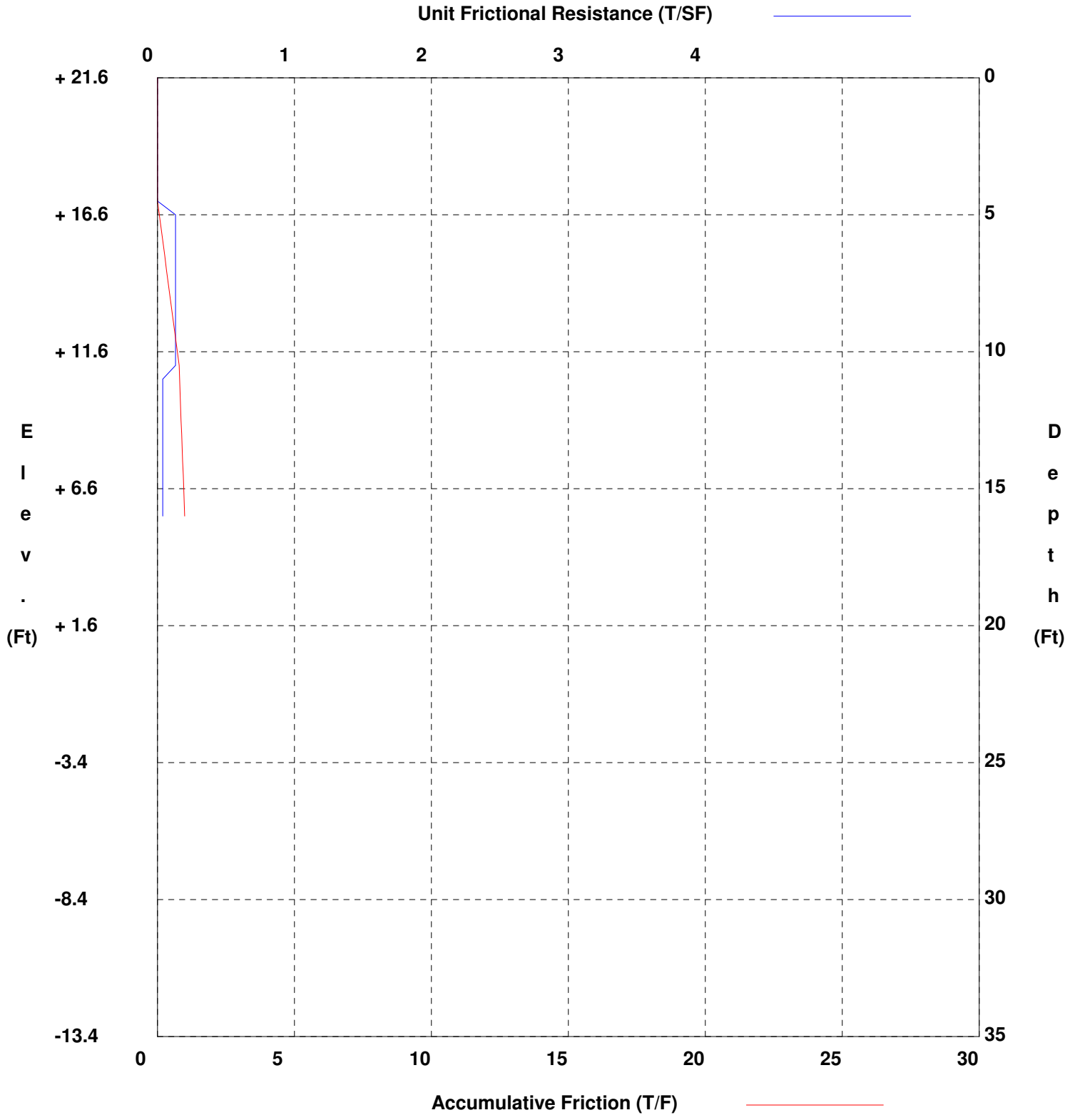
WinCore
Version 3.1

County	Harris	Hole	B-4	District	Houston
Highway	MKT Spur Con. & Bikeway	Structure	Pavement	Date	06.26.2015
Control		Station	8+28.32	Grnd. Elev.	21.62 ft
		Offset	11.50' L	GW Elev.	N/A

Piling Design: No Soil Reduction Factor

TAT Friction Values Used

Skin Friction Limit = 1.3 tsf



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Note: Disregard TCP blow count within the upper 10-ft depth

APPENDIX G

CALCULATIONS FOR REDUCING SOIL STRENGTH PARAMETERS TO ACCOUNT FOR WEATHERING EFFECTS

TWEI Project No.: **15.13.077**

Appendix G

Project: **MKT Spur Connector and Bikeway**
Houston, Texas

Boring: **B-1** **16' -18'** CU w/ pp Test Results

Effective Stress parameters for Long Term Condition

Depth of water = 14 ft PI = 23 γ_T = 132 pcf

γ_w = 62.4 pcf

Depth	Peak Shear Strength (psf)	Peak Cohesion (psf)	Density, pcf	Effective Stress (psf)	ϕ' (degree)	Correction Factor	Mob. Shear Strength, (psf)	ϕ' (degree)	Reduced Cohesion, psf
1	302	240	132	132.0	25	0.75	226	25.0	165
1.5	332	240	132	198.0	25	0.75	249	25.0	157
2	363	240	132	264.0	25	0.75	272	25.0	149
3	425	240	132	396.0	25	0.75	318	25.0	134
4	486	240	132	528.0	25	0.75	365	25.0	118
4.5	517	240	132	594.0	25	0.75	388	25.0	111
5	548	240	132	660.0	25	0.75	411	25.0	103
6	609	240	132	792.0	25	0.75	457	25.0	88
7	671	240	132	924.0	25	0.75	503	25.0	72

Note: CU Triaxial Test results from In-house Chart