FINAL
GEOTECHNICAL INVESTIGATION
BRINGHURST PEDESTRIAN BRIDGE OVER UPRR
WBS NO. N-000420-0045-3 / CSJ NO 0912-72-289
HOUSTON, TEXAS

SUBMITTED TO
SES HORIZON CONSULTING ENGINEERS, INC.
10101 SOUTHWEST FREEWAY; SUITE 400
HOUSTON, TEXAS 77074

SUBMITTED BY
HVJ ASSOCIATES, INC.
HOUSTON, TEXAS
JULY 17, 2013

REPORT NO. HG1218100
KEY MAP NO. 494 F

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July 17, 2013

Mr. Epifanio (Epi) Salazar, Jr., P.E.
SES Horizon Consulting Engineers, Inc.
10101 Southwest Freeway; Suite 400
Houston, Texas 77074

Re: Geotechnical Investigation
Bringhurst Pedestrian Bridge over UPRR
Houston, Texas
Owner: City of Houston
WBS No. N-000420-0045-3 / CSJ NO: 0912-72-289
HVJ Report No. HG1218100

Dear Mr. Epifanio:

Submitted herein is the final geotechnical report for the above referenced project. The study was performed in accordance with our proposal number HG1218100 dated December 19, 2012 (Revised December 28, 2012) and is subject to the limitations presented in this report.

It has been a pleasure to work for you on this project and we appreciate the opportunity to be of service. Please notify us if there are questions or if we may be of further assistance.

Sincerely,

HVJ ASSOCIATES, INC.
Texas Firm Registration No. F-000646

07-17-2013
Ziad AlAawar, PhD, PE
Project Manager

STATE OF TEXAS
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PROFESSIONAL ENGINEER

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Copies submitted: 2

The seal appearing on this document was authorized by Ziad AlAawar, PE 107115 on July 17, 2013. Alteration of a sealed document without proper notification to the responsible engineer is an offense under the Texas Engineering Practice Act.

The following lists the pages which complete this report:

- Main Text – 14 pages
- Plates – 3 pages
- Appendix A – 6 pages
- Appendix B – 7 pages
- Appendix C – 5 pages
- Appendix D – 2 pages
- Appendix E – 2 pages

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PLATES

APPENDICES

1 EXECUTIVE SUMMARY

HVJ Associates, Inc. was retained by SES Horizon Consulting Engineers, Inc. to provide geotechnical investigation for the proposed pedestrian bridge over UPRR facilities at Bringhurst in Houston, Texas. The purpose of this study is to provide geotechnical foundation design recommendations for the proposed pedestrian bridge construction. The geotechnical investigation, laboratory testing and report preparation were performed in accordance with TxDOT and Chapter 11 of the City of Houston Department of Public Works and Engineering Infrastructure Design Manual, July 2012. Subsurface conditions in the subject area were determined by drilling two soil borings to a depth of 60 feet each. Site vicinity map and plan of borings are presented on Plates 1 and 2 of the report, respectively.

Based on the subsurface conditions revealed by the soil borings, the findings and recommendations of this report are summarized below:

1. Borings BH-1 and BH-2. The subsurface soils generally comprise of soft to very stiff fat clays at the top 10 feet underlain by stiff to hard lean clays to 30 feet deep. Slightly compact to very dense sands were observed after 30 feet to the termination depth of both the borings. Calcareous nodules were encountered at various depths throughout the borings.

2. Groundwater was encountered at 32.0 feet at BH-1 and was not observed at BH-2 during the drilling operations. The 5 and 10 minutes water level readings were recorded at 28.0 and 27.75 feet at BH-1; while BH-2 was caved in before the readings were obtained. No piezometers were installed for this study.

3. A literature review of surface faults was made from published reports. The primary objective of this review was to evaluate available information from published reports and open file reports. Based on our review, Pecore fault is located at about 2.0 miles southwest of the project site. We believe that faulting should not impact the project site; however, it should be noted that unmapped faults that could impact the project site might exist within the project area. A detailed fault study was not within the scope of this study.

4. Driven pile and drilled shaft capacities were calculated at each boring location using the procedures described in the Texas Department of Transportation Geotechnical Manual dated December, 2012. The method described was adapted to Houston District practice. Allowable skin friction curves for drilled shafts along with end bearing capacity curves for each boring are presented in Appendix B. Allowable skin friction curves for driven piles are presented in Appendix C.

Please note that this executive summary does not fully relate our findings and opinions. These findings and opinions are only presented through our full report.

2 INTRODUCTION

2.1 Study Limits

HVJ Associates, Inc. was retained by SES Horizon Consulting Engineers, Inc. to provide geotechnical investigation for the proposed pedestrian bridge over UPRR facilities at Bringhurst in Houston, Texas. The UPRR facilities intersect Bringhurst at grade level as well as other intersecting streets within the project area. The UPRR facilities do not allow for vehicular traffic and pedestrian traffic to traverse Bringhurst and other intersecting streets when the train is in queue and stationary as it approaches an existing UPRR switch yard facility located East of Bringhurst. The proposed project allows for preliminary and final engineering of pedestrian bridge crossing alternatives, so that pedestrian traffic can safely traverse across the UPRR facilities from the surrounding residential area to the existing public schools and businesses located within the project area.

Subsurface conditions in the subject area were determined by drilling two soil borings to a depth of 60 feet each. Site vicinity map and plan of borings are presented on Plates 1 and 2 of the report, respectively.

2.2 Geotechnical Study Program

The primary objectives of this study were to gather information on subsurface conditions at the project site and to develop design and construction recommendations for the proposed bridge. The objectives were accomplished by:

1. Drilling and field testing two soil borings to a depth of 60 feet each to investigate soil stratigraphy and to obtain samples for laboratory testing,

2. Performing laboratory tests to determine physical characteristics of the soils, and

3. Performing design charts and engineering analyses to develop design guidelines and recommendations for the bridge foundations.

Subsequent sections of this report contain descriptions of the field exploration, laboratory testing program, general site and subsurface conditions, design charts and curves.

3 FIELD EXPLORATION

3.1 General

The field exploration program undertaken for the project was performed on May 6 and May 7 of 2013. Subsurface conditions at the project site were evaluated by drilling and sampling two soil borings to a depth of 60 feet below the existing ground surface. Drilling and sampling were performed in accordance with TxDOT procedures. Both the borings were backfilled with cement grout by tremie method in accordance with the City guidelines. The borings were drilled using dry auger and wet rotary drilling techniques using a truck-mounted drilling rig. The boring logs are presented in Appendix A.

Two borings were drilled with the following coordinates and depths:

- BH-1; 60 feet deep, Northing: 13,850,750.181, Easting: 3,131,499.917

- BH-2; 60 feet deep, Northing: 13,850,837.449, Easting: 3,131,465.517

3.2 Sampling Methods

Cohesive soil samples were obtained continuously using a 3-inch diameter thin walled tube pushed into soil in general accordance with the ASTM D1587 standard. Granular cohesionless soils were sampled in accordance with the ASTM D1586 standard. Each sample was removed from the sampler in the field, carefully examined and then classified using the Visual-Manual Procedure for Description and Identification of Soils in accordance with TxDOT Test Method Tex-141-E. The shear strength of the cohesive soils was estimated by TxDOT cone penetrometer in the field. Suitable portions of each sample were sealed and packaged for transportation to our laboratory.

The TxDOT cone penetrometer test was performed at approximately 5-foot intervals. 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 for a distance of 2 feet. Then it is driven for two consecutive 6-inch increments, and the blow counts for each increment are noted. In hard materials, the cone is driven with the resulting penetration in inches recorded for the 50 blows. 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 presented in Appendix A.

Detailed descriptions of the soils encountered in the borings are given in the boring logs presented in Appendix A. A key to the terms and symbols used for soil classification on the boring logs is also presented in Appendix A.

3.3 Groundwater Observations

Groundwater was encountered at 32.0 feet at BH-1 and was not observed at BH-2 during the drilling operations. The 5 and 10 minutes water level readings were recorded at 28.0 and 27.75 feet at BH-1; while BH-2 was caved in before the readings were obtained..

It should be noted that water levels determined during drilling may not accurately reflect the true groundwater conditions, and therefore should only be considered as approximate. These readings fluctuate seasonally and in response to rainfall.

4 LABORATORY TESTING

Selected soil samples were tested in the laboratory to determine applicable physical and engineering properties. All tests were performed according to the relevant TxDOT standards or ASTM Standards. The laboratory program included moisture content, Atterberg limits, percent finer than No. 200 sieve, pocket penetrometer and unconsolidated undrained (UU) triaxial compression tests.

The moisture content, Atterberg limits and percent finer than No. 200 sieve results were utilized to verify field classifications by the Unified Soils Classification System. The unconsolidated undrained (UU) triaxial compression tests were performed to obtain the undrained shear strength of the soil. The type and number of tests performed for this investigation are summarized in the following table:

Table 4-1 – Type and Number of Laboratory Tests

Laboratory Test Name	Number of Tests
Moisture Content (Tex-103E)	37
Liquid Limit (Tex-104E)	11
Plastic Limit (Tex-105E)	11
Unconsolidated Undrained Triaxial (ASTM D2850)	9

Laboratory Test Name	Number of Tests
Pass #200 Sieve (Tex-111E)	18

The laboratory test results are presented on the boring logs in Appendix A.

5 SITE CHARACTERIZATION

5.1 General Geology

There are two major surface geological formations that exist in the Houston area: the Beaumont formation and the Lissie formation. The Beaumont formation is a relatively younger formation generally found to the southeast of the Lissie formation. The Beaumont formation dips southeastward and extends beneath beach sand and waters of the Gulf of Mexico as far as the continental shelf. The project site is located in an area where the Beaumont formation is typically encountered.

The Beaumont formation was deposited on land near sea level in flat river deltas and in inter-delta regions. Soil deposition occurred in fresh water streams and in flood plains (as backwater marsh and natural levees). The courses of major streams and deltaic tributaries changed frequently during the period of deposition, generating within the Beaumont clay a complex stratification of sand, silt and clay deposits. Frequently, stream courses were diverted significant distances from a given point in a backwater marsh, and the water overlying the soil would evaporate since it was cut off from a drainage path. Such water, which would be highly alkaline, would precipitate large nodules of calcium carbonate (calcareous nodules) throughout the surface of evaporation. With the coming of the Second Wisconsin Ice Age, the nearby sea withdrew, leaving the formation several hundred feet above sea level and permitting the soil to desiccate. The process of desiccation compressed the clays in the formation such that they became significantly overconsolidated to a large depth. In addition to preconsolidating the soil, the process of desiccation, together with the later rewetting, produced a network of fissures and slickensides that are now closed but which represent potential planes of weakness in the soil.

5.2 Geologic Faulting

The tectonic history of the Texas Gulf Coast includes a relatively stable depositional cycle since the Cretaceous Period (about 65 million years). During this period the area has been subjected to deposition of clays, silts, and sands resulting in over 30 thousand feet of sedimentary rocks. Underlying this clastic sequence are salt formations, which have migrated upwards to produce the typical salt dome features associated with the Texas Gulf Coast. In conjunction with salt movement, dewatering and compaction of some of the deeper sediments in the basin have resulted in the development of growth faults.

A literature review of surface faults was made from published reports. The primary objective of this review was to evaluate available information from published reports and open file reports. Based on our review, Pecore fault is located at about 2.0 miles southwest of the project site. We believe that faulting should not impact the project site; however, it should be noted that unmapped faults that could impact the project site might exist within the project area. A detailed fault study was not within the scope of this study.

5.3 Soil Stratigraphy

Our interpretation of soil and groundwater conditions at the project site is based on information obtained at the boring locations only. This information has been used as the basis for our conclusions and recommendations. Significant variations at areas not explored by the project borings may require reevaluation of our findings and conclusions.

Based on our field investigation, the subsurface soils generally comprise of soft to very stiff fat clays at the top 10 feet underlain by stiff to hard lean clays to 30 feet deep. Slightly compact to very dense sands were observed after 30 feet to the termination depth of both the borings. Calcareous nodules were encountered at various depths throughout the borings. Detailed descriptions of the materials encountered in the borings are given on the boring logs presented in Appendix A. Key to the terms and symbols used for soil classification on the boring logs is given in Appendix A.

6 BRIDGE FOUNDATION RECOMMENDATIONS

6.1 General

We understand that the project involves the construction of pedestrian bridge over UPRR facilities located at Bringhurst in Houston, Texas.

6.2 Recommended Foundation Types

Due to the loads expected and the nature of the soils at the project site, deep foundations are generally required. Driven piles and drilled shafts are two common types of deep foundations used in the Houston and surrounding areas. Drilled shafts are usually acceptable, but driven piles are often used at sites where there is a considerable amount of sand present, as this eliminates the difficulties associated with sloughing of sand in drilled shaft excavations. Allowable skin friction curves for drilled shafts and driven piles were calculated using the Wincore program and are presented in Appendix B and Appendix C, respectively.

6.3 Analysis Criteria

The drilled shaft capacities were calculated using the procedures described in the Texas Department of Transportation (TxDOT) Geotechnical Manual dated December, 2012. The method described was adapted to Houston District practice as documented in the Sep. 12, 1988 memo to District 12 Designers and Laboratory Geotechnical Engineers titled Guidelines for Foundation Design.

The following summarizes the Houston District adaptations to the procedures described in the Geotechnical Manual based on the memo and comments from Houston District Laboratory staff.

- Drilled shafts are designed for both skin friction and end bearing. Allowable unit end bearing for drilled shafts is assumed to be a maximum of 2 tsf unless the drilled shaft diameter is 24 inches or smaller. For drilled shafts 24 inches in diameter or smaller end bearing is neglected. (Based on comments from the TxDOT Houston District Laboratory staff we understand that it is acceptable to compute unit end bearing for shafts greater than 48 inches in diameter based on the TCP blow counts and Figure 5-2 from the Geotechnical Manual, in which case the 2 tsf limit does not apply).

- Skin friction calculated for a drilled shaft is reduced by a soil reduction factor of 0.7 this reduces the maximum allowable unit skin friction to 0.875 tsf.

- For all drilled shafts, skin friction in fill should be disregarded.

- For drilled shaft foundations the skin friction in the upper ten feet should be disregarded due to moisture fluctuations and non-reliable friction transfer.

The Wincore computer program that incorporates TxDOT standard procedures was used to compute the allowable unit and accumulative skin friction for straight-sided drilled shafts for the project structures. A soil reduction factor of 0.7 was used to obtain the skin friction curves for the drilled shafts.

6.4 Drilled Shaft and Driven Pile Axial Capacity

Wincore was developed and is distributed by TxDOT. Soil Strength Analysis table printouts from the Wincore program are also presented in those appendices. The curves were developed for each boring location. The allowable values shown include a factor of safety of 2 according to the TxDOT Geotechnical Manual.

Allowable compressive capacity due to skin friction may be calculated from the curves by reading the accumulative skin friction value corresponding to the tip penetration of the shaft/pile and multiplying the value by the shaft/pile perimeter.

For drilled shaft foundations the allowable skin friction capacity for the upper 10 feet should be disregarded. For driven piles at least the upper 5 feet should be disregarded.

For drilled shafts an allowable end bearing capacity should be calculated by multiplying the shaft end area by the allowable unit end bearing pressure presented in Appendix B. The allowable end bearing capacity should be added to the allowable skin friction capacity (adjusted to remove the appropriate disregard depth) to determine the total allowable drilled shaft compressive capacity. The maximum allowable drilled shaft service load should be determined in accordance with Chapter 5; Section 3 of the TxDOT Geotechnical Manual dated December, 2012.

For driven piles, the total allowable compressive capacity is equal to the total allowable skin friction capacity adjusted to remove the appropriate disregard depth. It should be noted that 14-inch square precast concrete piles should not be used due to breakage problems, and 24-inch square precast concrete piles should not be used due to limited availability.

6.5 Lateral Capacity

Foundation elements often have to withstand significant lateral loads in addition to axial loads. Wind forces on bridges are forms of lateral loading. Lateral loads on a drilled shaft or driven piles will be countered by the mobilization of resistance in the surrounding soils as the shaft deflects. The lateral load capacity of the shaft or pile, therefore, will depend on its relative stiffness, and the strength of the surrounding soils.

A rational analysis of a problem involving lateral loading on a pile or shaft must consider the interaction of the soil and the structure. Equilibrium of forces and compatibility of displacements throughout the total system are the two fundamental conditions that are to be satisfied in the analysis.

For vertical piles or shafts subjected to small and transient wind or traction loads, it may be assumed that they can sustain horizontal loads of up to 10 kips per foot of pile/shaft diameter or width, and a transient load of 20 kips per foot of diameter or width. These values are allowable capacities, but do not restrict lateral deflection to a given value. Deflection associated with these loads should be within acceptable limits for bridge structures.

If higher lateral loads are anticipated, battered piles should be considered. If the higher lateral loads must be resisted with vertical piles/shafts, a more detailed study should be done to provide lateral load capacity curves.

Lateral load analysis was beyond the scope of this study and should be performed using software's such as LPILE, etc. The input parameters for lateral load analysis are presented in the table below (*LPILE Plus 4.0 for Windows Users Manual*).

Table 6-1 - Input Parameters for Lateral Load Analysis

Boring Number	Estimated Water Table Depth (ft)	Material Type	Depth (ft)		k, Lateral Modulus of Subgrade Reaction (lbs/in³)		Undrained Soil Shear Strength/ Internal Friction Angle		Effective Density (pci)	ε_{50}
			Top	Bottom	Static	Cyclic	ϕ (deg)	C_u (psf)		
BH-1	30 feet	Soft Clay	0	5	70	-	-	700	0.070	0.01
		Stiff Clay	5	10	500	200	-	1100	0.070	0.007
		Stiff to very stiff Clay	10	30	1000	400	-	3500	0.073	0.005
		Compact Sand	30	40	125	-	30	-	0.036	N/A
		Slightly compact Sand	40	55	60	-	28	-	0.036	N/A
		Dense Sand	55	60	125	-	34	-	0.036	N/A
BH-2	30 feet	Soft to very stiff Clay	0	6	100	-	-	900	0.073	0.01
		Stiff to hard Clay	6	30	1000	400	-	3000	0.074	0.005
		Compact Sand	30	40	125	-	30	-	0.036	N/A
		Compact to very dense Sand	40	60	125	-	34	-	0.036	N/A

6.6 Group Effects

Groups of shafts/piles should have a center-to-center spacing of at least 2.5D when designing foundations using one row group of shafts/piles and 3D for foundations using two or more rows of shafts/piles where D is the diameter of the shaft/pile. For greater spacing, the total capacity will be equal to the sum of the capacities of the individual shafts/piles in the group. The group capacity may be less than the sum of individual capacities at closer spacing. If spacing smaller is planned, HVJ Associates, Inc. should be contacted to assess group capacity.

6.7 Drilled Shaft Construction Recommendations

Drilled shaft construction and installation should follow TxDOT Standard Specification Item 416, TxDOT Construction Bulletin C-9, and ACI 336.1-89. Slurry displacement methods for drilled shaft construction are allowed under TxDOT Standard Specifications. 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. Before placing concrete, the shaft bottom should be cleaned out with a drilling bucket in order to remove any sediments that 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.

3. 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.

4. 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.

5. 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 will be left in place so we may provide revised shaft capacity calculations.

6. Shaft excavations should not be made within three shaft diameters (edge to edge) of shafts that have been concreted within the last 24 hours.

6.8 Driven Pile Construction Recommendations

Methods and effects of pile installation are important considerations in the choice and design of pile foundation systems. Piles normally experience their largest stresses during installation. Pile and soil properties, embedment length and driving equipment are a few of the variables that must be considered. Piling should be installed in accordance with TxDOT Standard Specification Items 404 and 409, TxDOT Construction Bulletin C-8, and ACI 543R-74.

We recommend that wave equation analyses be performed as a basis for selecting the installation equipment and procedures based on their ability to ensure installation to the required penetration without damage to the piles. In addition, the wave equation analyses should be used to determine an acceptable blow count at final penetration to be used to field verify the design pile capacity. The following guidelines should be followed when installing precast concrete piles.

1. Adequate cushioning material should be provided between the pile driver and the pile head. A six to twelve-inch thick cushion of softwood is usually adequate for piles that are over 50 feet long. Cushioning material condition should be carefully observed and the cushion must be changed if excessive compression occurs or at least every three piles.

2. Based on our experience, piles can usually be safely driven to about 8 blows per inch. Consistent blow counts above 100 blows per foot are not advisable. Difficult driving conditions are expected at some of the boring locations as discussed in Section 6.3.

3. Driving aids such as pilot holes may be needed to advance piles. Pilot holes can also be used to assist in pile alignment. Pilot holes, if used, should be developed using wet rotary or auger drilling methods. Jetting is not recommended for construction of pilot holes. Pilot holes constructed in accordance with TxDOT Standard Specification Section 404.3 can be large enough to cause a reduction in the skin friction capacity of square piling. The specification requirement allows a pilot hole depth of up to 5 feet, deeper pilot holes are allowed with the approval of the Engineer. Since the first 5 feet is within the disregard depth discussed in Sections 6.2 and 6.3 for driven piles there should be no impact on the allowable pile capacity. However, pilot holes constructed in accordance with Section 404.3 that extend deeper than five feet could reduce the allowable pile capacity depending on the diameter of the hole.

   We recommend that we be contacted to determine the potential impact on pile capacity if pilot holes for square piles that exceed two-thirds of the pile width extending deeper than 5 feet are used during construction.

4. The hammer, cushion and pile should be designed such that installation to design specifications can be realized with no damage to the pile.

5. The top of the pile should be perpendicular to the longitudinal axis in order to minimize damage to the pile edges during driving.

6. At the beginning of driving, when driving through relatively soft soils, or when driving through a pilot hole, driving stresses should be reduced by shortening the hammer stroke so that the pile will be less likely to develop damage due to reflected tensile stresses.

7. The pile driving cap should fit loosely around the top of the pile so that torsional stresses do not develop in the pile. The cap should, however, be able to control the alignment of the pile.

8. Prior to driving, the pile should be properly aligned and held with fixed leads. The pile should not be realigned once driving has begun.

9. Clays and some silty soils tend to undergo a reduction in strength during pile driving and regain strength after pile installation. This phenomenon is usually referred to as freeze or set-up. The number and duration of delays in the driving program should be minimized so as to control the effect of set-up and pile heaving. Pilot holes will also minimize this effect.

10. Piles should be handled so as to avoid tensile cracks and impact damage

7 SITE PREPARATION

Stripped areas should be appropriately graded and shaped to prevent ponding of water. Pumping may occur if the site becomes wet. All subgrade soils should be proof rolled in accordance with TxDOT Standard Specifications prior to placement of fill or paving. Fill material that is used should be placed and compacted in accordance with TxDOT Standard Specifications.

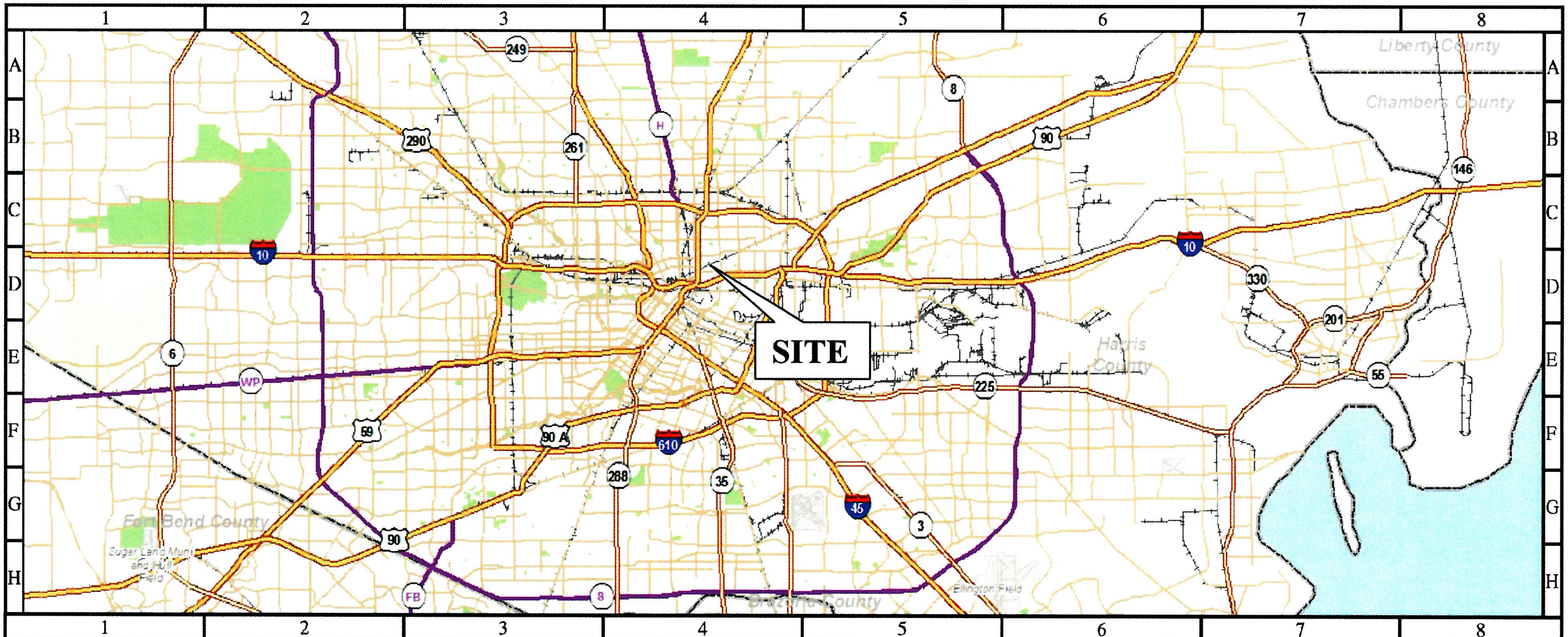
8 DESIGN REVIEW

HVJ Associates should be retained to review the final design plans and specifications for this project. During all excavation, grading and foundation construction phases of this project, HVJ should provide the materials testing verification and observation services so our geotechnical recommendations may be interpreted and implemented correctly.

9 LIMITATIONS

This investigation was performed for the exclusive use of SES Horizon Consulting Engineers, Inc. and City of Houston for the pedestrian bridge over UPRR facilities at Bringhurst in Houston, Texas. HVJ Associates, Inc. has endeavored to comply with generally accepted geotechnical engineering practice common in the local area. HVJ Associates, Inc. makes no warranty, expressed or implied. The analyses and recommendations contained in this report are based on data obtained from subsurface exploration, laboratory testing, the project information provided to us and our experience with similar soils and site conditions. The methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Samples cannot be relied on to accurately reflect the strata variations that usually exist between sampling locations. Should any subsurface conditions other than those described in our boring logs be encountered, HVJ Associates, Inc. should be immediately notified so that further investigation and supplemental recommendations can be provided.

PLATES



1 inch = 23146 feet

CITY OF HOUSTON
Department of Public Works and Engineering
Geographic Information & Management System (GIMS)

DISCLAIMER: THIS MAP REPRESENTS THE BEST INFORMATION AVAILABLE TO THE CITY. THE CITY DOES NOT WARRANT ITS ACCURACY OR COMPLETENESS. FIELD VERIFICATIONS SHOULD BE DONE AS NECESSARY.

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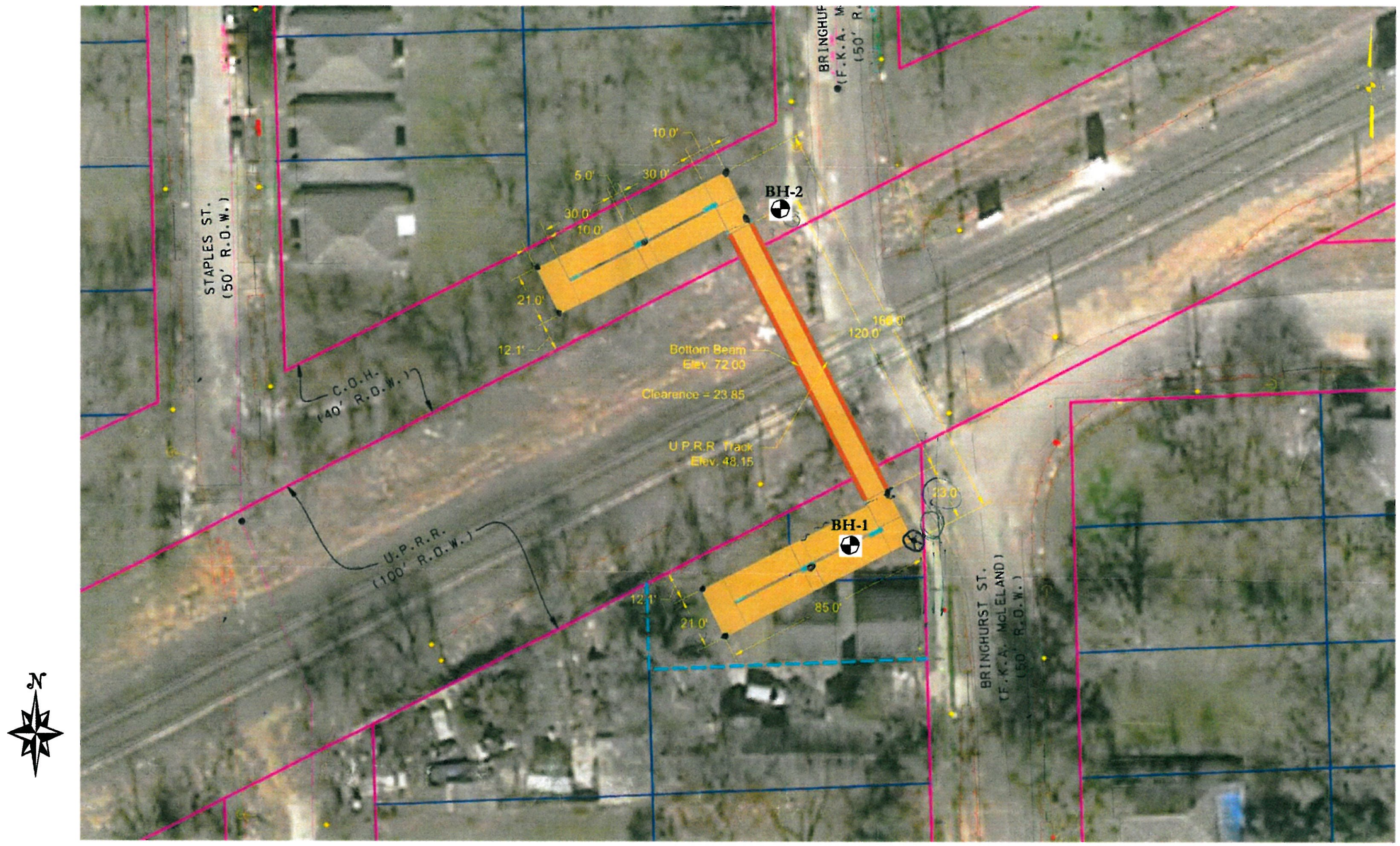
DATE: 5/20/2013	APPROVED BY: MM	PREPARED BY: NL

SITE VICINITY MAP
BRINGHURST PEDESTRIAN BRIDGE OVER UPRR
CSJ No. 0912-72-289; WBS No. N-000420-0045-3

PROJECT NO.: HG1218100	DRAWING NO.: PLATE 1



LEGEND:

APPROXIMATE BORING LOCATIONS

HVJ ASSOCIATES	6120 S. Dairy Ashford Road Houston, Texas 77072-1010 281.933.7388 Ph 281.933.7293 Fax	
DATE: 4/19/2013	APPROVED BY: MM	PREPARED BY: NL
PLAN OF BORINGS BRINGHURST PEDESTRIAN BRIDGE OVER UPRR CSJ No. 0912-72-289; WBS No. N-000420-0045-3		
PROJECT NO.: HG1218100	DRAWING NO.: PLATE 2	

APPENDIX A

BORING LOGS AND KEY TO TERMS AND SYMBOLS

DRILLING LOG

WinCore
Version 3.0

County	Harris	Hole	BH-1	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/7/2013
CSJ	0912-72-289	Station		Grnd. Elev.	43.16 ft
		Offset		GW Elev.	N/A

Elev. (ft)	Depth (ft)	Texas Cone Penetrometer	Strata Description
	0		CLAY, Fat, soft to stiff, brown and gray w/ sand at 4' (CH)
	5	4 (6) 4 (6)	
32.7	10	10 (6) 13 (6)	
			CLAY, Lean w/ Sand, stiff to hard, brown and gray, w/ calcareous nodules (CL)
	15	14 (6) 16 (6)	
26.7			CLAY, Fat, stiff to very stiff, brown w/ calcareous nodules (CH)
22.7	20	11 (6) 13 (6)	
			CLAY, Sandy, stiff to hard, reddish brown (CL)
	25	14 (6) 16 (6)	
12.7	30	10 (6) 12 (6)	
			SAND, w/ Silt, compact, brown (SP-SM)
7.7	35	20 (6) 25 (6)	
			SAND, Silty, compact, brown (SM)
2.7	40	25 (6) 37 (6)	
			SAND, w/ Silt, slightly compact to compact, brown (SP-SM)
	45	22 (6) 30 (6)	
	50	15 (6) 19 (6)	
-12.3	55	27 (6) 30 (6)	
			SAND, Silty, dense, brown w/ gravel at 58' (SM)
-17.3	60	50 (4) 50 (2)	
	65		
	70		
	75		

Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	Additional Remarks
		15				
		25				% Passing #200 Sieve: 82.4
		25				
5	16.2	30	74	52	121	
		30	76	54		
		16				% Passing #200 Sieve: 68.4
9	58.1	17	45	30	134	
		8				% Passing #200 Sieve: 59.4
13	37.1	23	50	33	128	
		18				
		15				% Passing #200 Sieve: 69.6
16	65	16	37	21	138	
		17				
18	49.7	27	31	14	127	
		18				
		20				% Passing #200 Sieve: 6.2
		20				% Passing #200 Sieve: 22.6
		20				% Passing #200 Sieve: 8.6
		19				% Passing #200 Sieve: 9.3
		20				% Passing #200 Sieve: 5.4
		16				% Passing #200 Sieve: 13.8

Remarks: Water was encountered at 32 feet below existing grade during drilling operations; at 28 and 27.75 feet after 5 and 10 minutes, respectively. Northing: 13,850,705.181 - Easting: 3,131,499.917. WBS# N-000420-0045-3

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

Driler: Van & Sons | Logger: SN | Organization: HVJ Associates, Inc.

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DRILLING LOG

WinCore
Version 3.0

County	Harris	Hole	BH-2	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/6/2013
CSJ	0912-72-289	Station		Grnd. Elev.	43.07 ft
		Offset		GW Elev.	N/A

Elev. (ft)	Depth (ft)	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 very stiff, brown and gray, w/ sand at 9' (CH)			23				
						25	59	41		
	5	5 (6) 9 (6)								
				5	45.8	24	75	57	127	
						26				
						28				% Passing #200 Sieve: 86.9
	10	9 (6) 12 (6)								
32.6			CLAY, Sandy, stiff to hard, brown and gray (CL)	8	63.5	10	29	14	128	
						18				
	15	10 (6) 15 (6)								
27.6			CLAY, Lean w/ Sand, stiff, brown and gray (CL)	11	23.7	17	41	28	118	
										% Passing #200 Sieve: 72.9
	20	15 (6) 21 (6)								
22.6			CLAY, Sandy, stiff to very stiff, brown and gray (CL)			15				% Passing #200 Sieve: 60.5
				15	43.1	15	36	23	139	
						15				
	25	12 (6) 20 (6)								
						17				% Passing #200 Sieve: 59.5
	30	15 (6) 25 (6)								
12.6			SAND, w/ Silt, compact, brown (SP-SM)			19				% Passing #200 Sieve: 9.1
	35	22 (6) 30 (6)								
7.6			SAND, Silty, compact, brown (SM)			18				% Passing #200 Sieve: 19.7
	40	22 (6) 32 (6)								
2.6			SAND, w/ Silt, compact to very dense, brown w/ gravel at 52' (SP-SM)			19				% Passing #200 Sieve: 11.7
	45	30 (6) 48 (6)								
	50	26 (6) 32 (6)								
						21				% Passing #200 Sieve: 6.8
	55	50 (1.5) 50 (0.5)								
-17.4	60	50 (2) 50 (0.5)								
	65									
	70									
	75									

Remarks: Boring caved in at 29 feet. Northing: 13,850,837.449 - Easting: 3,131,465.517. WBS# N-000420-0045-3

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

Driller: Van & Sons **Logger:** SN **Organization:** HVJ Associates, Inc.

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APPENDIX B

DRILLED SHAFT FOUNDATION DESIGN DATA

SOIL STRENGTH ANALYSIS

WinCore
Version 3.0

County	Harris	Hole	BH-1	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/7/2013
Control	0912-72-289	Station		Grnd. Elev.	43.16 ft
		Offset		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	43.2	32.7	0.22	1166	0.0	0.20	2.14
2	32.7	26.7	0.35	3996	0.0	0.70	6.34
3	26.7	22.7	0.34	2671	0.0	0.47	8.21
4	22.7	12.7	0.30	3427	0.0	0.60	14.21
5	12.7	7.7	0.39	0	0.0	0.39	16.18
6	7.7	2.7	0.54	0	0.0	0.54	18.89
7	2.7	-12.3	0.42	0	0.0	0.42	25.14
8	-12.3	-17.3	0.88	0	0.0	0.88	29.96

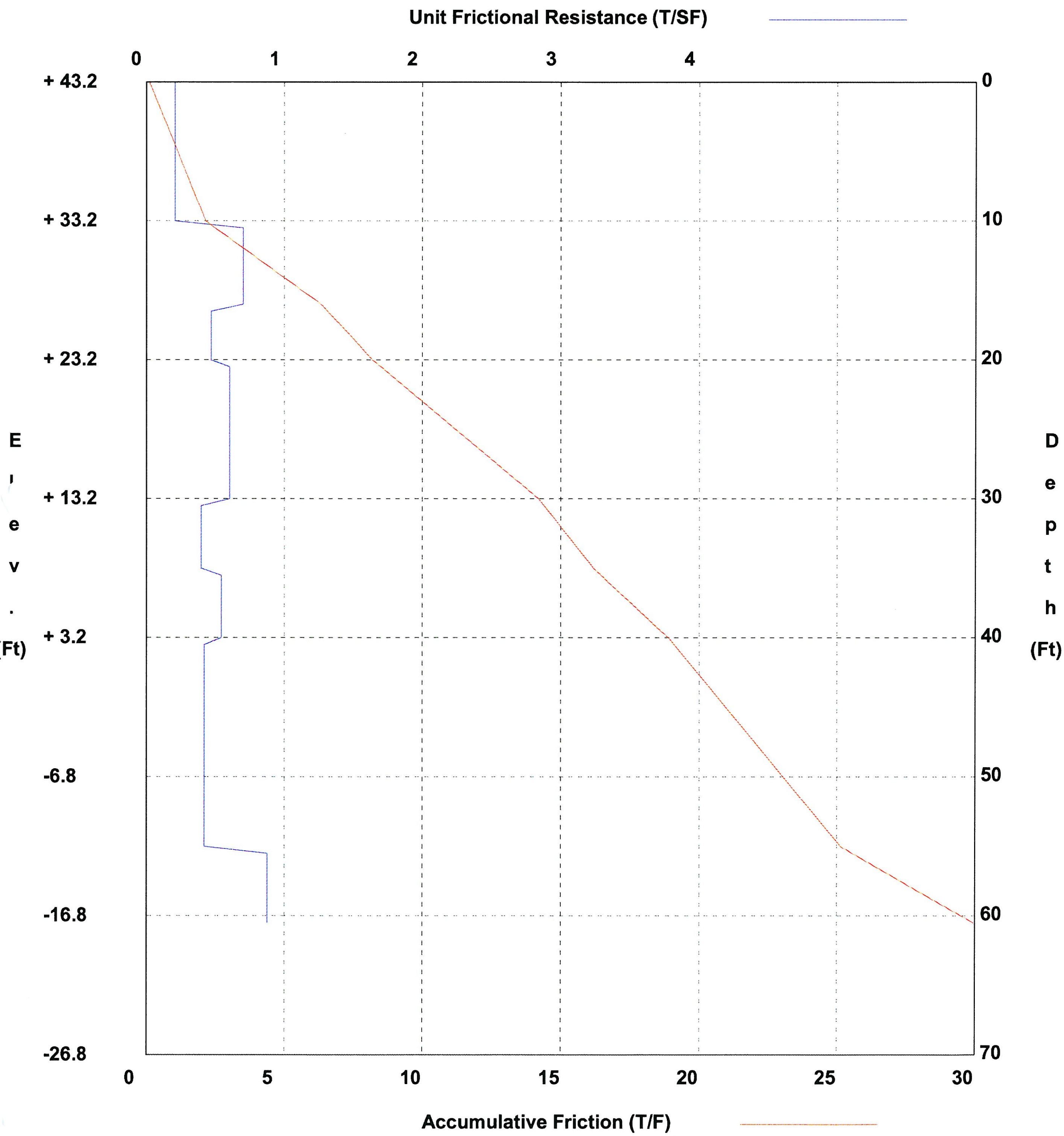
SKIN FRICTION DESIGN

WinCore
Version 3.0

County	Harris	Hole	BH-1	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/7/2013
Control	0912-72-289	Station		Grnd. Elev.	43.16 ft
		Offset		GW Elev.	N/A

Drilled Shaft Design: Soil Reduction Factor = 0.7

TAT Friction Values Used



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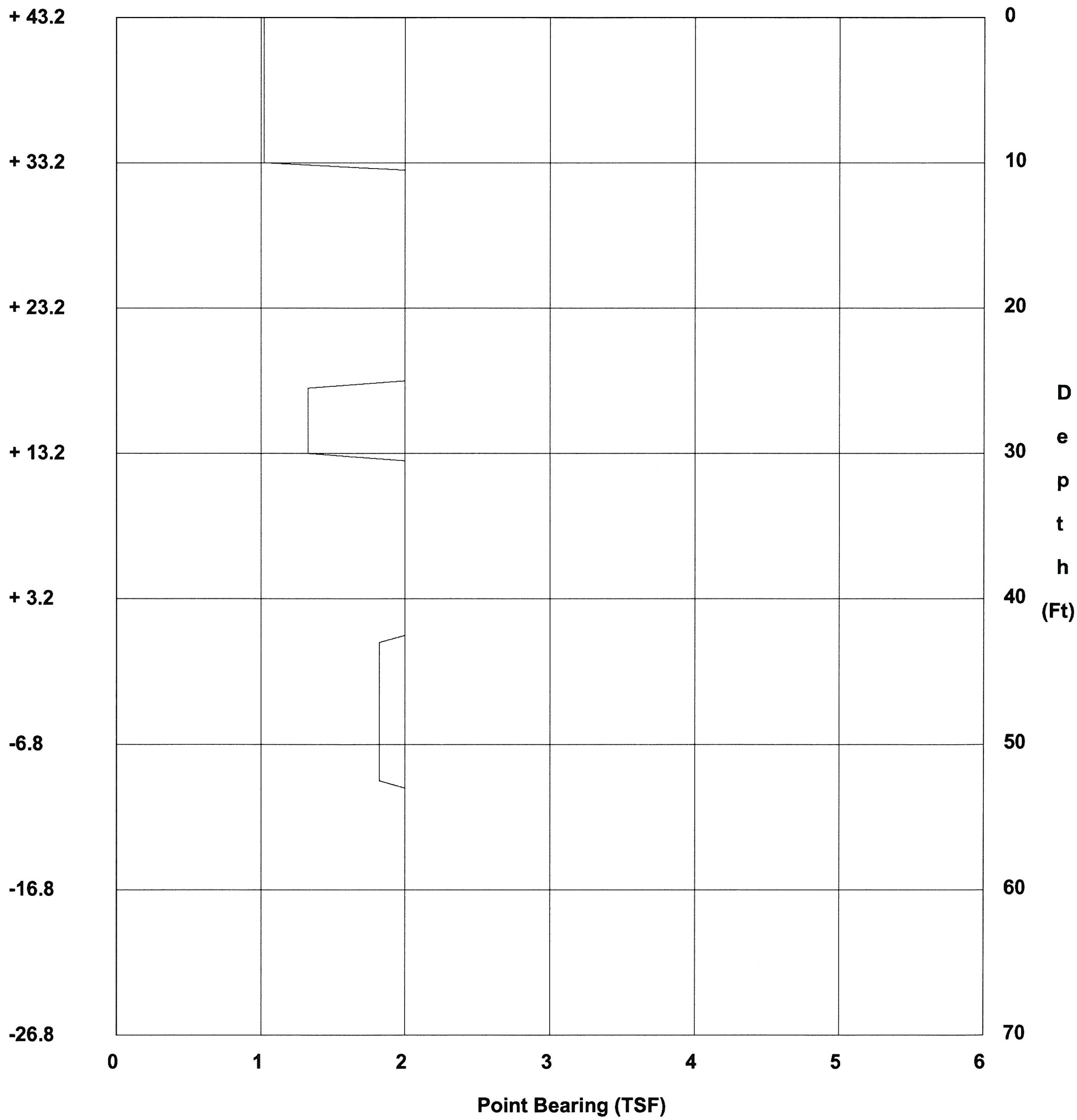
POINT BEARING DESIGN

WinCore
Version 3.0

County	Harris	Hole	BH-1	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/7/2013
Control	0912-72-289	Station		Grnd. Elev.	43.16 ft
		Offset		GW Elev.	N/A

Diameters Below Tip Checked = 2

TAT Bearing Values Used



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WinCore
Version 3.0

SOIL STRENGTH ANALYSIS

County	Harris	Hole	BH-2	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/6/2013
Control	0912-72-289	Station		Grnd. Elev.	43.07 ft
		Offset		GW Elev.	N/A

TAT Values Preferentially Used

Soil reduction factor of 0.7 applied

Strata No.	Elev. (Feet) From	Elev. (Feet) To	TCP Unit Friction	TAT Cohesion (PSF)	TAT Phi Degrees	TAT Unit Friction (TSF)	Accumulative Friction (T/F)
1	43.1	32.6	0.25	2765	0.0	0.48	5.08
2	32.6	27.6	0.29	3319	0.0	0.58	7.98
3	27.6	22.6	0.42	1706	0.0	0.30	9.48
4	22.6	12.6	0.42	3103	0.0	0.54	14.91
5	12.6	7.6	0.46	0	0.0	0.46	17.18
6	7.6	2.6	0.47	0	0.0	0.47	19.55
7	2.6	-17.4	0.74	0	0.0	0.74	34.61

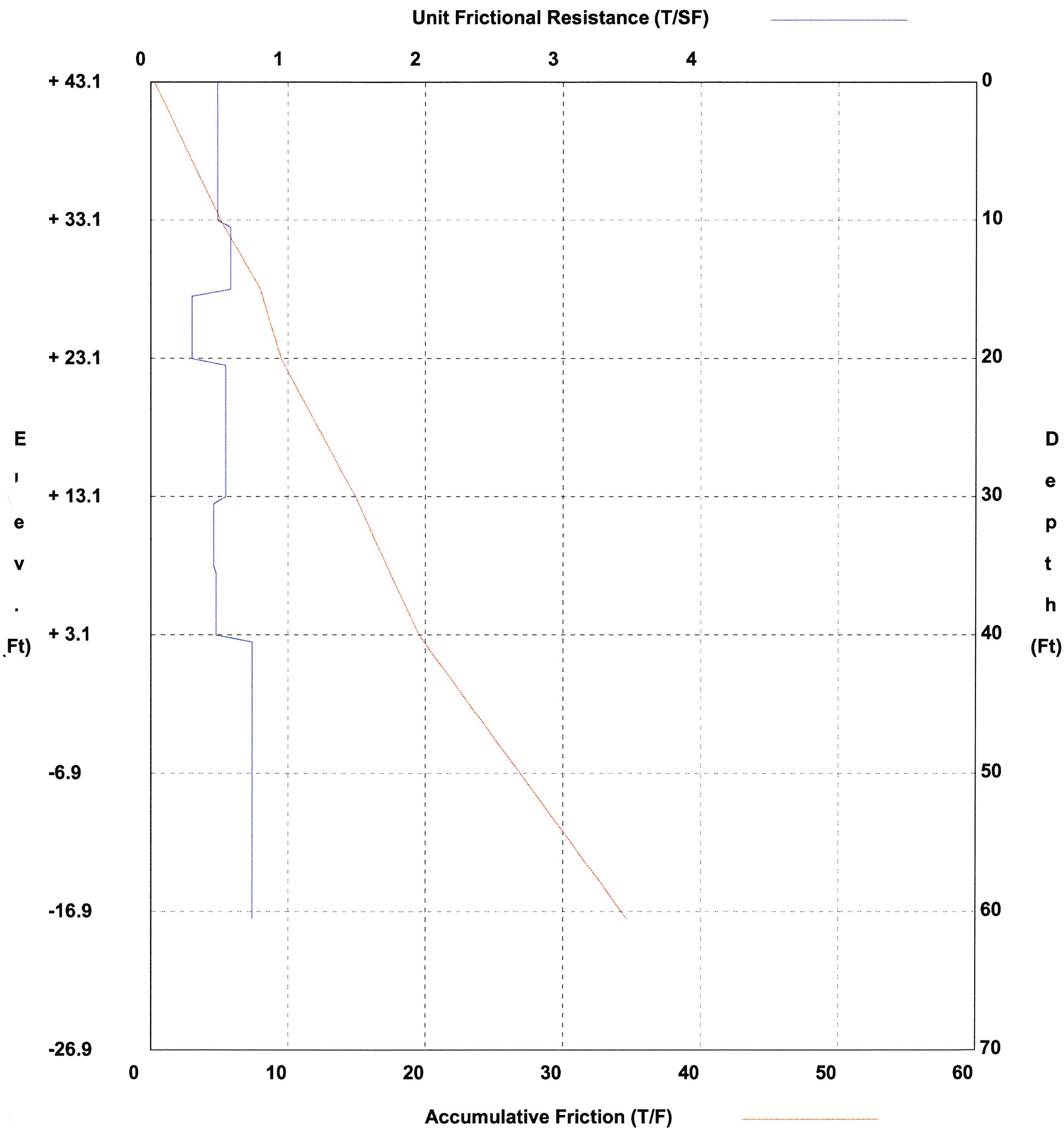
SKIN FRICTION DESIGN

WinCore
Version 3.0

County	Harris	Hole	BH-2	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/6/2013
Control	0912-72-289	Station		Grnd. Elev.	43.07 ft
		Offset		GW Elev.	N/A

Drilled Shaft Design: Soil Reduction Factor = 0.7

TAT Friction Values Used



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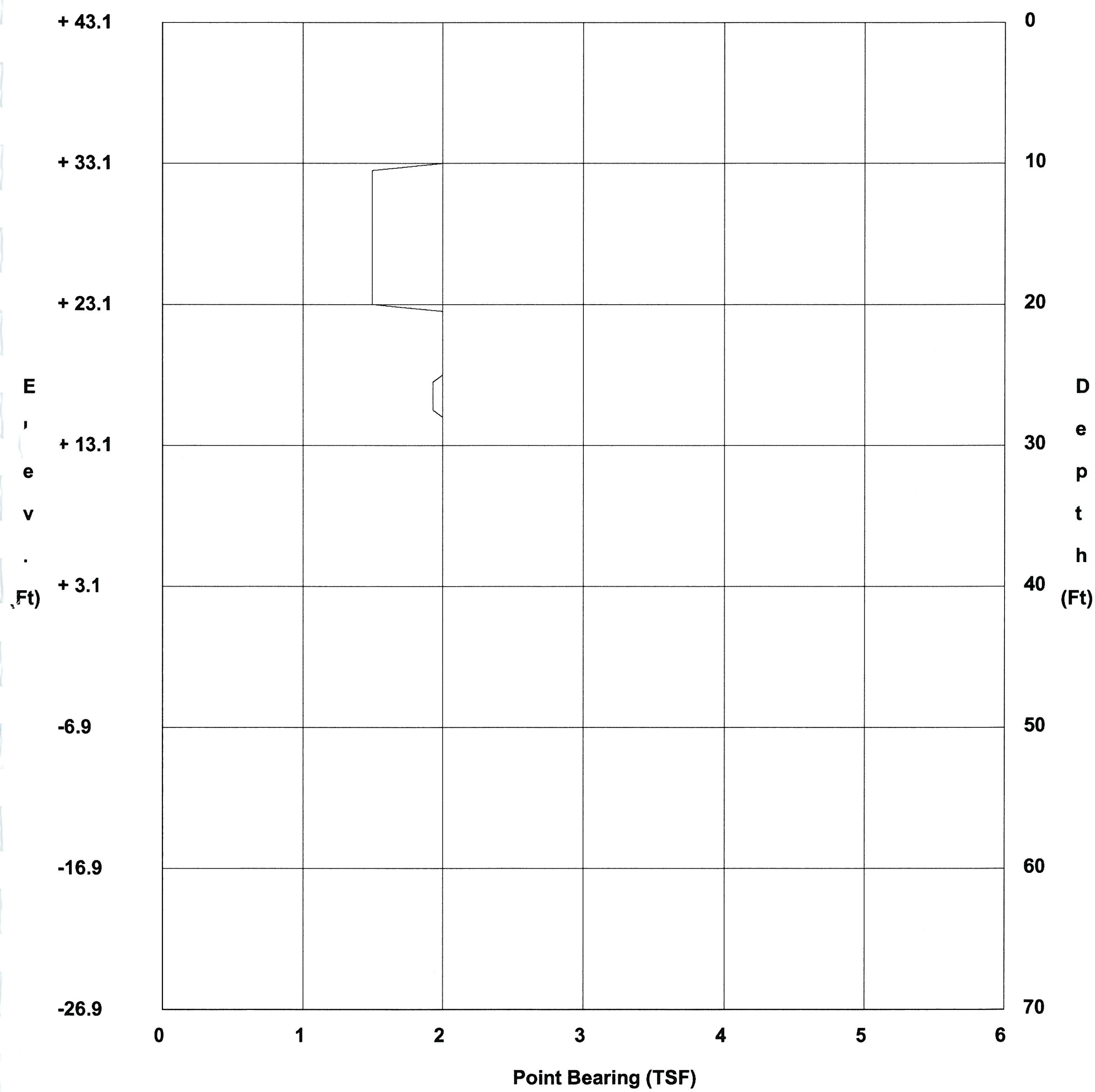
POINT BEARING DESIGN

WinCore
Version 3.0

County	Harris	Hole	BH-2	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/6/2013
Control	0912-72-289	Station		Grnd. Elev.	43.07 ft
		Offset		GW Elev.	N/A

Diameters Below Tip Checked = 2

TAT Bearing Values Used



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APPENDIX C

ALLOWABLE DRIVEN PILE SKIN FRICTION

WinCore
Version 3.0

SOIL STRENGTH ANALYSIS

County	Harris	Hole	BH-1	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/7/2013
Control	0912-72-289	Station		Grnd. Elev.	43.16 ft
		Offset		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	43.2	32.7	0.31	1166	0.0	0.29	3.06
2	32.7	26.7	0.50	3996	0.0	1.00	9.06
3	26.7	22.7	0.48	2671	0.0	0.67	11.73
4	22.7	12.7	0.43	3427	0.0	0.86	20.30
5	12.7	7.7	0.56	0	0.0	0.56	23.11
6	7.7	2.7	0.78	0	0.0	0.78	26.98
7	2.7	-12.3	0.60	0	0.0	0.60	35.92
8	-12.3	-17.3	1.25	0	0.0	1.25	42.80

SKIN FRICTION DESIGN

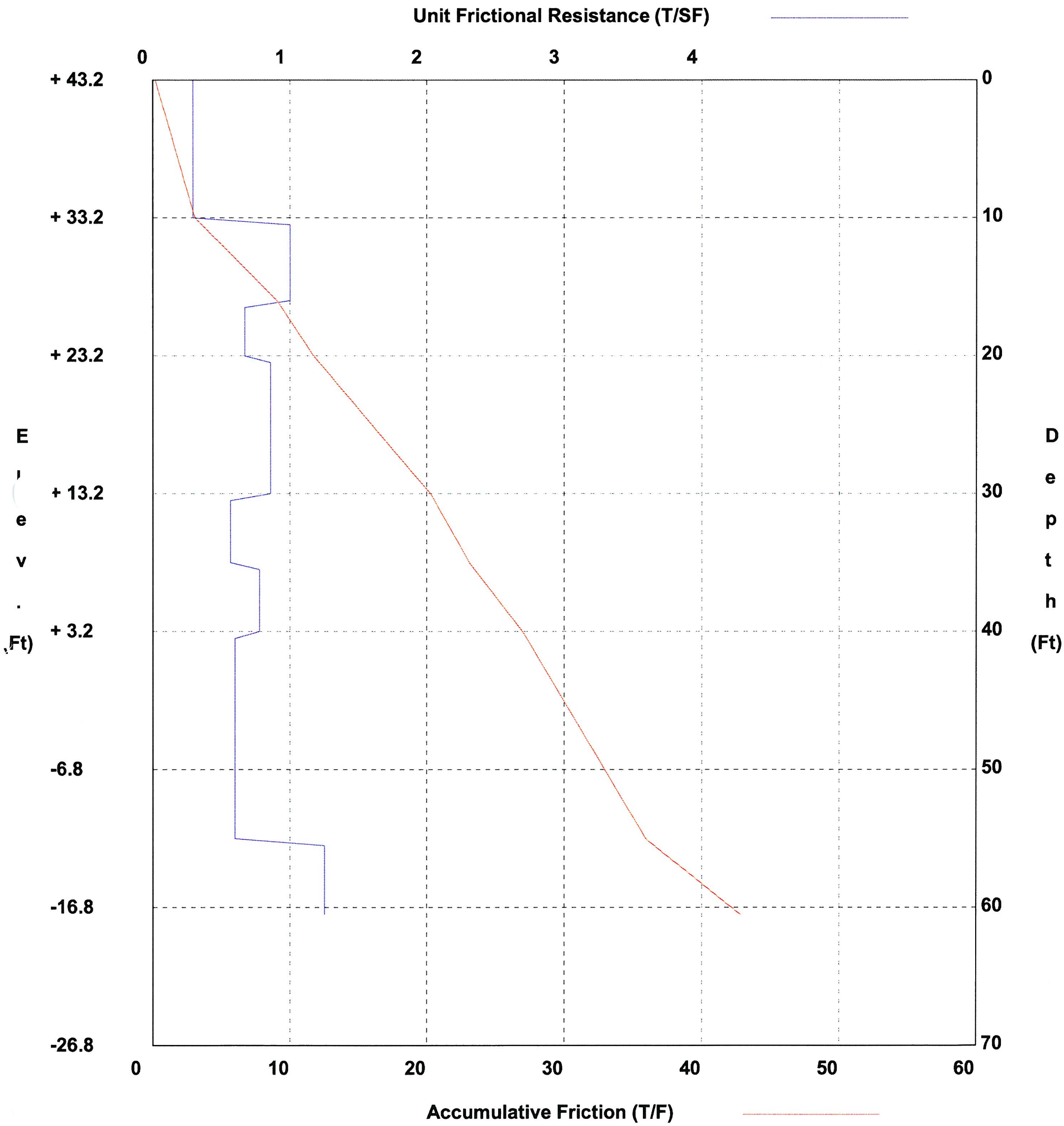
WinCore
Version 3.0

County	Harris	Hole	BH-1	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/7/2013
Control	0912-72-289	Station		Grnd. Elev.	43.16 ft
		Offset		GW Elev.	N/A

Piling Design: No Soil Reduction Factor

TAT Friction Values Used

Skin Friction Limit = 1.3 tsf



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SOIL STRENGTH ANALYSIS

WinCore
Version 3.0

County	Harris	Hole	BH-2	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/6/2013
Control	0912-72-289	Station		Grnd. Elev.	43.07 ft
		Offset		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	43.1	32.6	0.35	2765	0.0	0.69	7.26
2	32.6	27.6	0.42	3319	0.0	0.83	11.41
3	27.6	22.6	0.60	1706	0.0	0.43	13.54
4	22.6	12.6	0.60	3103	0.0	0.78	21.30
5	12.6	7.6	0.65	0	0.0	0.65	24.55
6	7.6	2.6	0.68	0	0.0	0.68	27.92
7	2.6	-17.4	1.05	0	0.0	1.05	49.45

SKIN FRICTION DESIGN

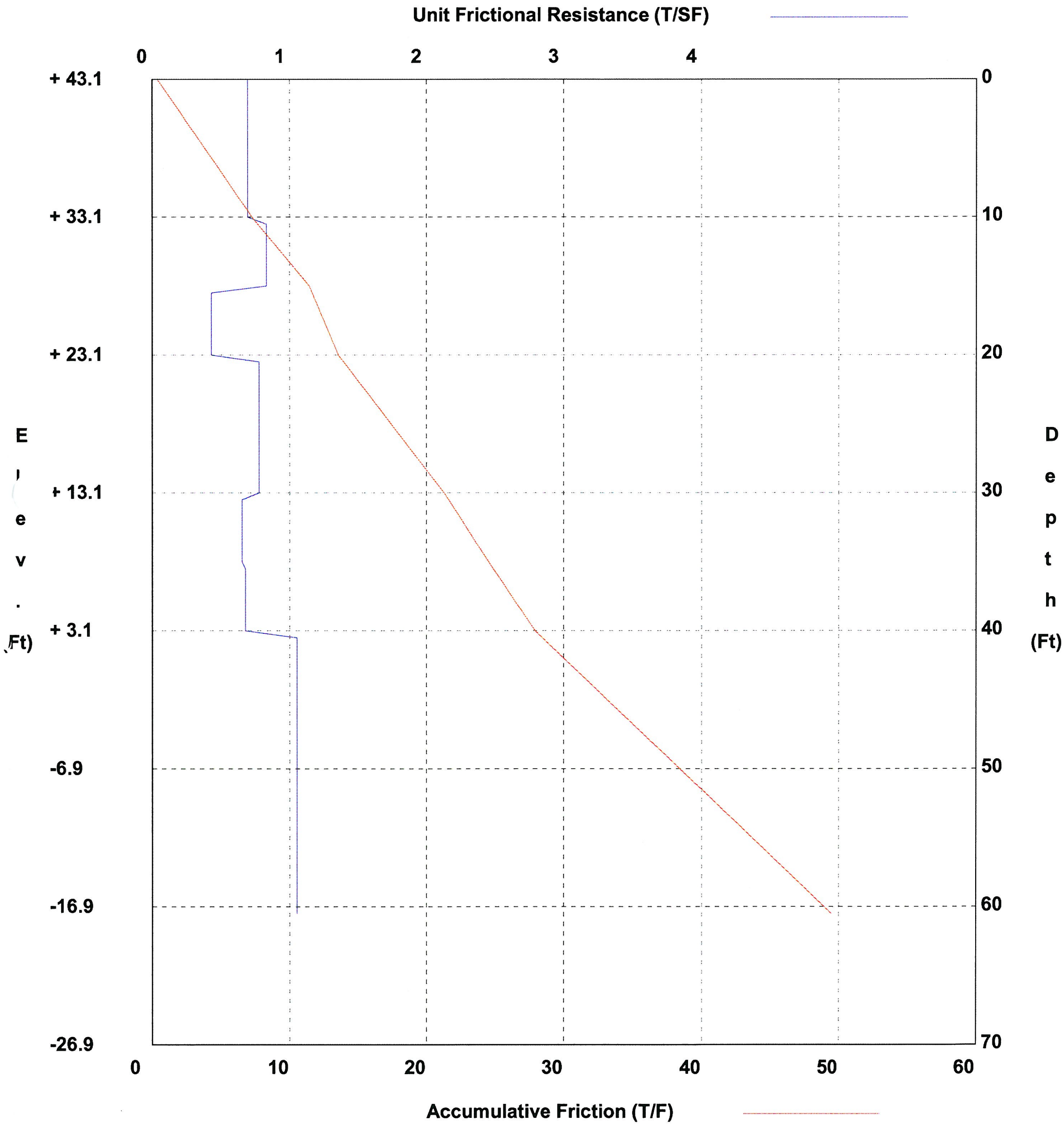
WinCore
Version 3.0

County	Harris	Hole	BH-2	District	Houston
Highway	Bringhurst Bridge	Structure	Pedestrian Bridge	Date	5/6/2013
Control	0912-72-289	Station		Grnd. Elev.	43.07 ft
		Offset		GW Elev.	N/A

Piling Design: No Soil Reduction Factor

TAT Friction Values Used

Skin Friction Limit = 1.3 tsf



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APPENDIX D

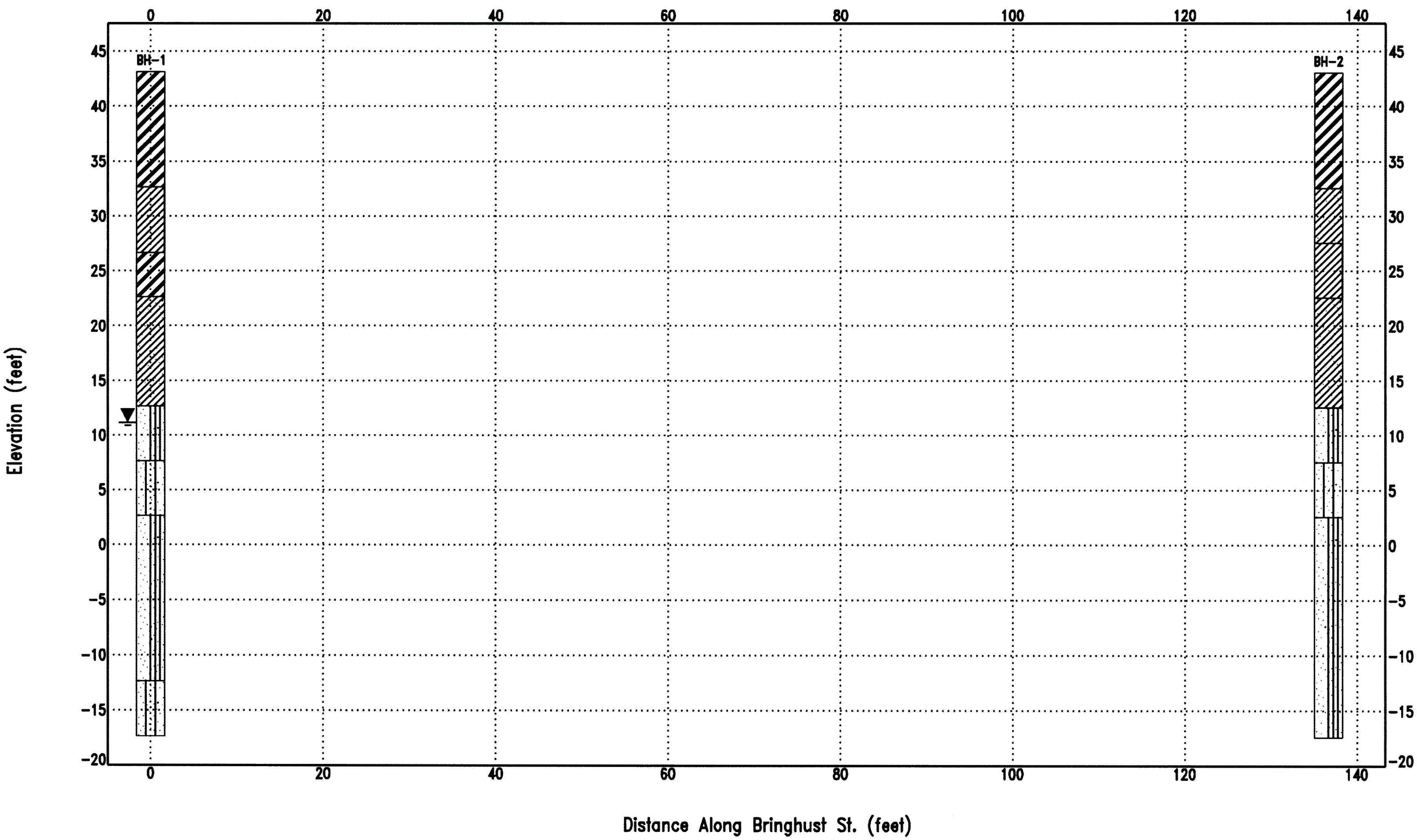
SUMMARY OF LABORATORY TESTINGS

Project: Bringhurst Bridge
Location: Houston, Texas
Number: HG1218100
WBS No. N-000420-0045-3
CSJ No. 0912-72-289

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	% Pass #200 Sieve	Moisture Content (%)	Unit Weight (pcf)	Shear Strength (UC) (psi)	Shear Strength (UU) (psi)
BH-1	2					15			
BH-1	4				82.4	25			
BH-1	6					25			
BH-1	7	74	22	52		30	121		16.2
BH-1	9	76	22	54		30			
BH-1	12				68.4	16			
BH-1	14	45	15	30		17	134		58.1
BH-1	16				59.4	8			
BH-1	19	50	17	33		23	128		37.1
BH-1	21					18			
BH-1	22				69.6	15			
BH-1	24	37	16	21		16	138		65
BH-1	26					17			
BH-1	27	31	17	14		27	127		49.7
BH-1	29					18			
BH-1	32				6.2	20			
BH-1	37				22.6	20			
BH-1	42				8.6	20			
BH-1	47				9.3	19			
BH-1	52				5.4	20			
BH-1	59				13.8	16			
BH-2	2					23			
BH-2	3	59	18	41		25			
BH-2	7	75	18	57		24	127		45.8
BH-2	8					26			
BH-2	9				86.9	28			
BH-2	12	29	15	14		10	128		63.5
BH-2	14					18			
BH-2	17	41	13	28		17	118		23.7
BH-2	19				72.9				
BH-2	21				60.5	15			
BH-2	23	36	13	23		15	139		43.1
BH-2	24					15			
BH-2	27				59.5	17			
BH-2	32				9.1	19			
BH-2	37				19.7	18			
BH-2	42				11.7	19			
BH-2	52				6.8	21			
Total		**11**	**11**	**11**	**18**	**37**	**9**	**0**	**9**

APPENDIX E

SOIL PROFILE



LEGEND:

- Concrete
- Fill (made ground)
- Sandy Clay
- Silt
- Asphaltic Concrete
- Fat Clay
- Base
- Clayey Sand
- Sand

Note:
Data concerning subsurface conditions obtained at boring locations only.

HVJ ASSOCIATES	7/15/2013	
	APPROVED BY: ZA	PREPARED BY: NL
BORING LOG PROFILE BRINGHUST PEDESTRIAN BRIDGE WBS No. N-000420-0045-3		
PROJECT NO.: HG1218100	DRAWING NO.: PLATE E-1	