ERDC Report to USIBWC

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Geotechnical Evaluation of the Brownsville Levee Cracking and Partial Slope Failure

Lucas A. Walshire, Joseph B. Dunbar, Isaac J. Stephens, Maureen K. Corcoran, Carla Roig-Silva, and Julie R. Kelley

July 2015

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Geotechnical and Structures Laboratory
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3909 Halls Ferry Road
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Final report

Prepared for U.S. International Boundary Water Commission
4171 North Mesa Street, C-100 El Paso, TX 79902

Monitored by Geotechnical and Structures Laboratory
U.S. Army Engineer Research and Development Center
Vicksburg, MS 39180-6199

DEPARTMENT OF THE ARMY
ENGINEER RESEARCH AND DEVELOPMENT CENTER, CORPS OF ENGINEERS
GEOTECHNICAL AND STRUCTURES LABORATORY
WATERWAYS EXPERIMENT STATION, 3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

REPLY TO
ATTENTION OF

July 13, 2015

Mr. Jose Nuñez, Principal Engineer
U.S. International Boundary Water Commission
U.S. Section
4171 North Mesa, Suite C-100
El Paso, Texas 79902-1441

Dear Mr. Nuñez:

At the request of the U.S. International Boundary Water Commission (USIBWC), the U.S. Army Engineer Research and Development Center (ERDC) conducted a geotechnical investigation to determine specific causes for levee cracking and partial slope failure on a portion of the Lower Rio Grande Valley flood control system in Brownsville, TX. The geotechnical investigation included characterizing the site geology and stratigraphic units, and performing slope stability modeling to identify the likely failure surfaces, failure mechanism(s), and underlying causes for the levee and floodplain deformation observed. The results of the investigation are documented in a technical report (TR) entitled "Geotechnical Evaluation of the Brownsville Levee Cracking and Partial Slope Failure", ERDC TR-15, June 2015. The report may be used by USIBWC to aid in the development of engineering solutions for remediation of the studied reach.

As Acting Director of the Geotechnical and Structures Laboratory, I certify that this geotechnical investigation was conducted by registered professional engineers and registered professional scientists.

Sincerely,

William P. Grogan, PhD, PE
Acting Director, Geotechnical and
Structures Laboratory

Abstract

The U.S. International Boundary and Water Commission (IBWC) discovered cracks and a partial slope failure on a newly refurbished levee section and adjacent floodplain along the Rio Grande River in Brownsville, TX. The partial failure followed a significant drop in water level in early-April 2014. A geotechnical investigation was performed by the U.S. Army Engineer Research and Development Center (ERDC) to determine the causes for the partial levee failure and provide remediation alternatives. A series of events, combined with the local geologic conditions, led to the partial slope failure. Events included the 2012 levee construction, fluctuation and rapid drawdown conditions in the Rio Grande, and a higher elevation of Lake Brown (an oxbow of the Rio Grande) relative to the river. Progressive or creep-type failure mode was identified as the probable mechanism to explain the deformation observed in the field, and this was confirmed by seepage and stability analyses. Based on this evaluation, recommendations for remediation include: (1) implementation of a vegetation control program, (2) short-term monitoring, (3) evaluation of other locations along the river with similar river geometry and groundwater conditions, (4) efforts to minimize sudden drawdown, (5) additional analyses using the design hydrograph, and (6) incorporating cost/benefit analyses for the different alternatives.

Contents

Figures and Tables

Figures

Tables

Preface

This study was conducted for the U.S. International Boundary Water Commission. The technical monitor was Jose Nunez.

The work was performed by the Geotechnical Engineering and Geosciences Branch (GSD) of the Geosciences and Structures Division (GS), U.S. Army Engineer Research and Development Center, Geotechnical and Structures Laboratory (ERDC-GSL). At the time of publication, Chad A. Gartrell was Chief, CEERD-GSD; Bartley P. Durst was Chief, CEERD-GS; and Dr. Michael K. Sharp, CEERD-GVT was the Technical Director for Water Resources Infrastructure. The Acting Deputy Director of ERDC-GSL was Dr. Gordon W. McMahon and the Acting Director was Dr. William P. Grogan.

The project was managed by the U.S. Army Corps of Engineers (USACE), Galveston District (SWG), under an Interagency Agreement. Enrique Villagomez was Project Manager. Gary Chow, SWG, was Senior Geotechnical Engineer/Technical Expert and Joshua Robbins, SWG, was Engineer-in-Training.

Drilling was conducted by USACE, Mobile District (SAM). Rhonda A. Capes, SAM, was Geology Lead and coordinated the drilling effort. The USACE, Savannah District (SAS), provided the cone penetrometer tests (CPTs). William McIntosh, SAS, coordinated the CPTs. Greg Armstrong and Sarwenaj Ashraf of the USACE, Fort Worth District (SWF) provided support for the installation and monitoring of the inclinometers.

LTC John T. Tucker III was the Acting Commander of ERDC, and Dr. Jeffery P. Holland was the Director.

Unit Conversion Factors

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	meters
inches	0.0254	meters
microinches	0.0254	micrometers
miles (US statute)	1,609.347	meters
pounds (force) per square foot	47.88026	pascals
pounds (mass)	0.45359237	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
pounds (mass) per square foot	4.882428	kilograms per square meter
square feet	0.09290304	square meters

1 Introduction

1.1 Background

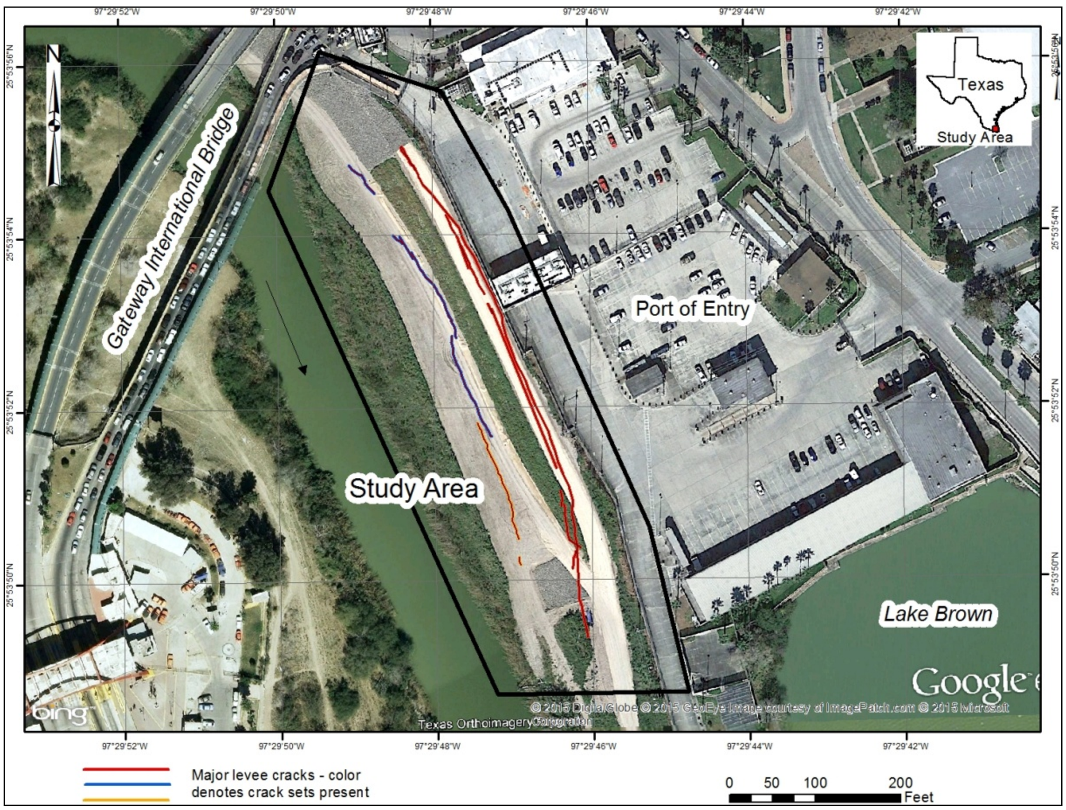
The U.S. section of the International Boundary and Water Commission (USIBWC) levee system was originally built by city and county governments within the Lower Rio Grande Valley (LRGV) during the late-1800s and early-1900s. Between 1900 and 1939, the Rio Grande River overflowed 23 times within the LRGV, with hurricanes hitting the area in 1910, 1913, and 1933 (Stubbs et al. 2003). During the depression years, the Texas border counties within the LRGV were unable to maintain the piecemeal network of private, local, and county levees because of the poor economy. Repeated flooding on the Rio Grande, combined with the economic conditions during this time, forced the border counties within the LRGV to petition the federal government to take over the existing levee system and provide comprehensive flood control protection.

The USIBWC assumed control and management of the local levee system in September 1932 and subsequently began rebuilding the entire LRGV flood control system during the 1930s and 40s (USIBWC 1992). These early levees were built according to local construction practice in the LRGV, using soils obtained from the nearby floodplain, and generally correspond to a standard levee section as defined by the U.S. Army Corps of Engineers (USACE) in USACE (2000): less than 25 ft in height, with side slopes flatter than or equal to 2H:1V.

Sections of the levee system have subsequently been refurbished, since they were federalized because of urban land use and environmental changes and, more recently, as part of the Upper Brownsville Levee Rehabilitation Project (UBLRP) between 2012 and 2013. The UBLRP involved raising the height of the levee an additional 3 ft between Donna Pump and Brownsville and regrading the levee slopes to meet the new project flood requirements. Construction of the levee raise between Donna Pump and Brownsville was completed in October 2013. The new levee construction in the Brownsville area was under Contract IBM 08T0036, IBM 09D0006, and IBM 13C0001 (Raba-Kistner 2009, 2011; Tetra Tech 2013).

The newly refurbished levee section and adjacent floodplain began cracking and a partial slope failure occurred following a significant drop in the Rio Grande water level in early-April 2014. The USIBWC (2014) discovered the levee cracks between Stations 1899+00 and 1904+85 in early-May 2014 (Figure 1.1). In July 2014, nearly a foot of slippage at the levee crest and two prominent cracks at the levee toe were observed by personnel from the U.S. Army Engineer Research and Development Center (ERDC) (Figures 1.2 to 1.4). USIBWC personnel reported that the river level had been rapidly drawn down several feet to satisfy local irrigation demands between April and June 2014 before the onset of the cracking and slippage of the crest.

Figure 1.1. Location of levee with cracking and partial slope failure.



1.2 Purpose

The purpose for this study was to determine the specific causes for the levee cracking and partial slope failure. A geotechnical investigation was performed for this study to characterize the site geology and stratigraphy, to evaluate the engineering properties of the underlying soils and stratigraphic units, and to perform slope stability modeling to identify the likely failure surfaces, failure mechanism(s), and underlying causes for the levee

Figure 1.2. Severe cracking at the levee crest.

Figure 1.3. Severe cracking and settlement at the levee crest and at the waterside toe.

Figure 1.4. Close-up view of levee cracking and settlement at the crest. View is looking upstream. Gateway Bridge is in the background and corresponds to the upper limits of the study area.

and floodplain deformation observed. The following investigation will be used to develop engineering solutions for remediation of this reach.

1.3 Scope of study

The scope of this investigation involved numerous tasks that were performed in a step-wise progression. This approach was designed to maximize the amount of information being collected, to better characterize the site conditions, and to guide subsequent steps in the data collection and evaluation process. In addition, because of both the uncertain nature of the site conditions encountered during the course of this study and the pre-existing data that were available to characterize the site initially, the following study was conducted in steps to obtain the necessary information for the subsequent analysis to answer basic questions regarding the underlying failure mechanism(s) and to develop remediation options for consideration.

Major tasks that were performed during this study include:

1. a comprehensive review of the previous geotechnical investigations that included those performed for design and construction documents;
2. an evaluation and historical reconstruction of the river reach under study to better understand prior levee performance issues and land use changes through time;
3. field investigations that included subsurface sampling involving cone-penetrometer tests (CPTs), soil borings and collection of Shelby-tube and split-spoon samples for laboratory testing and characterization of the underlying soils; installation and monitoring of inclinometers for measuring bank movements and piezometers for accurately determining groundwater levels and identifying the presence of permeable zones in the levee foundation; surface surveying to establish the post-cracking levee geometry and monitoring to quantify any subsequent surface movements that might occur; a bathymetric survey of the study reach to provide bathymetry of the submerged bank and bed of the river;
4. a geologic evaluation of the CPT results and soil boring to characterize and classify the soils, the stratigraphy, and the lateral and vertical extent of identified strata throughout the study reach;
5. seepage and stability modeling using state-of-practice slope stability programs and analysis of the levee foundation to determine the probable failure mechanism(s);

6. preparation of this report describing the investigation in greater detail, the methods used in this study, the results of analyses, and the important findings.

1.4 Study area

The reach of river under study is shown in Figure 1.1 and extends from Station 1899+00 to Station 1904+85 on the left bank of the Rio Grande downstream of the Gateway International Bridge. The study area is located on the U.S. side of the Rio Grande in Cameron County, Texas. Important features to be noted in Figure 1.1 are the Port-of-Entry (POE) parking lot and the port facility downstream of the Gateway International Bridge, as well as the prominent Rio Grande oxbow, or resaca, known as Lake Brown.

Geographically, both Lake Brown and the nearly right-angle Rio Grande course have been prominent and stable features within the study area for the past 170 years. As will be described in a later section of this report, the stability of the channel alignment at this location is noteworthy considering the numerous abandoned oxbows and courses within the floodplain of the Rio Grande (Brown et al. 1980; Bureau of Economic Geology 1976).

The study area is historically significant as being part of the War of 1846 battlefield between the United States and Mexico and was formerly part of the limits of the Fort Brown U.S. Military Base. Much of the land area within the former Fort Brown is under the jurisdiction of the USIBWC, which received title to the land with the decommissioning of the fort.

2 Previous Geotechnical Investigations

2.1 Introduction

The UBLRP encompassed a 51-mile stretch of a 65-mile levee that was raised 1- to 3-ft on the U.S. side. The project design was to rehabilitate the levee system to provide a 100-year level of flood protection, which would meet certification standards required by the Federal Emergency Management Agency (FEMA). As part of the overall design effort, the following tasks were completed:

- Detailed field inspection (performed by USIBWC);
- USIBWC document review;
- Visual inspection and survey of the existing gatewell structures (approx. 284);
- Geotechnical investigations and analyses (Raba-Kistner Inc. 2009, 2011);
- Field surveys of the levee centerline and right-of-way (ROW) mapping.

Documents were provided by USIBWC on previous geotechnical investigations from Tetra Tech, Inc. (Tetra Tech) and Raba-Kistner Consultants, Inc. (Raba-Kistner) that were prepared for the UBLRP. Additionally, ERDC personnel collected data from visits to the Cameron County Engineering Division at San Benito, TX, and the City of Brownsville Water and Sewer Department, Brownsville, TX.

2.2 Tetra Tech, Inc. and Raba-Kistner Consultants, Inc.

Geotechnical documents describing the UBLRP and Brownsville study area were produced by Tetra and Raba. Tetra Tech hired Raba to perform the geotechnical analysis of the levee system for their design, which was required to meet FEMA levee certification.

A summary description of the geotechnical reports produced by Tetra Tech Inc. and Raba-Kistner according to publication year is summarized below:

- Raba-Kistner Consultants, Inc. (2009) - Geotechnical Exploration and Engineering Evaluation of Levee System, The Lower Rio Grande Flood Control Project from Cameron County Line of Donna Pump to

Brownsville Levee Reach to its East-most Limit, July 24, 2009, USIBWC Task No. IBM08T0036, Final Technical Memorandum.
- Raba-Kistner Consultants, Inc. (2011) - Geotechnical Addendum-Subreach 4 For the Lower Rio Grande Flood Control Project Levee System – From Donna Pump to Brownsville Levee Reach, Hidalgo County and Cameron County, Texas, June 1, 2011.
- Tetra Tech, Inc. (2012) - Upper Brownsville Levee Rehabilitation, Design Report Final Submittal, Cameron Counties, Texas, May 2012, Contract No. IBM09D0006.
- Tetra Tech, Inc. (2013) - Upper Brownsville Levee Rehabilitation, Cameron Counties, Texas, May 2013, Plans and Specifications, 299 Sheets, Contract No. IBM09D0006.

2.2.1 Geotechnical exploration and engineering evaluation of the levee system, the Lower Rio Grande Flood Control Project from Cameron County Line of Donna Pump to Brownsville Levee Reach to its east-most limit (Raba-Kistner 2009)

Raba-Kistner performed a geotechnical investigation for the design of the levee improvements that included levee seepage, stability, and settlement analyses. As part of this effort, a total of 300 soil borings were drilled to characterize the in-situ conditions along the 51-mile stretch of levee. Only two of these borings were near the study area for this investigation: DP-201 and DP-202. Soil borings were advanced using straight flight augers in combination with mud rotary drilling techniques, and were backfilled with cement-grout. Samples were acquired with split spoon and Shelby tubes.

Soil Boring DP-202, located approximately 80 ft toward the river from the levee centerline, contained the only laboratory shear strength tests, which was a consolidated undrained triaxial test, from this series of reports. This test was conducted on a sample taken at a depth of 30 ft, which corre sponds to elevation 9.68 ft NAVD88. The sample was described as high plasticity clay. Table 2.1 reports the results of this test.

The loading conditions investigated during the seepage and stability analysis consisted of “end-of-construction” (undrained), steady state seepage from design flood stage (drained), and sudden drawdown condition. A traffic load was imposed along the levee crest and was equivalent to a uniform surcharge of 100 psf. The following material properties were assigned in the models (Table 2.2).

Table 2.1. DP-202 consolidated undrained test results for soil boring DP-202.

Boring No.	Depth, ft	Principle Stress Difference, ksf	Axial Strain %	Effective: Friction Angle, ϕ’ deg	Effective: Cohesion, c’ ksf	Total: Friction Angle, ϕ deg	Total: Cohesion, c ksf	Eff. Consol. Pressure, ksf
DP-202	30	3.2708	7.1	12.7	0.93	7.5	1.1	2.54
	30	3.9836	7.7					5.27
	30	8.2075	12.2					8.05

Table 2.2. Material properties used in stability analysis (Raba-Kistner 2009).

Case	Material	Unit Weight (pcf)	Short-Term and Sudden Drawdown (Undrained)		Long-Term (Drained)	
			Cohesion (psf)	ϕ (deg)	Cohesion (psf)	ϕ (deg)
1	Fill: High Plasticity, Fat Clay (CH)	125	400	15	500	8
	Silty Sand (SM)	115	0	29	0	29
	Sand (SP)	115	0	32	0	32
2	Fill, Low Plasticity, Lean Clay (CL)	125	250	30	650	19
	Silt, Low Plasticity (ML)	110	0	29	0	29
	Sandy Lean Clay (CL)	125	300	22	700	31
	Silty Sand (ML)	115	0	29	0	29
3	Fill: Lean Clay (CL)	125	250	30	650	19
	Fill: Silt (ML)	110	0	30	0	30
	Fat Clay (CH)	125	450	12	700	0
	Lean Clay (CL)	125	300	22	700	31
4	Fill: Fat Clay (CH)	125	400	15	500	8
	Fat Clay (CH)	125	450	12	550	0

The cases used in the analyses are defined as follows:

- Fat clay (CH) fill overlying non-cohesive soils (SM/SP)
- Lean clay (CL) fill overlying varied soils of silt, lean clay and sand (ML, CL)
- Irregular fill soils (with non-cohesive layers) overlying both fat (CH) and lean clays (CL)
- Fat clay (CH) fill overlying fat clay (CH)

At the time of this analyses (Raba-Kistner Inc. 2009), the conceptual drawings of the planned improvements were not available, so generalized geometry sections were used. Spencer's method in the limit equilibrium software program SLIDE developed by RocScience was used for the stability analysis. Considering the cases defined above, the levee section near borings DP-201 and DP-202 contained high plasticity clay used as levee fill material, which would indicate that Case 1 would be most applicable. Case 1 was defined as fat clay levee fill overlying silty and poorly graded sands. The results of the stability analyses for Case 1 are shown in Table 2.3.

Table 2.3. Factors of Safety from the Raba-Kistner (2009) stability analysis.

Slope	End of Const.	Steady State at Flood Stage-Waterside	Steady State at Flood Stage-Landside	Sudden Drawdown-Waterside	Sudden Drawdown-Landside
2.5H:1.0V	>2.0	>2.0	1.7	<1.0	1.7
3.0H:1.0V	>2.0	>2.0	1.8	1.2	>2.0

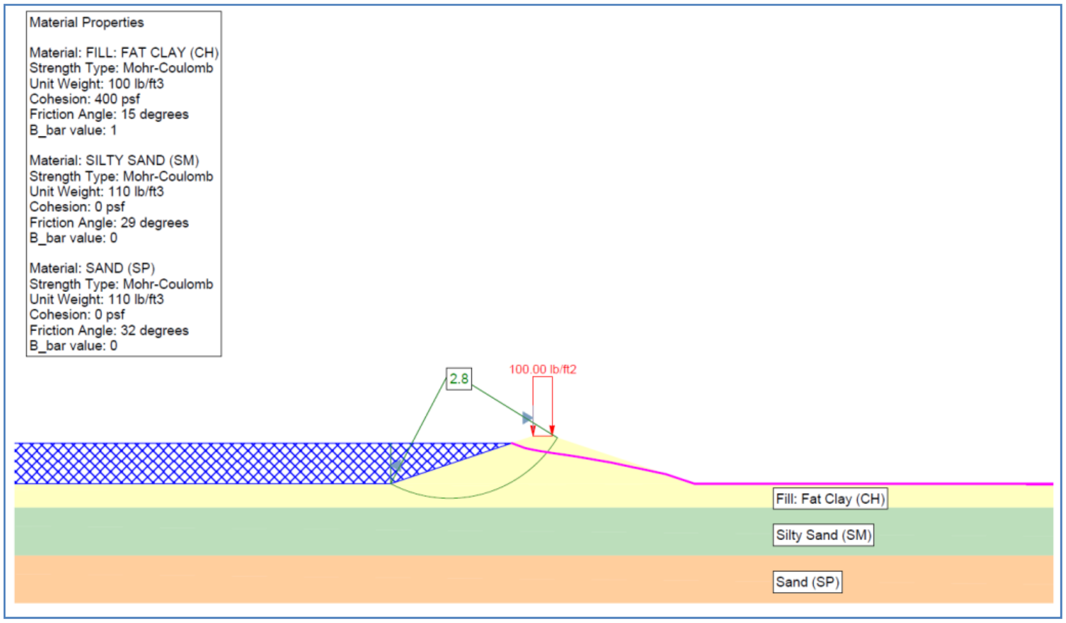
Raba-Kistner interpreted the results of the stability analysis to indicate that the levee would need side-slopes no steeper than 3.0H:1.0V. The results of the seepage analysis are shown in Table 2.4. The strata in the table refer to the different stratigraphy found along the levee reaches.

Table 2.4. Results of seepage analyses (Raba-Kistner 2009).

Case	Slope (H:V)	Stratum 1 Hydraulic Conductivity (K) cm/s	Stratum 2 Hydraulic Conductivity (K) cm/s	Stratum 3 Hydraulic Conductivity (K) cm/s	Stratum 4 Hydraulic Conductivity (K) cm/s	Stratum 5 Hydraulic Conductivity (K) cm/s	Calculated Max. Gradient, i_{max}
1a	2.5:1.0	1E-8	1E-2	1E-2	-	-	0.64
1b	3.0:1.0	1E-8	1E-2	1E-2	-	-	0.47
1c	4.0:1.0	1E-8	1E-2	1E-2	-	-	0.5

For the settlement analysis indicated for Case 1 containing a 3.0H:1.0V side slopes, the settlement would be on the range of 3.25 in. Figure 2.1 shows the Case 1 cross section developed by Raba-Kistner (2009).

Figure 2.1. Case 1 cross section (Raba-Kistner 2009).



2.2.2 Geotechnical addendum-Subreach 4 for the Lower Rio Grande Flood Control Project Levee System – from Donna Pump to Brownsville Levee Reach, Hidalgo County and Cameron County, Texas (Raba-Kistner 2011)

An addendum was submitted to revise Raba-Kistner's original Technical Memorandum (Raba-Kistner 2009) with the updated survey and levee geometry data provided by Tetra Tech (Raba-Kistner 2011). With the addition of the new data, Raba-Kistner felt that additional analyses were needed to evaluate seepage and slope stability conditions along the subreach. The critical cross sections for the analysis were chosen based on geotechnical and geometrical conditions (i.e., poor geometry and moderate soil conditions represented the critical case with respect to underseepage). Cross sections at Stations 1717+00 through 1746+00 were considered to be critical. This critical area is upstream of the study reach, which is between Stations 1899+00 to 1904+85. It was considered that the critical area had the potential to develop steady state seepage problems. A site visit was subsequently conducted and no signs of soft ground or steady state seepage conditions were identified; other sections were reviewed, and the critical section was chosen at another location.

Two analyses were conducted: one with respect to a critical section with regard to seepage, and the second, at a critical section with regard to geometry. Both of these analyses identified Station 1342+00 as the most critical section to evaluate. Table 2.5 contains the design hydraulic conductivities (K) used for the analyses.

Table 2.5. Design hydraulic conductivities (Raba-Kistner 2011).

Material	Des. Hydraulic Conductivity (K) (cm/s)
Poorly Graded Sands (SP)	1E-2
Poorly Graded Silty Sands (SM)	1E-4
Lean Clays (CL)	1E-6
Gravel (Drainage Blanket)	1E0
Clay Fill (CL and CH)	1E-6

Table 2.6 contains the shear strength data for the drained and undrained loading conditions. The results of the seepage analysis indicated that conditions exceeding the allowable exit gradient of 0.5 exist near the levee toe for subreach 4. Various alternatives were considered and a toe drain was selected as the most feasible. A toe drain near the area of the instability could not be constructed due to physical constraints. Raba-Kistner felt that there was sufficient blanket thickness to eliminate the toe drain in this area.

Raba-Kistner noted that a number of blow counts were less than 5, which may have indicated that some of the materials were weaker than was assumed for design. Raba-Kistner felt that correlations between blow counts and relative density indicated that for a friction angle of 32 deg, the relative density would be 30%; it was felt that this relative density did not reflect conditions observed at the site. A friction angle of 32 deg was considered the minimum likely friction angle.

The stability analysis was conducted using the stability software SLOPE/W, developed by GeoStudio. The results of the stability analyses reported a minimum factor of safety of all analyses of 1.7 for the end of construction condition on the landside. The results are shown in Table 2.7.

The results of the stability and seepage analysis indicated that the levee slopes were not of primary concern. Based on the results of the stability analysis slopes of 3:1 (H:V) and 2.5:1 (H:V) were considered sufficient for the waterside and landside slopes respectively.

Table 2.6. Shear strength parameters (Raba-Kistner 2011).

Loading	Material	Total Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
End of Construction (Total Stress)	Poorly Graded Sands with Silt (SP-SM)	117	0	33
	Poorly Graded Silty Sands (SM)	117	0	32
	Clays (CL & CH)	120	400	0
	Gravel (drainage blanket)	125	0	35
	Clay fill (CL and CH)	120	400	0
Steady State (Effective Stress)	Poorly Graded Sands with Silt (SP-SM)	117	0	33
	Poorly Graded Silty Sands (SM)	117	0	32
	Clays (CL & CH)	120	200	24
	Gravel (drainage blanket)	125	0	35
	Clay fill (CL and CH)	120	200	24

Table 2.7. Factors of safety from the results of design stability analysis (Raba-Kistner 2011).

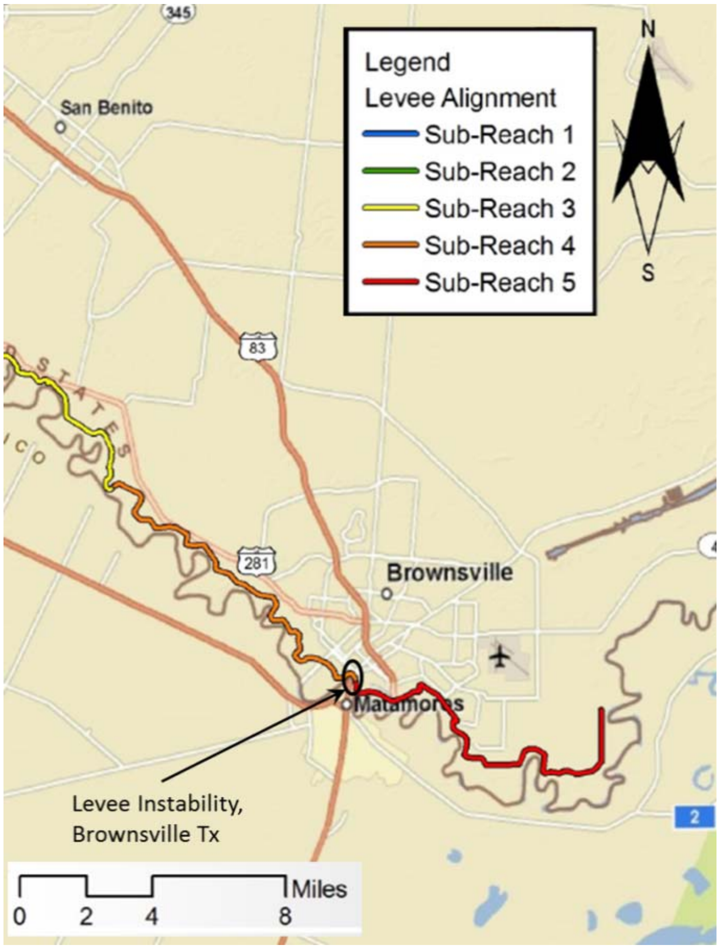
Landside of Levee		Floodside of Levee		
End of Construction	Steady State at Flood Stage	End of Construction	Steady State at Flood Stage	Sudden Drawdown
1.7	2.1	1.8	3.2	1.8

2.2.3 Upper Brownsville Levee Rehabilitation, Design Report Final Submittal, Cameron Counties, Texas (Tetra Tech 2012)

The final design report was provided by Tetra Tech in 2012 to bring the levee system from Donna Pump to Brownsville to current flood protection standards.

On the river side of the levee, the recommended levee side slope was 3H:1V and on the landside of the levee, the recommended side slopes varied between 3H:1V to 2.5H:1V. The top of the levee was reconstructed to provide a minimum width of 16 ft in most locations. The levee system was broken into five sub-reaches. The sub-reach that extends through the study area at Brownsville is in sub-reach 4 (Figure 2.2).

Figure 2.2. Location of levee sub-reaches near Brownsville (Tetra Tech 2012).



2.3 Hydraulic analysis

No hydrologic analysis was performed by Tetra Tech (2012) for the project. The design hydraulic analysis was based on work performed by the USIBWC in 2003. The expected 100-year flood event would result in flows of 20,000 cfs through the Brownsville-Matamoros area. The 100-yr flood elevation at the Gateway International Bridge would be 36.47 ft (NAVD 88).

2.4 Cameron County Engineering Division

The Gateway International Bridge is owned and maintained by Cameron County. A site visit by ERDC to the Cameron County Engineering Division at San Benito, TX, resulted in obtaining the International Bridge and Port-of-Entry (POE) approach drawings containing soil boring data from the floodplain and riverbank along the bridge alignment, as well as early topographic information shown on the drawings from the 1962 bridge design (Lockwood, Andrews, and Newnam, Inc. 1962a, 1962b, 1962c, 1968). This bridge design in Figure 2.3 corresponds to the second permanent bridge that was built at this location, based on historic land use changes in the study area.

The current bridge began experiencing settlement issues in 1984 at Pier No. 5, with distress visible in the concrete deck span. Subsequently, a geotechnical study was commissioned by the Cameron County Engineering Division to evaluate the soil conditions responsible for settlement (Professional Service Industries 1984). Included with this evaluation were geotechnical borings and results of laboratory soils testing. The remediation of the bridge pier involved the construction of deeper support piers and a support frame to the original pier (Figure 2.4). Soil layers responsible for the pier settlement are similar to those present in the area experiencing cracking.

Truck lane improvements at the bridge in 1992 resulted in another subsurface exploration program and a deep foundation design report (Trinity Testing Laboratories, Inc. 1992). Two additional soil borings were drilled in the riverbank along the bridge right-of-way and laboratory soil tests were also performed from selected samples during this effort to derive engineering properties of the soils.

2.5 City of Brownsville Water and Sewer Department

The POE area adjacent to the levee reach experiencing severe cracking contains a fire-hydrant that was visible through the border fence (Figure 2.5). A close inspection of this area was made during the initial site visit by ERDC and USACE District Galveston geotechnical personnel. There were no visible signs of distress (e.g., seepage, sinkholes, wet spots) within the POE area along the levee landside slope to suggest a utility was responsible for any soil being removed and contributing to the severe levee cracking.

A visit was made to the Brownsville Water and Sewer Department to obtain both inspection reports and pressure test data associated with the fire hydrant landside of the cracked levee area. The Water and Sewer

Figure 2.3. Gateway International Bridge details and borings (Lockwood, Andrews, and Newnam 1962c).

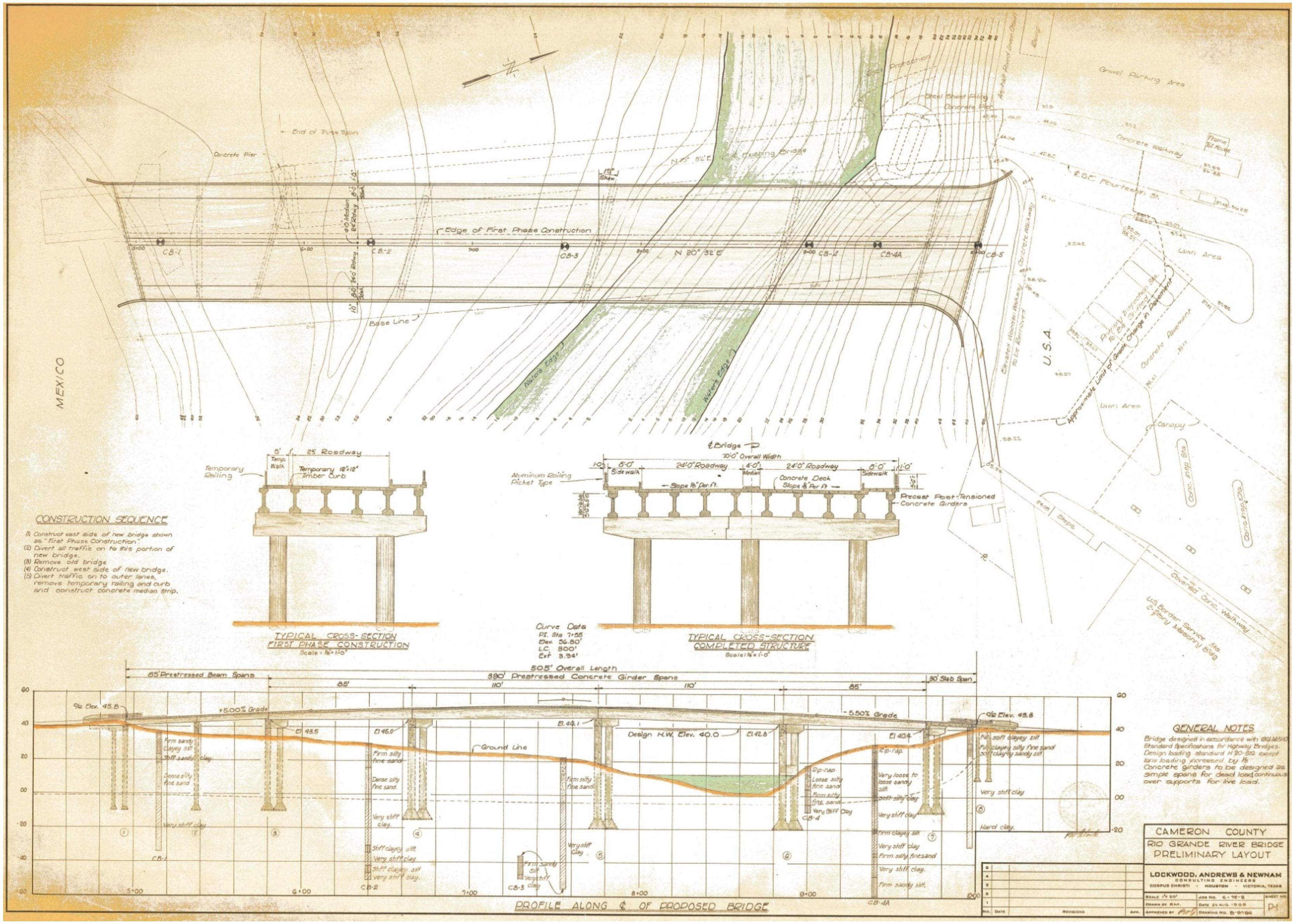


Figure 2.4. Support frame to remediate settlement of the International Gateway Bridge pier in 1984.

Figure 2.5. Location of fire-hydrant that was evaluated for possible leakage.

Department reported the fire hydrant at this location had active water pressure, and there were no reports of leakage.

The level of human foot traffic in this area and border maintenance activities observed at the POE adjacent to the cracking area would have alerted U.S. Customs or U.S. Border Patrol personnel if leakage was occurring with any of the buried water utilities and the fire hydrant nearby. Thus, it was concluded that buried utilities did not contribute to severe cracking within the study area.

2.6 Summary of existing geotechnical data

Boring data available at the onset of this investigation involved two borings that are located upstream and downstream of the distressed area from the Raba-Kistner (2009) work (i.e., DP-201 and DP-202) and nine soil borings from the Gateway International Bridge and POE area. These borings incorporate the foundation studies by Lockwood, Andrews, and Newnam, Inc. (1962c), Professional Service Industries (1984), and Trinity Testing Laboratories, Inc. (1992). These data were used in definition of the geologic site conditions.

3 Evaluation and Historical Reconstruction of the River Reach

3.1 Introduction

An important part of the technical literature review process was to determine historic land use changes and activities that have occurred in the study reach. A primary goal was to identify significant land use changes and activities that may have contributed to the levee instability within the study reach other than the levee rehabilitation work described above. This work, as previously described, involved raising the levee approximately 3 ft and re-grading the slopes to maintain the 3H:1V side slopes.

A concentrated effort was made to collect and review historic maps, charts, and photography to characterize the evolution of the river and subsequent land use changes within the study area through time. An important part of this effort were several visits made by the ERDC technical staff to the Cameron County Engineering Division, San Benito, TX, for the bridge data described in Chapter 2; the City of Brownsville Water and Sewer Department, Brownsville, TX, for data related to utilities at the POE (also described in Chapter 2); the Brownsville Historical Society, Brownsville, TX, for early photographs of the river front, and the U.S. National Park Service, Palo Alto Battlefield, Palo Alto, TX, for early historic map data. The discussion that follows presents various forms of historical information collected during this study, which has a direct bearing on the course of this investigation. A wealth of historic information exists from this area with selected information presented below to identify the major land use changes and activities that have occurred.

3.2 Historic maps

Historic maps and photographs from the study area were compiled and incorporated into a geographic information system (GIS) where applicable to compare the evolution of the study reach through time. Selected photographs and maps were spatially georeferenced to geographic position system (GPS) coordinates to permit accurate comparisons of historic land use and significant cultural features through time. Important data obtained during this study are briefly described here and their relevance to the study reach is summarized.

3.2.1 Capt. Mansfield map of 1846 (Figure 3.1)

The 1846 map is the earliest historic map obtained showing the study reach area. This map is of Fort Texas, which was later named Fort Brown in honor of Major Brown, who was killed defending the fort during the Mexican War of 1846 (Figure 3.1). Two features are noteworthy, the first being the acute orientation of the river north of the fort, and the second being the oxbow lake northeast of the fort. These two topographic features have been relatively stable since the map was made nearly 170 years ago.

Remnants of the earthen wall of Fort Brown are still visible today at the edge of the levee access road at the intersection of the golf course driving range. Embankment soils from the fort were likely incorporated into the present day USIBWC levee, which was originally a local city/county levee prior to 1932. This map shows a stable channel alignment through this area. The river has not migrated significantly since 1846.

3.2.2 International Boundary Commission (IBC) map of 1898 (IBC 1898; Figure 3.2)

This portion of the 1898 map shows nearly the same river orientation, oxbow, and the presence of Brownsville City streets and other cultural features. The remnants of the Old Fort Brown from 1846 are identified on the map, and an access road to the riverbank is shown, which likely corresponds to the first river front road right-of-way through the study reach. Also, noteworthy is the Custom's building, which is identified on Figure 3.2, and will be a prominent feature in many old maps and photographs that are subsequently presented.

3.2.3 International Boundary Commission (IBC) map of 1912 (IBC 1912; Figure 3.3)

This map is at a 1:10,000 scale and is significant to this study because of its accurate portrayal of topographic features and identification of surface elevations and channel bathymetry. This map adds four significant knowledge items to this study: (a) it depicts a 6-m-deep scour pool as evidenced by the contour lines within the cutbank of the channel, (b) the active channel through the bendway varies between 70 to 175 m wide, (c) the abandoned oxbow within Fort Brown is depicted as being 7.8-m-deep at the time of the map survey, and (d) an embankment (levee) is shown protecting the Custom's building and Brownsville downtown area.

Figure 3.1. Military reconnaissance map of Fort Brown that was sent by Capt Mansfield's in letter of 13 Jun 1846 to Brig. General Zackary Taylor.

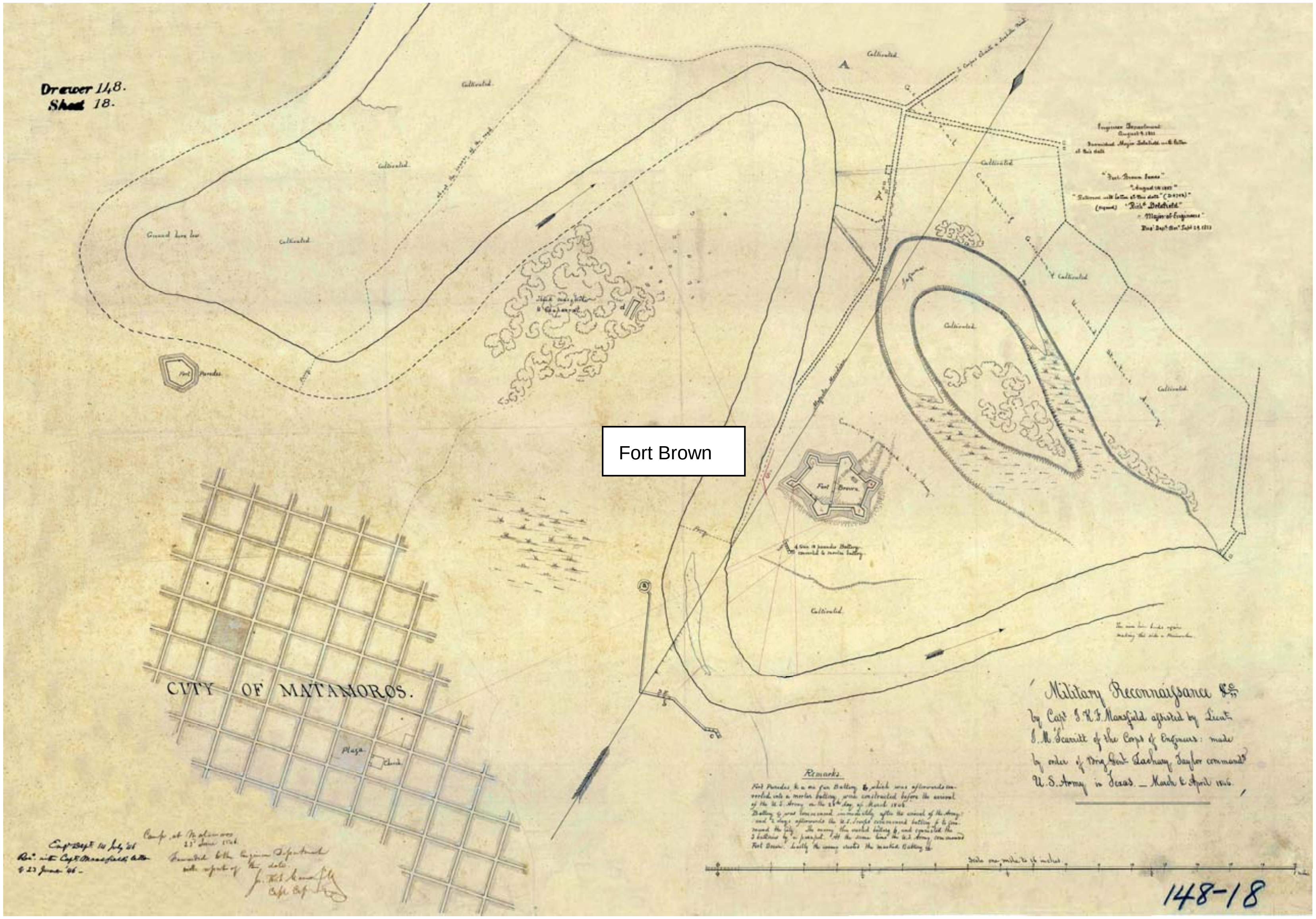


Figure 3.2. Portion of the IBC map no. 13 from survey of 1898. Red line corresponds to tentative boundary between the United States and Mexico at time of map publication in 1903. Note the location of the U.S. Customs building, which is present in old photographs of the river front.

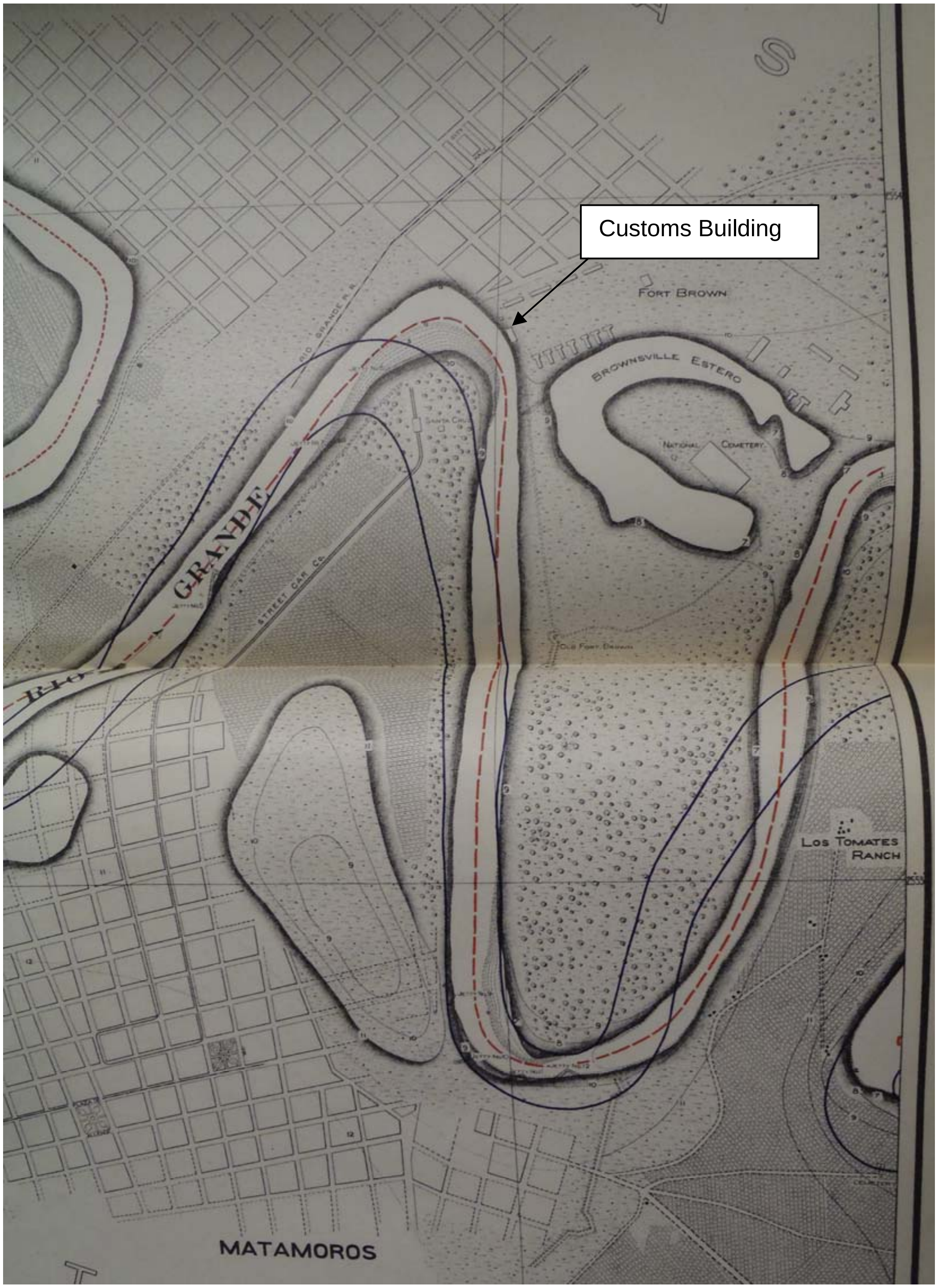


Figure 3.3. IBC map from 1912 that shows Rio Grande channel bathymetry and Lake Brown channel depth (IBC 1912).

3.2.4 USGS Brownsville topographic map of 1929 (USGS 1930; Figure 3.4)

This detailed topographic map identifies several important features: (a) the presence of the Gateway International Bridge, (b) width of the channel through the bendway is consistent with the 1912 data, (c) crossing of the 16-ft water surface contour at the downstream edge of the study area, (d) detailed elevation (contour) data from nearby abandoned oxbows, (d) the footprint of the city-county levee, and (e) presence of spot elevations of the water surface in the neighboring oxbows. The oxbows are labeled as bancos in Figure 3.4. Unfortunately, the water surface elevation at Lake Brown is not identified. However, the detailed contour data shown on the map at the upstream and downstream arms of Lake Brown identifies the water level at approximately 24 ft, which is consistent with the spot elevations identified for the nearby abandoned oxbows east of the Fort Brown area.

3.3 Historic photographs

A collection of old photographs archived at the Brownsville Historical Society show the river front in the Brownsville area since about the 1850s. A view of the waterfront riverbank from the late 1800s is presented in Figure 3.5, and a similar view between 1910 and 1915 is shown in Figure 3.6. The Customs building is a prominent feature in both of the early photographs. The riverbank at this time was absent of trees with a moderate bank slope being present as opposed to nearly vertical slopes that exist today.

The next three photographs are of the river front area during the construction of the first Gateway International Bridge in 1927 (Figures 3.7 to 3.9; courtesy of Brownsville Historical Society). The first photograph in this series (Figure 3.7) shows a view of the U.S. riverbank from Mexico during low water. The shallow sand bar identified on the 1912 IBC topographic map (Figure 3.3) is prominently visible in the middle part of the photograph, along with the early stages of construction of the Gateway International bridge pier at the U.S. side.

Figure 3.8 shows a close-up view of the timber works for the bridge pier construction, and riverbank conditions during this time, which corresponds to the area incorporating the study reach. Visible in this photograph is the presence of large stone riprap along the river edge, driftwood,

Figure 3.4. Portion of the USGS East Brownsville topographic map showing detailed 1-ft contour interval and spot elevations in feet MSL (mean sea level) (USGS 1930). Note the width of the river channel through the bend way, the presence of Gateway International Bridge, crossing of the water surface elevation contour of 16 ft MSL at downstream edge of the study area, the detailed contour information for the abandoned oxbow (banco), the levee alignment, and spot elevations shown on nearby oxbows.

Figure 3.5. Brownsville river front from late 1800s (photo courtesy of Brownsville Historical Society).

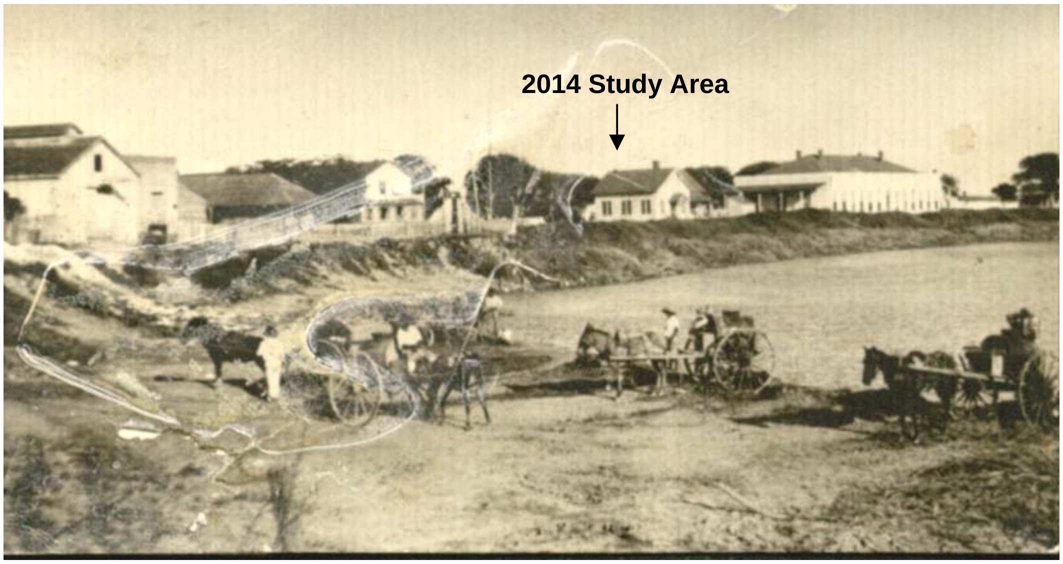
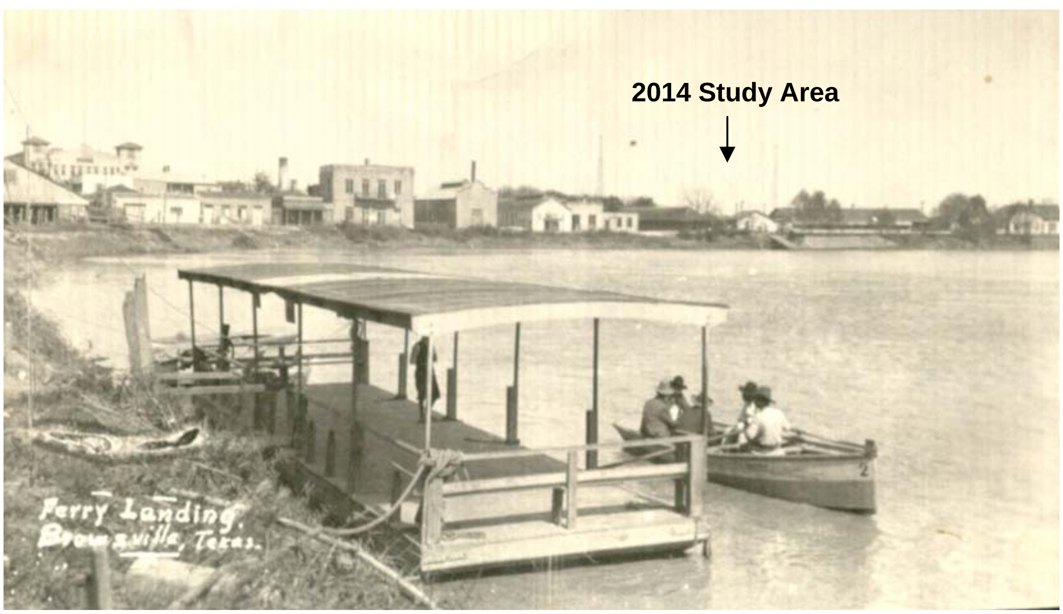
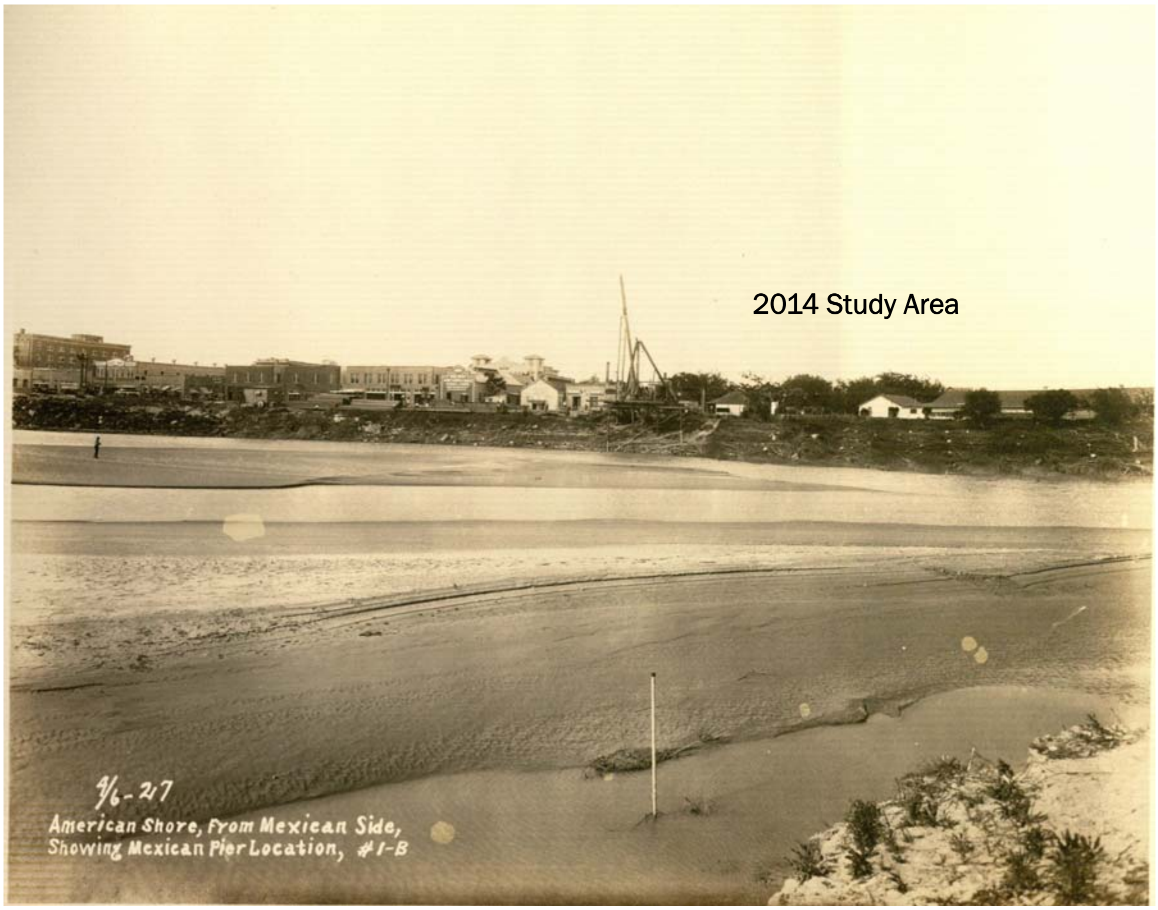


Figure 3.6. Brownsville river front between 1910 and 1915 (photo courtesy of Brownsville Historical Society).



and other debris deposited by a drop in river level. The light colored riprap shown in this photograph is possibly crystalline limestone, which was encountered in borings at 10- to 15-ft-depth drilled for this study in 2014 (described in next section). The riprap is not native to this area and likely obtained from sources outside of the LRGV to protect the bank from active migration. The bank has a relatively moderate slope to the river, and shows a much reduced levee prism than currently present, which was

Figure 3.7. View of U.S. riverbank in 1927 from the Mexican side of the river showing prominent sand bar during low water and initial construction of the pier for the Gateway International Bridge. Study area is right of the bridge pier construction.

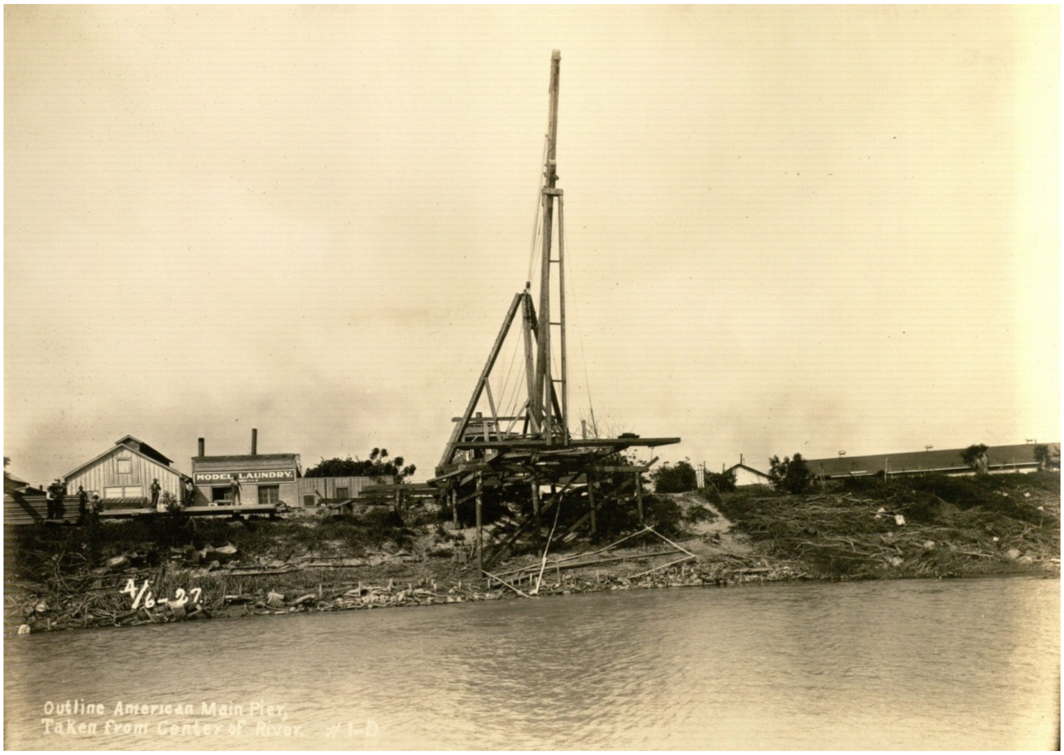


under city/county jurisdiction prior to being federalized by the USIBWC in the 1930s.

Figure 3.9 is taken from beneath the pier with a view looking downstream shows riverbank conditions along the entire study reach. As this photograph clearly shows, the river channel and bank are significantly different than conditions that currently exist today. The channel is much wider, with a moderate side slope that is approximately 1V to 3 H. The bridge was completed by 1928. The USGS topographic map from 1930 (Figure 3.4) identifies the bridge alignment and is consistent with the early photographs showing construction details.

Figure 3.10 shows parts of two Tobin aerial photographs from the Brownsville area (East and West Brownsville 7-1/2 topographic quadrangle maps) in 1930 and confirms the existence of a much wider river channel than

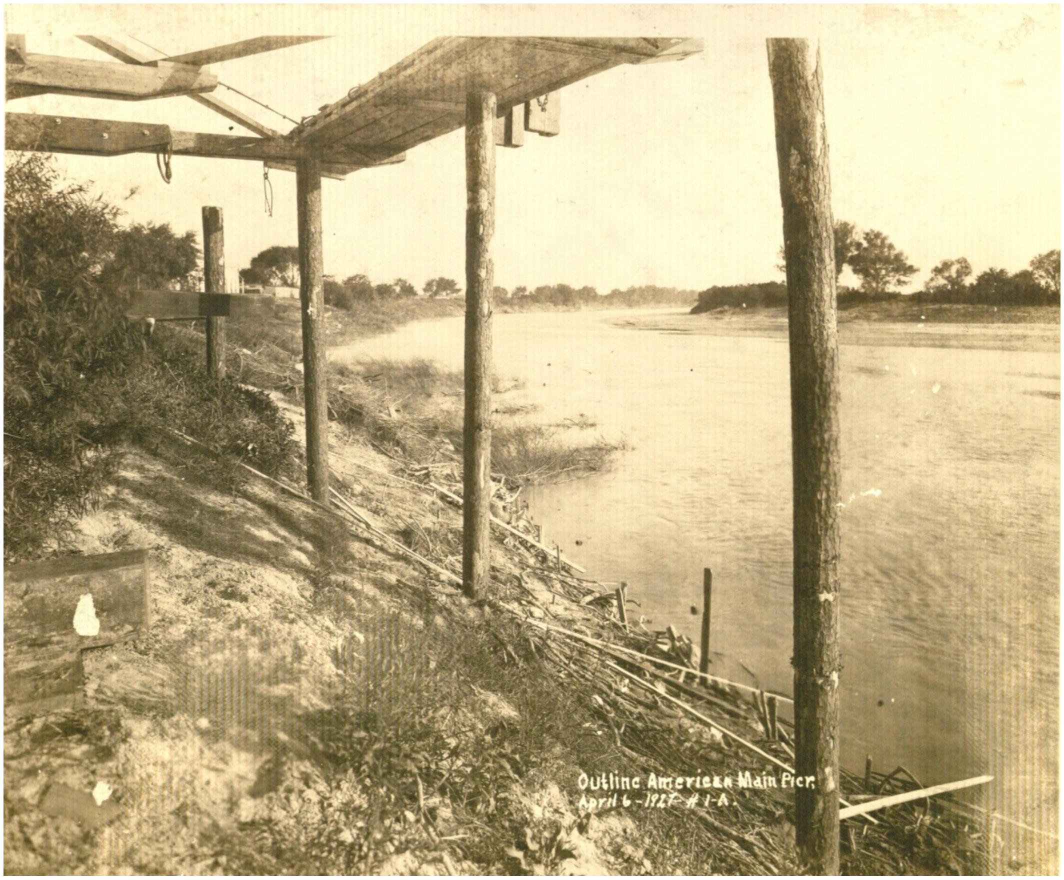
Figure 3.8. View of the riverbank in 1927 during initial construction of the pier for the Gateway International Bridge. Study area is right of the bridge pier construction.



present day limits. The 1930s channel limits have been imposed onto a 2014 Google Earth image of the study reach, which shows the infilling of the Rio Grande by river-borne sediment since 1930 (Figure 3.11). This infilling is the direct consequence of considerably reduced annual river flows caused by the construction of several upstream dams between 1950 and 1970, and the ever increased demands of agricultural irrigation use and water supply within the LRGV from expanding population growth. The maximum limits of the 1930 river channel generally correspond to the current day levee toe (Figure 3.10).

The final series of photographs are aerial obliques of the first and second Gateway International Bridge and surrounding area (Figures 3.12 and 3.13). These two photographs show the subsequent changes that have taken place since the late 1950s, but before the current POE facility was built. The two photographs are of the same river reach, but with different versions of the Gateway International Bridge shown. The steel frame bridge that was built in 1927 and 1928 was replaced during the early 1960s with the current two bridge design. The first pier of the new bridge on the

Figure 3.9. View of the study area riverbank in 1927 from beneath the frame support for the Gateway International Bridge pier. View is looking downstream and shows a bank slope of approximately 3H:1V.



U.S. side began experiencing problems with settlement in 1984 as previously described (Figure 2.3).

3.4 Summary

The preceding historical review of the land use changes in the study reach is used to evaluate the horizontal and vertical limits of the alluvial soils, the nature of the bank stratigraphy present, and the underlying prehistoric deposits within the study reach. The drilling and soil sampling part of this investigation involves definition of both the horizontal and vertical limits of the different alluvial soil units comprising the bank, definition of their associated engineering properties, and interpretation of these soils in terms of their historic and prehistoric context.

Figure 3.10. East and West Brownsville 1930 Tobin Photographs with channel limits of Rio Grande outlined in yellow.

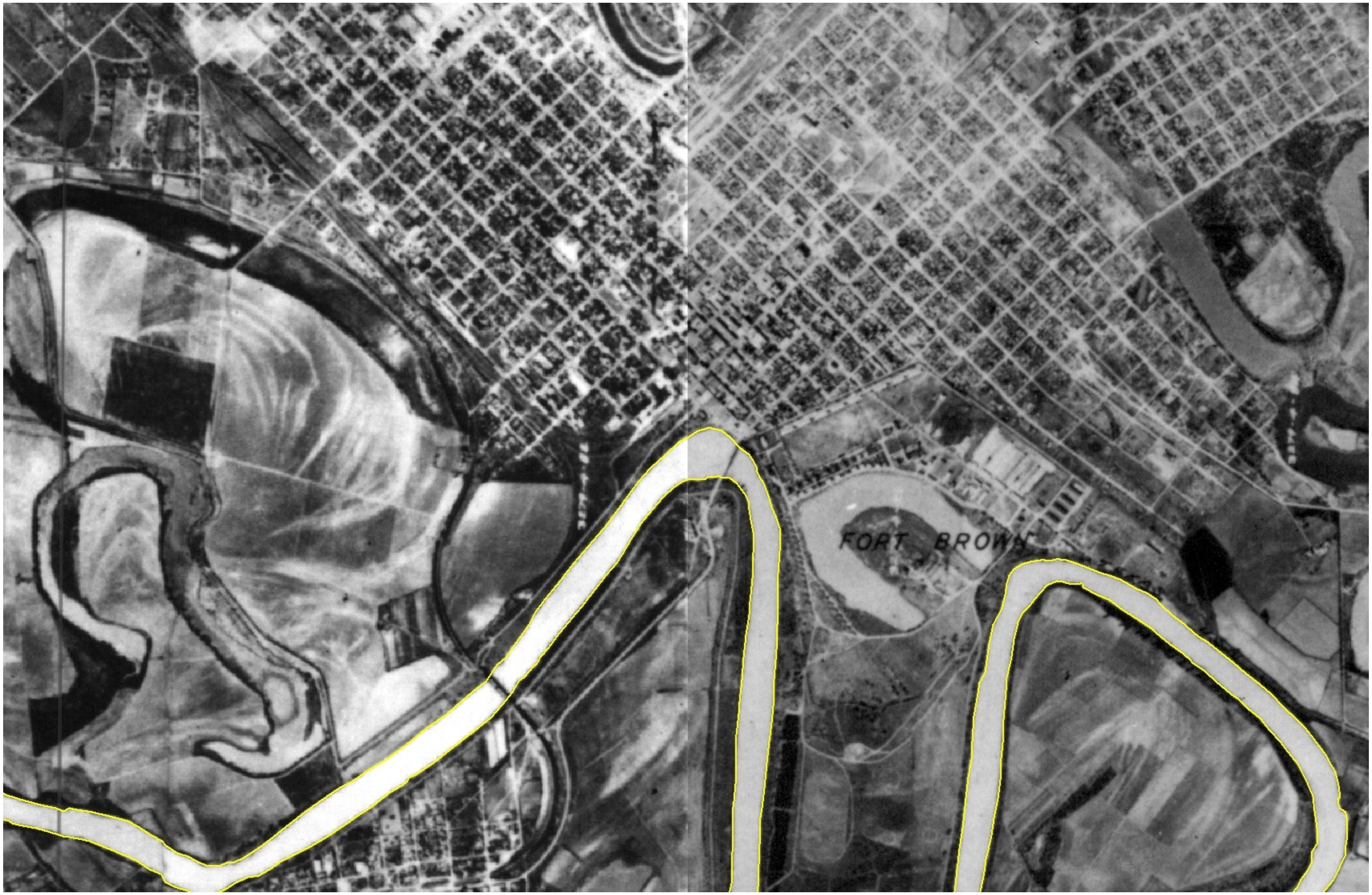


Figure 3.11. 2013 Bing image with 1930 Rio Grande channel limits shown by yellow lines. Edge of the 1930 river channel corresponds to approximate toe of current levee. Gateway International Bridge shown is the second bridge at this location.

Figure 3.12. View of first Gateway International Bridge approximately middle–to-late 1950s (courtesy of Brownsville Historical Society). Note the Customs building is still present in this photograph, which has been prominent landmark on past historic maps and early photographs.

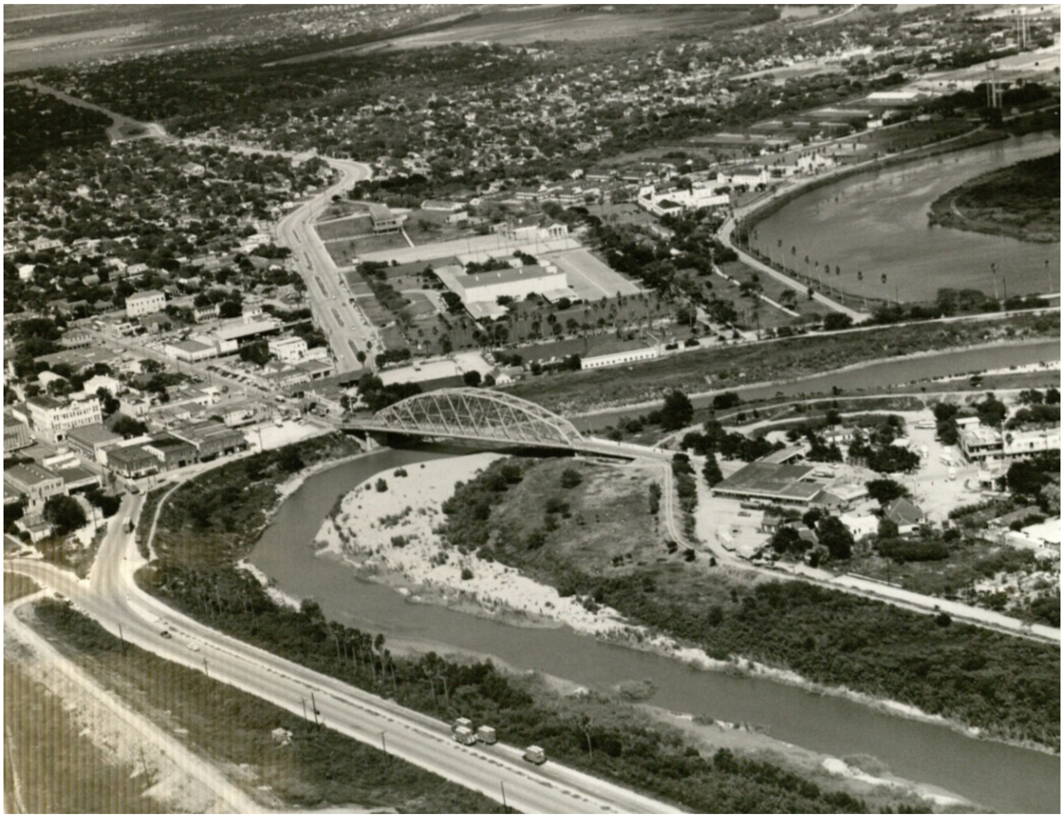
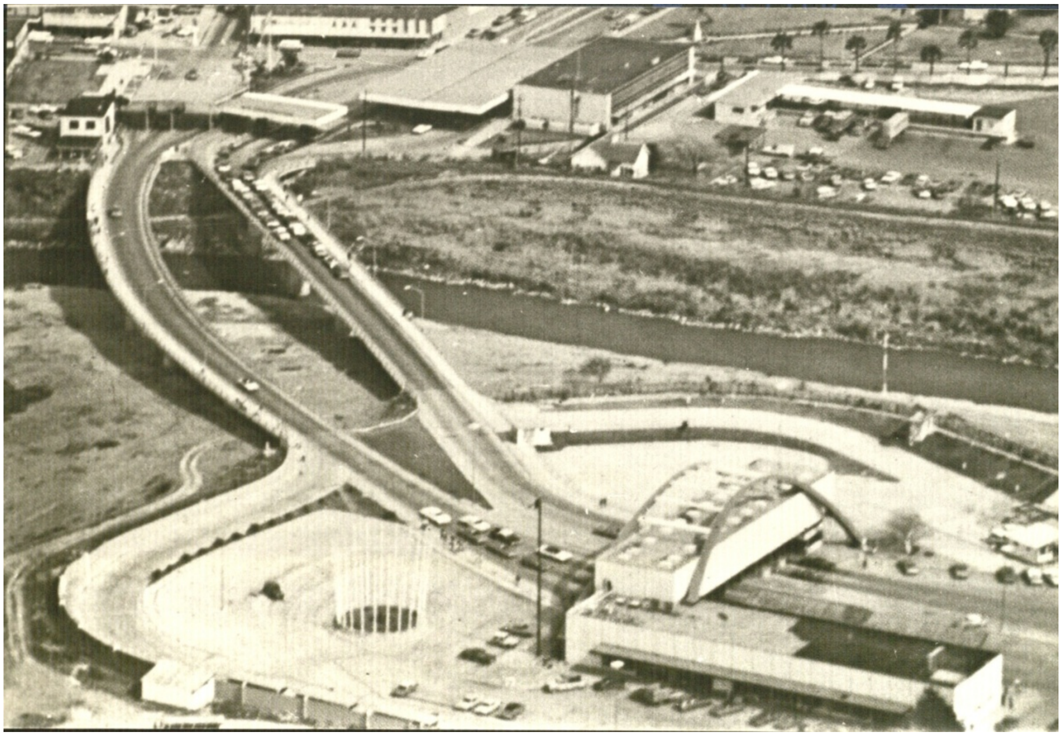


Figure 3.13. View of second Gateway International Bridge from Mexico approximately late 1960s or early 1970s (courtesy of Brownsville Historical Society).



4 Field Investigations

4.1 Introduction

The field investigations began with a reconnaissance of the site by the ERDC geotechnical team. This work was followed by cone penetration tests (CPTs), geotechnical borings, slope stability instrumentation, installation of piezometers, real-time groundwater monitoring, bathymetric and terrestrial LiDAR (Light Detection And Ranging) surveys of the levee reach, and periodic elevation surveys of the land surface to determine the magnitude of any ongoing movements.

The primary focus of this chapter is to present the different field data that were collected and to provide a general framework for subsequent discussions about these different data. The various field investigation activities performed to identify and evaluate the causes of levee cracking are further described in this section in the order of their occurrence during this study.

4.2 Site visit

An initial site visit to the Brownsville levee reach was conducted during the first week of July 2014 by members of the ERDC geotechnical team and USIBWC personnel. Accompanying the team were geotechnical personnel from USACE Galveston District and Headquarters, USACE (HQUSACE). The ERDC team requested the presence of the HQUSACE member because of his long-term experience with slope stability problems when he worked at ERDC. The purpose for the site visit was to assess the nature of the cracking problem and to develop a strategy for the field investigation phase of the study (Appendix A).

Three longitudinal crack sets, extending between levee stations 1898+00 to 1904+00, had developed as shown by Figure 4.1 (see also Figures 1.2 to 1.4). These crack sets were grouped based on their position and by the displacement exhibited. Cracking may likely extend beneath the riprap that was used to armor the slope beneath and downstream of the Gateway International Bridge. However, rock was not removed between stations 1897+00 to 1898+00 to verify the crack limits under the riprap section. Pin flags with colors designated in Figure 4.1 were used to highlight and mark the major crack sets as shown by Figure 4.2 to 4.4. GPS mapping with a Trimble model GeoXH was subsequently performed to accurately

Figure 4.1. Location of major crack sets in the study reach denoted by color and levee stationing in yellow (merged Bing and Google images).

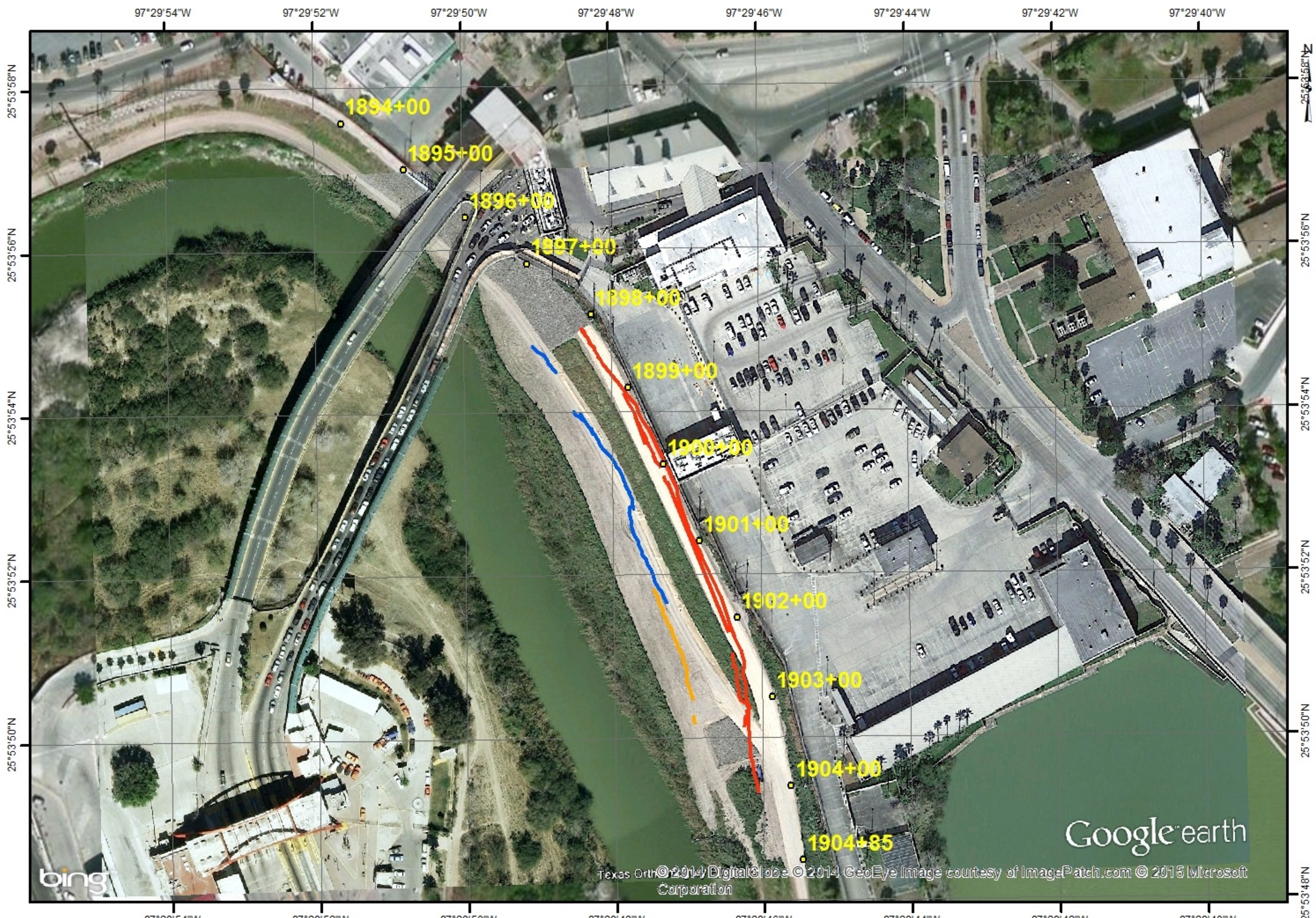


Figure 4.2. Two prominent cracks through the levee embankment at downstream end of study area that merge with cracks at the levee crest (see also Figures 1.2 to 1.4).

Figure 4.3. View of crack set at levee toe looking downstream. Crack extends through the gravel ridge in middle part of the lower photo and to the levee access road in top photo (see Figure 4.1 for location of crack).

Figure 4.4. Longitudinal crack along the downstream end of the riverbank (see Figure 4.1 for location).

locate the cracks. Results of this mapping effort are shown on the aerial image of the study site in Figure 4.1 using GIS technology.

4.3 Drilling and sampling program

A soil boring and sampling program was conducted in September 2014 as part of the field investigation program to collect site-specific geotechnical properties of the subsurface, to map the stratigraphy of the levee and riverbank soils for use in conducting slope stability analyses. The soils exploration program consisted of 32 CPTs performed in a phased approach to obtain maximum information about the levee site (Figure 4.5). Following completion of the CPT program, six soil borings were made at selected locations for visual correlation of the CPT data, to obtain soil samples for laboratory testing, and for installing instrumentation (Figure 4.6). Soil sampling included collection of both undisturbed (3-in. Shelby-tube) and disturbed (split-spoon) samples.

Figure 4.5. Location of CPTs (merged Bing and Google Earth images).

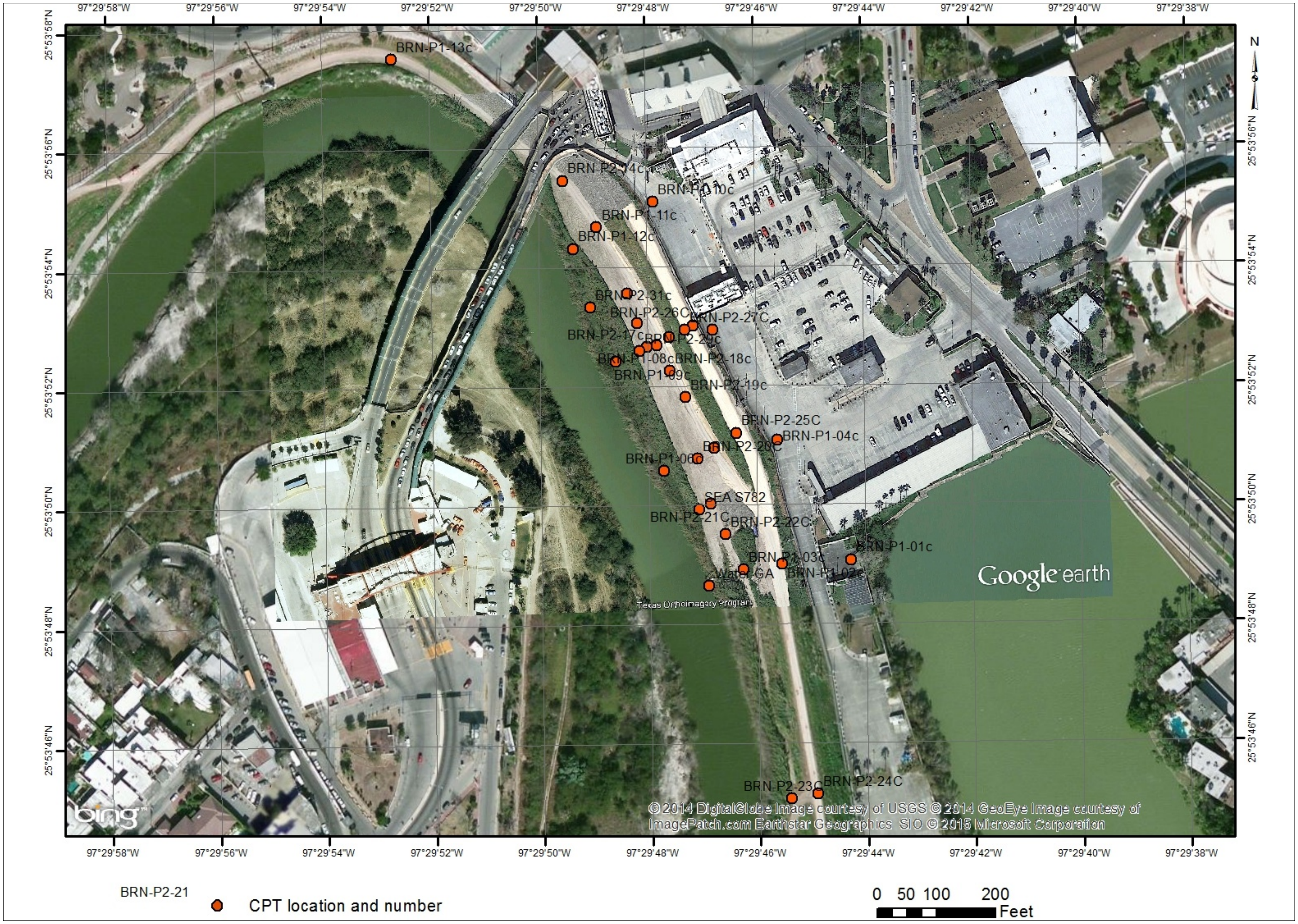
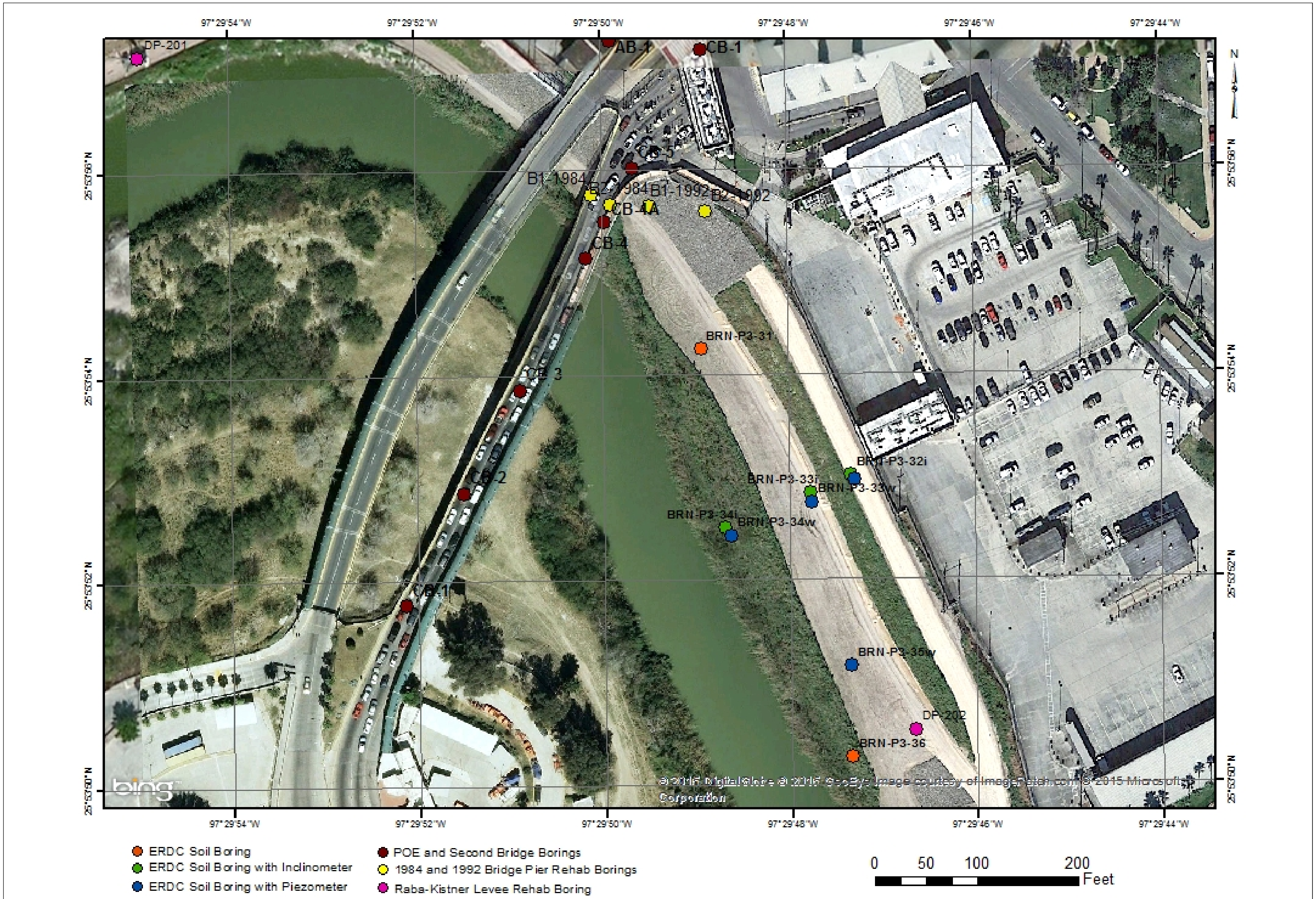


Figure 4.6. ERDC drilled borings showing location of inclinometers (green), piezometers (blue), and lithology borings (red). Backdrop is a Google Earth image of the site from 2014 prior to levee cracking and slumping.



4.3.1 CPT soundings

Phase I and Phase II CPT fieldwork began on 29 July 2014. The CPTs were pushed by USACE Savannah District and were completed on 5 August 2014. CPT locations are shown on Figure 4.5. CPT interpretive plots of the results logs of the individual soundings are presented in Appendix B. Depth of the investigation by the CPT soundings ranged from less than 10 ft to 70 ft. Soundings were taken on the crest levee, levee toe, and near the edge of the riverbank. CPTs near the edge of the river encountered buried riprap between 5- and 10-ft depth and were not pushed beyond the refusal limit. Soil borings identified a 2- to 5-ft-thick limestone rock interval at relatively shallow depths near the edge of the river that likely represents local historic bank stabilization efforts.

Soil type and soil stratigraphy from CPT data were interpreted using empirical relationships developed by Robertson et al. (1986), Robertson (2014), and Geologismiki Geotechnical Software (2014), herein referred to as Geologismiki software. CPT cross sections were compiled from these data using the Geologismiki software and are presented in Appendix C. Similarly, soil strength models from the CPT soundings were developed using the Geologismiki software and are presented in Appendix D.

Soil profiles and soil strength models in Appendices C and D were used to plan the locations of the geotechnical borings that were drilled in the next phase of this study. This information was also used and to determine the depth of the soil beneath the levee and floodplain. The soil borings were drilled to visually inspect the underlying soils and stratigraphy, verify relationships observed in the CPT data, and obtain soil samples for laboratory testing. Samples were tested in the laboratory to determine soil strength properties for use in the geotechnical analysis.

CPT data identify a riverbank and levee formed of mainly fine-grained, low shear strength soils. Soil profile plots presented in Appendix C and D represent the first approximation of the horizontal and vertical extent of soil texture and provide a general measure of soil strength between borings. CPT soundings are an ideal method to rapidly explore a site and to correlate basic properties, such as soil texture and general stratigraphy across the site. Results of CPT soundings and associated soil strength models will be described in detail in subsequent chapters of this report where applicable.

It should be noted that the interpretation of the CPT data does not account for unconformities in the stratigraphy because of erosion and chronologic breaks in deposition of sediment by different Rio Grande courses, or from weathering due to changes in the river's base level resulting from global sea level fluctuations. Recognition of age- and stratigraphy-related features requires visual examination of soil cores to identify fundamental soil properties, which includes texture, color, grain size, mineralogy, consistency, stiffness, presence of mottling, occurrence of concretions, organics, fossils, buried soil horizons, and other evidence of chemical and physical weathering of the underlying soil.

4.3.2 Pore pressure dissipation tests

Quick pore pressure dissipation tests, generally no more than seven minutes in duration, were performed in CPT soundings whenever increased tip resistance indicated probable sand layers. Tests where pressures came to equilibrium in this period indicated the presence of sand layers and associated hydrostatic water levels. Dissipation test results are presented in Appendix E.

4.3.3 Soil borings

Six soil borings ranging in depth from 50 to 70 ft were drilled at the levee crest, levee toe, or near the edge of the riverbank (Figure 4.6). Soil sampling was generally accomplished using Standard Penetration Test (SPT) methods in the borings, with continuous undisturbed sampling using Shelby tubes performed at selected depths in borings P3-33 and P3-34.

Split-spoon sampling was performed using a standard split-spoon, 140-lb hydraulic hammer, and a 30-in. weight drop. Blow counts were recorded for each 6-in. of sample penetration. Sample refusal was defined as more than 25 blows per 6 in. Split-spoon samples were logged in the field by a geologist and sealed in jars for later laboratory classification, sieve testing, and water contents.

Undisturbed samples of the levee embankment and riverbank were recovered using 3-in. Shelby tubes having a length of 30 in. Shelby tube samples were sealed in the field to preserve soil moisture, and they were later extruded in the laboratory under controlled climate conditions. Engineering properties measured in the laboratory for selected recovered samples include soil texture, grain-size distribution, moisture content, Atterberg Limits, and shear strength.

Field logs were prepared for each boring. The logs described the sampling methods employed and other data that are relevant to geotechnical-type soil sampling—namely, texture based on the Unified Soils Classification System (USCS), number of blow counts per 6 in. of spilt-spoon penetration, soil color, moisture, groundwater occurrence, consistency or stiffness, grain size characteristics, bedding properties, mineralogy, presence of organics, weathering, and other relevant data. A pocket penetrometer was used on the fine-grained samples to estimate the soil strength with depth. These values are included on the field boring logs (Appendix F).

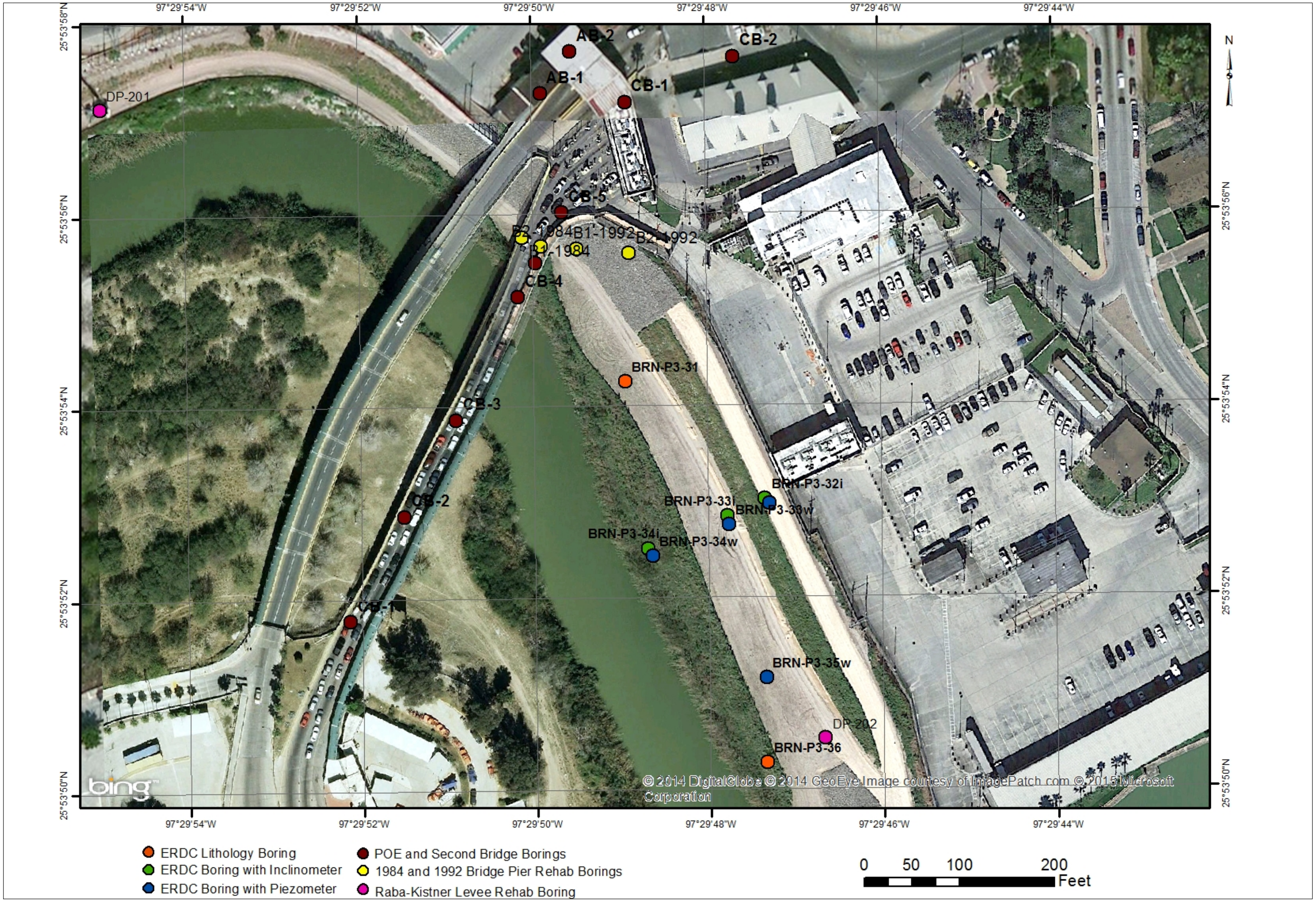
In addition to the borings and CPT data collected by USACE, historic boring and laboratory test data from the study site were examined and evaluated to characterize the site's soils and stratigraphy. The locations of the borings are presented in Figure 4.7. A complex stratigraphy was interpreted from the results of the boring program. The riverbank is composed primarily of a 30- to 35-ft-thick layer of gray to dark gray, fine-grained historic alluvium, with blow counts ranging from 2 to 4 blows per 6 in. penetration, underlain by a stiff to very stiff, uniform tan or brown layer of alluvial clay that is estimated as Late Pleistocene (between 10,000 to 120,000 years before present) as evidenced by its physical and engineering properties.

Blow counts recorded for the Pleistocene clays were normally higher than the overlying historic fill and ranged from 4 and 10 blows per 6 in. penetration. The uniform tan-to-brown color and increased stiffness are considered to be diagnostic soil properties. This color and stiffness correspond to alluvial sediments that were likely oxidized and underwent weathering of the exposed alluvial surface more than 15,000 years ago when sea level was much lower, because of the presence of wide-spread continental ice sheets that covered much of the North America continent.

4.3.4 Monitoring program

Three different monitoring methods were used at the levee site to detect the occurrence of continuing movements and deformation of the levee slope and riverbank.

Figure 4.7. Location of all borings used to characterize the levee site. Backdrop is a merged 2013 Bing and 2014 Google Earth image of the site prior to the levee cracking and slumping.



Instruments included piezometers to determine the elevation of the groundwater in pervious strata, inclinometers to determine both depth and rate of movement, and surface elevation surveys for performing continuous monitoring of the levee reach to monitor and to also establish base line conditions for later surveys. Types of elevation surveys performed included placing reference markers along the levee slope and bank at selected locations to monitor surface movements in the x (easting), y (northing), and z (elevation) directions; a bathymetric survey of the river channel to determine characteristics of the channel itself; and a ground LiDAR survey of the levee slope and exposed river bank to accurately measure deformation across the study reach and to establish a base line reference for future surveys. These different monitoring techniques and methods are described in more detail in this section.

4.3.5 Groundwater monitoring

Four piezometers were installed to assess groundwater conditions (see Figure 4.6 for locations). Piezometers were built using 1-1/2-in. schedule 40 PVC casing and 5-ft lengths of manufactured well screen with slot openings of 0.006 to 0.125 in. Screen length was variable and was dependent on the underlying stratigraphy. Table 4.1 identifies relevant information about the four piezometers that were installed. Information identified in Table 4.1 includes the screen depth and corresponding elevation, min and max water level depths recorded over the period of record (14 October to 16 December 2014) and corresponding elevation, and stratigraphic interval that was screened. The stratigraphic intervals identified in Table 4.1 were classified as being either historic, Holocene, or Pleistocene alluvium.

Well completion for each piezometer involved placing fine-grained filter sand around the well screen in the boring annulus to approximately 2 ft above the top of screen, followed by a 2-ft interval of bentonite pellets, and topped with a standard Portland cement and bentonite grout mix to the surface. A concrete pad containing a flush mount steel cover was constructed over each monitoring well location. Flush-mounted construction was designed to prevent damage to the piezometer from mowing equipment and other vehicle activities.

Grouting of the piezometers was by way of a tremie pipe such that the dense grout mix would displace any water from the borehole as grouting progressed to the surface. Mixing of the grout was accomplished by using a

Table 4.1. Characteristics of screened interval for Brownsville piezometers. Water level data range from 14 Oct to 16 Dec 2014. Water level data recorded with Solinst levelloggers in each well.

Boring	Elv Top Casing	Depth (ft) Top of Screen	Depth (ft) Bottom Screen	Elv Top Screen (NAVD88)	Elv Bottom Screen NAVD88)	Min Depth (ft) Water	Max Depth (ft) Water	Max Elv Water (NAVD88)	Min Elv Water (NAVD88)	Screen Interval
BRN-P3-32W shallow	39.93	20.00	35.00	19.93	4.93	16.13	17.38	23.80	22.55	Holocene
BRN-P3-32W deep	39.95	65.00	75.00	-25.05	-35.05	18.60	19.77	21.35	20.17	Pleistocene
BRN-P3-33W	30.61	14.00	25.00	16.61	5.61	12.23	13.03	18.39	17.58	Historic
BRN-P3-34W	23.08	15.00	20.00	8.08	3.08	10.15	11.86	12.94	11.22	Historic
BRN-P3-35W*	31.67	46.50	62.80	-14.83	-31.13	8.25	9.36	23.42	22.31	Pleistocene

* Estimated elevation – top of concrete slab needs to be surveyed.

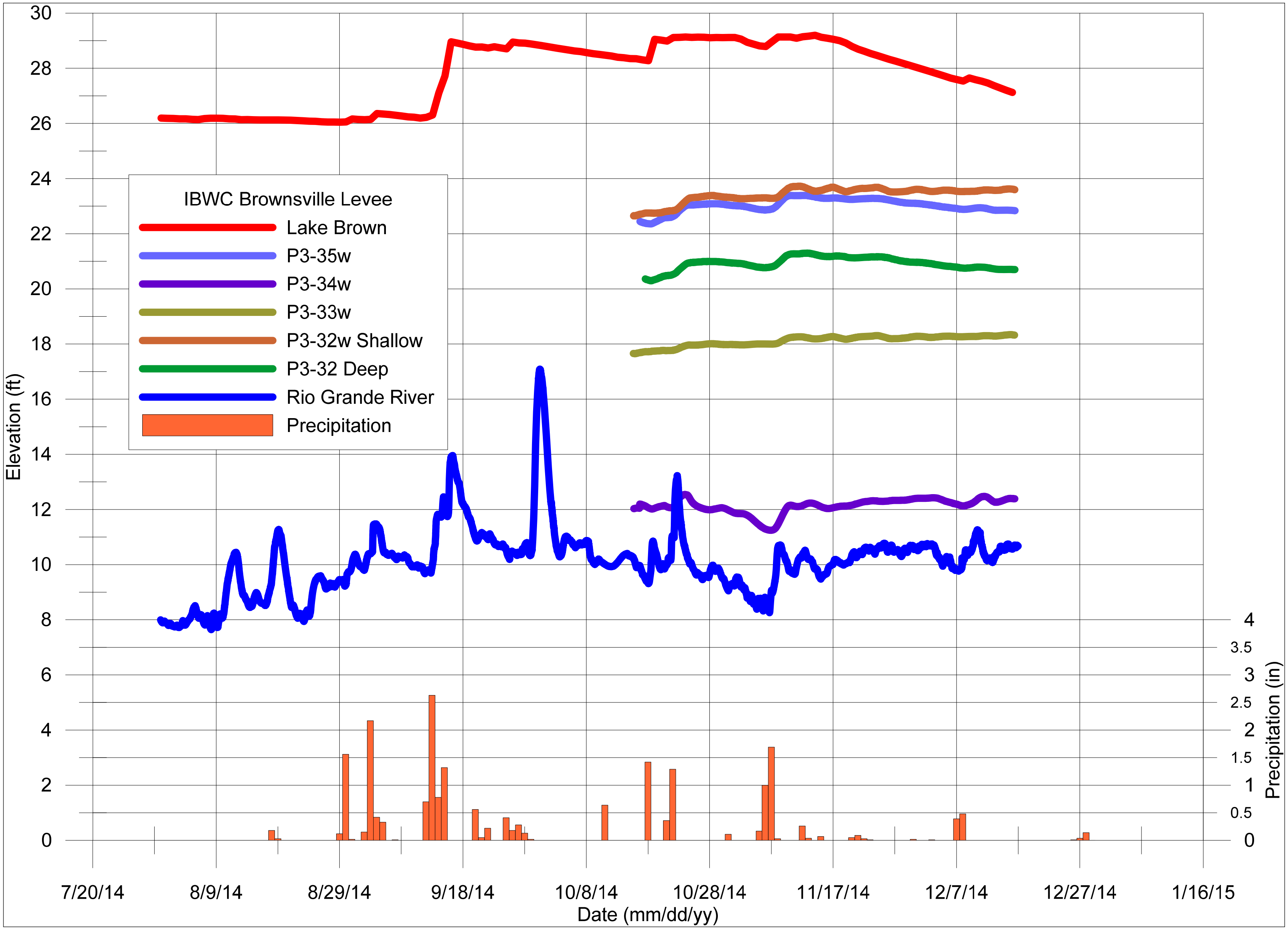
portable gasoline type piston pump. Well completion diagrams for each piezometer are included with the boring logs in Appendix F. Monitoring wells were developed using a bailer and removing 3 to 5 volumes of water from the pipe. This development technique ensured that the water in the screened interval was clear and any fines present would be removed.

Automated Solinst Levelogger pressure sensors/recorders were installed in each piezometer to permit real-time water level monitoring. A barometric pressure sensor was installed at the levee crest (BRN-P3-32 shallow) and used to correct the data due to changes in atmospheric pressure. Water level sensors were placed into service in each well about mid-October 2014. The loggers were set to read every hour. Data were downloaded from the loggers on 16 December 2014, with a portable data logger. The period of record for water level data during this reporting phase of the report is mid-October to 16 December 2014.

Plots of the groundwater data are presented in Figure 4.8 and show minor fluctuations over the period of record. Water level data in BRN-P3-34W, which is nearest the river, tends to have the greatest variability as would be expected because of the precipitation relationship to the river. However, all the wells reflect a sudden change in groundwater levels at the end of October 2014 due to heavy rainfall at this time. Groundwater elevations indicate a general connection between the different strata as evidenced by the graph in Figure 4.8, and the well screen intervals identified in Table 4.1.

An automated water level recorder was placed in Lake Brown at the start of the study to monitor lake level fluctuations. In addition to local surface drainage to the lake, the water level in the lake is maintained by personnel from the Brownsville Water and Sewer Department pumping water, as needed, from the Rio Grande. Pumping of water from the Rio Grande is conducted with a trailer-mounted pump that is placed into service as needed. The pump location is located at the downstream end of the study area, near the access road to the river bank. No historical records of pumping frequency or duration were found for filling of Lake Brown. ERDC technical personnel concluded from discussion with city employees that the water level was maintained locally on an as-needed basis. It was further understood that the University of Texas, Southmost Campus, was withdrawing water from the lake for cooling water for campus cooling and heating equipment and discharging the heated effluent back into the lake.

Figure 4.8. Lake Brown stage and monitoring well elevation vs. time.



The water level in Lake Brown during the study period varied between elevations 27 to 29 ft. Locally, the lake level elevation corresponds to the groundwater surface, while near the Rio Grande, the piezometric surface in BRN-P3-34 is at elevation ~12 ft, nearly a 15- to 17-ft head difference between Lake Brown and the river. The lake contains a hydraulic connection to the underlying stratigraphy beneath the levee foundation through pervious point bar deposits that were formed during the oxbow migration and cut-off process. These pervious sediments form the southern limits of the study area near the levee access road to the riverbank and road under the Gateway International Bridge. Geologic data will be presented in detail in the next section of this report, which will help clarify groundwater conditions and provide a better understanding of the hydraulic relationships in the stratigraphy.

4.3.6 Inclinometers

Instrumentation included installation of three inclinometer casings in boreholes at the levee crest, toe, and at the edge of the riverbank, approximately in the middle of the levee reach (Station 1900+13), to monitor for signs of ongoing slope movements (see Figure 4.6 for inclinometer locations). The goal for installation of the inclinometer casing was to determine the specific depth of the slide zone/surface, the soil layers responsible for the underlying movement, and to quantify the rate of movement should the slide wedge still be active. This information is needed to fully understand the magnitude of the problem and develop effective long-term remediation solutions.

Plastic acrylonitrile butadiene styrene (ABS), QC-type, inclinometer casing from Durham Geo Slope indicator (DGSI) was installed into the levee crest, toe, and riverbank boreholes to depths of 80, 70, and 60 ft, respectively. Each casing was built using 10-ft lengths of QC casing with 3.34-in.- (85-mm-) outside-diameter (OD) and grouted into place through a quick-connect valve at the bottom of the casing. The grout mix was a 500-lb/ft^2 compressive strength mix containing Portland cement, bentonite, and water according to specifications in DGSI (1997). Grout was mixed using a small gasoline powered piston type pump. The grout mix was customized by weighing the components (i.e., water, cement, and bentonite) to match the volume of the grout pump used by the USACE drill crew. This grout mix was designed to match the strength and deformation characteristics of the surrounding embankment and riverbank soils. A concrete pad containing flush-mount steel covers was constructed over each inclinometer.

Flush-mounted construction was to prevent damage to the inclinometer from mowing equipment and other vehicular activities.

The basis for measuring deformation in a borehole involves a slotted inclinometer casing, a portable probe with two tilt meters oriented 90 deg apart, and an electrical cable, which transmits the output of the tilt meters to a console unit at surface. The tilt meter unit rides in the slotted casing, and measurements are made in the plane of interest through the entire casing depth. The output unit presents the angle of inclination in the x and y directions according to depth (Figure 4.9). Slots in the casing are oriented 90 deg apart, parallel and perpendicular to the levee axis and the casing (referred to as “a” and “b” in Figures 4.9 through 4.11). The casing was installed vertically. Readings were taken at each 2-ft interval along the casing depth. Stable ground above and below the zone of movement serves as a datum from which the deformation is measured. Depth of the casing was estimated to be below the zone of potential movement.

Baseline readings were taken by ERDC field personnel on 14 October 2014, using the inclinometer probe from USACE Fort Worth District. Subsequent readings were taken on 16 December 2014 and, most recently, between 27 and 28 January 2015. Results of the three inclinometer surveys to date are graphically shown in Figures 4.9 through 4.11. Inclinometer data measured to date identifies a zone of movement between 34 and 38 ft deep at the levee crest in I32 (Figure 4.9), between 40 and 44 ft deep at the levee toe in I33 (Figure 4.10), and between 32 and 36 ft deep at the edge of the riverbank in I34 (Figure 4.11). Maximum total displacement of the bank at all three inclinometers is approximately 1 in. riverward since the first measurement was taken on 15 October 2014.

4.3.7 Surveying

The final monitoring technique employed during the course of this study involved three different types of survey methods: traditional, bathymetry and side scan sonar, and LiDAR. Traditional elevation surveys along the levee embankment and riverbank were performed to determine the extent of horizontal and vertical movements at three surface profiles through time. A bathymetry and side scan sonar survey was performed of the river channel to determine channel topography below the water surface. Last, a ground-based LiDAR survey was made to determine the surface topography as of 12 September 2014, and measure surface displacement across the entire riverbank and levee slope between the current condition and

Figure 4.9. Inclinometer data as of 27 Jan 2015 for I32.

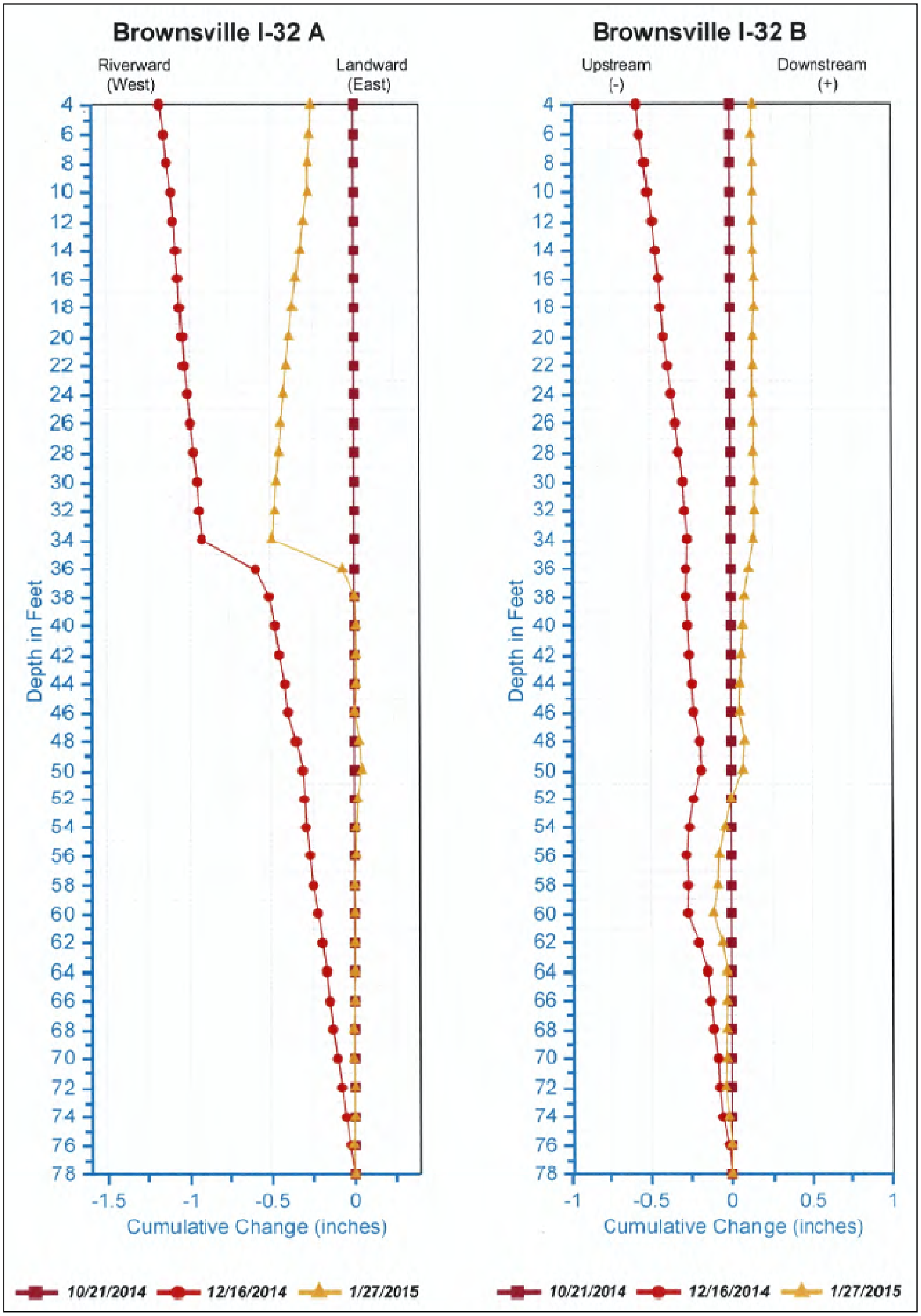


Figure 4.10. Inclinometer data as of 27 Jan 2015 for I33.

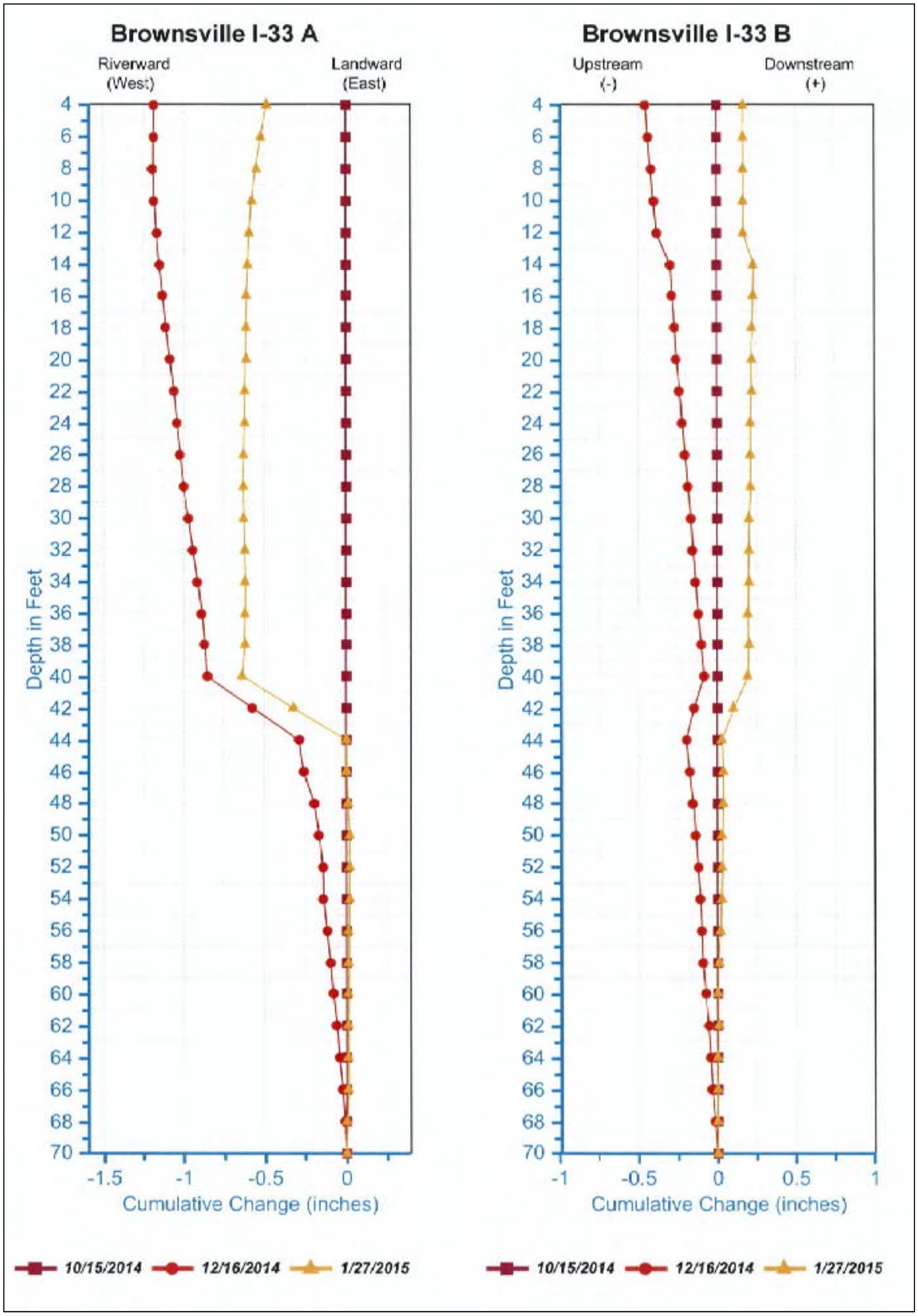
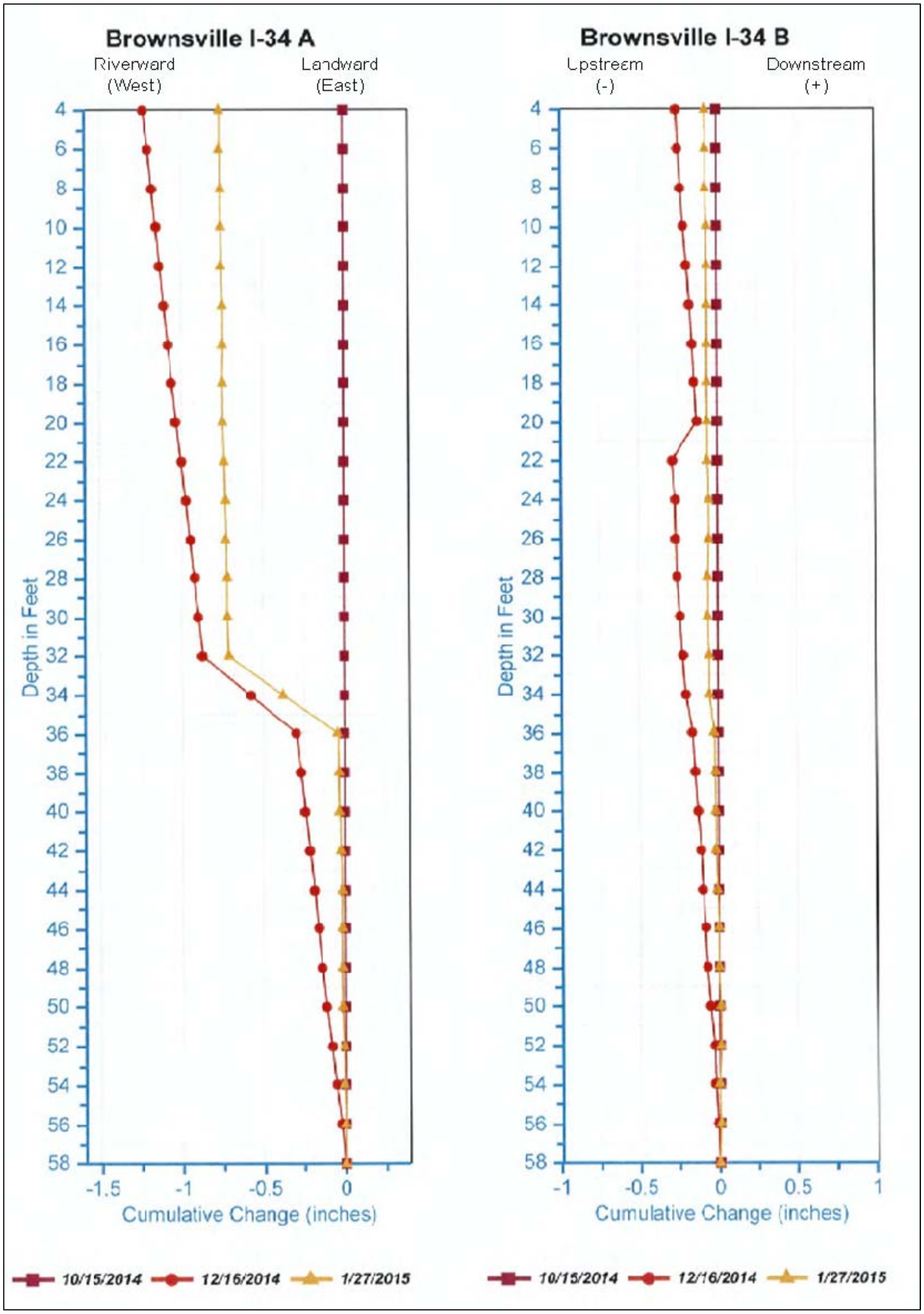


Figure 4.11. Inclinometer data as of 27 Jan 2015 for I34.



those shown on the plans and spec data following the rehabilitation. Additionally, the LiDAR dataset establishes the base line conditions for later surface surveys if warranted.

4.3.8 Survey profiles

Survey pins or markers were installed along three transects or profile locations (Figure 4.12). Repeat surveys were performed by two different survey groups during the course of this study. Galveston District survey crews conducted surveys from the end of July to August 2014, and an ERDC survey crew began surveys in late August 2014. Additionally, initial pre-failure survey data were provided by Vista Sciences Corporation, a USIBWC contractor performing construction inspection of the rehabilitation work following the new construction, and shortly after the cracking manifested itself on 29 May 2014. It was found that total station surveys utilizing fixed base stations were required to obtain the precision needed for meaningful comparisons of bank movements as opposed to using only GPS based methods.

Comparison of point measurements along each of the three survey transects are presented in Tables 4.2 to 4.4 for the upstream, center, and downstream profiles. These profiles corresponds to locations where geologic cross-sections were constructed from the boring and CPT data (i.e., sections B-B', C-C', and D-D', respectively). Values shown in the referenced tables correspond to the cumulative differences measured for each point in the x (easting), y (northing), and z (elevation) components for surveys made on 26 August and 8 October 2014. Negative (down) and positive (up) values indicate the direction of the cumulative movement that was measured for the two surveys. Values measured were in the hundredths to thousandths of a foot range as shown by Tables 4.2 to 4.4 (points A at crest and G and H are at riverbank). This measured range of movement was considered relatively insignificant in terms of the cumulative displacement that was observed by visual inspection. It was concluded, that the major period of surface deformation occurred before the monitoring network was established. The range of movements measurements indicated that longer periods between surveys were warranted.

Figures 4.2 to 4.4 show the vertical (elevation) changes that were measured from multiple surveys along the three transects. Similarly, the vertical displacement between the first (26 September) and last (8 October 2014) survey shows differences that were in the hundredths to

Figure 4.12. Location of survey profiles to monitor bank movements. Merged Bing and Google Earth background image is from 2014 prior to levee cracking.

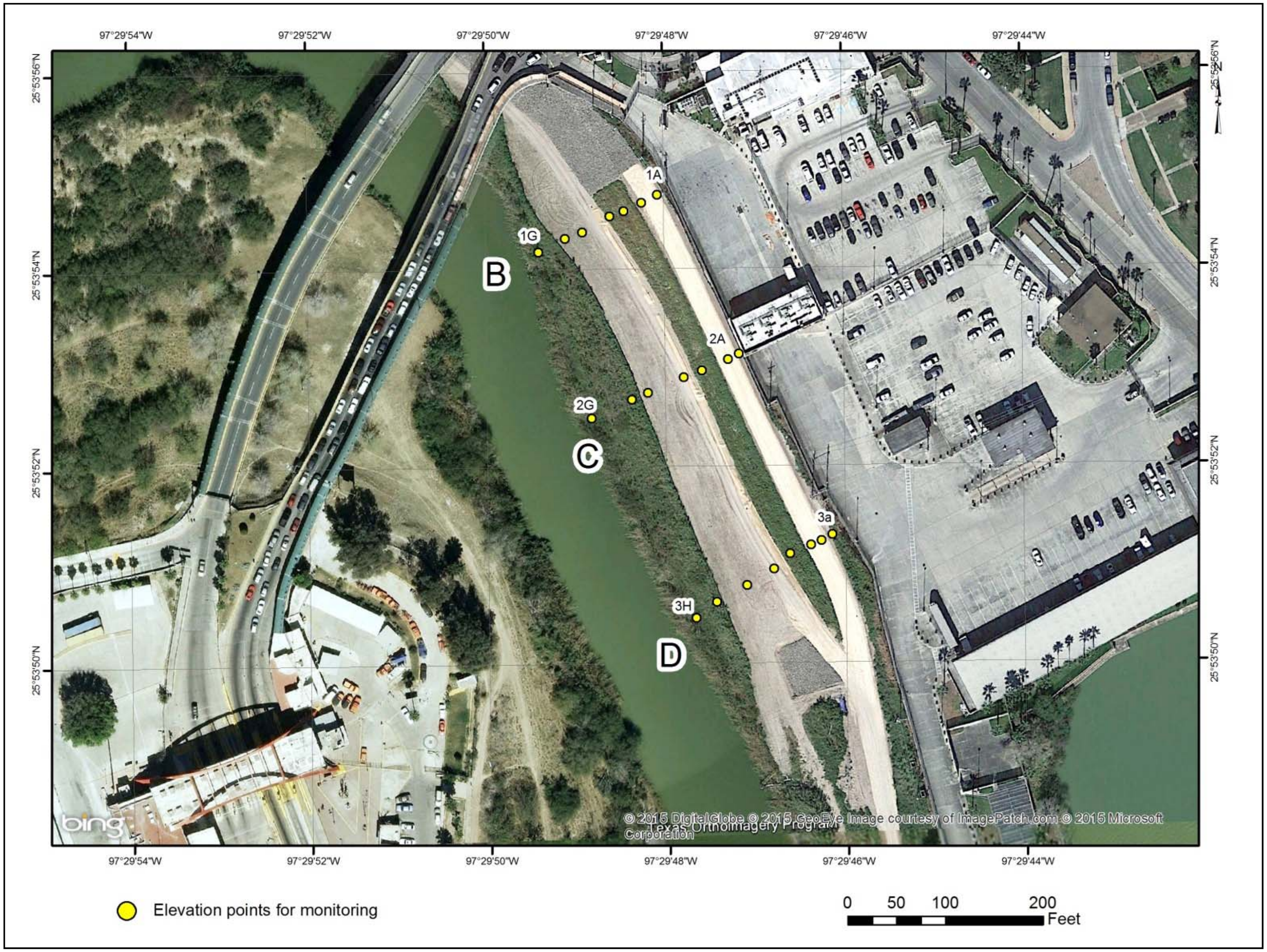


Figure 4.13. Total station survey profile for upstream levee section (location corresponds to geologic cross section B-B') showing levee and bank geometry, and the absence of appreciable movement between survey periods 26 Aug to 8 Oct 2014 (see Table 4.2).

Table 4.2. Net change in Northing, Easting, and Elevation between 26 Aug and 8 Oct 2014 for upstream profile (location roughly corresponds to geologic cross section B-B').

Station (TX-ID)	Distance	y- Northing	x- Easting	z- Elevation
	X axis ft	ft	ft	ft
1A	0.00	-0.15	-0.18	0.03
1B	18.38	-0.15	-0.03	-0.03
1C	39.05	0.04	-0.03	-0.03
1D	55.08	0.01	-0.11	-0.06
1E	87.56	-0.10	-0.05	-0.03
1F	106.36	-0.09	0.13	-0.02
1G	136.72	-0.04	0.00	-0.03

thousandths of a foot range. It was determined from the earlier surveys made that GPS based survey methods alone did not have the level of precision needed to quantify the range of movements observed, thus the reason for the switch to total station methods.

Figure 4.14. Total station survey profile for center levee section (location roughly corresponds to geologic cross section C-C') showing levee and bank geometry, and the absence of appreciable movements between survey periods 26 Aug to 8 Oct 2014 (see Table 4.3).

Table 4.3. Net change in Northing, Easting, and Elevation between 26 Aug and 8 Oct 2014 for center profile (location roughly corresponds to geologic cross section C-C').

Station (TX-ID)	Distance	y- Northing	x- Easting	z- Elevation
	X axis ft	ft	ft	ft
2A	0.00	0.02	0.05	0.00
2B	12.83	-0.02	-0.03	0.02
2C	42.68	-0.03	0.11	0.01
2D	62.34	-0.01	0.13	-0.02
2E	102.46	-0.02	0.08	0.00
2F	120.52	-0.03	0.06	0.00
2G	166.08	0.00	0.00	-0.04

Figure 4.15. Total station survey profile for downstream levee section (location roughly corresponds to geologic cross section D-D') showing levee and bank geometry, and the absence of appreciable movements between survey periods 26 Aug to 8 Oct 2014 (see Table 4.4).

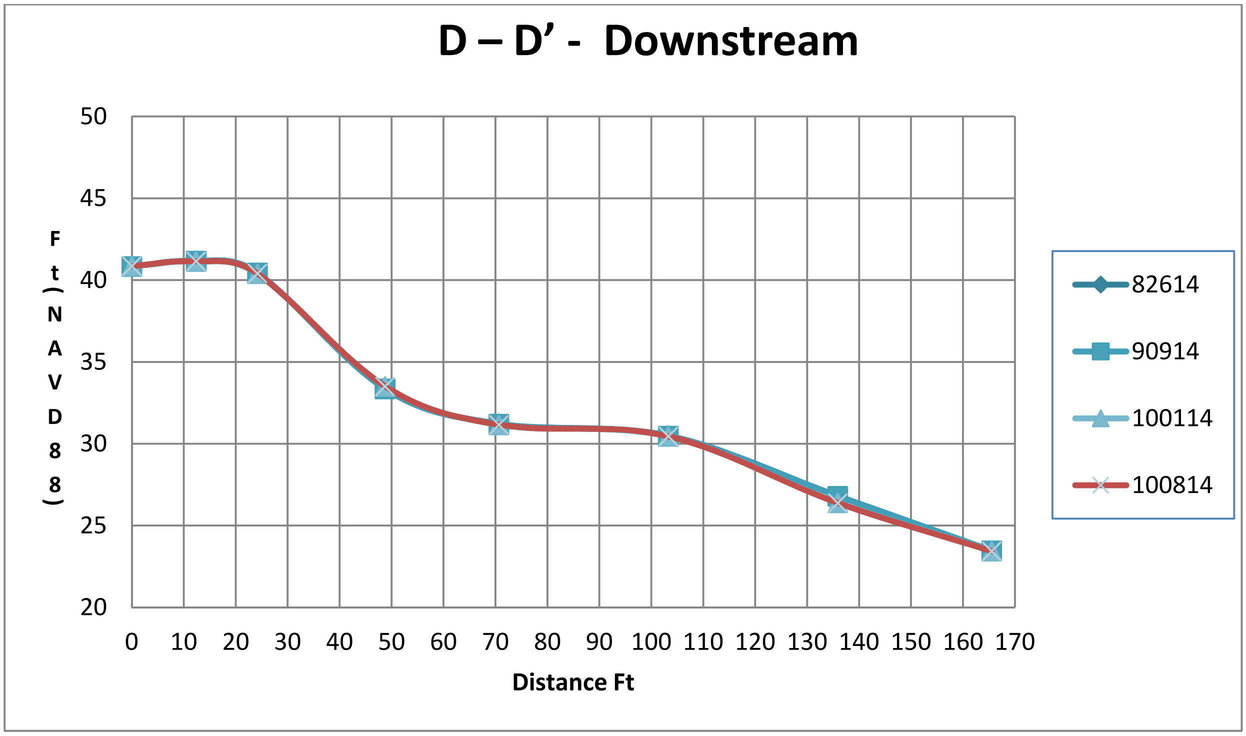


Table 4.4. Net change in Northing, Easting, and Elevation between 26 Aug and 8 Oct 2014 for downstream profile (location roughly corresponds to geologic cross.

Station (TX-ID)	Distance	y- Northing	x- Easting	z- Elevation
	X axis ft	ft	ft	ft
3A	0.00	0.03	-0.04	-0.01
3B	12.38	-0.01	-0.03	0.00
3C	24.19	-0.05	-0.04	0.01
3D	48.74	0.02	0.02	0.00
3E	70.67	-0.08	-0.06	-0.01
3F	103.28	-0.06	-0.01	-0.01
3G	135.90	-0.04	-0.01	-0.02
3F	165.52	0.05	-0.12	-0.01

4.3.9 Bathymetry survey

A bathymetric survey of the Rio Grande channel was made between 10 and 12 September 2014, by personnel from ERDC's Coastal Hydraulic

Laboratory (CHL). The purpose for the survey was to obtain elevation and topographic information of the channel bottom and submerged bank to determine the extent of scouring below the water surface and for accurate topographic information in the slope stability analysis.

Bathymetric and side scan sonar data were collected with a 25-ft Coast Guard Defender vessel with twin 225-hp outboard engines. Bathymetric data were collected using a GeoAcoustics GeoSwath Plus 250-kHz system, which simultaneously collects bathymetry and side scan sonar data. The horizontal datum for the project was in the North American Datum of 1983 (NAD83), State Plane Zone Texas Southern 4205 in U.S. survey feet. Similarly, the vertical datum was in the North American Vertical Datum (NAVD) 1988, also in U.S. survey feet. Motion and speed compensation of the vessel were corrected using data processing software to eliminate any errors associated with the boat motion.

Figure 4.16 presents the bathymetry data collected from the study area taken 10 to 12 September 2014. Elevation data from this survey identify a deep scour hole present in the bendway, extending downstream of the bridge, and with an elevation of less than 1 ft NGVD. The thalweg (deepest point in the river) then crosses toward the Mexico side of the river, where the channel bottom elevation begins to rise to between 2 and 4 ft NGVD. Further downstream the elevation is between 6 and 7 ft NGVD. Also noticeable in this figure is the hummocky topography of the U.S. channel bank, which displays a scallop outline and indicates a history of past bank slumping activity. Bank slumping was noted by ERDC personnel during field visits to the site in 2014 (Figure 4.17).

The rough nature of the U.S. bank is apparent in the side scan sonar images in Figures 4.18 and 4.19. The northern part of the study area shows a channel containing displaced bank material at the edge of the channel, as compared to the southern end where the submerged bank slope is generally devoid of any in-channel debris. The thalweg (deepest point in the channel) crosses toward the Mexico side of river downstream of the bridge.

A close-up view of the sonar image from the upstream half of the study area is shown in Figure 4.19. The submerged lower riverbank displays several areas likely containing active bank slides as evidenced by the scalloped nature of the upper bank and the presence of displaced bank

Figure 4.16. Bathymetry data showing elevation of the channel bottom. Bathymetry data were collected on 10 to 12 Sep 2014. Note the jagged U.S. bank line and scallop topography below the water surface.

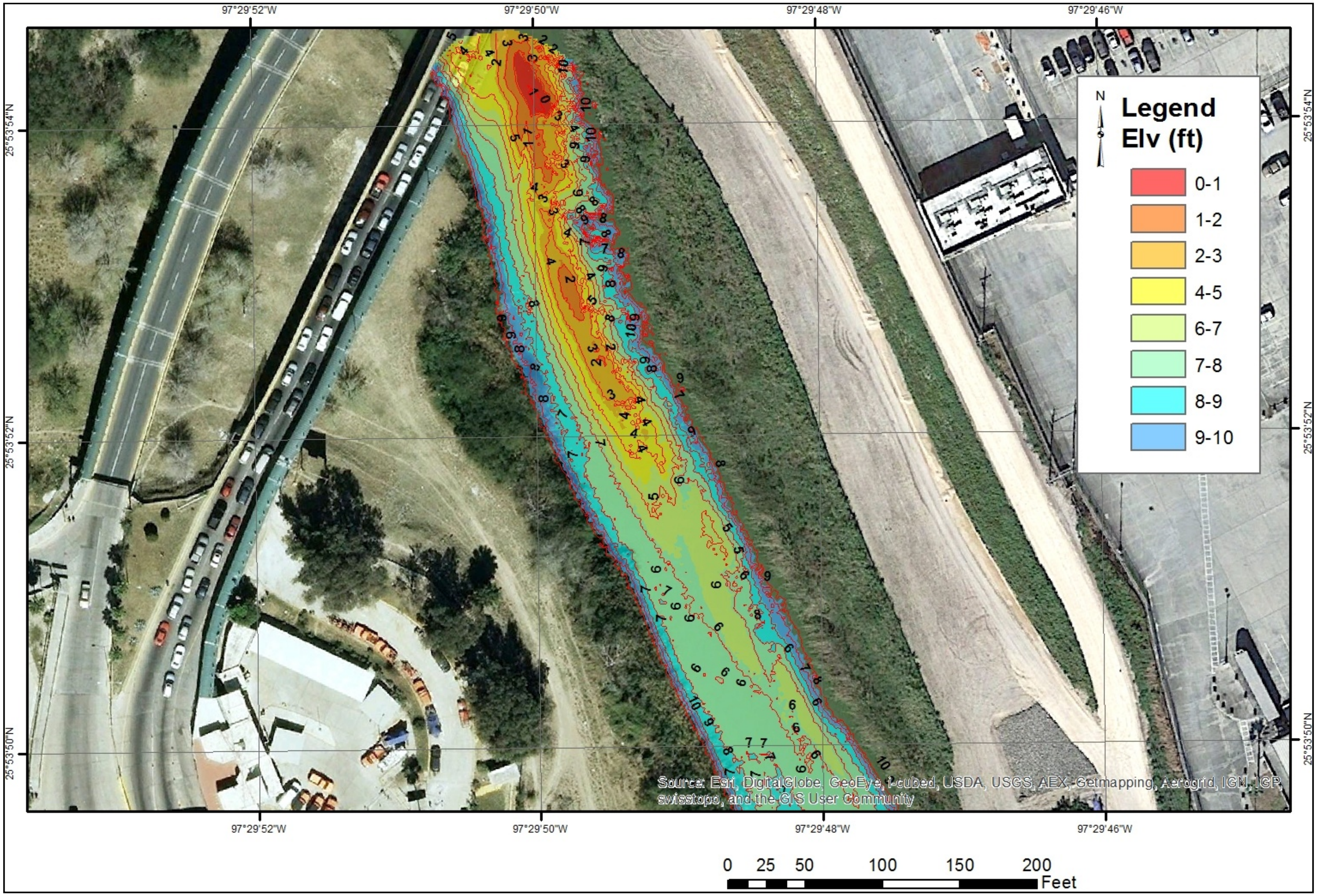
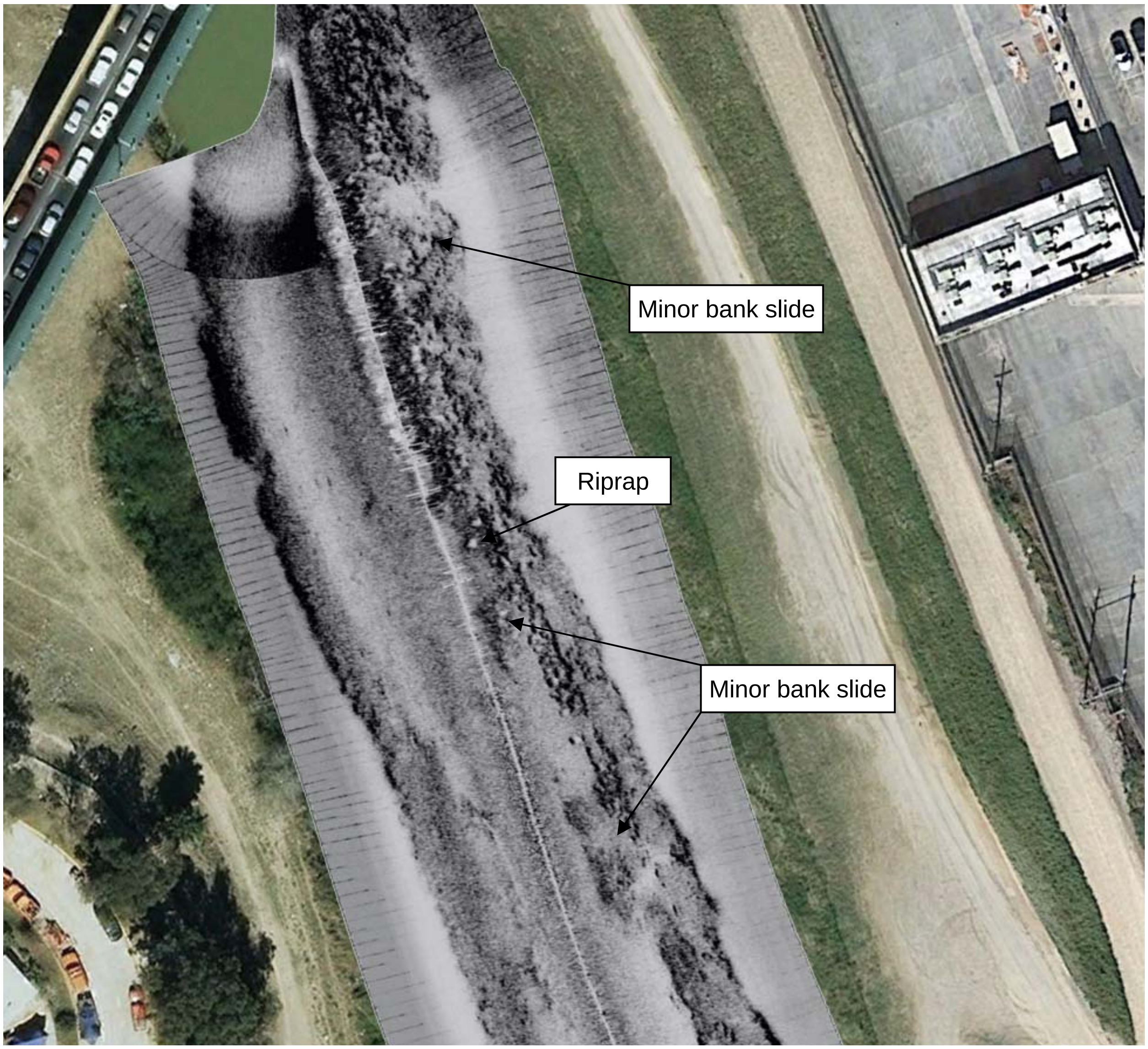


Figure 4.17. Active bank slumping occurring along riverbank adjacent to person in photograph. Photograph is in the middle of the study reach. Photograph was taken in Jul 2014 after the brush was cleared from the bank to permit close inspection.

Figure 4.18. Side scan sonar image overlain on 2014 Google image showing the channel of the Rio Grande through the study area. Close-up view presented in Figure 4.19 with prominent features noted.

Figure 4.19. Close-up view of the side scan sonar data overlain on 2014 Google image. Note the presence of rip-rap and small bank slumps/ slides on U.S side. White line corresponds to center of boat track moving toward the bridge.



resting on the bottom edge of channel, which is at a higher elevation. Also noteworthy is the presence of stone riprap along the channel edge, which is either due to rock moving from the upstream bend way or is present in the lower bank because of past armoring that occurred. As previously noted, limestone riprap was encountered in borings close to river's edge, approximately 10 to 12 ft below the ground surface. The presence of stone riprap is clearly visible in the photograph in Figure 4.20, which was taken during the low water on 12 April 2014. The sudden drop in water level which exposed the bank and channel bottom in this photograph is thought to be the likely initial trigger for the slope failure that resulted in extensive levee cracking. This photograph is significant as it shows conditions prior to the onset of the slope failure. The vegetated bank appears to be nearly vertical in this photograph, while the riprapped channel bottom is relatively horizontal looking downstream. The location of the photograph corresponds to a view looking downstream from the vicinity of the bridge.

4.4 Terrestrial LiDAR survey

A terrestrial LiDAR survey was performed by CHL personnel during the same time as the bathymetry data were being collected. A Reigl VZ400 Laser Scanner was used for terrestrial data collection. A Trimble R8 receiver on a 2.1-m tripod positioned on a known survey control point collected raw GPS data during the data collection period. This receiver also generated the Real Time Kinematic (RTK) corrections for real-time use. Five individual reflective locations were needed to collect the elevation site data.

The raw GPS data file was used in post-processing with Trimble Business Center software to achieve centimeter level horizontal and vertical accuracy. These selective target locations were used to correlate the scan data positions and produce a geo-referenced point cloud dataset using Reigl's RiSCAN PRO software. This dataset was filtered to remove the woody vegetation and then integrated with the bathymetric data.

Two images of the LiDAR data are presented in Figures 4.21 and 4.22. The first view is from the levee crest, looking upstream from the south end of the study area. The image captures 0.5 to 0.7 ft of displacement along the scarp at the roadway crest. The second view is looking downstream from the levee toe, approximately midway in the study area and shows the wide crack at the levee toe, which cuts across the gravel ridge and parallels the levee toe upstream from the viewer's perspective (note the upstream extent of the blue crack in Figure 1.1).

In summary, the LiDAR data provides an elevation baseline should future surveys be warranted and these data permit accurate measurement of the change in elevation across the levee slope. Unfortunately, the extent of the brush growth along the bank at the time of the survey prevents detailed resolution of the ground surface at the riverbank and a critical examination of the slumping adjacent to the bank. The cracking at the levee crest may extend beneath the riprap that was used to armor the upper bank

Figure 4.20. View of the exposed riverbank and channel bottom during the low water event on 12 Apr 2014 (photograph courtesy of Ramon Navarro, Engineering Services Division, USIBWC).

Figure 4.21. LiDAR image looking upstream and showing 0.5 to 0.7 ft of down slope displacement of the crest road.

Figure 4.22. LiDAR image looking downstream showing the large crack separation at the toe, the crack crossing the gravel mound, and continuing upstream toward the viewer. White circular features in the image are the LiDAR stations where the instrument was placed to conduct the scan of the bank.

downstream of the bridge. Coarse stone used to armor the slope prevents the Terrestrial LiDAR from measuring minor elevation changes that possibly reflect the continuation of the crack (Figure 4.23).

Figure 4.23. LiDAR image of the crack at the levee crest. Rip-rap at the upstream end of study area prevents examination of the ground surface to verify the upstream crack extent. LiDAR data does not provide additional resolution because of the coarse nature of the stone.

5 Geology

5.1 Geologic setting

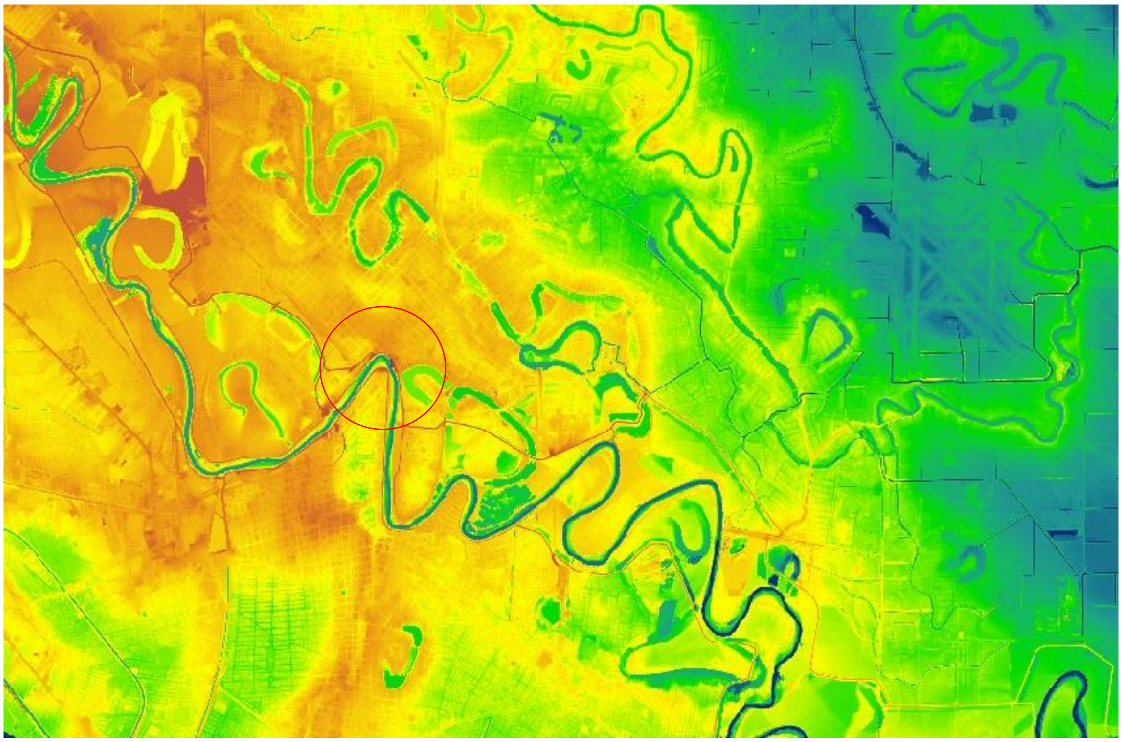
Alluvial sediments in the study area involve historical (since 1846) and Holocene (<10,000 years) age deposits. These sediments were formed by the migration of the Rio Grande in the LRGV during the Holocene (Figure 5.1) and are related to Rio Grande course changes that are present in the Brownsville area as shown in Figure 5.1. These different Rio Grande courses also correspond to different Rio Grande delta systems that were active during the past 10,000 years (Figure 5.2).

Underlying the historic and Holocene age alluvial deposits in the study area are Pleistocene sediments that were exposed to intense weathering during the last glacial maximum and corresponding low sea level stand, which ended approximately 12,000 to 15,000 years ago. Periods of maximum world-wide sea level drop during the Pleistocene correspond to periods of ice sheet build-up with continental glaciers extending across the North America continent. The corresponding drop in sea level would have exposed the existing Pleistocene drainage network and led to a period of prolonged weathering and deep seated oxidation of this surface. Sea level is estimated to have dropped by 350 ft world-wide, and caused widespread erosion of the drainage network, valley down-cutting, and widening in the LRGV. The shoreline now would have been near the edge of the continental shelf.

A long-term break in deposition (e.g., long term exposure and weathering of the Pleistocene surface) would imprint a distinct signature that is much different than the younger sediments that overlie this surface. Diagnostic characteristics that support these geologic processes involve marked differences in soil color, stiffness, shear strength, texture, and other physical signs. A break in deposition in the geologic record is known as an unconformity and is marked by characteristic soil profiles developed upon the exposed surface. This Pleistocene surface has subsequently been buried by the deposition of younger Holocene and historic alluvial sediment within the study area from the Rio Grande courses shown in Figure 5.1.

The Pleistocene history of the LRGV is complex (Brown et al. 1980; Bureau of Economic Geology 1976; Leblanc 1958; Lohse; 1958). The

Figure 5.1. Holocene Rio Grande courses shown on 2011 LiDAR and Bing image of the Brownsville area, TX. Study area is within red circle (higher elevation corresponds to red tones).



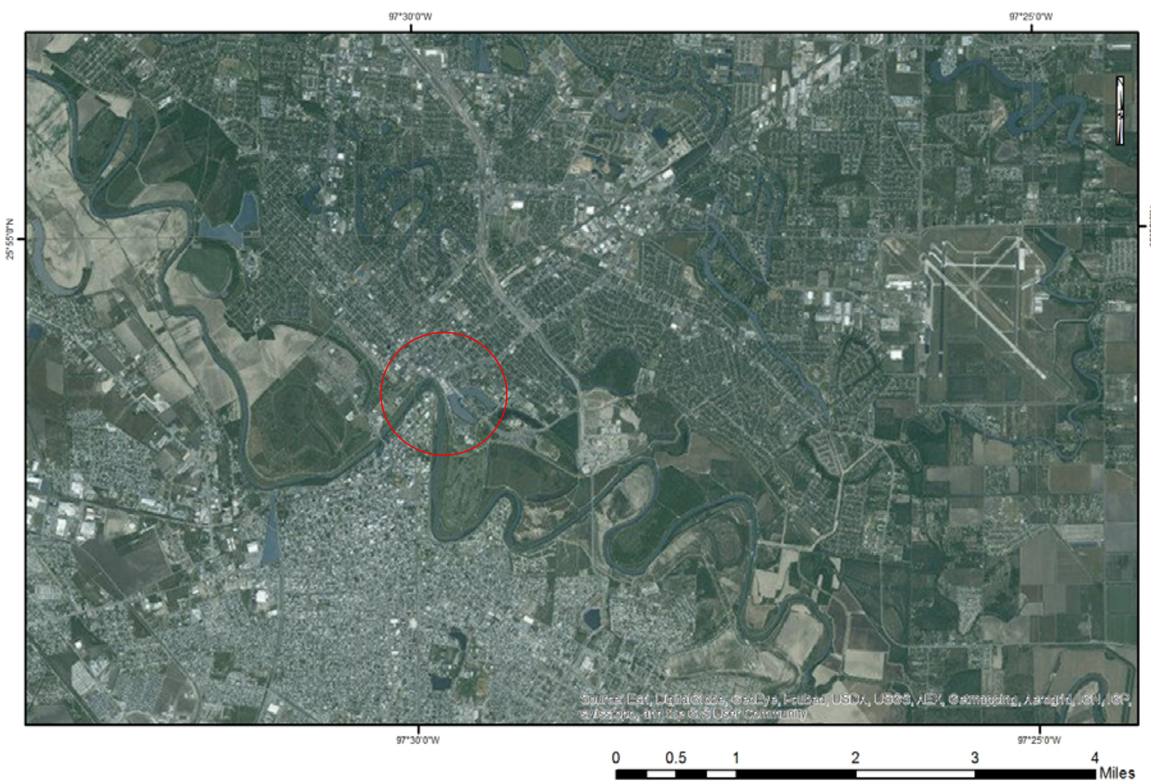
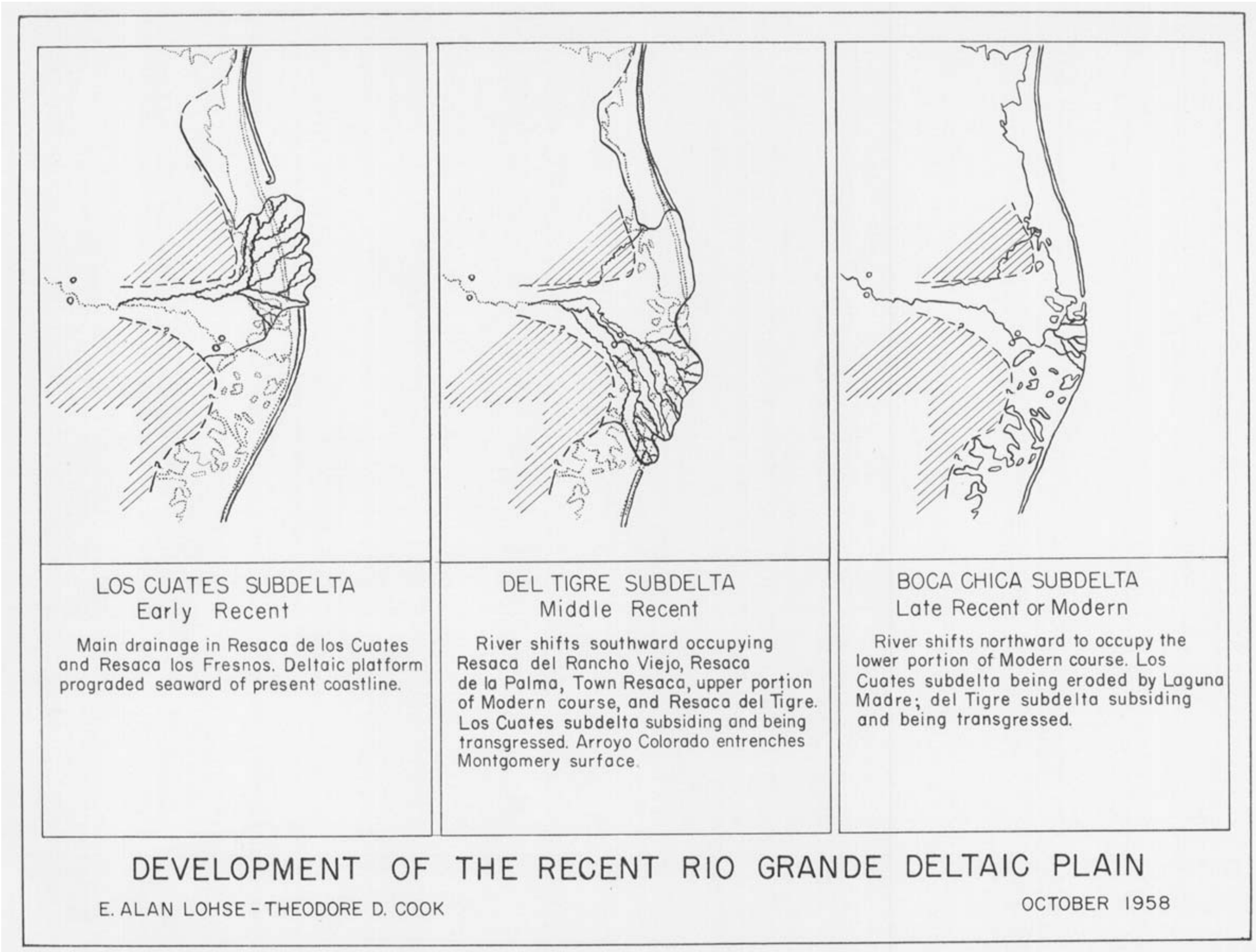
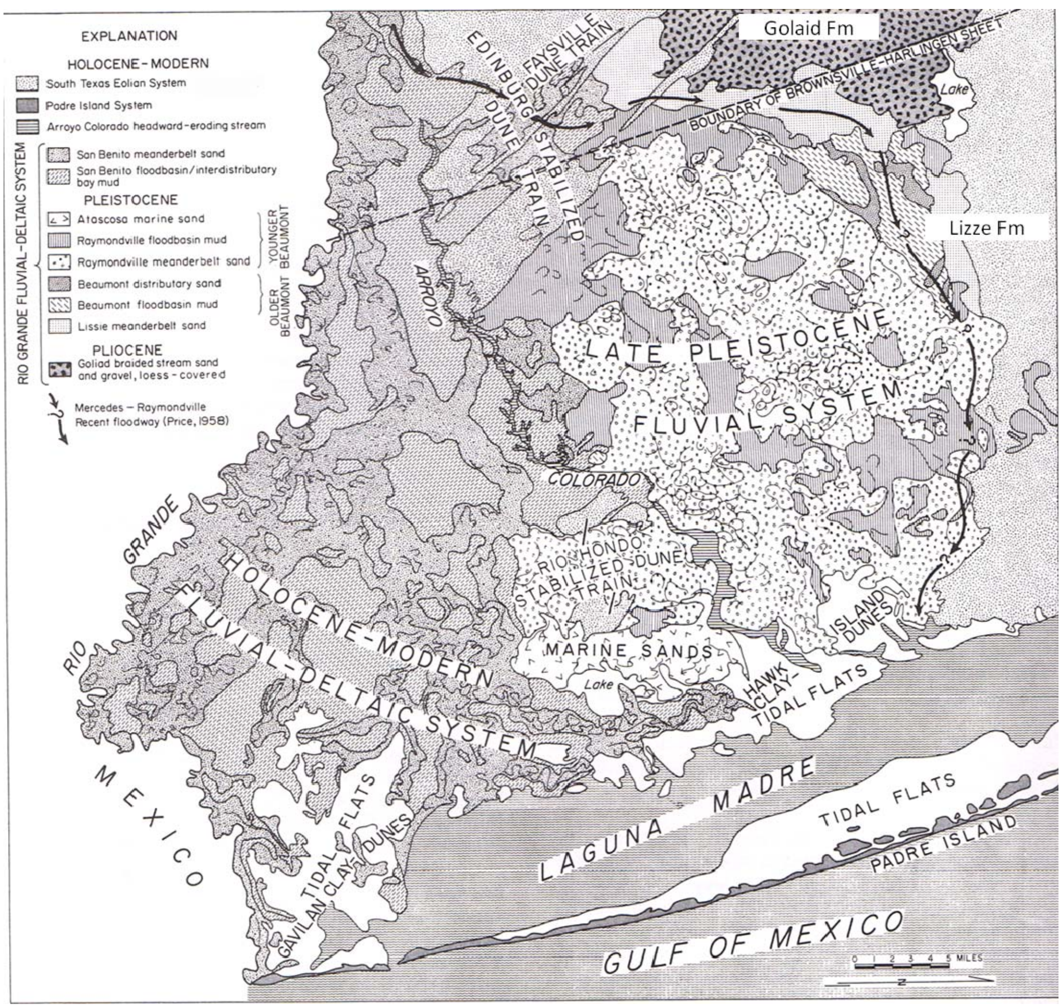


Figure 5.2. Major delta systems in the LRGV during the Holocene rise in sea level, which began 12,000 years ago and reached the present stand 3,000 to 5,000 years ago (Lohse 1958).



history involves the horizontal and vertical movement of the Rio Grande channel in response to base level or sea level changes caused by glacial events in the northern latitudes (Figure 5.3). The Rio Grande moved repeatedly across its alluvial valley during the Pleistocene, as shown from Figures 5.1 to 5.3, and left a geologic record of past floodplain surfaces containing associated fluvial and windblown deposits in its wake. Much of the floodplain in the study area has received extensive sediment within the past 75 years as evidenced by the historic map data that were compiled as part of this study. Additionally, Lake Brown is another tangible example of active horizontal migration that has occurred in the study area within a relatively short time span.

Figure 5.3. Regional geologic map (scale 1:630,000) of Rio Grande fluvial-deltaic system in the LRGV and the subdivision of the Pleistocene Beaumont Formation into a younger (Eunice) and older (Oberlin) deltaic system (Brown et al. 1980). Map area extends from east of 98° W. Longitude. Floodway identified by arrows corresponds to the location of the USIBWC floodway.



5.2 Geologic cross sections

Boring and CPT data were compiled into four cross sections to show the horizontal and vertical limits of the stratigraphy within the study area. The locations of the cross section are shown in Figure 5.4. Included on the cross sections are the top and bottom depths of the well screens on the respective cross-sections. A longitudinal section (section A-A') extends from upstream of the bridge starting with the Raba-Kistner boring DP-201 to downstream of the study area at CPT P2-24C (see Figures 5.4 and 5.5). This section shows the different stratigraphic units present from upstream of the bridge to downstream of the failure reach. The primary changes through this section are the depth of the Pleistocene surface.

The Pleistocene surface is defined by the green-dashed line and is relatively shallow at boring DP-201 at elevation 26 ft. The contact deepens in the bend way of the river at the bridge pier borings (B1-1984, B2-1984, and CV-4A) where it ranges between 0- and -10-ft elevation. Through the failure reach, this surface varies between -10- and -20-ft elevation. Near boring P3-36B, the surface rises again to about 15-ft elevation. Downstream of the study area at CPT P2-24C the surface drops to the 0-ft elevation, which corresponds to maximum depth of fluvial scouring observed at the bridge by the bathymetry data.

The deepening of the Pleistocene surface in the bend way is due to scouring by the Rio Grande in this reach as evidenced by the historic map and photographic data presented in Chapter 2. Sediments that overlie this surface are historic, fine-grained alluvial fill. These sediments are primarily dark gray in color, very soft, and have low blow counts. Above the water table, these sediments generally become stiffer and contain more sand.

Historical sediments that form the riverbank area are adjacent to the Rio Grande. The upper bank near the river is sandy in composition. Zones where the stratigraphy was sandy are identified by the orange dashed line in the section. Likewise, areas where uniform clay is present are shown by the blue dashed line.

A break in the longitudinal section is shown between 1,400 and 1,700 ft along the x-axis. The section break presents the Lake Brown water surface elevation for comparison purposes. The lake elevation along with the 1912 measured channel depth (see Figure 3.3) is identified by the arrow length

Figure 5.4. Location of geologic cross sections.

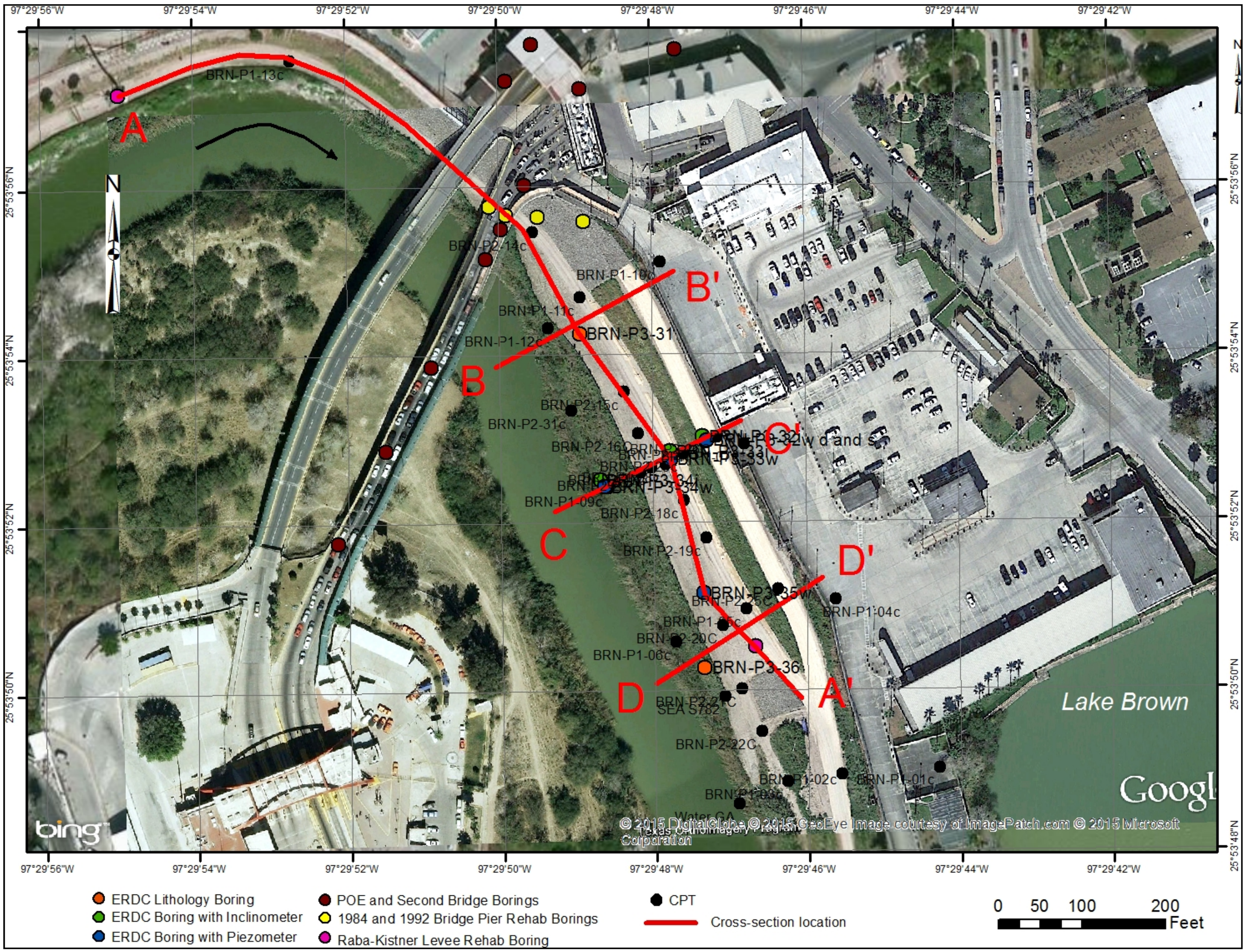


Figure 5.5. Longitudinal geological cross section. This section extends from upstream of the bridge starting with the Raba-Kistner boring DP-201 to downstream of the study area at CPT P2-24C (see Figure 5.4).

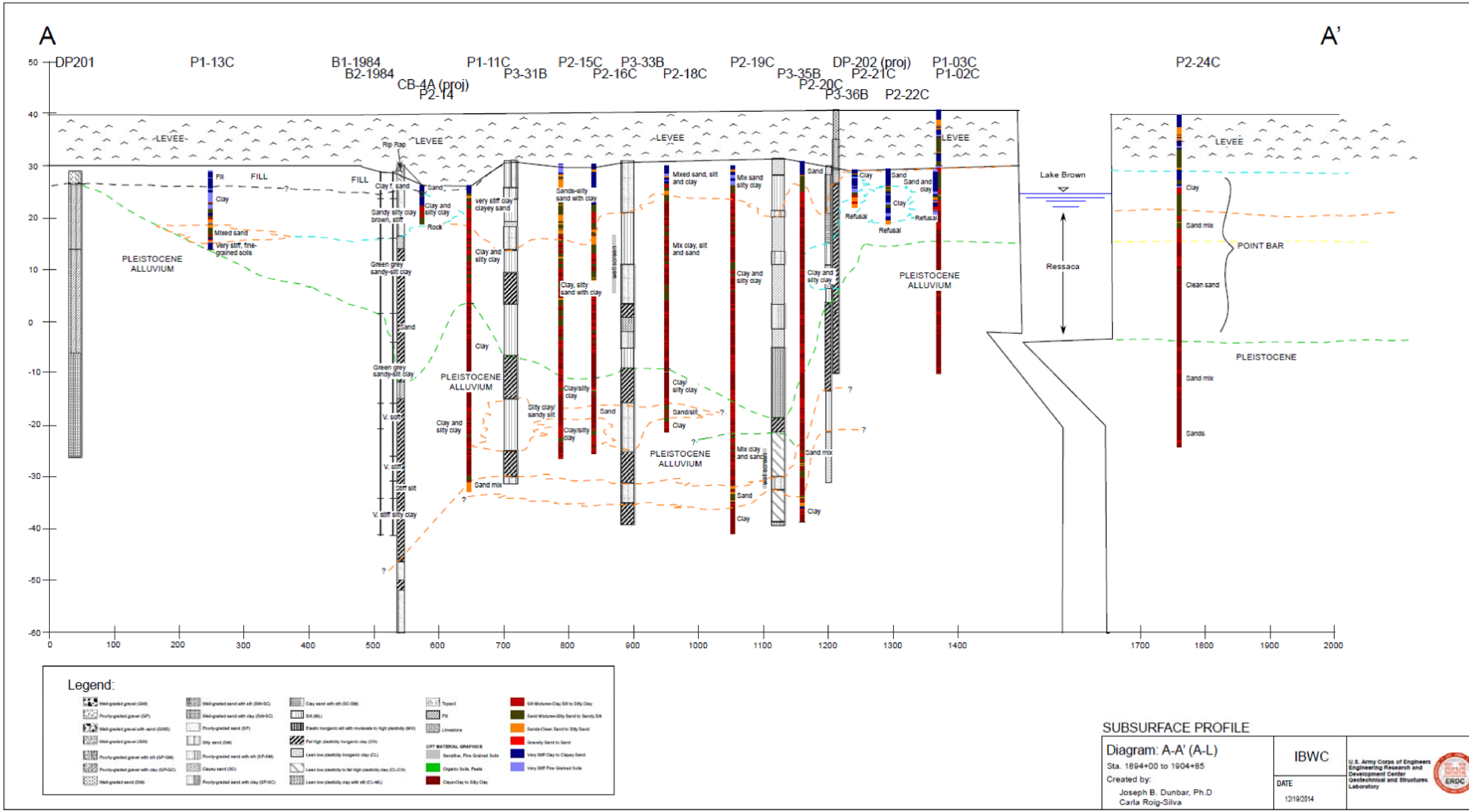


Figure 5.6. Geological cross-section B-B'.

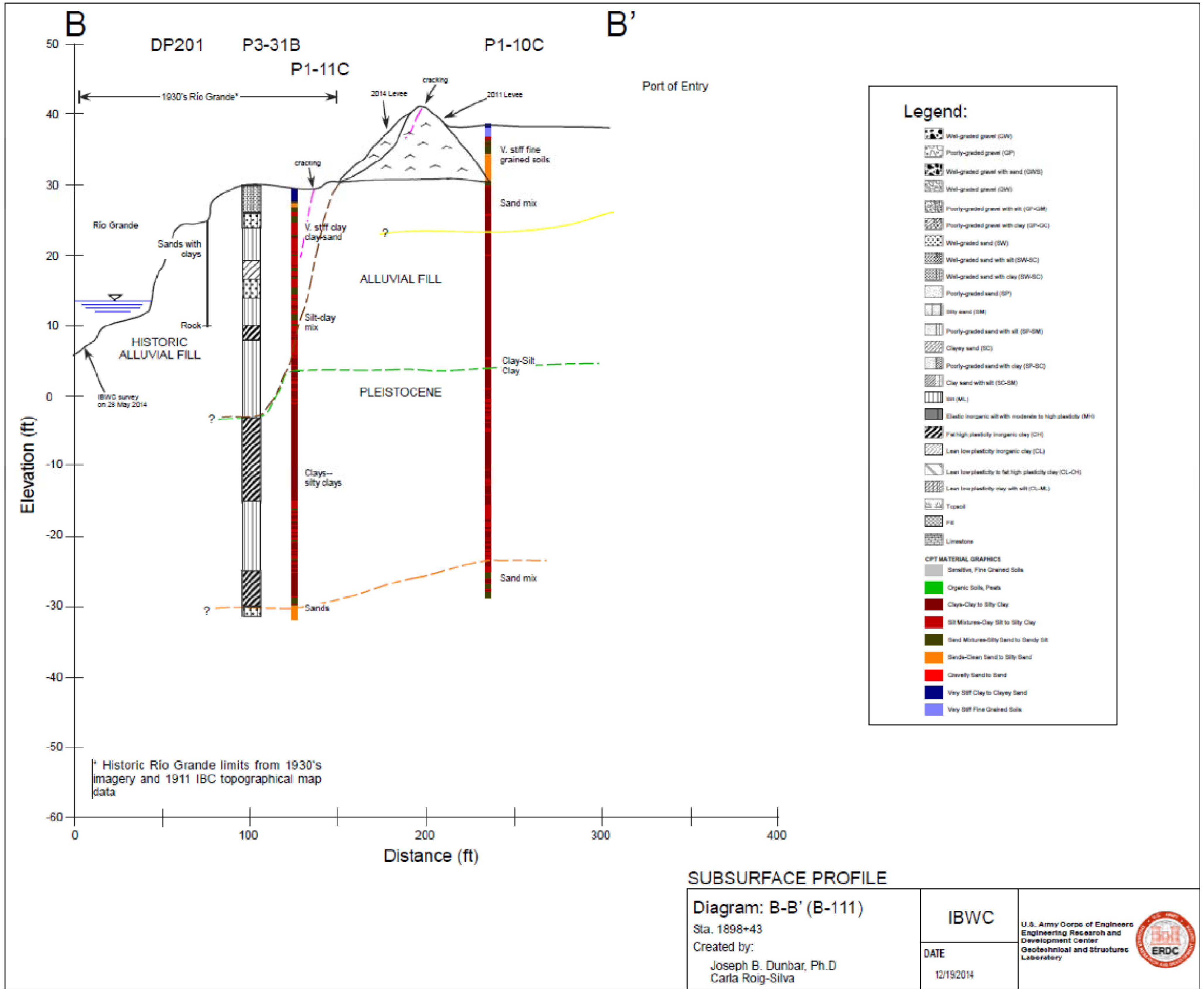


Figure 5.7. Geological cross-section C-C'.

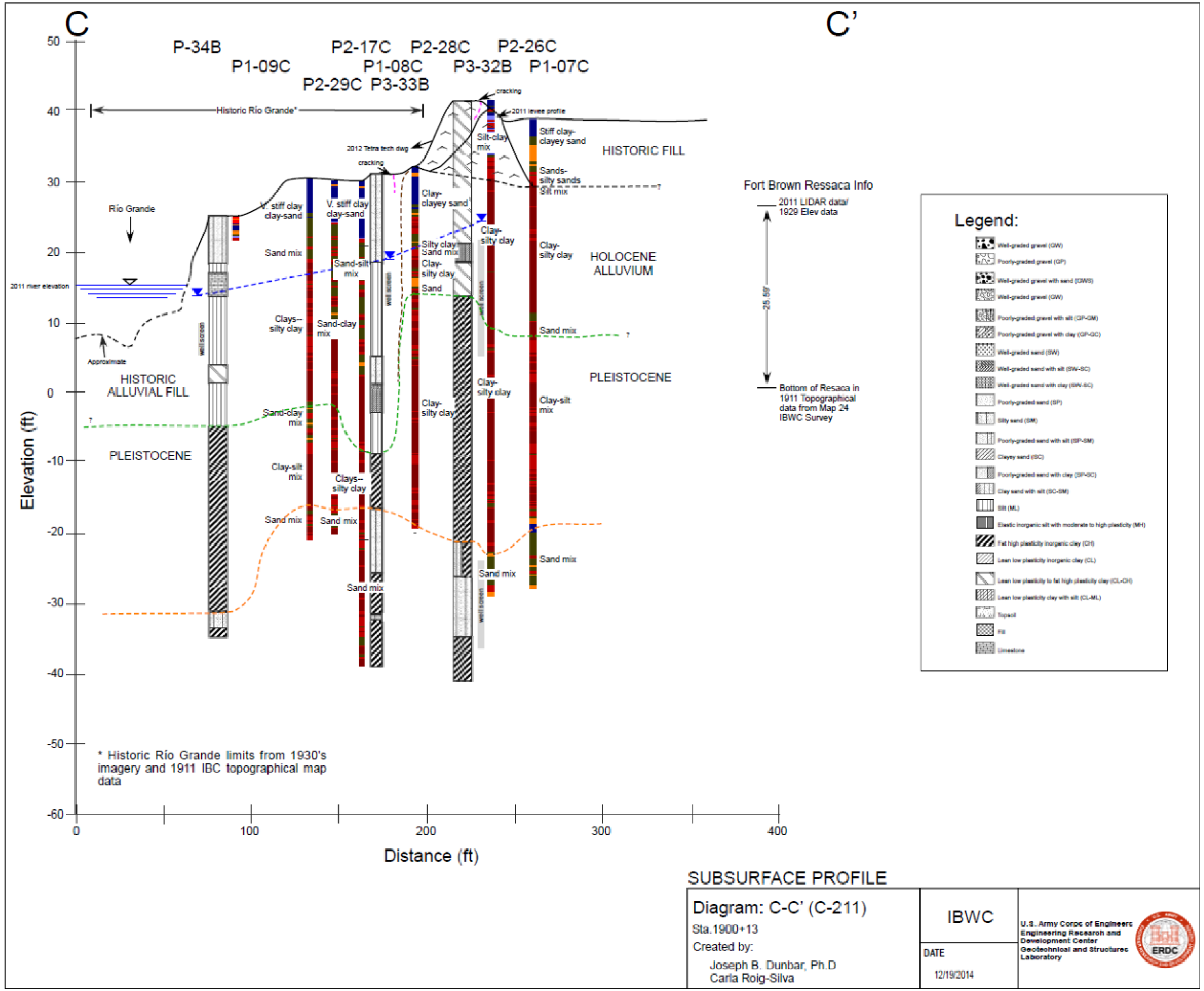
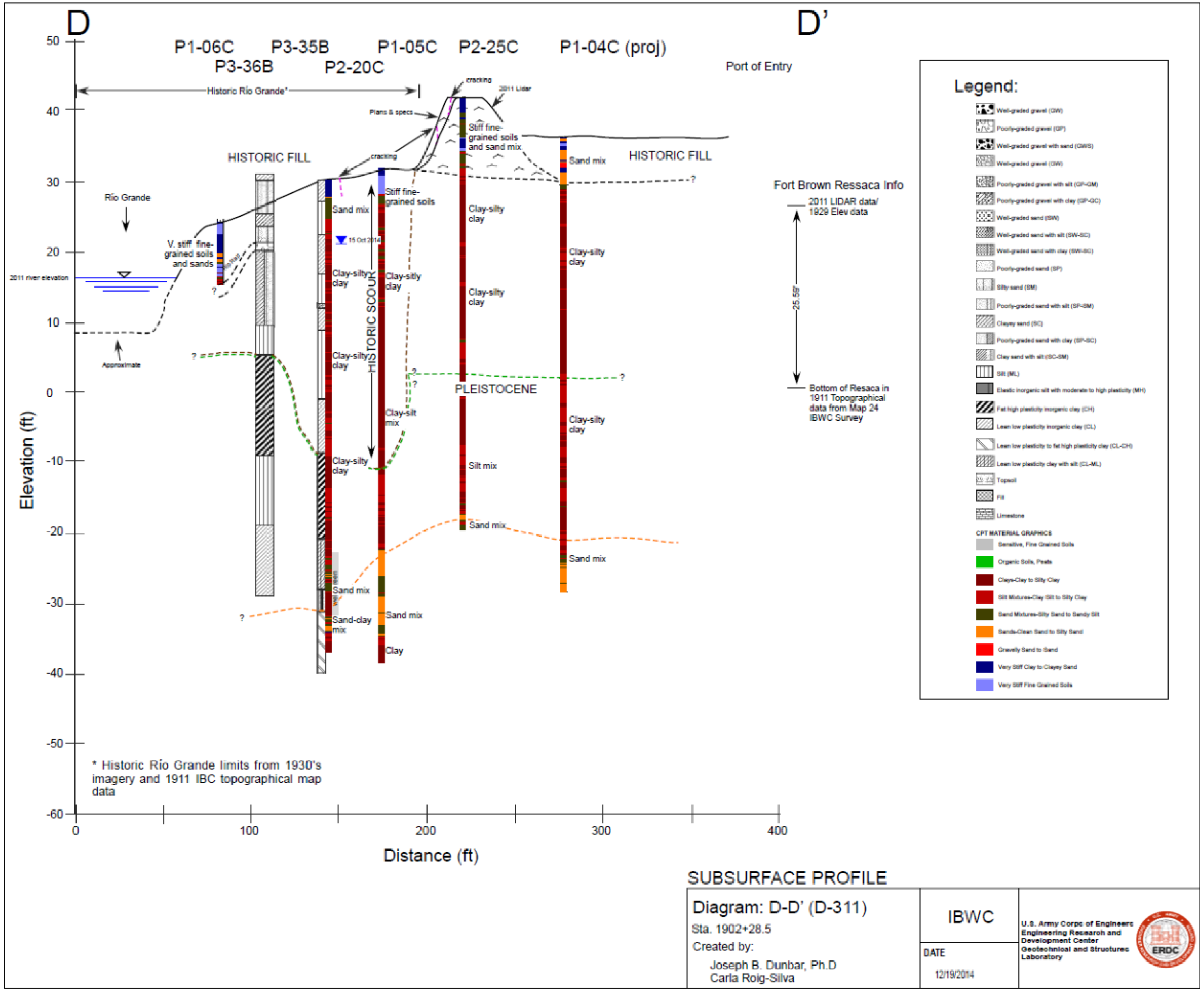


Figure 5.8. Geological cross-section D-D'.



for illustration purposes. To the right of the section break is CPT P2-24C, which identifies a classic point bar stratigraphy of a fining-upwards sequence related to river migration and eventual neck cutoff of Lake Brown from the main Rio Grande channel before 1846. Point bar stratigraphy at this location contains basal coarse sand, overlain by fine silty sand, and a fine grained top stratum or blanket composed of clay. The Pleistocene surface at this location is about at elevation 0 ft. The point bar sands at this location are nearly 20 ft thick. Point bar deposits correspond to Case I stratigraphy evaluated by Raba-Kistner for the geotechnical evaluation of the UBLRP.

Point bar deposits are especially pervious and noted for their seepage potential during flood stage. During river flooding, horizontal flow through the pervious sands can extend great distances landward in the shallow aquifer because of the steep hydraulic gradients produced by the river. Conversely, a rapid drawdown of the river, combined with a stable lake level that is significantly higher than the river, and a pervious substratum permits elevated pore pressure conditions to be generated locally in the shallow aquifer by the sudden drawdown condition.

Three cross sections perpendicular to the longitudinal levee cross sections were developed from the CPT and boring data as shown by Figure 5.4. These sections are identified as B-B' (Figure 5.6), C-C' (Figure 5.7), and D-D' (Figure 5.8) with sections beginning at the upstream edge of the study area and progressing downstream. Surface elevations shown on the sections were developed using post-rehabilitation topographic information contained on design sheet 70, Plan and Profile, Station 1894+00 to 1904+00 (Tetra Tech 2013) and 2011 LiDAR data obtained from the USIBWC. The 2011 LiDAR data were used as part of the design of the levee rehabilitation project.

The profile sections show changes to the levee geometry post-2011 from the 2013 planned rehabilitation work. Included on the levee cross sections are the locations of the surface cracks that were mapped by ERDC in July 2014. Also shown are the well screen horizons, the piezometric surface, and the nearby Lake Brown water level. Interestingly, the 1929 water level in the lake does not vary significantly from the 2011 LiDAR elevation, or the 2014 data compiled from the instrumentation data monitoring the lake level. The Pleistocene surface is identified in the three sections by the dashed green line based on the CPT tip resistance and lithologic data in borings obtained during the field investigation phase of this study. The limits of the river scouring into Pleistocene surface are identified at each section location along with the abrupt vertical boundary between the historic channel fill and the Holocene alluvium that formed the riverbank before the construction of main-stem dams upstream resulted in the channel filling with sediment. The POE land area behind the levee in each profile generally contains sand fill beneath the concrete roadway.

A deep sand layer is present beneath the Pleistocene surface that is interpreted to be a buried point bar alluvial sequence (i.e., fine-grained top stratum or blanket and pervious sandy substratum). This sand layer is marked by the orange-dashed line. In terms of the regional geology described above, this sequence is probably correlative to floodplain deposits associated with the Late Pleistocene Younger Beaumont formation (Figure 5.3) when sea level was at a lower elevation (Brown et al. 1980). What is significant about the lower sand zone is the hydraulic response measured by piezometers that were screened in this sand as compared to the shallow piezometers (see Figure 4.8). Borings P3-32 (section C-C', Figure 5.7) and P3-35 (section D-D', Figure 5.8) contain well screens that were tipped in the lower sand unit. Their response through time and water level elevation changes would indicate a hydraulic connection with the shallow stratigraphy. Both of the deep wells have water levels near the level of the shallow wells.

Historic alluvial sediments are primarily fine-grained, gray to dark gray in color, soft to very soft, and contain organic materials (wood, roots, charcoal), and/or historic debris, such as glass and buried riprap. Historic sediments become sandy near the surface, and are finer-grained with depth. Wood is often present below the water table. In contrast, the Pleistocene sediments are clay-rich, more uniform, brown to tan in color, stiff to very stiff, mottled, and contain small carbonate concretions. The clay is usually dry unless sand lenses are present. Where sand lenses exist, the clay can be soft where it is wet as revealed in borings that were drilled into this top stratum unit.

5.3 Groundwater

With only the four piezometers installed during this study, it is only possible to infer basic observations about groundwater conditions in the study area as compared to a detailed piezometric map that shows groundwater flow from numerous wells. Groundwater flow is generally toward the river and to the Gulf of Mexico from basic understanding of groundwater hydrology in alluvial aquifer settings and the measured water level data recorded during this study.

As shown by Figure 5.1, abandoned oxbows are present throughout the greater Brownsville area. Lake levels have been relatively stable during the past as identified by historic topographic map data (Figure 3.4). The abandoned oxbows and former Rio Grande courses (multiple interconnected channels that contain several oxbows which together form a meander belt and constitute a former river course) presently serve as sinks for urban surface drainage and locally feed the shallow alluvial aquifer. Thus, aquifer flow is locally to the river and regionally toward the coast.

Lake Brown maintains a relatively stable lake level due to surface drainage into to the lake and pumping from the Rio Grande to the lake by the City of Brownsville for the Southmost Campus. This lake is hydraulically connected to the river as evidenced by local sand layers in the Holocene alluvium, the point bar stratigraphy at the southern edge of the study reach (Figure 4.5), and the measured response of the monitoring wells to changes in the river level (Figure 4.8). The response is especially noticeable, as would be expected, in well BRN-P3-34W, which is nearest the river channel (Figure 4.6), and less so in the wells at the levee crest (BRN-P3-32) and at the levee toe (BRN-P3-33W). These wells show a flatter and delayed response to water level changes in the river and in the lake due to precipitation.

The water level elevation and the monitored response in BRN-P3-35W are interesting, as this well is screened in the Pleistocene (see Figure 4.8) and the water level is shallow and deep, which is comparable to the levels in BRN-P3-32. Thus, the lower and upper stratigraphy in the study area are likely tied together because of past river migration and channel scouring into Pleistocene deposits, which in turn has caused juxtaposition of different stratigraphic units with pervious point bar deposits. The higher elevation and flatter response of these wells is likely related to the nearly constant water level in Lake Brown. In summary, the groundwater surface is locally towards the river and any change in river stage can cause fluctuations in the local gradient.

5.4 Rio Grande gage data

The water level in the channel as indicated by river gage data can have a significant effect on the local hydraulic gradient. The sudden drawdown that occurred in early April 2014 is considered to have a direct impact on the levee cracking and the partial bank slope failure. Brownsville gage data for the first six months of 2014 are presented in Table 5.1. Low flow periods of less than 1 cms are highlighted in yellow and occurred in 2014 on 10 to 12 April, 4 to 6 May, 7 to 14 June, and 4 to 7 July. Discharge measurements in Table 5.1 require adjustments for elevation at the study area.

The Brownsville gage is located 7.2 miles downstream of the Gateway International Bridge (Figure 5.9). To place the water elevation at the gage station into its proper context at the levee study area requires an adjustment in terms of the elevation between the two points. This adjustment is directly related to the longitudinal profile of the water surface elevation of the river between the bridge and the downstream gage location and the stage discharge relationship for the gage. A plot of the river stage versus discharge, for the period 27 December 2014 to 30 January 2015, is presented in Figure 5.10. The zero for the Brownsville gage is at the 0 ft elevation (personal communication with Glen Smith, Water Accounting Division, USIBWC). Thus, the corresponding discharge readings (cms) in Table 5.1 can be correlated to river stage (meters) at the Brownsville location. Thus, a discharge of 11 cms in Figure 5.10 corresponds to a water surface elevation of 1 m. However, the water level that is important to this study is the corresponding water surface elevation at the Gateway International Bridge.

To adjust for the drop in water surface elevation along the 7.2-mile stretch between the bridge and the gage, or the higher elevation at the study area, requires a corresponding location and elevation adjustment be made for this 7.2-mile difference in distance. The basis for this adjustment in elevation is derived from Figure 5.11 showing the 2011 LiDAR data and the change in elevation between the Gateway International Bridge and the Brownsville gage. A grade control structure is present at 213 m (700 ft) above the gage (see Figure 5.12 and 5.13). For consistency of English units used throughout this report, the vertical change in elevation between the bridge and gage is 7.8 ft. Therefore, to estimate the water surface elevation at the levee reach an adjustment of 7 ft was used to model the water levels identified in Table 5.1 during the period of this study. A 7.0 ft correction

Table 5.1. Brownsville gage data for first six months of 2014.
Gage is located at 25° 52’ 32.40” North Latitude,
97° 27’ 16.86 West Longitude.

International Boundary and Water Commission
U.S. Section

08475000 Rio Grande Near Brownsville, Texas
And Matamoros, Tamaulipas

2014

Mean Daily Discharge in Cubic Meters per Second

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	15.9	4.30	7.05	9.44	1.79	4.49	5.49					
2	15.8	6.30	6.13	6.54	1.42	3.65	3.10					
3	14.6	5.91	5.23	2.59	1.00	3.16	1.32					
4	12.8	5.67	5.04	1.88	.40	4.43	.83					
5	11.2	5.76	6.51	2.02	.43	2.44	.79					
6	9.51	5.21	5.66	1.97	.72	1.35	.81					
7	7.64	4.19	4.72	2.31	1.13	.90	1.16					
8	6.88	5.58	4.83	1.36	1.24	.76	1.39					
9	5.93	7.91	5.69	1.03	3.94	.60	.75					
10	5.16	7.62	4.53	.77	6.82	.64	.66					
11	5.07	5.79	2.92	.92	10.2	.49	.72					
12	4.09	4.58	3.86	.77	16.3	.54	1.61					
13	3.73	6.94	3.04	1.05	25.5	.66	5.05					
14	4.35	9.47	2.61	1.37	28.2	.85						
15	5.74	10.5	5.88	3.53	30.3	1.25						
16	5.65	9.96	9.02	2.26	26.0	4.23						
17	4.49	9.18	7.85	1.97	20.8	4.15						
18	5.13	6.03	6.21	2.60	18.0	2.49						
19	6.01	3.35	7.69	2.59	18.7	1.79						
20	5.80	4.51	8.06	2.75	20.1	1.34						
21	5.37	3.82	6.12	6.84	17.7	1.28						
22	6.39	2.59	5.64	6.25	9.59	1.22						
23	5.10	3.61	4.17	3.37	5.42	1.47						
24	4.71	4.46	2.77	1.96	4.42	3.40						
25	7.13	4.22	1.71	1.34	4.54	3.78						
26	6.96	4.43	1.93	1.32	6.57	3.39						
27	5.51	5.42	2.63	1.66	6.89	3.72						
28	4.12	5.48	2.45	1.71	3.58	2.94						
29	4.72		2.12	2.01	2.26	2.57						
30	4.00		1.64	2.20	1.44	3.34						
31	3.54		3.70		1.98							
Sum	213.03	162.79	147.41	78.38	297.38	67.32						
Mean	6.87	5.81	4.76	2.61	9.59	2.24						
Max	15.9	10.5	9.02	9.44	30.3	4.49						
Min	3.54	2.59	1.64	0.77	0.40	0.49						
TCM	18,406	14,065	12,736	6,772	25,694	5,816						

Figure 5.9. Location of Brownsville gage in relation to the study area. Gage is 7.2 miles downstream of the study area.

Figure 5.10. Water stage versus discharge for the Brownsville gage. The example time line shown above is for period 27 Dec 2014 to 30 Jan 2015. The zero of the Brownsville gage is at the 0 ft elevation (personal communication, Glen Smith, Water Accounting Division, USIBWC). Water stage elevation at the Gateway International Bridge was estimated from the discharge curve by adjusting for the difference in the longitudinal elevation upstream of the gage (see Figure 5.11).

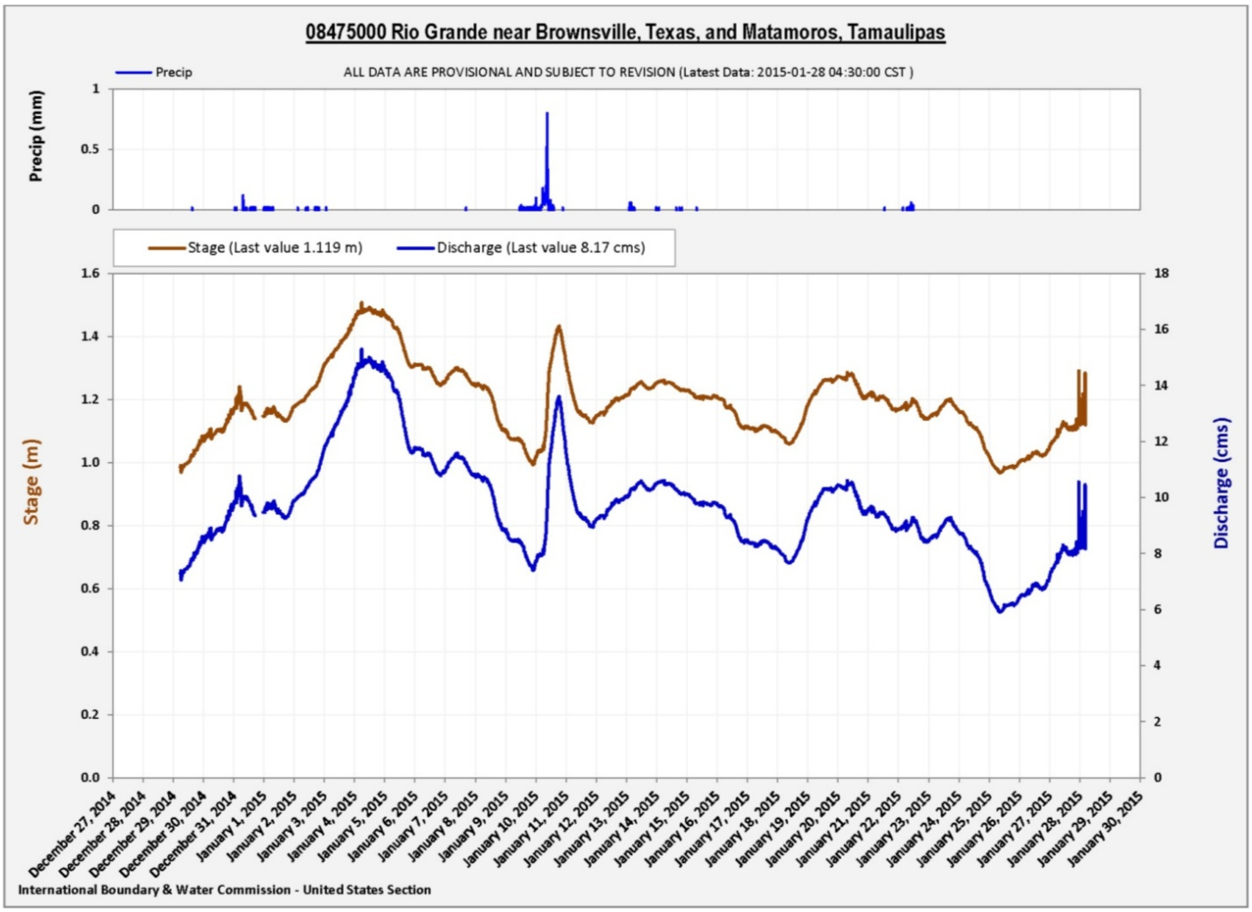


Figure 5.11. Change in water surface elevation between the Gateway International Bridge and the Brownsville gage. The presence of a grade control structure occurs 213 m (750 ft) upstream of the gage. Water surface data presented above derived from the 2011 LiDAR data of the Brownsville area.

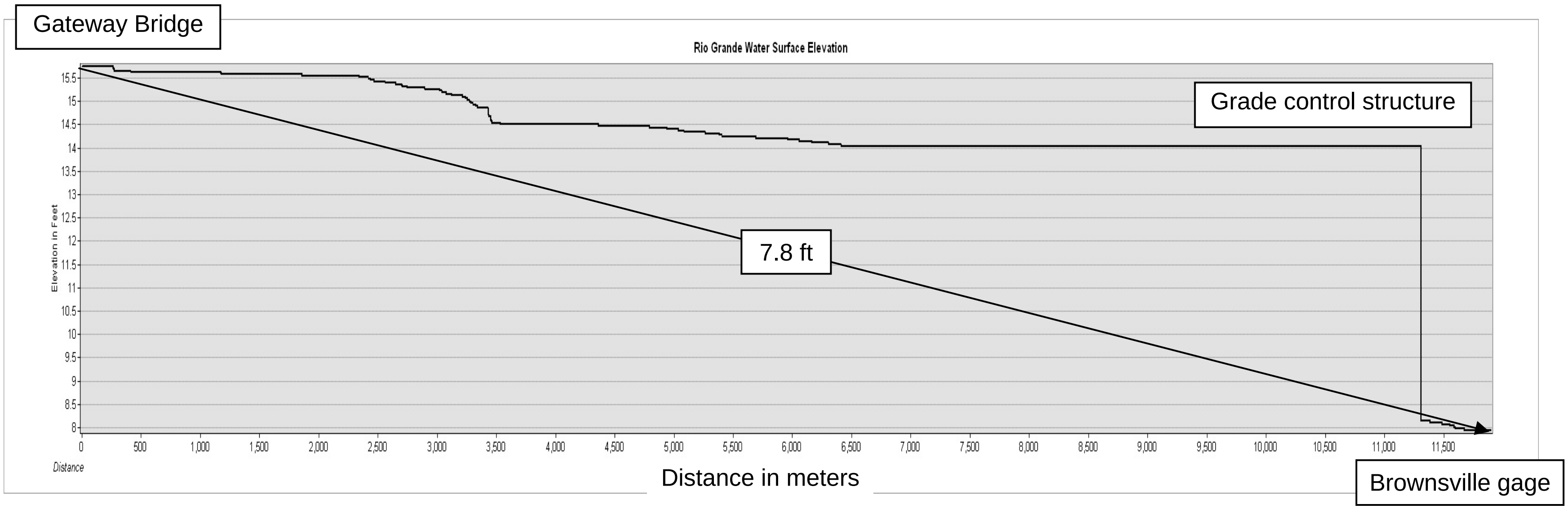


Figure 5.12. Close-up of grade control structure showing rock construction upstream of the gage (2014 Google Earth Image).

Figure 5.13. Google image of the rock boulders and cobbles that were used to build the grade control structure upstream of the Brownsville gage.

factor used in the seepage analysis was a conservative estimate rather than the actual value of 7.8 ft.

The impact of the grade control structure is to maintain the channel from vertical degradation and upstream migration of a nick point. The coarse nature of the rock used to construct the grade control structure shown in Figure 5.13 will not prevent low discharge conditions from occurring at the bridge area. The photograph of the exposed channel area downstream of the bridge area in Figure 4.20 on 12 April 2014 has an elevation of about 7 to 8 ft, which is the close to the estimated water elevation in Table 5.1 as determined from the bathymetric data presented in Figure 4.16. Thus, a 7-ft adjustment for water surface elevation in Table 5.1 and Figure 5.11 to represent the drawdown elevation at the study area site is considered a reasonable approximation of the conditions that would have occurred.

5.5 Failure time line

A time line of events leading to and contributing to the cracking and partial slope failure facilitates an understanding of the underlying causes. Reference is made to Table 5.1 and the highlighted low water periods that have been identified in yellow. The period 10 to 12 April 2014 was the first time in 2014 where discharge levels were below 1 cms. This event was captured in a previously referenced photograph of the exposed channel and vertical side slopes of the riverbank (Figure 4.12). Prior to this period, discharges were above 1 cms and reached a high of 9 cms on a few occasions in the preceding months.

The next low water period occurred from 4 to 6 May 2014. USIBWC personnel photographed the extent of cracking at the levee crest and toe on 6 May 2014 (Nunez 2014). This series of photographs is presented as an Appendix J to this report for a record of the time line. Selected photographs of the crest are shown in Figure 5.14.

The large scale displacements at the levee crest observed in early July 2014 (see Figures 1.2 and 1.3) are not present at this time. High water discharges of greater than 25 cms occur during the period 11 to 20 May 2014, followed by another low period with discharges below 1 cms between 7 and 14 June 2014. It is suspected that the 7 to 14 June 2014 period culminated with the large crest displacements observed by ERDC personnel from 1 to 3 July 2014. The maximum displacements observed at this time were between 0.5 and 0.7 ft at the levee crest.

Figure 5.14. Photographs of levee cracking on 6 May 2014.

Three low water events occurred within a 60-day period starting in early April 2014, separated by moderate to very high flow periods. These low water events likely corresponded to times where slope displacements occurred as a series of “creep” type movements triggered by the sudden increase in the hydraulic gradient in the bank during low water events lasting a few to several days in extent.

5.6 Site stratigraphy and inclinometer data

The relationship between the site stratigraphy and initial results from the inclinometer data is examined in this section. To date, three sets of readings have been collected; therefore, it is not yet possible to draw any firm conclusions regarding the behavior and history of bank movements. It is believed that regular readings during the next 6 to 12 months should be performed before any definitive conclusions are drawn from these data.

Figures 5.15 to 5.17 present the inclinometer curves for BRN-P3 - 32I, 33I, and 34I, respectively, with the basic stratigraphy added to show where slope movements have been measured in relationship to the stratigraphy. All three inclinometers tend to show some deflection along their entire length compared to their initial readings that were performed on 21 October 2014.

What is surprising is the interval where deflection begins to occur in relation to the basic stratigraphy. At the levee crest, 32I deflection starts in the upper Pleistocene stratigraphy between 34 and 36 ft. At the levee toe, 33I deflection occurs at the contact between the historic fill and the

Figure 5.15. Inclinometer data with basic stratigraphy as of 27 Jan 2015 for I32.

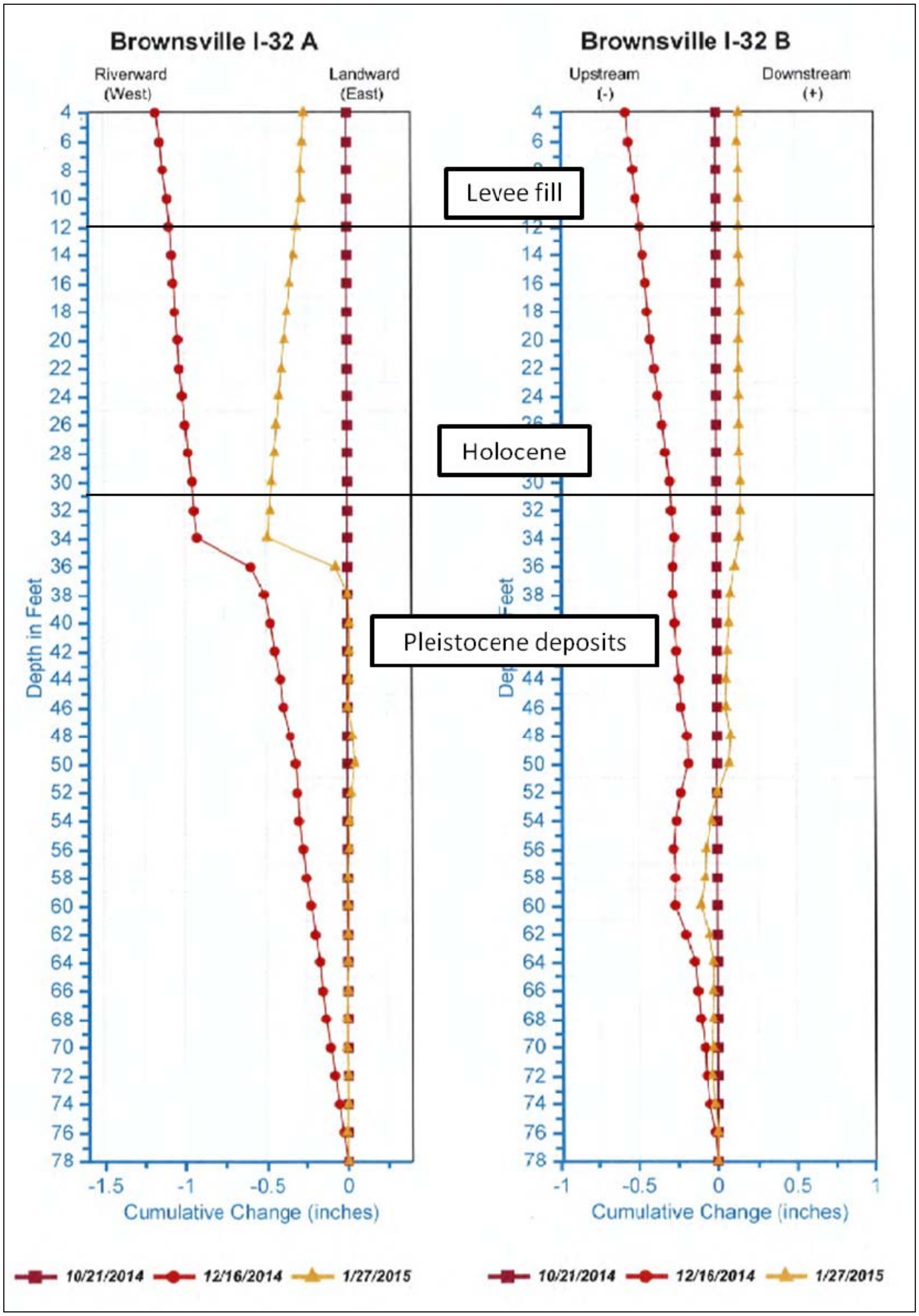


Figure 5.16. Inclinometer data with basic stratigraphy as of 27 Jan 2015 for I33.

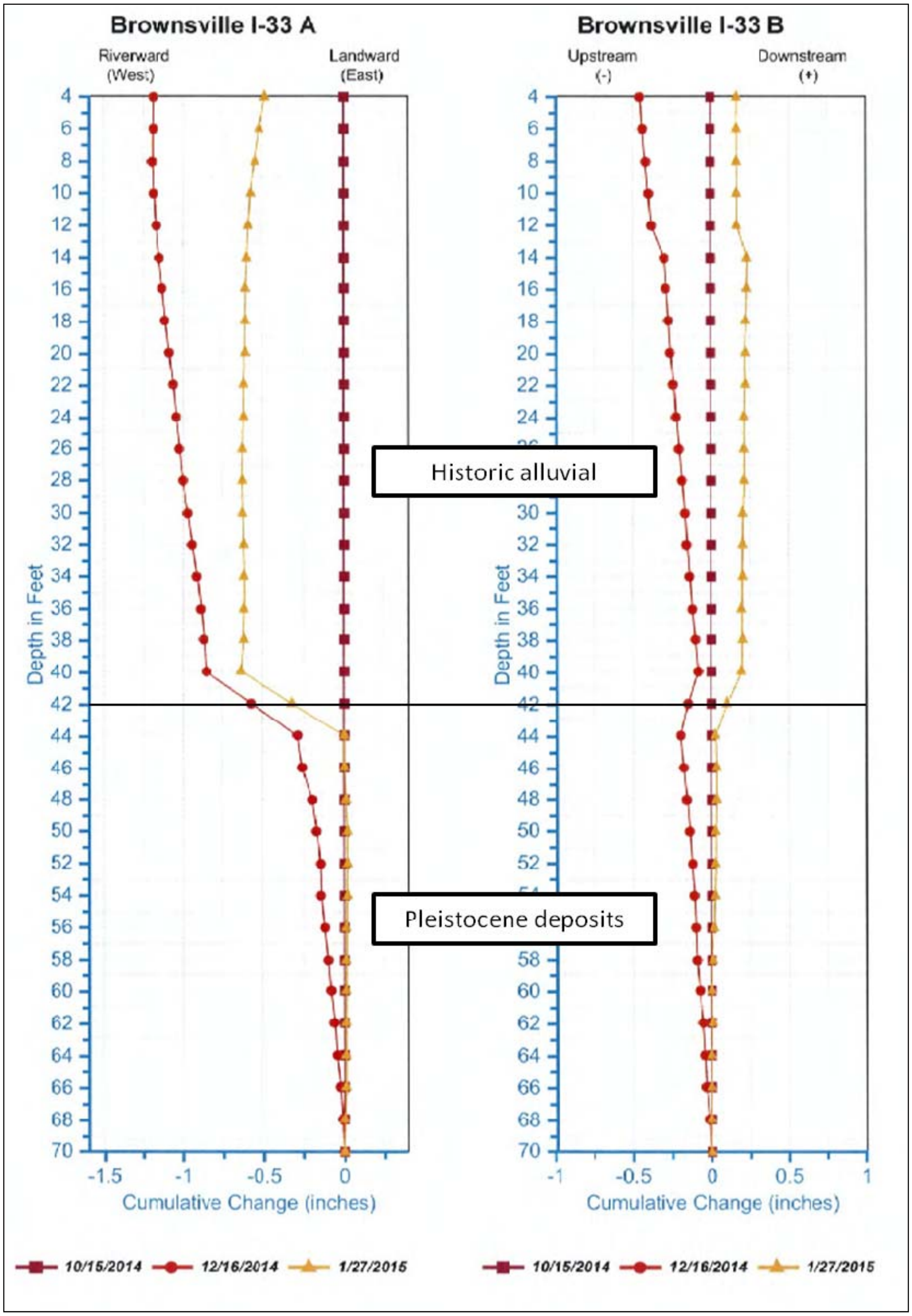
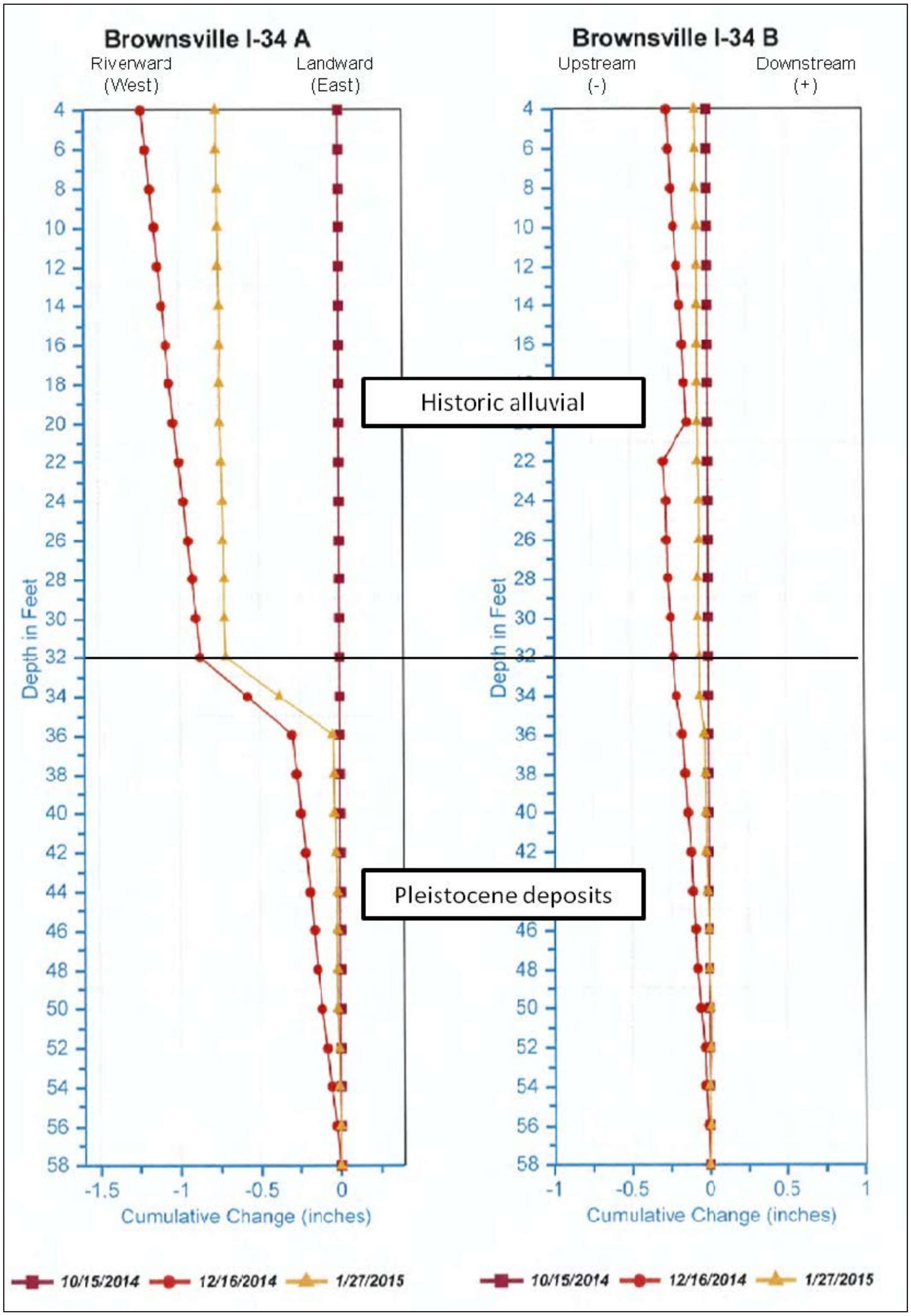


Figure 5.17. Inclinometer data with basic stratigraphy as of 27 Jan 2015 for I34.



Pleistocene surface. Near the river at 34I, the deflection interval starts below the Pleistocene contact between 32 and 36 ft.

These data suggest the Pleistocene surface is behaving as a stiff layer compared to the overlying soft sediments comprised of historic fill and Holocene alluvium. The sharp deflection within the upper Pleistocene sediments is thought to correspond to a hinge point or zone because of the deformation that is occurring in the overlying, younger, and softer sediments. A simple analogy is the inclinometer casing is acting much like a common soda straw, which is being pushed with one hand near the top of the straw, while firmly holding the straw at its midpoint with the other hand.

The combined range of movement is similar and relatively minor in the three plots, which is on the order of 0.5 to nearly 1 in. However, the most recent measurement indicates 0.5 in. or less. Again these data should be considered preliminary at this point. Additional measurements need to be performed before specific trends can be attributed to the data.

6 Seepage and Stability Analyses

6.1 Introduction

Seepage and stability analyses were conducted to better understand the partial levee failure from the rapid fall and rise in river stage that began in April 2014. The hydrograph shown in Figure 6.1 displays a peak value of 14.31 ft in the middle of May 2014. The top of bank elevation is approximately 25 ft, and the levee toe at an elevation of 30 ft, as shown in Figure 5.7. The hydrograph indicates that the incident that may have initiated the levee instability was a relatively minor fluctuation of river stage. If this drawdown in river stage triggered the movement of the levee as a series of creep events, the factor of safety against this type of failure mode was very low.

The numerical modeling effort focused on analyzing the response of the levee during the hydrograph loading using data collected during the field investigation phase of the instability investigation. As previously discussed, the field investigation data consisted of surveys, CPT, soil borings, and the results of geotechnical laboratory testing from samples collected in the field. Hydraulic data were obtained from USIBWC river gage 08-4750.00 located downstream of Brownsville. These data were scaled to represent the conditions at the location of the levee instability. Three sections were numerically modeled using the seepage and stability software SEEP/W and SLOPE/W distributed by GeoStudio. Figure 5.6, Figure 5.7, and Figure 5.8 shows the sections used in the analyses.

Review of historic maps showing the previous courses of the Rio Grande indicates that the area on the waterside of the levee between the toe and river channel contains younger alluvial fill as compared to that on the landside of the levee. Yellow lines in Figure 6.2 show the location of the 1930 river channel, and the blue, red, and orange lines represent the locations of the levee cracking that occurred in 2014. The presence of the younger alluvial fill on the waterside of the levee indicates that there may be weak zones present in the area providing support to the levee.

The numerical analysis conducted consisted of a finite element seepage analysis using the software SEEP/W to investigate steady state seepage conditions at both low and high river stages and to perform a transient

Figure 6.1. River stage and post instability sequence of events.

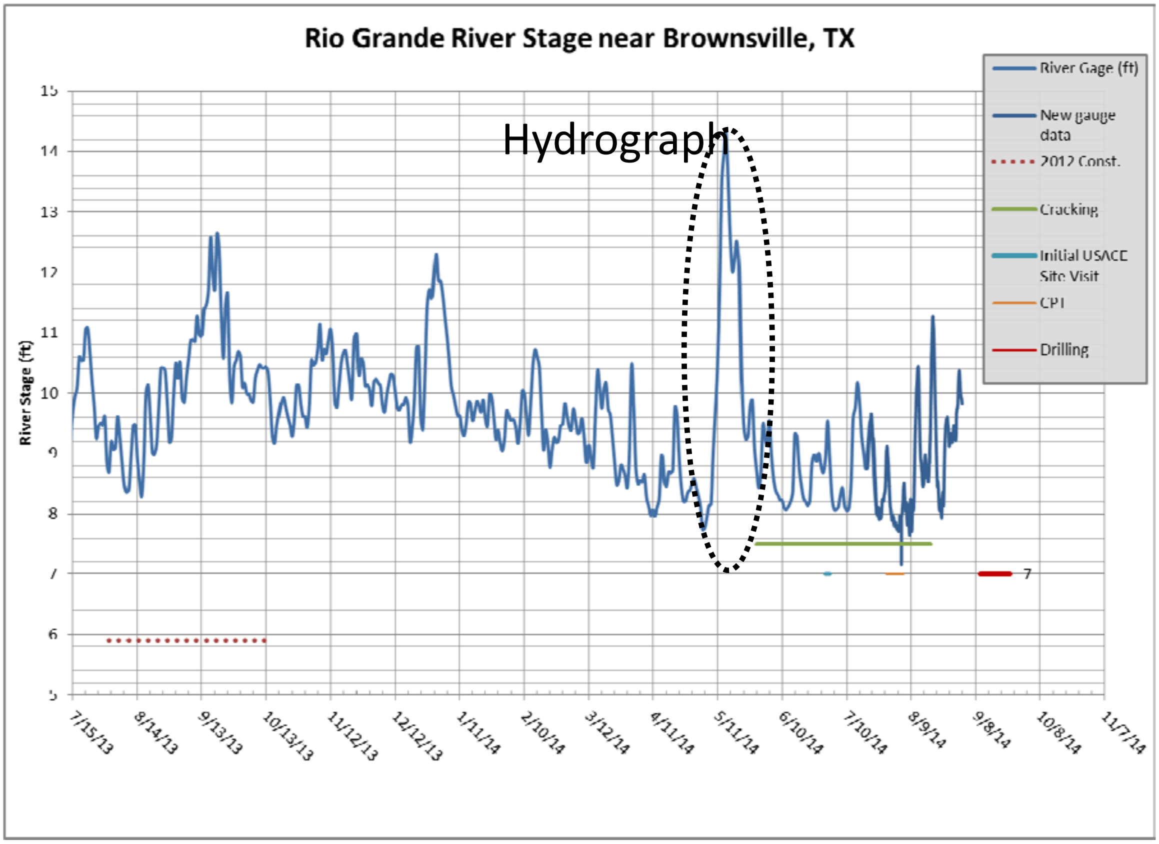
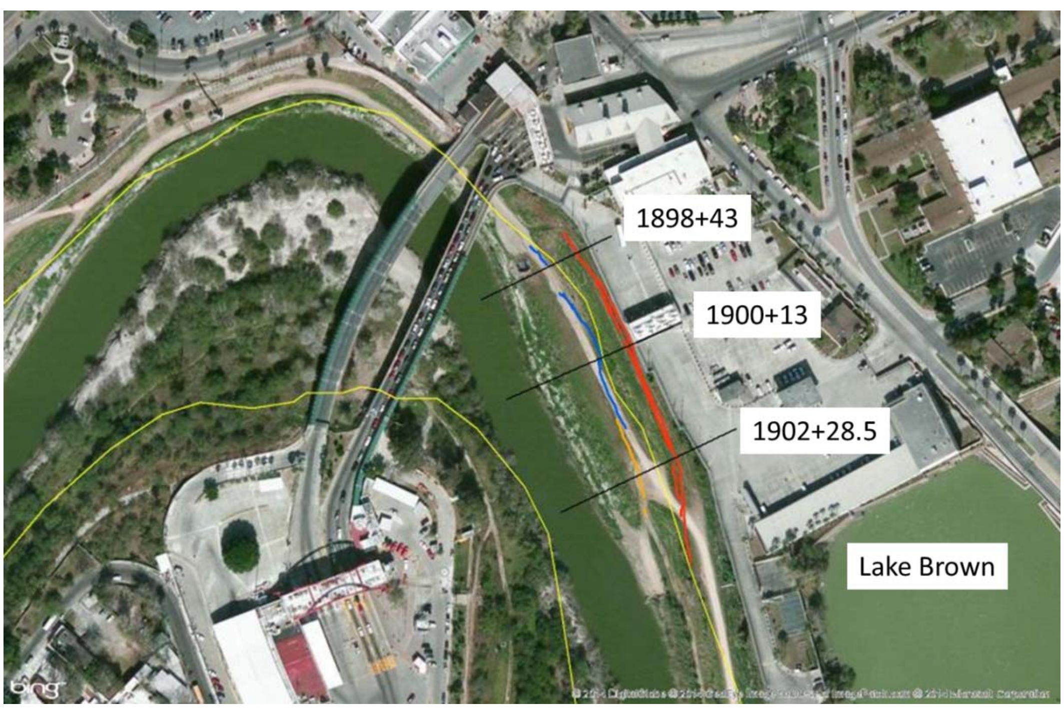


Figure 6.2. Section and cracking locations; see Figure 3.10 for old channel limits (yellow line).



seepage analysis. The transient seepage analysis consisted of using the results of the steady state seepage analysis at low river stage as the initial condition. Pore pressures that were calculated with the river at low stage were used as a starting point (i.e., time equals zero), and then the hydrograph shown in Figure 6.1 was applied. When the hydrograph was applied, each river stage was held for a certain amount of time, and the pore pressures were calculated at each time. At discrete times, a stability analysis was conducted that allowed for the factor of safety to be reported at various times during the hydraulic loading and/or unloading. The stability software used was SLOPE/W, and the Spencer method of slices, which was developed on the basis of limit equilibrium, was selected for the analysis. A rapid drawdown analysis was conducted using total stresses, and steady state stability analyses were conducted. A sensitivity analysis was performed on the critical section at Station 1900+13; the shear strength of the soft ML was varied to investigate this parameters' impact on the factor of safety.

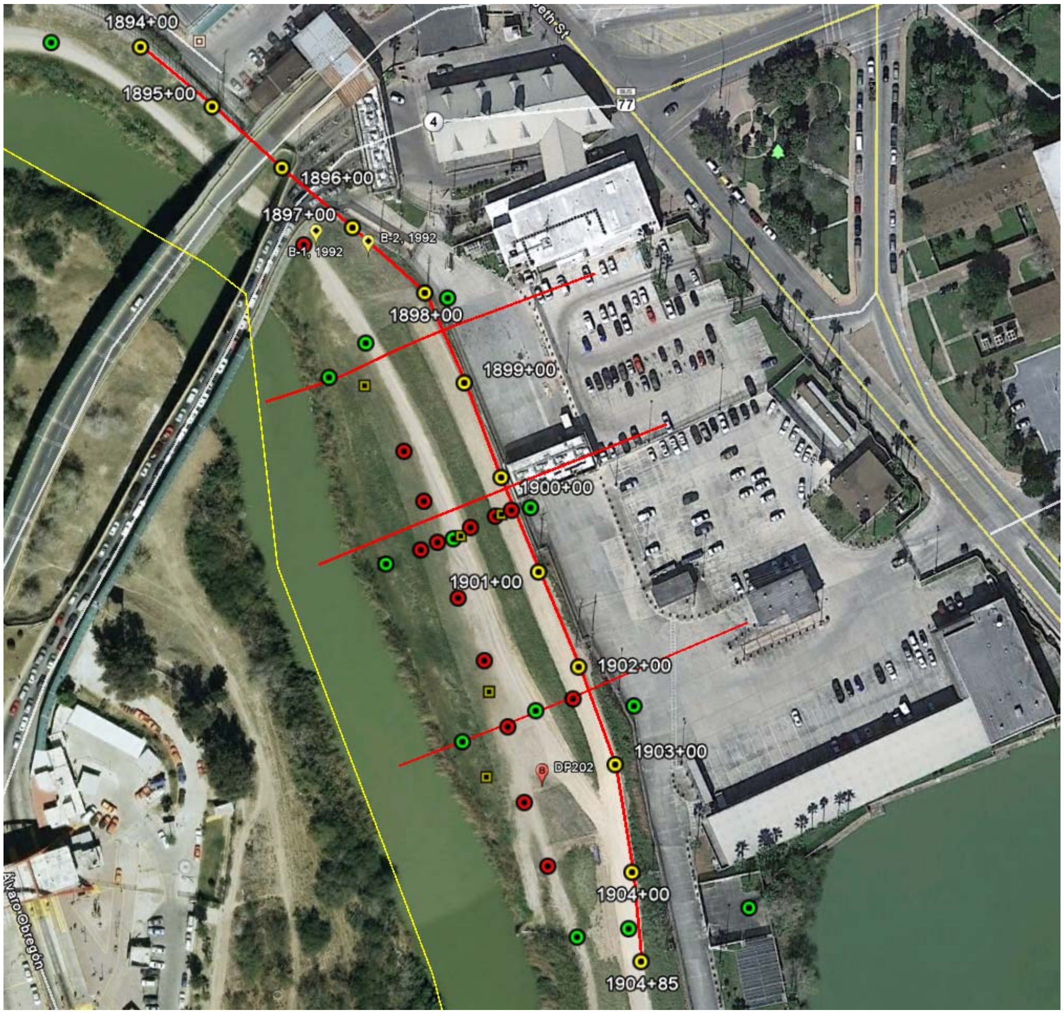
6.2 Cross sections

Three cross sections were developed using the results of the soil borings and CPTs conducted as part of the field investigation phase for the numerical model analysis. Existing maps were reviewed and used to construct the historic river channel over time. The design report from the UBLRP (Tetra Tech 2012) provided valuable information regarding the levee alignment. The purpose of the UBLRP was to raise the levee to provide 100-yr flood level protection.

Three cross sections were evaluated and are located at the upstream edge (Station 1898+43), middle (Station 1900+13), and downstream edge (Station 1902+28.5) of the levee cracking as shown in Figure 6.2. Figure 6.3 shows the cross sections with regard to the CPT and soil boring data.

The cross section at Station 1900+13 is shown in Figure 5.7; the model cross section is shown in Figure 6.4. The material located on both sides of the levee differs in age and behavior. A weak zone (labeled "soft ML") is located near elevation 12 ft. The crest of the levee in this section is located at elevation 41.2 ft, and the top of the riverbank is at elevation 24 ft. The ground surface was obtained from 2011 LiDAR data and 2014 survey data. The bank of the river and channel was obtained from the 2002 HEC-RAS

Figure 6.3. CPT boring locations with regard to three cross sections.



model data presented in the 2012 Design Report by Tetra Tech (2012). The slope of the riverbank is 1.92H:1V.

Figure 6.5 shows the cross section used in the analysis of Station 1898+43. The section at this station exhibits the same general type of stratigraphy of that in Station 1900+28.5 with more recent alluvial deposits on the water-side of the levee. The top of the riverbank is at elevation 24 ft, and the slope of the bank is 2.25H:1V.

Figure 6.6 shows the section analyzed at Station 1902+28.5, the farthest section downstream. This section is different when compared to the other two sections; the weak zone is lower in elevation, and there is a zone of riprap and a low plasticity clay layer (CL). The elevation of the top to the riverbank is 22.54 ft, and the slope of the bank is approximately 1.95H:1.0V.

Figure 6.4. Cross section at Station 1900+13.

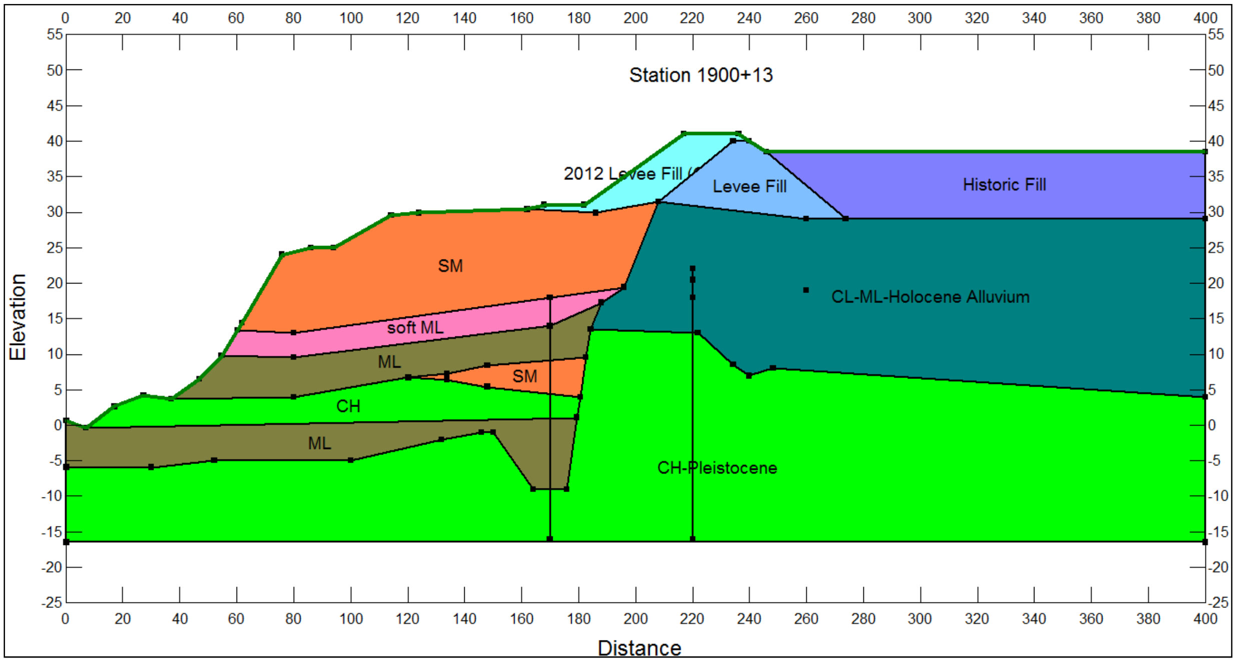


Figure 6.5. Cross section at Station 1898+43.

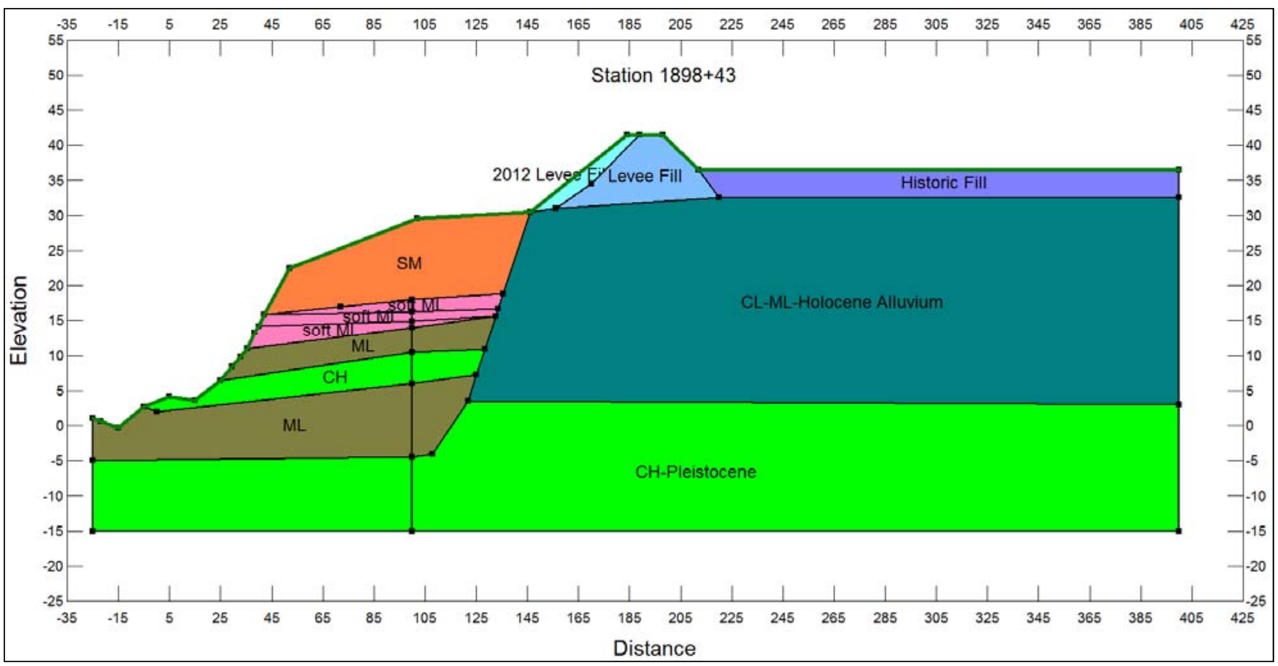
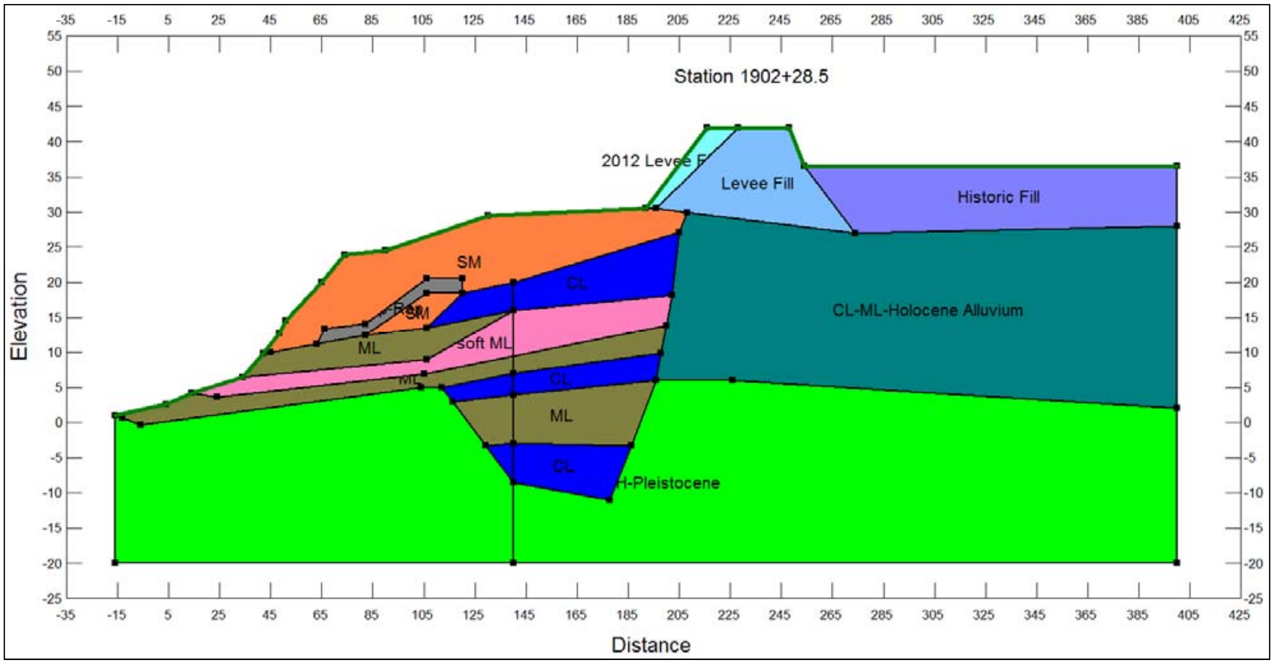


Figure 6.6. Cross section at Station 1902+28.5.



6.3 Material properties

Geotechnical laboratory tests were conducted on soil samples collected from the soil borings obtained during the field investigation. The intent of the testing was to further characterize the behavior of the soils at the site. The CPTs identified a few weak zones in the foundation material on the waterside of the levee. The laboratory tests consisted of the following:

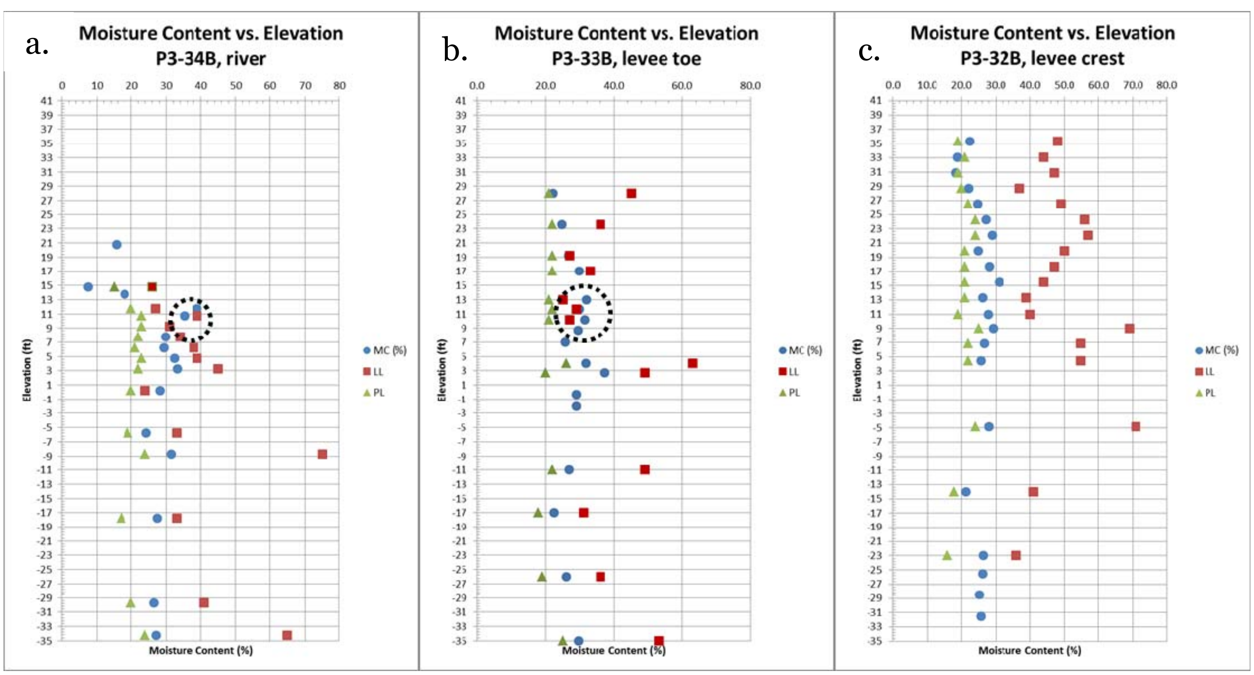
1. Grain-size analysis
2. Atterberg limits
3. Classification of soils
4. Consolidation tests
5. Direct shear test
6. Unconsolidated-Undrained (UU) triaxial
7. Moisture content

Grain-size data and Atterberg limits make it possible to classify and characterize the soils. Consolidation tests were performed to better understand the compressibility of the soil samples. Direct shear box testing was conducted to ascertain drained shear strength, and UU tests were performed to measure undrained shear strength. Due to the limited undisturbed samples collected, only single point UU tests were performed. The UU tests were performed to corroborate the undrained shear strength empirically derived from the CPTs. The results of the laboratory testing program are in Appendix I.

Moisture content profiles at Station 1900+13 are shown in Figure 6.7; encompassing boring P3-34B is located near the river, and P3-33B is located on the waterside toe of the levee and P3-32B at the crest. The red squares represent the value of the liquid limit (LL), the green triangles represent the value of the plastic limit (PL), and the blue circles represent the natural water content of the soil. The difference between the LL and the PL is the range of water contents that the soil will behave plastically is known as the plasticity index (PI). If a soil's natural moisture content is closer to the plastic limit, larger magnitude shear strength would be expected compared to if the soil was near its liquid limit. Wroth and Wood (1978) found that a soil at its plastic limit would have near 100 times the shear strength of a same soil at its liquid limit.

Profiles a and b, shown in Figure 6.7, exhibit smaller plasticity indices compared to the profile c indicating that there is a material difference. This difference supports a change between the Holocene alluvium and the younger alluvial materials in the soil cross section.

Figure 6.7. Moisture content profiles at station 1900+13.



The natural moisture content is larger than the liquid limit between elevation 9 and 13 ft in both profiles A and B, indicating a zone of lower shear strength. This lower shear strength could be due to a failure surface passing through this zone. Another possibility is that the material could have mobilized and softened due to displacements that occurred during the levee instability.

A CPT cross section showing undrained strength (S_u) normalized with effective confining pressure (p') is shown in Figure 6.8. From elevation 10 ft to 17 ft, extremely low S_u/p' values were identified ranging from 0.2 to 0.4 that start near the levee toe and extends toward the river (see Figure 6.8).

Figure 6.8. CPT predicted undrained shear strength at Station 1900+13.

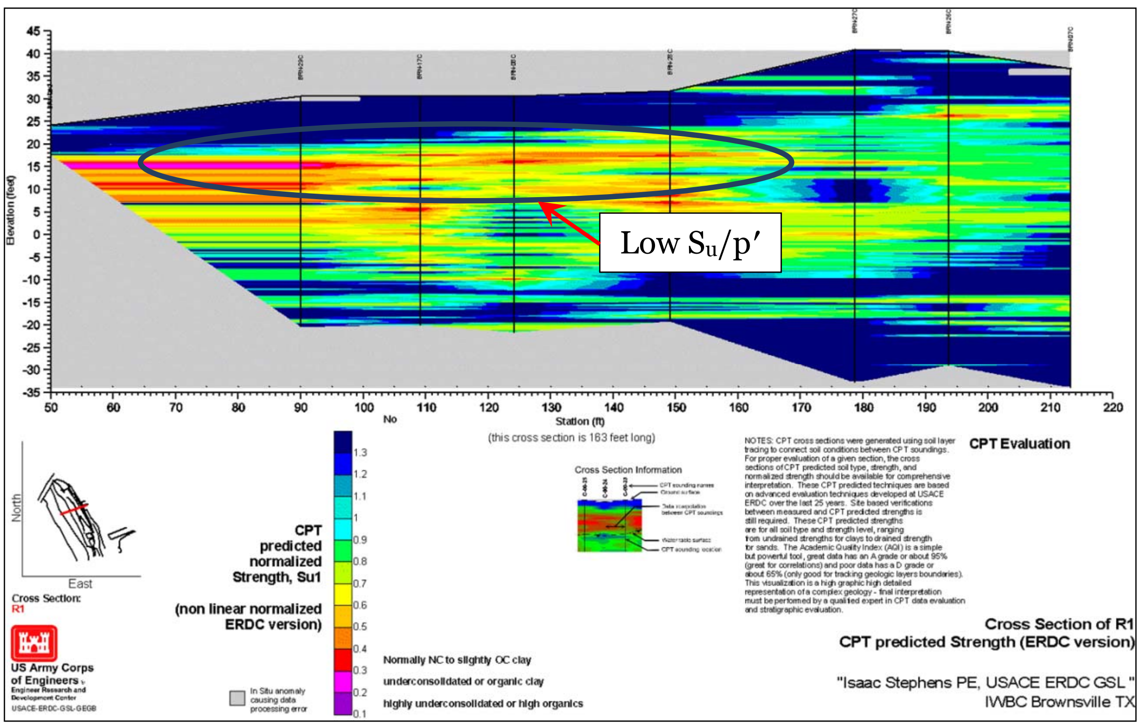
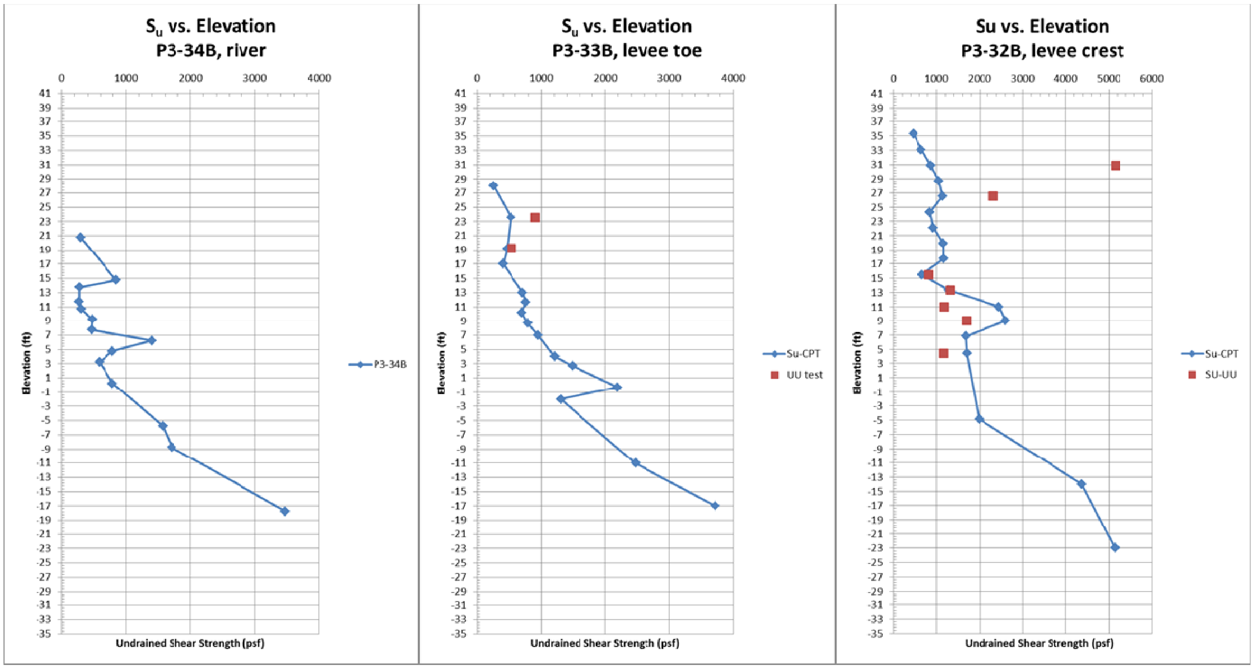


Figure 6.9 shows the undrained strength profiles for the three borings used in the moisture content profiles (Figure 6.7). The undrained strength profiles shown in blue were derived from the CPT correlations and the red squares represent the results of the UU triaxial testing. Relatively good agreement can be seen between the two tests especially near the zone where the high LL and low S_u/p' values were found. In areas above and below the weak zone, the CPT values under and over predict the value of the undrained shear strength compared to the results of the UU tests.

Figure 6.9. Undrained shear strength profiles, CPT and UU test.



Standard penetration tests (SPT) conducted during soil boring resulted in blow counts of less than five in this zone of higher water contents and in some places was less than one, meaning that the SPT fell under its own weight. A sensitivity analysis was conducted by varying the undrained shear strengths for the soft material shown in Figure 6.4. The range of undrained shear strengths used in the sensitivity analysis ranged from 140 to 225 psf. At the other two sections, the soft zone were assigned an undrained shear strength value that correlated to the value found from the CPT and were 260 psf at Station 1902+28.5 and 200 psf at Station 1898+43, respectively.

The unit weights used in the stability analysis were derived from dry densities acquired as part of the laboratory testing program. For each material type, the dry unit weight was averaged and Equation 1 was used to calculate the total unit weight.

$$\gamma_{tot} = \gamma_d + nS\gamma_w \qquad (1)$$

Where γ_{tot} is the total unit weight (pcf), γ_w is the unit weight of water (pcf), γ_d is the dry unit weight (pcf), n is porosity and S is saturation. Krahn (2004) recommends using Equation 1 because the difference between using effective and total unit weight is negligible when considering its impact on the factor of safety. Total unit weights were used during the analysis.

The shear strength and unit weight parameters used in the analysis are shown in Table 6.1. The shear strength parameters for the soft ML, levee fill, ML and CL Holocene materials were derived from the current investigations. Shear strength parameters from Tetra Tech (2012) were used for SM and CH Pleistocene. A majority of the total stress parameters were derived from the Tetra Tech report as well except for the soft ML layer where the undrained strength values were used.

Table 6.1. Material properties at Station 1900+13.

Material	Unit Weight (pcf)	c' (psf)	phi' (deg)	c (psf)	phi (deg)
CH Pleistocene	121.98	200.00	24.00	2320.00	0.00
CL-Holocene	123.37	800.00	17.30	400.00	0.00
SM	117.00	0.00	32.00	0.00	32.00
ML	119.38	300.00	32.60	0.00	29.00
2012 Levee Fill	127.34	620.00	29.20	5000.00	0.00
Levee Fill	127.34	620.00	29.20	5000.00	0.00
Historic Fill	127.34	200.00	24.00	400.00	15.00
soft ML	125.98	200.00	0.00	200.00	0.00

6.4 Hydraulic properties

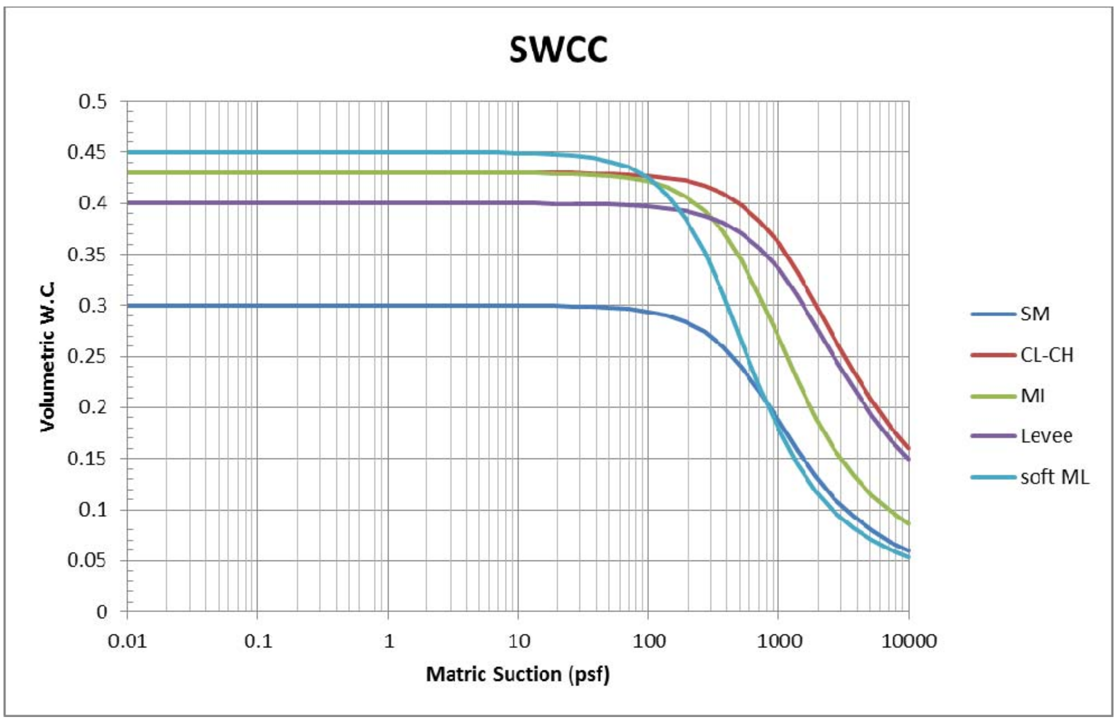
The properties necessary to conduct a steady state seepage analysis consist of the saturated hydraulic conductivity (horizontal), saturated volumetric water content (porosity), ratio of vertical to horizontal hydraulic conductivity, and volume compressibility. The saturated hydraulic conductivity values and ratio for the different parameters were taken from Tetra Tech (2012) and adjusted to conform to recommended values from Terzaghi et al. (1996). The coefficient of volume compressibility and porosity values were derived from the laboratory testing program. Table 6.2 shows the saturated hydraulic material properties used in the models. The saturated properties are necessary for both the steady state and transient seepage analyses.

Unsaturated soil properties used for the transient seepage analysis include the soil water characteristic curve (SWCC) and hydraulic conductivity function (HCF). The SWCCs used in the model are shown in Figure 6.10. The SWCCs were obtained using the sample functions available in

Table 6.2. Saturated hydraulic properties used in the numerical models.

Material	K_{sat} (ft/s)	*n*	m_v (1/psf)	Ratio
CH Pleistocene	3.30E-08	0.44	3.60E-06	0.2
CL-Holocene	3.30E-08	0.43	2.50E-06	0.2
SM	3.30E-07	0.3	5.00E-06	0.2
ML	1.00E-07	0.43	1.00E-05	0.2
2012 Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Historic Fill	3.30E-08	0.4	3.74E-06	0.2
Soft ML	1.00E-07	0.45	1.00E-05	1

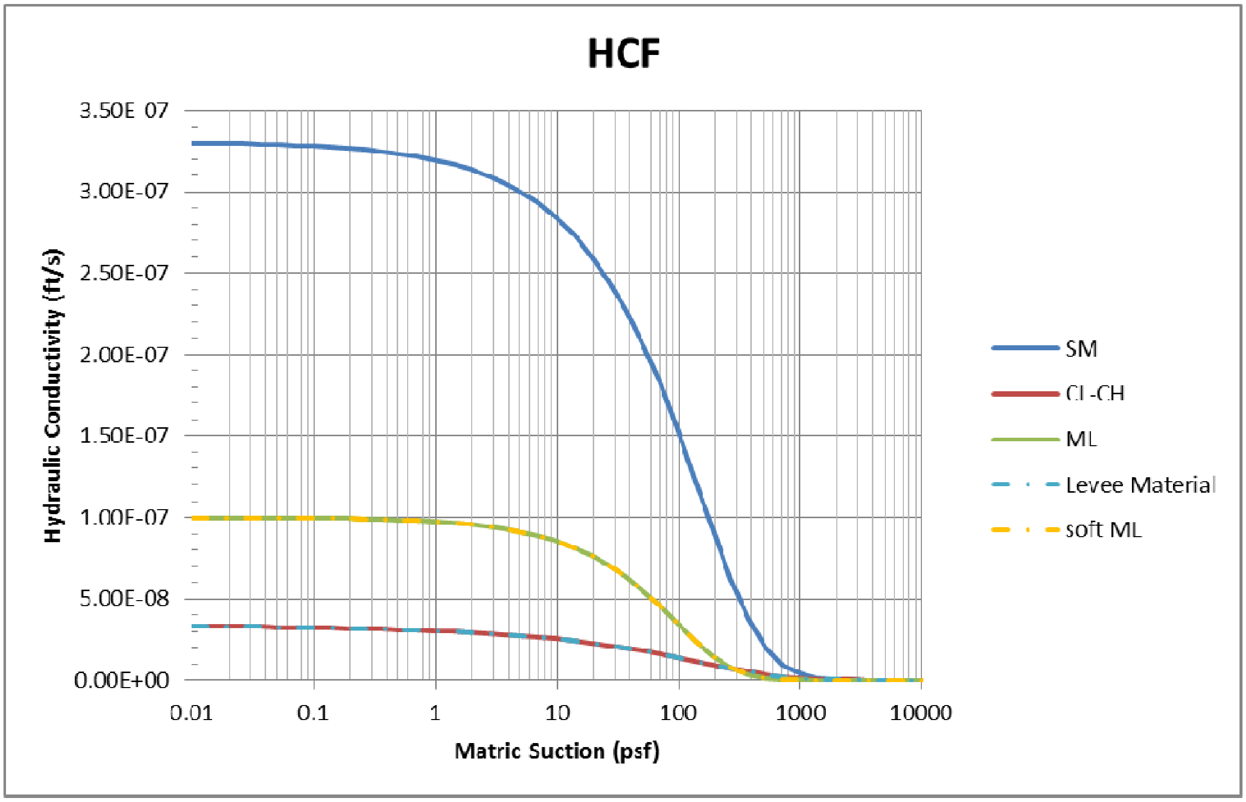
Figure 6.10. SWCC used in transient seepage analysis.



SEEP/W and setting the porosity equal to the saturated volumetric water content.

The HCFs were obtained by using van Genuchten's method (implementation outlined in Krahn 2004), the saturated hydraulic conductivity, and the SWCC parameters. The HCF is used above the phreatic surface to assign a hydraulic conductivity value lower than the saturated value due to discontinuities in the pore fluid and the presence of negative pore pressures. The HCFs used in the model are shown graphically in Figure 6.11.

Figure 6.11. HCFs used in the transient seepage analysis.



6.4.1 Boundary conditions

The key to an accurate seepage analysis is the application of proper boundary conditions. Lake Brown is located 500 ft downstream from the center of the levee cracking area and is held at an artificially high water level because the city of Brownville is using it as a storm water retention basin. In July 2014, a well logger was installed at Lake Brown, and the elevation of the water surface was monitored. These data are shown in Figure 4.8.

The plot shows that in July and August 2014, the elevation of Lake Brown was near 26.5 ft while through the fall and winter, the water surface elevation increased to near 29 ft. The elevation of the river at the time of the cracking incident was near 14 ft at the highest elevation and 7 ft at the lowest elevation. Even when the river stage is at the highest elevation, the Lake Brown is still almost 12 ft higher. The difference in elevation between the lake and the river allows for elevated pore pressures on the landside of the levees and low pore pressures on the waterside of the levee. These boundary conditions were captured by assigning total head boundary conditions on the landside and waterside boundaries.

Five different monitoring wells (Figure 6.12) were installed during the site investigation conducted in 2014, and well loggers were used to measure the water level and store the data until it could be downloaded.

The well labeled P3-32w has both a shallow and a deep well associated with it. Well P3-34w is located near the river and responds closely to the water elevation of the river while the other wells lie between the water elevations of the river and Lake Brown. This condition indicates that the lake is contributing to the increase in the pore pressures on the landside of the models and that these high pore pressures decrease across the model until the base elevation of the river is reached.

The hydrograph that was used for the transient analysis is shown in Figure 6.13. The initial stage of the river was at elevation 7.77 ft and the peak was at elevation 14.31 ft. The peak elevation occurs $8.64(10^6)$ sec from the initiation.

6.5 Summary of seepage and stability analyses

Three cross sections were developed for the seepage and stability analysis. There were steady state seepage and stability analyses performed on each section at high and low river stages. There was also a transient seepage and stability analysis performed on each section. The results of the analysis are fully displayed in the Appendix L and the critical section will be discussed further. Table 6.3 shows the results of the numerical analysis. The lowest factors of safety were in the section representing Station 1900+13. Factor of safety is defined as the ratio of resisting forces to the applied forces (gravity and applied surcharges). A factor of safety of 1.00 indicates that failure has occurred. The USACE requires a long-term factor of safety of 1.3 and the end-of-construction factor of safety to be 1.4. The section analyzed at Station 1900+13 was found to have the lowest factors of safety and because of these results; it is considered the critical section.

Figure 6.14 shows the results of both the transient seepage and stability analyses conducted at Station 1900+13. The factor of safety was calculated to be 1.02 and occurred at a time of 1.64e6 sec after the hydrograph peak. The undrained shear strength assigned to the weak zone was 150 psf. Figure 6.15 shows the river elevation on the left vertical axis plotted against time and the factor of safety calculated at discrete points (right vertical axis).

Figure 6.12. Locations of monitoring wells.

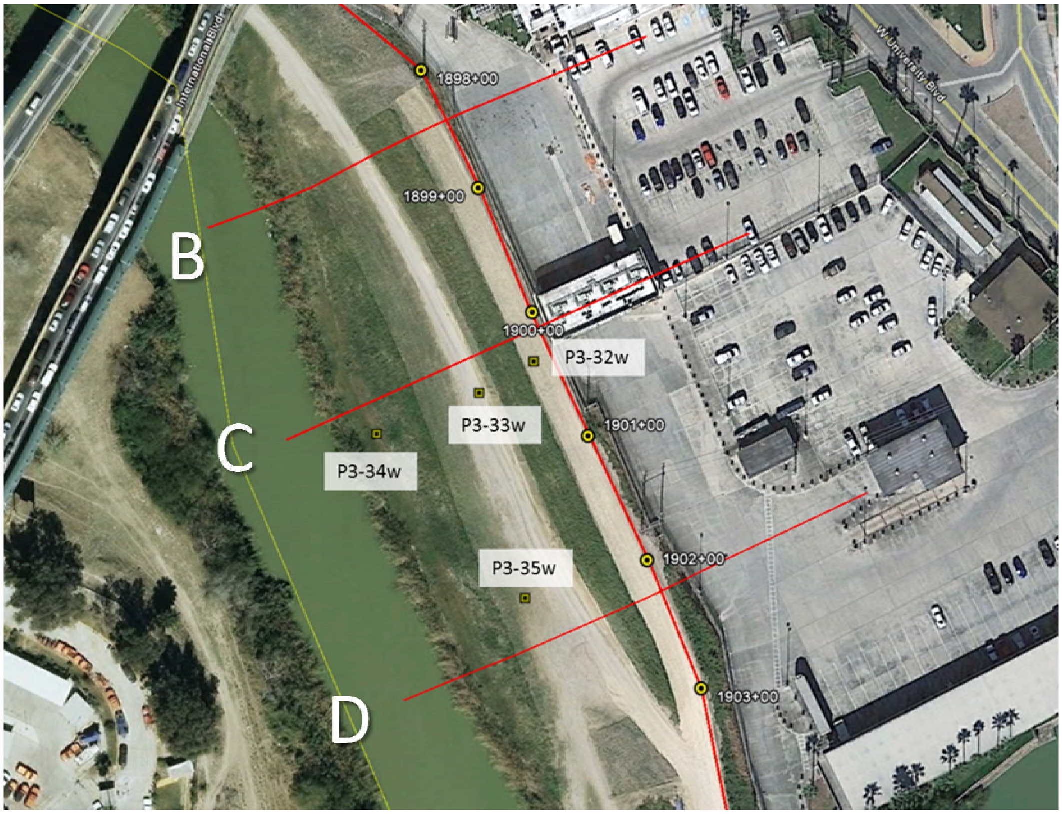


Figure 6.13. Hydrograph used in transient seepage analysis.

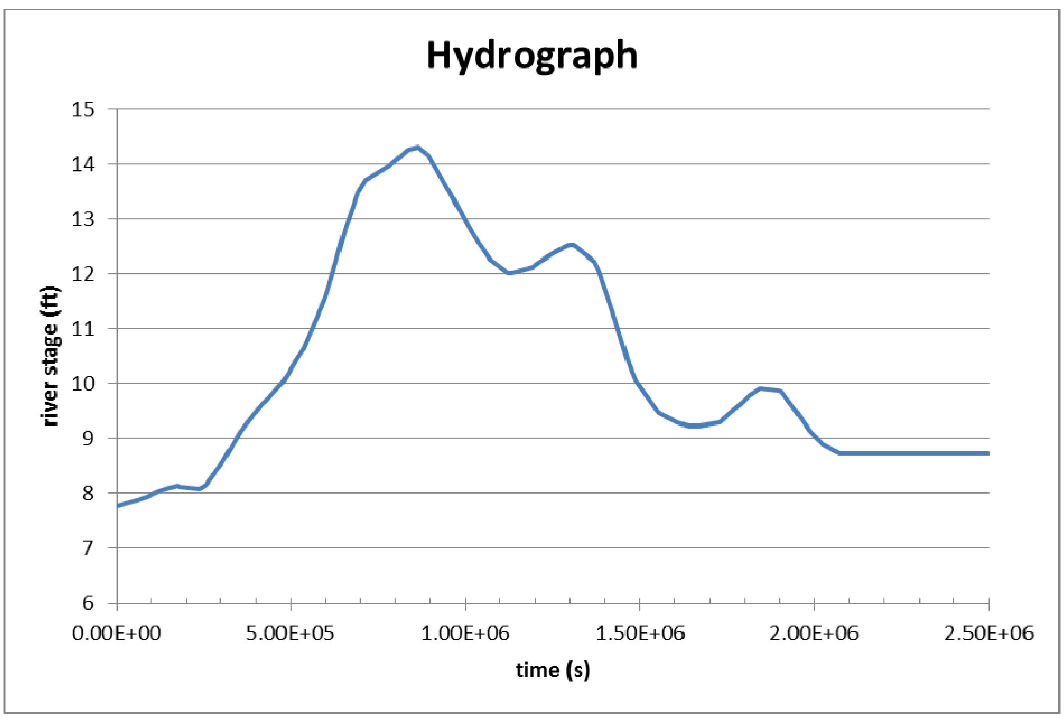


Table 6.3. Results of the numerical analysis.

Stability		Analysis			Hydrograph
Section	Station	SS low	SS high	RD	Minimum
111	1898+43	1.11	1.10	1.06	1.10
211	1900+13	1.26	1.10	1.00	1.02*
311	1902+28.5	1.20	1.17	1.17*	1.12

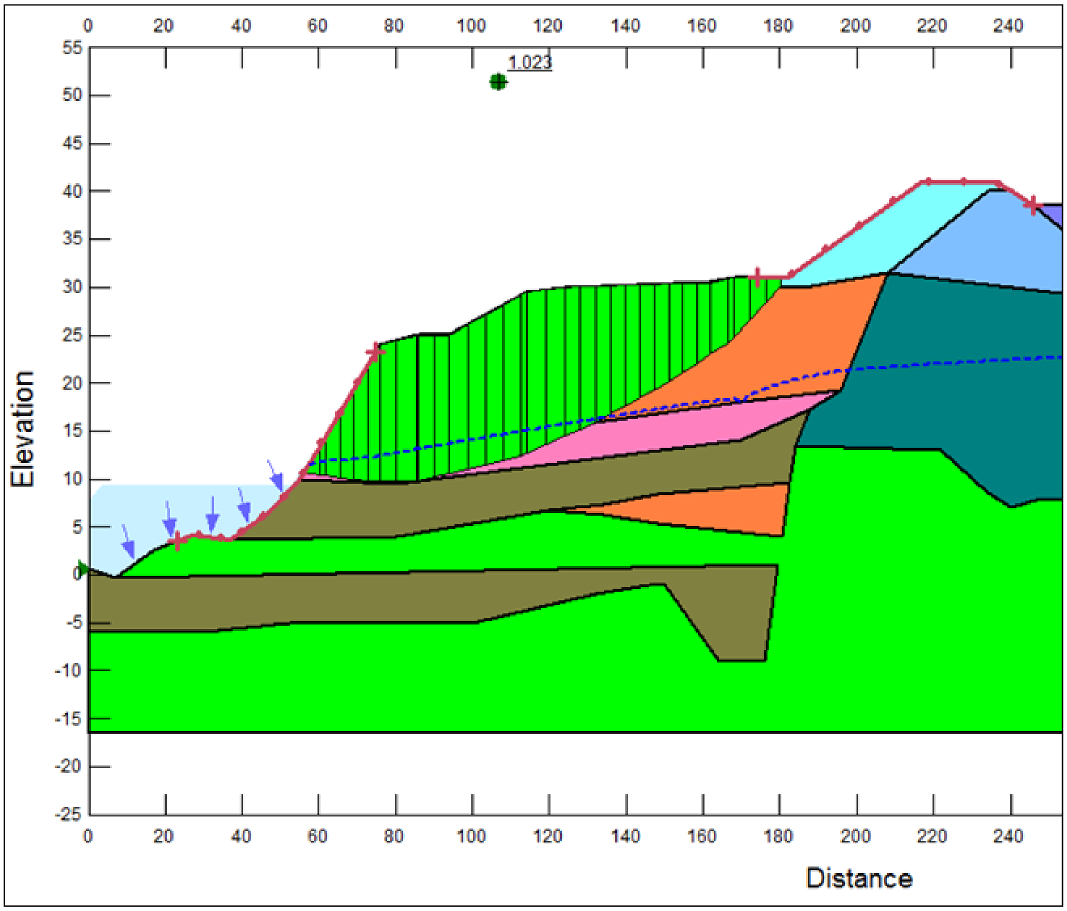
**lower factor of safety calculated, see discussion*
SS-steady state
RD-rapid drawdown

The factors of safety shown in Figure 6.15 were as expected. As the hydrograph peak was occurring, the highest factor of safety (1.15) was calculated (1.15) because of the stabilizing effect of the water weight against the riverbank. Once the river stage drops, the factor of safety decreases with a slight increase due to a second smaller increase in river elevation then the factor of safety decreases rapidly enough that the materials above and below the soft ML layer do not have time to dissipate the pore pressures generated during the flood loading. There is an increased weight associated with the excess pore fluid. The reason the factor of safety decreases after the peak of the hydrograph has been reached is due to the excess weight of the saturated materials with no reinforcing effect of the water.

The type of failure (i.e., from the levee toe through the alluvial material) shown in Figure 5.16 would have led to a progressive type of failure mode as described by Bjerrum (1967). The material located between the levee and the river provides support for the levee material, but when this material mobilized and displaced toward the river, the crest material slumped in the same direction. A quick analysis was performed to investigate this theory by removing the initial slide material from the model and then calculating a factor of safety, which was found to be less than 1.0. This indicates that removal of the toe material would lead to failure of the levee crest.

A sensitivity analysis of the undrained shear strengths in the soft ML material was conducted to better understand the response of the levee to the drawdown event that seemed to have triggered the levee cracking. Figure 6.16 shows the result of this sensitivity study. The undrained shear strength of the soft zone was varied from 140 to 225 psf. The same general trend of increasing factor of safety with increasing river stage until the peak of the hydrograph occurs is found for all the shear strength values

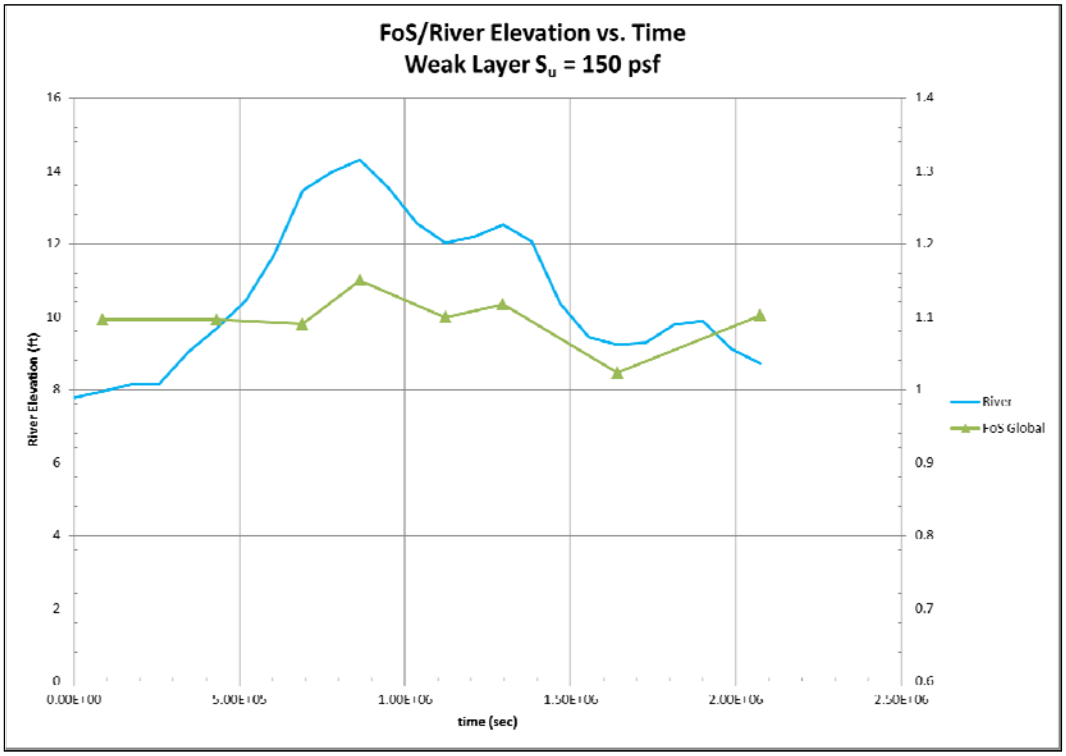
Figure 6.14. Results of stability analysis Station 1900+13, hydrograph loading t = 1.64E7s.



used. The trend skews when a value of 169 psf or less is used. This is due to a change in failure surface geometry. Figure 6.17 show the failure surface with the soft ML assigned a shear strength value of 215 psf. The same failure surface search limits are used as for the rest of the sensitivity analysis. In this case, the entry and exit limits of the search are defined as well as the minimum tangent for the circle. When the failure surface intercepts the end of the entry line (red line segment with crosses at the beginning and end), it means that the minimum surface may be located outside the bounds of the current search. In order to find the minimum, either another search method can be used or the entry line must be relocated. When another search method (block search method) was applied, it revealed that the minimum surface was located closer to the river.

When the block search method was employed, a minimum failure surface that was close to riverbank and the minimum factor of safety was 1.07 in this case at t = 1.12e6 s. If the search was set closer to the bank of the river,

Figure 6.15. Results of transient seepage/stability analysis at Station 1900+13; FoS = Factor of safety.



an infinite failure mode was obtained. This is a product of the SM material not having a cohesion value and the steepness of the riverbank. Factors of safety for infinite slope analysis were less than 1.0 indicating that the bank should not be that steep and may be actively failing or that the friction angle for the SM material was set too low. Observations made during a December 2014 visit by ERDC indicate that the bank is sloughing due to its steepness and possible river undercutting (see Figure 6.19). The higher factor of safety was reported in Table 6.3 to provide continuity between search methods and failure modes.

6.6 Conclusions

The results of the field and laboratory testing indicate the presence of soft alluvial sediments in the foundation material located between the river and levee. The soft material was found to have natural water contents above the liquid limit, which indicates a very low shear strength. CPT testing in this area correlated the cones tip and side resistance to a minimum undrained strength ratio (S_u/p') of near 0.2 and corresponded to a soft zone. The soft zone was not considered in the design of the levees.

Figure 6.16. Results of sensitivity analysis on Station 1900+13.

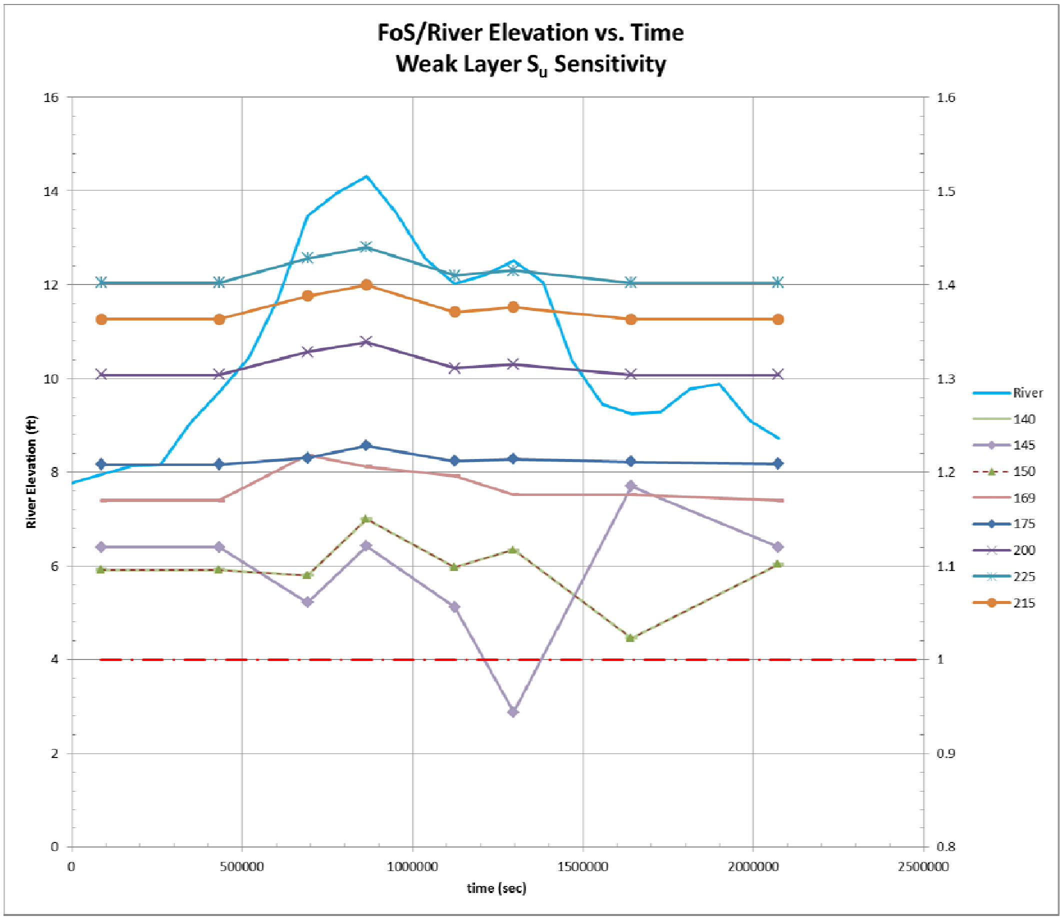


Figure 6.17. Station 1900+13 with weak zone shear strength at 215 pcf.

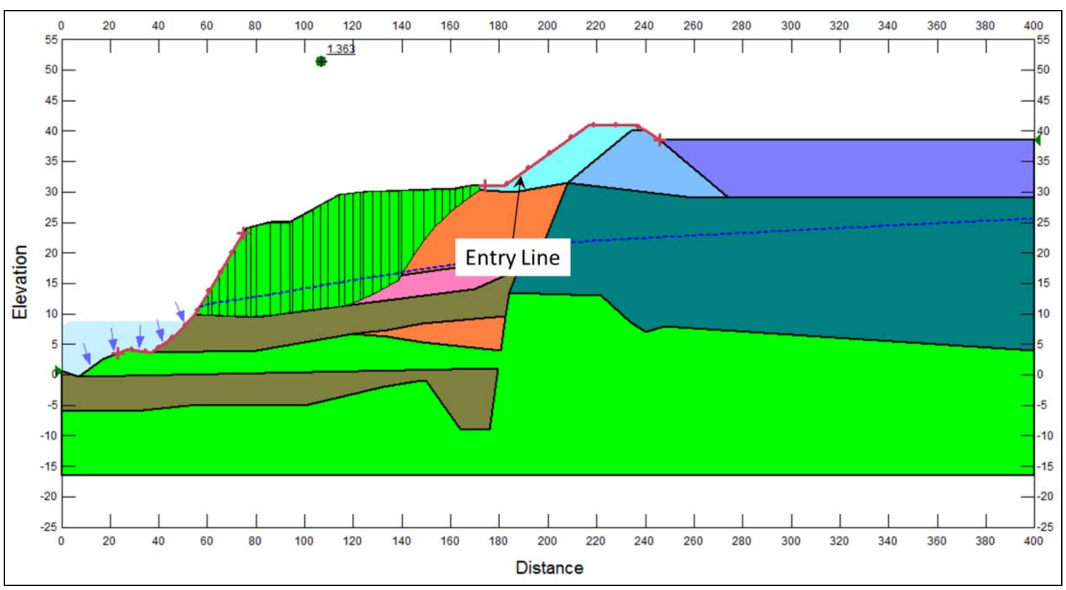
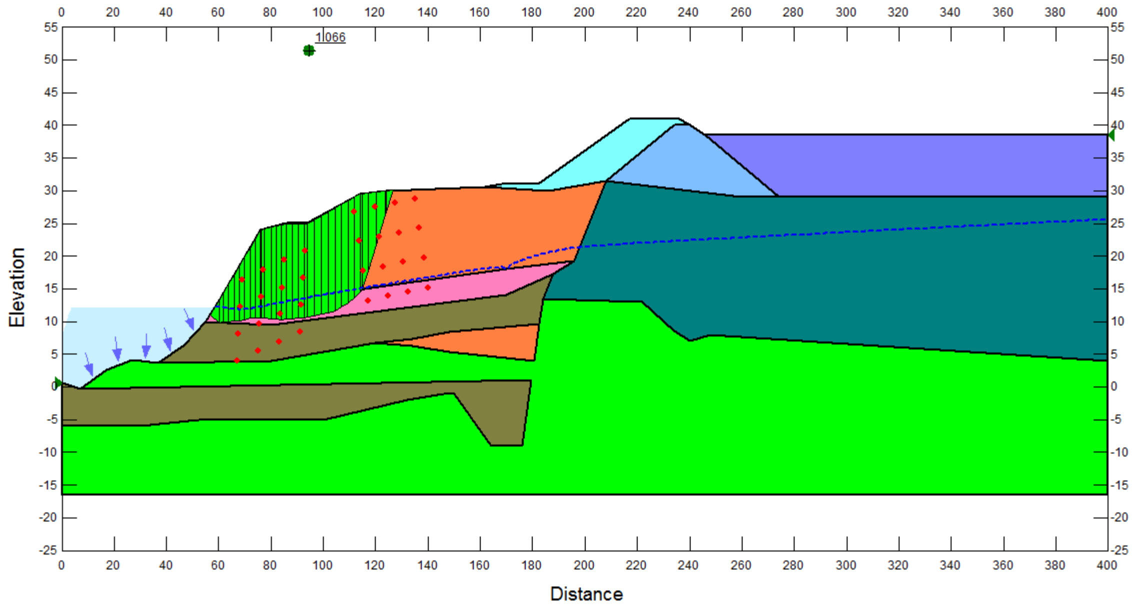


Figure 6.18. Failure surface at station 1900+13, riverbank failure.



The factor of safety that exists in reality is much less than required for a robust flood protection system.

The models indicated that a blocky progressive failure mode may have been the cause of the cracking witnessed on the Brownsville levee. This type of failure is likely attributed to the relatively weak alluvial materials located in the area between the toe of the levee and the river. It is likely that a series of events, including the 2012 construction, multiple river drawdown events, and high water level in Lake Brown, contributed to the instability.

The failure zone is estimated to be above elevation 7 ft on the riverside toe and likely bisects through the levee embankment. The shear surface is highlighted by the high water contents shown in Figure 6.7 and the weak shear strengths illustrated in Figure 6.8. The full depth of cracking is unknown, but likely deeper than 1 ft. Initial cracking was noted early April 2014; the photo shown in Figure 6.20 was taken in July 2014. IBWC staff noted that the cracks were much deeper. When ERDC staff arrived on site, there had been several rain events that washed debris into the cracks.

Figure 6.19. Minor riverbank sloughing in December 2014.

Figure 6.20. Depth of levee cracking, July 2014.

7 Discussion

7.1 Overview of the geotechnical study activities

An initial site visit to the Brownsville levee reach was conducted during the first week of July 2014 by members of the ERDC geotechnical team to evaluate the extent of the surface cracking and develop a strategy for the subsequent field investigation phase of the study. Three longitudinal crack sets, extending between levee stations 1898+00 to 1904+00, had developed when the ERDC team first visited the levee site.

A drilling and sampling program was subsequently initiated in September 2014 as part of the field investigation program to collect site-specific geotechnical properties of the subsurface, determine the vertical and horizontal limits of the levee and riverbank soils and associated stratigraphic contacts to conduct slope stability analyses. The soils exploration program consisted of 32 CPTs and 6 soil borings. Soil samples were obtained and monitoring instrumentation installed in the soil borings. Soil sampling involved both undisturbed (3-in. Shelby tube) and disturbed (split-spoon) sampling techniques.

As part of the field investigation efforts, three different monitoring systems were used at the levee site. Instruments included piezometers to determine the elevation of the ground water in pervious stratigraphic zones and inclinometers to determine rate of movement and depth to the shear zone. Elevation surveys of the levee reach, were used to perform continuous monitoring and establish base line condition for later surveys.

Additionally, a comprehensive review of the design and construction documents was made to fully understand the UBLRP activities that were performed in the study reach. An important part of this study included a historic evaluation and reconstruction of land use changes in the study area to better understand past levee performance issues and major land use changes through time that may have contributed to the partial slope failure.

A numerical modeling analysis was conducted to investigate how the levee would respond to different loading conditions for three sections. The loading conditions were based on a hydrograph obtained from gage data and consisted of steady state analyses, rapid drawdown with total stresses and an effective stress rapid drawdown analysis. The intent of these analyses was to better understand the type of failure mode that may have impacted the levee during the partial slope failure.

7.2 History of levee past performance

The levee reach has been relatively stable since the USIBWC assumed control of the levee system in the 1930s. Stable side slopes in this area have been 3H:1V, as shown in historic photos (see Chapter 3). No record of past performance issues were discovered during this study. In general, the channel alignment through the levee reach has been relatively stable since the early 1900s. However, much of the floodplain adjacent to the river in the study area was formed only during the past 75 years as evidenced by the historic map data that were compiled as part of this study. The channel has decreased significantly in width since the 1930s because of reduced river flows and associated sedimentation due to the construction of upstream dams, which has regulated river flow, and because of increased water use in the LRGV by agricultural and urban population growth. The limits of the 1930 river channel generally correspond to the current day levee toe in the study reach.

7.3 Geology

The riverbank and levee foundation in the study area are composed of historic, Holocene, and Pleistocene deposits as determined from the CPT and soil borings made during this study. Holocene age alluvial deposits in the study area are associated with active river migration by the Rio Grande. The nearby Lake Brown is an example of horizontal river migration that occurred in the study area within a relatively short time span. This oxbow was likely abandoned by the Rio Grande River some 200 to 300 years ago. Point bar deposits are present in the study area associated with this oxbow.

Pleistocene sediments are present in the levee and riverbank foundation between depths of 30- and 50-ft depth. These older sediments are significantly different in terms of their engineering properties from the younger sediments that overlie them. Pleistocene sediments beneath the levee were exposed to intense weathering approximately 12,000 to 15,000 years ago during the last glacial maximum when sea level was more than 300 ft lower than the present day stand.

Historic and Holocene alluvial sediments are primarily fine-grained, gray to dark gray in color, soft to very soft, and contain organic materials (e.g., wood, roots, and charcoal). Historic sediments contain cultural debris, such as glass, rusted metal, and buried riprap. Historic channel fill sediments generally become sandy near the surface and are finer-grained with depth. Wood is often present below the water table. In contrast, the Pleistocene sediments are different. These sediments are brown to tan in color, clay-rich, more uniform, stiff to very stiff, mottled, and contain carbonate concretions.

7.4 Groundwater

Groundwater flow is locally toward the river and regionally to the Gulf of Mexico based on basic understanding of ground water hydrology in alluvial aquifer settings. Abandoned oxbows are present throughout the greater Brownsville area and these have lake levels that have been relatively stable during the past as identified by historic topographic map data. Lake Brown maintains a relatively constant lake level due to surface drainage into the lake and pumping water from the Rio Grande into the lake by the city of Brownsville for the Southmost Campus.

Lake Brown is hydraulically connected to the river as evidenced by the presence of local sand layers in the Holocene alluvium, the occurrence of point bar stratigraphy at the southern edge of the study reach, and the measured response of water levels in piezometers installed for this study in the historic, Holocene, and Pleistocene stratigraphy.

Water levels in Lake Brown vary between elevation 27 and 29 ft. Interestingly, the 1929 lake level does not vary significantly from the present day level. Locally the lake corresponds to the upper groundwater surface, while near the Rio Grande; the piezometric surface corresponds to the river level, a 15 to 17 ft hydraulic head difference between the lake and the river depending on river stage. Lake Brown has a hydraulic connection to the river through pervious point bar sediments and deep scouring as evidenced by historic map data.

Point bar deposits are especially noted for their seepage potential during flood stage. During river flooding, horizontal flow through the pervious sands can extend great distances landward in the shallow aquifer because of the steep hydraulic gradients produced by the river. Conversely, a rapid drawdown of the river, combined with a stable lake level that is significantly higher than the river, and a pervious substratum permits elevated pore pressure conditions locally in the shallow aquifer by a sudden drawdown condition.

7.5 Inclinometer data

To date only three sets of readings have been collected and it is not yet possible to draw any firm conclusions regarding the behavior and history of bank movements. Measurements to date from the three inclinometers indicate the Pleistocene surface is behaving as a stiff layer compared to the overlying softer historic and Holocene sediments. Deflections start in the upper Pleistocene sediments and are thought to represent a hinge point because of the deformation that is occurring in the overlying, younger, and softer sediments. A simple analogy is the inclinometer casing is acting like a common soda straw which is being pushed with one hand near the top of the straw (column), while firmly holding the straw at its midpoint (Pleistocene surface) with the other hand. Recent measurement indicates 0.5 in. or less and these data should be considered preliminary in nature at this point.

7.6 Survey data

Survey data involved periodic elevation surveys of the bank and levee at three transects; a terrestrial LiDAR survey to establish base line conditions and permit measurements of the surface deformation, and bathymetric and side-scan sonar surveys of the river channel. Elevation monitoring surveys performed during this study were started well after the major displacements occurred. As of October 2014 surface surveys of elevation have not identified any appreciable movements.

Bathymetric and side-scan sonar data identify a channel bank on the U.S. side of the river which shows historic bank instability as evidenced by the scallop bankline topography, both above and below the level of the river. A deep scour pool between 0- and 1-ft elevation is present beneath the Gateway International Bridge that extends to about the limits of the upper bank riprap that is immediately downstream of the bridge. Discontinuous stone riprap is also present in the channel as evidenced by side-scan sonar data and a low water photograph of the channel that was made by USIBWC personnel on 12 April 2014. The present day river channel has nearly vertical banks as opposed to the much large channel that existed in the 1930s, with side slopes of about 3H:1V.

7.7 Timeline of 2014 partial failure

Three low water events occurred within a 60-day period starting in early April 2014, separated by moderate to very high flow periods. These low water events correspond to times when slope displacements began occurring as a series of “creep” type movements. These episodic movements were likely triggered by the rapid increase in the hydraulic gradient in the bank during low water events lasting a few days in extent. Photographs taken during these low water flow events lend support to this viewpoint.

7.8 Seepage and stability analyses

Three cross sections were analyzed for both the seepage and slope stability. Station 1900+13 was the most critical section. This station roughly corresponds to the center of the cracking identified during the preliminary site investigation. All three sections had low factors of safety in part due to the low shear strength assigned to the soft alluvial sediments located between the levee and the river channel. Low shear strengths were supported by the S_u/p' charts derived from CPTs and the water content profiles attained from the laboratory testing program. The water content profiles identified areas where the natural water content was well above the liquid limit, indicating a zone of low shear strength.

The hydraulic loading conditions that were used in the models were rather minor hydrologic events, and the water surface at its highest elevation barely reached the midpoint of the riverbank. It is likely that the factor of safety of the system before the cracking was decreasing over time, and the different drawdown events were enough to initiate movement of the levee. It is likely that a related series of events, UBLRP (i.e., 2012-2013 levee construction), multiple river drawdowns, and the high water levels in Lake Brown, contributed to the levee instability.

8 Remediation Alternatives

8.1 Introduction

Tetra Tech (2012) presented a memorandum (Appendix D, dated March 2011) of possible hydraulic improvements to the levee just downstream of the International Gateway Bridge. Remediation alternatives described in this chapter are based on the results of the ERDC study and evaluation of the Tetra Tech hydraulic alternatives.

The 2011 memorandum is presented in Appendix M of this report. The memorandum was concerned with improving the levee in a manner that would reduce the impacts of scour and erosion due to the bend of the river at the bridge. Scour and active bank slumping were observed in the 2014 bathymetric survey. The alternatives that were presented in the 2011 memorandum are listed below:

1. Riprap revetment of upper bank only
2. Riprap revetment of entire bank
3. Launchable rock
4. Sheetpile

All of the 2011 alternatives involved the placement of riprap on the bank for protection against scouring. The memo states that the recommended rock gradation of the riprap would be D_{100} of 9 in. and D_{50} of 6 in. and a thickness of 12 in.

The alternatives laid out in the 2011 memorandum were considered in this Chapter to understand the general benefits to the project. The same boundary and loading conditions to evaluate these alternatives as were used in the ERDC analysis, described earlier in the seepage and slope stability chapter. The section (Station 1900+13) defined in the ERDC study as critical was used for this evaluation.

8.2 Existing conditions

In order to quantify the possible effects of the alternatives, it was necessary to determine the existing conditions at the Brownsville Levee. An existing conditions analysis was conducted to understand the current factor of safety at the Brownsville Levee. This analysis was conducted using the bathymetric and LiDAR data collected by ERDC in September of 2014 to define the surface of the model. The stratigraphy was the same as that used in the stability models performed earlier in this study. Figure 8.1 shows how the surface changed from before to after the levee instability occurred. Major changes occurred in the channel, with material sloughing off the riverbank and into the channel.

Figure 8.1. Model surface data comparison before and after levee instability.

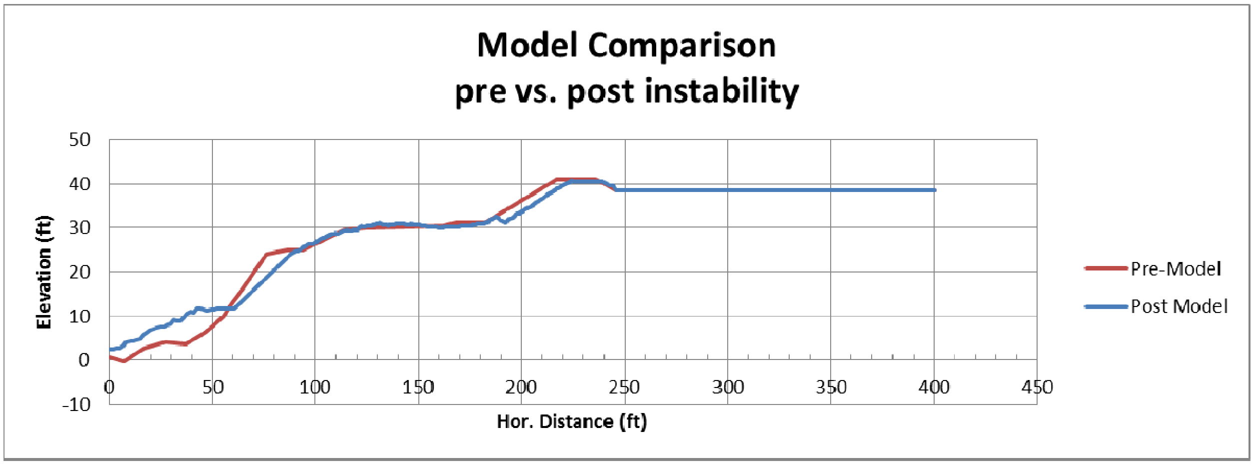


Figure 8.2 shows the model geometry used to evaluate the existing conditions. The difference between the surface shown in Figure 8.1 and that used in Figure 8.2 was any material that was considered to be sloughing material was left out of the analysis. This removal was done, because in a high water event, river velocities in this part of the channel would likely wash this material away.

Results of the analyses under the same loading conditions that were assumed for the triggering event to the instability were used for evaluating the remediation alternatives. The existing condition results are shown in Table 8.1.

Table 8.1 shows the factors of safety for the steady state analysis are relatively insensitive to the loading condition. The reason is that both analyses have the same type of failure surface, and there are a lot of slices that are well below the phreatic surface in both the high and low water evaluations. An important finding of this study is that the water elevation of Lake Brown is contributing to the low factors of safety in both evaluations. It is also important to note that the shear strength of the “soft ML” material is

Figure 8.2. Post-levee instability model.

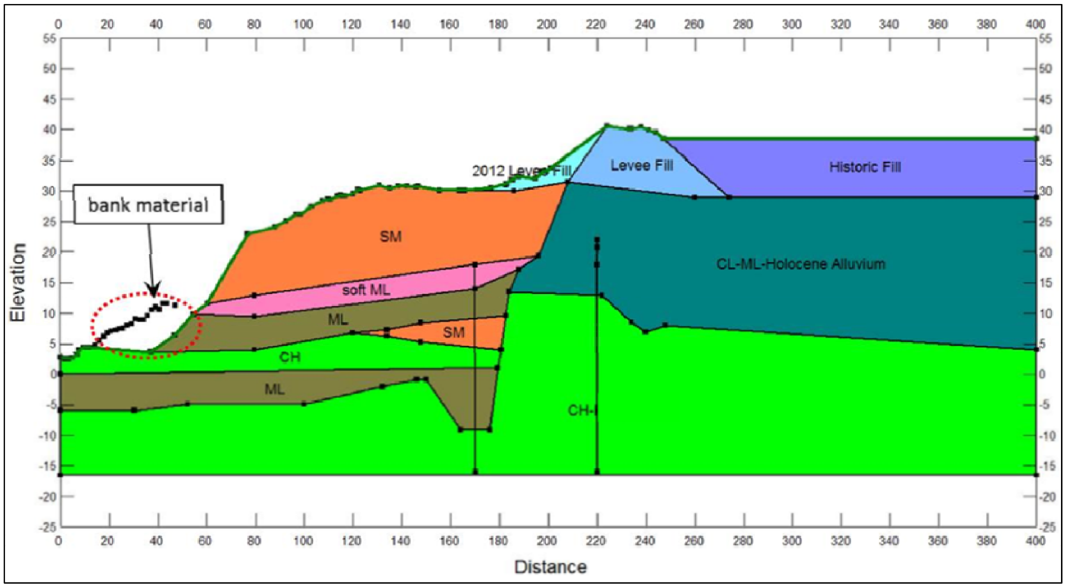


Table 8.1. Factors of safety for four different loading conditions using both the pre- and post-instability models.

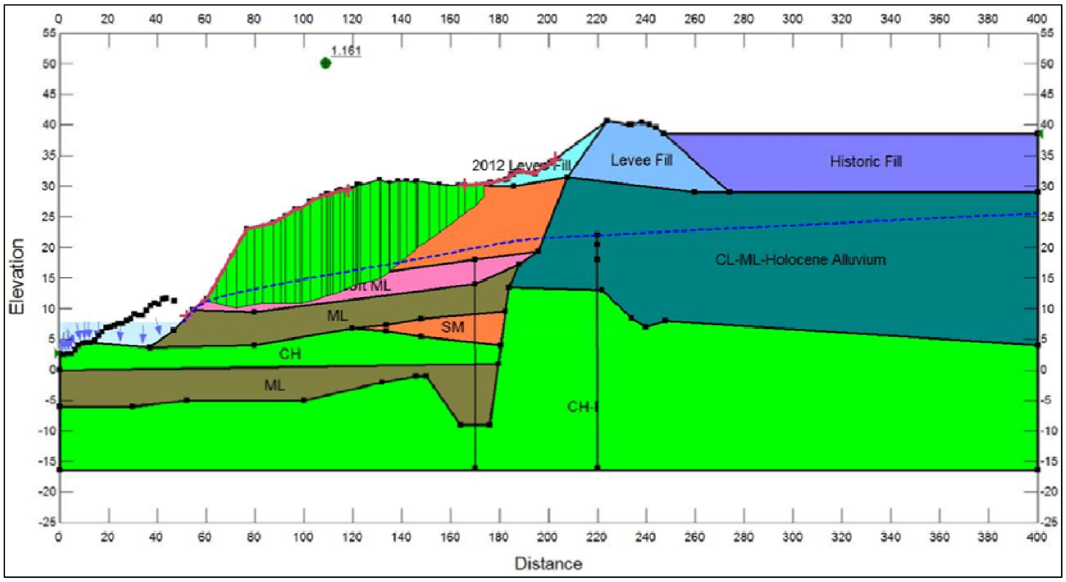
Analysis	Steady State Low (7.77 ft)	Steady State High (14.31 ft)	Rapid Drawdown (14.31 ft to 7.77 ft)	Hydrograph (transient analysis)
Pre-model	1.26	1.10	1.00	1.02
Post-model	1.16	1.17	1.00	1.15

modeled as undrained. If the soft material was allowed to drain or load slowly, the long-term shear strength would likely provide more resistance.

The results of the rapid drawdown analysis for the post-slope surface are about the same as before the instability, but there is a slight increase in factors of safety between the two hydrograph analyses. The case outlined in the numerical modeling section where the riverbank material is in an unstable condition was found during this analysis as well. The current unstable condition is due to the overly steep riverbank that presently exists, and because high river velocities are assumed to be washing away bank material.

Results of this type of analysis indicate the riverbank requires reinforcing so that the levee system would be brought to a stable state with an increased factor of safety. In addition to reinforcing the riverbank, in situ

Figure 8.3. Results of the steady state loading condition, river elevation 7.77 ft.



modification of the softer bank materials along the river could add additional stability improvements.

8.3 Potential remediation alternatives

The following engineering alternatives are based on the results of both the ERDC model analyses and data obtained during this study.

1. Regrade the bank to a 1H:5V slope with riprap protection from the edge of the access road to below the elevation of the softer material. This alternative is similar to Alternative III in the 2011 memorandum. The key to the success of this method is to fully reinforce the toe of the riverbank (long term, >5 yrs).
2. Install a sheetpile wall behind the existing riverbank to reinforce the soft alluvial sediments. Buried riprap in bank may make this alternative difficult to construct (long term, >5 yrs).
3. Improve the soil strength at the toe of the levee using soil mixing techniques by installing either a continuous wall or panels to improve the shear strength and rigidity of the foundation materials (long term, >5 yrs).
4. Monitor the existing "as is" condition during the short-term and maintain stable river elevations. If possible, avoid rapid drawdown events. Perform quarterly surveys and read inclinometers on a monthly basis for the next 10 to 12 months (short term, 2-5 yrs).

The alternatives are briefly summarized below with sketches showing the basic geometry as they were modeled.

8.3.1 Alternative I

Alternative I consists of modifying the geometry of the area between the toe of the levee and the river channel. It would include excavating material from the toe of the levee and regrading to make a 5H:1V slope. The intent here is to bring the riverbank to a stable configuration. Launchable riprap should be placed at the riverbank toe, similar to the procedure outlined in the 2011 Tetra Tech memorandum. Figure 8.4 shows the general configuration. Riprap placed at the regarded riverbank toe and slope will provide additional reinforcement and stability.

8.3.2 Alternative II

Alternative II consists of driving a sheetpile wall through the soft alluvial material to provide reinforcement. The target depth would be just into the Pleistocene material, near an elevation of -10 ft. This alternative would provide the needed support and rigidity that the system needs, but there may be potential issues with driving sheetpile due to riprap being encountered in the CPT and soil borings. Figure 8.5 shows the basic configuration of this alternative. A sensitivity analysis is recommended to understand the best location for the wall.

8.3.3 Alternative III

The third alternative is to improve shear resistance of the soft alluvial sediments located between the toe of the levee and the river. This alternative could be accomplished via soil mixing by installing either panels or a continuous wall to an elevation of 0 ft. This method may be more costly, but will improve the stability of the levee system. However, this alternative may not reduce scour at the toe of the levee. Figure 8.6 shows the configuration of the soil mixing wall. The modeling was performed assuming a cement bentonite mix would be used.

8.4 Alternative IV

Alternative IV consists of monitoring the levee over the short-term to select the most appropriate remediation strategy. This approach includes developing a monitoring plan for this alternative. Additionally, investigate a method to regulate the river stage at the study location, thus avoiding

Figure 8.4. Alternative I configuration.

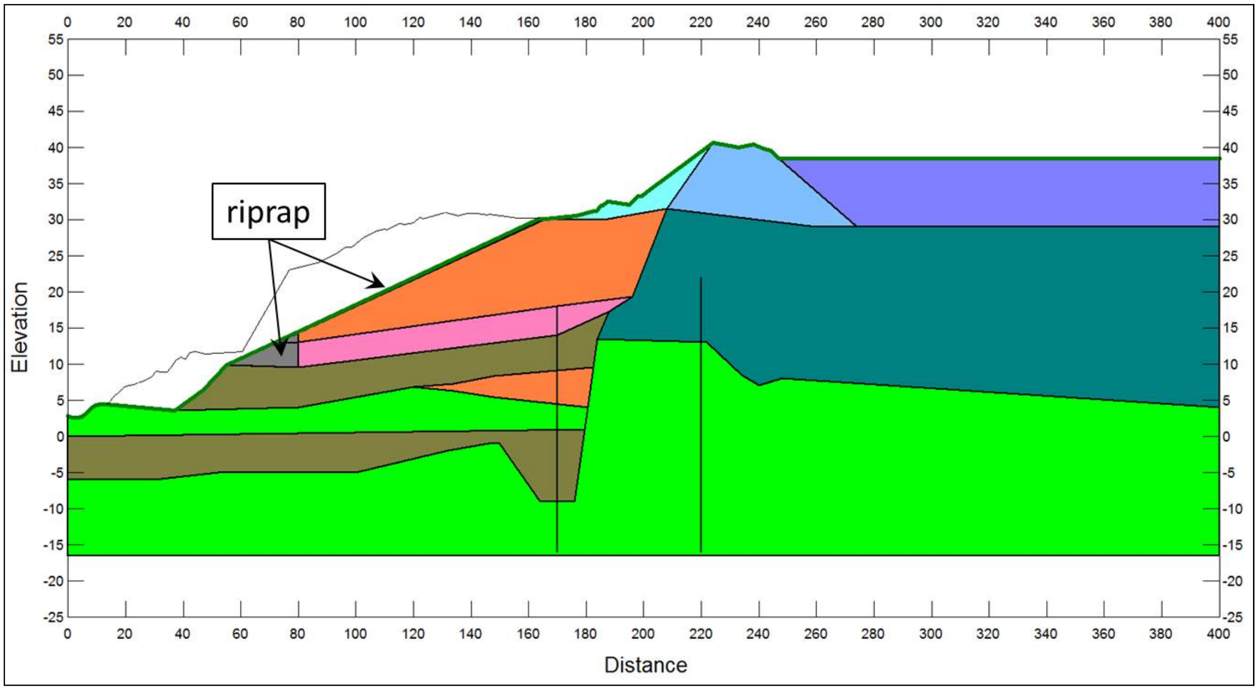


Figure 8.5. Alternative II configuration.

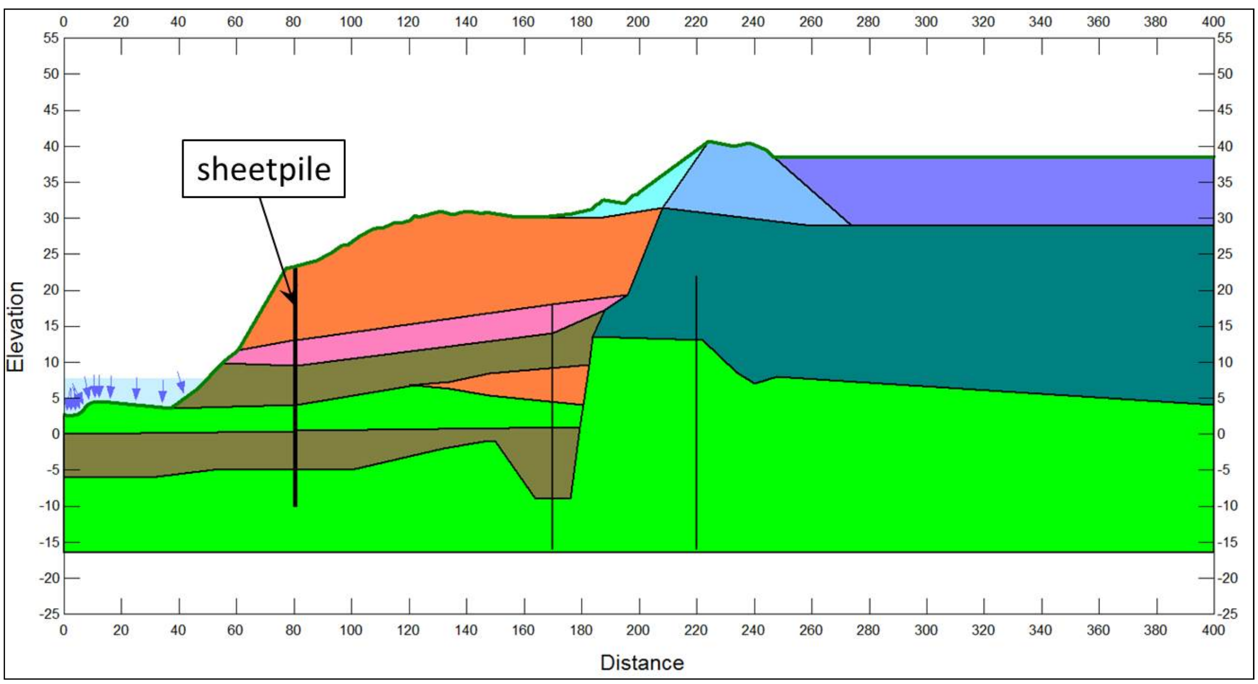
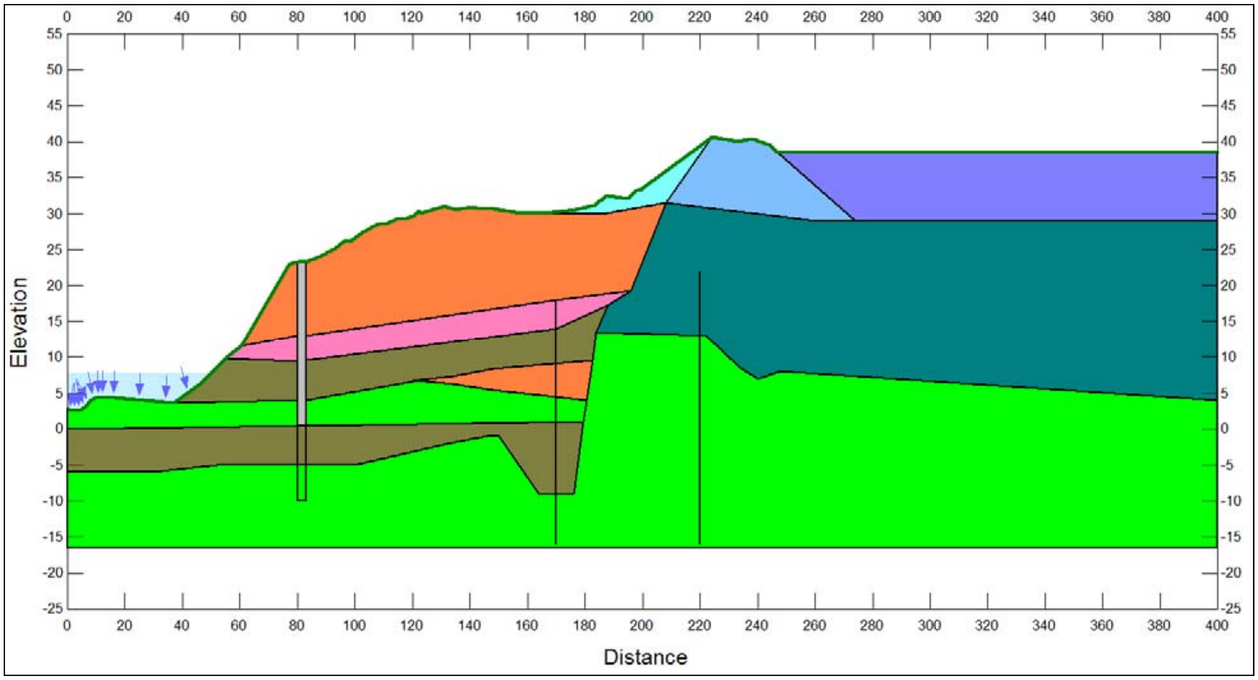


Figure 8.6. Alternative III configuration.



rapid drawdown situations. This alternative should incorporate a risk/benefit evaluation and may include an emergency action plan. The parapet wall at the POE provides additional flood protection, and this structure ensures a certain level of freeboard will be maintained.

This alternative involves reading inclinometers, monitoring wells, and performing additional surveys on a scheduled frequency. The inclinometers should be read at least quarterly to gather data to identify any trends. Additionally, perform both bathymetric and LIDAR surveys and compare these data to that collected in 2014. Further analysis of these data may provide insight into other possible system modifications that could be made to improve the flood protection system. A monitoring strategy in the short-term provides additional time to evaluate longer-term alternatives.

8.4.1 Results and discussion

Results of the alternatives analyses performed are shown in Table 8.2 with greatest increase in safety factor resulting from Alternative III followed closely by Alternative II. Alternative I showed some improvement and could be optimized by different slope and riprap configurations.

Table 8.2. Results of alternative analysis.

Analysis	Steady State Low (7.77 ft)	Steady State High (14.31 ft)	Rapid Drawdown (14.31 ft to 7.77 ft)	Hydrograph (transient analysis)
Pre-instability	1.26	1.10	1.00	1.02
Post-instability	1.16	1.17	1.00	1.15
Alt. I	1.24	1.22	1.20	1.56
Alt. II	1.62	1.55	1.34	1.64
Alt. III	1.56	1.67	2.19	2.32

Other alternatives could be considered and would reinforce the alluvial deposits (i.e., soil nails) and armor the riverbank against scour and erosion. At a minimum, Alternative IV should be adopted for short-term understanding until a long-term engineering solution is adopted.

Additional analysis, not part of this study, using a finite element (or difference) approach will be needed to fully understand the benefits of each alternative. This type of analysis would enable the calibration of both the observation well data as well as modulus data to observations made during the site investigation. The model could then be extrapolated to investigate these alternatives under a 100-yr hydraulic loading event. Factors of safety or displacements at critical locations could be compared for each alternative analysis in order to understand the possible benefits of each approach. This type of analysis would incorporate cost/benefit analysis for the different alternatives.

If Alternative I is selected, the factor of safety could be increased by varying the angle of the slope and the size of the riprap. Riprap should be sized such that it will withstand the expected velocities of the new channel profile. Due to the high water level in Lake Brown and the subsequent seepage through the foundation a filter (that meets current design standards) will need to be incorporated into the riprap design. The following list of manuals is included for reference to get the designer started but is by no means complete.

- EM 1110-2-1901, Engineering and Design Seepage Analysis and Control for Dams

- EM 1110-2-1913 Engineering and Design, Design and Construction of Levees
- EM 1110-2-1601 Engineering Design, Hydraulic Design of Flood Control Channels
- FEMA Filters for Embankment, Best Practices for Design and Construction

9 Conclusions and Recommendations

9.1 Conclusions

A series of unrelated events combined with the local geologic conditions led to the partial slope failure at the Brownsville, TX, levee. Events include the 2012 levee construction (i.e., UBLRP), fluctuation and rapid drawdown conditions in the Rio Grande, and higher elevation of Lake Brown relative to the river. The local geology consists of a soft soil that was not encountered in the widely-spaced geotechnical design borings drilled in 2009. Soft historic alluvial sediments were deposited less than 70 years ago and form the bank at the levee toe. These sediments are prone to be saturated and have low undrained shear strengths because of their depositional environment. The likely trigger for the partial slope failure was multiple rapid rise and rapid drawdown events beginning in early-April 2014. The factor of safety against this type of failure mode was very low.

Progressive or creep-type failure mode is the probable mechanism to explain the deformation observed in the field and was confirmed by seepage and stability analyses. The unstable nature of the riverbank sediments, combined with scour and erosion of the riverbank toe, contributed to the partial failure. Active slumping, as confirmed from field observations (i.e., bathymetry, visual inspection), is occurring along this river reach. These contributors may not be localized to the study alone; other areas along the river may be prone to similar type levee failure. Reaches with a similar geology and hydraulic setting are at risk for levee stability issues. Monitoring wells, water level data, and the ERDC stability analyses confirmed the impact of Lake Brown's water elevation on the stability of the levee system. Preliminary inclinometer data indicate that there is movement above the stiff Pleistocene surface and in the softer alluvial sediments. Results of the total station survey data indicate that surface movement was not occurring between August 2014 and October 2014. However, inclinometer data read in January 2015 indicates minor displacement in the subsurface.

9.2 Recommendations

9.2.1 Short-term recommendations (<5 years):

1. **Develop monitoring plan.** A monitoring plan that describes the procedures, schedule, and types of monitoring to be performed, as well as suggesting the organizations to collect measurements and/or perform the monitoring, would be prepared by ERDC and submitted to USIBWC for review and approval. This information gathered from the monitoring is needed to effectively plan, design, and budget for a permanent remediation strategy. The monitoring plan will include:
    a. **Visual inspection.** This inspection is especially important during periods where river stages are subject to wide fluctuations in stage from a large rainfall event and/or irrigation demands on the river. A record of inspections should be maintained to accurately note observations and any details.
    b. **Instrument monitoring.** The failure mechanism identified during this investigation involves a creep-type mechanism, which may not have attained stability. Although at the conclusion of this geotechnical investigation in February 2015 an immediate threat to the levee stability did not seem likely, it is important that the inclinometers and piezometers continue to be read to identify if movement is occurring. Measurements will be used to quantify the rate and magnitude of the deformation.
    c. **Elevation surveys.** Total station elevation measurements of the crest and slope are necessary to establish a baseline survey after the levee surface is regraded.
    d. **Assessment reports.** The results of the monitoring should be evaluated and reports prepared on a quarterly basis to provide an assessment of levee conditions. After each quarterly assessment, the monitoring plan should be reviewed and adjusted, if needed.
2. **Vegetation control.** A vegetation control program is necessary to provide a reliable inspection of both the bank and levee slopes.
3. **Regrading the levee profile.** Regrading the levee crest and toe to pre-failure conditions would permit a new baseline to be established in terms of the topographic profile. Regrading would also improve the aesthetic condition of the levee. During the regrading, caution should be taken to prevent disturbing the piezometers and inclinometers.

9.2.2 Long-term recommendations (>5 years):

1. Incorporate cost/benefit analyses for the different alternatives described in this report.
2. Perform additional analyses using the design hydrograph to fully assess the benefits of each remediation alternatives.
3. Remediation alternatives I-III should be coupled with an updated hydraulic analysis assessing the design flood.
4. Conduct LiDAR and side-scan sonar surveys if displacements are observed or measured during monitoring. These surveys would be coupled with the elevation surveys.

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Appendix A: Scope of Work

DEPARTMENT OF THE ARMY
ENGINEER RESEARCH AND DEVELOPMENT CENTER, CORPS OF ENGINEERS
GEOTECHNICAL AND STRUCTURES LABORATORY
WATERWAYS EXPERIMENT STATION, 3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

REPLY TO
ATTENTION OF:

16 June 2014

Office of Technical Directors

SUBJECT: Request for Proposal (RFP) IBM14-15, for Brownsville Levee Geotechnical Investigation, dated 15 May 2014

José A. Nuñez, P.E.
Principal Engineer
International Boundary and Water Commission, United States Section
4171 North Mesa, Suite C-310
El Paso, TX 79902-1441

Dear Mr. Nuñez:

Reference the attached scope-of-work (SOW) for the analysis of the Brownsville Levee Geotechnical Investigation. In response to your RFP, we are submitting a preliminary cost estimate to perform Tasks 1 and 2 of the SOW. Tasks 1 involves the site visit and inspection, and Task 2 involves the development of a detailed site investigation plan for performing the geotechnical study to determine the severity, extent, and remediation of the levee failure at the Brownsville, Texas.

Table 1 is an itemized cost estimate for ERDC to perform only tasks 1 and 2. The total cost for performing this work is estimated at $43,025. If you have questions about this estimate, please contact Dr. Maureen K. Corcoran at 601-634-3334, or Dr. Joseph B. Dunbar at 601-634-3315.

Sincerely,

Maureen K. Corcoran

Maureen K. Corcoran, PhD
Associate Technical Director

Encl:
Scope of Work
Cost estimate

DEPARTMENT OF THE ARMY
ENGINEER RESEARCH AND DEVELOPMENT CENTER, CORPS OF ENGINEERS
GEOTECHNICAL AND STRUCTURES LABORATORY
WATERWAYS EXPERIMENT STATION, 3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

16 June 2014

REPLY TO
ATTENTION OF:

Geosciences and Structures Division

SUBJECT: Request for Proposal (RFP) IBM14-15, for Brownsville Levee Geotechnical Investigation, dated 15 May 2014

Mr. Frank Delgado
International Boundary and Water Commission, United States Section
4171 North Mesa, Suite C-310
El Paso, TX 79902-1441

Dear Mr. Delgado:

Reference the attached scope-of-work (SOW) for the analysis of the Brownsville Levee Geotechnical Investigation. In response to your RFP, we are submitting a preliminary cost estimate to perform Tasks 1 and 2 of the SOW. Tasks 1 involves the site visit and inspection, and Task 2 involves the development of a detailed site investigation plan for performing the geotechnical study to determine the severity, extent, and remediation of the levee failure at the Brownsville, Texas.

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Sincerely,

Maureen K. Corcoran

Maureen Corcoran, PhD
Associate Technical Director

Encl:
Scope of Work
Cost estimate

Preliminary Scope of Work

Geotechnical Investigation of Brownsville Levee Failure, Brownsville, TX

Background

Levee cracking has occurred along an 800 ft reach of the north (left) bank levee of the Rio Grande River at Brownsville, TX, between stations 1898+00 and 1904+85 following a rapid drawdown of the river in late-March 2014. The levees are under the jurisdiction of the U.S. Section of the International Boundary and Water Commission (USIBWC), headquartered in El Paso, Texas. The USIBWC has requested a scope of work from the U.S. Engineer Research and Development Center (ERDC), Geotechnical and Structures Laboratory (GSL), Geotechnical Engineering and Geosciences Branch (GEGB), to conduct a geotechnical investigation of the levee reach that has displayed signs of cracking described as a slope failure in the USIBWC request for proposal (RFP), *Geotechnical Investigation Services to Determine the Cause of an Embankment Failure, USIBWC Upper Brownsville Rehabilitation Levee, Lower Rio Grande Flood Control Project, Cameron, Texas*. The RFP was sent to ERDC by the USIBWC that describes requirements to perform a geotechnical investigation on the cause of the cracking. The RFP is attached to this proposal as enclosure 1 and is the basis for discussion of the technical items that will be described in the geotechnical investigation plan presented herein.

Purpose and Scope

The proposed work will provide geotechnical services to address the underlying causes of the levee cracking at Brownsville, TX, between stations 1898+00 and 1904+85. Because of the uncertain nature of the site conditions that will be encountered in the field and the nature of the pre-existing data that are available to characterize the site, the following study will be conducted in phases to obtain the necessary information for the subsequent analysis needed to identify the underlying mechanisms producing the cracking and provide remediation options.

Description of Major Tasks

Multiple tasks will be performed to characterize the levee reach, evaluate the data collected, and report upon the study findings. These tasks are described in more detail as follows:

Task 1. Initial Site Visit. This task will involve an initial site visit by the ERDC geotechnical staff to determine the site conditions, the extent of the cracking, and discuss the data that is available in the USIBWC project files (note: digital files were provided by USIBWC on a ftp site). The initial visit is the basis for Task 1 and will involve a site visit by two geotechnical engineers and a geologist. The geotechnical staff will consist of professional engineers (PE) and a registered professional geologist (RPG).

Task 2. Preparation of Site Investigation Plan. Information gathered from this visit will form the basis for the preparation of the detailed Site Investigation Plan. This plan will specify the locations of cone-penetrometer test (CPT) borings and conventional borings to obtain soil samples from the levee and foundation in the levee reach where cracking is evident. The site investigation plan will incorporate several levels of information to characterize the levee conditions and geometry (i.e., embankment soils, their engineering properties, foundation geology, foundation soil types, engineering properties of the foundation, extent of cracking, and vertical extent into the subsurface. The latter being determined from a combination of backhoe trenches, CPTs, and/or borings.

Another important component of the detailed study plan will include the proposed use of waterborne geophysics to obtain high resolution bathymetry and images of the levee slope and toe in the failure area to determine surface and subsurface channel geometry, bathymetry, and/or bed forms to accurately model the slope and determine conditions above and beneath the water surface. Additionally, fixed survey profiles of the levee geometry and slope will be required to accurately model any movements through the course of the investigation.

A requirement for the site investigation plan is the requirement for complying with the state and federal environmental regulations (see attached RFP, section C, Specific Work Requirements, Task 1e). The primary activity that will cause soil disturbance during this investigation is the need for drilling and sampling of floodplain and levee soils and backhoe trenches. ERDC will operate under the jurisdiction of the USIBWC and their environmental governances for levee maintenance activities. The USIBWC has jurisdiction over the international boundary and the floodplain easement between the levees. The ERDC team will coordinate with the USIBWC environmental officer for drilling and sampling activities to ensure environmental compliance.

Task 3. Field Data Collection. The information contained in the site plan will be used to address the fundamental engineering properties and geologic conditions within the failure reach that are described in the enclosed RFP, Task 2, Final Site Investigation Report, specifically items 2a through 2h. It is envisioned that the data needed to address items 2a through 2h would be an iterative process in that CPT and borings would likely be performed in separate phases.

Task 4. Laboratory soil testing. This task would involve laboratory soil testing. More than one soil testing laboratory will be used and would be a combination of USACE-approved soil laboratories and/or ERDC soil testing laboratory. A goal for using more than one soil laboratory would be to minimize transport and disturbance of undisturbed samples and provide QA/QC of the laboratory data.

Task 5. Analysis of engineering and geologic data. Field and laboratory data collected from elevation surveys, CPTs, and borings will be used to characterize the subsurface stratigraphy, engineering soil properties, develop geologic cross-sections, and develop models for slope stability analysis. These data will address item 2j in the enclosed RFP. The requirement for equally- spaced profiles at 50 ft intervals containing geologic information would typically involve a boring or CPT at the levee crest, levee toe, and midway on the floodplain bench to provide stratigraphic details along the levee reach, which spans approximately 800 ft of the Rio Grande. This requirement would require a minimum of 48 borings and CPTs be drilled (800 ft /50ft x 3 = 48) along a fixed spacing plus the additional borings or CPTs needed for identifying anomalous features of interest. Furthermore, there is need to drill borings outside of the failure area to compare conditions in reaches that have not failed, which would add to the number of borings outside of the minimum specified by the RFP. It is recommended that spacing be conducted at 100 ft spacing initially, and a determination will be made for the requirement for the CPTs and borings to be spaced at a 50 ft interval. This minimum 50 ft spacing may be in excess of what will be required to make a determination of the factors responsible. The precise number and spacing of borings can be determined by a cost assessment in Task 2 and coordination with technical staff at the USIBWC. It is anticipated that borings and CPT would be obtained in a staged approach to fully complete requirements described in Task 3 and presented in Task 5.

Task 6. Slope stability modeling. – Slope stability modeling of the embankment reach will be made using the results of the geotechnical data collected in Tasks 3 through 5 to determine specific mechanisms leading to cracking that match site conditions and surface geometry. Stability modeling will incorporate data from the rehabilitation project documentation that is described in Section H, Information Provided by USIBWC, of the attached RFP. Two-dimensional (2D) slope stability modeling will be performed using standard geotechnical engineering software (i.e., Slope/W), and incorporating several representative profiles in the distressed area. The report of study will model and evaluate at least three engineering alternatives for remediation.

Task 7. This task involves compiling the data into a technical report and reporting on the study findings. It is anticipated that the geotechnical team would presented findings and results at the USIBWC office in El Paso, TX. The report of findings will present at least three engineering alternatives for remediation of the failure reach.

Project Technical Personnel

Technical personnel involved in this investigation will be senior level staff that will be registered in their respective field and have the necessary experience in conducting geotechnical investigations. A list of the senior level technical staff is presented below. Supporting the investigation will be other engineers, geologists, and support staff that have between 4 and 10 years of professional experience and completed course work in graduate studies.

Name	**Position**	**E-mail**	**Office phone**	**Cell phone**
Dr. Joe Dunbar	Senior Geologist	Joseph.B.Dunbar@usace.army.mil	601-634-3315	601-529-3315
Don Yule	Senior Geotechnical Engineer	Don.E.Yule@usace.army.mil	601-634-2964	601-529-9653
Isaac Stephens	Geotechnical Engineer	Isaac.J.Stephens@usace.army.mil	601-634-3610	

Schedule/Budget

Task	**Date(s)**	**Budget**	**Deliverable**
Task 1. Initial Site Visit	23-26 June	$25,525	Trip report of initial findings; basis to develop

			site preparation plan
Task 2. Preparation of Site Investigation Plan	10 July	$17,500	Detailed plan on conducting site investigation
TOTAL Task 1 and Task 2		**$43,025**	
Task 3. Field Data Collection	Dependent on previous tasks	Dependent on previous tasks	
Task 4. Laboratory soil testing.	Dependent on previous tasks	Dependent on previous tasks	
Task 5. Analysis of engineering and geologic data	Dependent on previous tasks	Dependent on previous tasks	
Task 6. Slope stability modeling	Dependent on previous tasks	Dependent on previous tasks	
Task 7. Preparation of the report	Discussion with IBWC		

Operating Environment and Safety

Personnel conducting drilling in the levee right-of-way will be operating in an environment that may be troublesome with cross-border vandalism. Thus, drilling equipment will be moved daily from the work site to ensure safety of the equipment against potential vandalism. The ERDC support team will require close coordinating with USIBWC and the Department of Homeland Security (DHS) personnel to allow close-by storage of the equipment, as well as communication to ensure the study activity will be monitored and allowed to proceed and not adversely impact the mission.

Scope of Work
Geotechnical Investigation Services to Determine the Cause of an Embankment Failure, USIBWC Upper Brownsville Rehabilitation Levee, Lower Rio Grande Flood Control Project Cameron County, Texas

Table of Contents

A. General Project Description:

1. The newly refurbished United States Section of the International Boundary and Water Commission (USIBWC) levee section that was part of the Upper Brownsville Levee Rehabilitation Project and adjacent floodplain, is encountering cracking and slope failure after a significant drop of the Rio Grande water level in the month of March 2014. This area has experienced moderate dry climate conditions for a number of decades, and a subsurface investigation is required in order to evaluate the existing conditions and provide remediation solutions to keep the area from further slope failures.

2. The work shall be on the USIBWC levee and floodplain, located in Brownsville Texas, near Station 1890+00 to Station 1908+00, east of the International Gateway Bridge located in Brownsville, Texas, as indicated on the provided plan set for Contract IBM13C0001 - UBL. This location will be shown by the Contracting Officer's Representative (COR) upon arrival. The exact location of work will be determined by the Contractor as they investigate the full extent of the failure area.

3. The Contractor shall provide all equipment and personnel (qualified and licensed) necessary to perform a Geotechnical Study of the affected area. The Contractor will be required to submit a Geotechnical Report which shall summarize the root causes of the slope failure and will also include three (3) options on how to stop the failure and how to remediate the failure area. The report will be signed and sealed by a Professional Engineer and shall consist of the components defined under Section C of this Scope of Work.

4. The Contractor shall identify any structures that may be affected by any continued slope failure and the remediation options.

(1) The work includes geotechnical engineering services to investigate the root cause failure of the slope to include:

i. The full extent of the failure area

ii. The soil stratification within the existing failure arca

iii. Conclusions of the investigation in a written report

iv. A minimum of three (3) recommendations and options for repair/reconstruction of the affected area

v. Identification of any structures that may be affected by any continued slope failure and the remediation options

(2) This work will involve:

i. Perform an initial site visit to determine the required subject matter experts, labor, and equipment required to perform a complete and comprehensive site investigation.

ii. Perform site investigation to include, if necessary: boring of test holes, geotechnical soil testing, site surveying, and/or any other items the Contractor deems necessary to perform a full investigation.

iii. The Contractor shall provide a complete and comprehensive written report which details the findings of the site investigation. The report shall include all results and data collected. The report shall also include at least three recommendations for a permanent repair of the affected levee area.

5. **The performance period is 120 calendar days. The amount of performance period days are also considered a negotiable item at time of the Contractors bid submission.**

B. Project Background:

1. On March 29, 2014, the USIBWC discovered levee cracks between Station 1898+00 and Station 1904+85 on the Upper Brownsville Levee Rehabilitation Project. In several of the reports received about this slope failure, it was noted that the river level had recently dropped several feet. This river drawdown condition is assumed to be the most likely trigger of the slope failure, since existing boreholes indicate that fluvial depositional environmental created layers of lean clay, fat clay, and sand varying from about four (4) feet in depth to over twenty five (25) feet in this area.

2. The construction in this area under Contract IBM13C0001, Upper Brownsville Levee Rehabilitation, was completed in October 2013. Tetra Tech, Inc. (Tetra) was the design firm for this project which provided the following deliverables to the USIBWC: Design Report, Geotechnical Report, Construction Plans, and Construction Specifications. Tetra Tech hired Raba-Kistner Consultants, Inc. (Raba) to perform a geotechnical analysis of the site for their design, which was required to meet FEMA levee certification requirements.. A geotechnical report entitled *Geotechnical Addendum - Subreach 4, Lower Rio Grande Flood Control Project Levee System - From Donna Pump to Brownsville Levee Reach* dated June 1, 2011 was prepared by Raba. Additionally, Tetra prepared a Final Design Report for Upper Brownsville Levee Rehabilitation in May 2012.

C. Specific Work Requirements

Task 1 Embankment Failure Site Investigation Plan

a. The Contractor shall provide the services of a qualified geotechnical "expert" to provide a detailed plan for site investigation consisting of borings, subsurface soil sample logs and identification, slope stability analysis, and any other services which may be required to determine the failure mechanism.

b. The Contractor shall coordinate all fieldwork with the USIBWC office, as required. All fieldwork shall be performed in accordance with local, state, and federal laws. The Contractor shall submit their Site Investigation Plan to the COR for review and compliance confirmation. The material to be submitted shall include, but not be limited to, the following:

(1) The Contractor shall provide in detail the scope of the soil investigation including the number and types of borings or soundings, the equipment used to drill and sample, the in situ testing equipment, and the laboratory testing program. The investigation program shall be determined by a registered design professional and shall be included in the Site Investigation Plan.

(2) The Contractor shall include an aerial map depicting the number and spacing of borings to be taken. The Contractor shall also include a proposed plan to seal boring holes once investigation is completed.

(3) The Contractor shall include general equipment and procedures that will be used throughout the geotechnical investigation.

(4) Additional studies shall be detailed under the Site Investigation Plan as necessary to evaluate slope stability, soil strength, position and adequacy of load-bearing soils, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction, and expansiveness.

c. The soil boring and sampling procedure and apparatus shall be described in the Site Investigation Plan. These items shall be in accordance with generally accepted engineering practice. The registered design professional shall have a fully qualified representative on the site during all boring and sampling operations. The qualified representative must have a minimum of five (5) years' experience with operating the proposed apparatus and shall possess all licensing certificates to operate said apparatus.

d. The Contractor's process regarding soil classification shall be listed under their Site Investigation Plan. The soil classification shall be based on observation and any necessary tests of the materials disclosed by borings, test pits, or other subsurface exploration made in appropriate locations.

e. When developing the Site Investigation Plan, the Contractor is responsible for complying with the Texas Commission on Environmental Quality, the Environmental Protection Agency Region 6 Office in Dallas for any National Pollutant Discharge Elimination System (NPDES) Construction General Permit and Storm Water Pollution Prevention Plan. If permit is not required by State or Federal agency with jurisdiction on federal sites, Contractor shall provide written documentation referencing reason for waiver.

f. When developing the Site Investigation Plan, the Contractor shall include a Spill Prevention Plan for all equipment and materials to be used onsite and in any associated staging areas.

Task 2 Final Site Investigation Report

The final site investigation report shall include the following information:

a. A plot showing the location of test borings and/or excavations referenced from existing benchmarks. Boring locations shall be surveyed in the field following completion of drilling activities.

b. A complete record of the soil samples taken. All samples shall be classified and recorded using standard reporting procedures. A summary test data sheet shall be included in the Final Site Investigation Report.

c. A record of the soil profiles and layers encountered.

d. Identify and evaluate the existing soil composition and strength parameters of the current levee and underlying strata.

e. Evaluate the relevant engineering properties of the sampled collected and create boring logs for representation of in situ soils.

f. Calculate strength parameters of the existing soil and underlying strata to be used in structural evaluation.

g. Characterize engineering properties of the geology, top stratum, substratum, and groundwater conditions.

h. Elevation of the water table, if encountered.

i. Recommendations for soils remediation and design criteria, including but not limited to: bearing capacity of soils; provisions to mitigate the existing soils; mitigation of the effects of slope failure and varying soil strength; and the effects of adjacent loads.

j. Cross sections located perpendicular to the Rio Grande at a maximum of fifty (50) foot intervals extending from the river bank up to the levee landside toe, at a minimum. Cross sections shall show stratigraphy (including top stratum and substratum thickness at specific points beneath the levee), USCS soil types, and their horizontal and vertical distribution and relationships used in structural evaluation within any identified potential problem areas.

k. The Contractor shall submit, in the final site investigation report, any and all final drawings demonstrating the determined affected failure area and the recommended repair area limits. These drawings shall be detailed with coordinate system used in the provided construction plans as described under Section H of this Scope of Work.

l. Identification of any structures that may be affected by any continued slope failure and/or the remediation options presented by the Contractor.

m. At least three (3) remediation options ranging from least complex to most complex regarding technical viability shall be presented. Conceptual drawings shall be prepared showing the work extent and work components.

n. **It is the Contractors responsibility to provide any additional information that is required to address the slope failure and the three (3) remediation options the Contractor proposes as part**

of their Final Site Investigation Report. The items listed under Section C Task 2 of this Scope of work are only USIBWC minimum report content recommendations.

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Task 3 Personnel Requirements

a. The Contractor shall provide a complete listing of the project team inclusive of individual resumes and qualifications. After award of the this Contract, the Contractor shall not remove or exchange personnel listed on the Contractors Proposal without written approval from the Contracting Officer. In the event that this does occur, only replacements that match or exceed the current team member's qualifications will be considered for replacement by the government.

b. The Contractor shall identify key personnel to be used on this project and their areas of responsibility. Explain how the proposed personnel along with the Contractor's work plan will meet the requirements of this project.

c. The qualifications and experience of the selected Geotechnical "Expert.

(1) The Geotechnical Expert shall have a minimum of ten (10) years of proven experience in the implementation of equivalent required services.

(2) The Geotechnical Expert shall be a Licensed Professional Engineer.

d. The Contractor shall provide and be responsible for all equipment and items required for personnel to perform this Contract. At a minimum, field personnel are expected to have:

(1) Computer and necessary software.

(2) Vehicle appropriate for the site conditions with appropriate safety equipment.

(3) Personal protective equipment as well as inspection and measurement items. Minimum personal protective equipment is hard hat, safety vest, hearing protection, steel toed boots, and safety glasses. Hard hats shall have the name of the consulting firm visibly displayed.

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b. The Contractor is responsible for verification of all dimensions and existing site conditions.

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d. If work is conducted during the Migratory Bird Treaty Act (MBTA) bird breeding season of March 1 through August 31, bird nesting surveys will be required of the project area prior to starting the Site Investigation. Bird nesting surveys will be required once every seven calendar days to ensure compliance with the MBTA.

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1. USIBWC compliance confirmation is required for submittals. Submittals not receiving compliance confirmation must be resubmitted to the Government for approval.

2. The COR shall have a maximum of fourteen (14) days to review and provide responses to all submittals required prior to the start of site work.

3. The COR shall have a maximum of twenty one (21) days to review and provide responses to all other submittals.

4. The Contractor shall submit thc following prior to the start of site work:
   a. Progress Schedule including Site Investigation Time Line and Final Deliverables Time Line.
   b. Site Investigation Crew Organization Chart and Resumes.
   c. Storm Water Pollution Prevention Plan (SWPPP) per NPDES permit requirements
   d. Spill Prevention Plan
   e. Site Investigation Plan.
   f. Utility Locate Report.
   g. Entry Authorization List (EAL).
   h. Materials for backfilling of bore holes.

5. Other submittals required under this Contract:
   a. Preliminary Slope Failure Extent Drawings (90% Complete)
   b. Final Slope Failure Extent Drawings
   c. Conceptual Drawings for Site Remediation (90% Complete).
   d. Final Conceptual Drawings for Site Remediation.
   e. Site Investigation Report (90% Complete).
   f. Final Investigation Report (to include items listed under C.Task 2 of this Scope).

E. Occupancy of Premises / Access

The levee area near the embankment failure will not be occupied during performance of work under this Contract, except by the US Customs Border Protection Agents in the event of criminal activities. Before work is started, the Contractor shall arrange with the COR a sequence of procedure, means of access, space for storage of materials and equipment.

Task 1 Security Requirements

a. Access to the USIBWC levee area near the Gate Way International Bridge is controlled by the USIBWC. All contracted personnel entering the sites shall be on an Entry Authorization List (EAL) The Contractor shall provide complete written, valid, and legible data that shall include lcgible photocopies or scanned electronic documents to be used to produce the initial EAL prior to their initial commencement of work at the site for this project.

b. If existing access to the site is to be temporarily blocked, temporary access shall be properly provided by the Contractor. The Contractor shall notify the COR two (2) calendar days prior to any interruption of access to the sites. Date, site(s) affected, length of time, and alternate entry method for Site Interruption Plan shall be submitted in writing for approval.

c. The work is located completely within the United States, but is directly adjacent to the international border with Mexico. Security is a major concern adjacent to the international boundary. The Contractor is responsible for securing the work site, equipment, and materials from vandalism and theft.

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a. Company Identification (logo) must be clearly, legibly, and identifiable at a minimum of thirty (30) foot distance and displayed on each side of all vehicles and equipment brought onto or operated on site. Vehicles and equipment without such identification may be denied access to the site, and maybe subject to being stopped by the U.S. Customs and Border Protection.

b. The access road leading from the main road adjacent to the site is owned by the City of Brownsville, Texas. The area off to the sides of the access road is either private property, federal, state, or county property. Parking vehicles on the access road is allowed with permission that has been obtained from the USIBWC.

c. Authorized Contractor vehicles and equipment will be placed so as not to interfere with gates and emergency escape routes.

F. Dig Permits

1. No excavation will begin without first conducting a Utility Locate Report. This shall be accomplished via Texas 811 or through a private utility locate company.

2. The Contractor is also responsible for contacting the USIBWC Operations and Maintenance Office in Mercedes Texas (USIBWC O&M) to inquire about any buried cables/structures within the construction area. Items encountered and damaged within three (3) feet on either side of a marked line or around a marked point of items shall be repaired by the Contractor. The Contractor is to confirm with USIBWC O&M within (2) calendar days prior to any excavation regarding any possible utilities within this area (USIBWC O&M Office POC Joel Saldivar, 915-832-4777).

G. Deliverables

a. The schedule of deliverables the Contractor shall submit to the project COR includes, but is not limited to:

	Item	No. of Copies
i.	Drawings of Full Extent of Slope Failure (90%, Final Drawings)	5
ii.	Conceptual Drawings for Proposed Floodplain Failure Remediation for each Remediation Recommendation (90%, Final Drawings)	5

iii. Site Investigation Report (90%, and Final Report) 5

b. Unless otherwise noted, the number of copies specified above refers to hard copies for 90% and final submittals.

c. One set of the hard copies specified above shall be delivered directly to the Mercedes Field Office. All drawing deliverables to the Mercedes Field Office shall be 24 inch x 36 inch in size (ANSI D).

d. Drawing deliverables to El Paso Headquarters Office shall be 11 inch x 17 inch (ANSI B) and shall be printed at true half scale.

e. In addition, the Contractor shall submit four (4) electronic copies for both 90% and for final submittals USIBWC's Headquarters Office in El Paso, TX. Electronic copies shall be provided on CD, DVD, or USB drives.

f. All written reports shall be printed on paper containing 30% post-consumer fiber (30 PC). All deliverables shall also be furnished in electronic format. Electronic format of the report shall be in Portable Document Format (pdf) and Microsoft Word 2007, while electronic format for all drawings shall be in pdf and in AutoCAD/AutoCAD Civil 3D 2012.

g. The Final Submittals shall include the Contractor's written response to all USIBWC comments generated during the review of all the 90% deliverables. In addition, the Contractor shall provide marked up copies of the 90% deliverables (all deliverables that required revisions) showing all changes made on the 90% after USIBWC comments. A meeting between the USIBWC and the Contractor shall be conducted after the Contractor receives and reviews USIBWC's comments on the 90% deliverables, if concurrence is not reached on comments.

h. Only after acceptance of the 90% responses by the USIBWC shall the Contractor provide final submittals.

H. <u>Information Provided by USIBWC</u>

1. The USIBWC shall provide the following existing project documents to the Contractor in digital format:

   (1) Tetra Design Report Titled: *"Upper Brownsville Levee Rehabilitation, Cameron Counties, Texas, Design Report, Final Design Submittal"* May 2012, By Tetra Tech Inc.

   (2) Tetra Construction Drawings Titled: *"Upper Brownsville Levee Rehabilitation, Cameron County, Texas, Conformed Project Drawings"* June 2012, By Tetra Tech Inc.

   (3) Tetra Construction Specifications Titled: *"Technical Specifications for Lower Rio Grande Flood Control Upper Brownsville Levee Rehabilitation"* June 21, 2012, By Tetra Tech Inc.

   (4) Tetra Geotechnical Report Titled: *"Geotechnical Exploration and Engineering Evaluation of Levee System for the Lower Rio Grande Flood Control Project from Cameron County Line of Donna Pump to Brownsville Levee Reach to its Eastmost Limit, Final Technical Memorandum"* July 24, 2009, By Raba-Kistner Consultants, Inc.

   (5) Tetra Geotechnical Report Titled: *"Geotechnical Addendum – Subreach Four for the Lower Rio Grande Flood Control Project Levee System-from Donna Pump to Brownsville Levee Reach, Hidalgo County and Cameron County, Texas"* June 1, 2011, By Raba-Kistner Consultants, Inc.

(6) Area Environmental Assessment Titled: *"Final Environmental Assessment, Improvements to the Donna-Brownsville Levee System, September, 2007."* September 2007, By USIBWC.

(7) Photo Documentation of the Area spanning from March 29, 2014 to present day.

2. It is the Contractor's responsibility to print all items provided in electronic format. All of the paper documents provided to the Contractor are/shall remain property of the USIBWC and shall be returned at the end of the project.

3. Information provided by the USIBWC in the form of reports or data cannot be used for work outside of the current SOW without written consent of the USIBWC.

END OF SCOPE OF WORK

Scope of Work
Geotechnical Investigation Services to Determine the Cause of an Embankment Failure,
USIBWC Upper Brownsville Rehabilitation Levee,
Lower Rio Grande Flood Control Project
Cameron County, Texas

Table of Contents

A. General Project Description:

1. The newly refurbished United States Section of the International Boundary and Water Commission (USIBWC) levee section that was part of the Upper Brownsville Levee Rehabilitation Project and adjacent floodplain, is encountering cracking and slope failure after a significant drop of the Rio Grande water level in the month of March 2014. This area has experienced moderate dry climate conditions for a number of decades, and a subsurface investigation is required in order to evaluate the existing conditions and provide remediation solutions to keep the area from further slope failures.

2. The work shall be on the USIBWC levee and floodplain, located in Brownsville Texas, near Station 1890+00 to Station 1908+00, east of the International Gateway Bridge located in Brownsville, Texas, as indicated on the provided plan set for Contract IBM13C0001 - UBL. This location will be shown by the Contracting Officer's Representative (COR) upon arrival. The exact location of work will be determined by the Contractor as they investigate the full extent of the failure area.

3. The Contractor shall provide all equipment and personnel (qualified and licensed) necessary to perform a Geotechnical Study of the affected area. The Contractor will be required to submit a Geotechnical Report which shall summarize the root causes of the slope failure and will also include three (3) options on how to stop the failure and how to remediate the failure area. The report will be signed and sealed by a Professional Engineer and shall consist of the components defined under Section C of this Scope of Work.

4. The Contractor shall identify any structures that may be affected by any continued slope failure and the remediation options.

May 15, 2014

(1) The work includes geotechnical engineering services to investigate the root cause failure of the slope to include:

i. The full extent of the failure area

ii. The soil stratification within the existing failure arca

iii. Conclusions of the investigation in a written report

iv. A minimum of three (3) recommendations and options for repair/reconstruction of the affected area

v. Identification of any structures that may be affected by any continued slope failure and the remediation options

(2) This work will involve:

i. Perform an initial site visit to determine the required subject matter experts, labor, and equipment required to perform a complete and comprehensive site investigation.

ii. Perform site investigation to include, if necessary: boring of test holes, geotechnical soil testing, site surveying, and/or any other items the Contractor deems necessary to perform a full investigation.

iii. The Contractor shall provide a complete and comprehensive written report which details the findings of the site investigation. The report shall include all results and data collected. The report shall also include at least three recommendations for a permanent repair of the affected levee area.

5. **The performance period is 120 calendar days. The amount of performance period days are also considered a negotiable item at time of the Contractors bid submission.**

B. <u>Project Background:</u>

1. On March 29, 2014, the USIBWC discovered levee cracks between Station 1898+00 and Station 1904+85 on the Upper Brownsville Levee Rehabilitation Project. In several of the reports received about this slope failure, it was noted that the river level had recently dropped several feet. This river drawdown condition is assumed to be the most likely trigger of the slope failure, since existing boreholes indicate that fluvial depositional environmental created layers of lean clay, fat clay, and sand varying from about four (4) feet in depth to over twenty five (25) feet in this area.

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d. Drawing deliverables to El Paso Headquarters Office shall be 11 inch x 17 inch (ANSI B) and shall be printed at true half scale.

e. In addition, the Contractor shall submit four (4) electronic copies for both 90% and for final submittals USIBWC's Headquarters Office in El Paso, TX. Electronic copies shall be provided on CD, DVD, or USB drives.

f. All written reports shall be printed on paper containing 30% post-consumer fiber (30 PC). All deliverables shall also be furnished in electronic format. Electronic format of the report shall be in Portable Document Format (pdf) and Microsoft Word 2007, while electronic format for all drawings shall be in pdf and in AutoCAD/AutoCAD Civil 3D 2012.

g. The Final Submittals shall include the Contractor's written response to all USIBWC comments generated during the review of all the 90% deliverables. In addition, the Contractor shall provide marked up copies of the 90% deliverables (all deliverables that required revisions) showing all changes made on the 90% after USIBWC comments. A meeting between the USIBWC and the Contractor shall be conducted after the Contractor receives and reviews USIBWC's comments on the 90% deliverables, if concurrence is not reached on comments.

h. Only after acceptance of the 90% responses by the USIBWC shall the Contractor provide final submittals.

H. Information Provided by USIBWC

1. The USIBWC shall provide the following existing project documents to the Contractor in digital format:

   (1) Tetra Design Report Titled: *"Upper Brownsville Levee Rehabilitation, Cameron Counties, Texas, Design Report, Final Design Submittal"* May 2012, By Tetra Tech Inc.

   (2) Tetra Construction Drawings Titled: *"Upper Brownsville Levee Rehabilitation, Cameron County, Texas, Conformed Project Drawings"* June 2012, By Tetra Tech Inc.

   (3) Tetra Construction Specifications Titled: *"Technical Specifications for Lower Rio Grande Flood Control Upper Brownsville Levee Rehabilitation"* June 21, 2012, By Tetra Tech Inc.

   (4) Tetra Geotechnical Report Titled: *"Geotechnical Exploration and Engineering Evaluation of Levee System for the Lower Rio Grande Flood Control Project from Cameron County Line of Donna Pump to Brownsville Levee Reach to its Eastmost Limit, Final Technical Memorandum"* July 24, 2009, By Raba-Kistner Consultants, Inc.

   (5) Tetra Geotechnical Report Titled: *"Geotechnical Addendum – Subreach Four for the Lower Rio Grande Flood Control Project Levee System-from Donna Pump to Brownsville Levee Reach, Hidalgo County and Cameron County, Texas"* June 1, 2011, By Raba-Kistner Consultants, Inc.

(6) Area Environmental Assessment Titled: *"Final Environmental Assessment, Improvements to the Donna-Brownsville Levee System, September, 2007."* September 2007, By USIBWC.

(7) Photo Documentation of the Area spanning from March 29, 2014 to present day.

2. It is the Contractor's responsibility to print all items provided in electronic format. All of the paper documents provided to the Contractor are/shall remain property of the USIBWC and shall be returned at the end of the project.

3. Information provided by the USIBWC in the form of reports or data cannot be used for work outside of the current SOW without written consent of the USIBWC.

END OF SCOPE OF WORK

Appendix B: CPT Logs

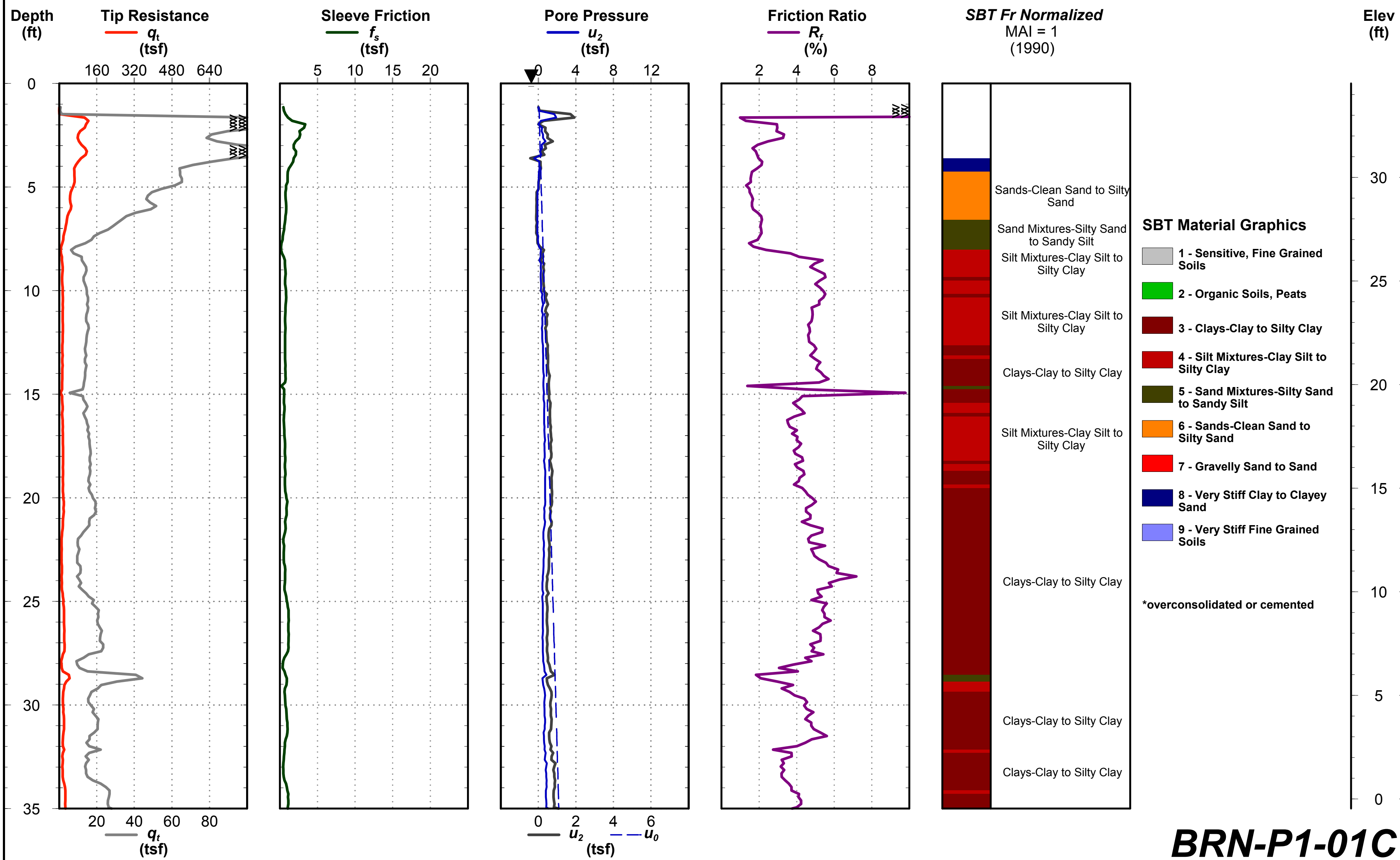
Cone Penetration Test BRN-P1-01C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489194.5
Easting: 1314445.6
Elevation: 34.5

Total Depth: 50.7 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-01C

Electronic File Name: BRN-P1-01C.cpt

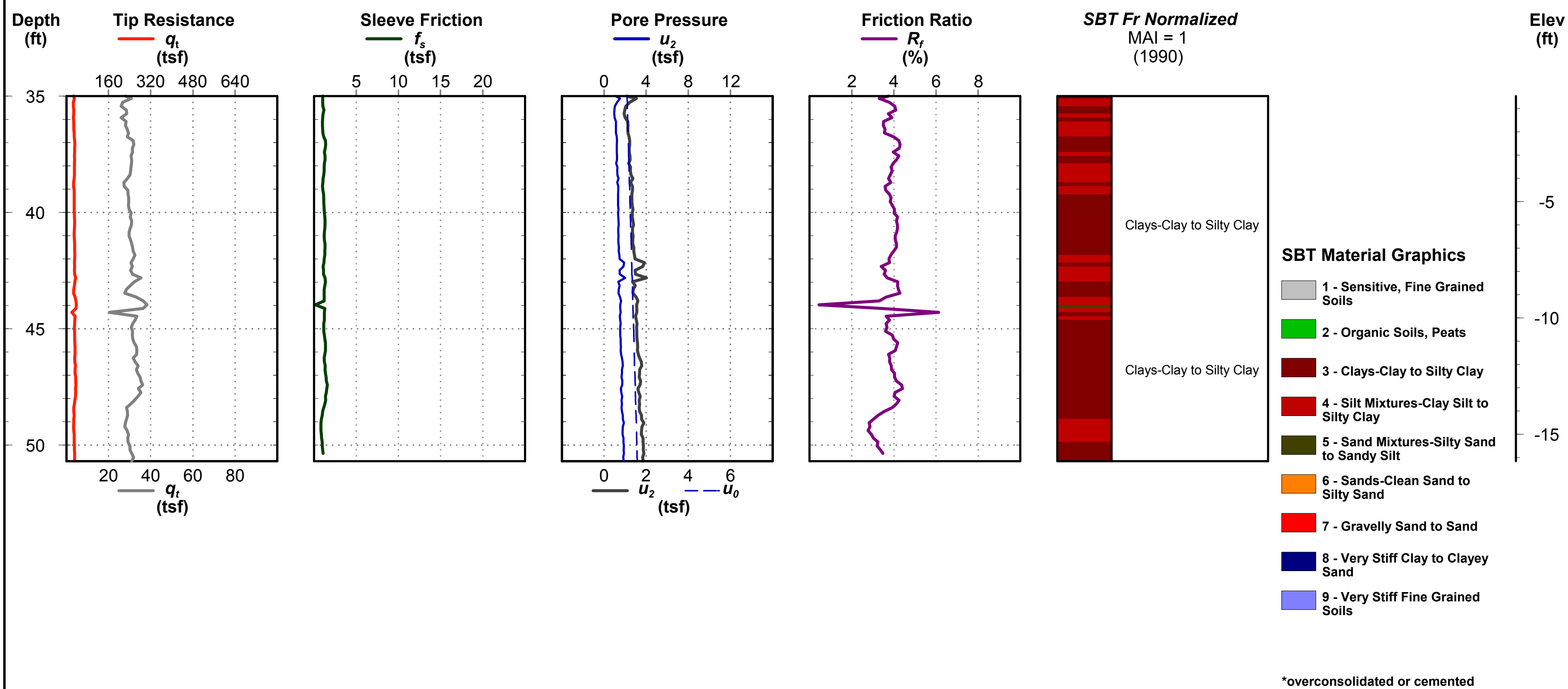
Cone Penetration Test BRN-P1-01C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489194.5
Easting: 1314445.6
Elevation: 34.5

Total Depth: 50.7 ft
Termination Criteria:
Cone Size:



BRN-P1-01C

Electronic File Name: BRN-P1-01C.cpt

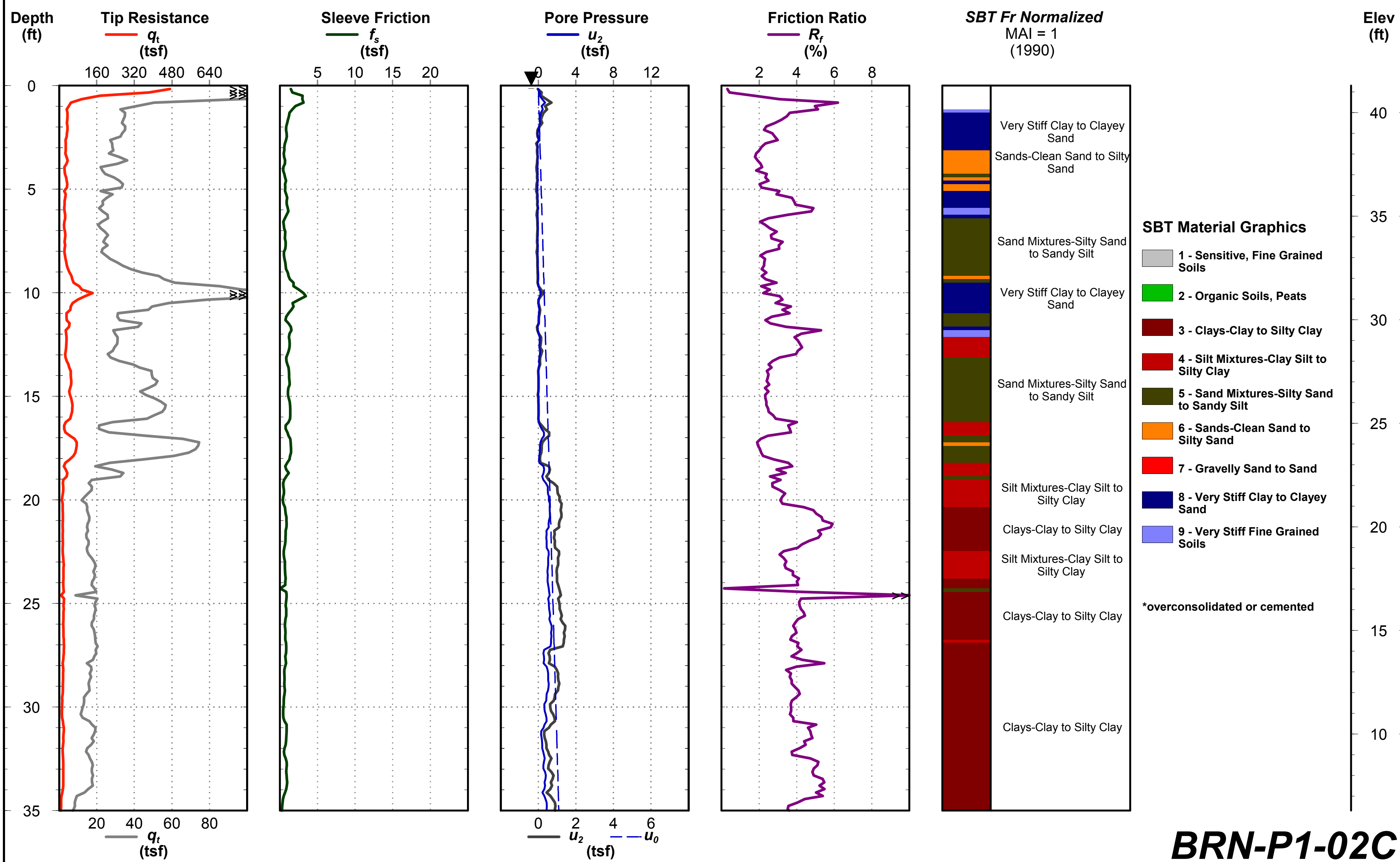
Cone Penetration Test BRN-P1-02C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489186.8
Easting: 1314329.0
Elevation: 41.3

Total Depth: 50.7 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-02C

Electronic File Name: BRN-P1-02C.cpt

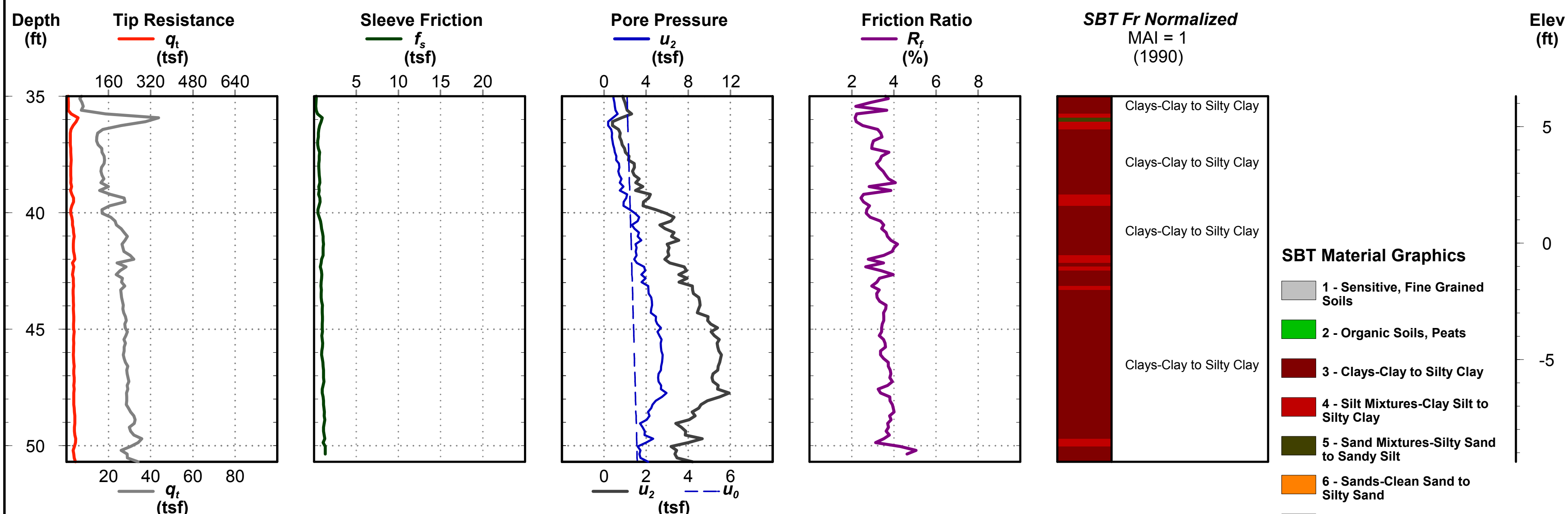
Cone Penetration Test BRN-P1-02C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489186.8
Easting: 1314329.0
Elevation: 41.3

Total Depth: 50.7 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-02C

Electronic File Name: BRN-P1-02C.cpt

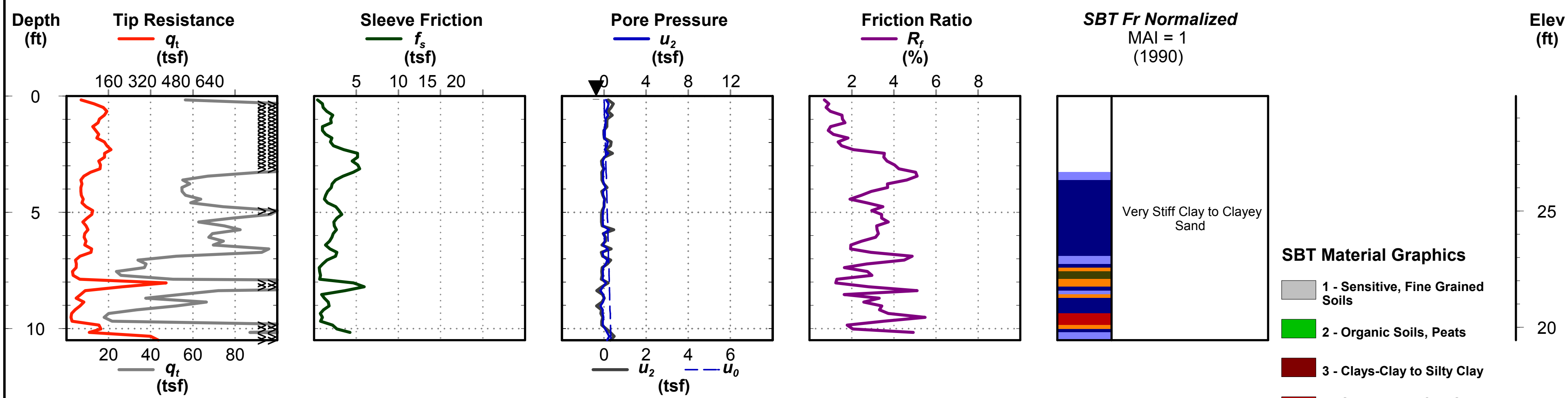
Cone Penetration Test BRN-P1-03C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489177.3
Easting: 1314263.5
Elevation: 29.9

Total Depth: 10.5 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-03C

Electronic File Name: BRN-P1-03C.cpt

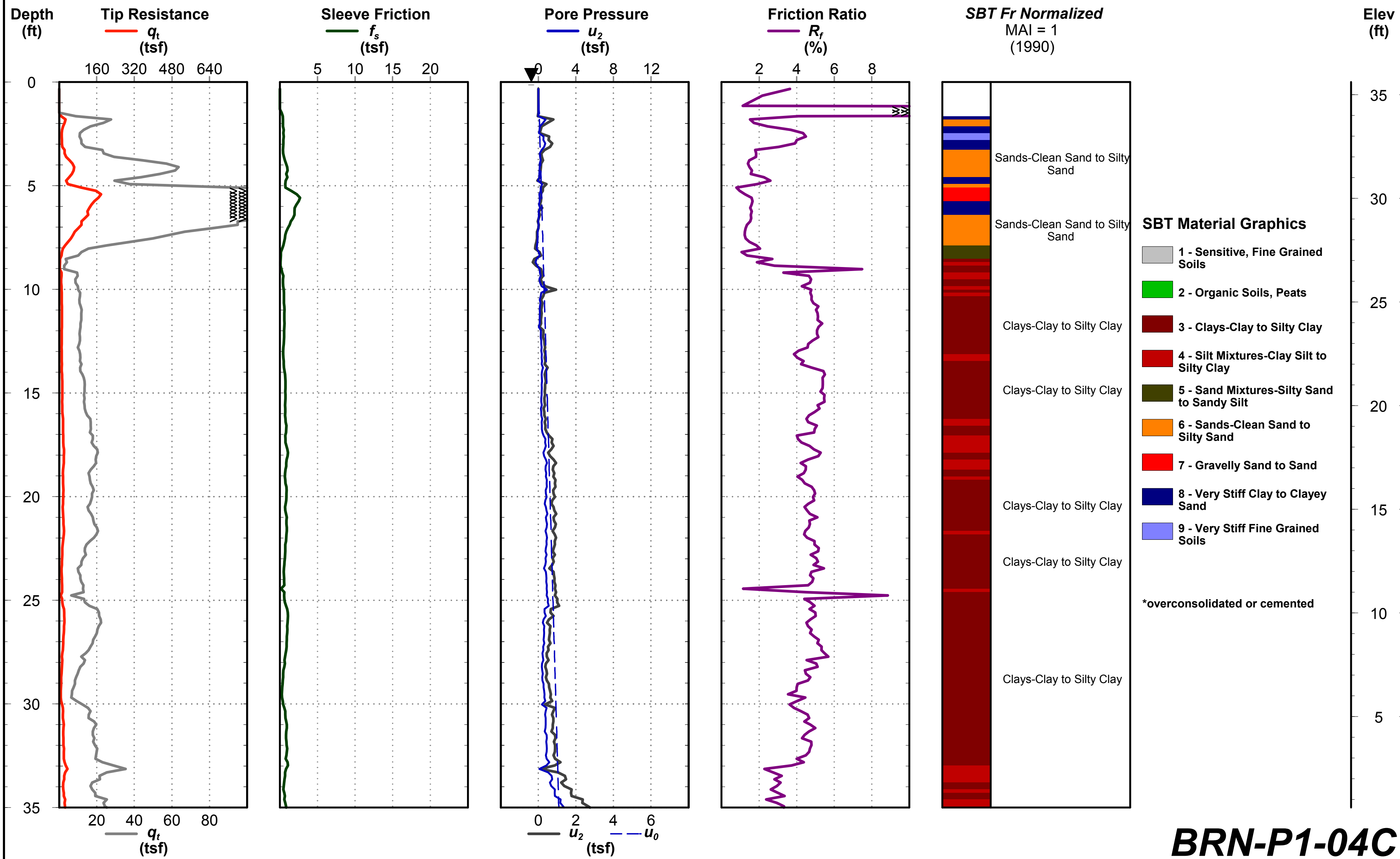
Brownsville, Tx
Project Number :IBWC

Cone Penetration Test BRN-P1-04C

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489396.7
Easting: 1314322.0
Elevation: 35.6

Total Depth: 61.8 ft
Termination Criteria:
Cone Size:



BRN-P1-04C

Electronic File Name: BRN-P1-04C.cpt

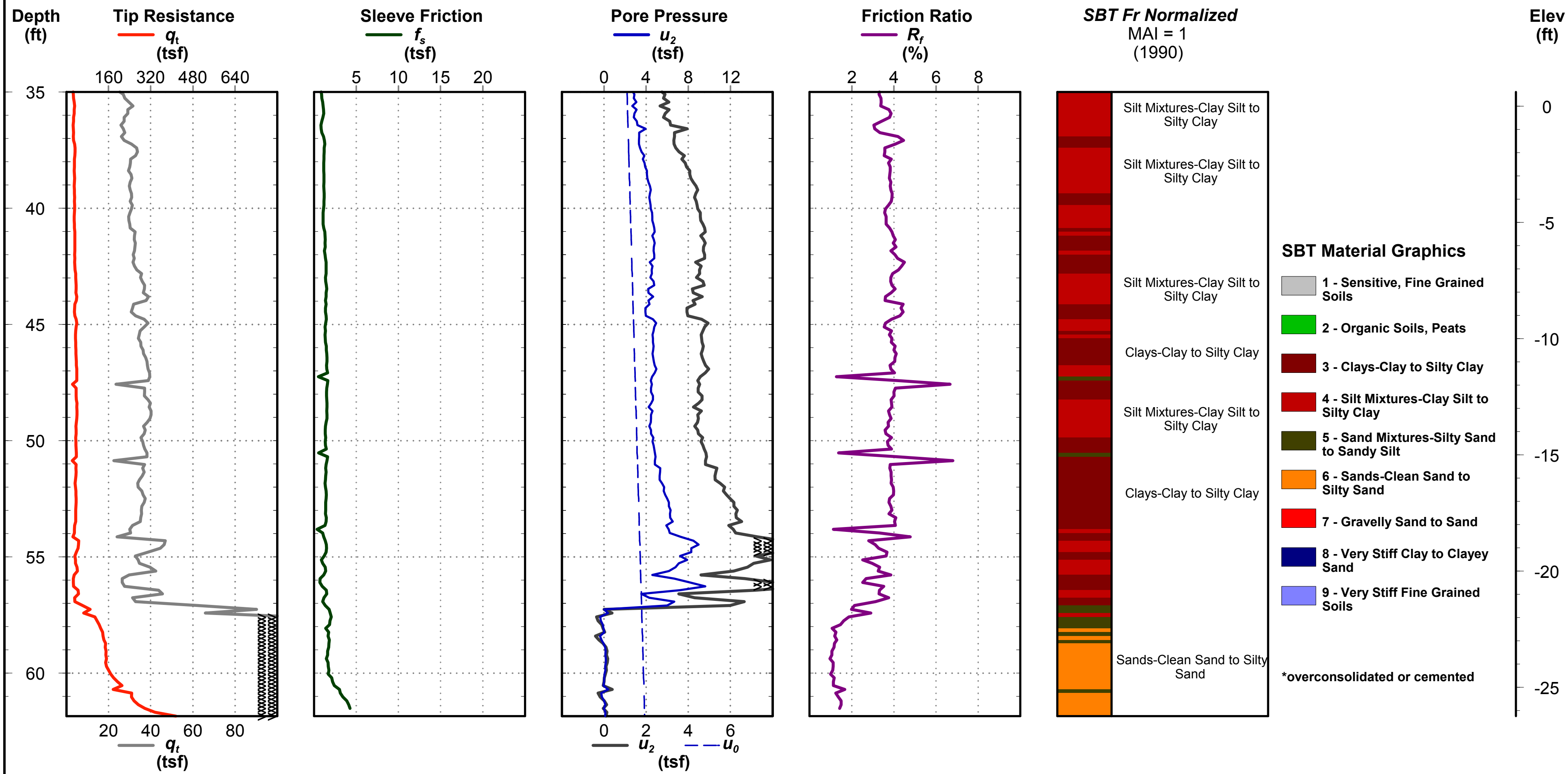
Cone Penetration Test BRN-P1-04C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489396.7
Easting: 1314322.0
Elevation: 35.6

Total Depth: 61.8 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-04C

Electronic File Name: BRN-P1-04C.cpt

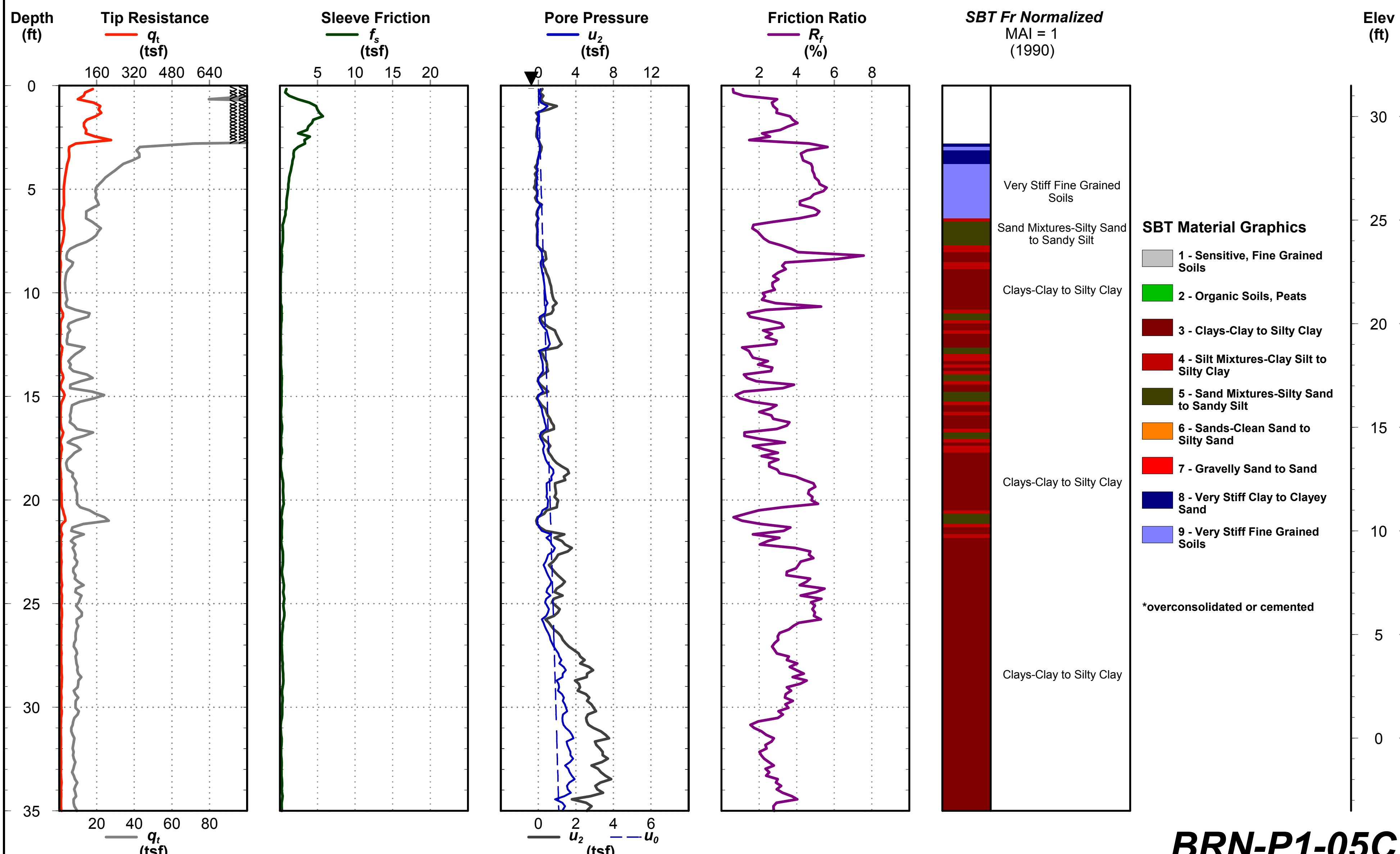
Cone Penetration Test BRN-P1-05C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489384.4
Easting: 1314214.4
Elevation: 31.5

Total Depth: 70.4 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-05C

Electronic File Name: BRN-P1-05C.cpt

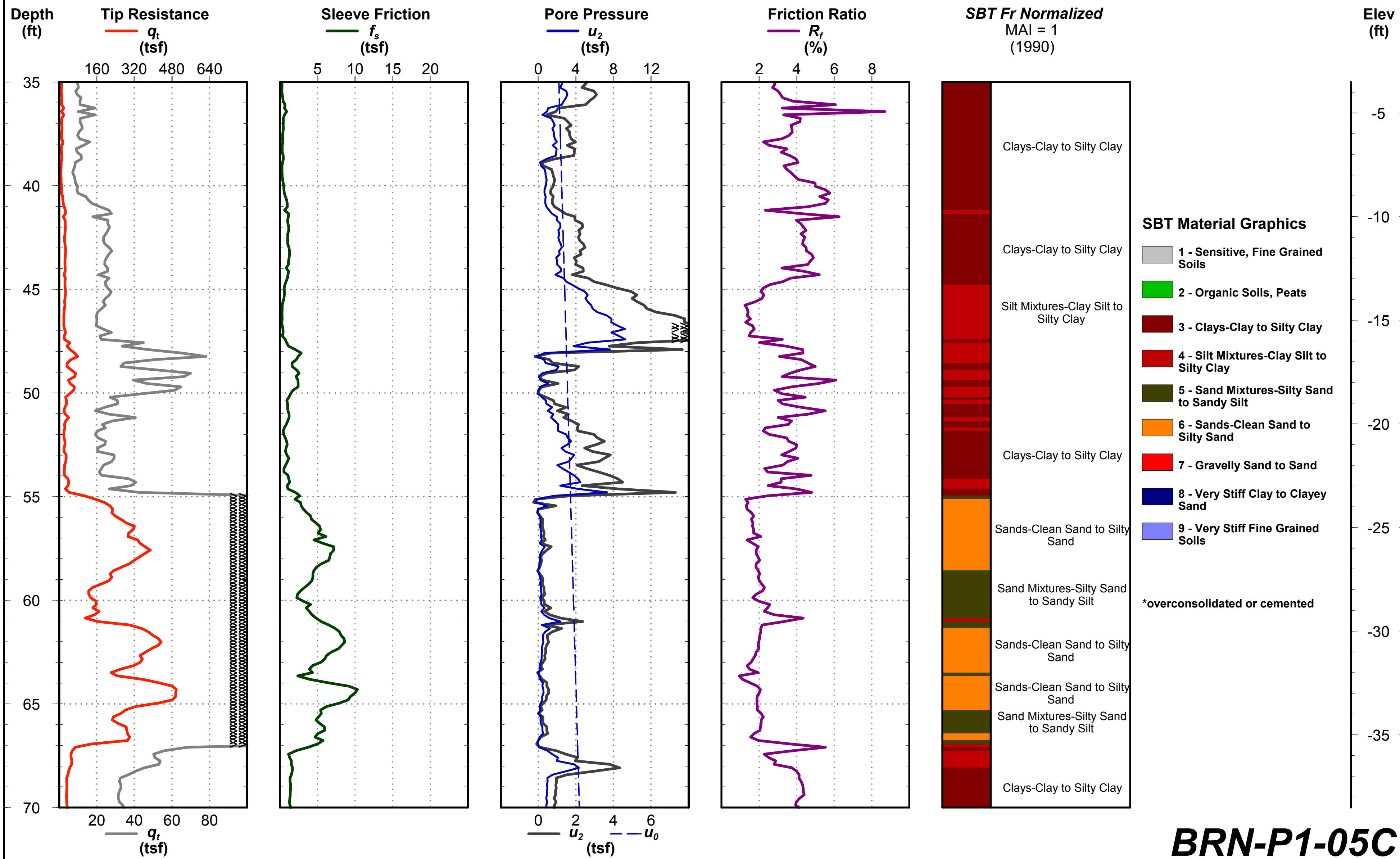
Cone Penetration Test BRN-P1-05C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489384.4
Easting: 1314214.4
Elevation: 31.5

Total Depth: 70.4 ft
Termination Criteria:
Cone Size:



BRN-P1-05C

Electronic File Name: BRN-P1-05C.cpt

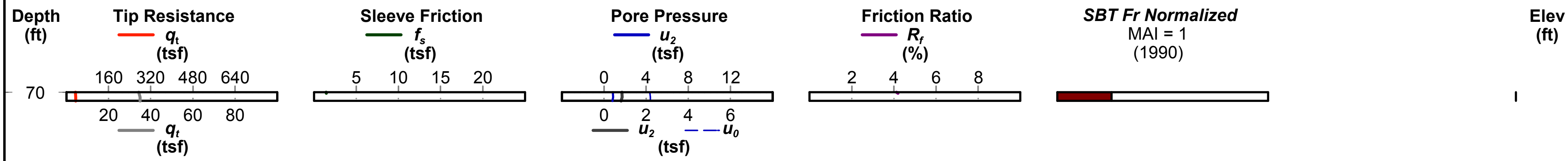
Cone Penetration Test BRN-P1-05C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489384.4
Easting: 1314214.4
Elevation: 31.5

Total Depth: 70.4 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-05C

Electronic File Name: BRN-P1-05C.cpt

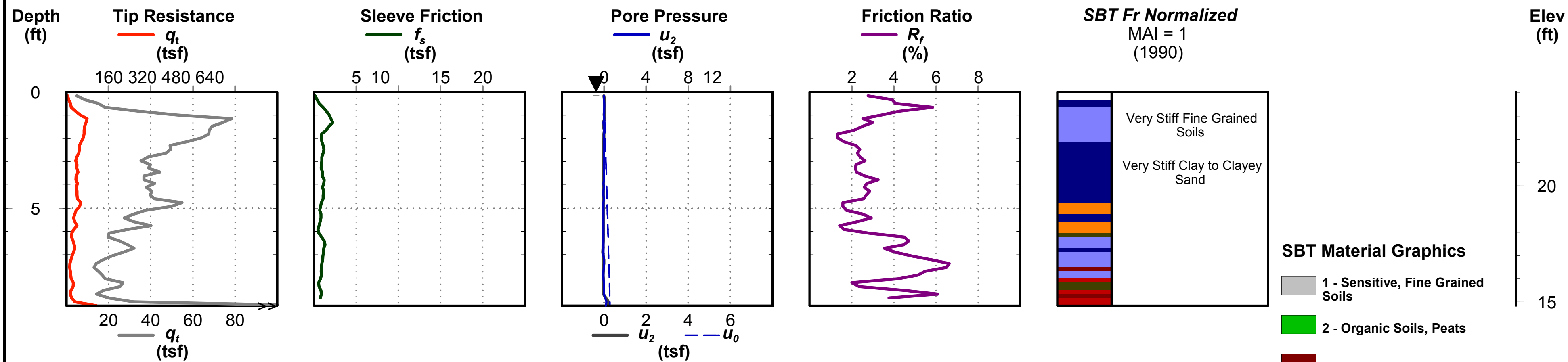
Cone Penetration Test BRN-P1-06C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489345.2
Easting: 1314130.4
Elevation: 24.0

Total Depth: 9.2 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

1 - Sensitive, Fine Grained Soils
2 - Organic Soils, Peats
3 - Clays-Clay to Silty Clay
4 - Silt Mixtures-Clay Silt to Silty Clay
5 - Sand Mixtures-Silty Sand to Sandy Silt
6 - Sands-Clean Sand to Silty Sand
7 - Gravelly Sand to Sand
8 - Very Stiff Clay to Clayey Sand
9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-06C

Electronic File Name: BRN-P1-06C.cpt

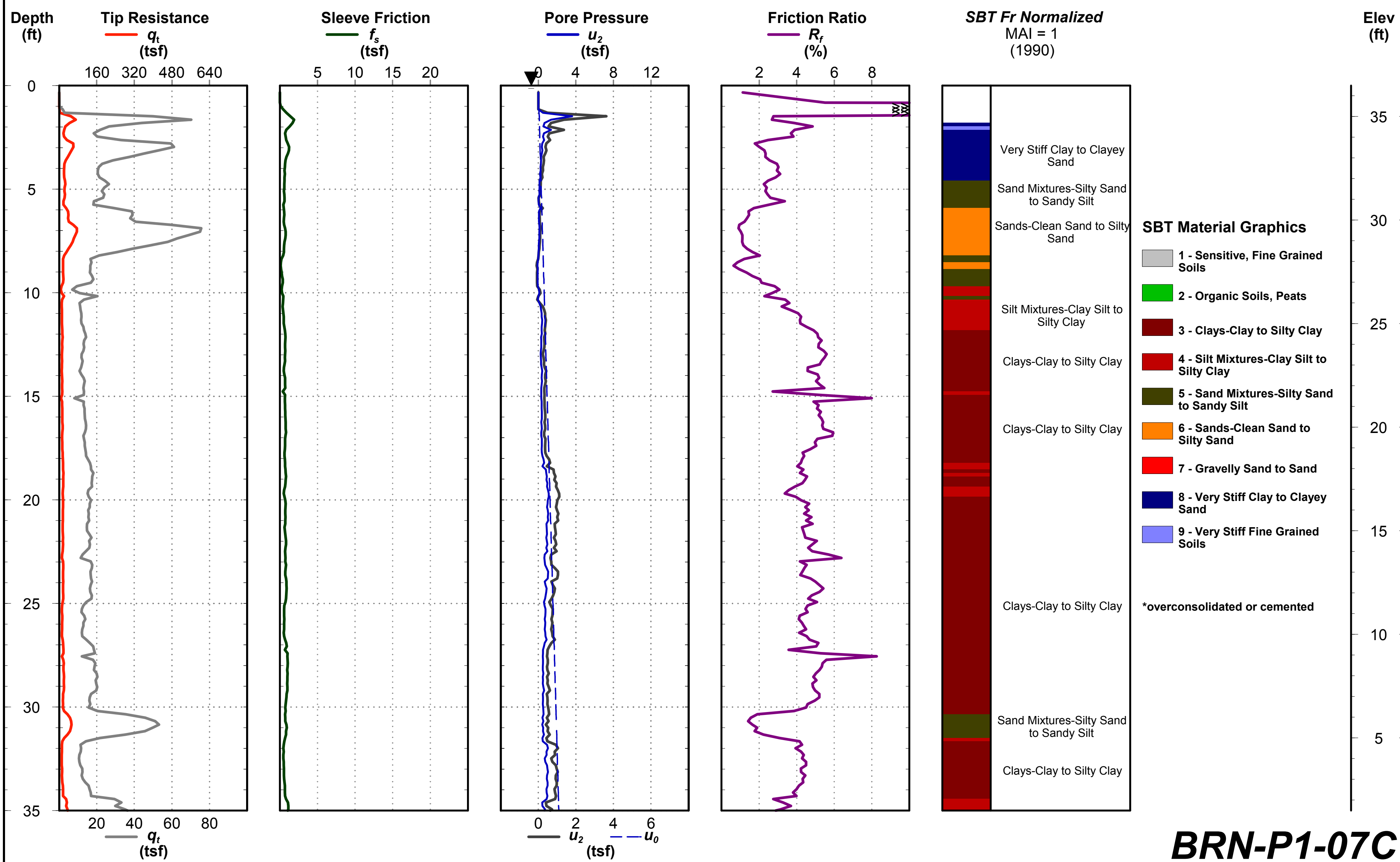
Cone Penetration Test BRN-P1-07C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489583.2
Easting: 1314212.7
Elevation: 36.5

Total Depth: 70.5 ft
Termination Criteria:
Cone Size:



BRN-P1-07C

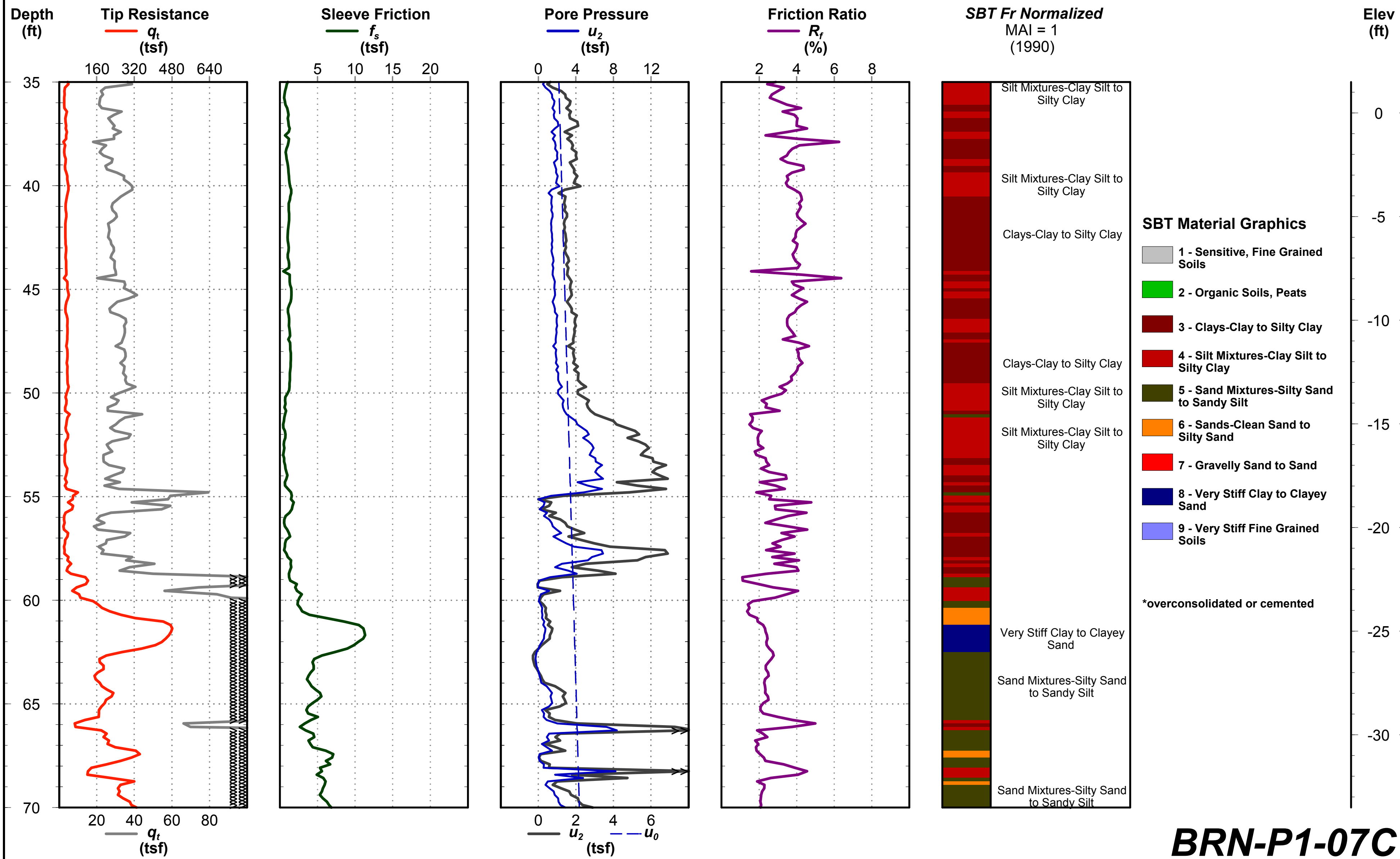
Brownsville, Tx
Project Number :IBWC

Cone Penetration Test BRN-P1-07C

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489583.2
Easting: 1314212.7
Elevation: 36.5

Total Depth: 70.5 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-07C

Electronic File Name: BRN-P1-07C.cpt

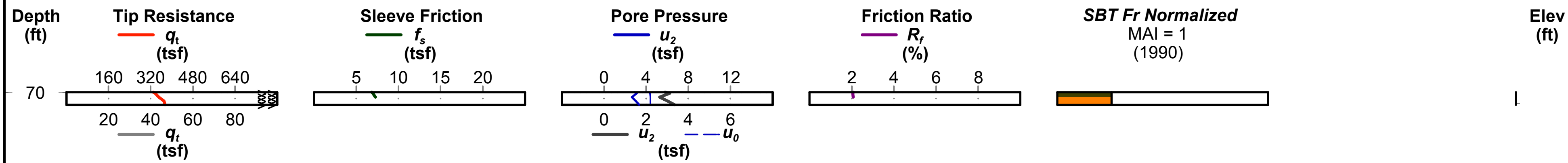
Cone Penetration Test BRN-P1-07C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489583.2
Easting: 1314212.7
Elevation: 36.5

Total Depth: 70.5 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-07C

Electronic File Name: BRN-P1-07C.cpt

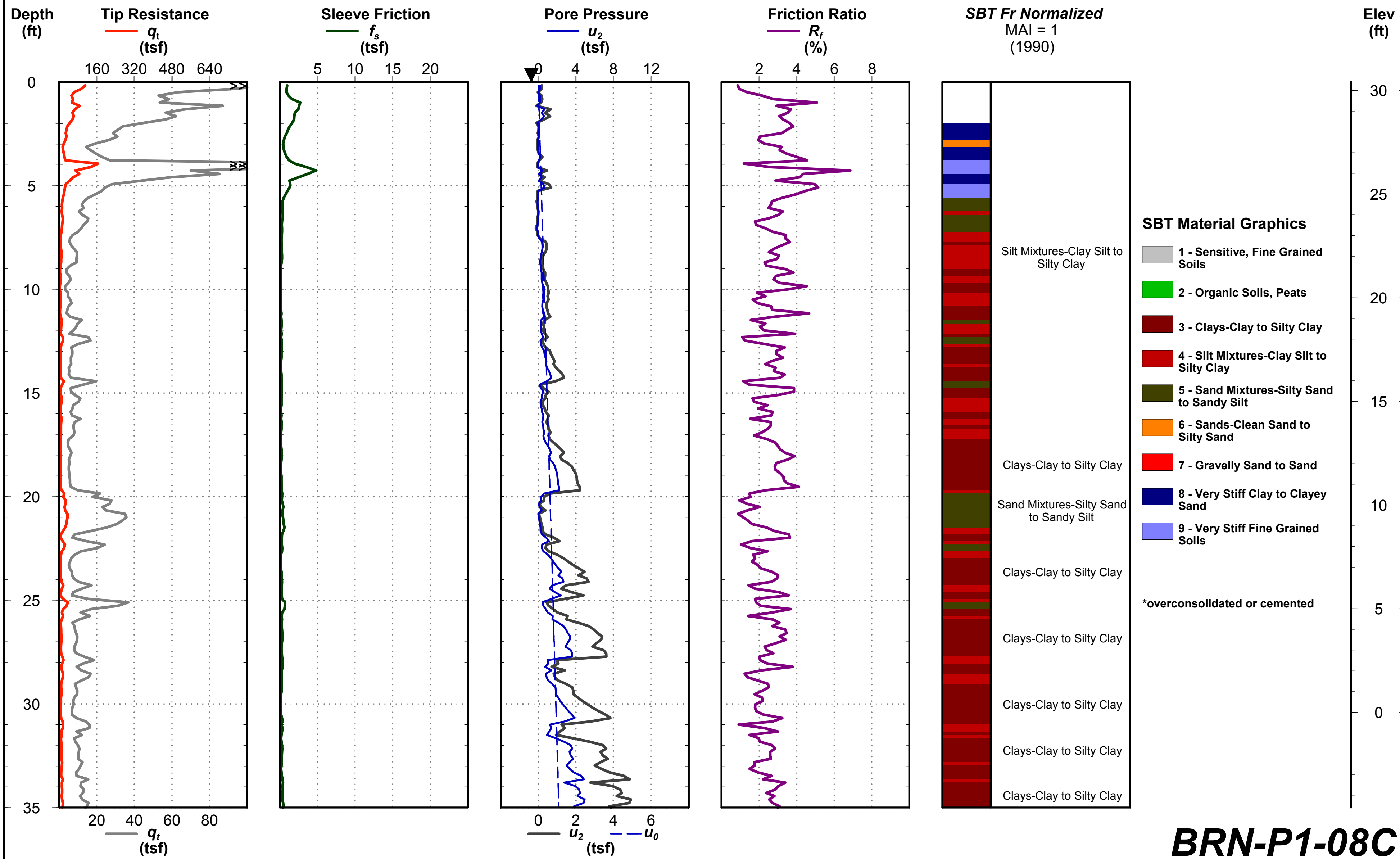
Cone Penetration Test BRN-P1-08C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489557.7
Easting: 1314117.9
Elevation: 30.4

Total Depth: 70.4 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-08C

Electronic File Name: Rename-BRN-P1-08C.cpt

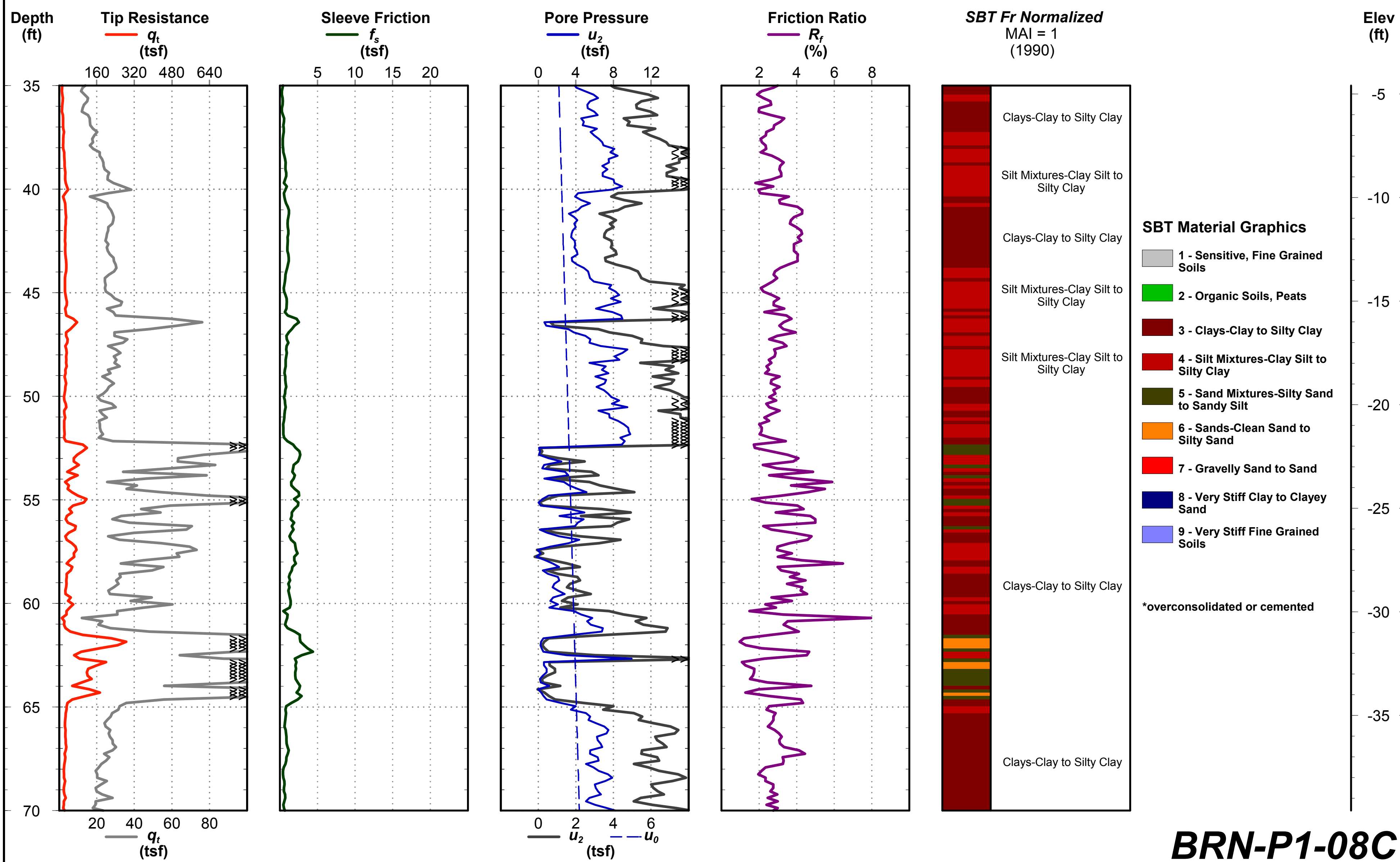
Cone Penetration Test BRN-P1-08C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489557.7
Easting: 1314117.9
Elevation: 30.4

Total Depth: 70.4 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-08C

Electronic File Name: Rename-BRN-P1-08C.cpt

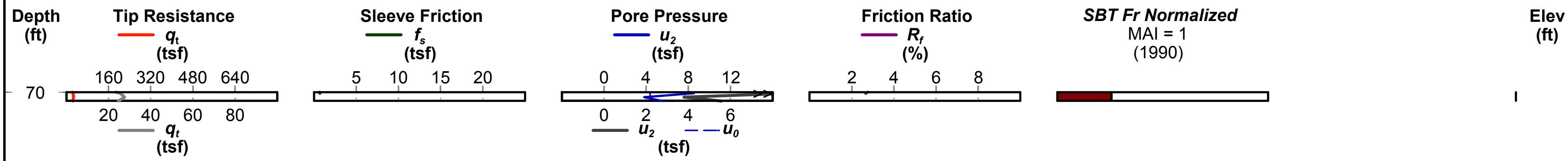
Cone Penetration Test BRN-P1-08C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489557.7
Easting: 1314117.9
Elevation: 30.4

Total Depth: 70.4 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-08C

Electronic File Name: Rename-BRN-P1-08C.cpt

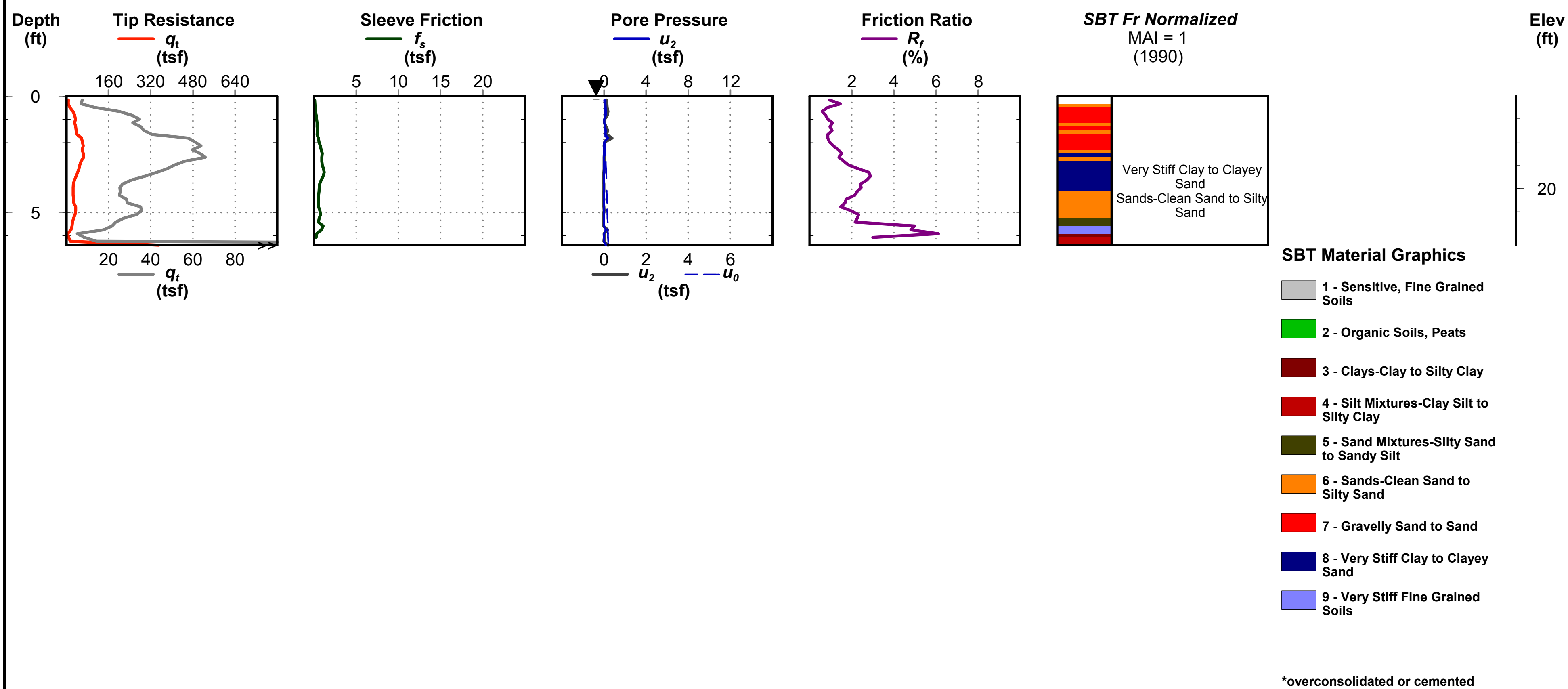
Cone Penetration Test BRN-P1-09C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489529.7
Easting: 1314048.6
Elevation: 24.0

Total Depth: 6.4 ft
Termination Criteria:
Cone Size:



BRN-P1-09C

Electronic File Name: BRN-P1-09C.cpt

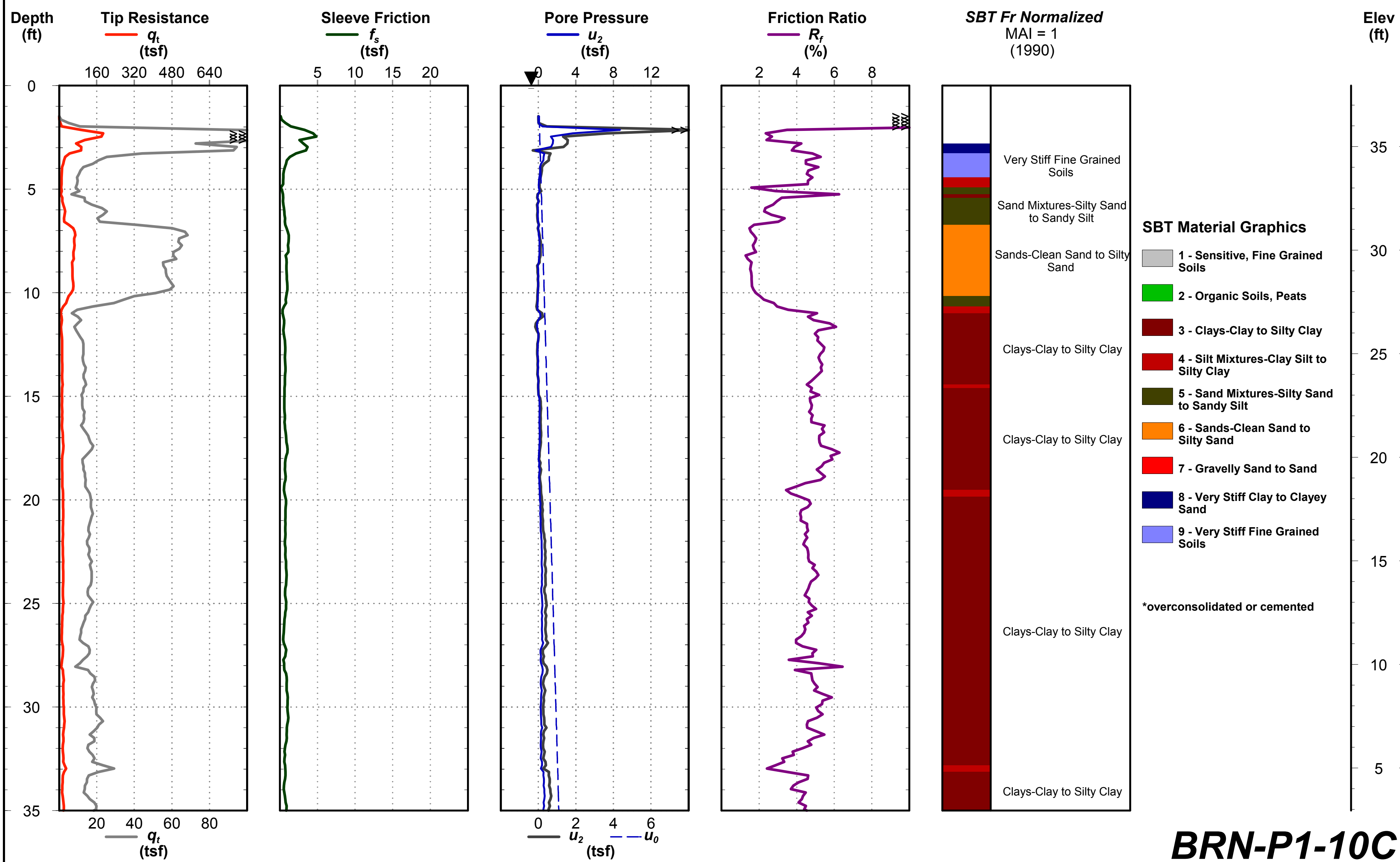
Cone Penetration Test BRN-P1-10C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489801.0
Easting: 1314112.1
Elevation: 37.9

Total Depth: 63.8 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-10C

Electronic File Name: BRN-P1-10C.cpt

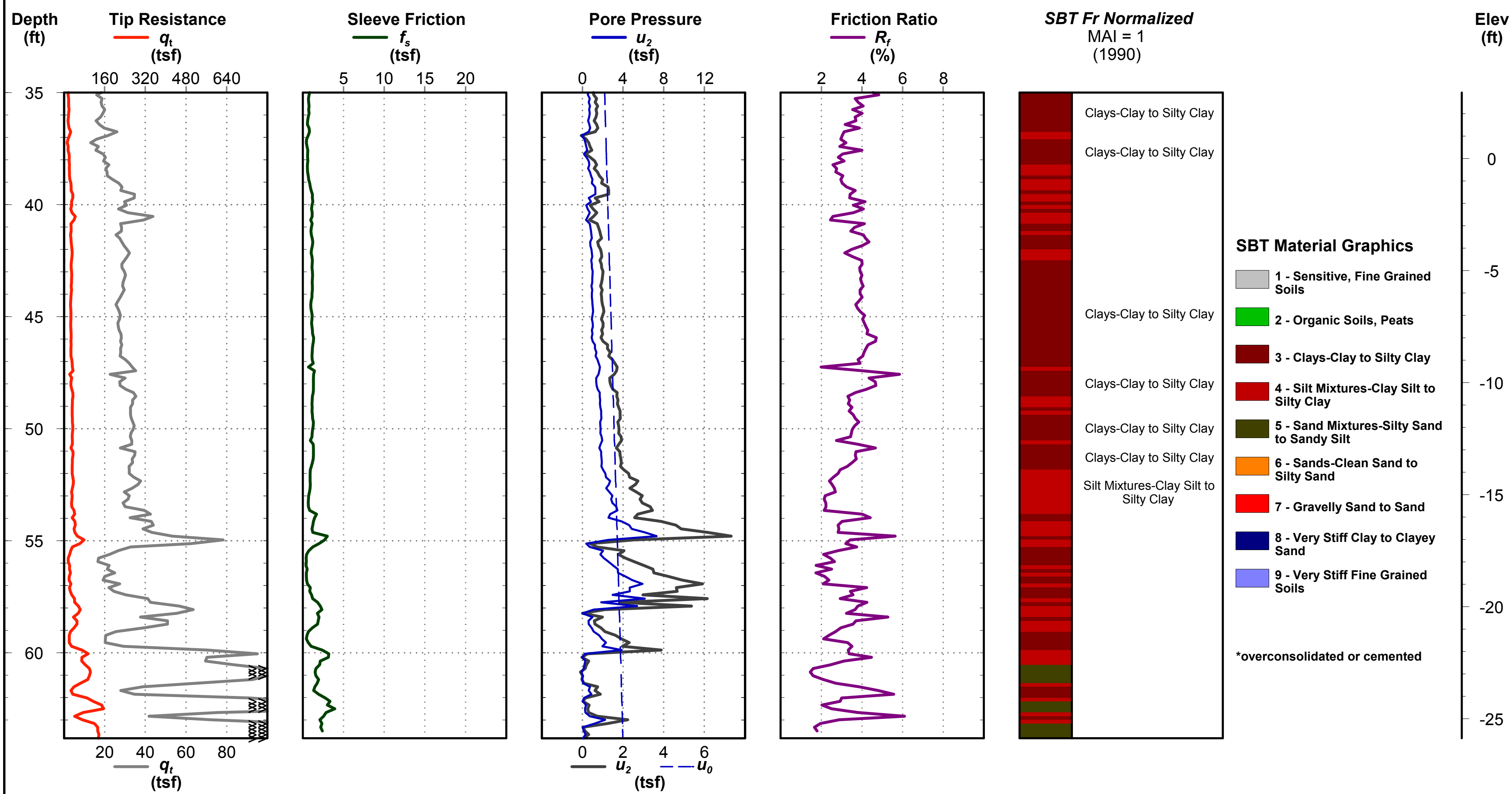
Cone Penetration Test BRN-P1-10C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489801.0
Easting: 1314112.1
Elevation: 37.9

Total Depth: 63.8 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-10C

Electronic File Name: BRN-P1-10C.cpt

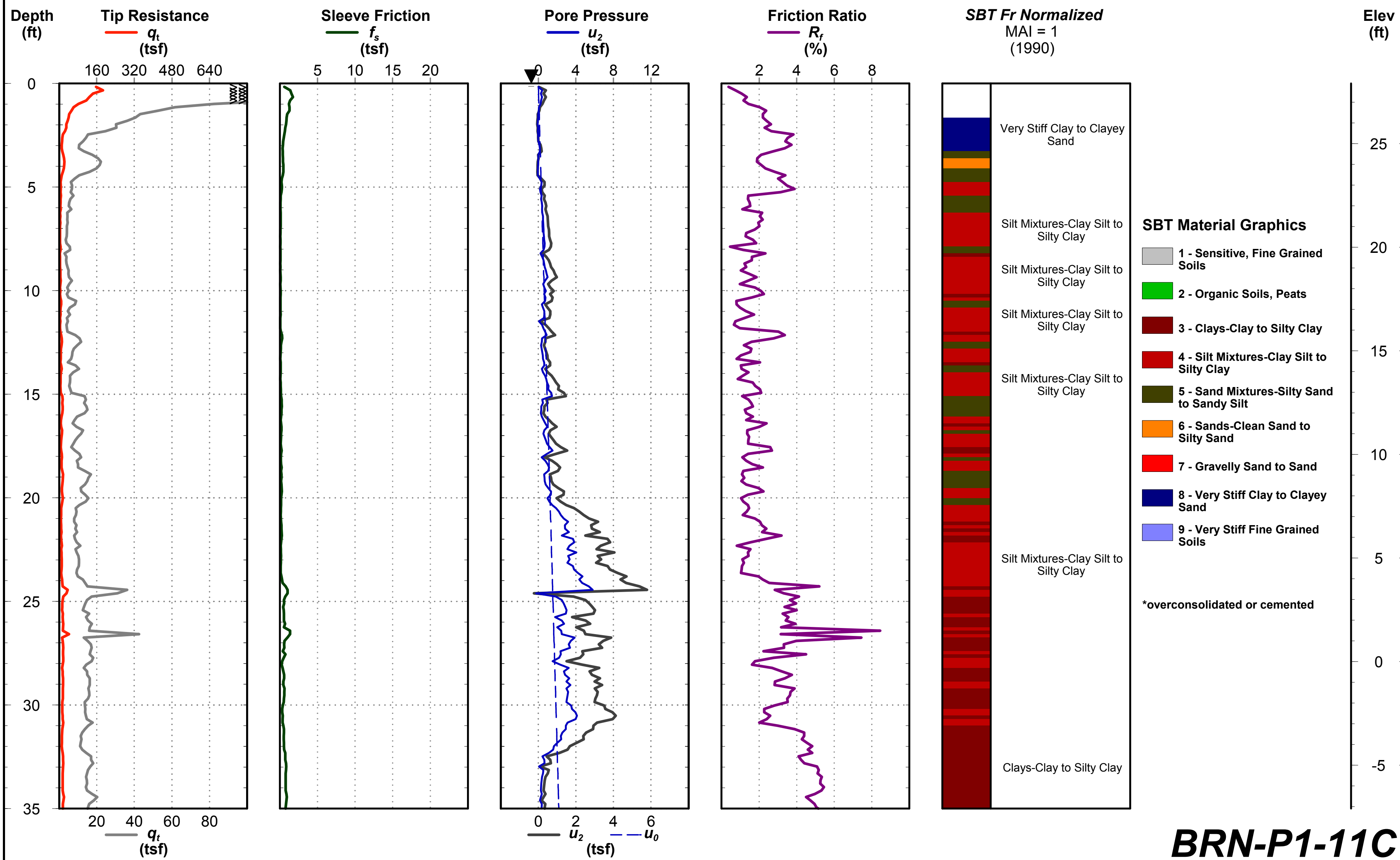
Cone Penetration Test BRN-P1-11C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489758.2
Easting: 1314016.0
Elevation: 27.9

Total Depth: 60.2 ft
Termination Criteria:
Cone Size:



BRN-P1-11C

Electronic File Name: BRN-P1-11C.cpt

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

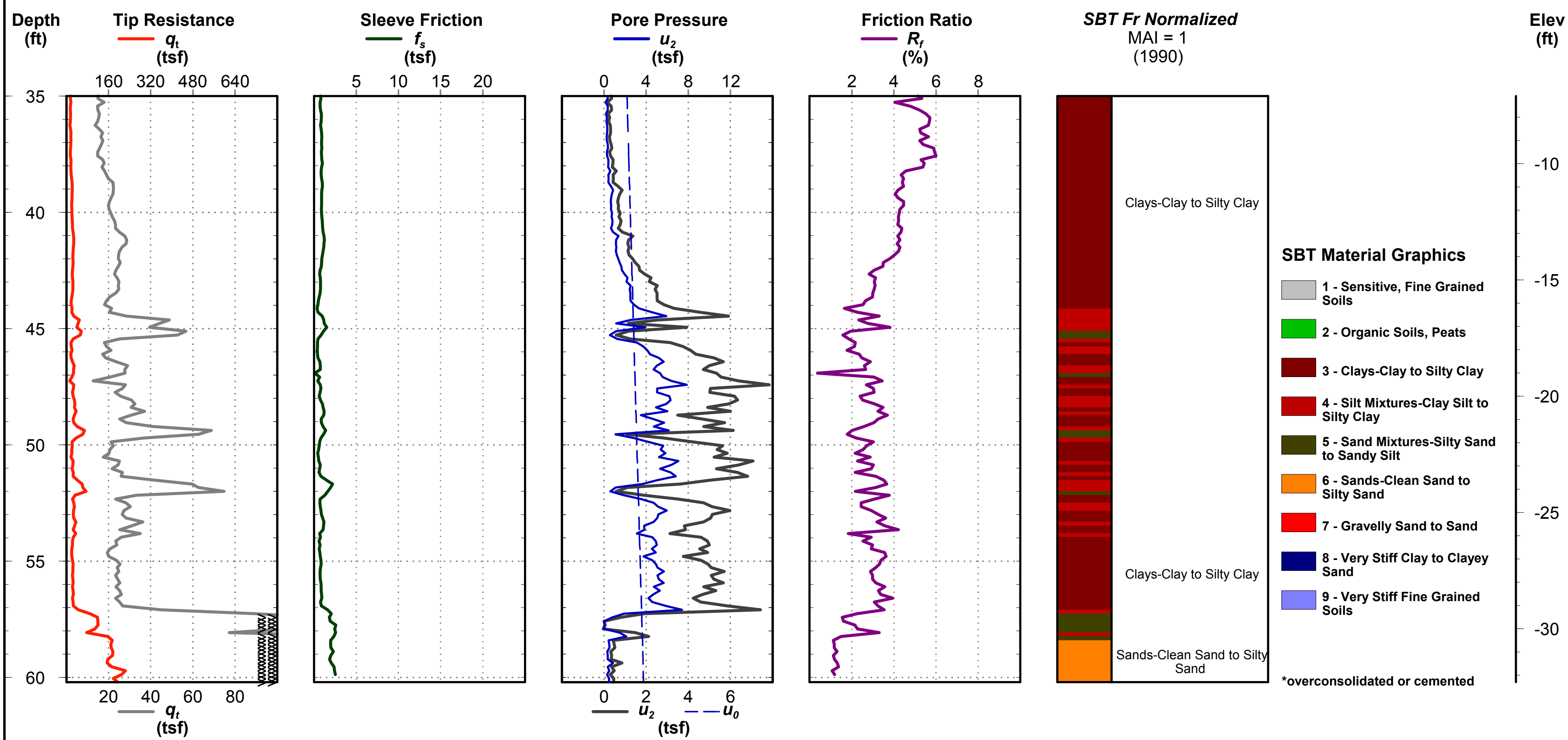
Cone Penetration Test BRN-P1-11C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489758.2
Easting: 1314016.0
Elevation: 27.9

Total Depth: 60.2 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-11C

Electronic File Name: BRN-P1-11C.cpt

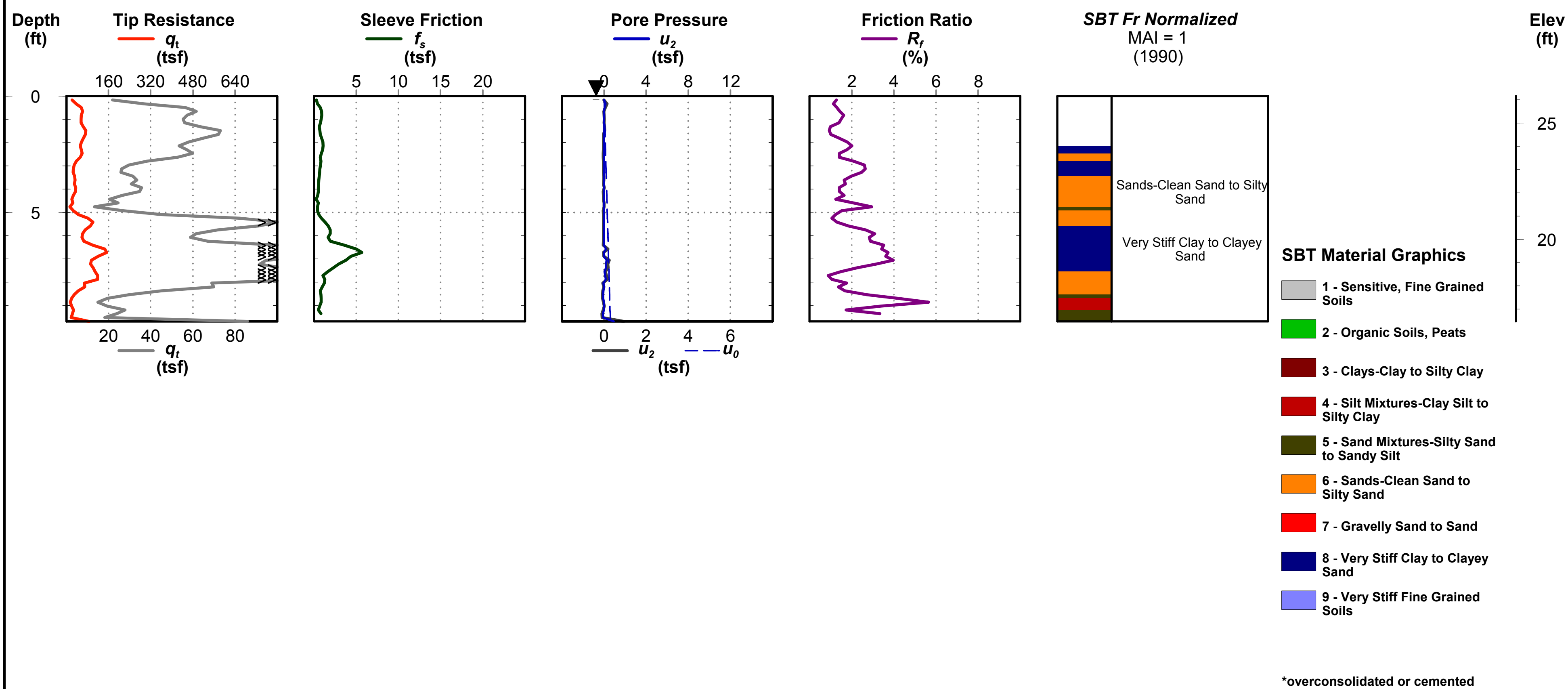
Cone Penetration Test BRN-P1-12C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489721.4
Easting: 1313977.4
Elevation: 26.2

Total Depth: 9.7 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-12C

Electronic File Name: BRN-P1-12C.cpt

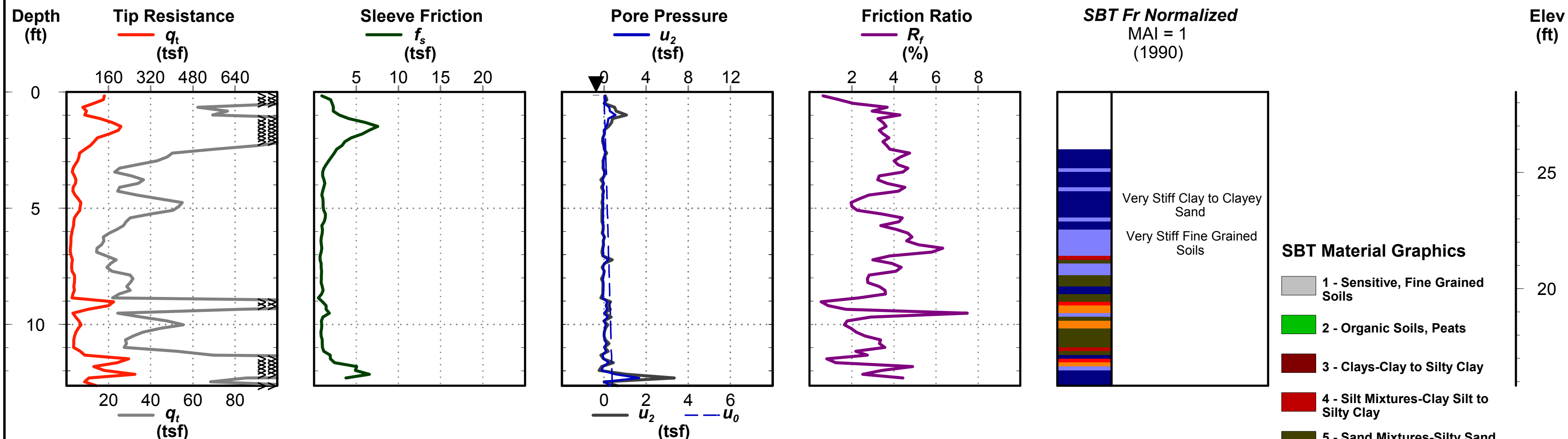
Cone Penetration Test BRN-P1-13C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16490042.6
Easting: 1313668.8
Elevation: 28.5

Total Depth: 12.6 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P1-13C

Electronic File Name: BRN-P1-13C.cpt

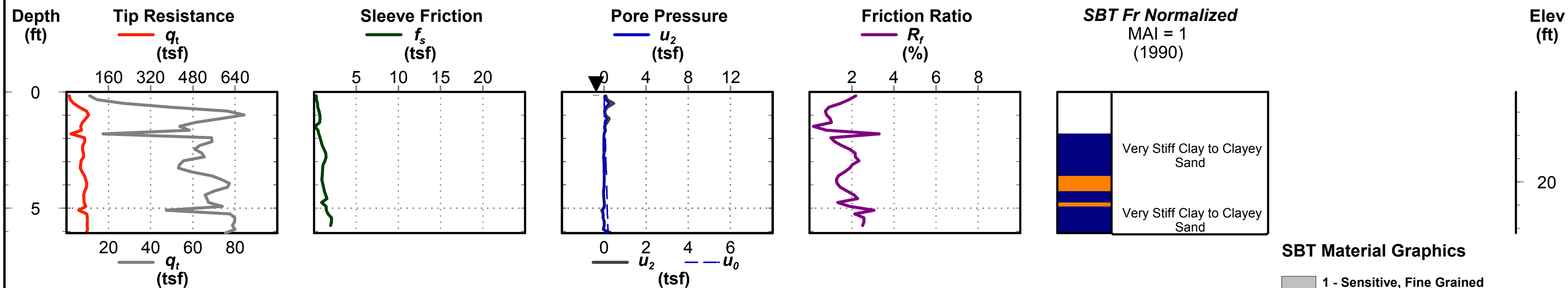
Cone Penetration Test BRN-P1-31C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489622.4
Easting: 1314005.1
Elevation: 23.9

Total Depth: 6.1 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

Electronic File Name: BRN-P2-31C.cpt

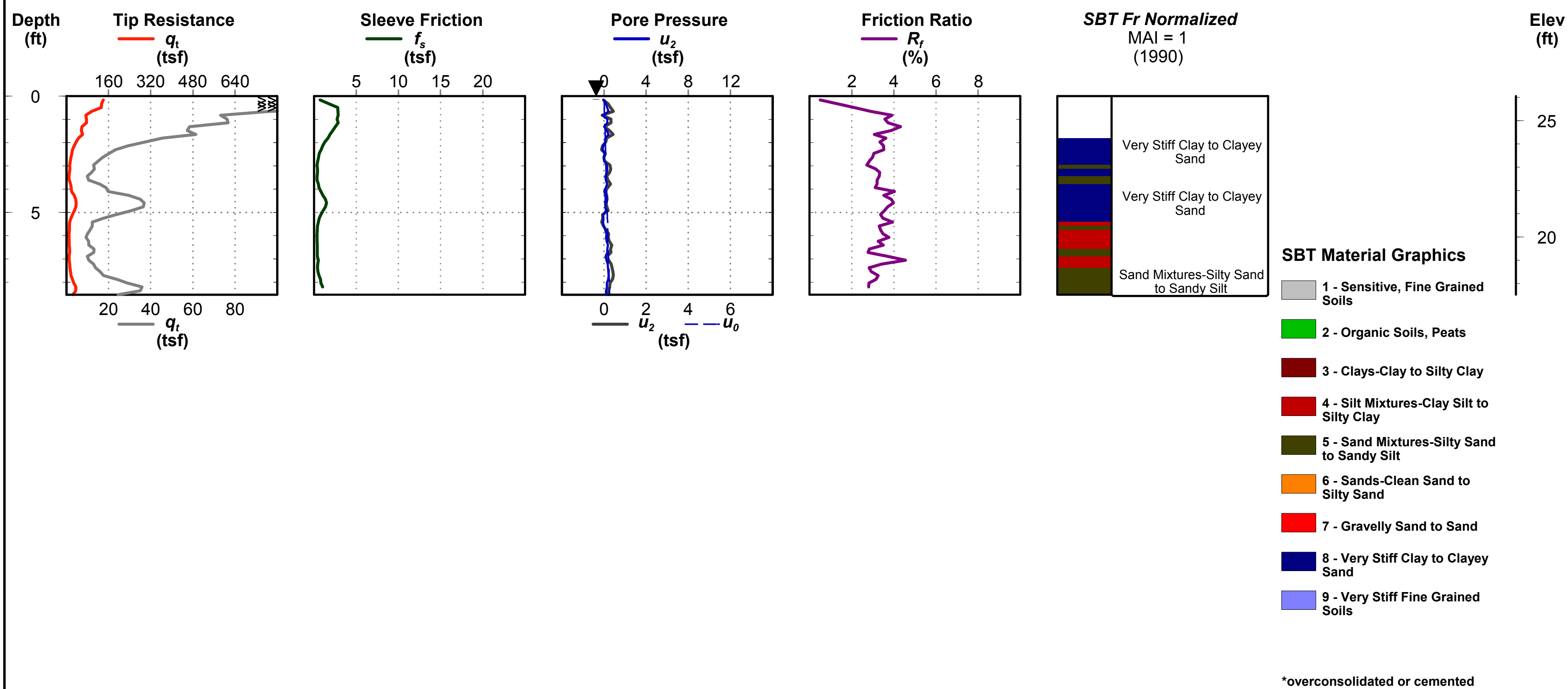
Cone Penetration Test BRN-P2-14C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489836.7
Easting: 1313959.2
Elevation: 26.1

Total Depth: 8.5 ft
Termination Criteria:
Cone Size:



BRN-P2-14C

Electronic File Name: BRN-P2-14C.cpt

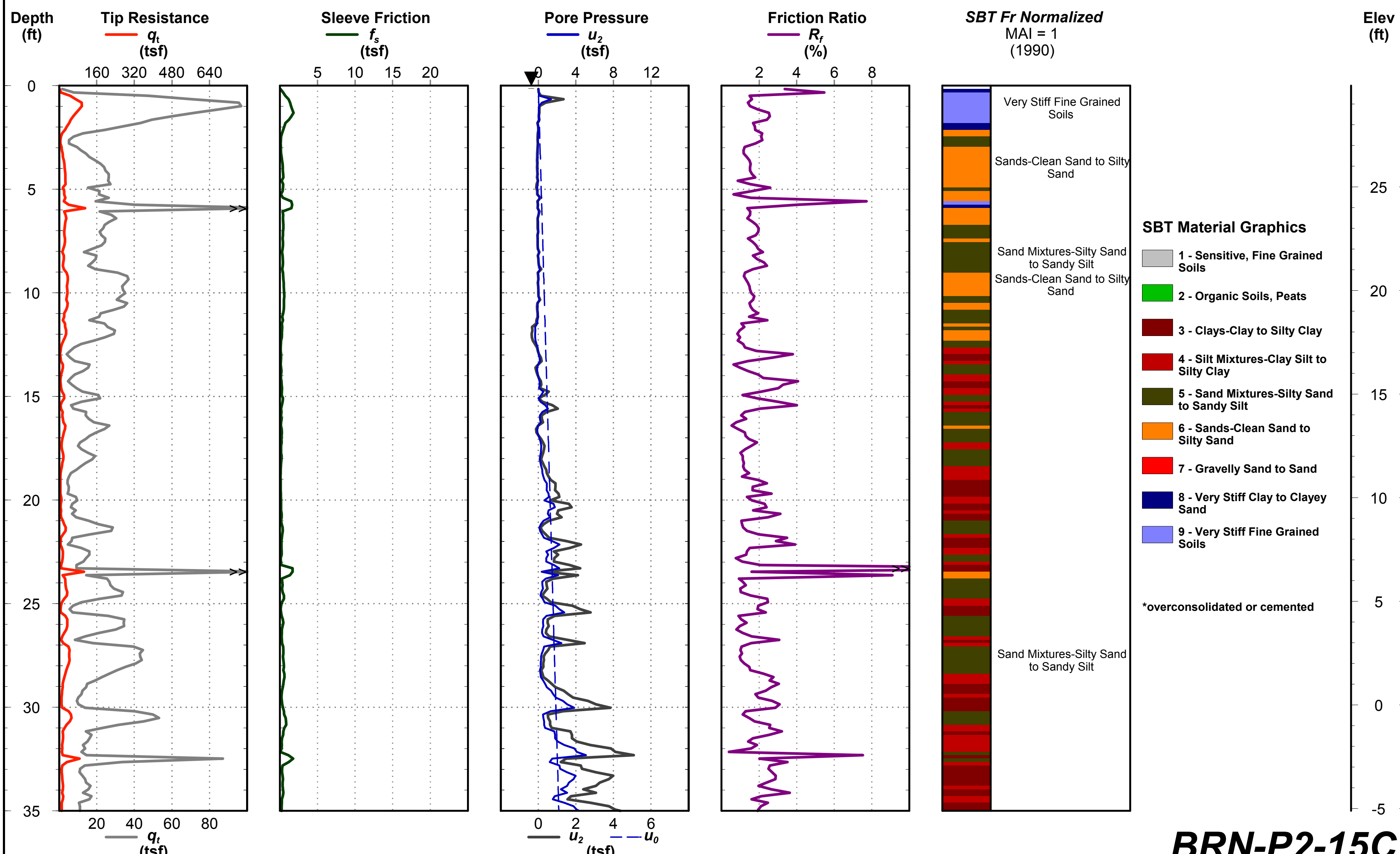
Cone Penetration Test BRN-P2-15C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489644.9
Easting: 1314068.6
Elevation: 29.9

Total Depth: 51.2 ft
Termination Criteria:
Cone Size:



BRN-P2-15C

Electronic File Name: BRN-P2-15C.cpt

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

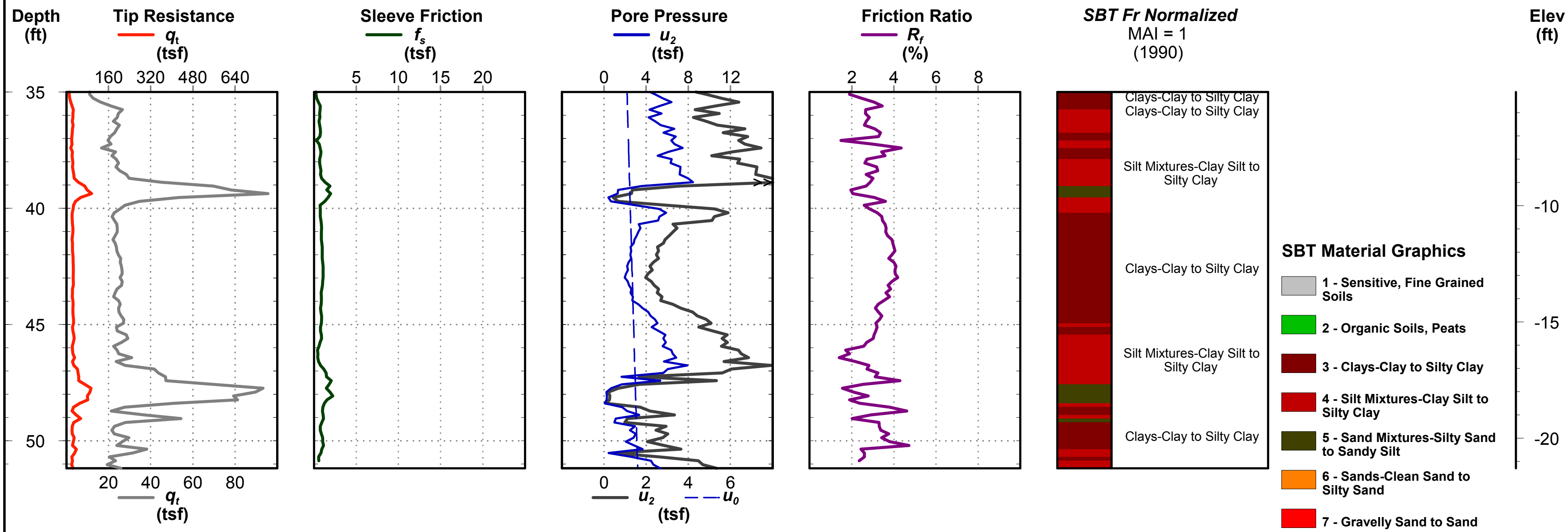
Cone Penetration Test BRN-P2-15C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489644.9
Easting: 1314068.6
Elevation: 29.9

Total Depth: 51.2 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-15C

Electronic File Name: BRN-P2-15C.cpt

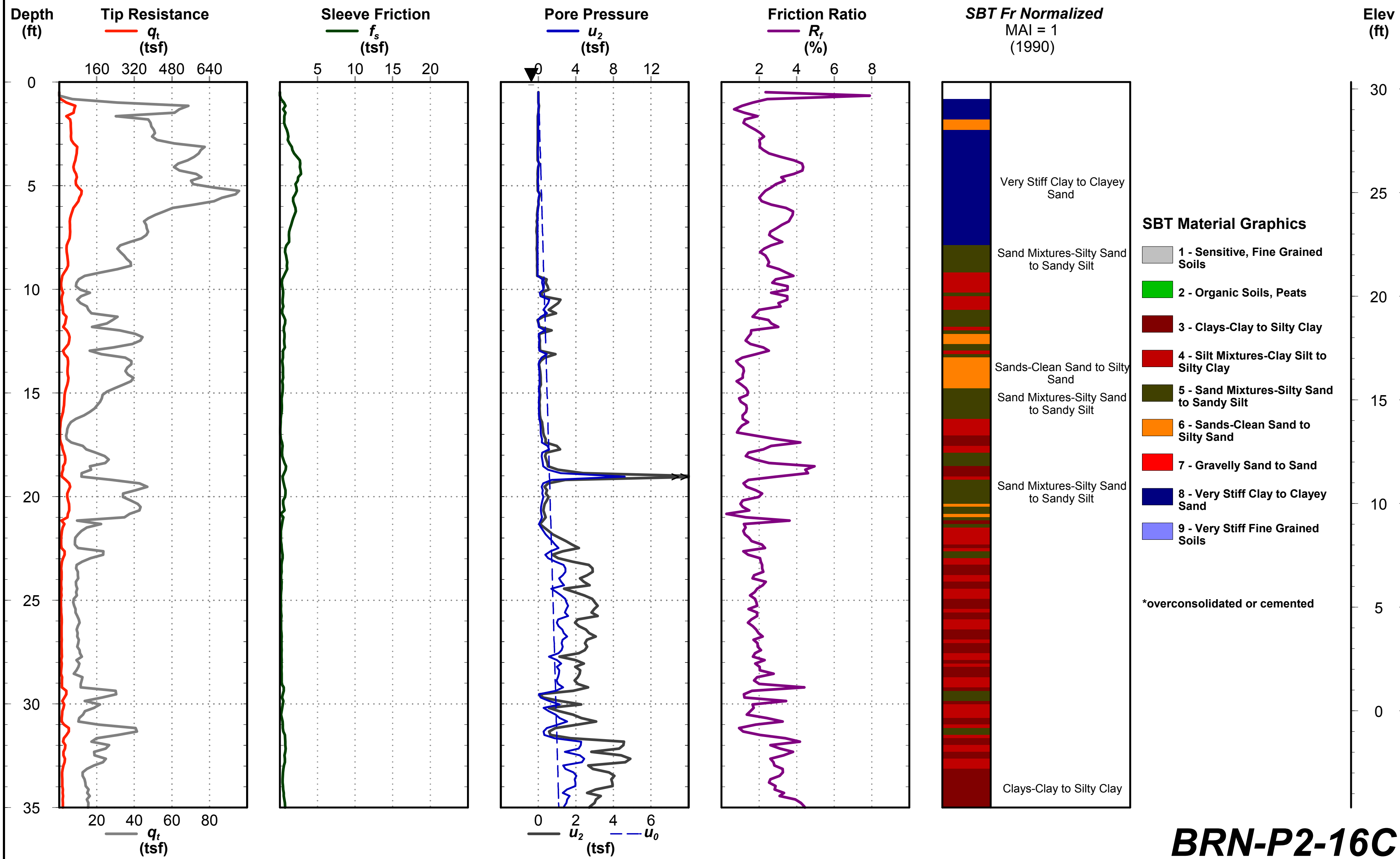
Brownsville, Tx
Project Number :IBWC

Cone Penetration Test BRN-P2-16C

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489595.3
Easting: 1314085.1
Elevation: 30.3

Total Depth: 50.4 ft
Termination Criteria:
Cone Size:



BRN-P2-16C

Electronic File Name: BRN-P2-16C.cpt

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

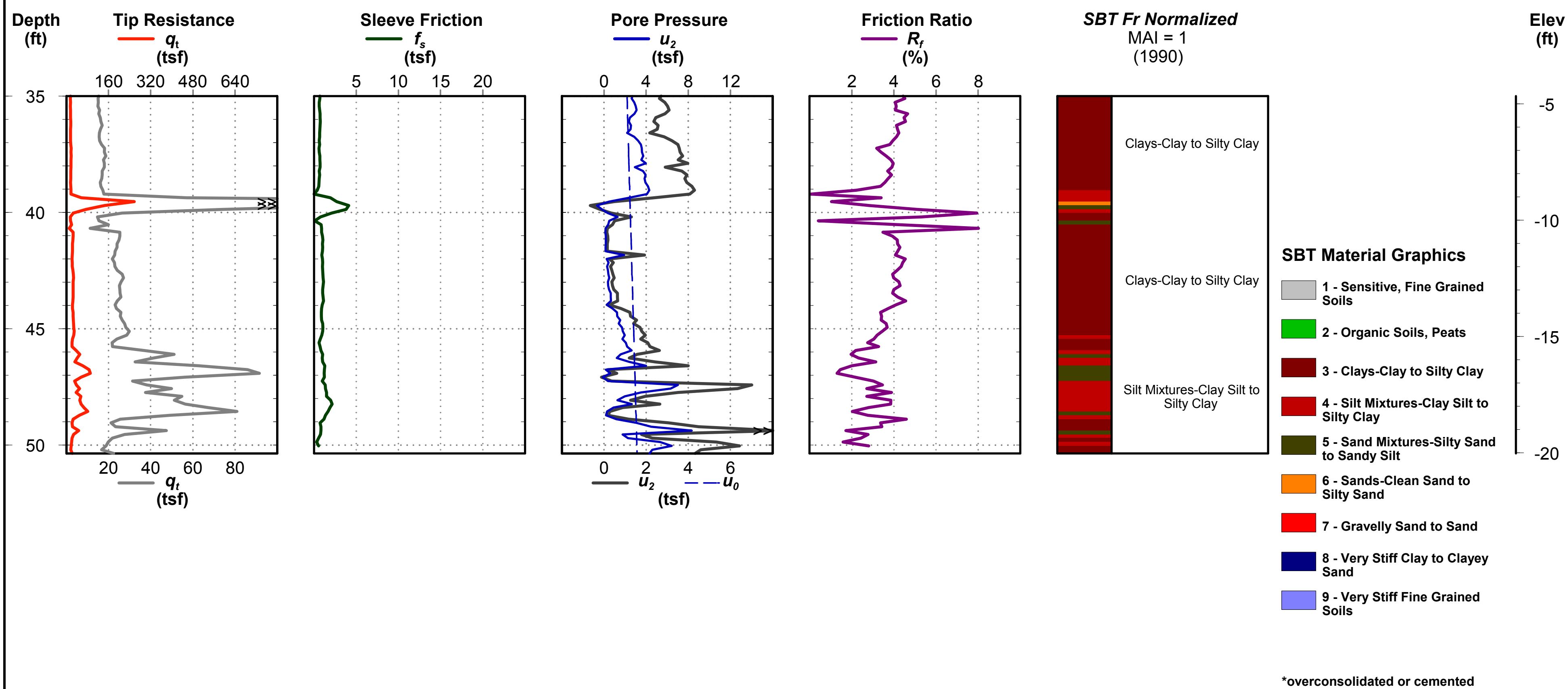
Cone Penetration Test BRN-P2-16C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489595.3
Easting: 1314085.1
Elevation: 30.3

Total Depth: 50.4 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-16C

Electronic File Name: BRN-P2-16C.cpt

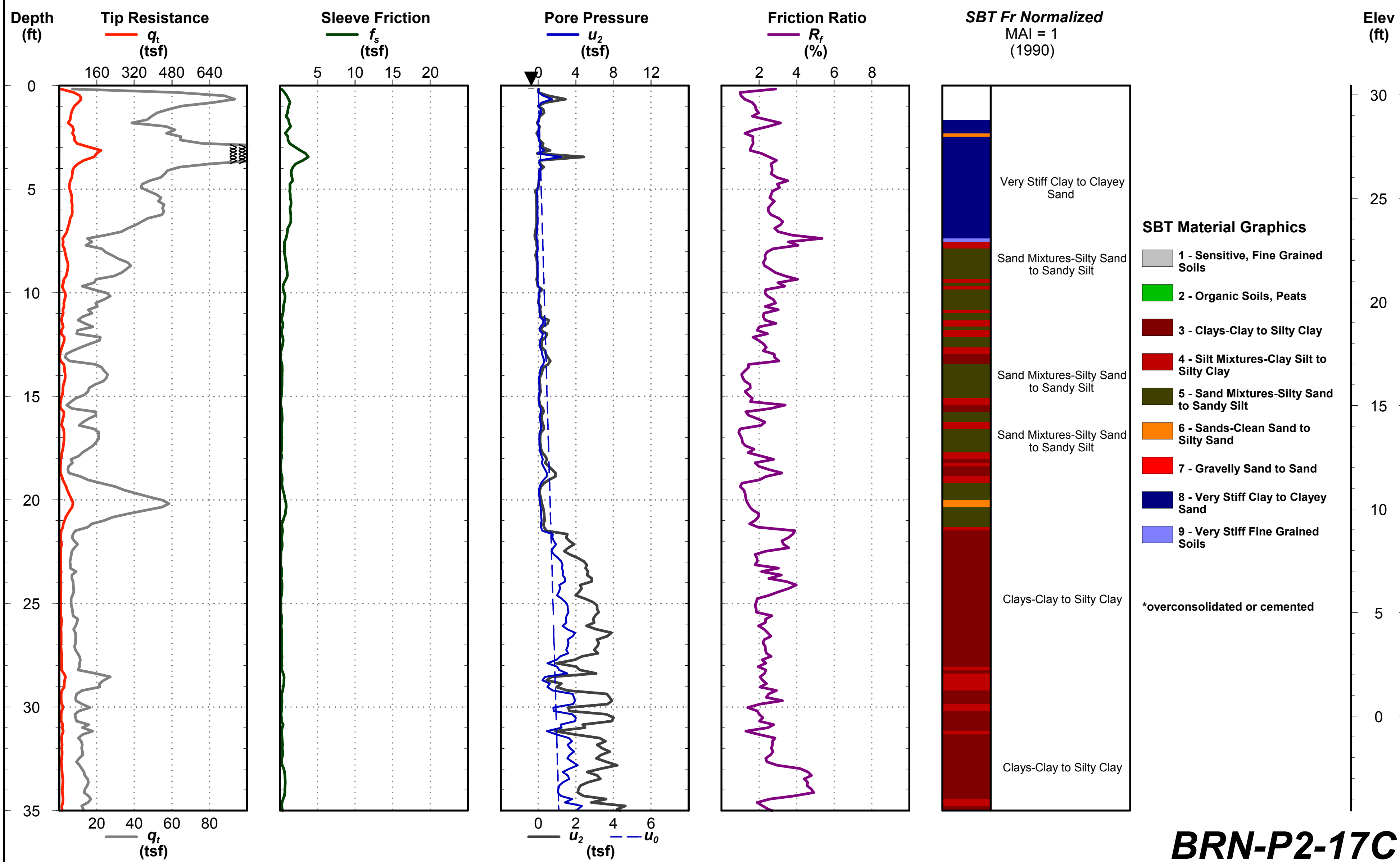
Cone Penetration Test BRN-P2-17C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489554.7
Easting: 1314100.7
Elevation: 30.4

Total Depth: 50.5 ft
Termination Criteria:
Cone Size:



BRN-P2-17C

Electronic File Name: BRN-P2-17C.cpt

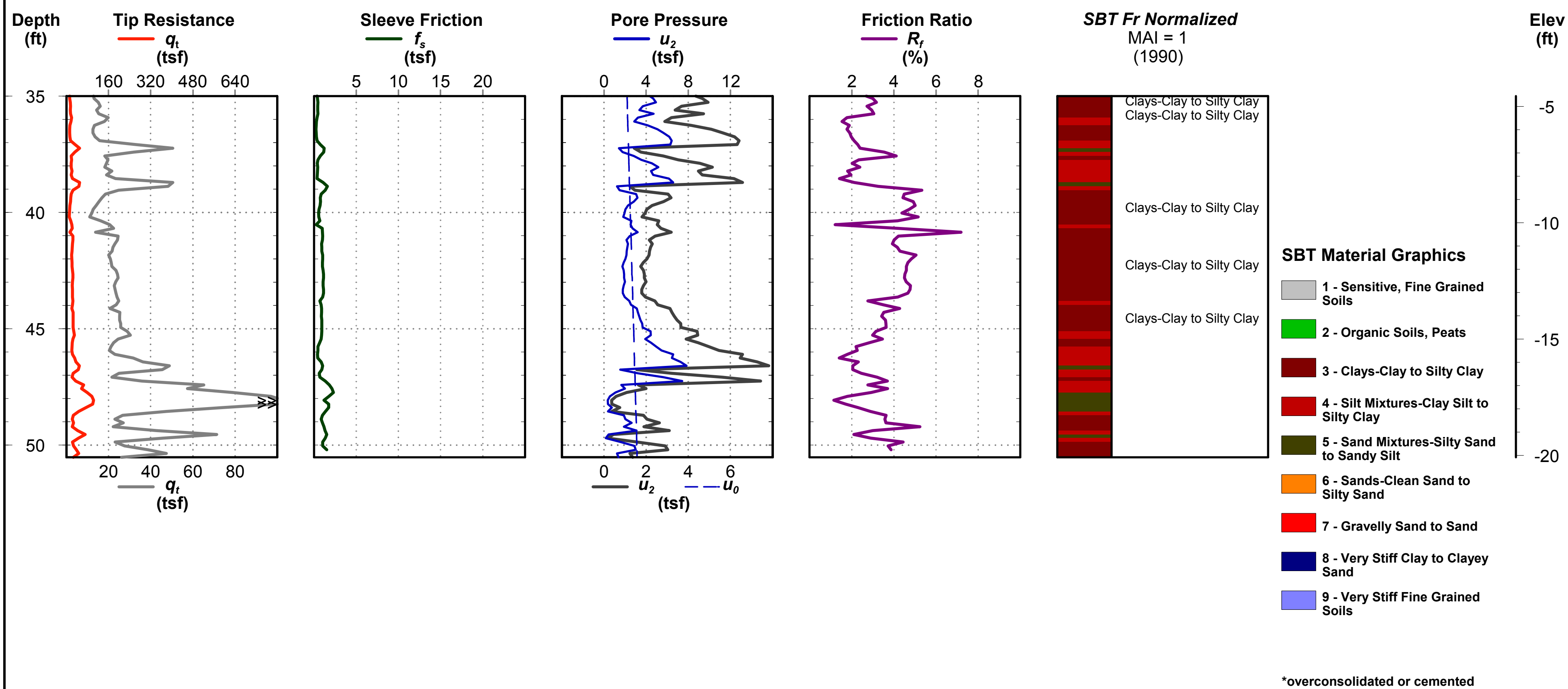
Cone Penetration Test BRN-P2-17C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489554.7
Easting: 1314100.7
Elevation: 30.4

Total Depth: 50.5 ft
Termination Criteria:
Cone Size:



BRN-P2-17C

Electronic File Name: BRN-P2-17C.cpt

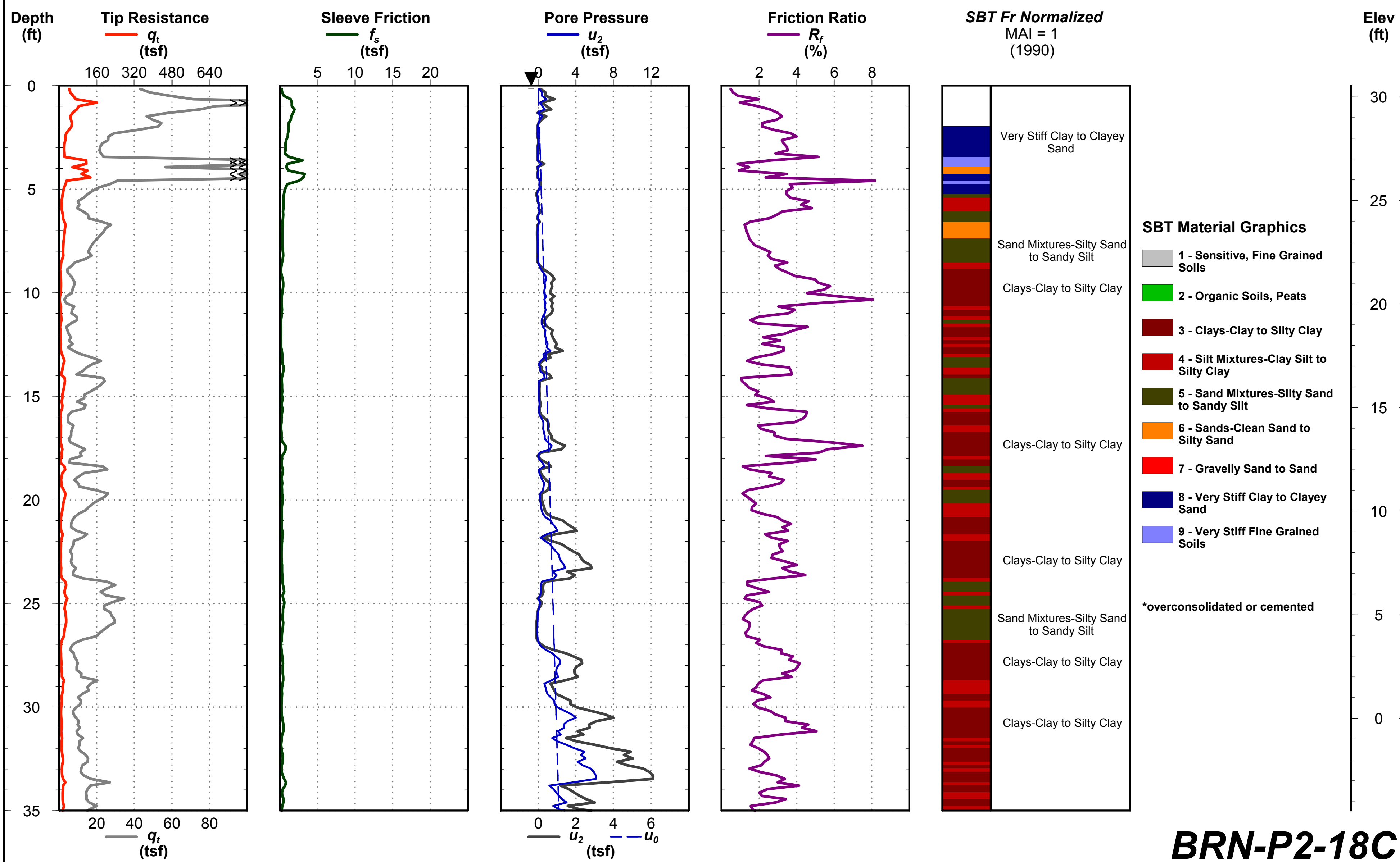
Cone Penetration Test BRN-P2-18C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489514.8
Easting: 1314139.8
Elevation: 30.5

Total Depth: 50.9 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-18C

Electronic File Name: BRN-P2-18C.cpt

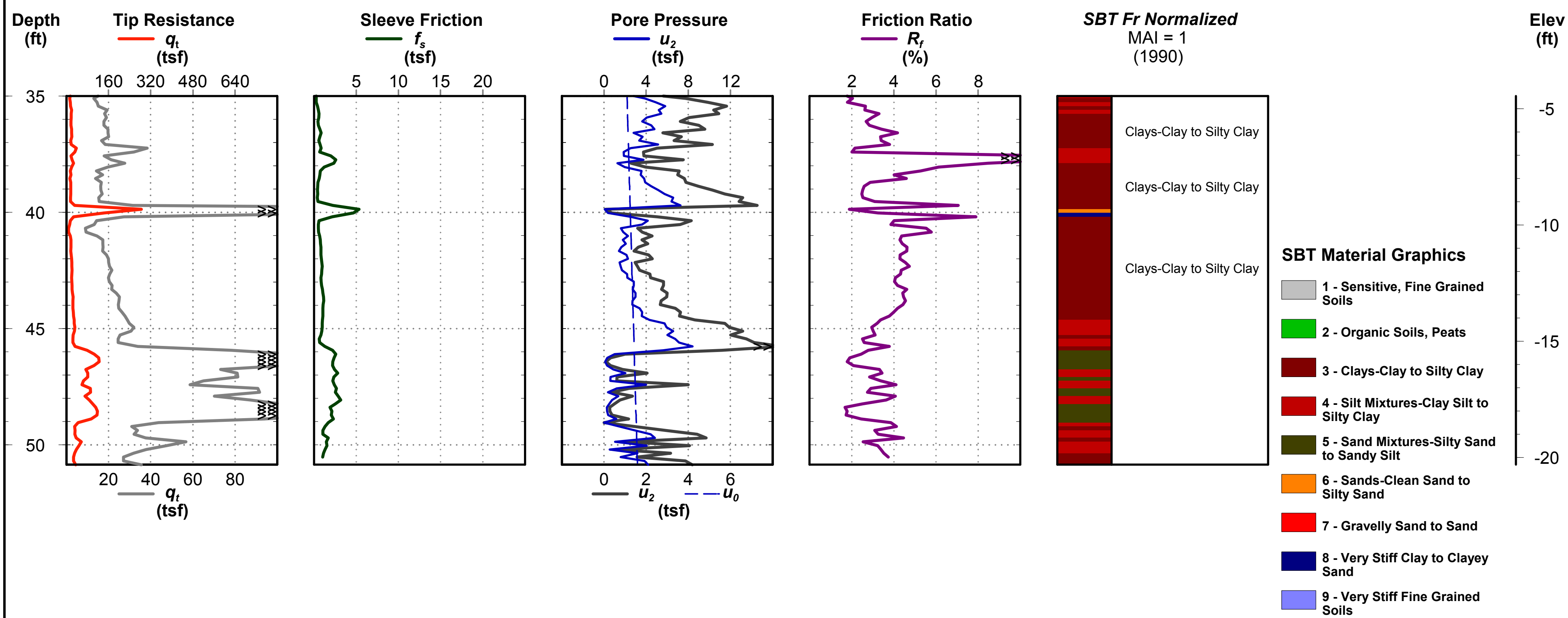
Cone Penetration Test BRN-P2-18C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489514.8
Easting: 1314139.8
Elevation: 30.5

Total Depth: 50.9 ft
Termination Criteria:
Cone Size:



*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

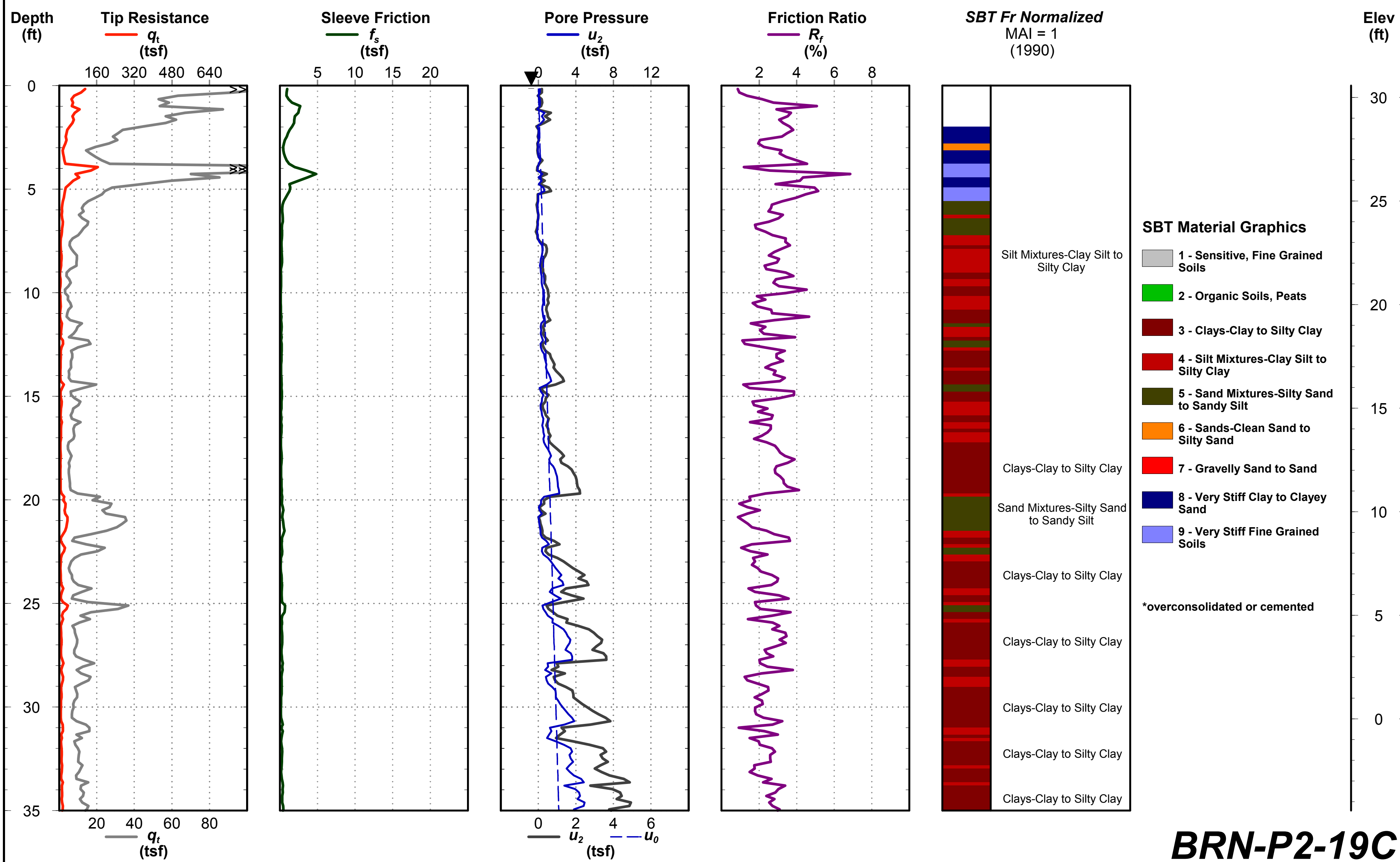
Cone Penetration Test BRN-P2-19C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489470.3
Easting: 1314166.9
Elevation: 30.6

Total Depth: 70.4 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-19C

Electronic File Name: BRN-P2-19C.cpt

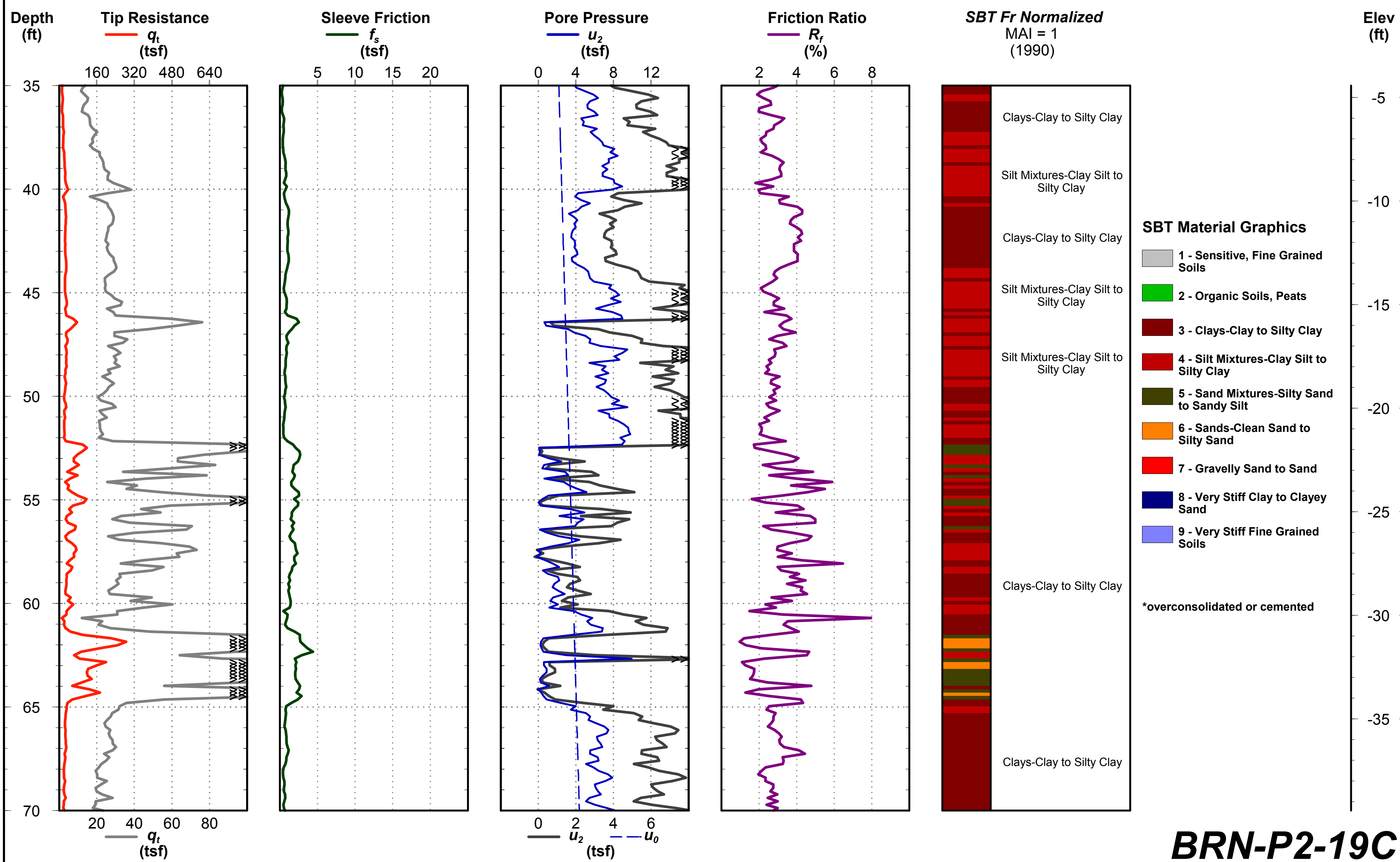
Cone Penetration Test BRN-P2-19C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489470.3
Easting: 1314166.9
Elevation: 30.6

Total Depth: 70.4 ft
Termination Criteria:
Cone Size:



BRN-P2-19C

Electronic File Name: BRN-P2-19C.cpt

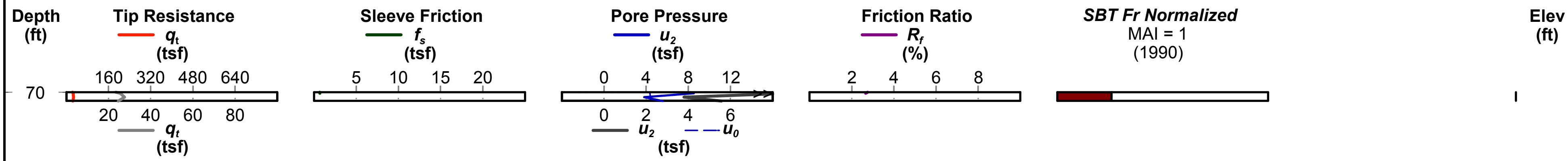
Cone Penetration Test BRN-P2-19C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489470.3
Easting: 1314166.9
Elevation: 30.6

Total Depth: 70.4 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-19C

Electronic File Name: BRN-P2-19C.cpt

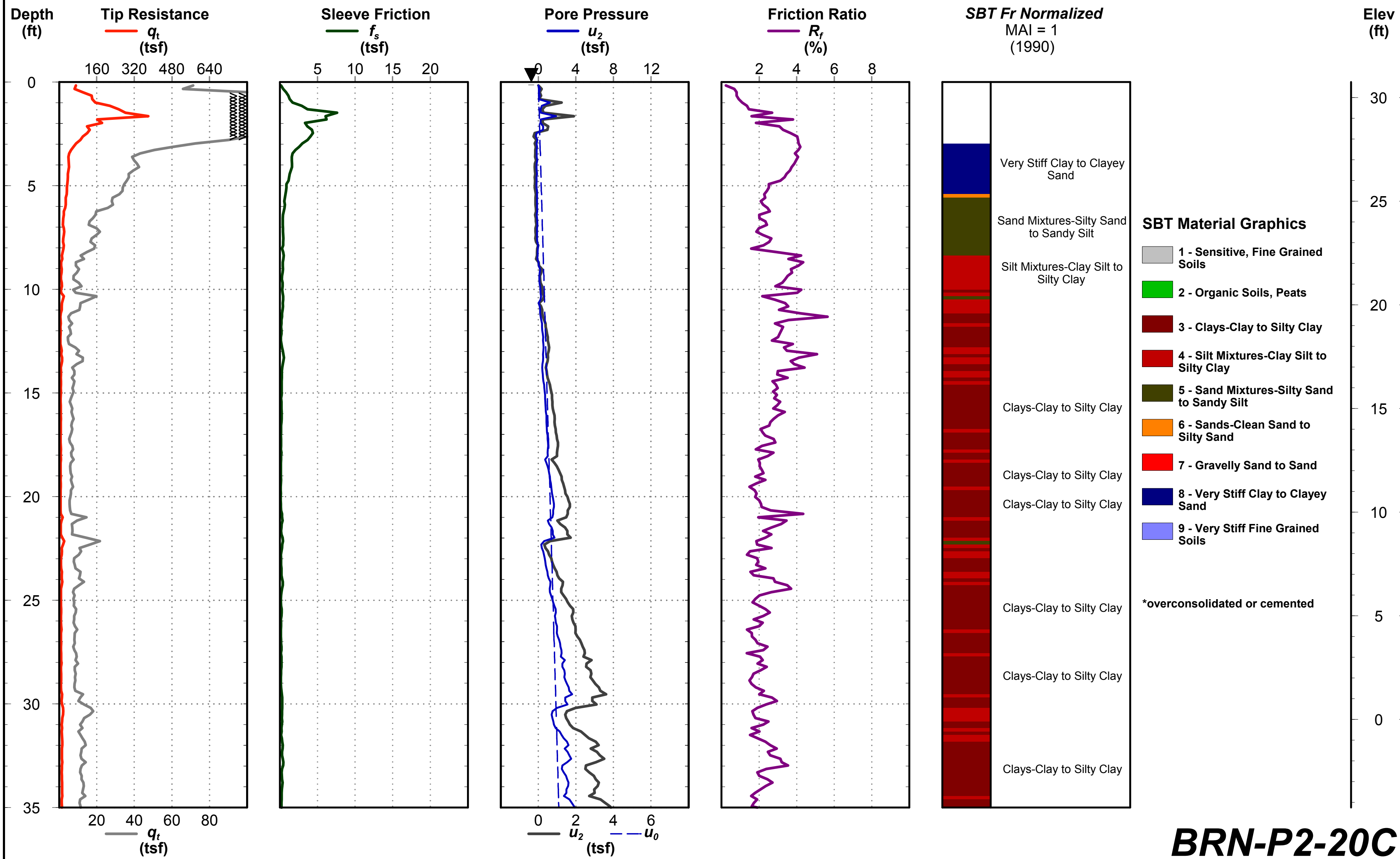
Cone Penetration Test BRN-P2-20C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489365.3
Easting: 1314186.3
Elevation: 30.7

Total Depth: 66.9 ft
Termination Criteria:
Cone Size:



BRN-P2-20C

Electronic File Name: BRN-P2-20C.cpt

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

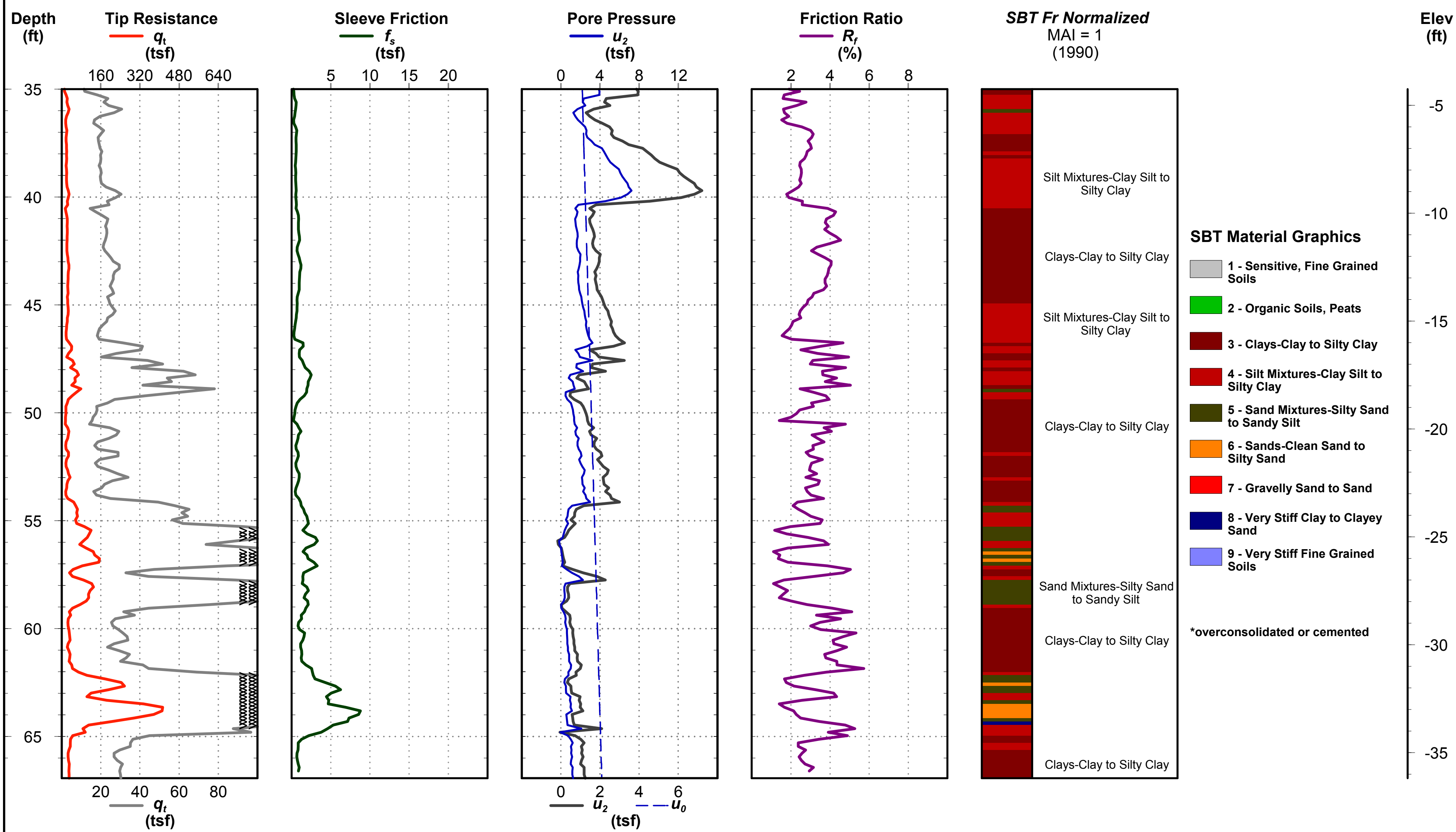
Cone Penetration Test BRN-P2-20C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489365.3
Easting: 1314186.3
Elevation: 30.7

Total Depth: 66.9 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-20C

Electronic File Name: BRN-P2-20C.cpt

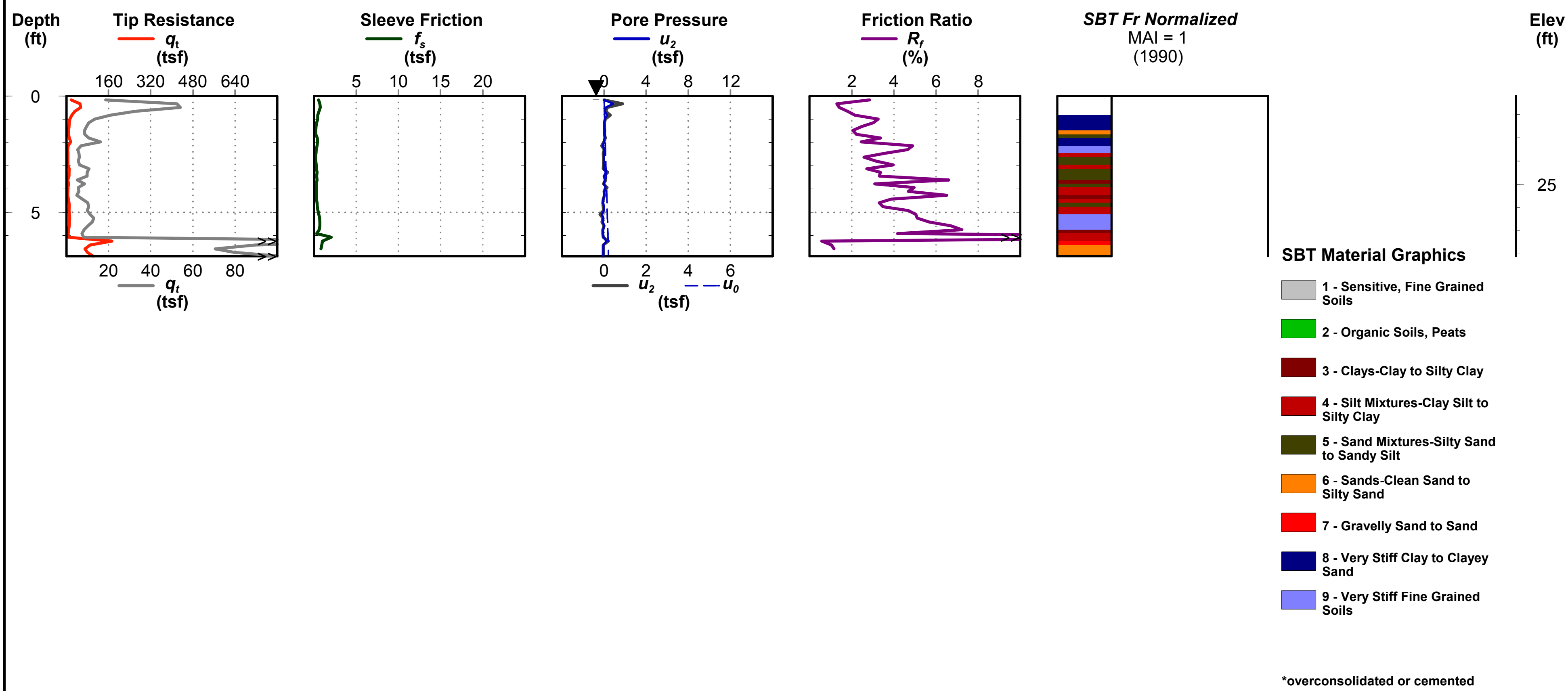
Cone Penetration Test BRN-P2-21C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489288.7
Easting: 1314209.4
Elevation: 28.8

Total Depth: 6.9 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-21C

Electronic File Name: BRN-P2-21C.cpt

ERDC

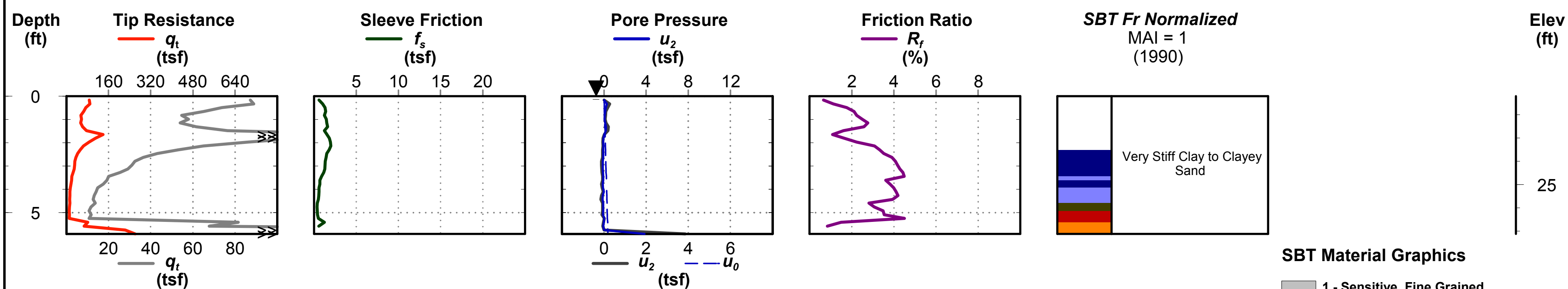
Brownsville, Tx
Project Number :IBWC

Cone Penetration Test BRN-P2-21C-a

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489288.7
Easting: 1314209.4
Elevation: 28.8

Total Depth: 5.9 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

1 - Sensitive, Fine Grained Soils
2 - Organic Soils, Peats
3 - Clays-Clay to Silty Clay
4 - Silt Mixtures-Clay Silt to Silty Clay
5 - Sand Mixtures-Silty Sand to Sandy Silt
6 - Sands-Clean Sand to Silty Sand
7 - Gravelly Sand to Sand
8 - Very Stiff Clay to Clayey Sand
9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-21C-a

Electronic File Name: BRN-P2-21C-a.cpt

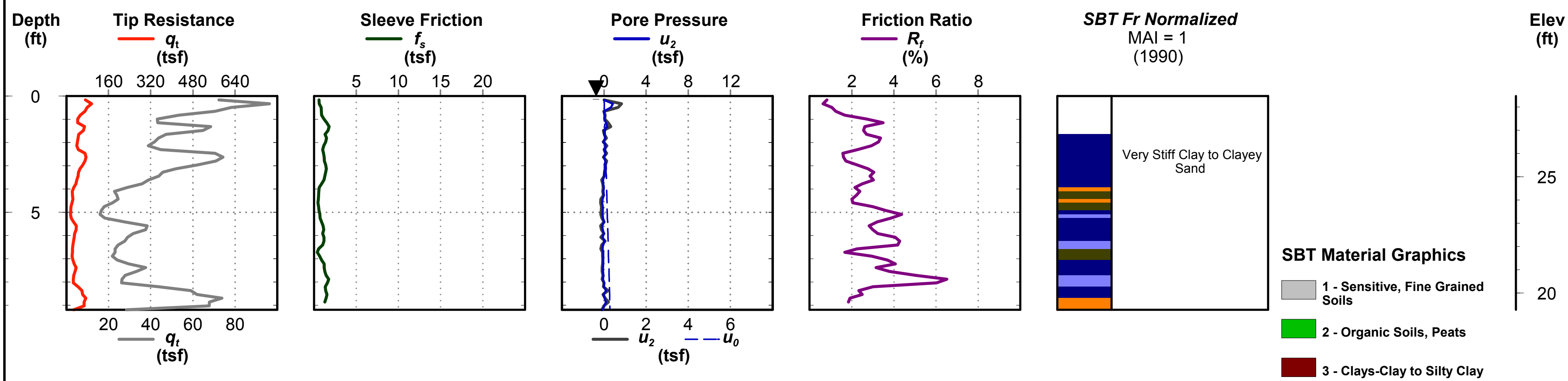
Cone Penetration Test BRN-P2-22C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489237.5
Easting: 1314233.1
Elevation: 28.5

Total Depth: 9.2 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

1 - Sensitive, Fine Grained Soils
2 - Organic Soils, Peats
3 - Clays-Clay to Silty Clay
4 - Silt Mixtures-Clay Silt to Silty Clay
5 - Sand Mixtures-Silty Sand to Sandy Silt
6 - Sands-Clean Sand to Silty Sand
7 - Gravelly Sand to Sand
8 - Very Stiff Clay to Clayey Sand
9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-22C

Electronic File Name: BRN-P2-22C.cpt

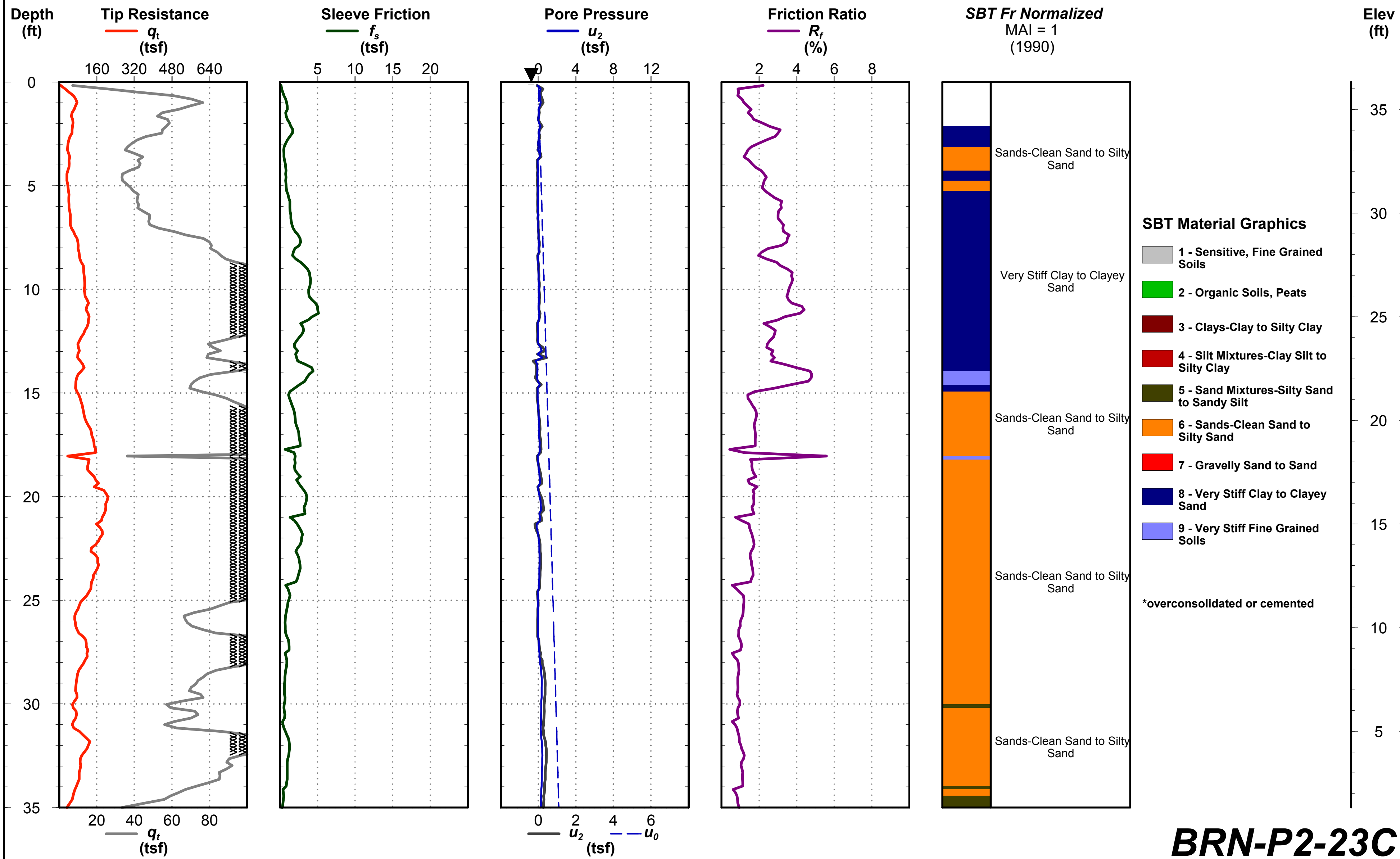
Cone Penetration Test BRN-P2-23C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16488789.9
Easting: 1314344.0
Elevation: 36.3

Total Depth: 53.8 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-23C

Electronic File Name: BRN-P2-23C.cpt

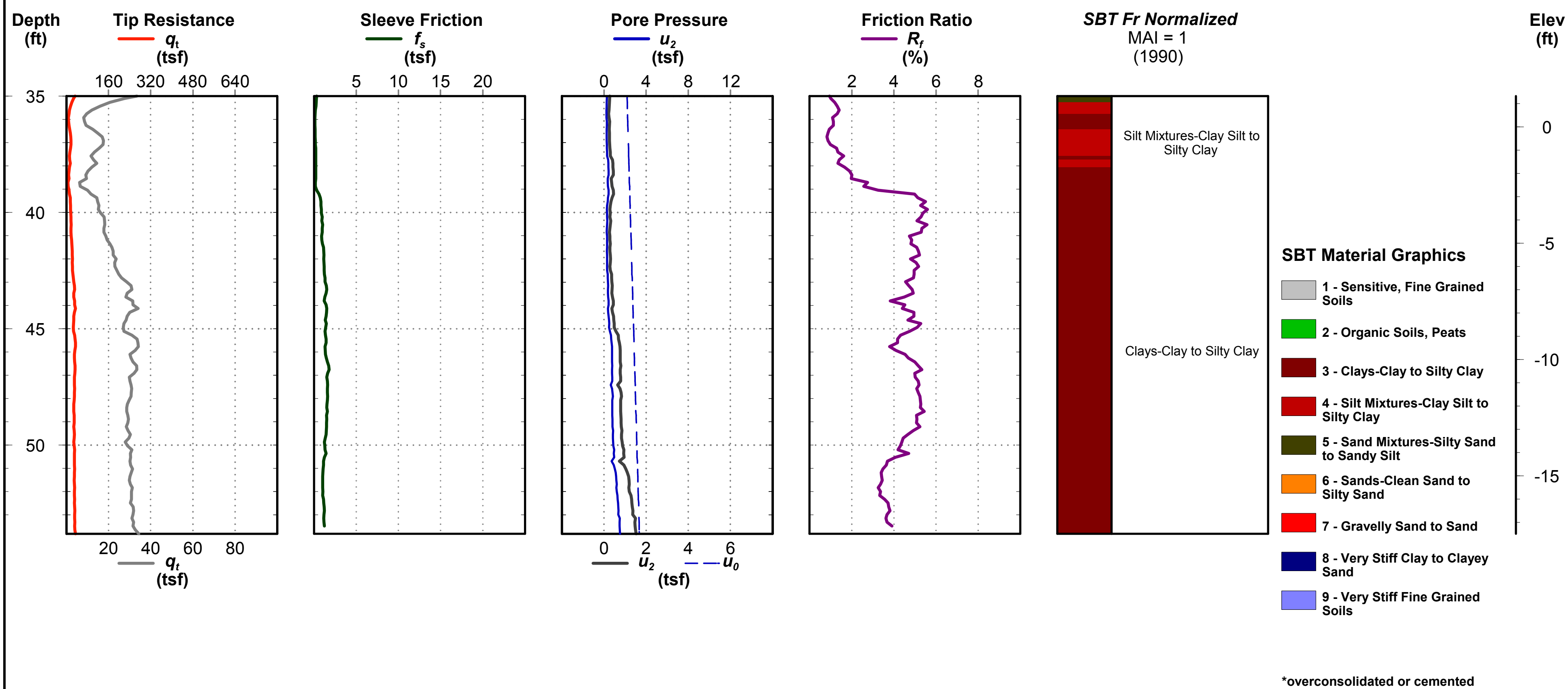
Cone Penetration Test BRN-P2-23C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16488789.9
Easting: 1314344.0
Elevation: 36.3

Total Depth: 53.8 ft
Termination Criteria:
Cone Size:



BRN-P2-23C

Electronic File Name: BRN-P2-23C.cpt

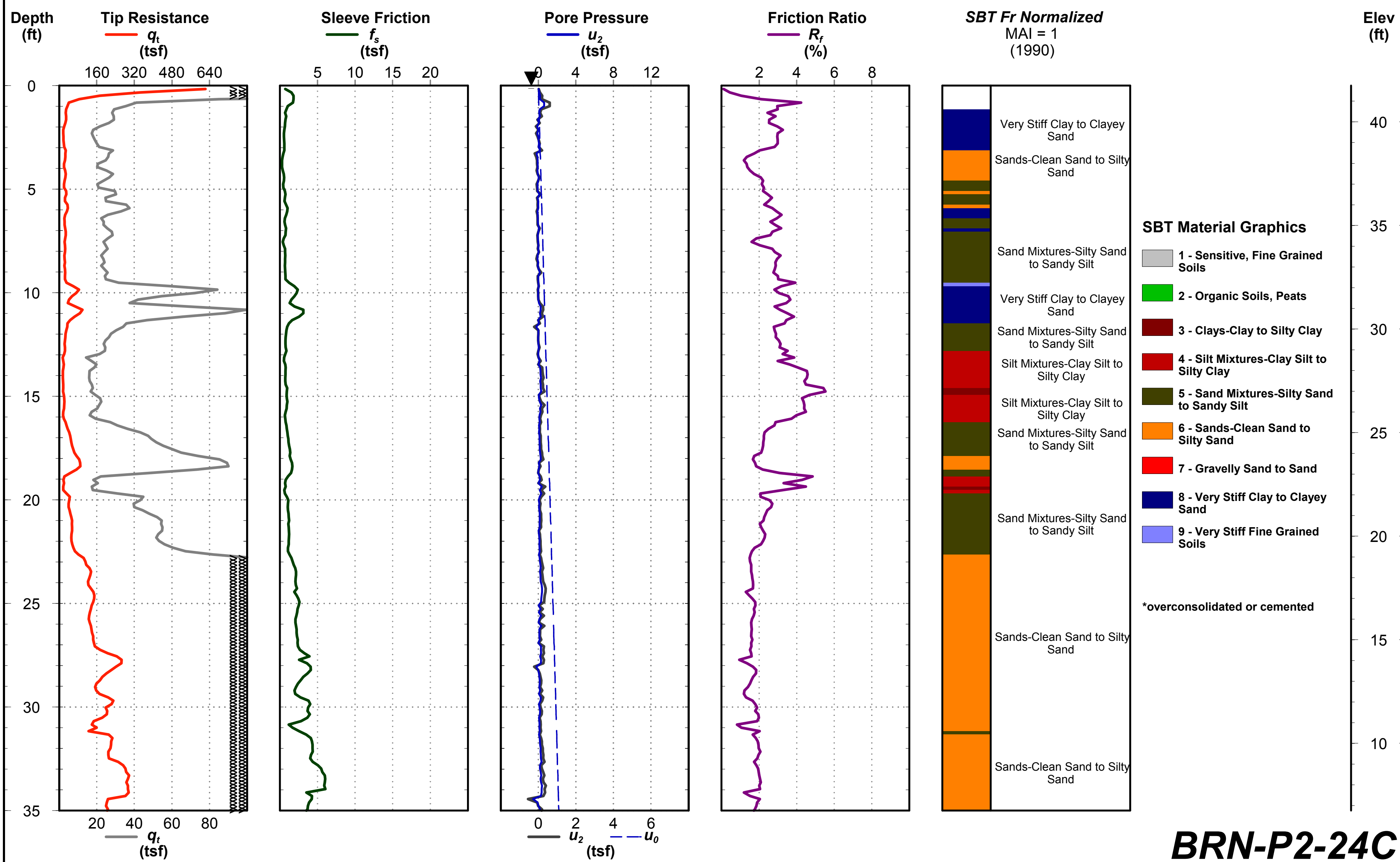
Cone Penetration Test BRN-P2-24C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16488798.1
Easting: 1314388.9
Elevation: 41.8

Total Depth: 63.7 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-24C

Electronic File Name: BRN-P2-24C.cpt

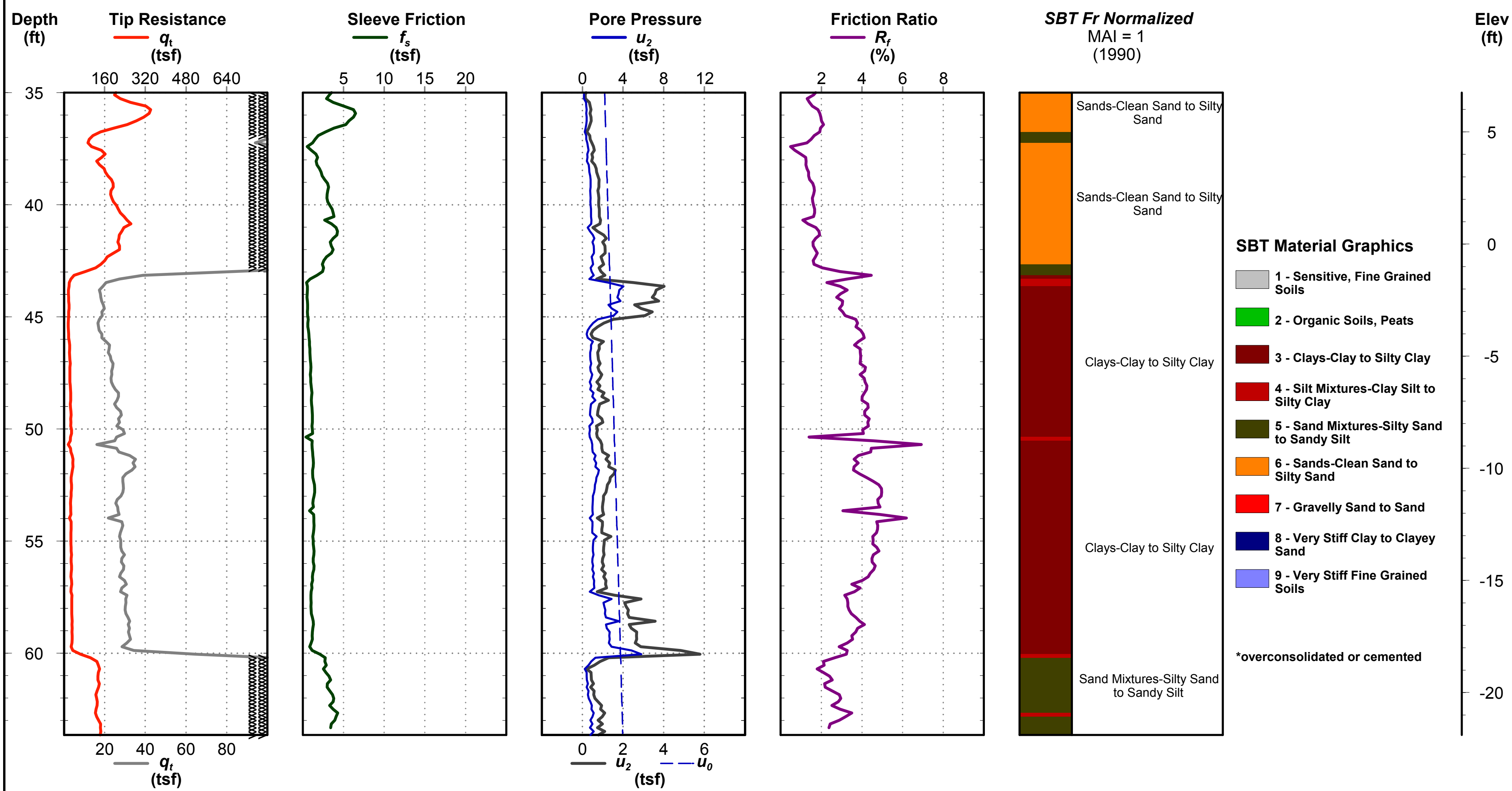
Cone Penetration Test BRN-P2-24C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16488798.1
Easting: 1314388.9
Elevation: 41.8

Total Depth: 63.7 ft
Termination Criteria:
Cone Size:



BRN-P2-24C

Electronic File Name: BRN-P2-24C.cpt

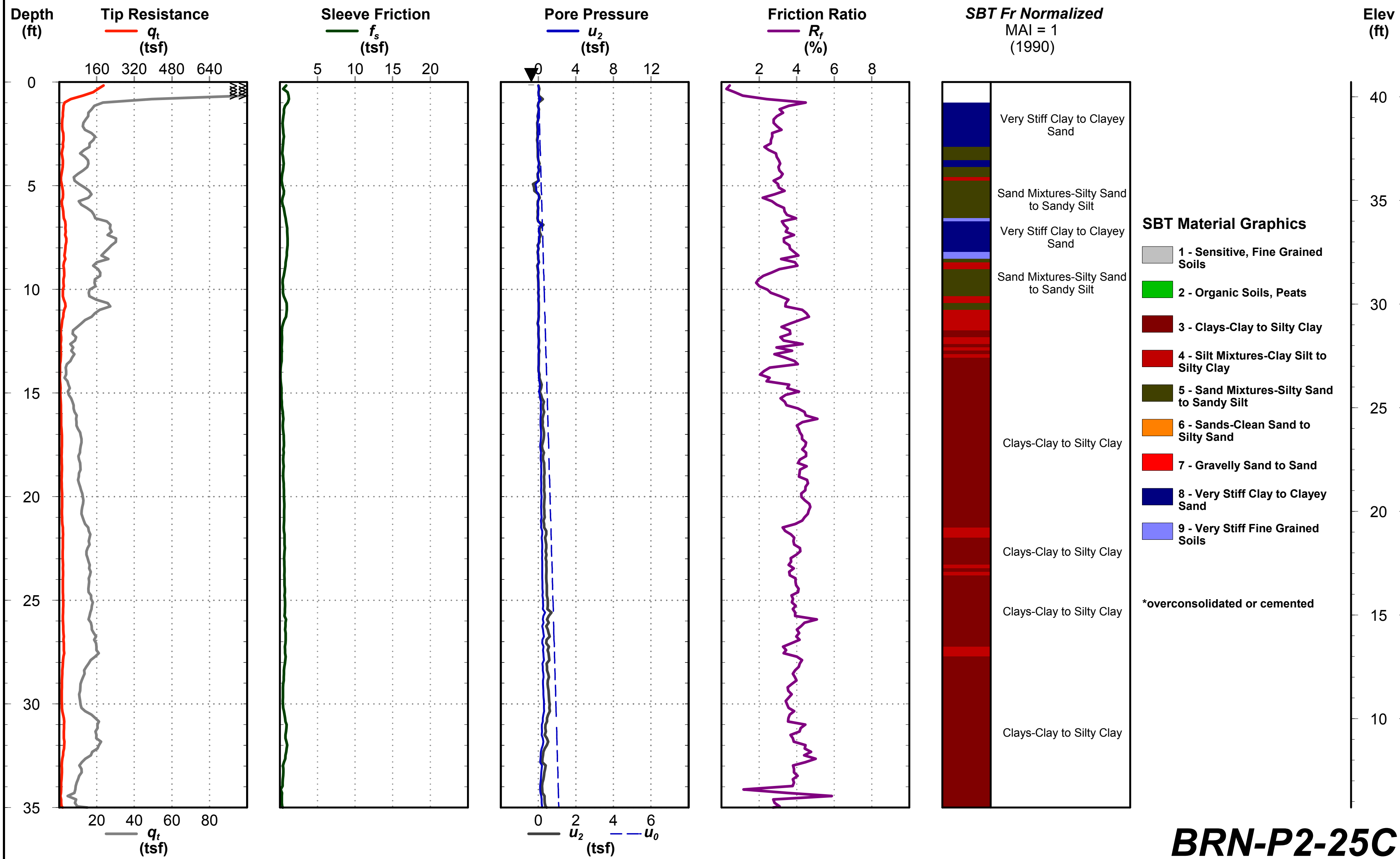
Cone Penetration Test BRN-P2-25C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489409.2
Easting: 1314252.4
Elevation: 40.7

Total Depth: 62.3 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-25C

Electronic File Name: BRN-P2-25C.cpt

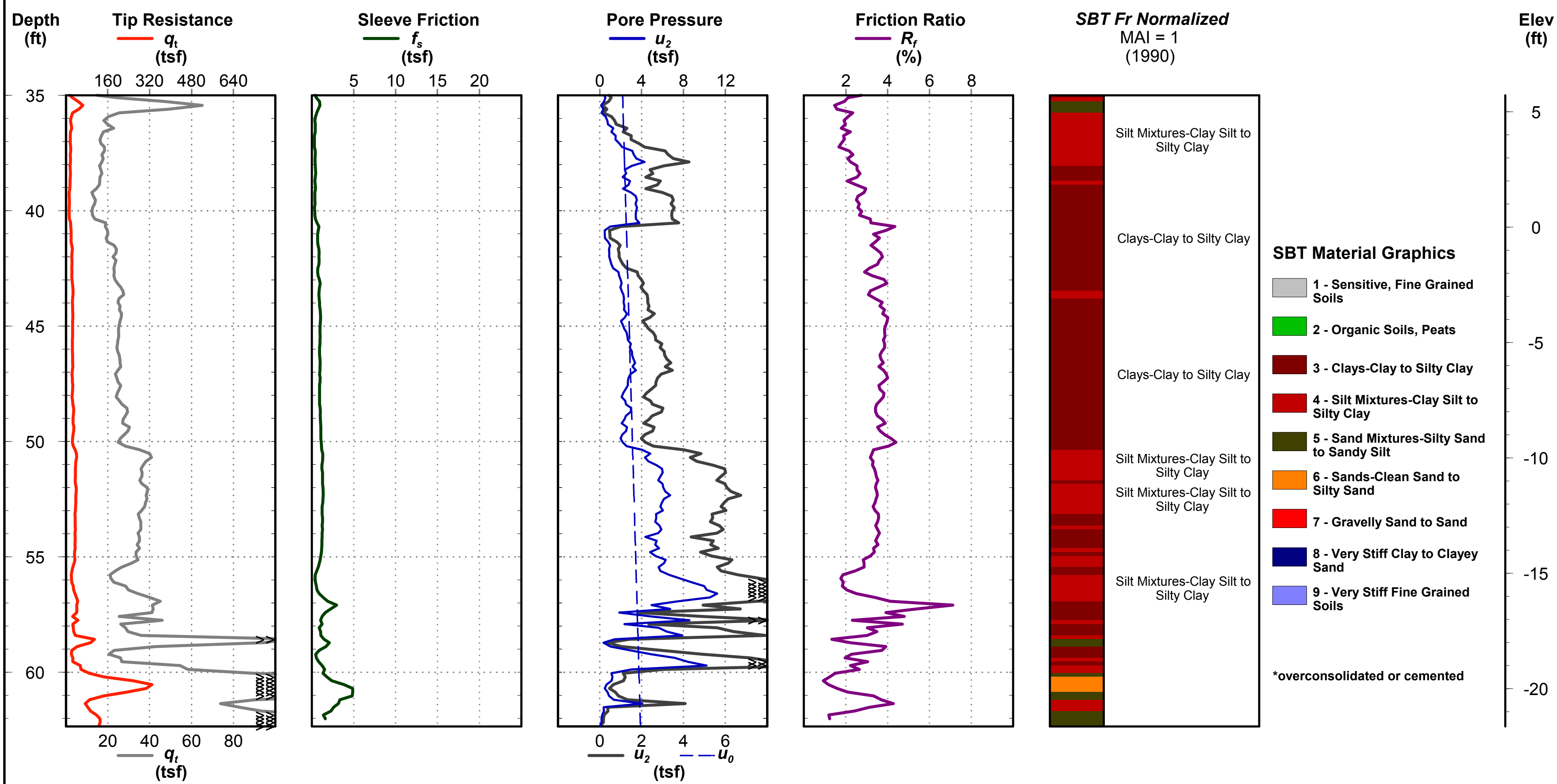
Cone Penetration Test BRN-P2-25C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489409.2
Easting: 1314252.4
Elevation: 40.7

Total Depth: 62.3 ft
Termination Criteria:
Cone Size:



BRN-P2-25C

Electronic File Name: BRN-P2-25C.cpt

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

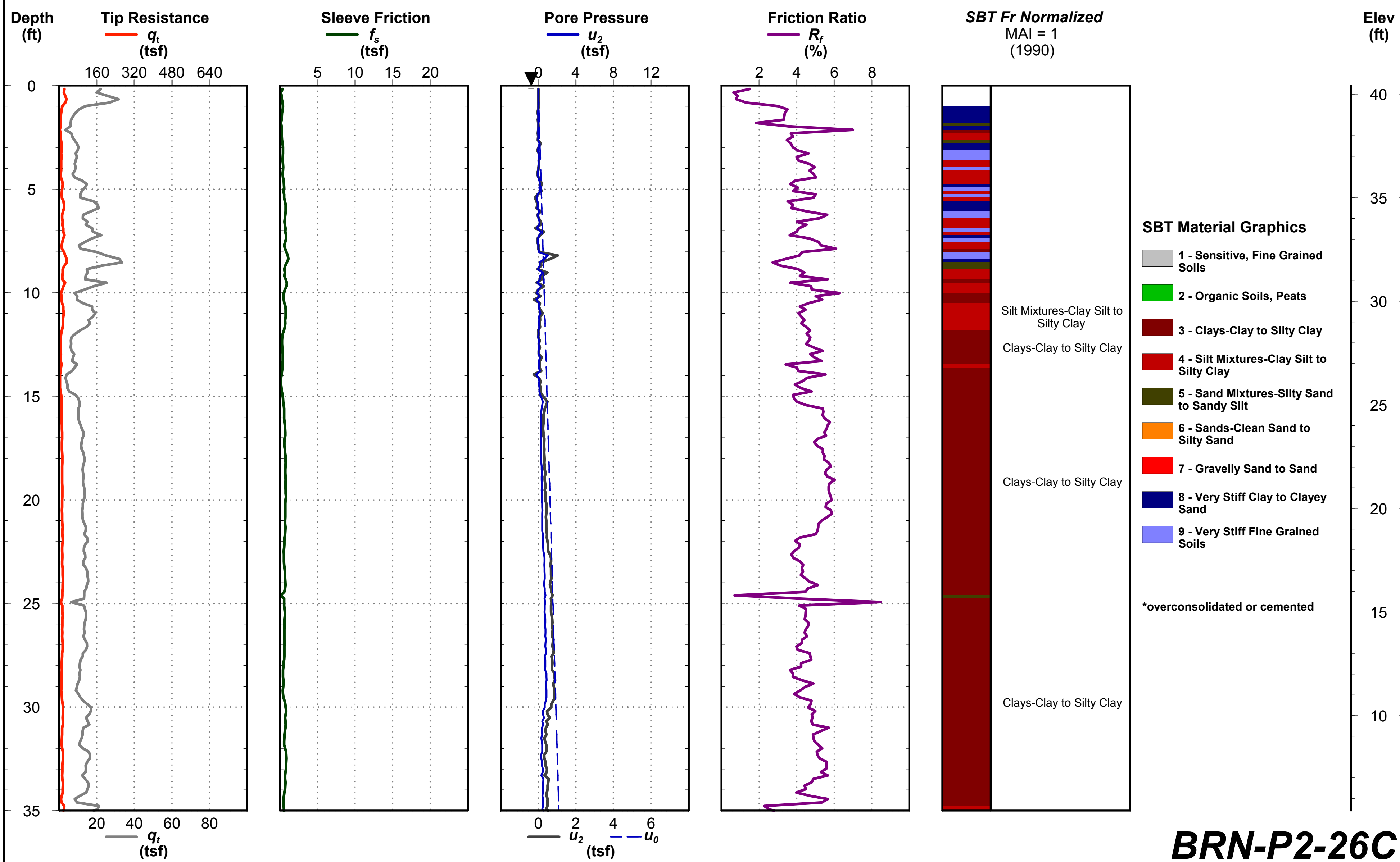
Cone Penetration Test BRN-P2-26C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489591.3
Easting: 1314179.4
Elevation: 40.4

Total Depth: 69.4 ft
Termination Criteria:
Cone Size:



BRN-P2-26C

Electronic File Name: BRN-P2-26C.cpt

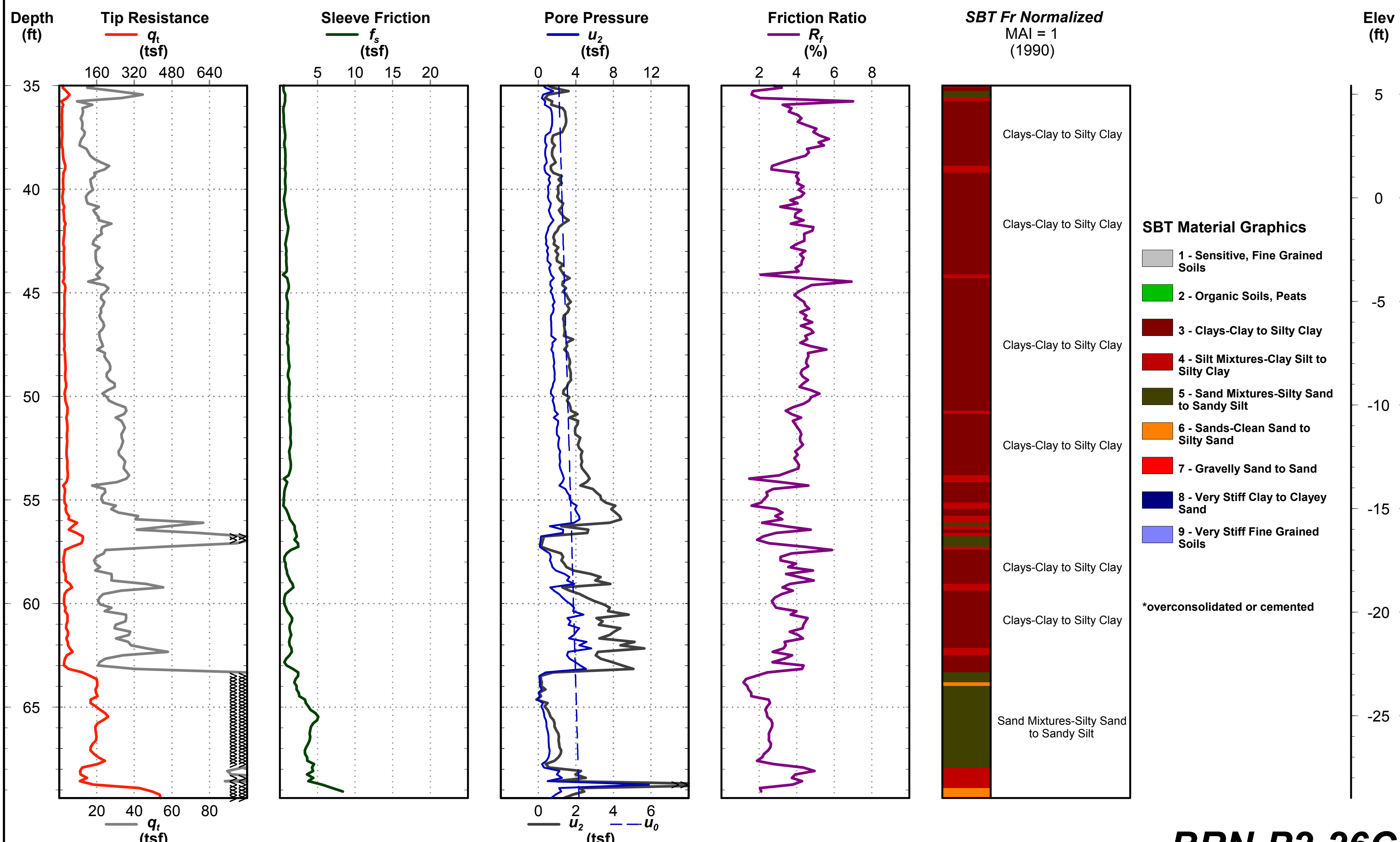
Cone Penetration Test BRN-P2-26C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489591.3
Easting: 1314179.4
Elevation: 40.4

Total Depth: 69.4 ft
Termination Criteria:
Cone Size:



BRN-P2-26C

Electronic File Name: BRN-P2-26C.cpt

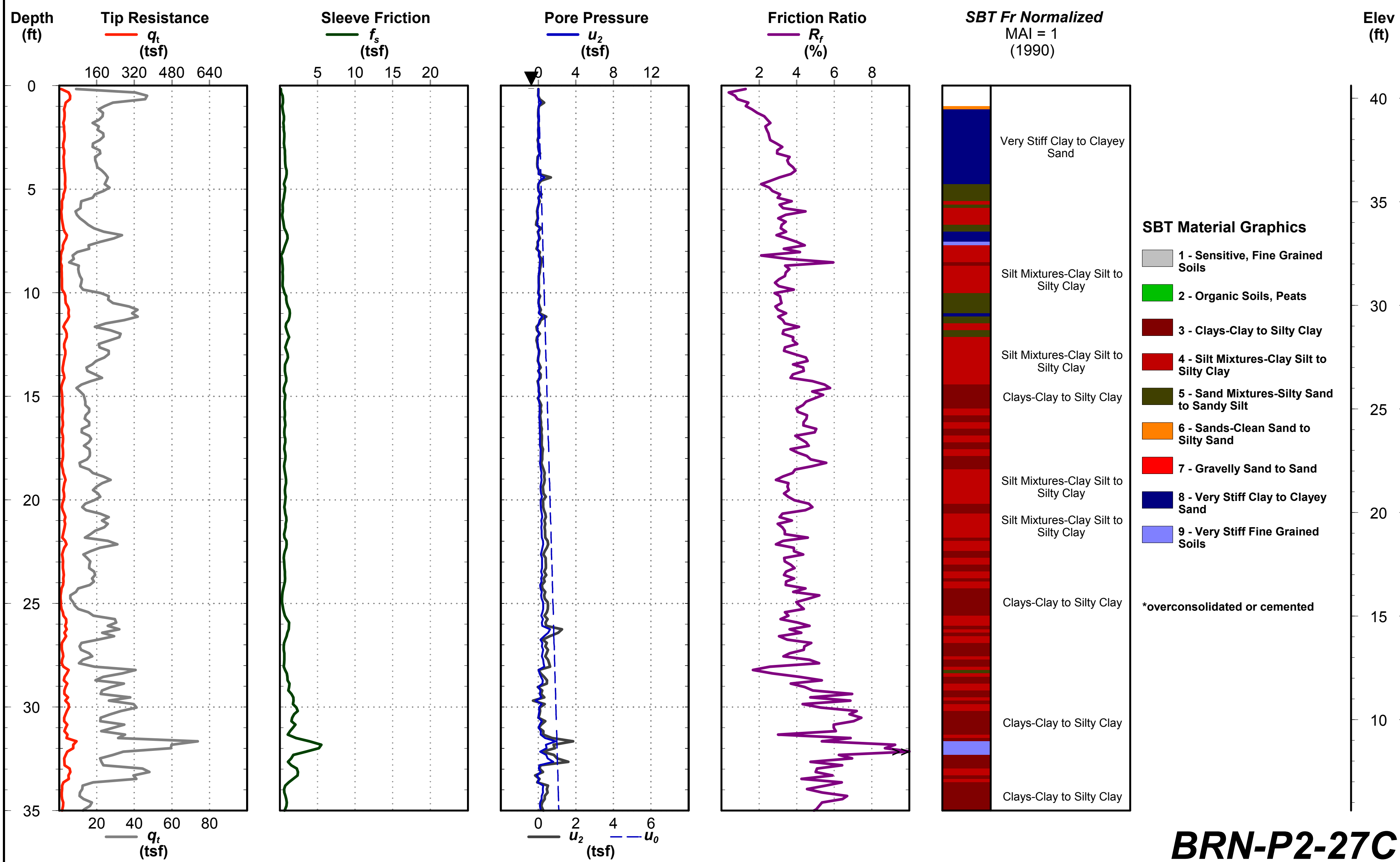
Cone Penetration Test BRN-P2-27C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489584.4
Easting: 1314165.2
Elevation: 40.6

Total Depth: 73.5 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-27C

Electronic File Name: BRN-P2-27C.cpt

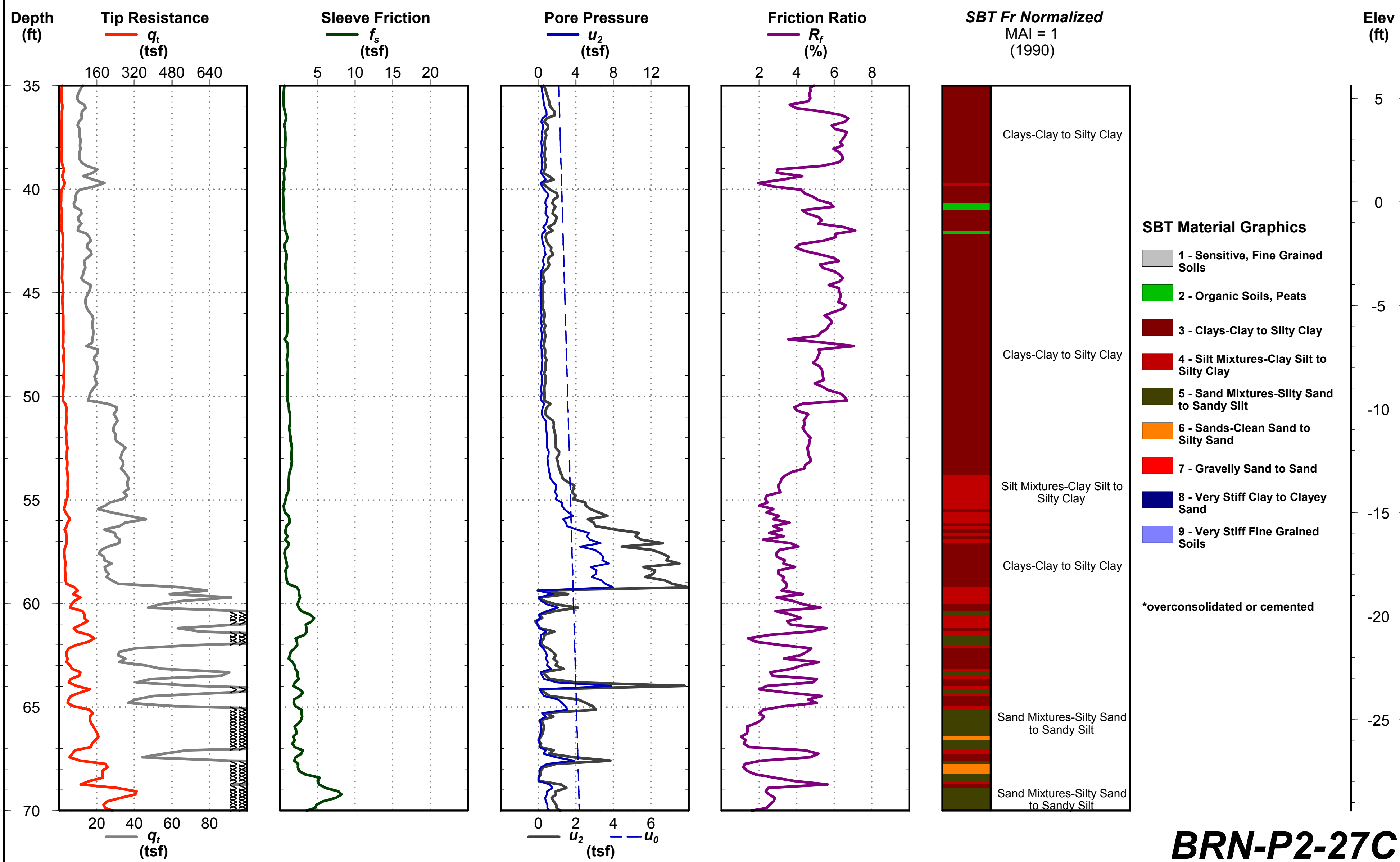
Cone Penetration Test BRN-P2-27C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489584.4
Easting: 1314165.2
Elevation: 40.6

Total Depth: 73.5 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-27C

Electronic File Name: BRN-P2-27C.cpt

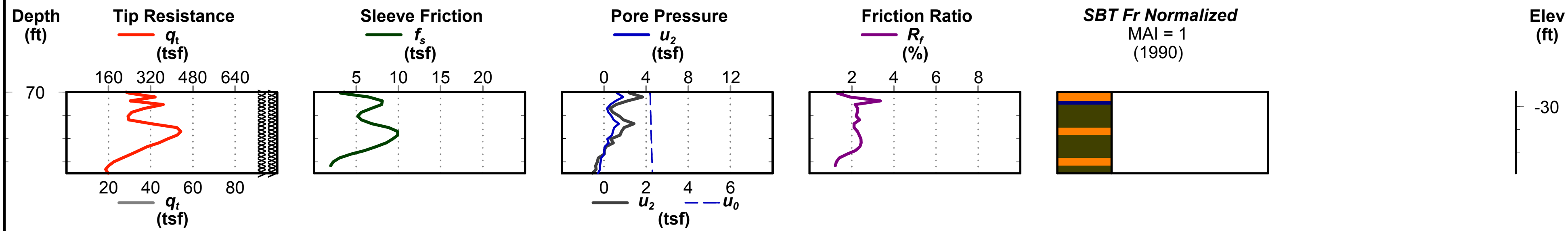
Cone Penetration Test BRN-P2-27C

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489584.4
Easting: 1314165.2
Elevation: 40.6

Total Depth: 73.5 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-27C

Electronic File Name: BRN-P2-27C.cpt

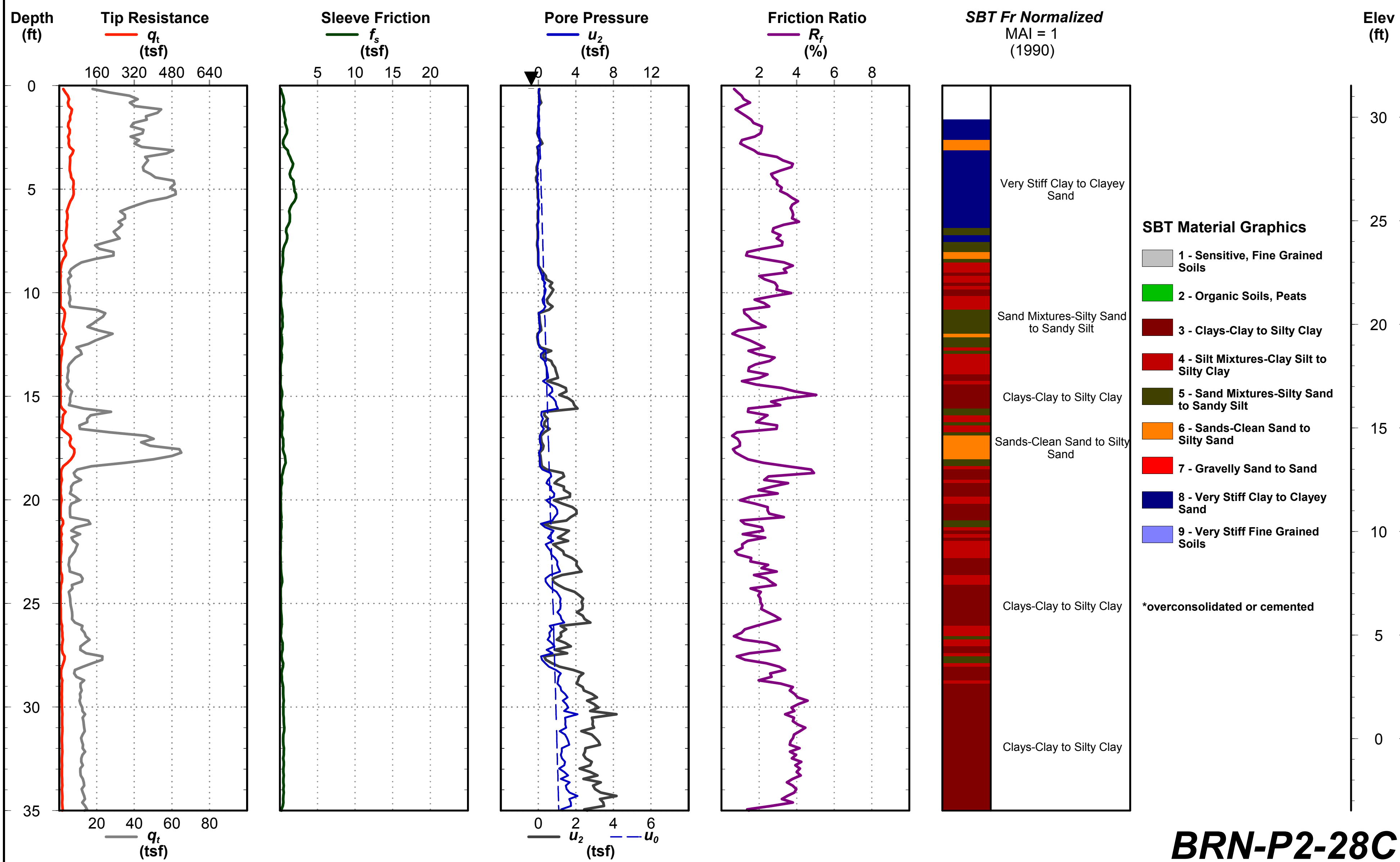
Cone Penetration Test BRN-P2-28C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489571.1
Easting: 1314138.7
Elevation: 31.5

Total Depth: 50.9 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-28C

Electronic File Name: BRN-P2-28C.cpt

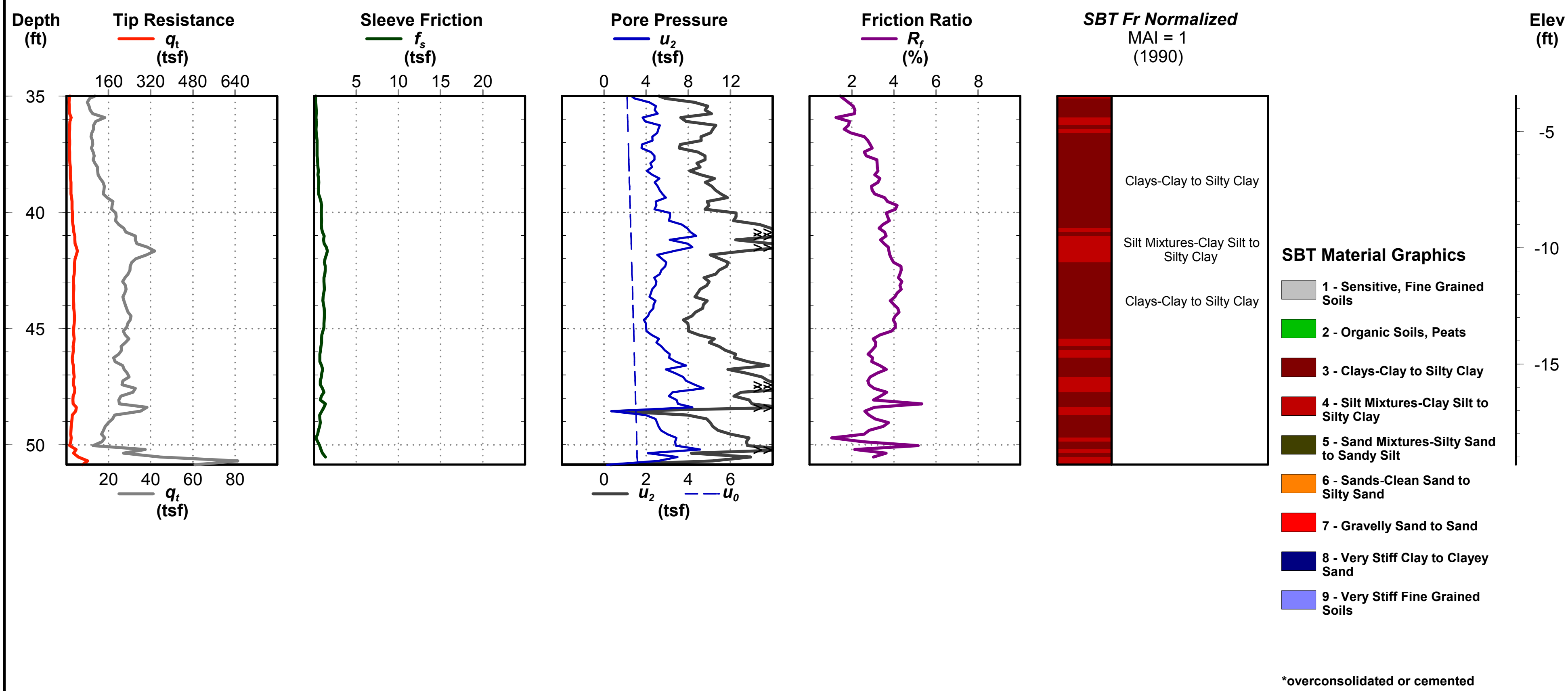
Brownsville, Tx
Project Number :IBWC

Cone Penetration Test BRN-P2-28C

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489571.1
Easting: 1314138.7
Elevation: 31.5

Total Depth: 50.9 ft
Termination Criteria:
Cone Size:



CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

BRN-P2-28C

Electronic File Name: BRN-P2-28C.cpt

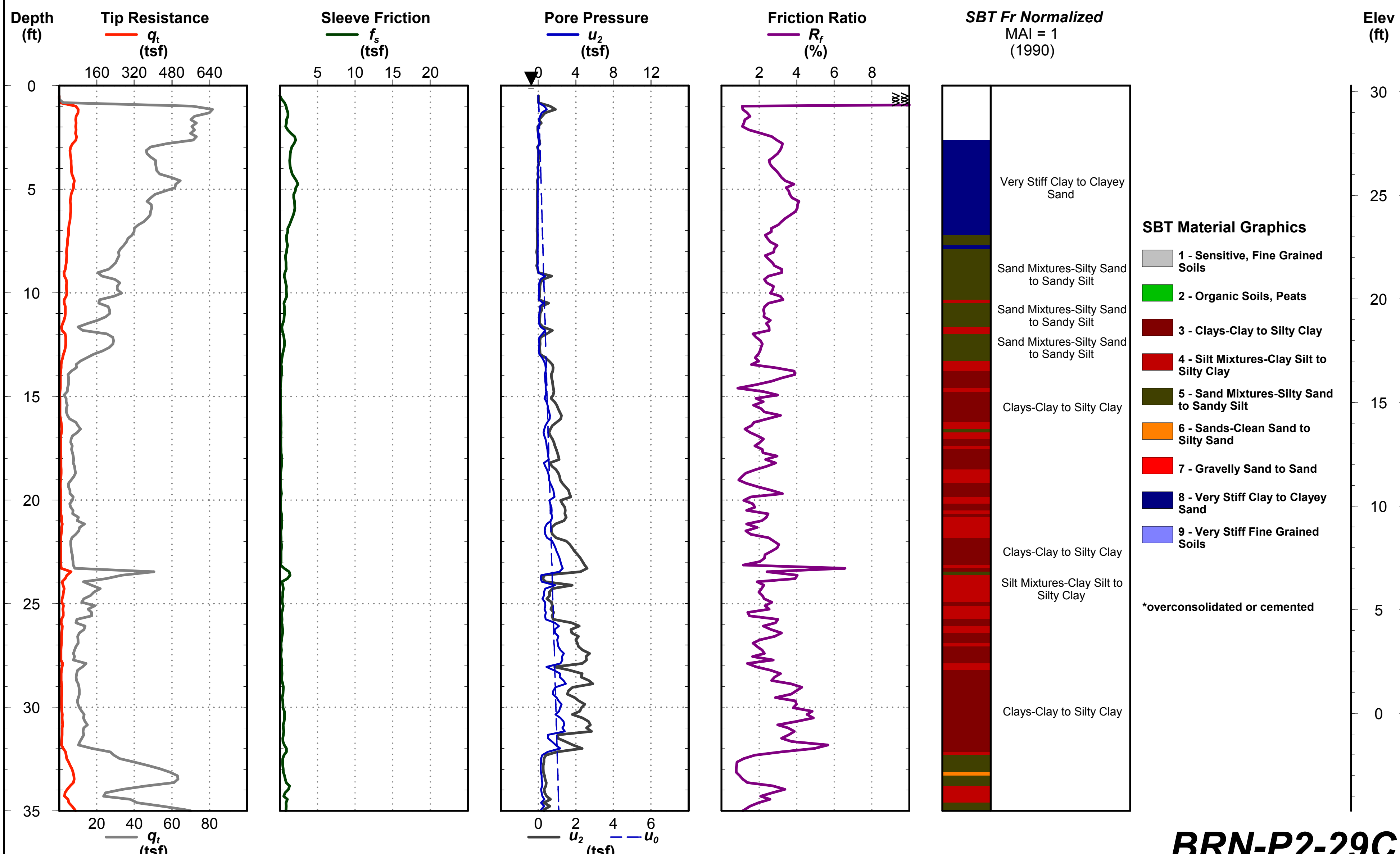
Cone Penetration Test BRN-P2-29C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489548.2
Easting: 1314088.2
Elevation: 30.3

Total Depth: 50.7 ft
Termination Criteria:
Cone Size:



BRN-P2-29C

Electronic File Name: BRN-P2-29C.cpt

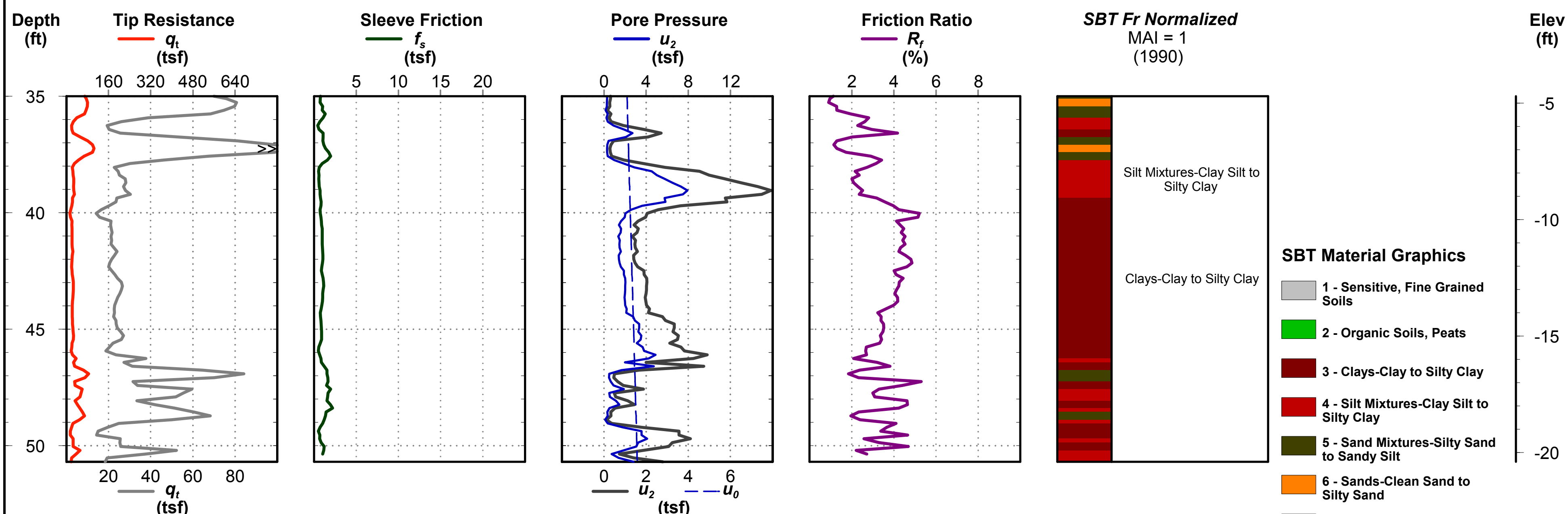
Cone Penetration Test BRN-P2-29C

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489548.2
Easting: 1314088.2
Elevation: 30.3

Total Depth: 50.7 ft
Termination Criteria:
Cone Size:



SBT Material Graphics

- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

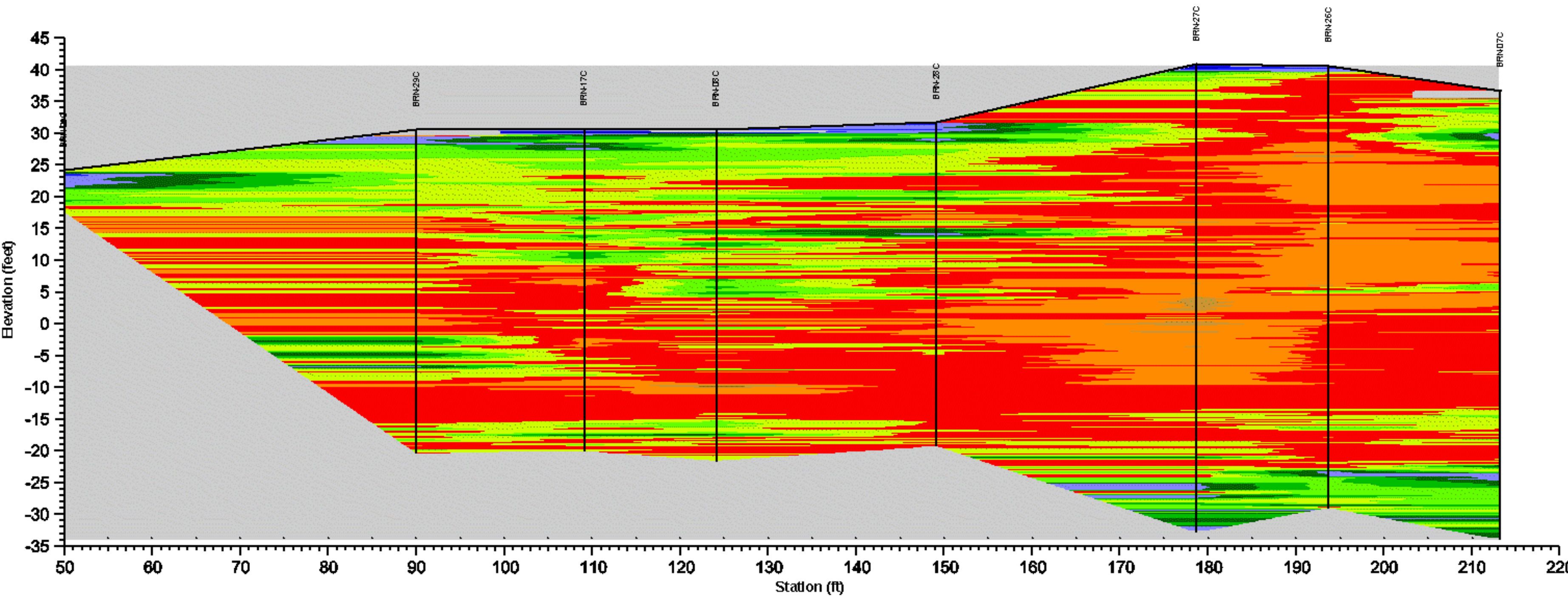
*overconsolidated or cemented

CPT REPORT - STANDARD WITH LEGEND BRN.GPJ CPT V3.0.GDT 8/15/14

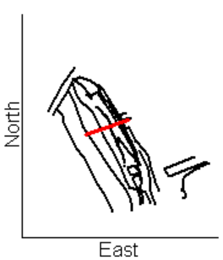
BRN-P2-29C

Electronic File Name: BRN-P2-29C.cpt

Appendix C: CPT Profiles



(this cross section is 163 feet long)



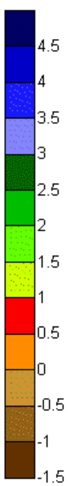
Cross Section:
R1

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

CPT Predicted Soil classification (SCN)

(non linear normalized ERDC 07 version)

In Situ anomaly causing data processing error



SP to GM

SP and SM

SP to SM
SP-SM SAND, with less than 5% fines

ML to SM

ML (by defination)

Clay mixtures, influenced by silt interaction
BOUNDRY for Silty clay to clayey mixtures
Clays NOT influenced by silt interaction

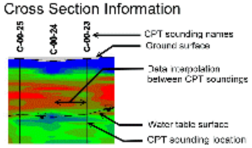
CL to CH clays

CH and CL clays (and NC organic clays)

CH clays, organic clays, or peats

Organic clays to Peats

Cross Section Information

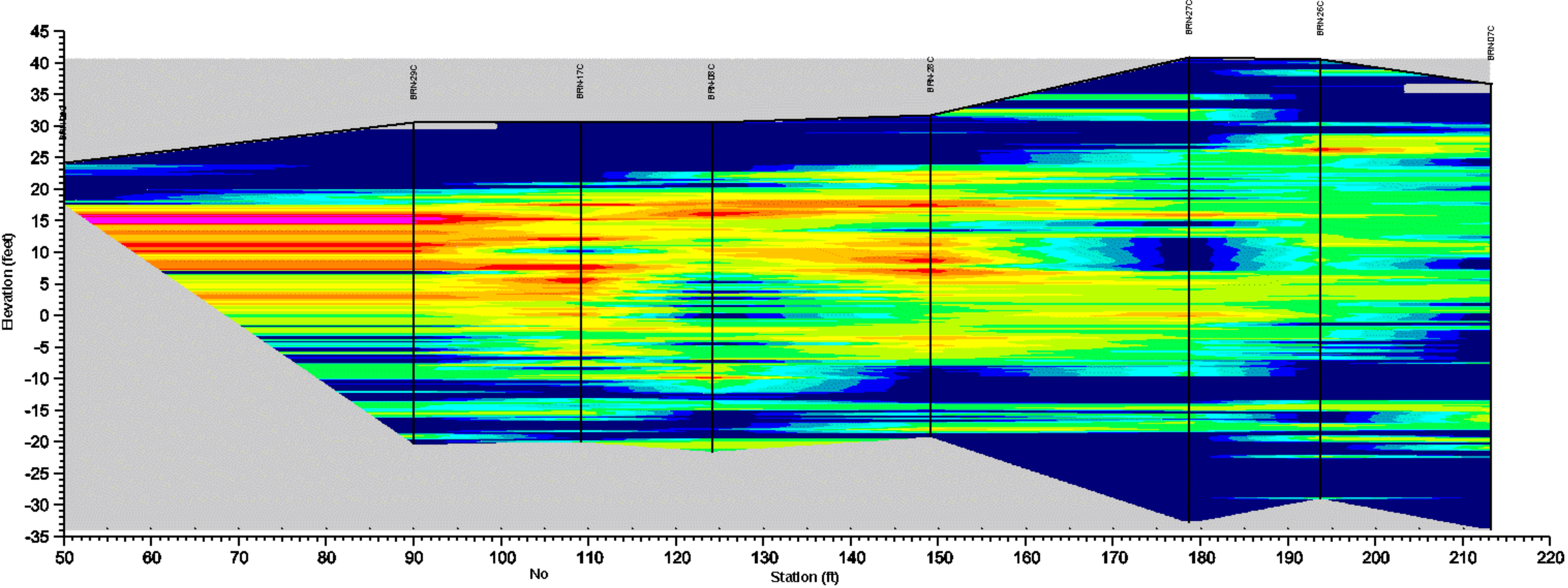


NOTES: CPT cross sections were generated using soil layer tracing to connect soil conditions between CPT soundings. For proper evaluation of a given section, the cross sections of CPT predicted soil type, strength, and normalized strength should be available for comprehensive interpretation. These CPT predicted techniques are based on advanced evaluation techniques developed at USACE ERDC over the last 25 years. Site based verifications between measured and CPT predicted strengths is still required. These CPT predicted strengths are for all soil type and strength level, ranging from undrained strengths for clays to drained strength for sands. The Academic Quality Index (AQI) is a simple but powerful tool, great data has an A grade or about 95% (great for correlations) and poor data has a D grade or about 65% (only good for tracking geologic layers boundaries). This visualization is a high graphic high detailed representation of a complex geology - final interpretation must be performed by a qualified expert in CPT data evaluation and stratigraphic evaluation.

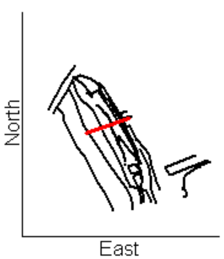
CPT Evaluation

Cross Section of R1
CPT predicted Soil Classification (ERDC version)

"Isaac Stephens PE, USACE ERDC GSL"
IWBC Brownsville TX



(this cross section is 163 feet long)

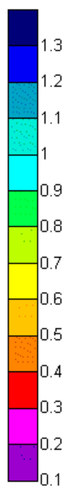


Cross Section:
R1

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

CPT predicted normalized Strength, Su1

(non linear normalized ERDC version)



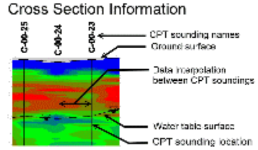
In Situ anomaly causing data processing error

Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information

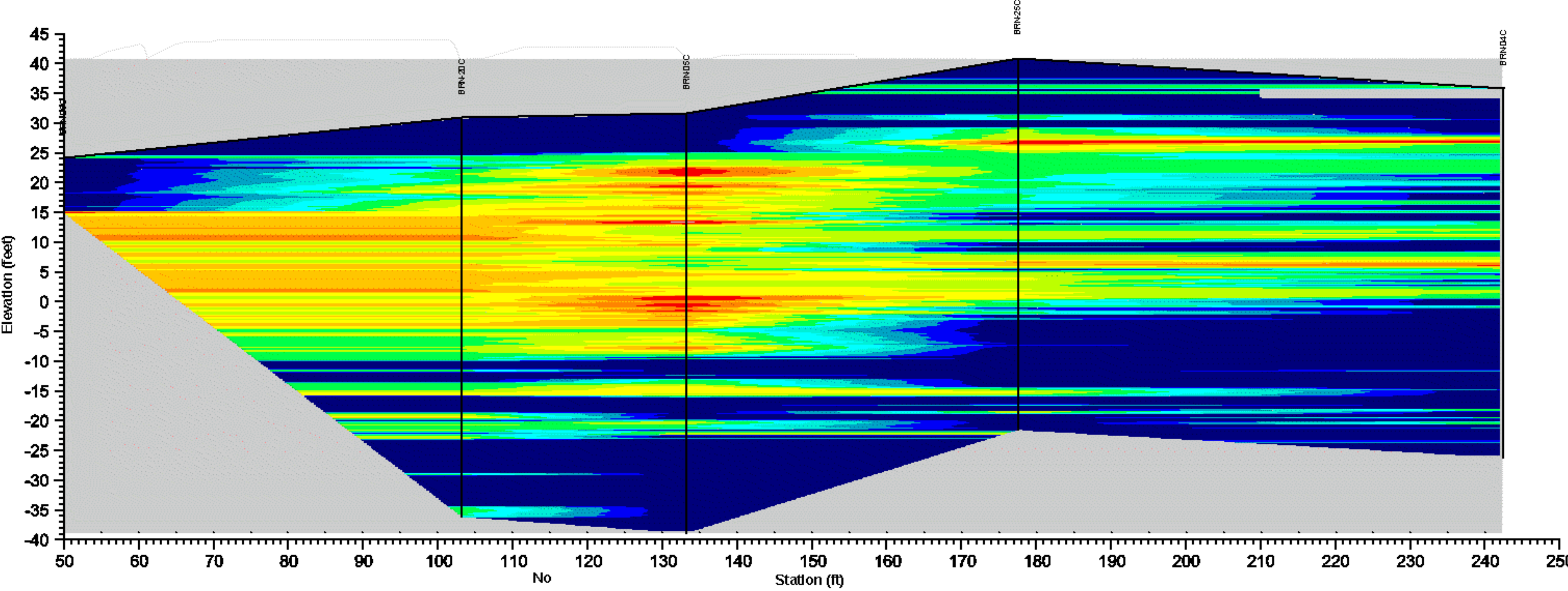


NOTES: CPT cross sections were generated using soil layer tracing to connect soil conditions between CPT soundings. For proper evaluation of a given section, the cross sections of CPT predicted soil type, strength, and normalized strength should be available for comprehensive interpretation. These CPT predicted techniques are based on advanced evaluation techniques developed at USACE ERDC over the last 25 years. Site based verifications between measured and CPT predicted strengths is still required. These CPT predicted strengths are for all soil type and strength level, ranging from undrained strengths for clays to drained strength for sands. The Academic Quality Index (AQI) is a simple but powerful tool, great data has an A grade or about 95% (great for correlations) and poor data has a D grade or about 65% (only good for tracking geologic layers boundaries). This visualization is a high graphic high detailed representation of a complex geology - final interpretation must be performed by a qualified expert in CPT data evaluation and stratigraphic evaluation.

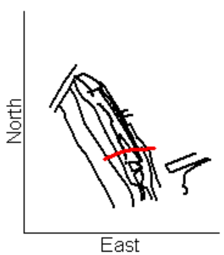
CPT Evaluation

Cross Section of R1
CPT predicted Strength (ERDC version)

"Isaac Stephens PE, USACE ERDC GSL "
IWBC Brownsville TX



(this cross section is 192 feet long)

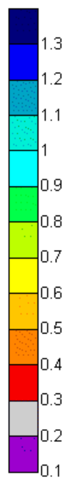


Cross Section:
R2

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

CPT predicted normalized Strength, Su1

(non linear normalized ERDC version)



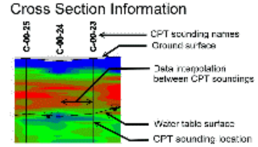
In Situ anomaly causing data processing error

Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information



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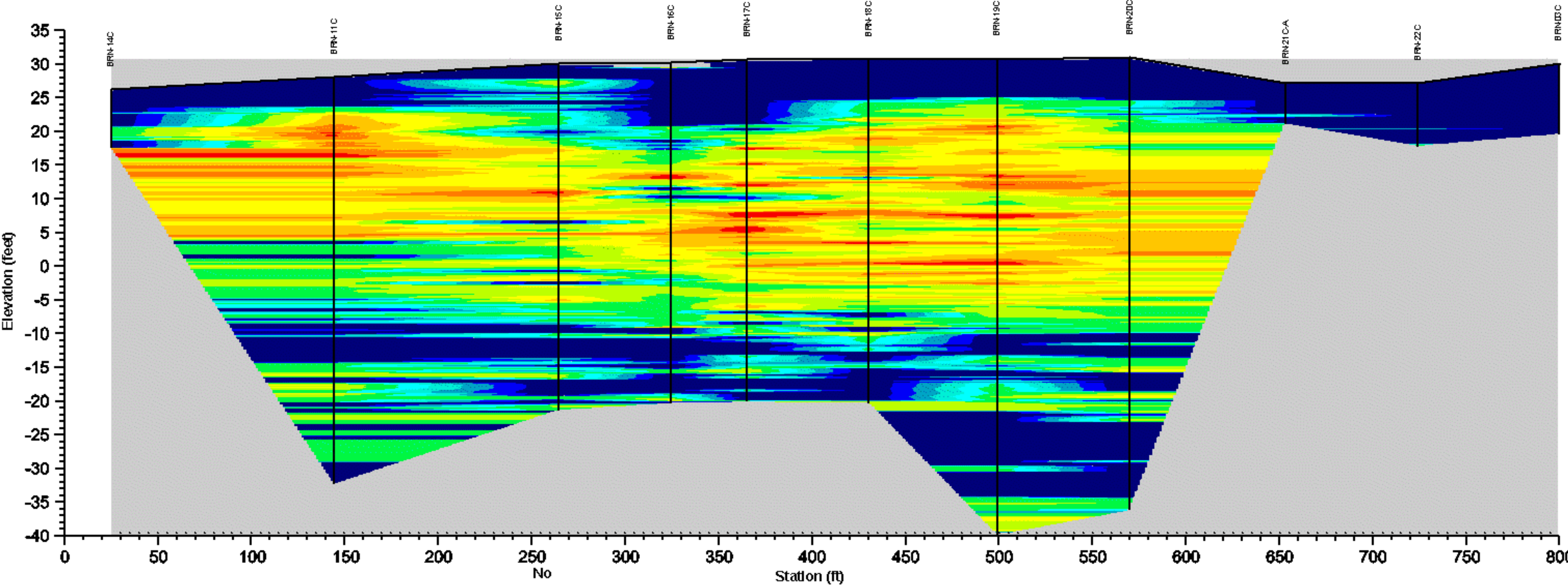
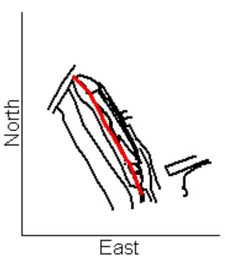
CPT Evaluation

Cross Section of R2
CPT predicted Strength (ERDC version)

"Isaac Stephens PE, USACE ERDC GSL "
IWBC Brownsville TX



(this cross section is 774 feet long)



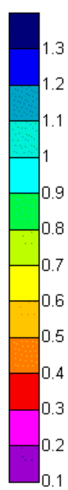
Cross Section: P1

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

CPT predicted normalized Strength, Su1

(non linear normalized ERDC version)

In Situ anomaly causing data processing error

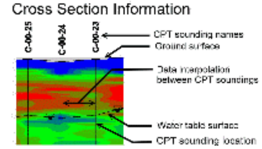


Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information



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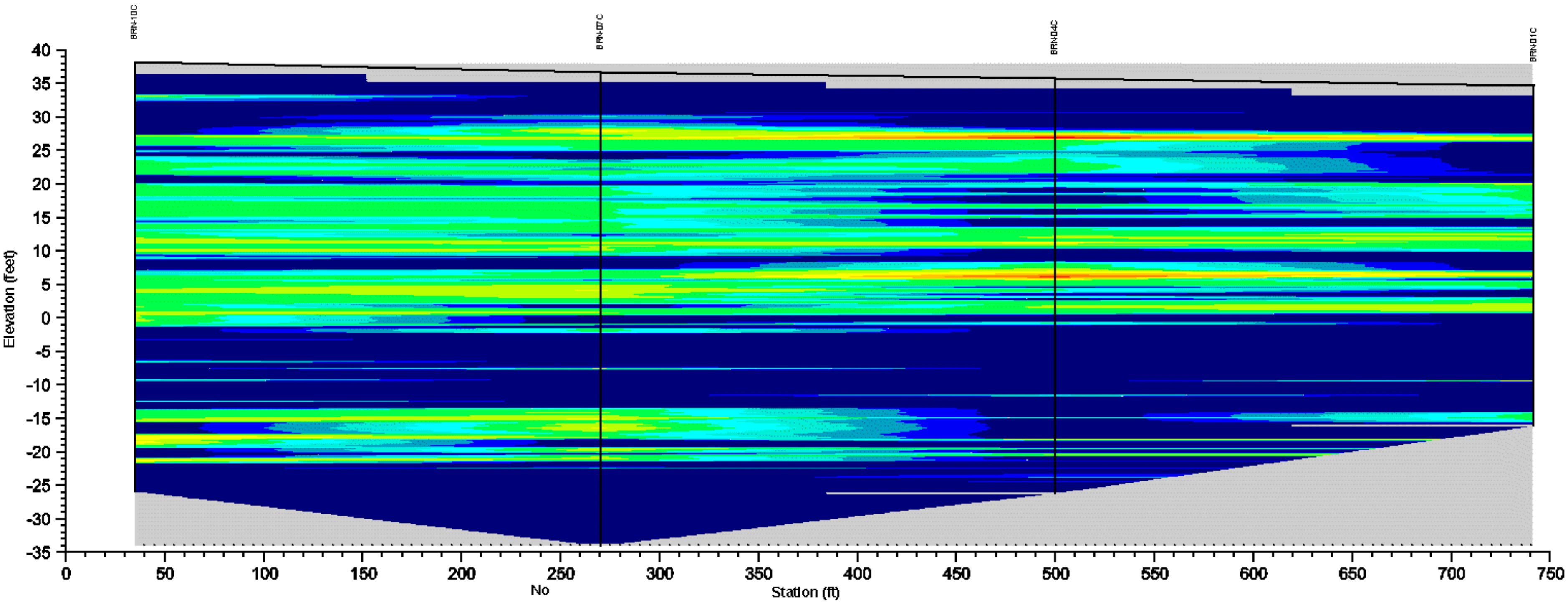
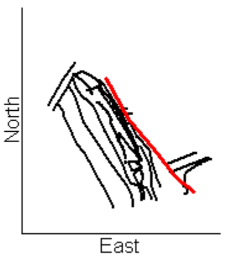
CPT Evaluation

Cross Section of P1
CPT predicted Strength (ERDC version)

"Isaac Stephens PE, USACE ERDC GSL "
IWBC Brownsville TX



(this cross section is 706 feet long)

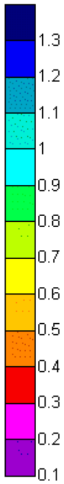


Cross Section:
P2

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

CPT predicted normalized Strength, Su1

(non linear normalized ERDC version)



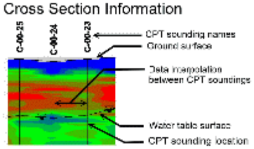
In Situ anomaly causing data processing error

Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information



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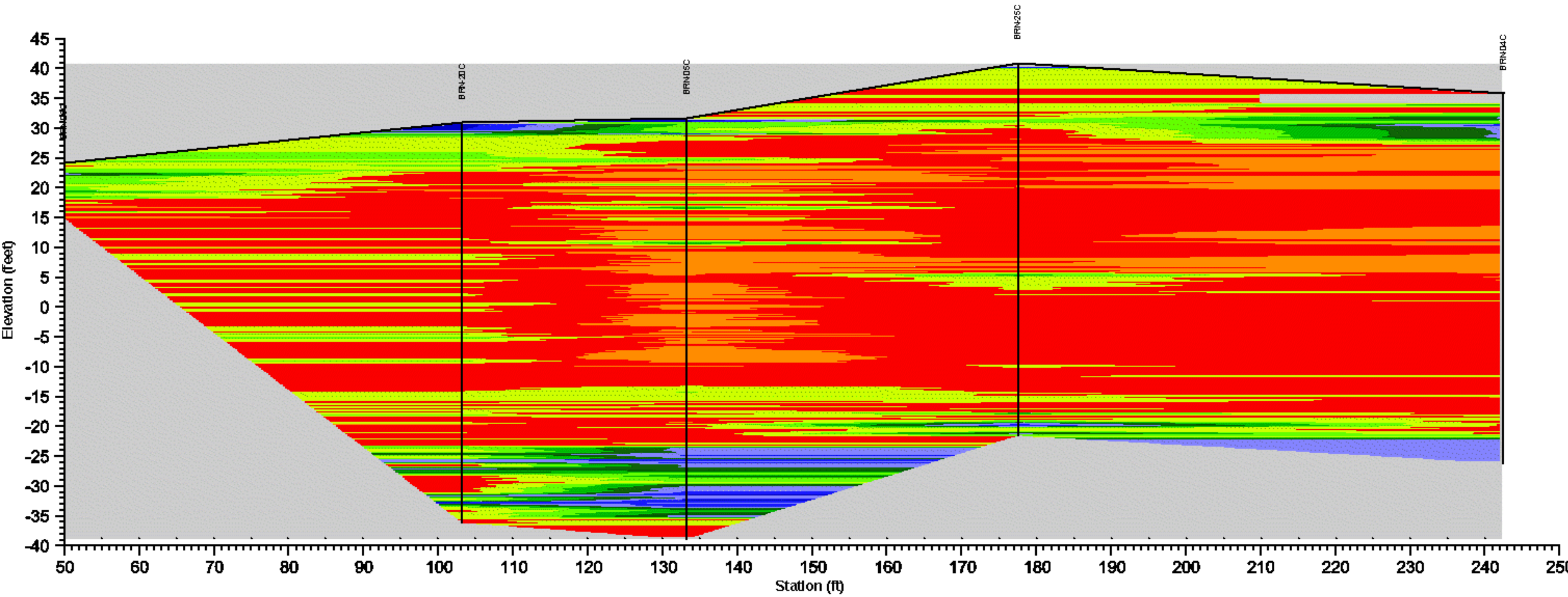
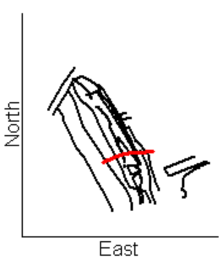
CPT Evaluation

Cross Section of P2
CPT predicted Strength (ERDC version)

"Isaac Stephens PE, USACE ERDC GSL "
IWBC Brownsville TX



(this cross section is 192 feet long)

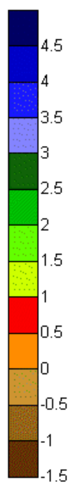


Cross Section:
R2

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

CPT Predicted Soil classification (SCN)

(non linear normalized ERDC 07 version)



SP to GM

SP and SM

SP to SM
SP-SM SAND, with less than 5% fines

ML to SM

ML (by defination)

Clay mixtures, influenced by silt interaction
BOUNDRY for Silty clay to clayey mixtures
Clays NOT influenced by silt interaction

CL to CH clays

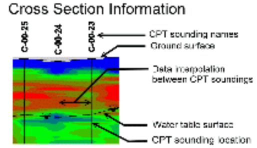
CH and CL clays (and NC organic clays)

CH clays, organic clays, or peats

Organic clays to Peats

In Situ anomaly causing data processing error

Cross Section Information



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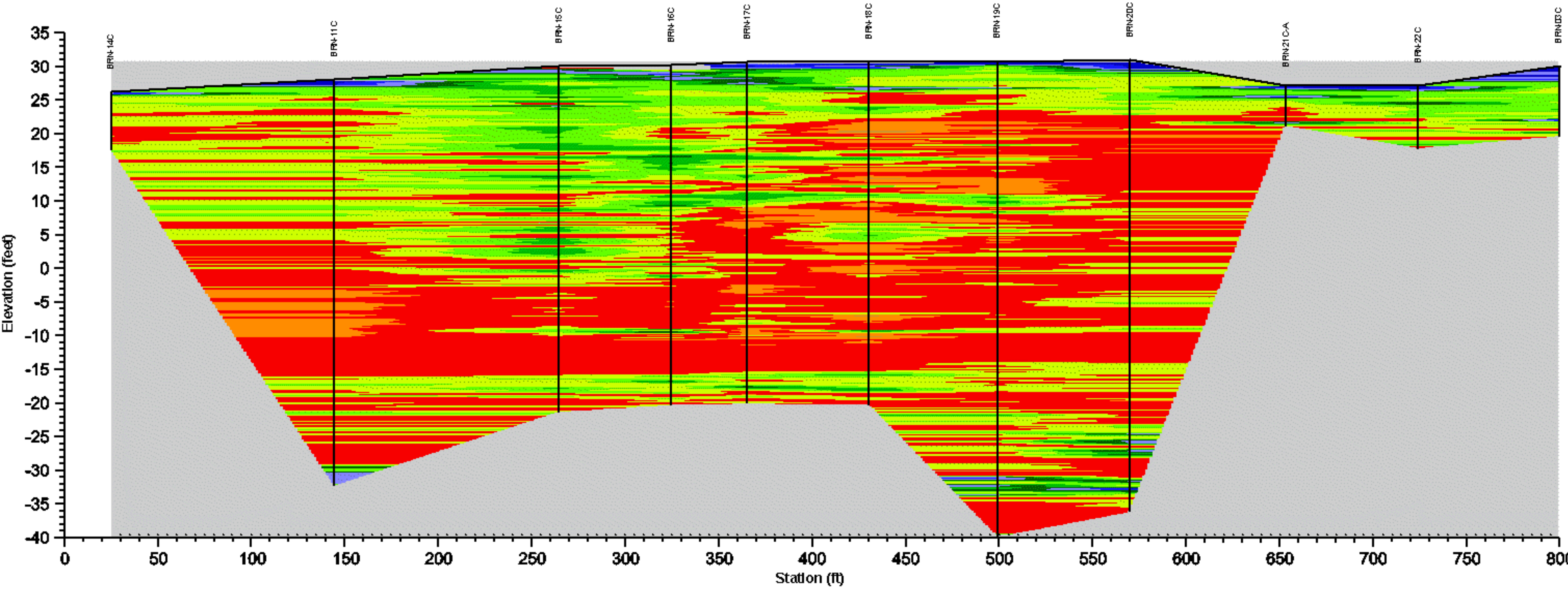
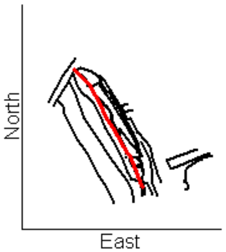
CPT Evaluation

Cross Section of R2
CPT predicted Soil Classification (ERDC version)

"Isaac Stephens PE, USACE ERDC GSL"
IWBC Brownsville TX



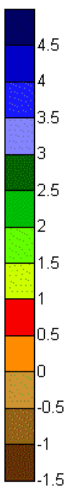
(this cross section is 774 feet long)



Cross Section:
P1

CPT Predicted Soil classification (SCN)

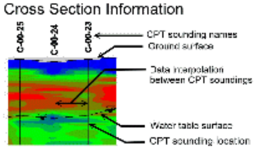
(non linear normalized ERDC 07 version)



SP to GM
SP and SM
SP to SM
SP-SM SAND, with less than 5% fines
ML to SM
ML (by defination)
Clay mixtures, influenced by silt interaction
BOUNDRY for Silty clay to clayey mixtures
Clays NOT influenced by silt interaction
CL to CH clays
CH and CL clays (and NC organic clays)
CH clays, organic clays, or peats
Organic clays to Peats

In Situ anomaly causing data processing error

Cross Section Information



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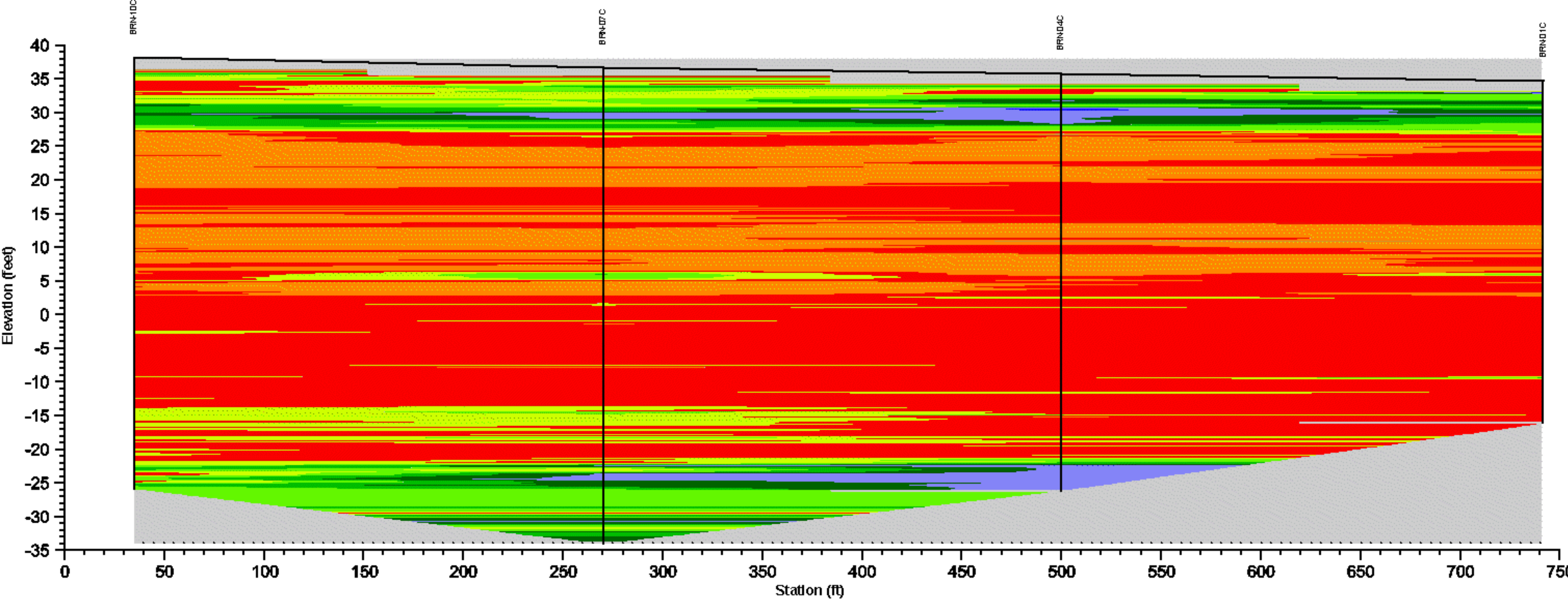
CPT Evaluation

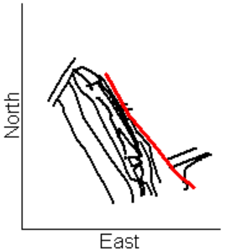
Cross Section of P1
CPT predicted Soil Classification (ERDC version)

"Isaac Stephens PE, USACE ERDC GSL "
IWBC Brownsville TX

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB



(this cross section is 706 feet long)



Cross Section:
P2

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

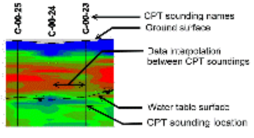
CPT Predicted Soil classification (SCN)

(non linear normalized ERDC 07 version)

4.5 – 4	SP to GM
4 – 3.5	SP and SM
3.5 – 3	SP to SM SP-SM SAND, with less than 5% fines
3 – 2.5	ML to SM
2.5 – 2	ML (by definition)
1.5 – 1	Clay mixtures, influenced by silt interaction BOUNDRY for Silty clay to clayey mixtures Clays NOT influenced by silt interaction
0.5 – 0	CL to CH clays
0 – -0.5	CH and CL clays (and NC organic clays)
-0.5 – -1	CH clays, organic clays, or peats
-1 – -1.5	Organic clays to Peats

In Situ anomaly causing data processing error

Cross Section Information



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CPT Evaluation

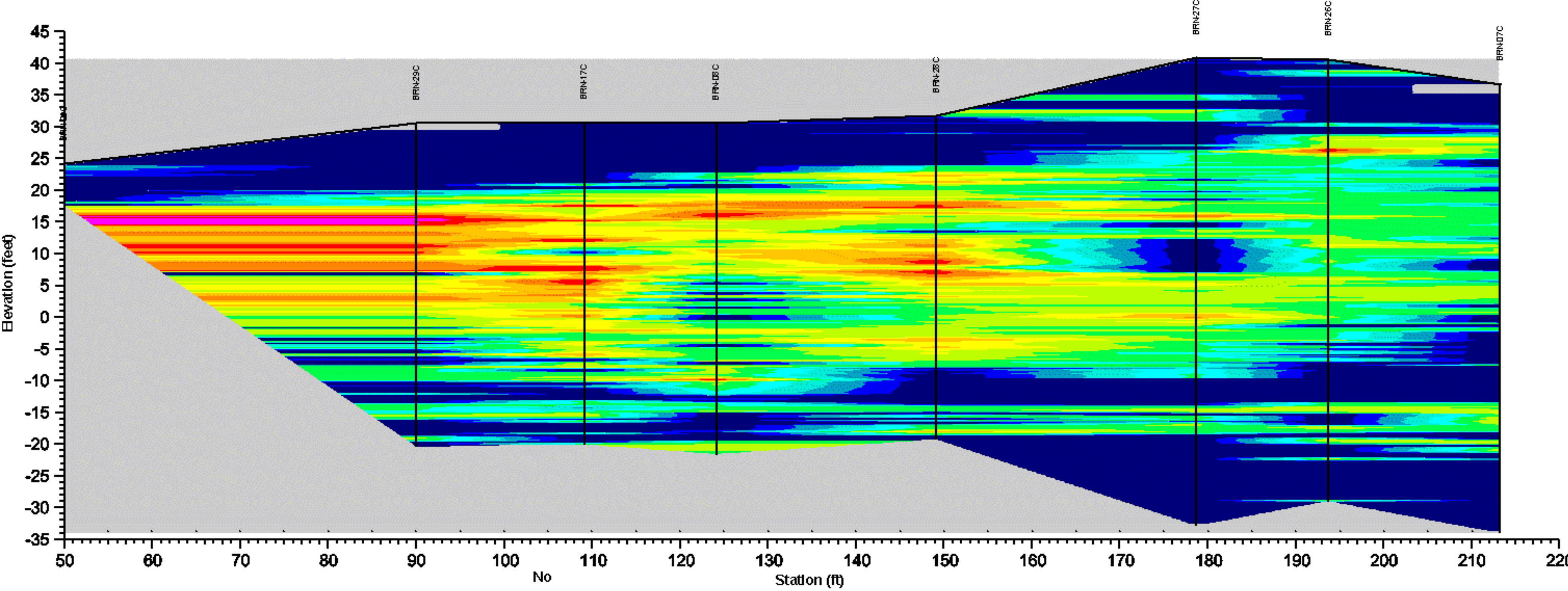
Cross Section of P2

CPT predicted Soil Classification (ERDC version)

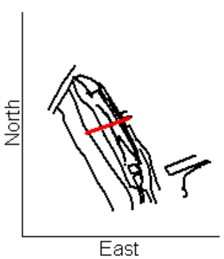
"Isaac Stephens PE, USACE ERDC GSL"

IWBC Brownsville TX

Appendix D: CPT Predicted Strengths



(this cross section is 163 feet long)

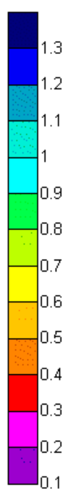


Cross Section:
R1

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

CPT predicted normalized Strength, Su1

(non linear normalized ERDC version)



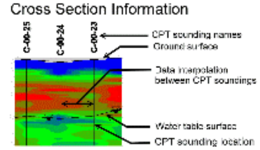
In Situ anomaly causing data processing error

Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information

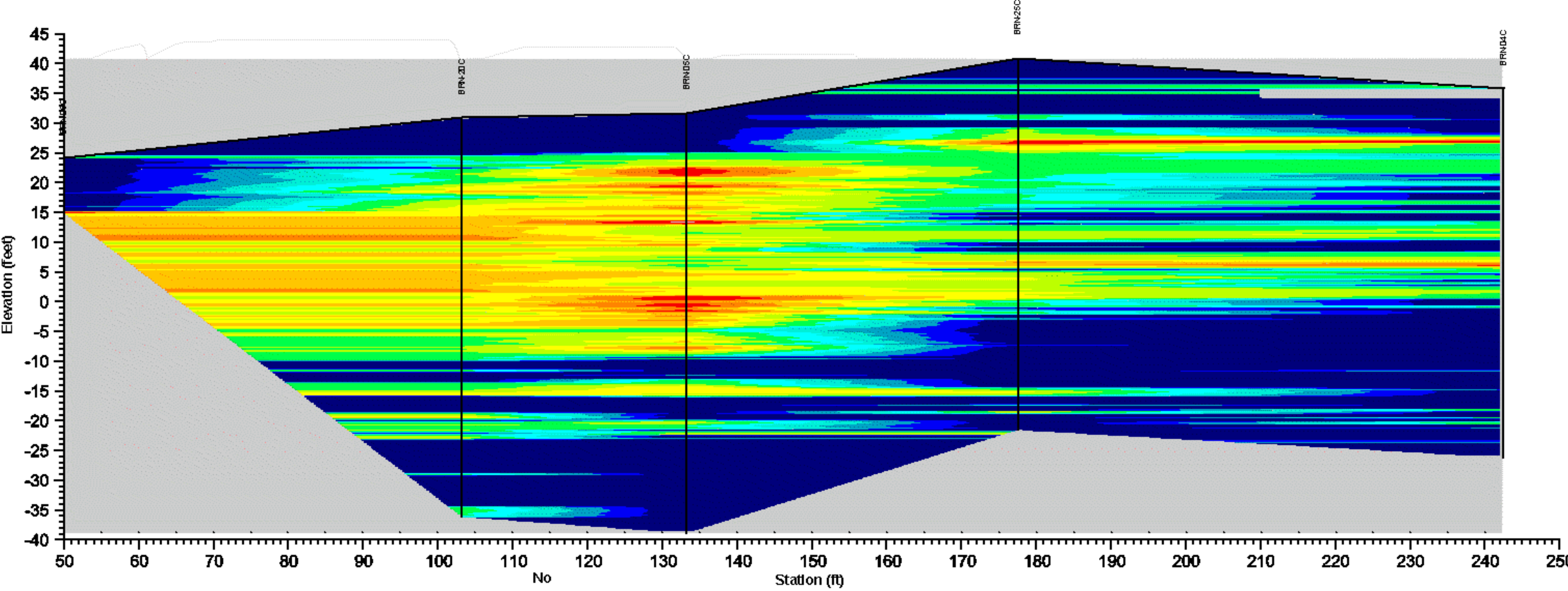


NOTES: CPT cross sections were generated using soil layer tracing to connect soil conditions between CPT soundings. For proper evaluation of a given section, the cross sections of CPT predicted soil type, strength, and normalized strength should be available for comprehensive interpretation. These CPT predicted techniques are based on advanced evaluation techniques developed at USACE ERDC over the last 25 years. Site based verifications between measured and CPT predicted strengths is still required. These CPT predicted strengths are for all soil type and strength level, ranging from undrained strengths for clays to drained strength for sands. The Academic Quality Index (AQI) is a simple but powerful tool, great data has an A grade or about 95% (great for correlations) and poor data has a D grade or about 65% (only good for tracking geologic layers boundaries). This visualization is a high graphic high detailed representation of a complex geology - final interpretation must be performed by a qualified expert in CPT data evaluation and stratigraphic evaluation.

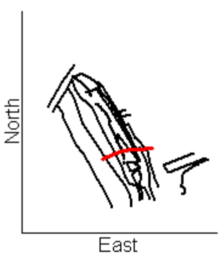
CPT Evaluation

Cross Section of R1
CPT predicted Strength (ERDC version)

"Isaac Stephens PE, USACE ERDC GSL "
IWBC Brownsville TX



(this cross section is 192 feet long)



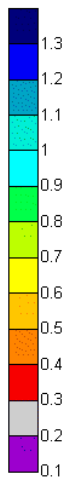
Cross Section:
R2

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

CPT predicted normalized Strength, Su1

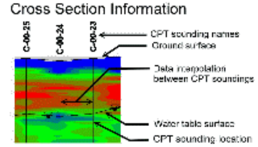
(non linear normalized ERDC version)

In Situ anomaly causing data processing error



Normally NC to slightly OC clay
underconsolidated or organic clay
highly underconsolidated or high organics

Cross Section Information

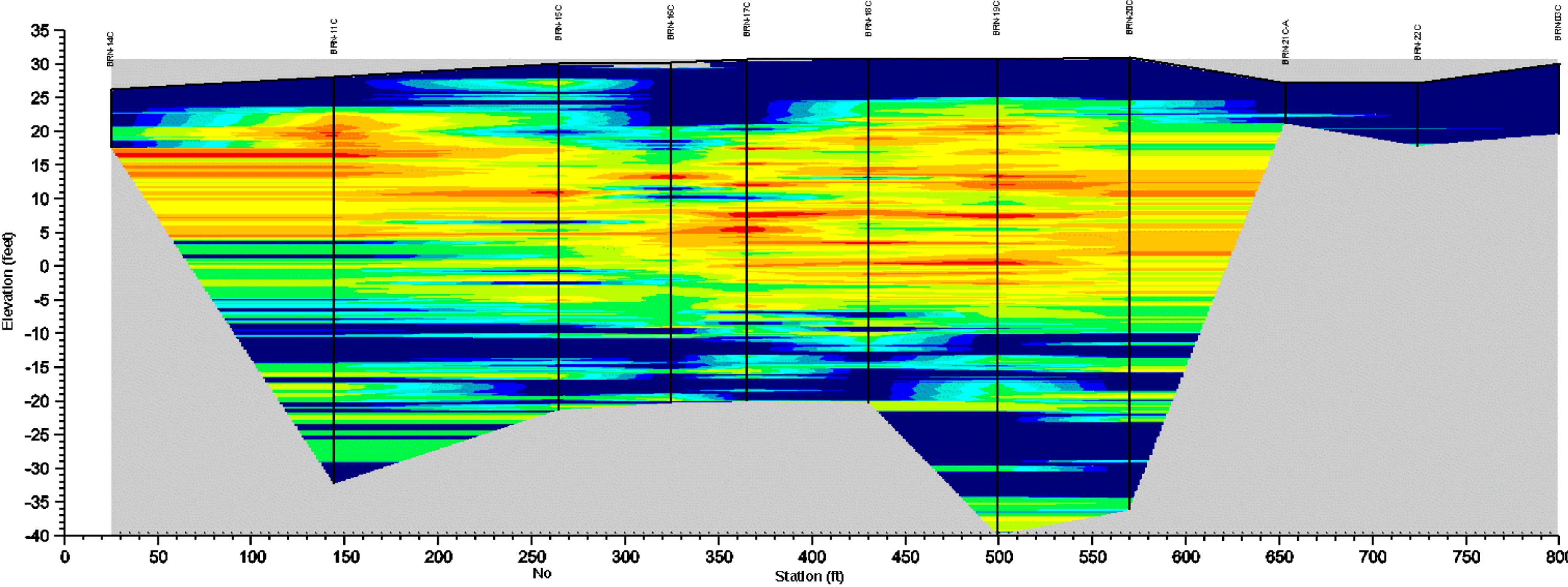


NOTES: CPT cross sections were generated using soil layer tracing to connect soil conditions between CPT soundings. For proper evaluation of a given section, the cross sections of CPT predicted soil type, strength, and normalized strength should be available for comprehensive interpretation. These CPT predicted techniques are based on advanced evaluation techniques developed at USACE ERDC over the last 25 years. Site based verifications between measured and CPT predicted strengths is still required. These CPT predicted strengths are for all soil type and strength level, ranging from undrained strengths for clays to drained strength for sands. The Academic Quality Index (AQI) is a simple but powerful tool, great data has an A grade or about 95% (great for correlations) and poor data has a D grade or about 65% (only good for tracking geologic layers boundaries). This visualization is a high graphic high detailed representation of a complex geology - final interpretation must be performed by a qualified expert in CPT data evaluation and stratigraphic evaluation.

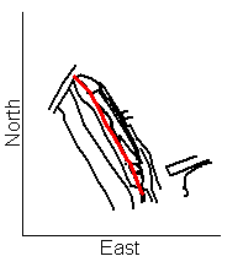
CPT Evaluation

Cross Section of R2
CPT predicted Strength (ERDC version)

"Isaac Stephens PE, USACE ERDC GSL "
IWBC Brownsville TX



(this cross section is 774 feet long)



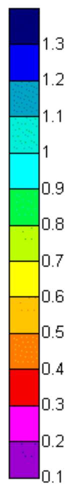
Cross Section:
P1

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

CPT predicted normalized Strength, Su1

(non linear normalized ERDC version)

In Situ anomaly causing data processing error

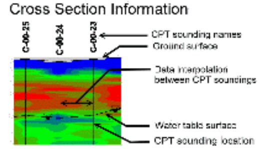


Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information

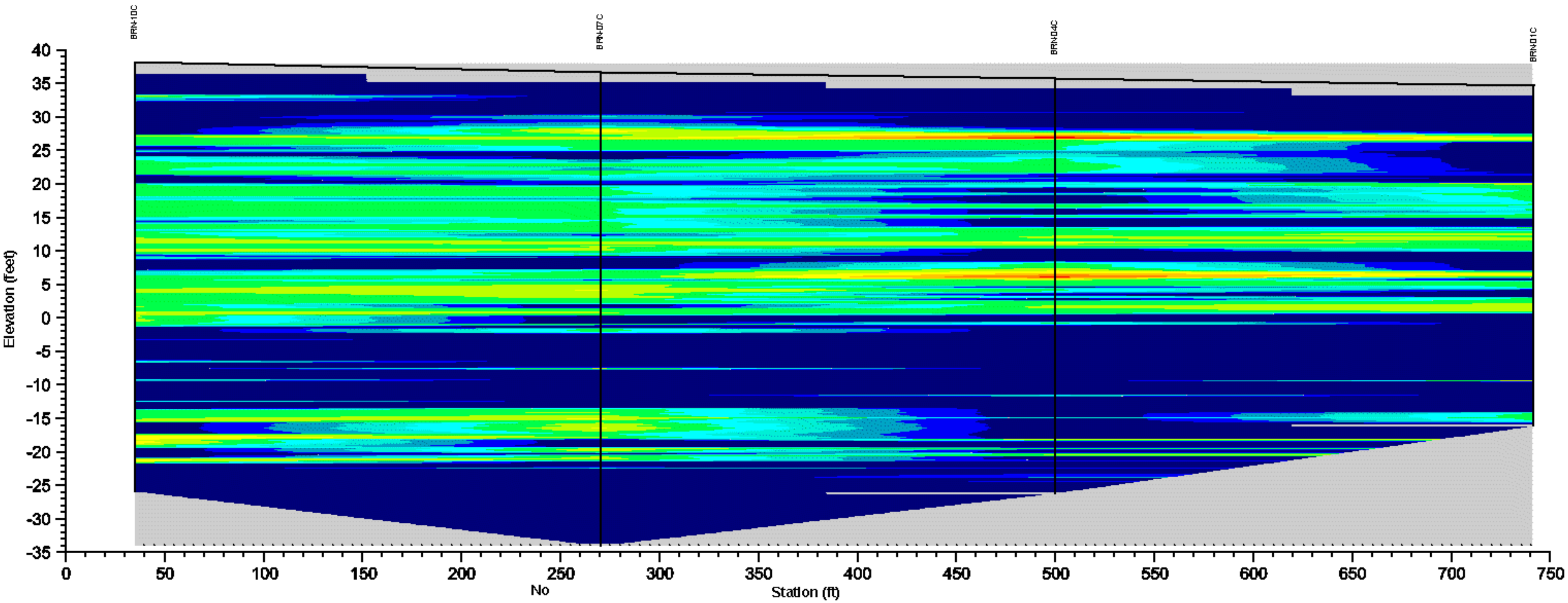


NOTES: CPT cross sections were generated using soil layer tracing to connect soil conditions between CPT soundings. For proper evaluation of a given section, the cross sections of CPT predicted soil type, strength, and normalized strength should be available for comprehensive interpretation. These CPT predicted techniques are based on advanced evaluation techniques developed at USACE ERDC over the last 25 years. Site based verifications between measured and CPT predicted strengths is still required. These CPT predicted strengths are for all soil type and strength level, ranging from undrained strengths for clays to drained strength for sands. The Academic Quality Index (AQI) is a simple but powerful tool, great data has an A grade or about 95% (great for correlations) and poor data has a D grade or about 65% (only good for tracking geologic layers boundaries). This visualization is a high graphic high detailed representation of a complex geology - final interpretation must be performed by a qualified expert in CPT data evaluation and stratigraphic evaluation.

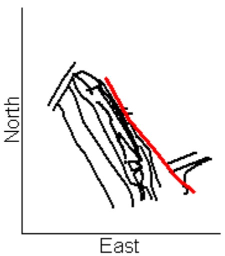
CPT Evaluation

Cross Section of P1
CPT predicted Strength (ERDC version)

"Isaac Stephens PE, USACE ERDC GSL "
IWBC Brownsville TX



(this cross section is 706 feet long)

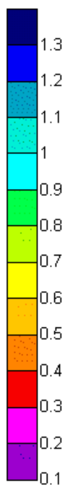


Cross Section:
P2

US Army Corps of Engineers
Engineer Research and Development Center
USACE-ERDC-GSL-GEGB

CPT predicted normalized Strength, Su1

(non linear normalized ERDC version)



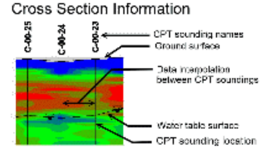
In Situ anomaly causing data processing error

Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information



NOTES: CPT cross sections were generated using soil layer tracing to connect soil conditions between CPT soundings. For proper evaluation of a given section, the cross sections of CPT predicted soil type, strength, and normalized strength should be available for comprehensive interpretation. These CPT predicted techniques are based on advanced evaluation techniques developed at USACE ERDC over the last 25 years. Site based verifications between measured and CPT predicted strengths is still required. These CPT predicted strengths are for all soil type and strength level, ranging from undrained strengths for clays to drained strength for sands. The Academic Quality Index (AQI) is a simple but powerful tool, great data has an A grade or about 95% (great for correlations) and poor data has a D grade or about 65% (only good for tracking geologic layers boundaries). This visualization is a high graphic high detailed representation of a complex geology - final interpretation must be performed by a qualified expert in CPT data evaluation and stratigraphic evaluation.

CPT Evaluation

Cross Section of P2
CPT predicted Strength (ERDC version)

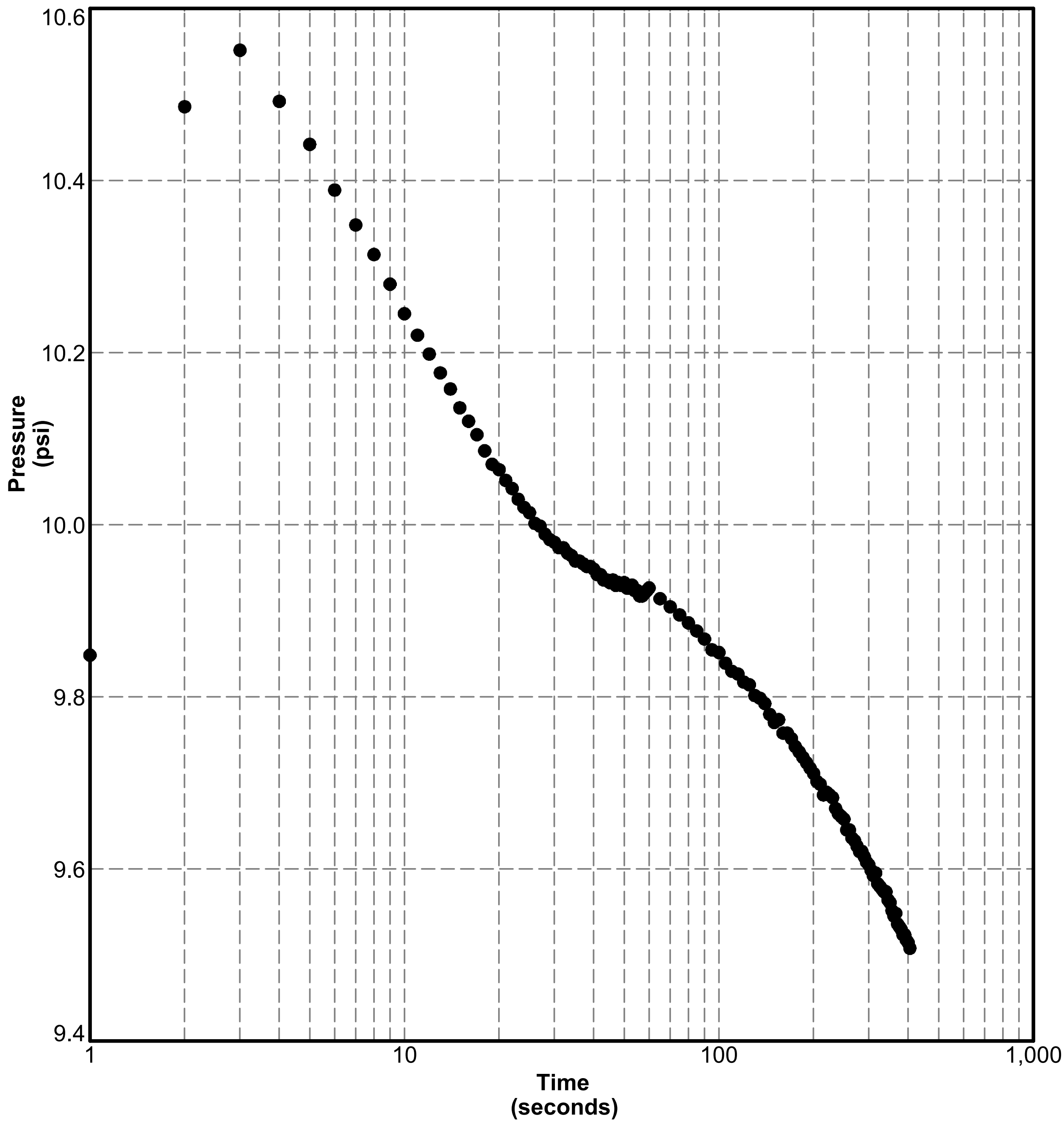
"Isaac Stephens PE, USACE ERDC GSL "
IWBC Brownsville TX

Appendix E: Dissipation Tests

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489194.5
Easting: 1314445.6
Elevation: 34.5
Total Depth: 50.7 ft
Termination Criteria:
Test Depth: 20.3 ft



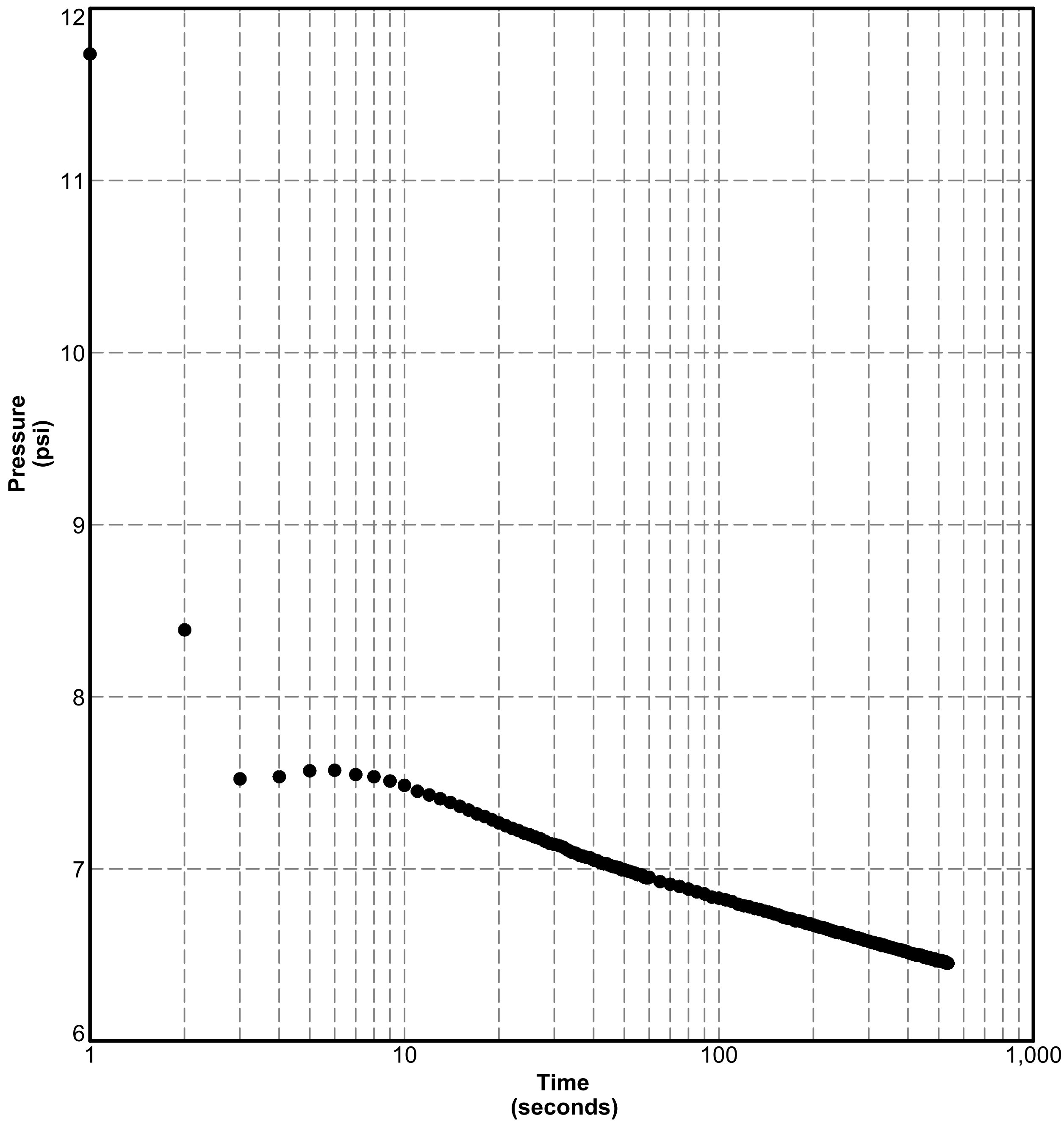
BRN-P1-01C

Electronic File Name: BRN-P1-01C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489194.5
Easting: 1314445.6
Elevation: 34.5
Total Depth: 50.7 ft
Termination Criteria:
Test Depth: 28.5 ft



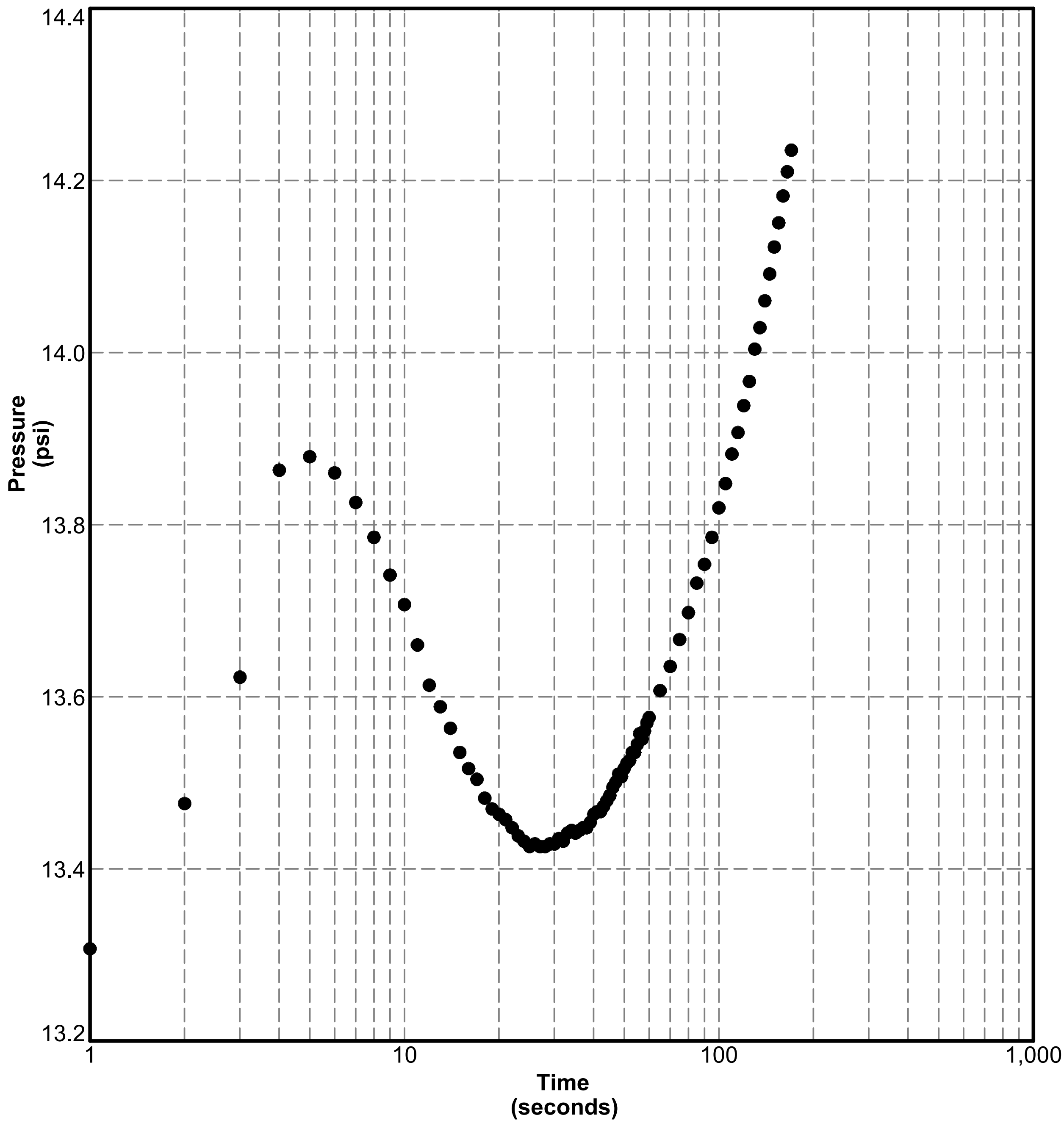
BRN-P1-01C

Electronic File Name: BRN-P1-01C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date:	Aug. 1, 2014	**Northing:**	16489194.5	**Total Depth:**	50.7 ft
Estimated Water Depth:	0 ft	**Easting:**	1314445.6	**Termination Criteria:**	
Rig/Operator:	Markov	**Elevation:**	34.5	**Test Depth:**	35.8 ft



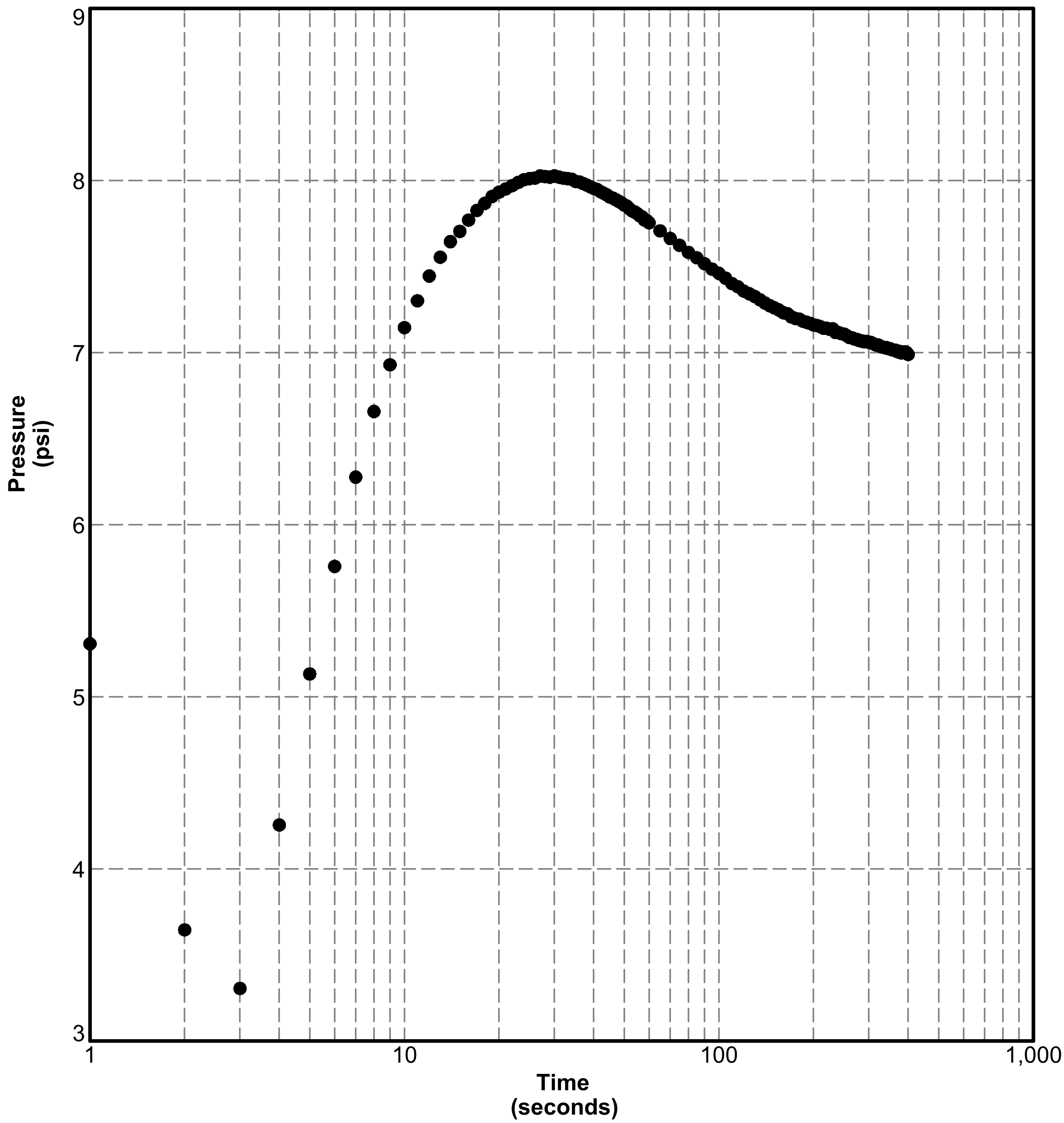
BRN-P1-01C

Electronic File Name: BRN-P1-01C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489186.8
Easting: 1314329.0
Elevation: 41.3
Total Depth: 50.7 ft
Termination Criteria:
Test Depth: 36.1 ft



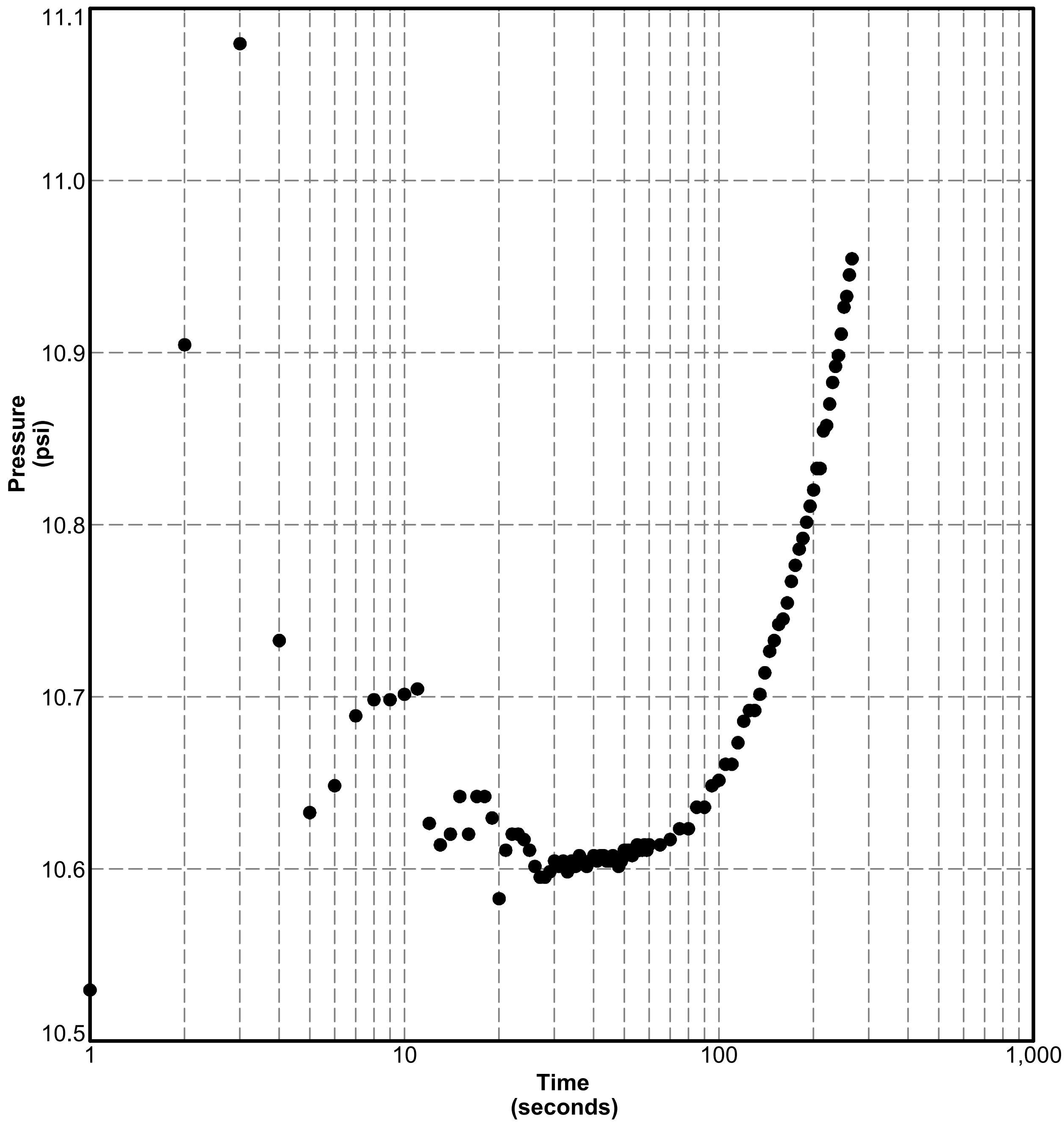
BRN-P1-02C

Electronic File Name: BRN-P1-02C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489396.7
Easting: 1314322.0
Elevation: 35.6
Total Depth: 61.8 ft
Termination Criteria:
Test Depth: 25.8 ft



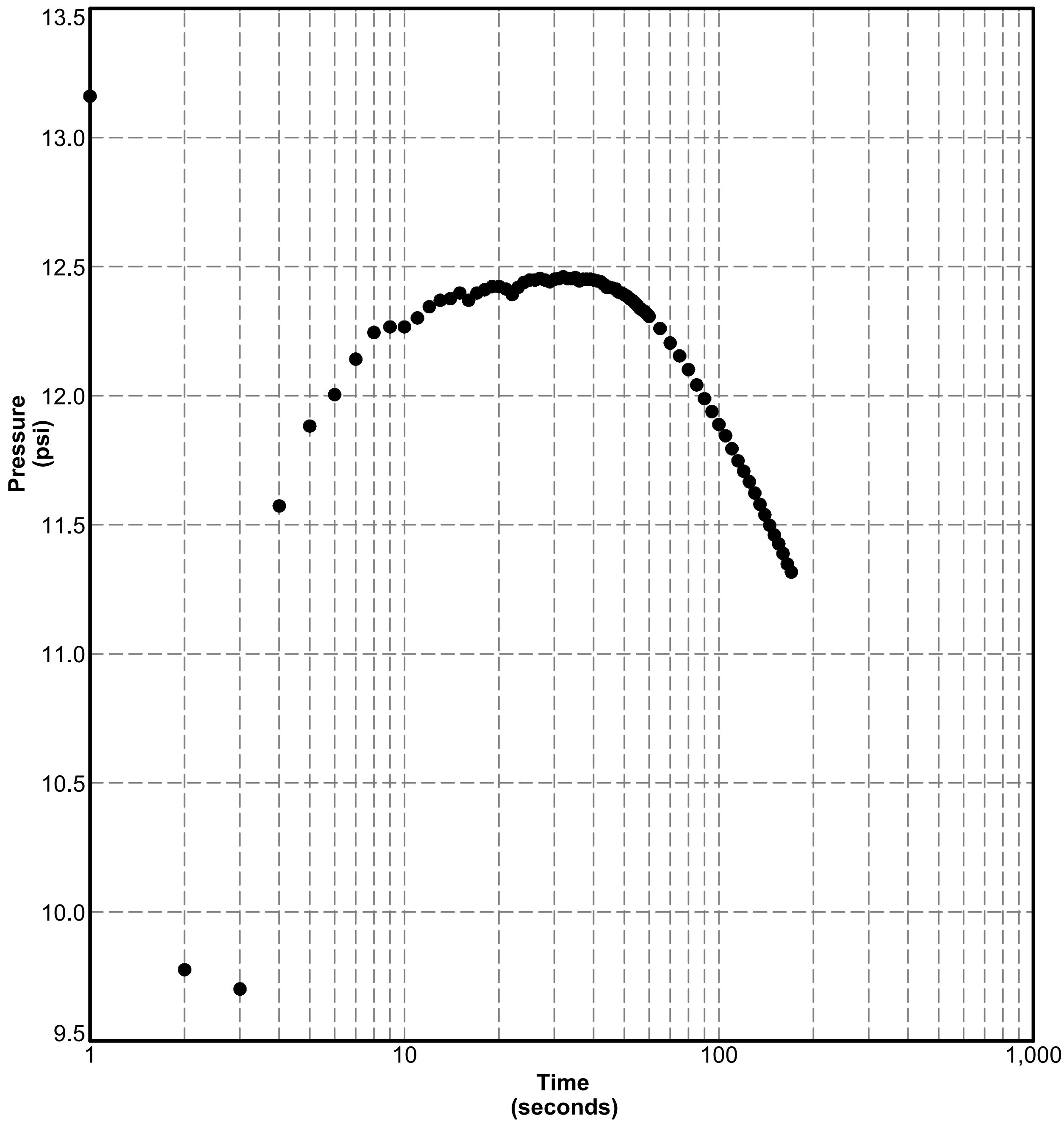
BRN-P1-04C

Electronic File Name: BRN-P1-04C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014 | **Northing:** 16489396.7 | **Total Depth:** 61.8 ft
Estimated Water Depth: 0 ft | **Easting:** 1314322.0 | **Termination Criteria:**
Rig/Operator: Markov | **Elevation:** 35.6 | **Test Depth:** 33.0 ft



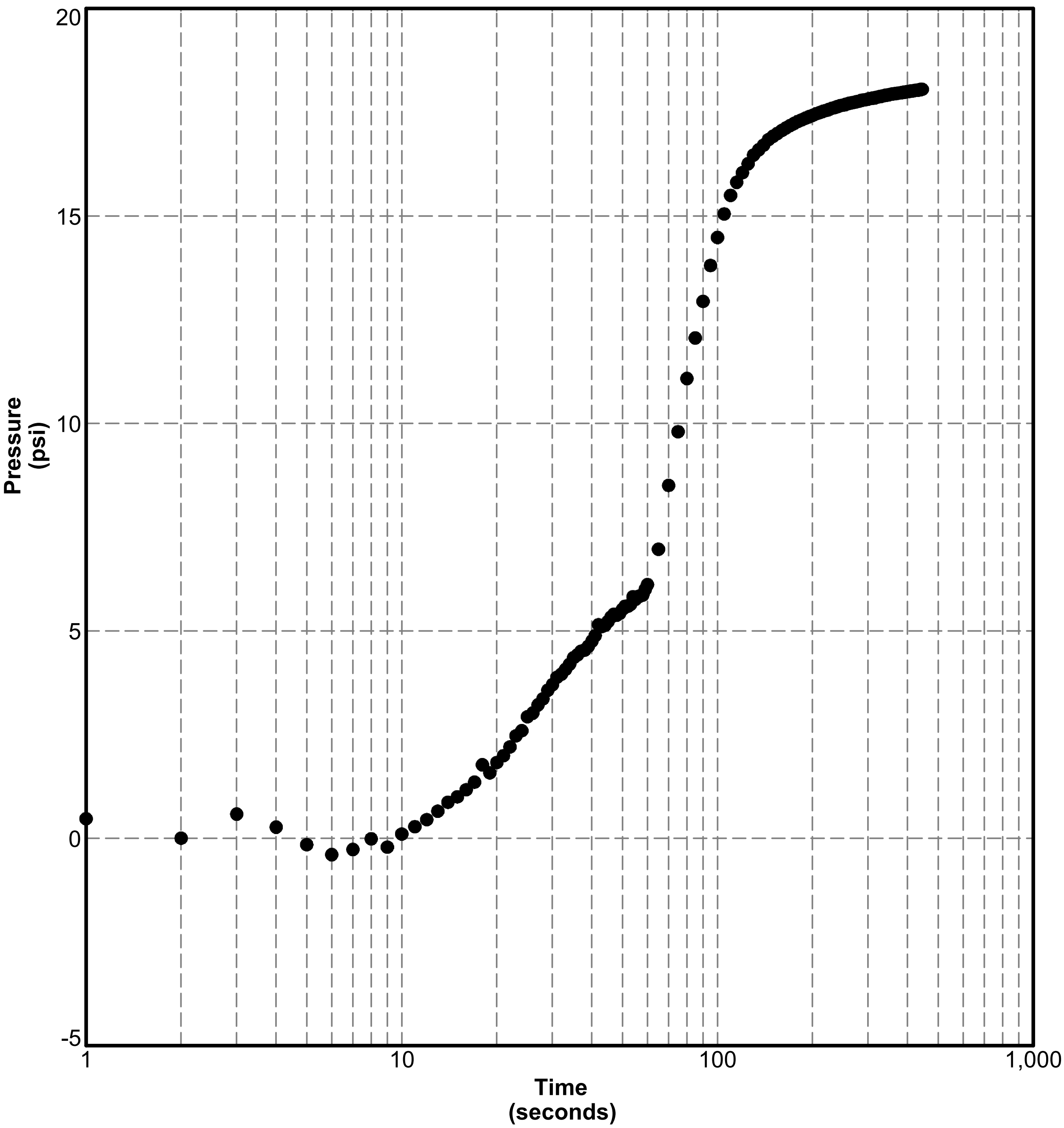
BRN-P1-04C

Electronic File Name: BRN-P1-04C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489396.7
Easting: 1314322.0
Elevation: 35.6
Total Depth: 61.8 ft
Termination Criteria:
Test Depth: 58.2 ft



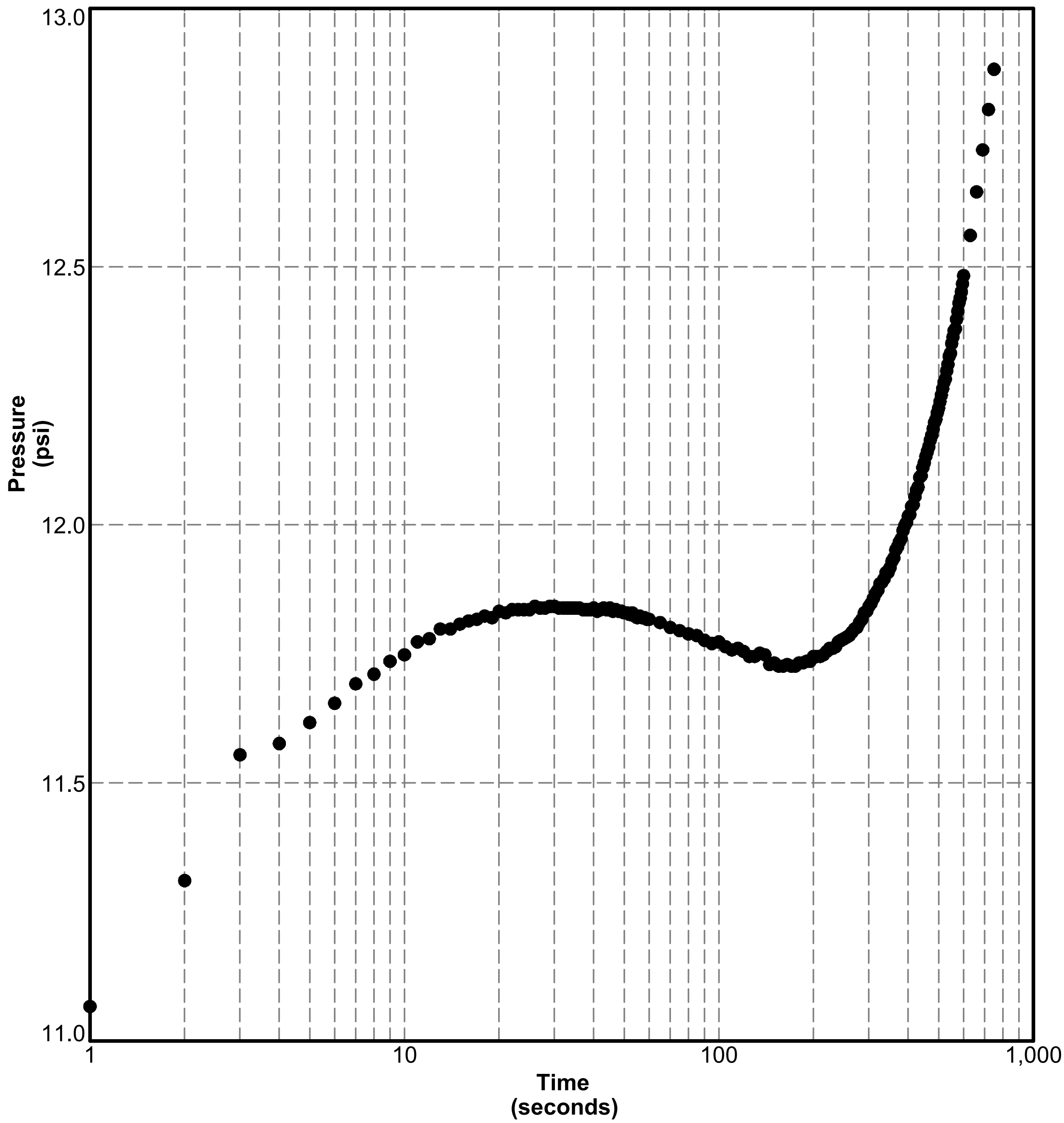
BRN-P1-04C

Electronic File Name: BRN-P1-04C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014 **Northing:** 16489384.4 **Total Depth:** 70.4 ft
Estimated Water Depth: 0 ft **Easting:** 1314214.4 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 31.5 **Test Depth:** 40.5 ft



BRN-P1-05C

Electronic File Name: BRN-P1-05C.dis

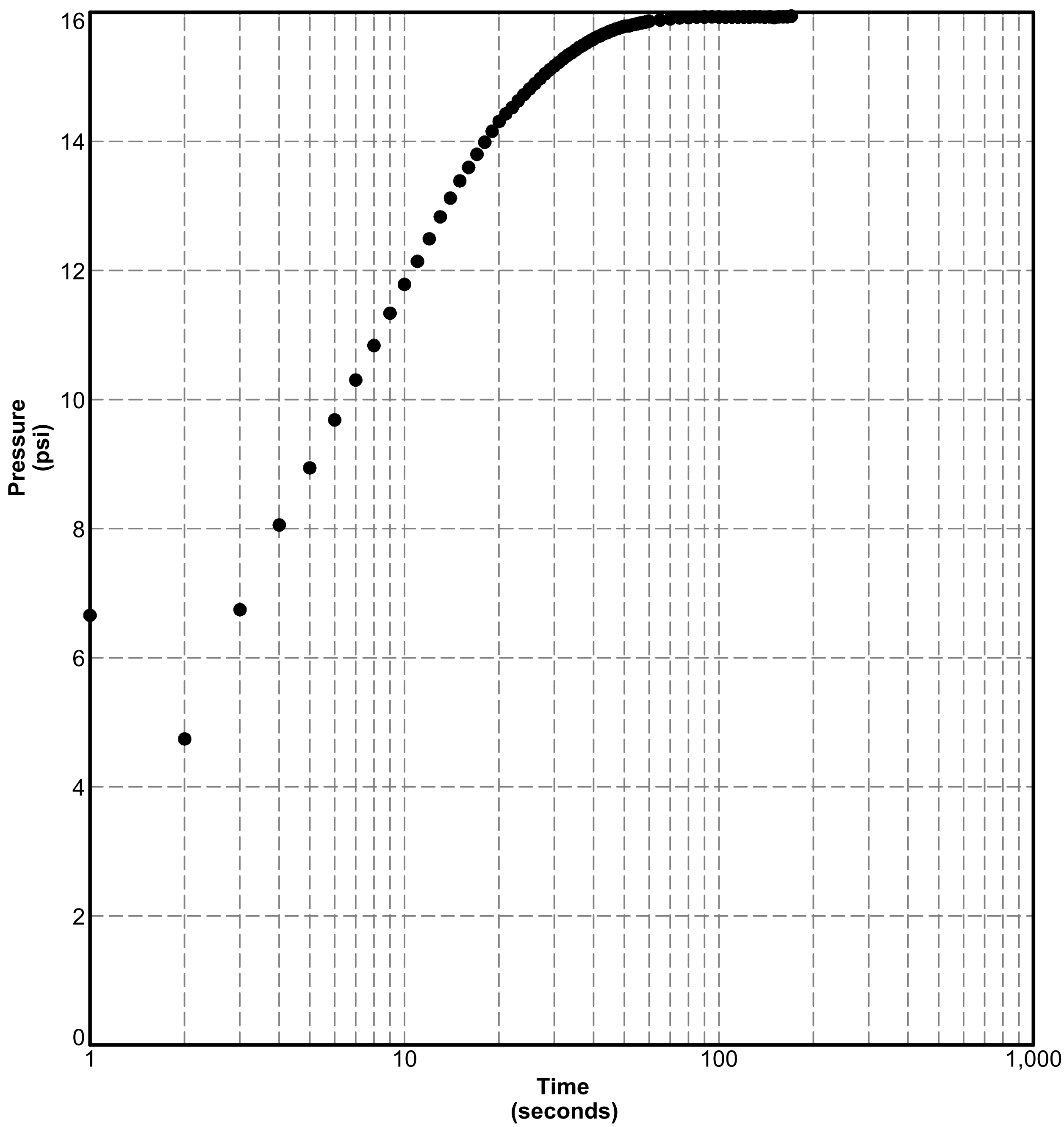
Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489384.4
Easting: 1314214.4
Elevation: 31.5

Total Depth: 70.4 ft
Termination Criteria:
Test Depth: 48.1 ft



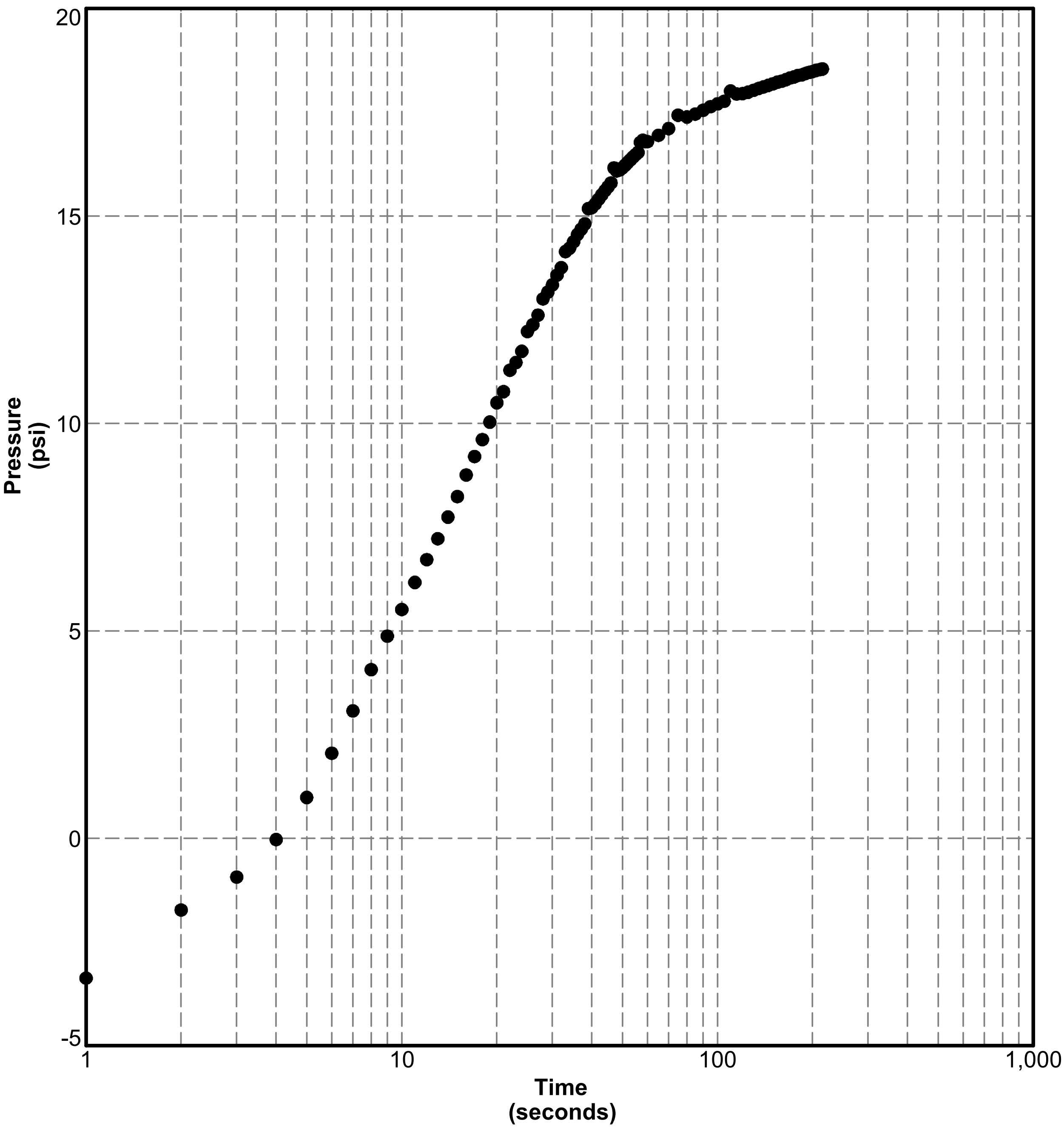
BRN-P1-05C

Electronic File Name: BRN-P1-05C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489384.4
Easting: 1314214.4
Elevation: 31.5
Total Depth: 70.4 ft
Termination Criteria:
Test Depth: 55.3 ft



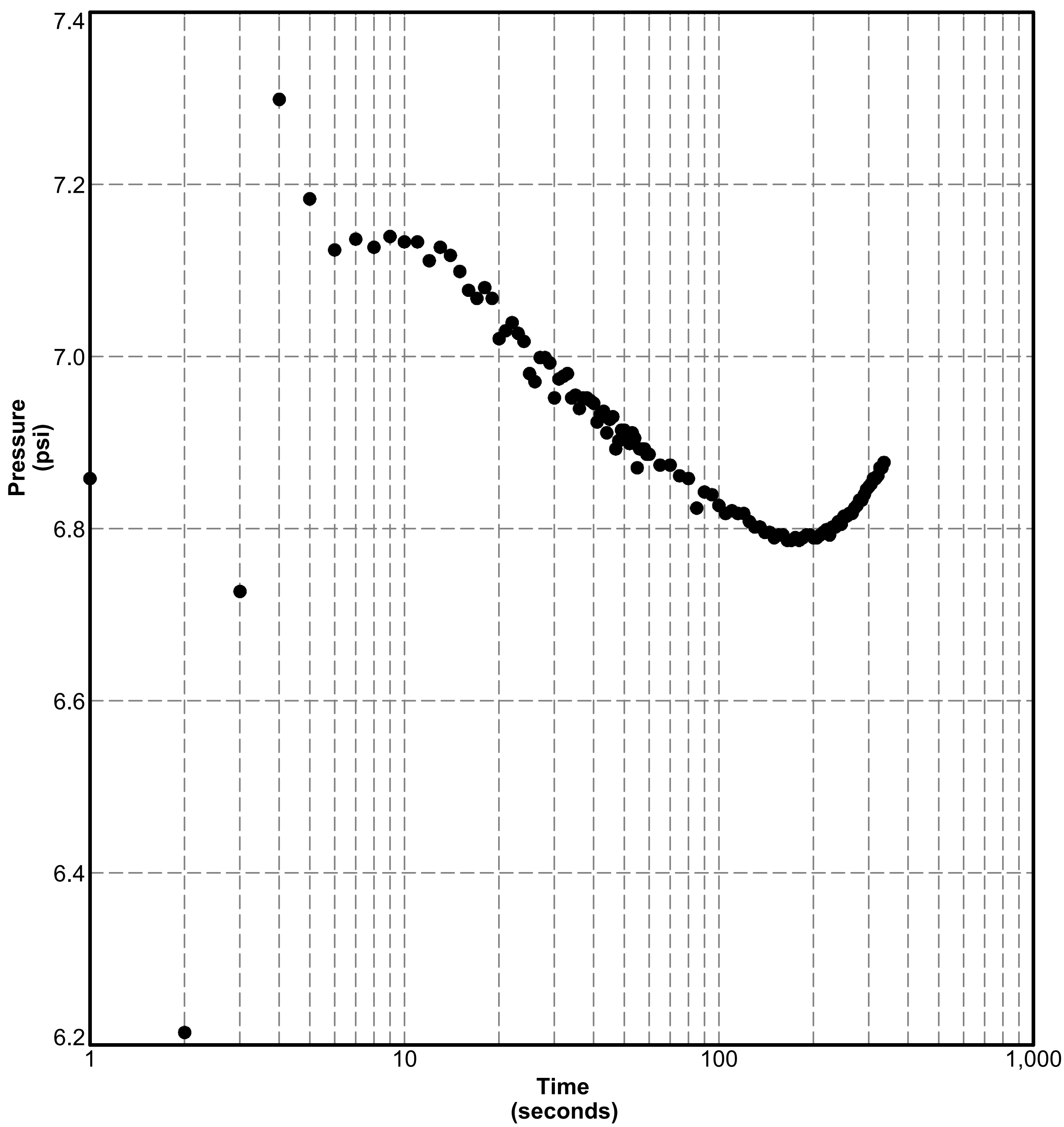
BRN-P1-05C

Electronic File Name: BRN-P1-05C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489583.2
Easting: 1314212.7
Elevation: 36.5
Total Depth: 70.5 ft
Termination Criteria:
Test Depth: 27.4 ft



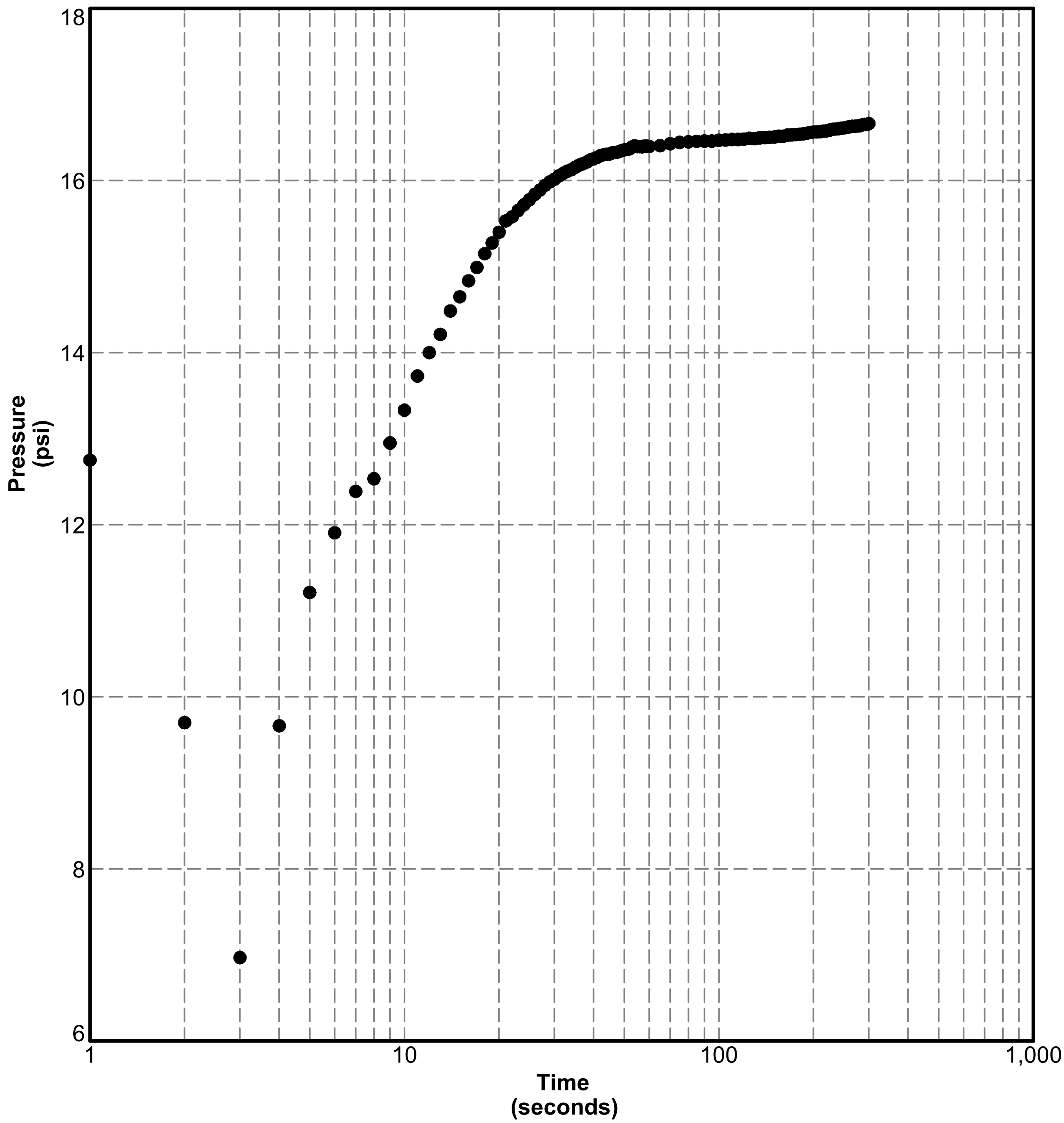
BRN-P1-07C

Electronic File Name: BRN-P1-07C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489583.2
Easting: 1314212.7
Elevation: 36.5
Total Depth: 70.5 ft
Termination Criteria:
Test Depth: 54.8 ft



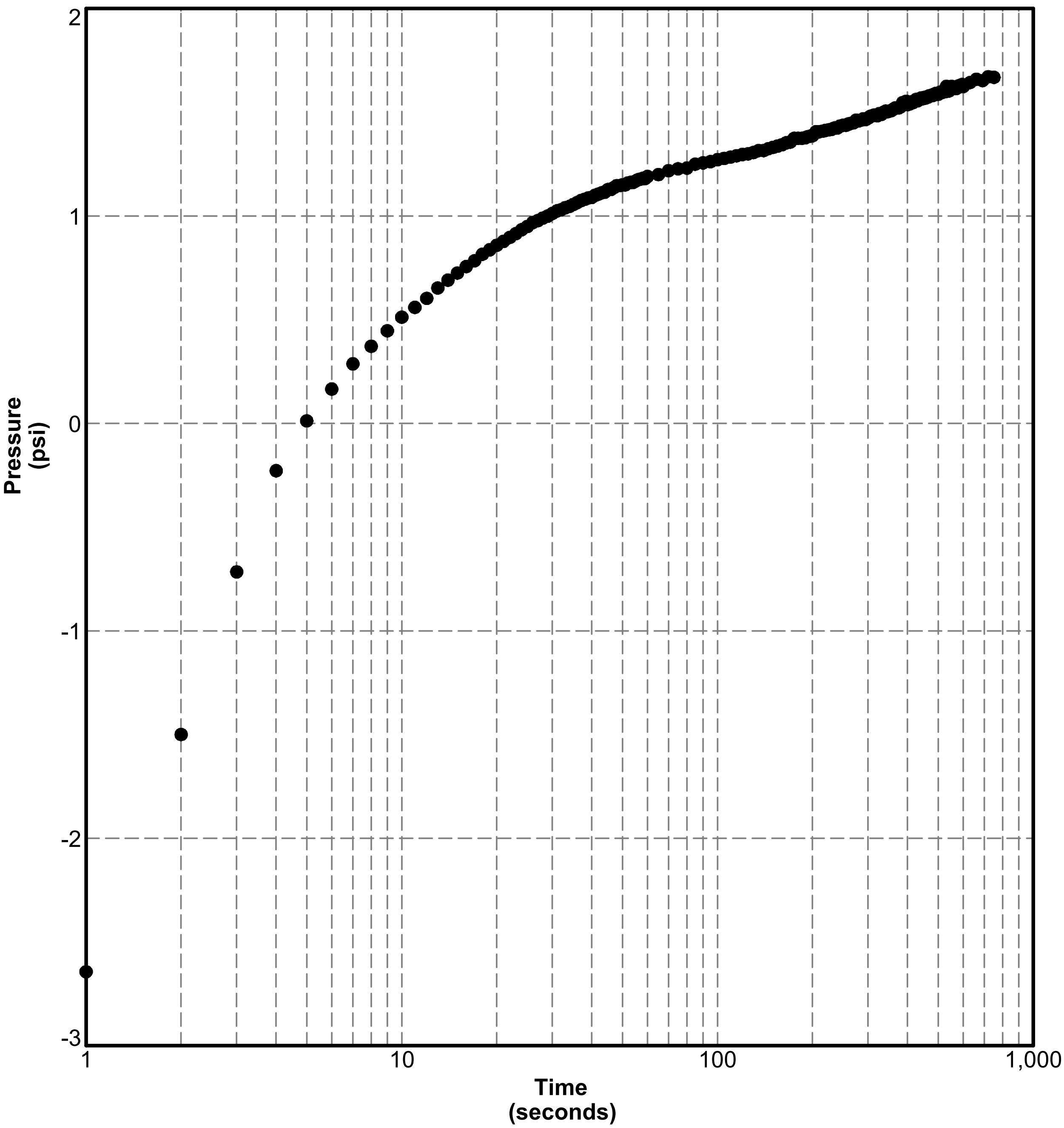
BRN-P1-07C

Electronic File Name: BRN-P1-07C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014 **Northing:** 16489557.7 **Total Depth:** 70.4 ft
Estimated Water Depth: 0 ft **Easting:** 1314117.9 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 30.4 **Test Depth:** 15.9 ft



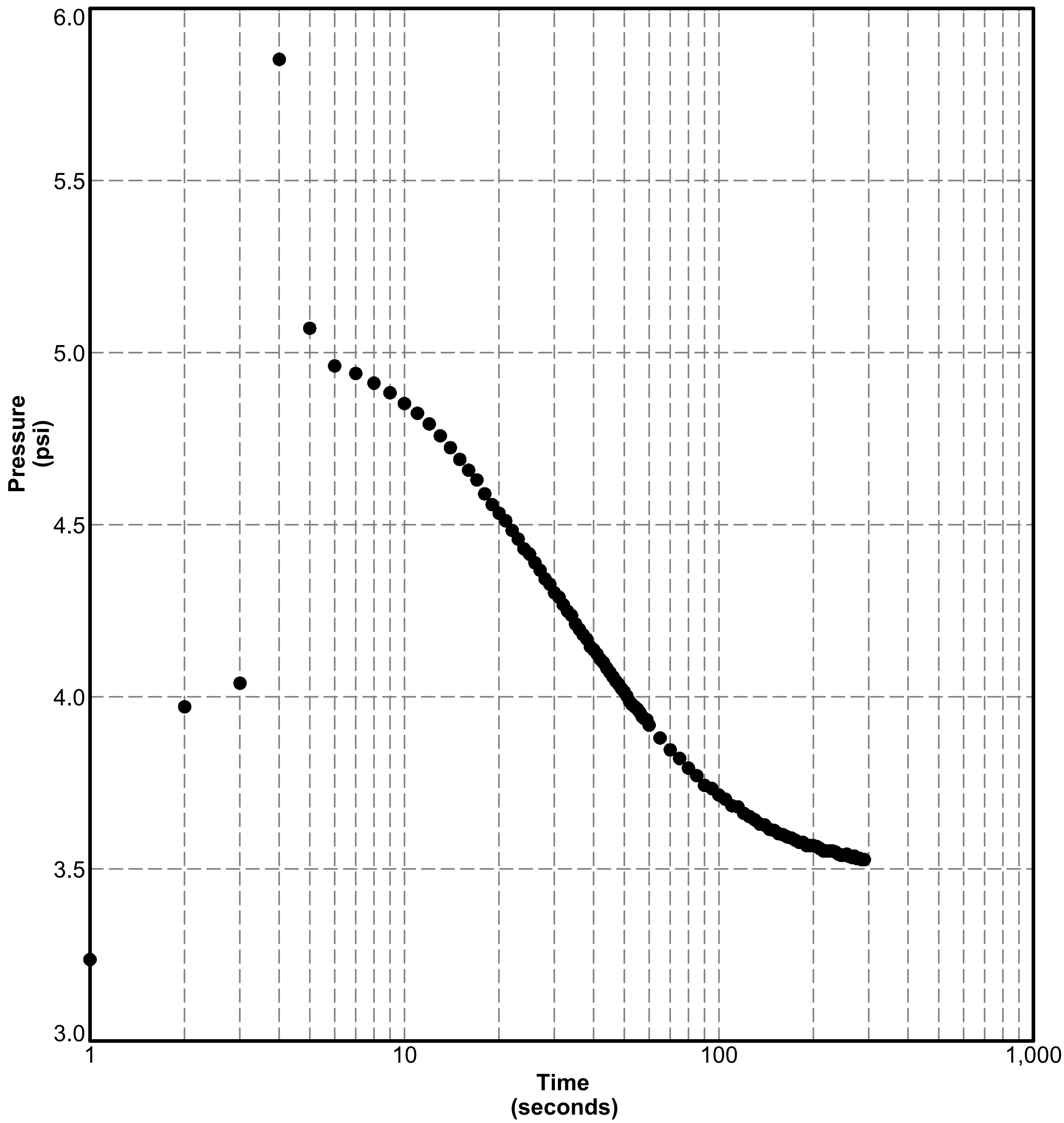
BRN-P1-08C

Electronic File Name: BRN-P1-08C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489557.7
Easting: 1314117.9
Elevation: 30.4
Total Depth: 70.4 ft
Termination Criteria:
Test Depth: 21.5 ft



BRN-P1-08C

Electronic File Name: BRN-P1-08C.dis

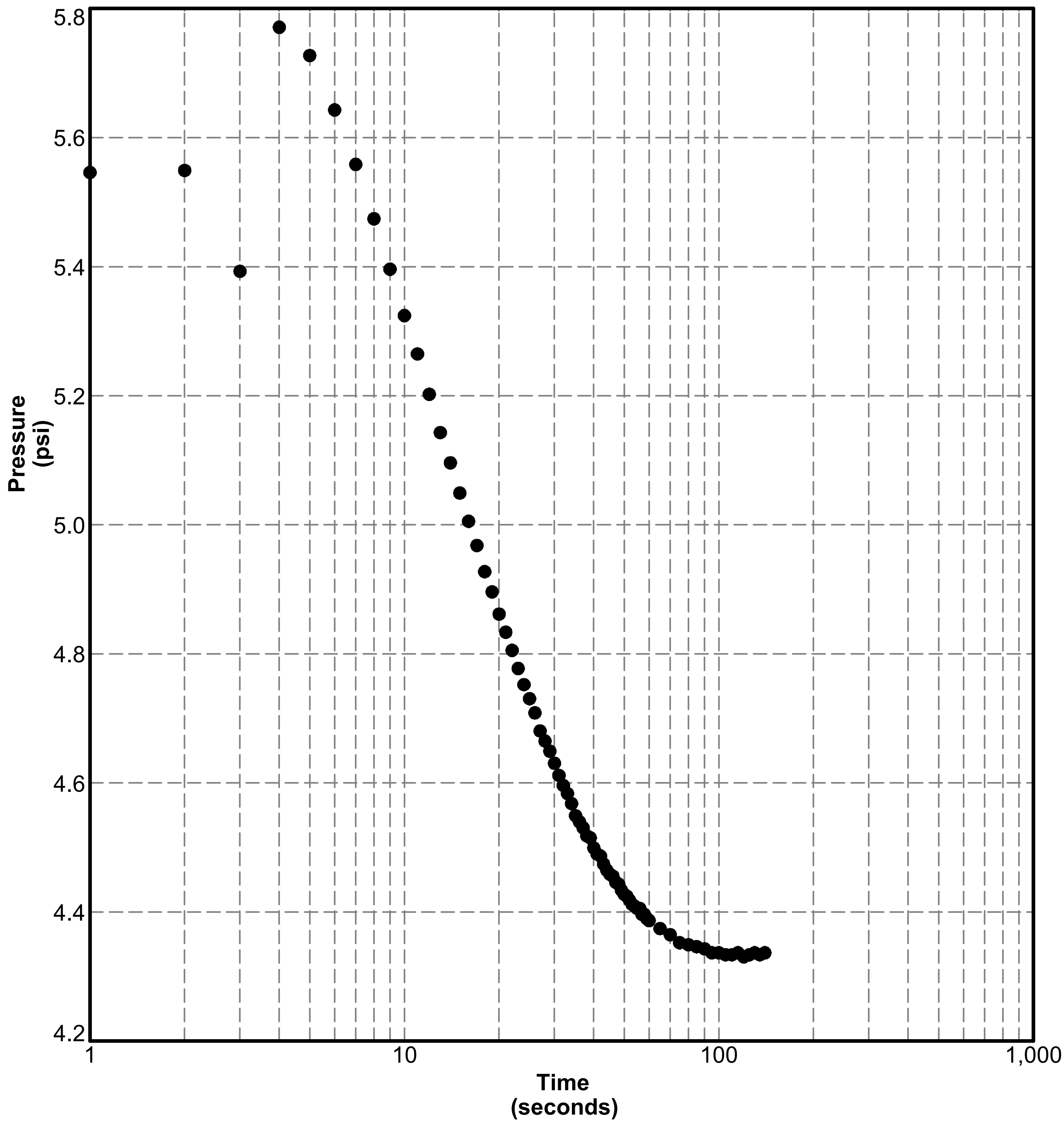
Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489557.7
Easting: 1314117.9
Elevation: 30.4

Total Depth: 70.4 ft
Termination Criteria:
Test Depth: 24.0 ft



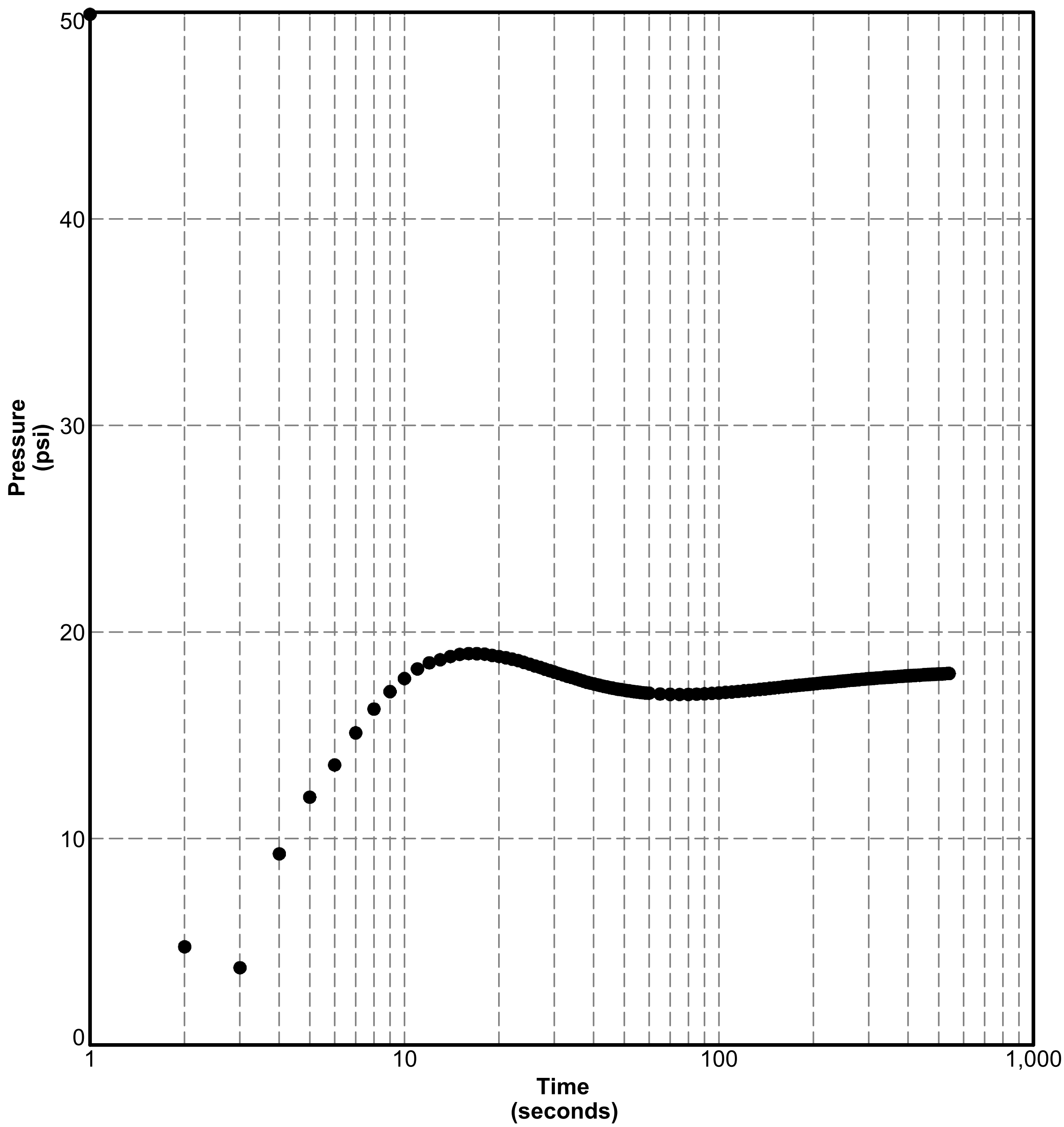
BRN-P1-08C

Electronic File Name: BRN-P1-08C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014 **Northing:** 16489557.7 **Total Depth:** 70.4 ft
Estimated Water Depth: 0 ft **Easting:** 1314117.9 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 30.4 **Test Depth:** 52.0 ft



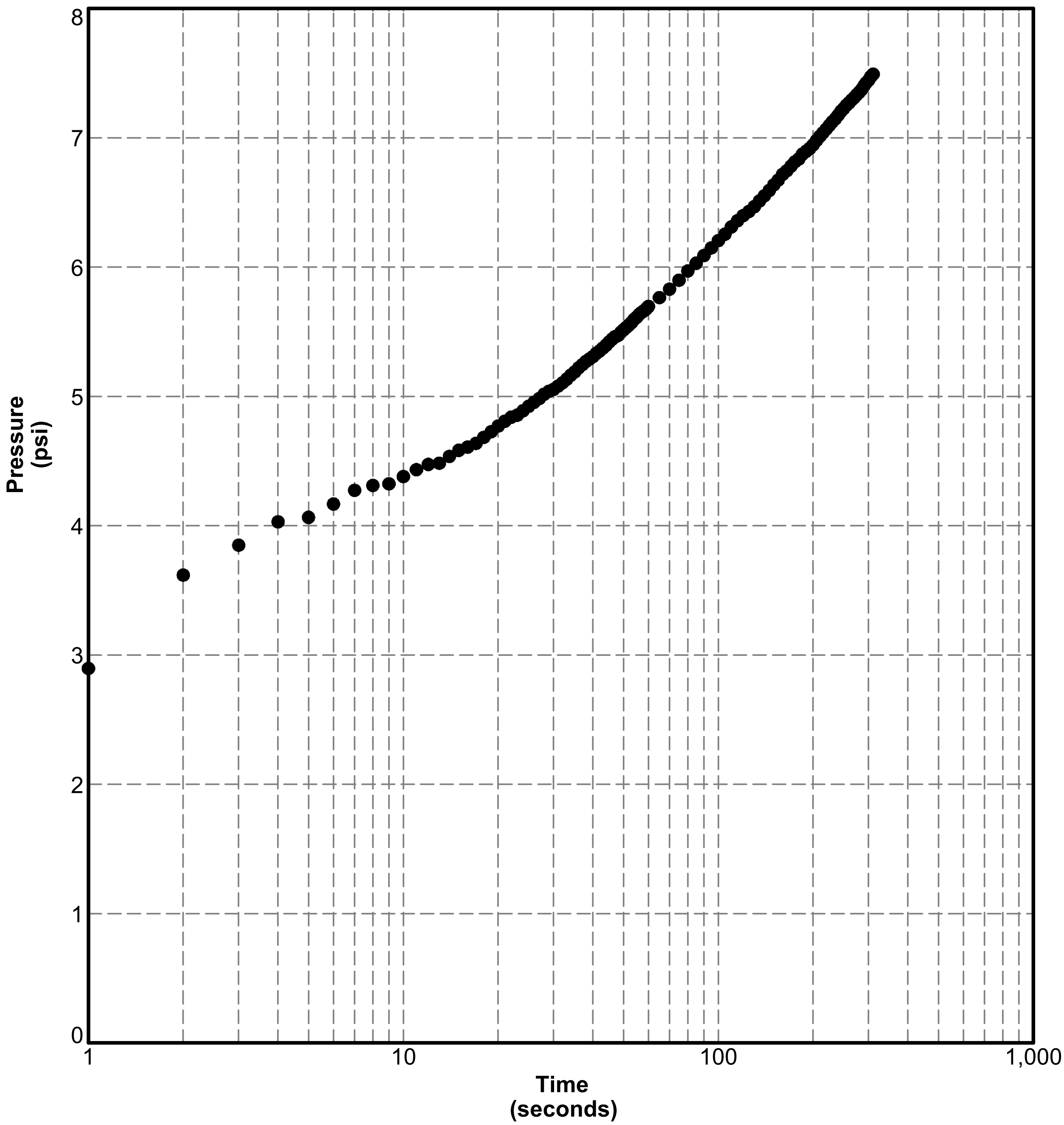
BRN-P1-08C

Electronic File Name: BRN-P1-08C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489801.0
Easting: 1314112.1
Elevation: 37.9
Total Depth: 63.8 ft
Termination Criteria:
Test Depth: 32.5 ft



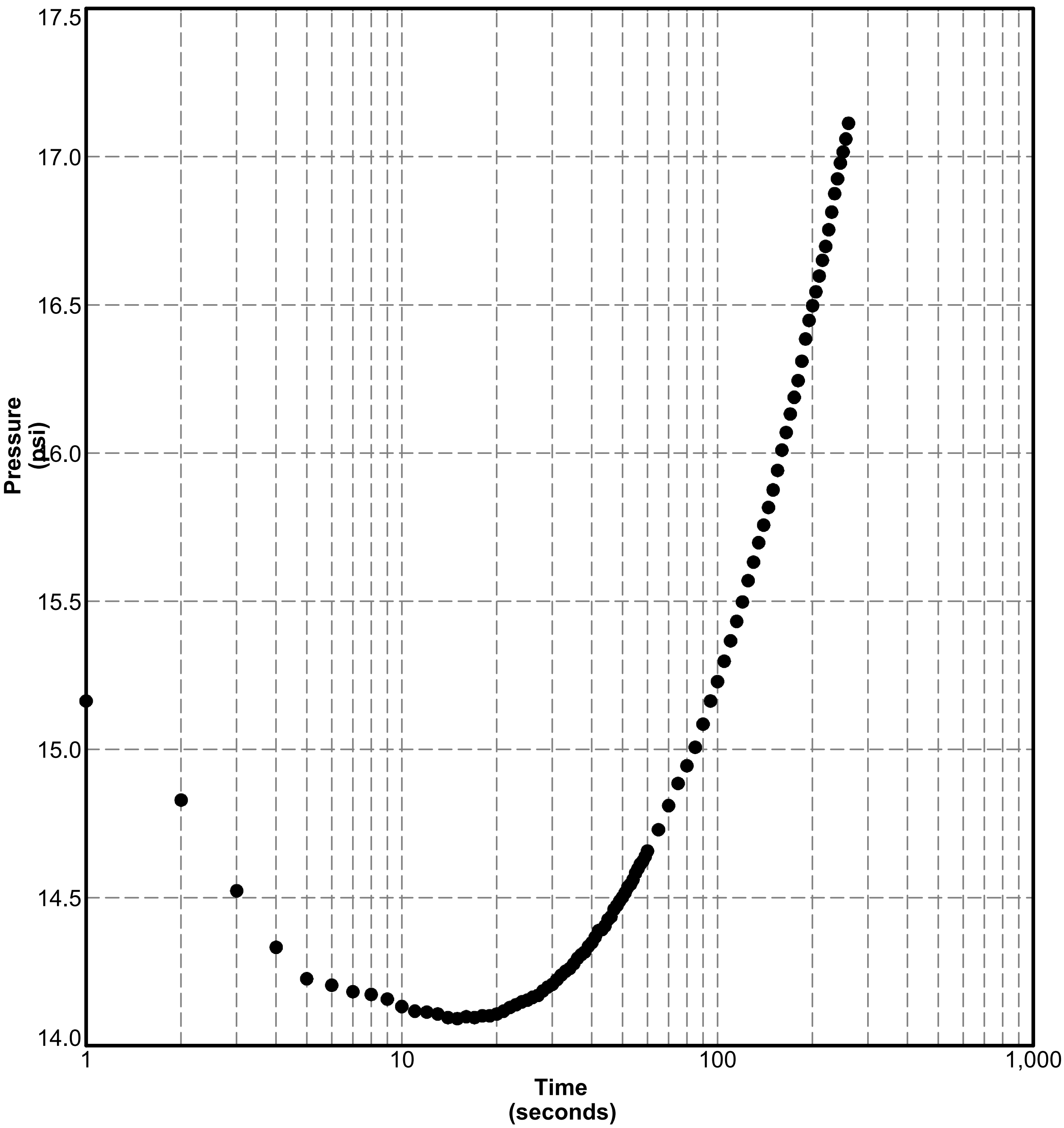
BRN-P1-10C

Electronic File Name: BRN-P1-10C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014 **Northing:** 16489801.0 **Total Depth:** 63.8 ft
Estimated Water Depth: 0 ft **Easting:** 1314112.1 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 37.9 **Test Depth:** 46.1 ft



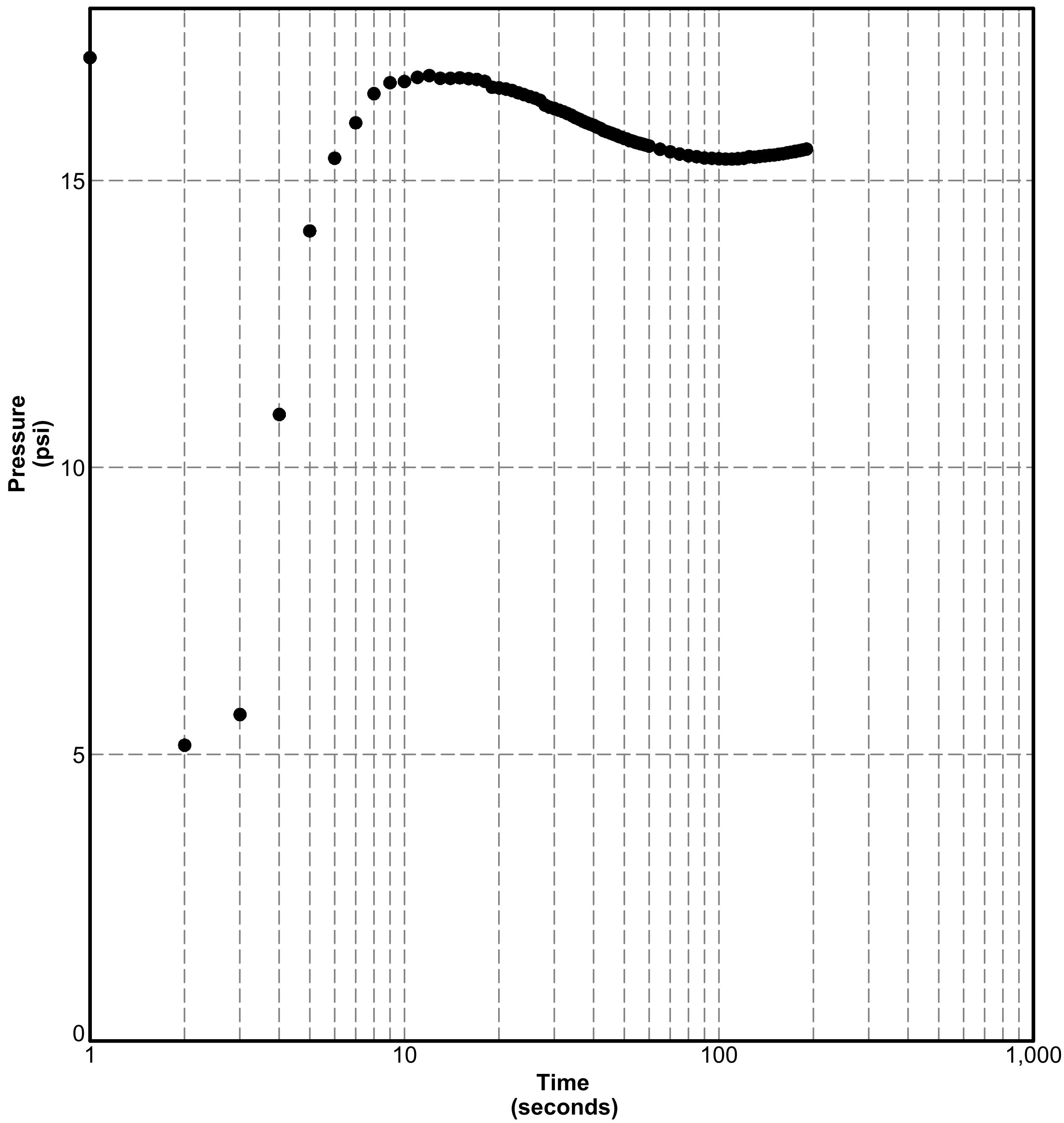
BRN-P1-10C

 Electronic File Name: BRN-P1-10C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014 **Northing:** 16489801.0 **Total Depth:** 63.8 ft
Estimated Water Depth: 0 ft **Easting:** 1314112.1 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 37.9 **Test Depth:** 55.0 ft



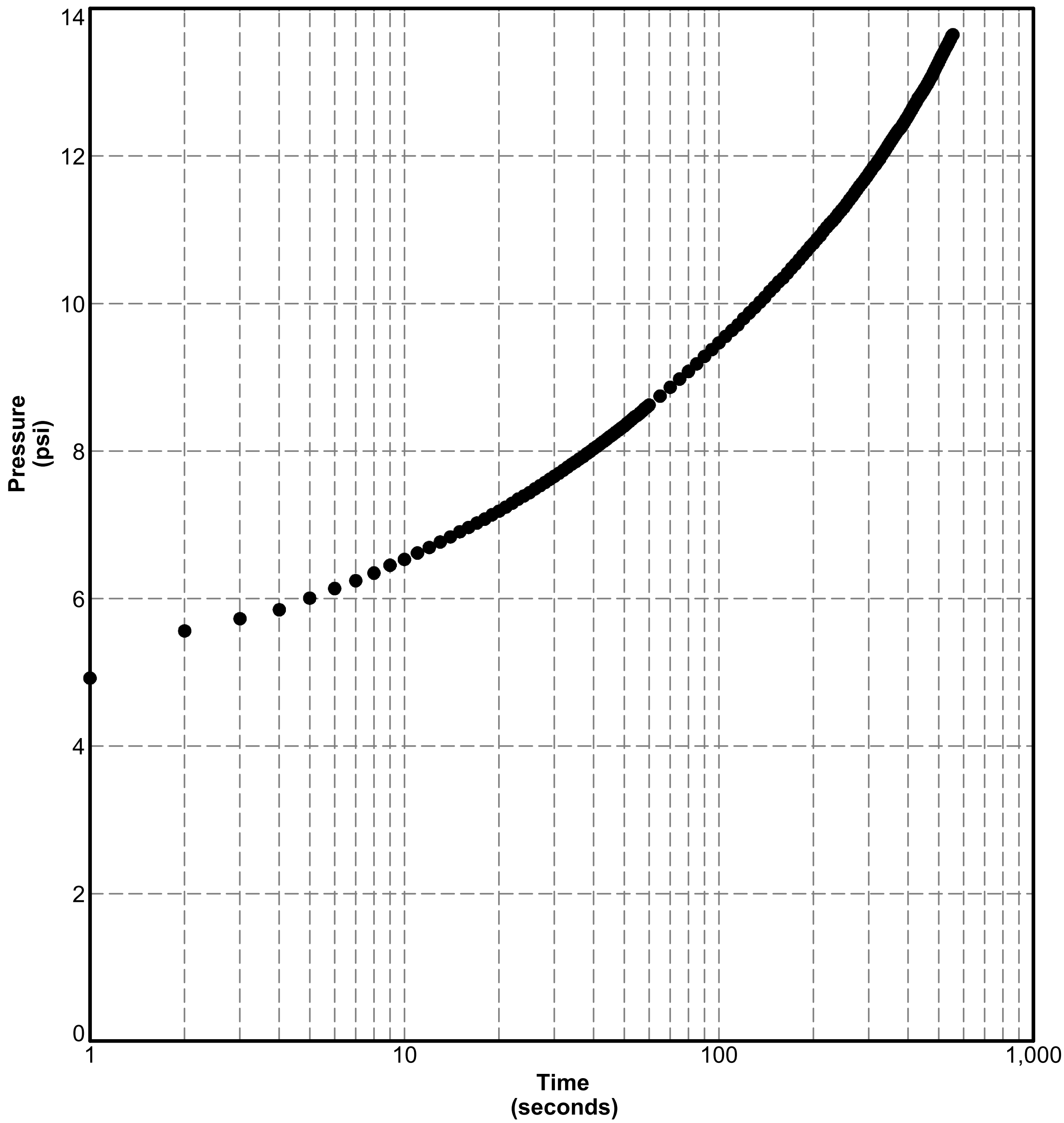
BRN-P1-10C

Electronic File Name: BRN-P1-10C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489758.2
Easting: 1314016.0
Elevation: 27.9
Total Depth: 60.2 ft
Termination Criteria:
Test Depth: 34.8 ft



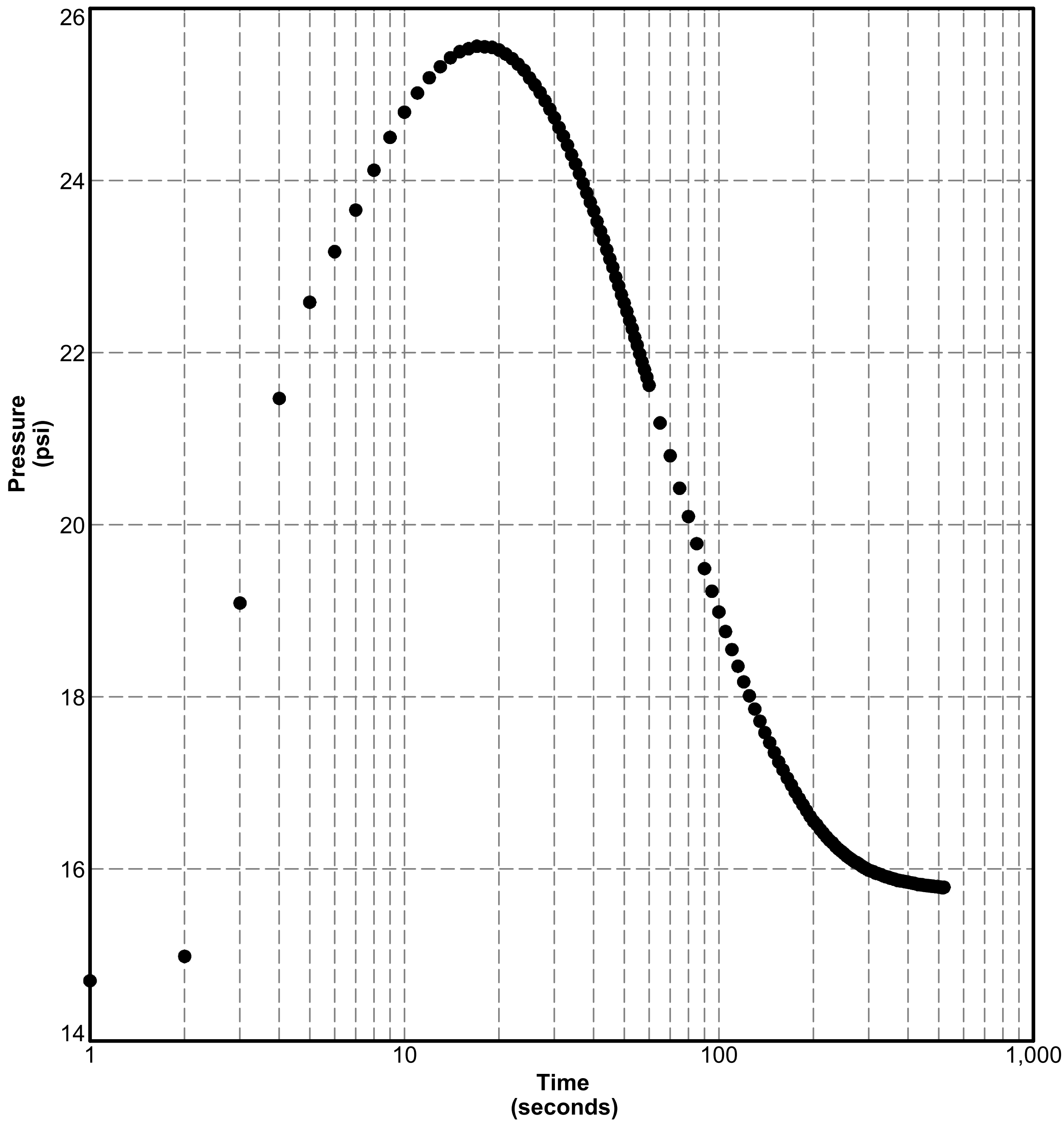
BRN-P1-11C

Electronic File Name: BRN-P1-11C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014 **Northing:** 16489758.2 **Total Depth:** 60.2 ft
Estimated Water Depth: 0 ft **Easting:** 1314016.0 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 27.9 **Test Depth:** 45.1 ft



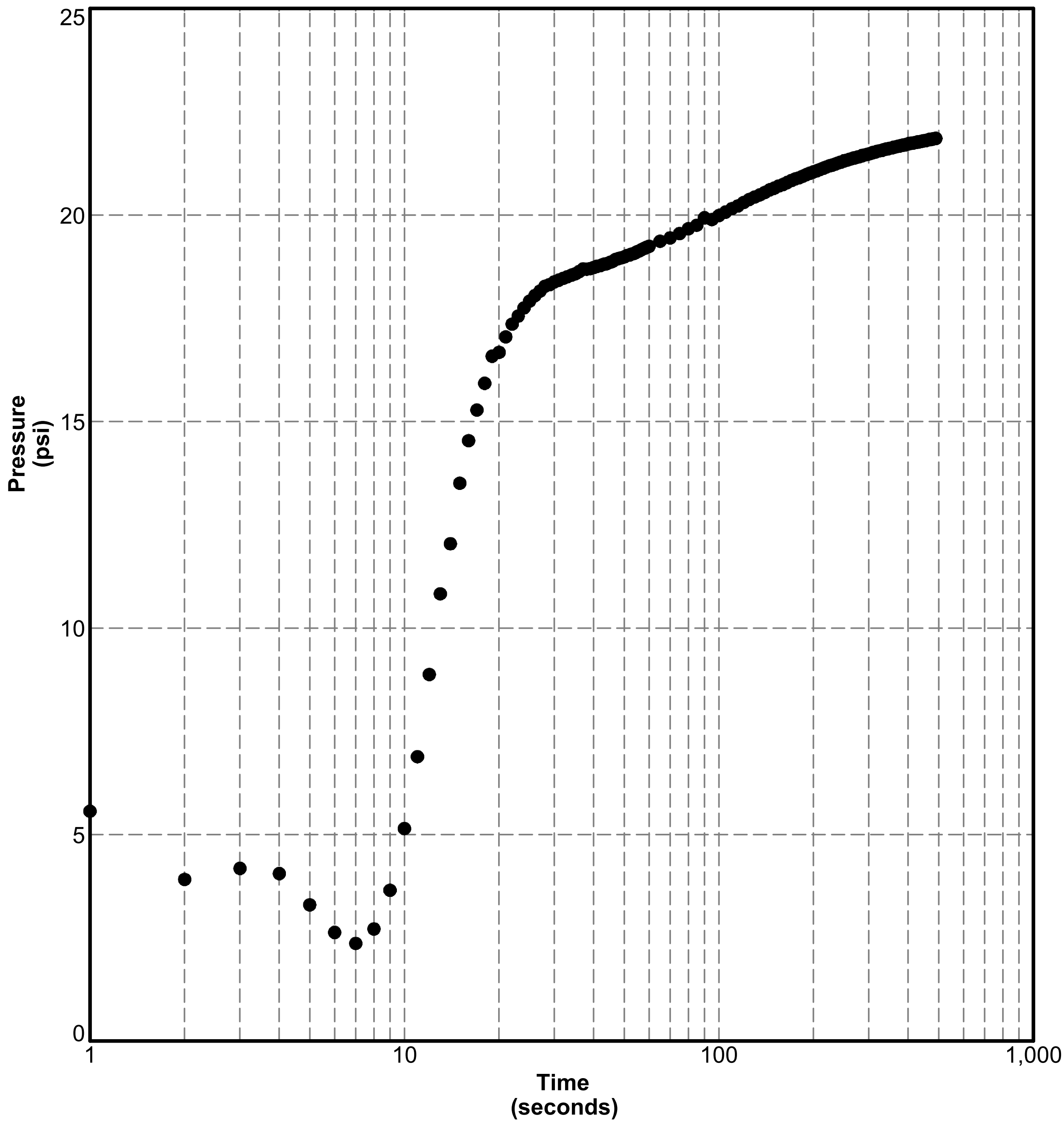
BRN-P1-11C

Electronic File Name: BRN-P1-11C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489758.2
Easting: 1314016.0
Elevation: 27.9
Total Depth: 60.2 ft
Termination Criteria:
Test Depth: 60.2 ft



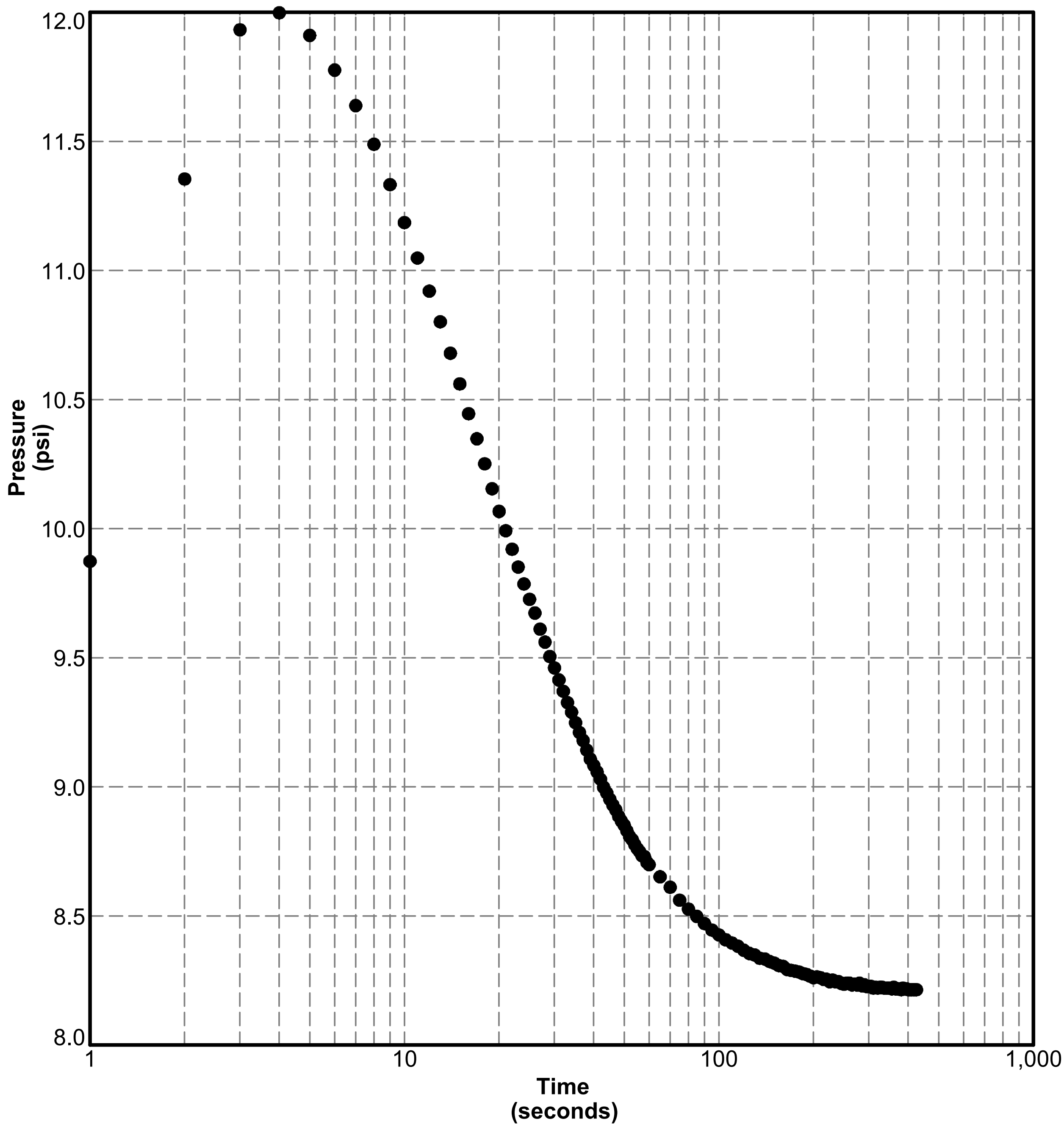
BRN-P1-11C

Electronic File Name: BRN-P1-11C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489644.9
Easting: 1314068.6
Elevation: 29.9
Total Depth: 51.2 ft
Termination Criteria:
Test Depth: 30.7 ft



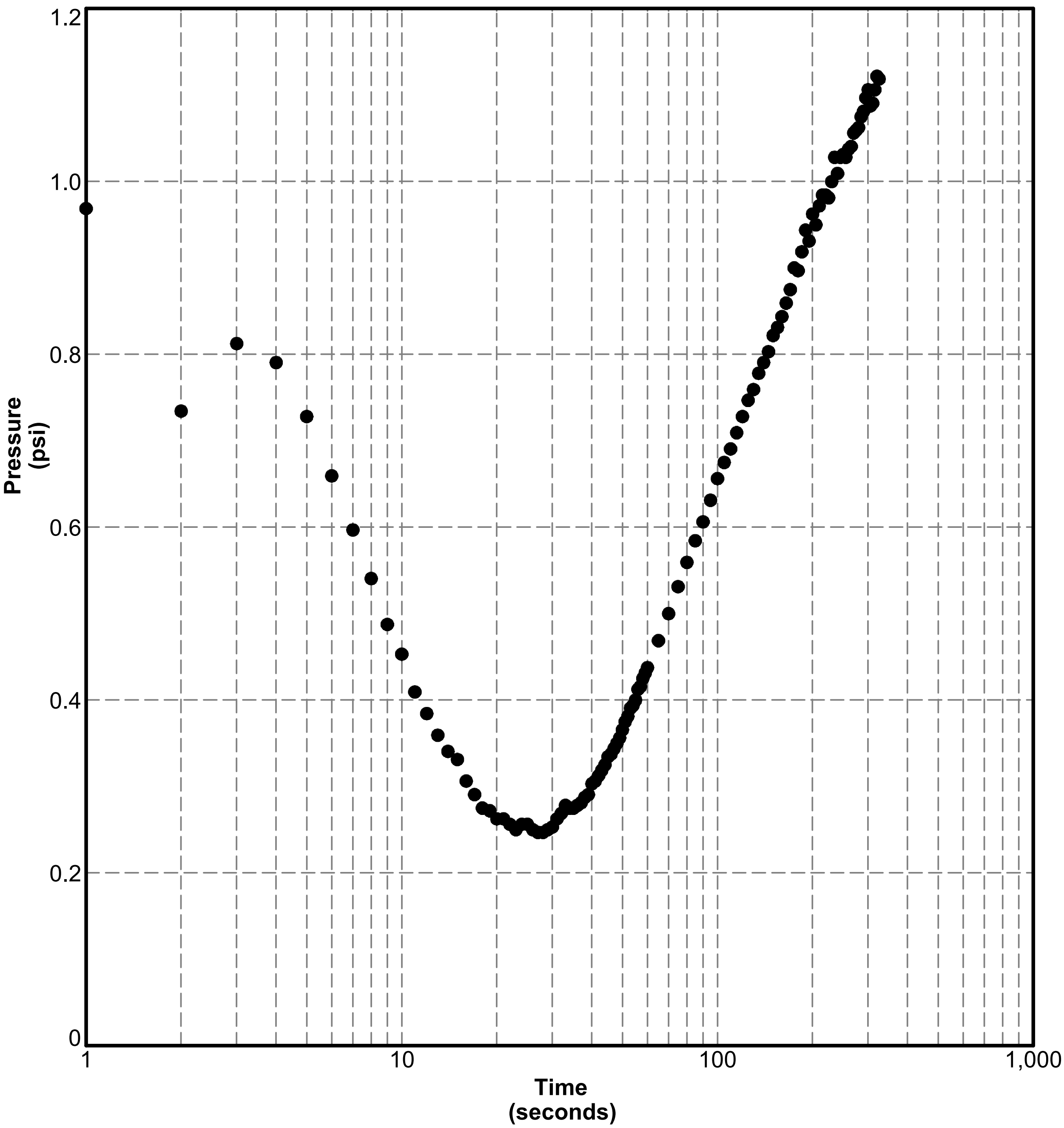
BRN-P2-15C

Electronic File Name: BRN-P2-15C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489595.3
Easting: 1314085.1
Elevation: 30.3
Total Depth: 50.4 ft
Termination Criteria:
Test Depth: 12.6 ft

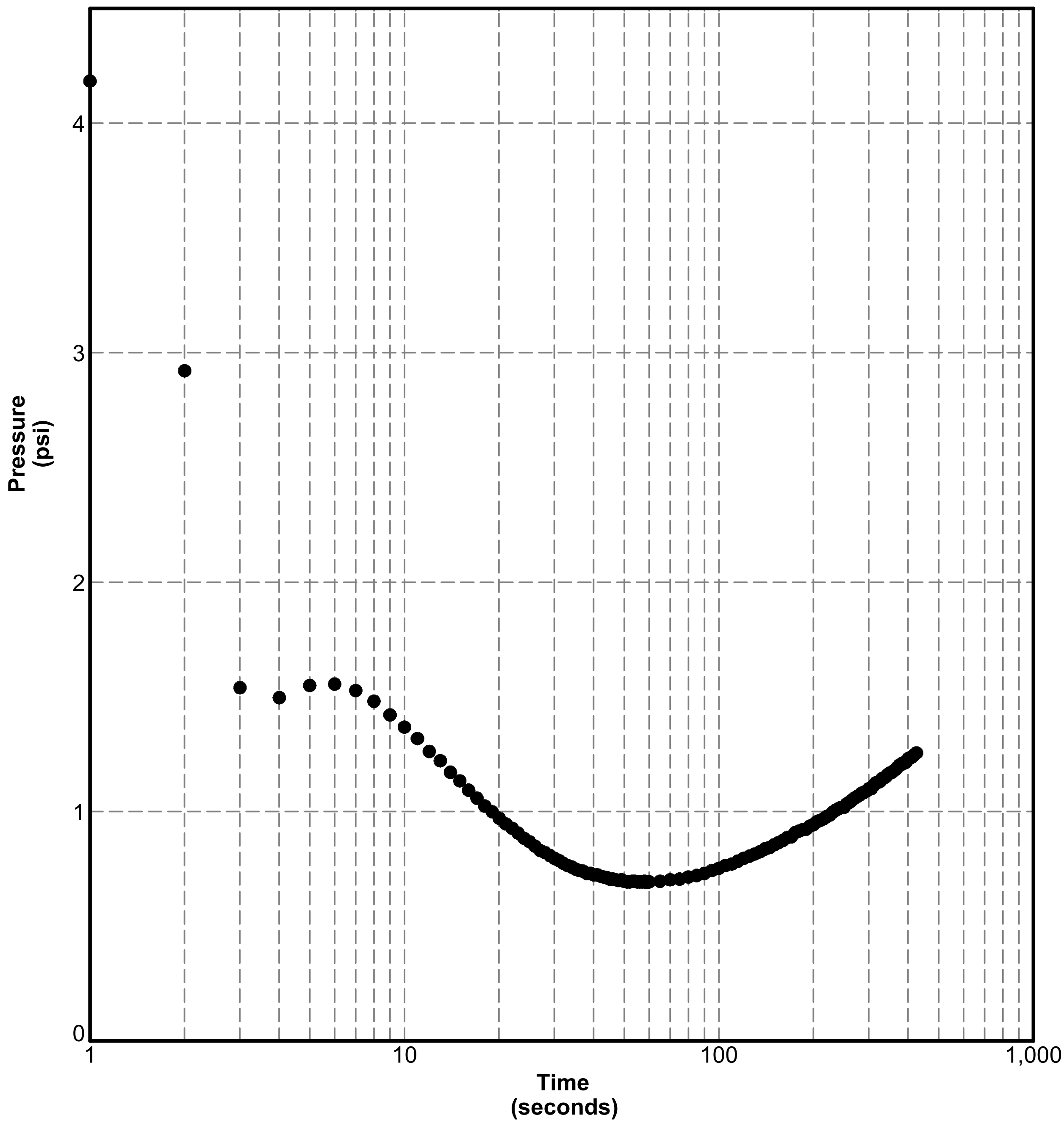


BRN-P2-16C

Electronic File Name: BRN-P2-16C.dis

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489595.3
Easting: 1314085.1
Elevation: 30.3
Total Depth: 50.4 ft
Termination Criteria:
Test Depth: 13.3 ft



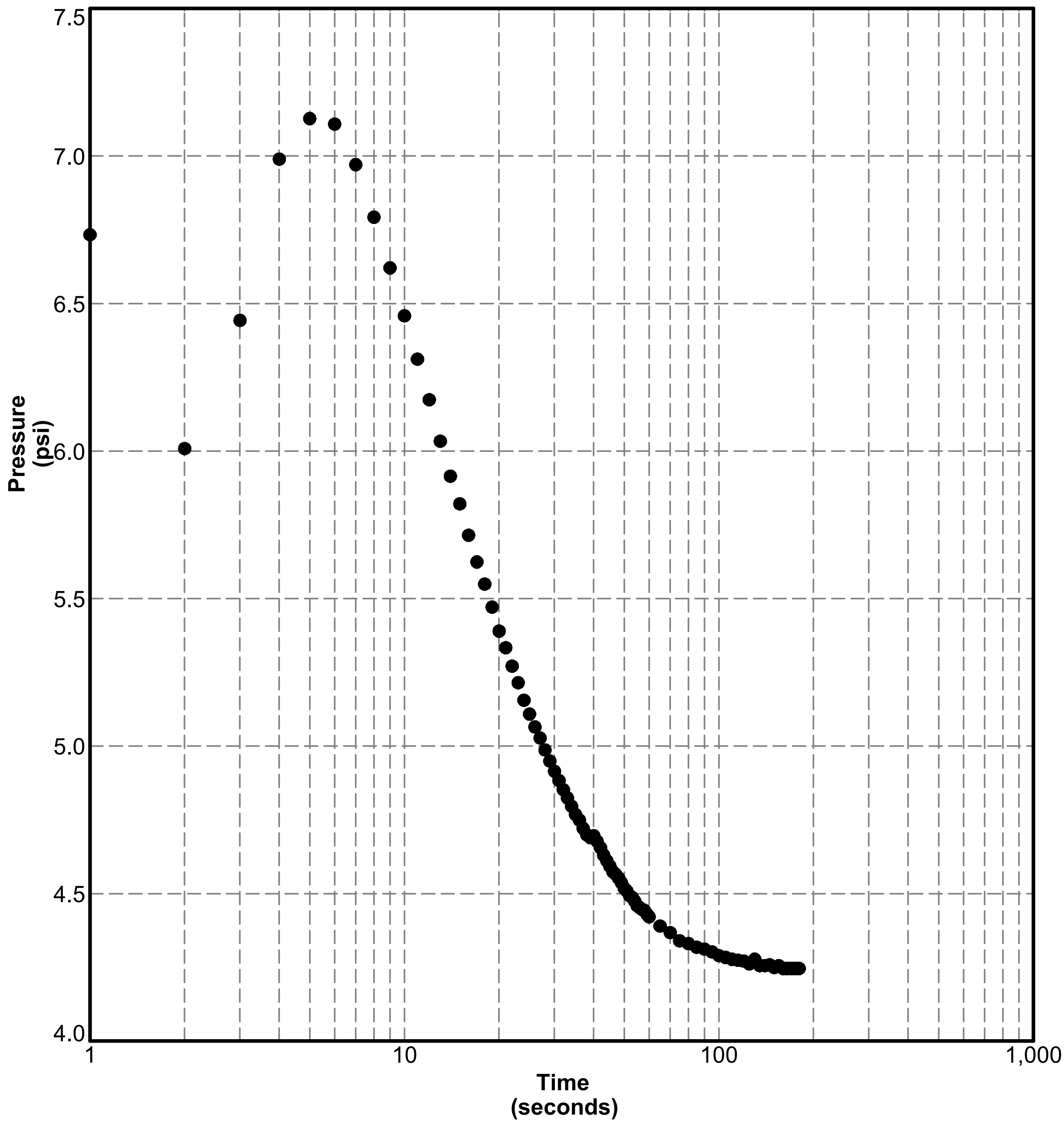
BRN-P2-16C

Electronic File Name: BRN-P2-16C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014 **Northing:** 16489595.3 **Total Depth:** 50.4 ft
Estimated Water Depth: 0 ft **Easting:** 1314085.1 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 30.3 **Test Depth:** 19.4 ft



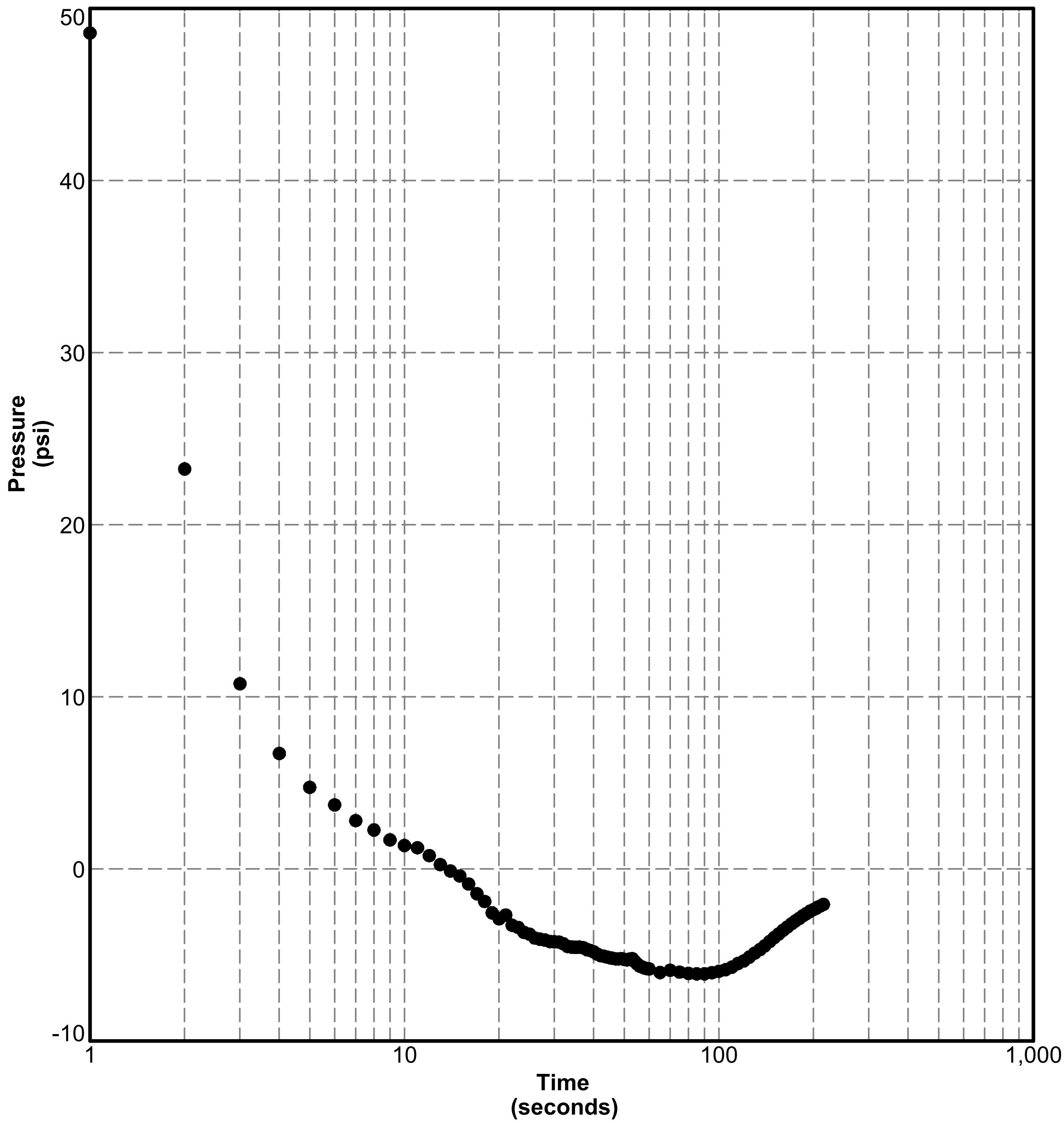
BRN-P2-16C

Electronic File Name: BRN-P2-16C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489595.3
Easting: 1314085.1
Elevation: 30.3
Total Depth: 50.4 ft
Termination Criteria:
Test Depth: 39.4 ft



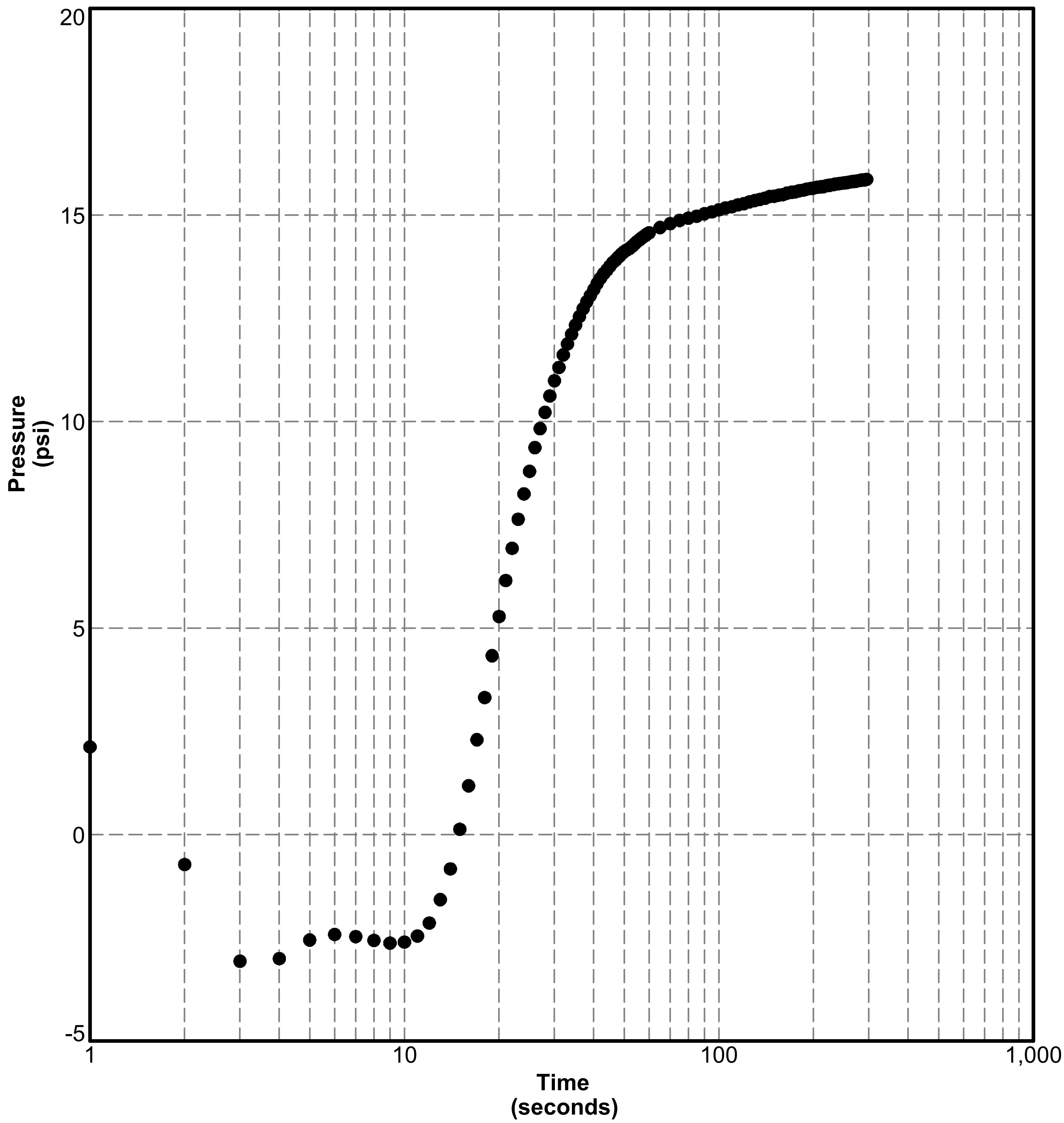
BRN-P2-16C

Electronic File Name: BRN-P2-16C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 30, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489595.3
Easting: 1314085.1
Elevation: 30.3
Total Depth: 50.4 ft
Termination Criteria:
Test Depth: 46.8 ft



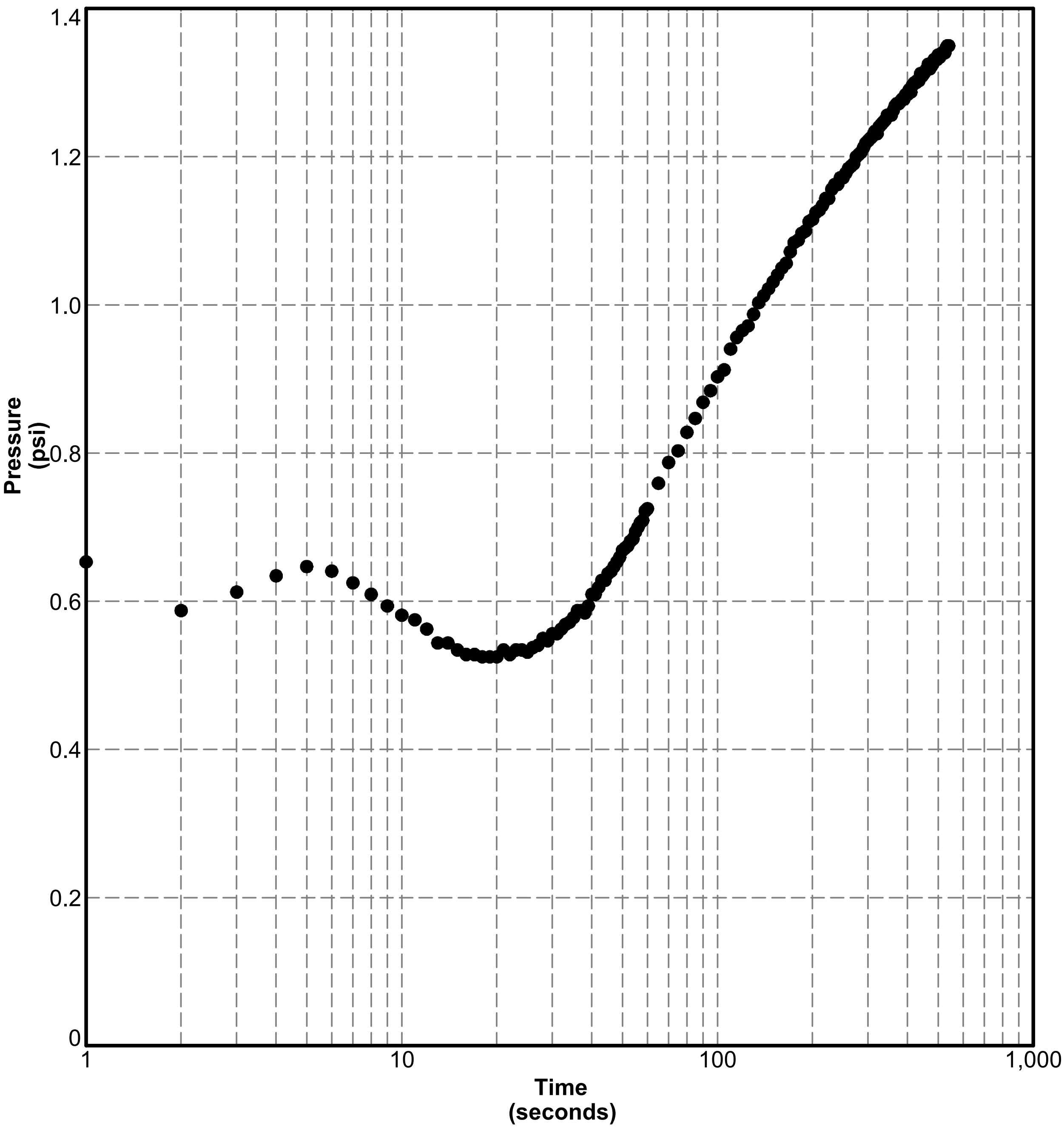
BRN-P2-16C

Electronic File Name: BRN-P2-16C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489554.7
Easting: 1314100.7
Elevation: 30.4
Total Depth: 50.5 ft
Termination Criteria:
Test Depth: 14.3 ft



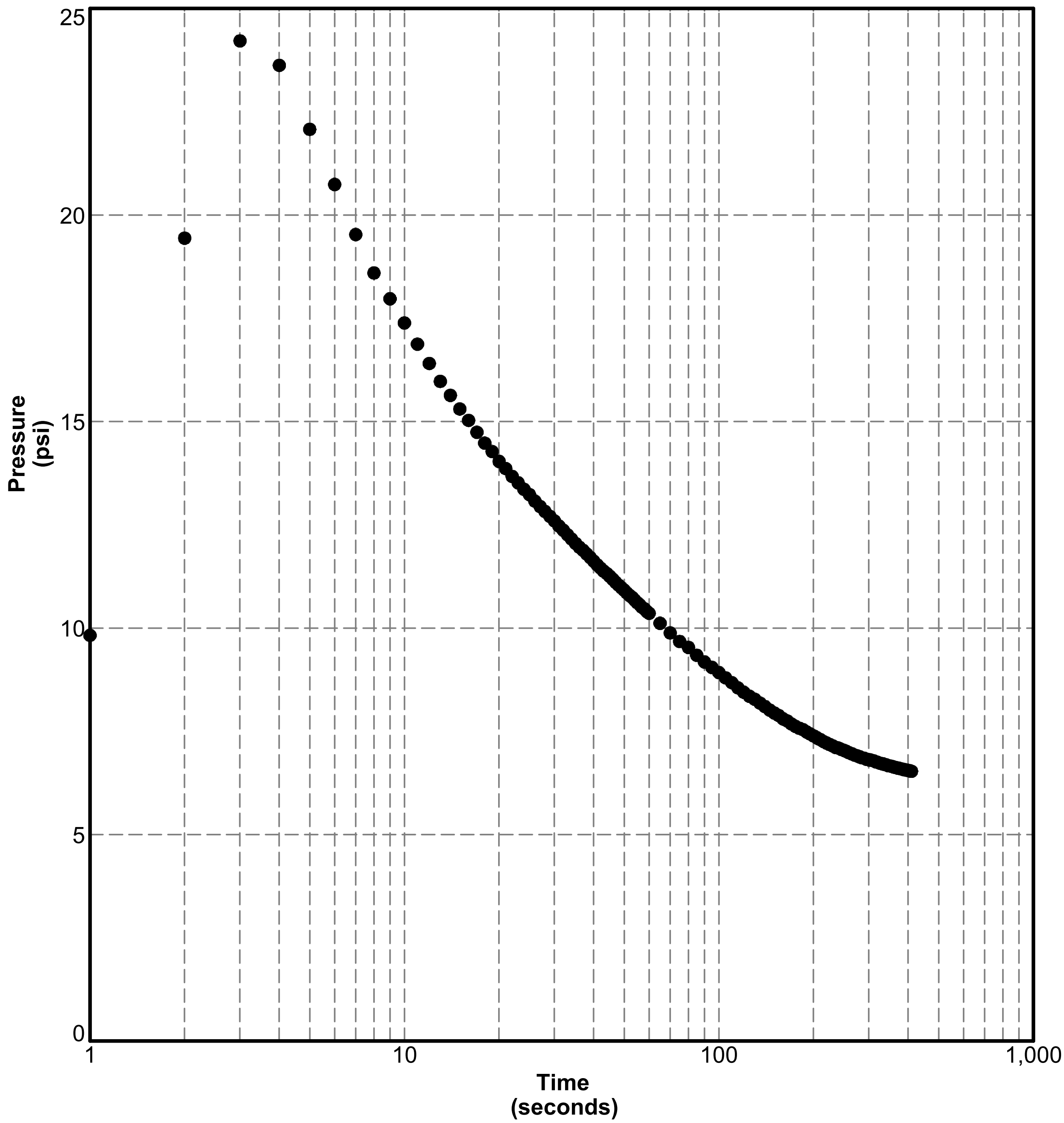
BRN-P2-17C

Electronic File Name: BRN-P2-17C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489554.7
Easting: 1314100.7
Elevation: 30.4
Total Depth: 50.5 ft
Termination Criteria:
Test Depth: 28.5 ft



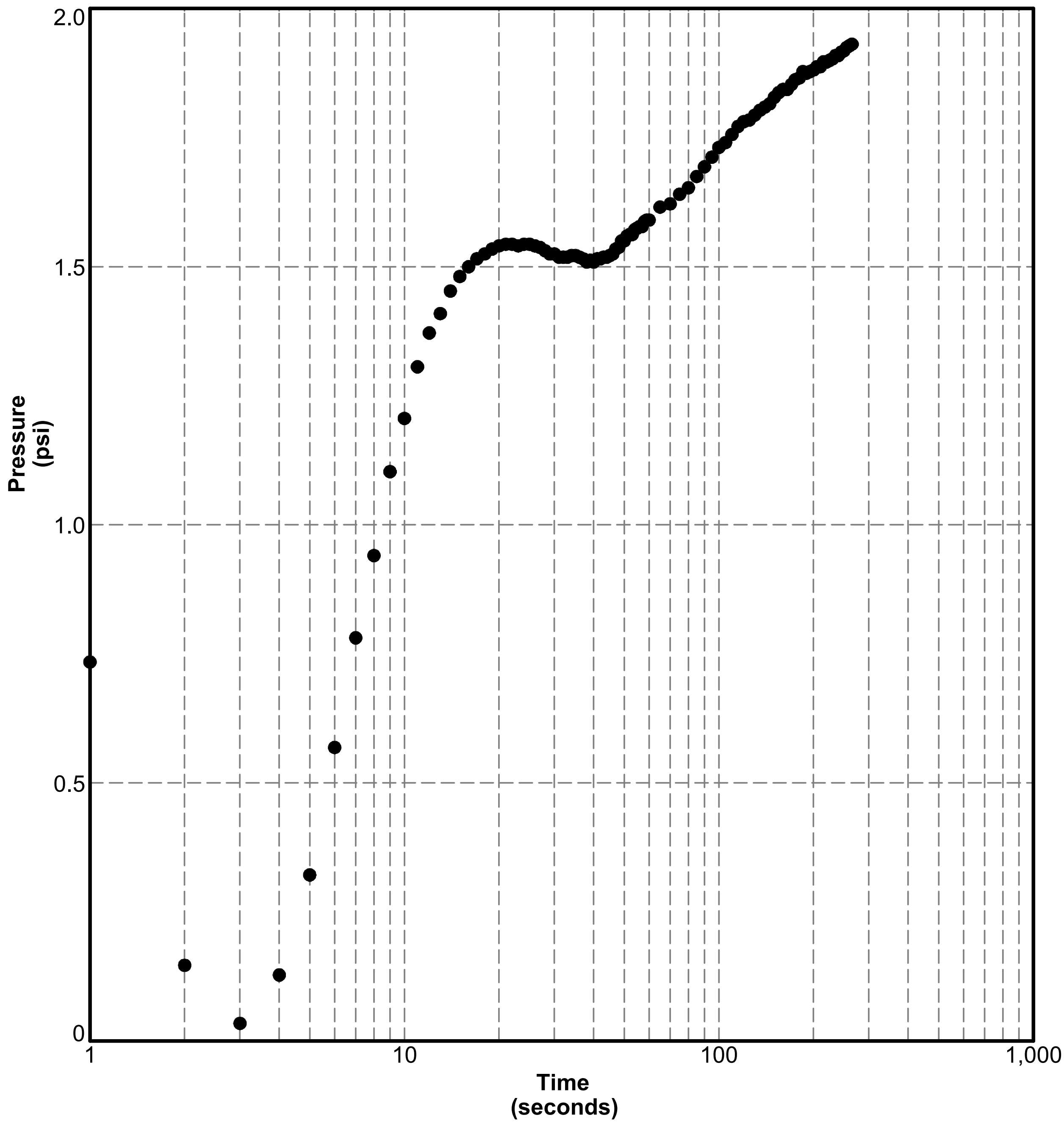
BRN-P2-17C

Electronic File Name: BRN-P2-17C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014 **Northing:** 16489514.8 **Total Depth:** 50.9 ft
Estimated Water Depth: 0 ft **Easting:** 1314139.8 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 30.5 **Test Depth:** 14.6 ft



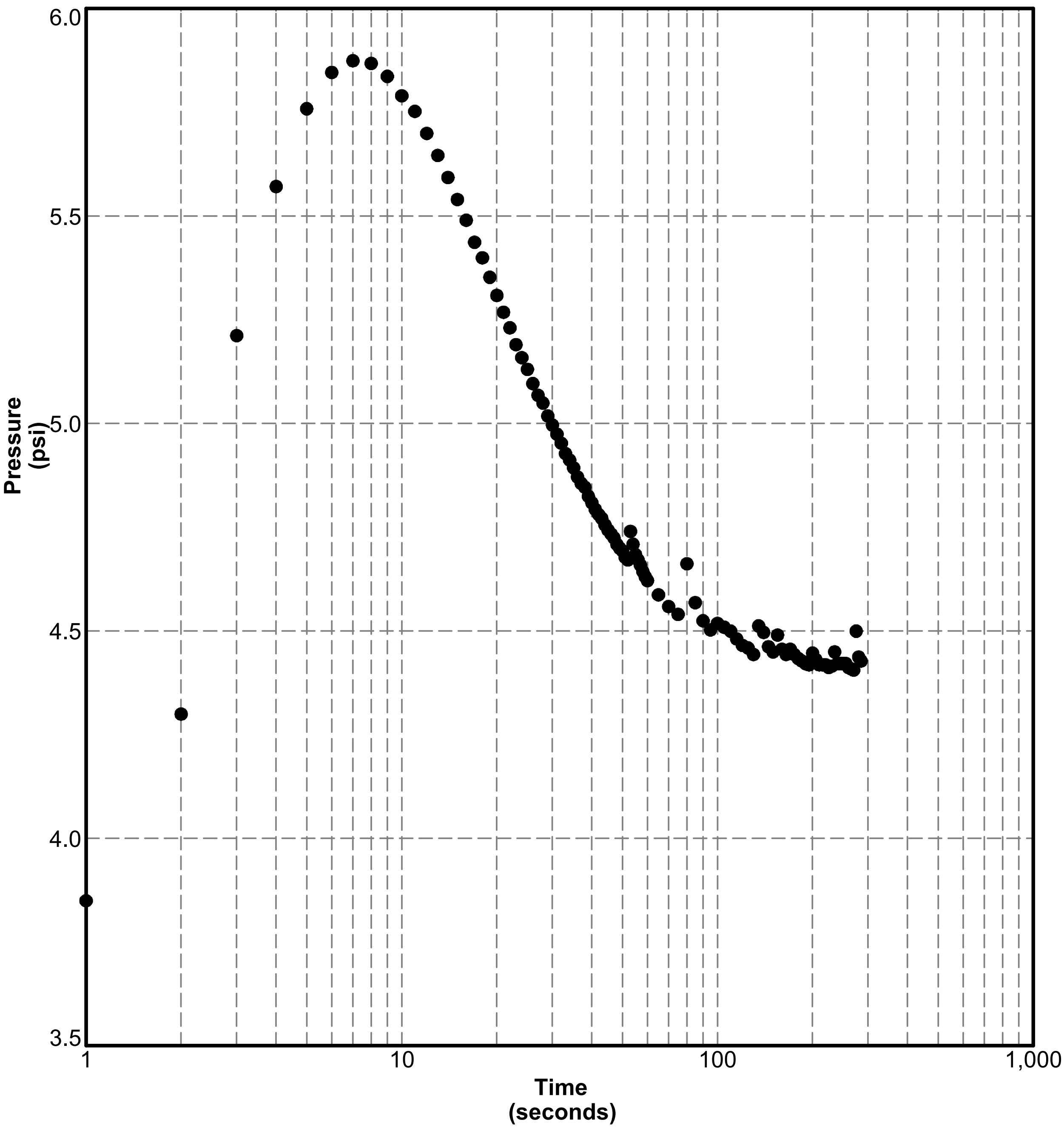
BRN-P2-18C

Electronic File Name: BRN-P2-18C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489514.8
Easting: 1314139.8
Elevation: 30.5
Total Depth: 50.9 ft
Termination Criteria:
Test Depth: 24.3 ft



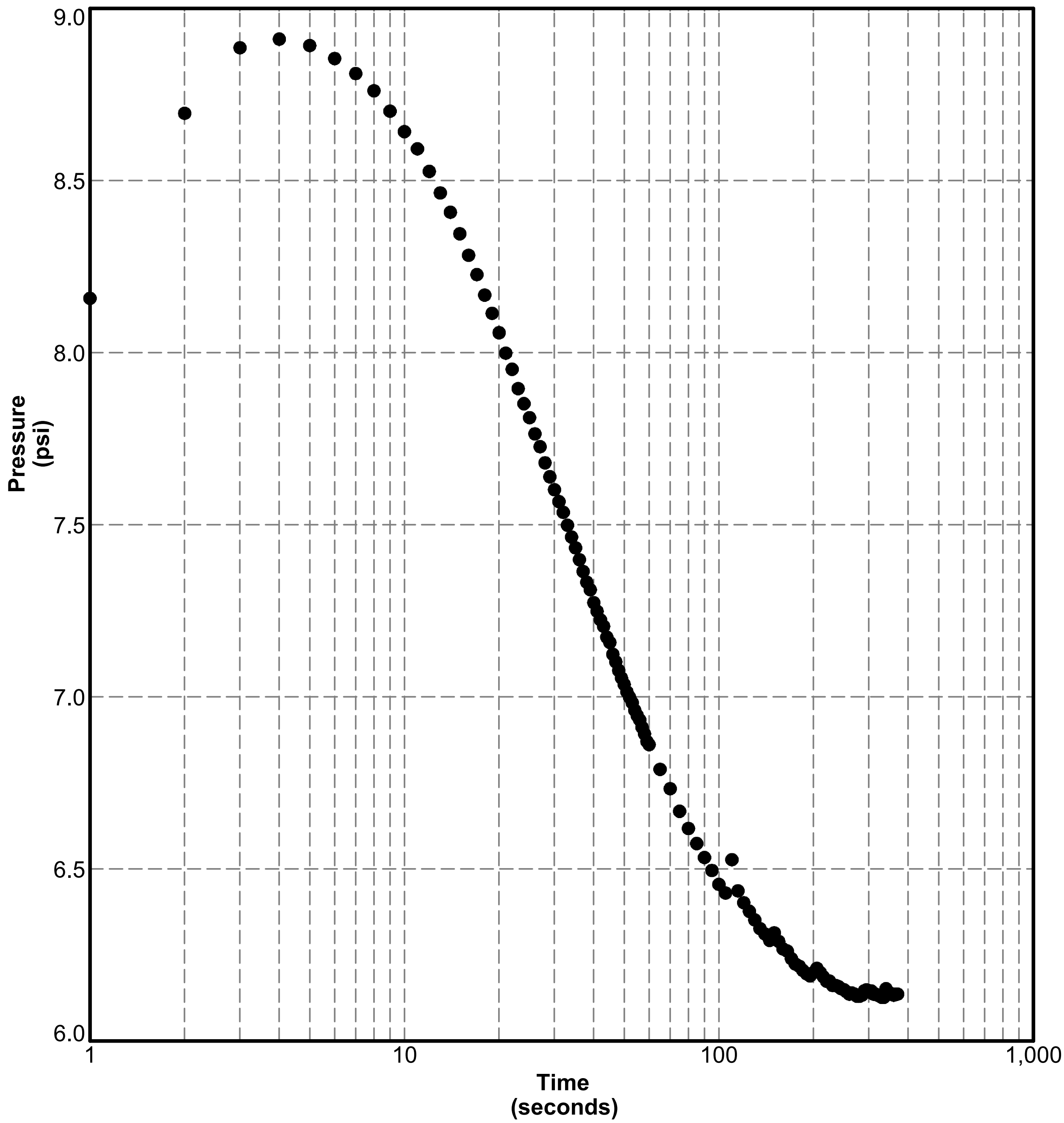
BRN-P2-18C

Electronic File Name: BRN-P2-18C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489470.3
Easting: 1314166.9
Elevation: 30.6
Total Depth: 70.4 ft
Termination Criteria:
Test Depth: 25.3 ft



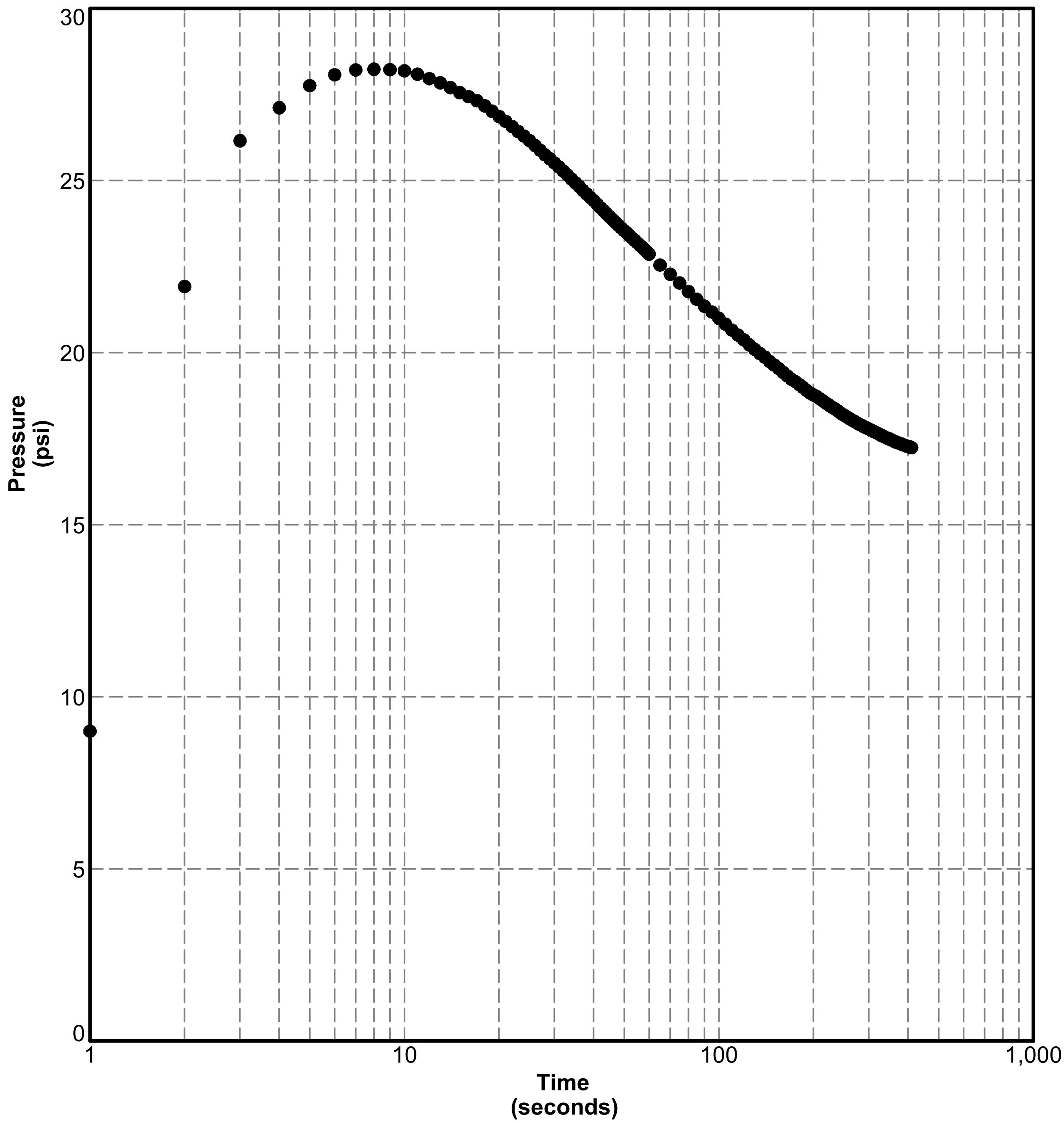
BRN-P2-19C

Electronic File Name: BRN-P2-19C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489470.3
Easting: 1314166.9
Elevation: 30.6
Total Depth: 70.4 ft
Termination Criteria:
Test Depth: 46.4 ft



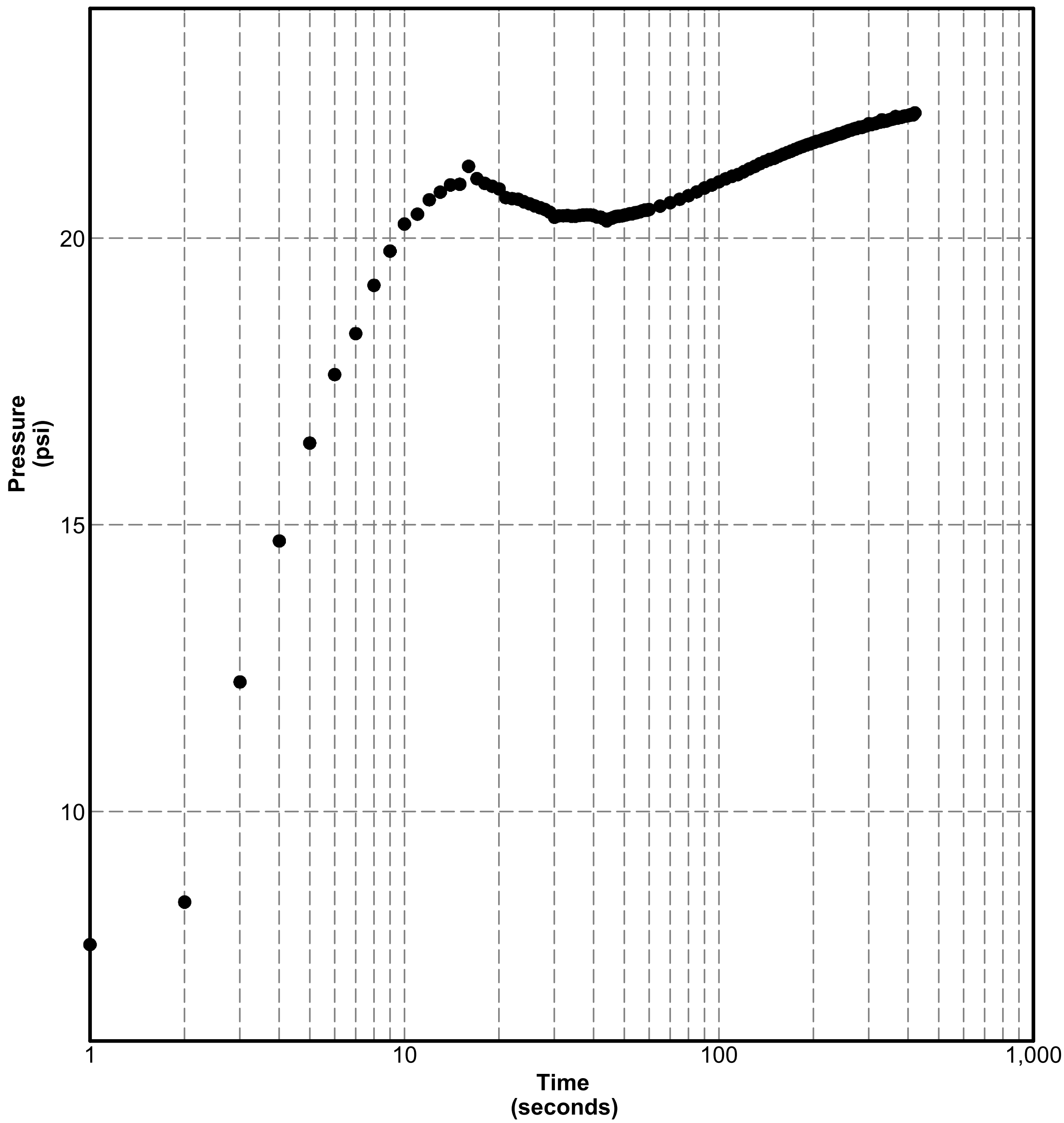
BRN-P2-19C

Electronic File Name: BRN-P2-19C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 29, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489470.3
Easting: 1314166.9
Elevation: 30.6
Total Depth: 70.4 ft
Termination Criteria:
Test Depth: 62.3 ft



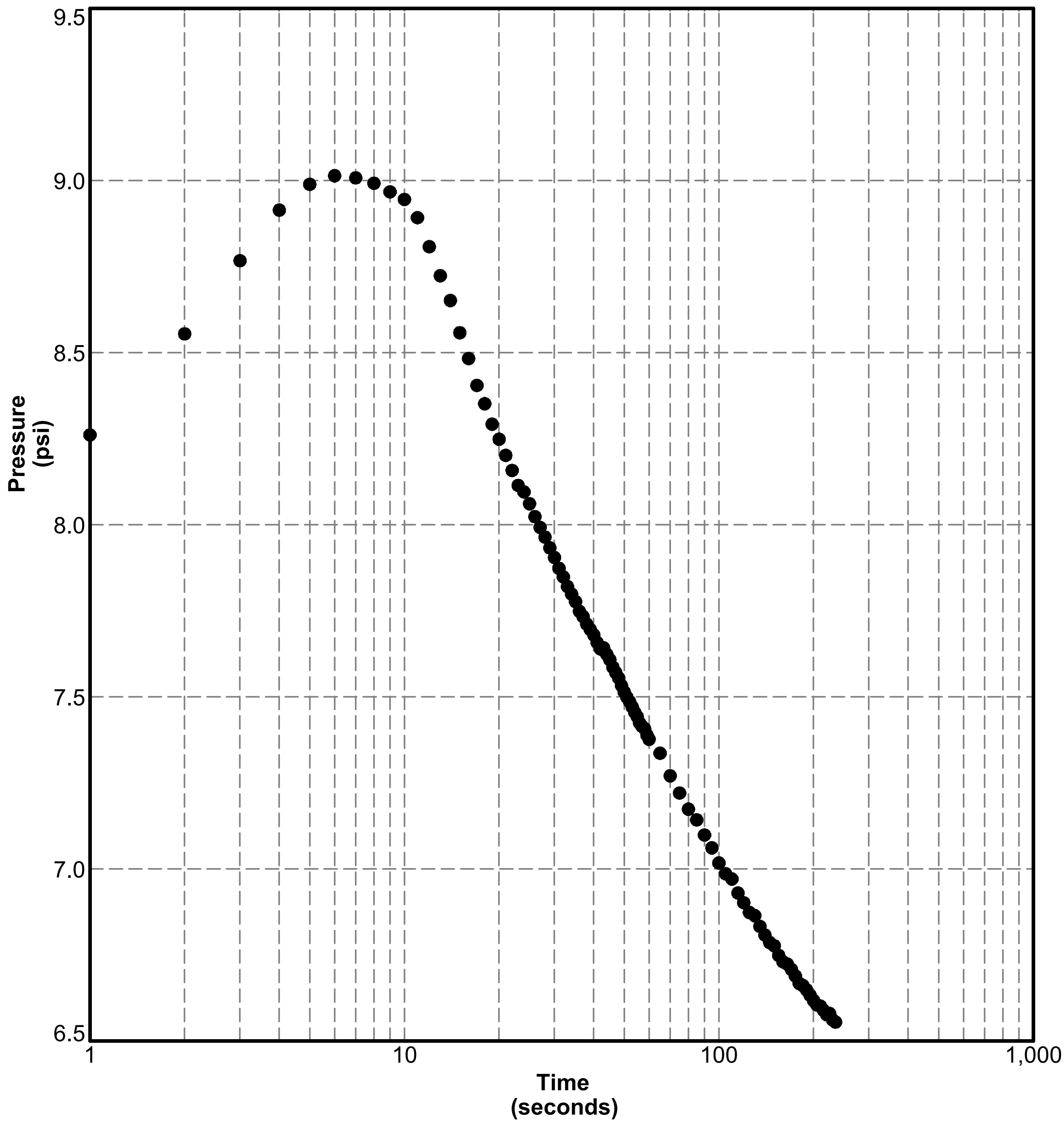
BRN-P2-19C

Electronic File Name: BRN-P2-19C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014 **Northing:** 16489365.3 **Total Depth:** 66.9 ft
Estimated Water Depth: 0 ft **Easting:** 1314186.3 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 30.7 **Test Depth:** 22.1 ft



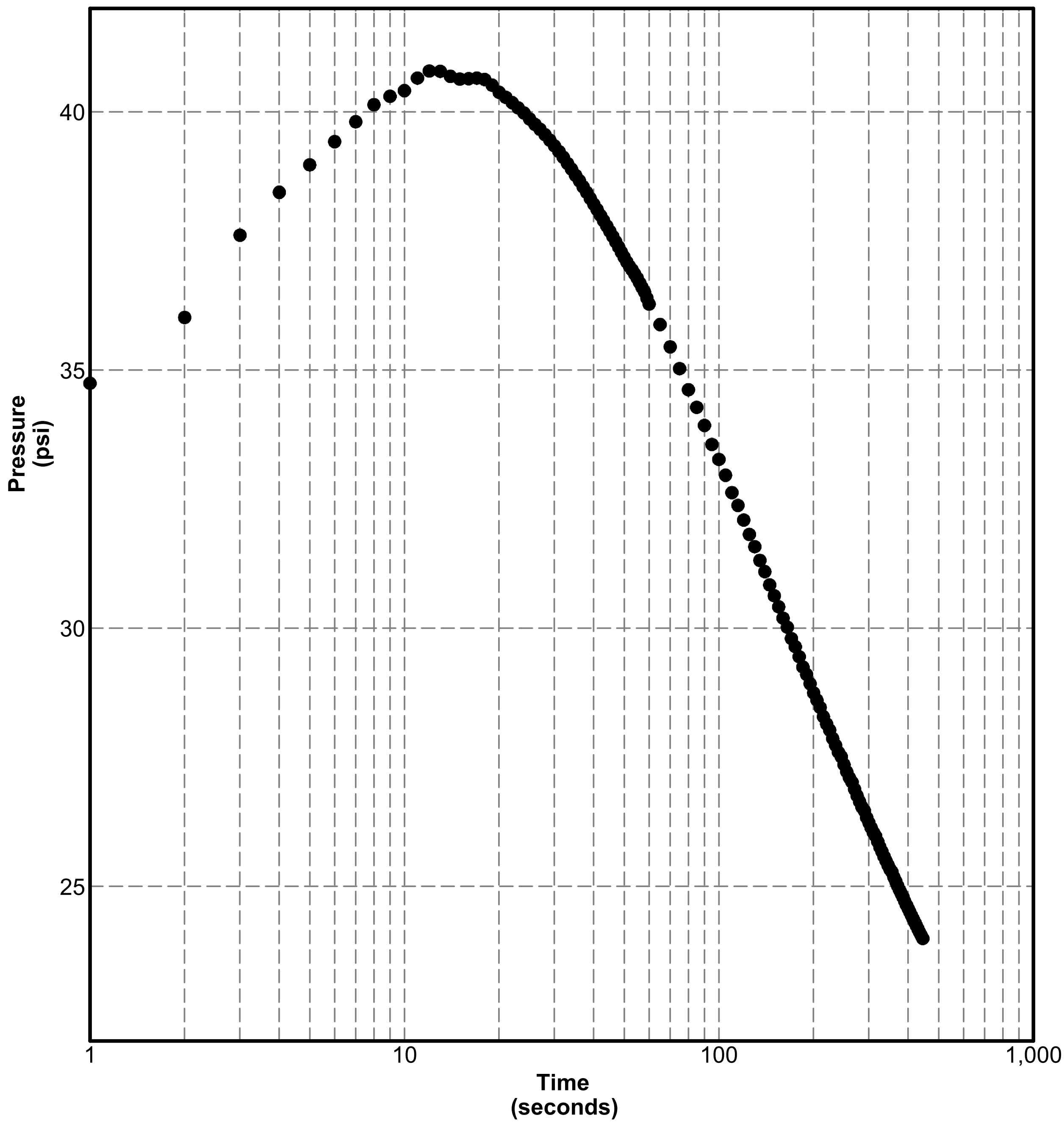
BRN-P2-20C

Electronic File Name: BRN-P2-20C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489365.3
Easting: 1314186.3
Elevation: 30.7
Total Depth: 66.9 ft
Termination Criteria:
Test Depth: 46.9 ft



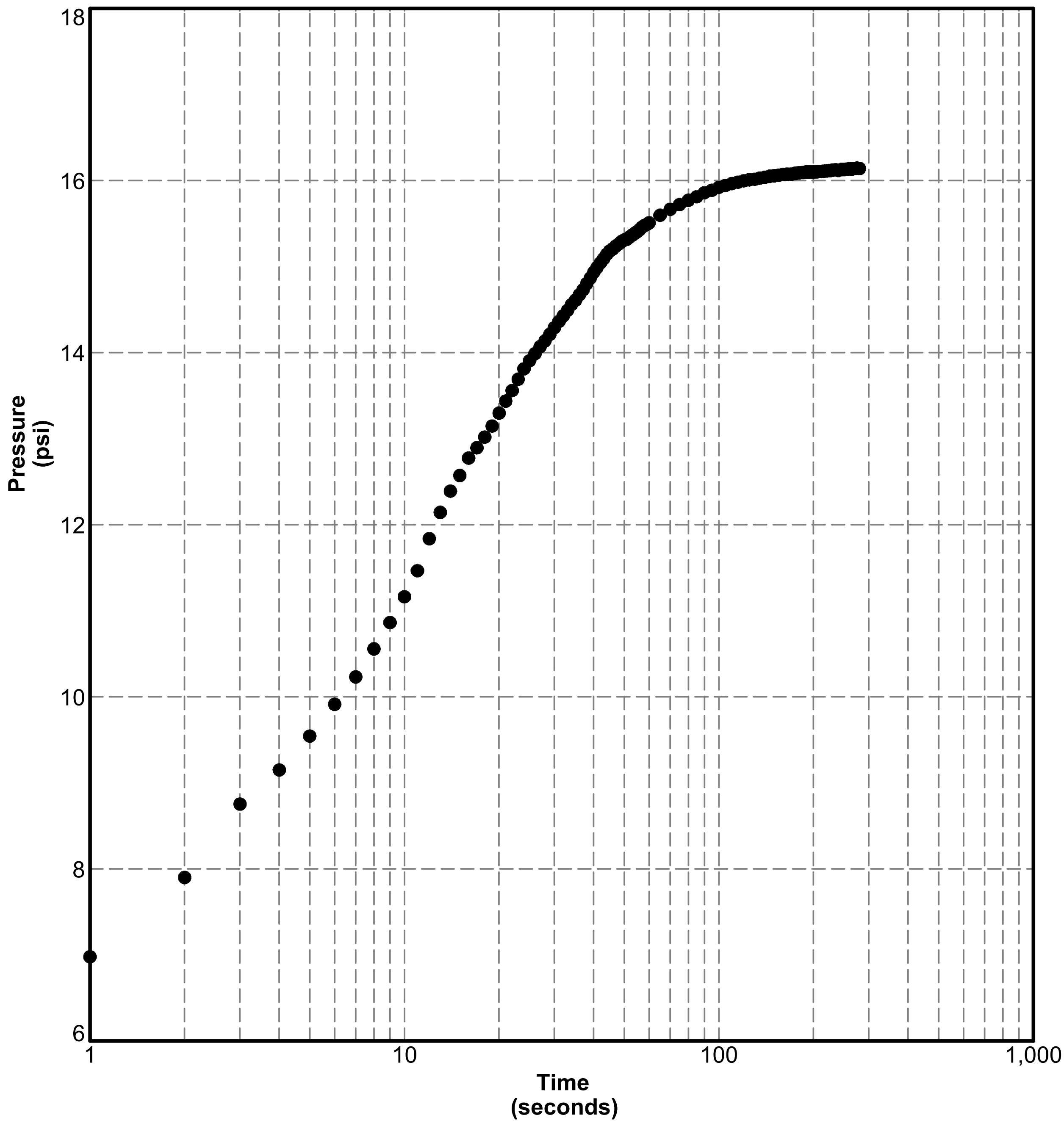
BRN-P2-20C

Electronic File Name: BRN-P2-20C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489365.3
Easting: 1314186.3
Elevation: 30.7
Total Depth: 66.9 ft
Termination Criteria:
Test Depth: 49.0 ft



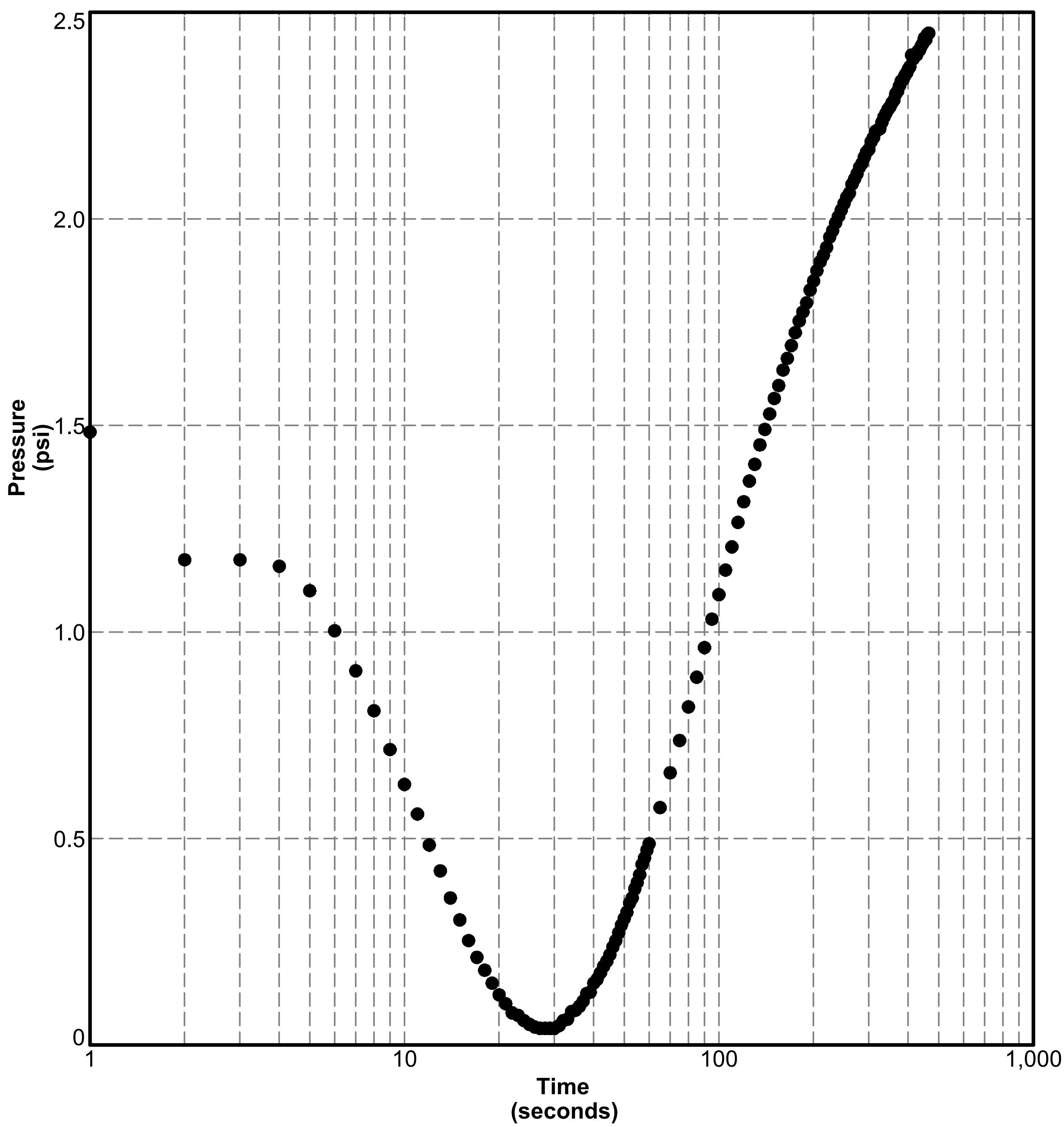
BRN-P2-20C

Electronic File Name: BRN-P2-20C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16488789.9
Easting: 1314344.0
Elevation: 36.3
Total Depth: 53.8 ft
Termination Criteria:
Test Depth: 27.7 ft



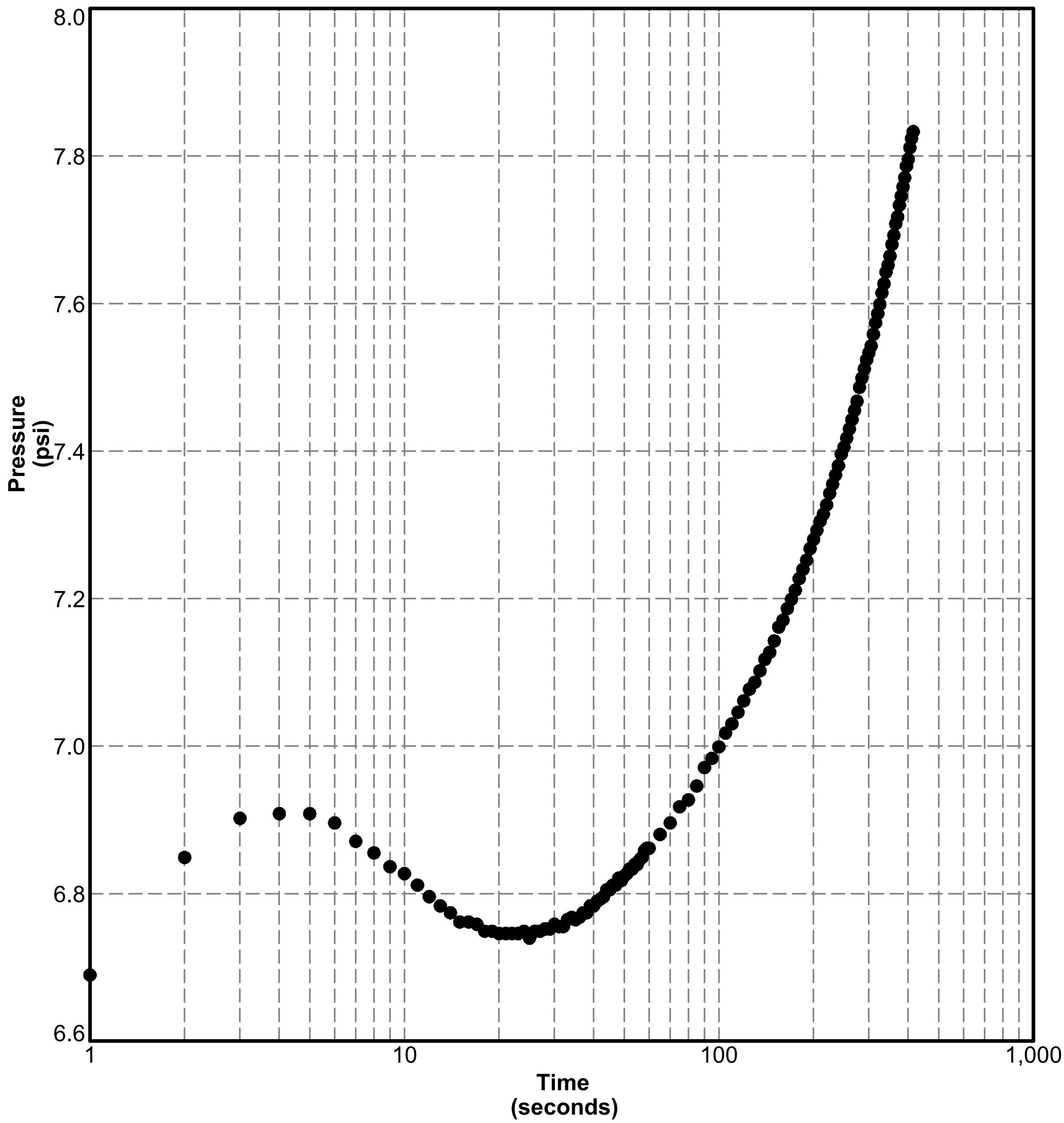
BRN-P2-23C

Electronic File Name: BRN-P2-23C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014 **Northing:** 16488789.9 **Total Depth:** 53.8 ft
Estimated Water Depth: 0 ft **Easting:** 1314344.0 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 36.3 **Test Depth:** 44.9 ft



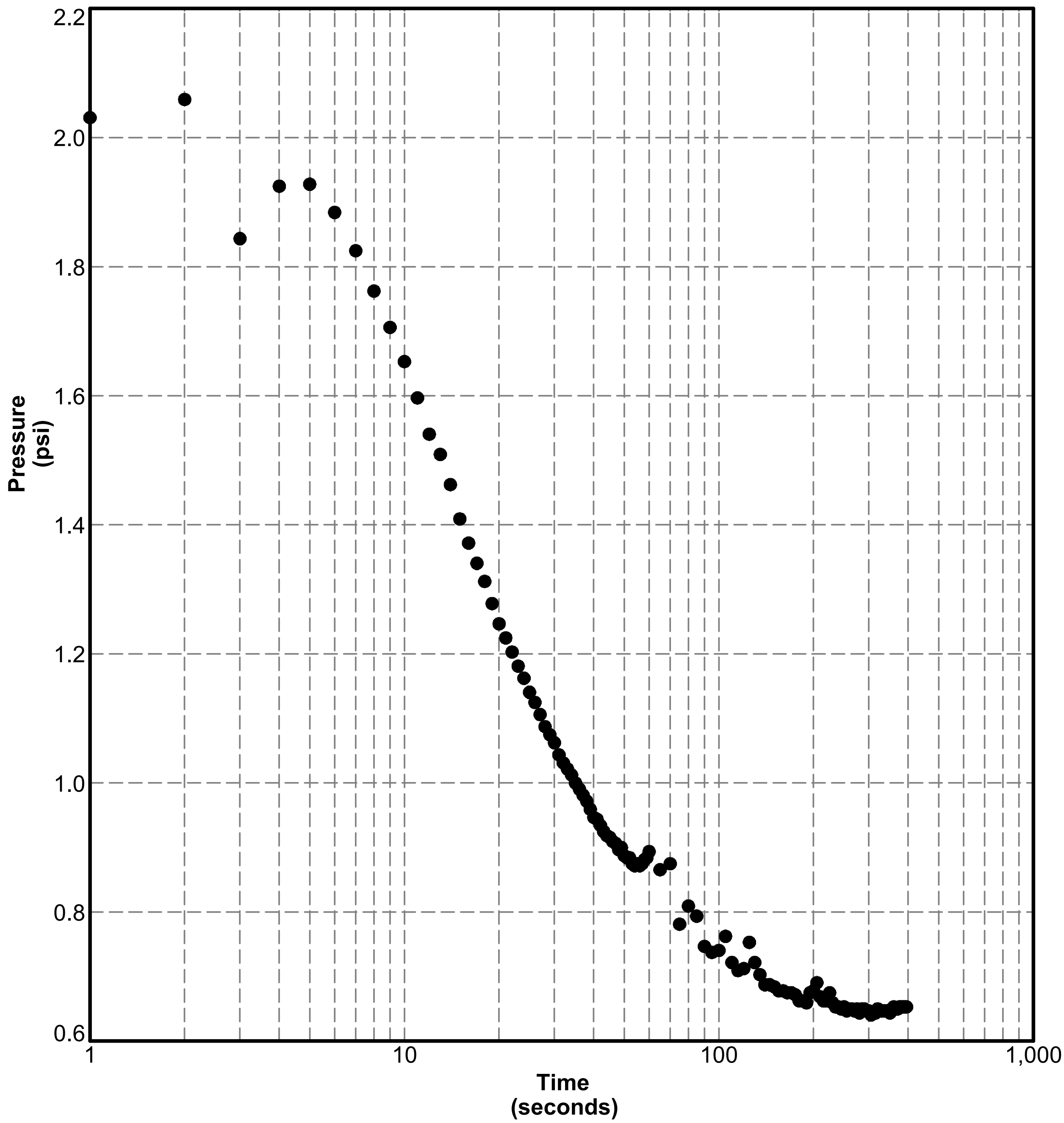
BRN-P2-23C

Electronic File Name: BRN-P2-23C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16488798.1
Easting: 1314388.9
Elevation: 41.8
Total Depth: 63.7 ft
Termination Criteria:
Test Depth: 22.5 ft



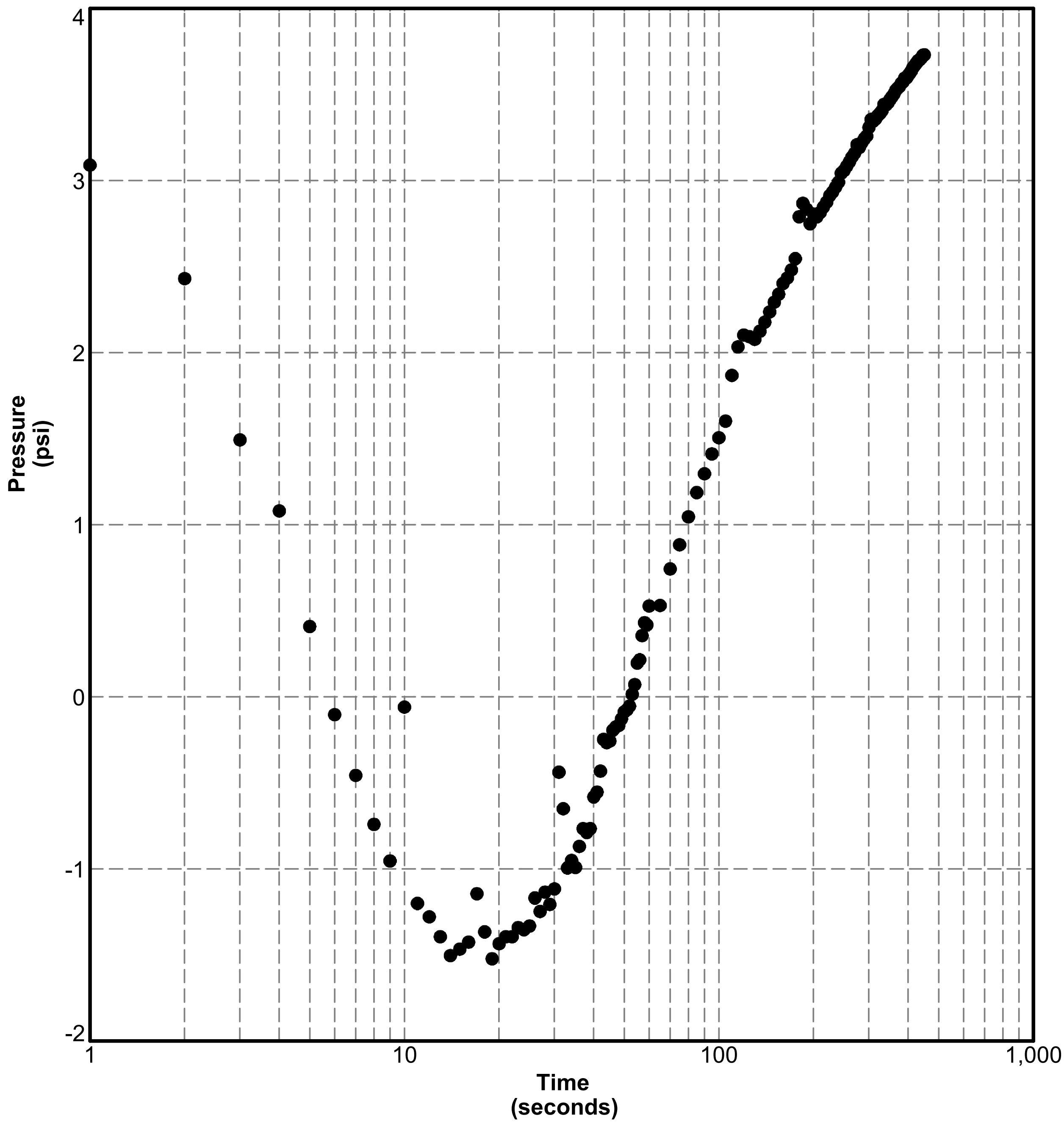
BRN-P2-24C

Electronic File Name: BRN-P2-24C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date:	Aug. 2, 2014	**Northing:**	16488798.1	**Total Depth:**	63.7 ft
Estimated Water Depth:	0 ft	**Easting:**	1314388.9	**Termination Criteria:**	
Rig/Operator:	Markov	**Elevation:**	41.8	**Test Depth:**	36.7 ft



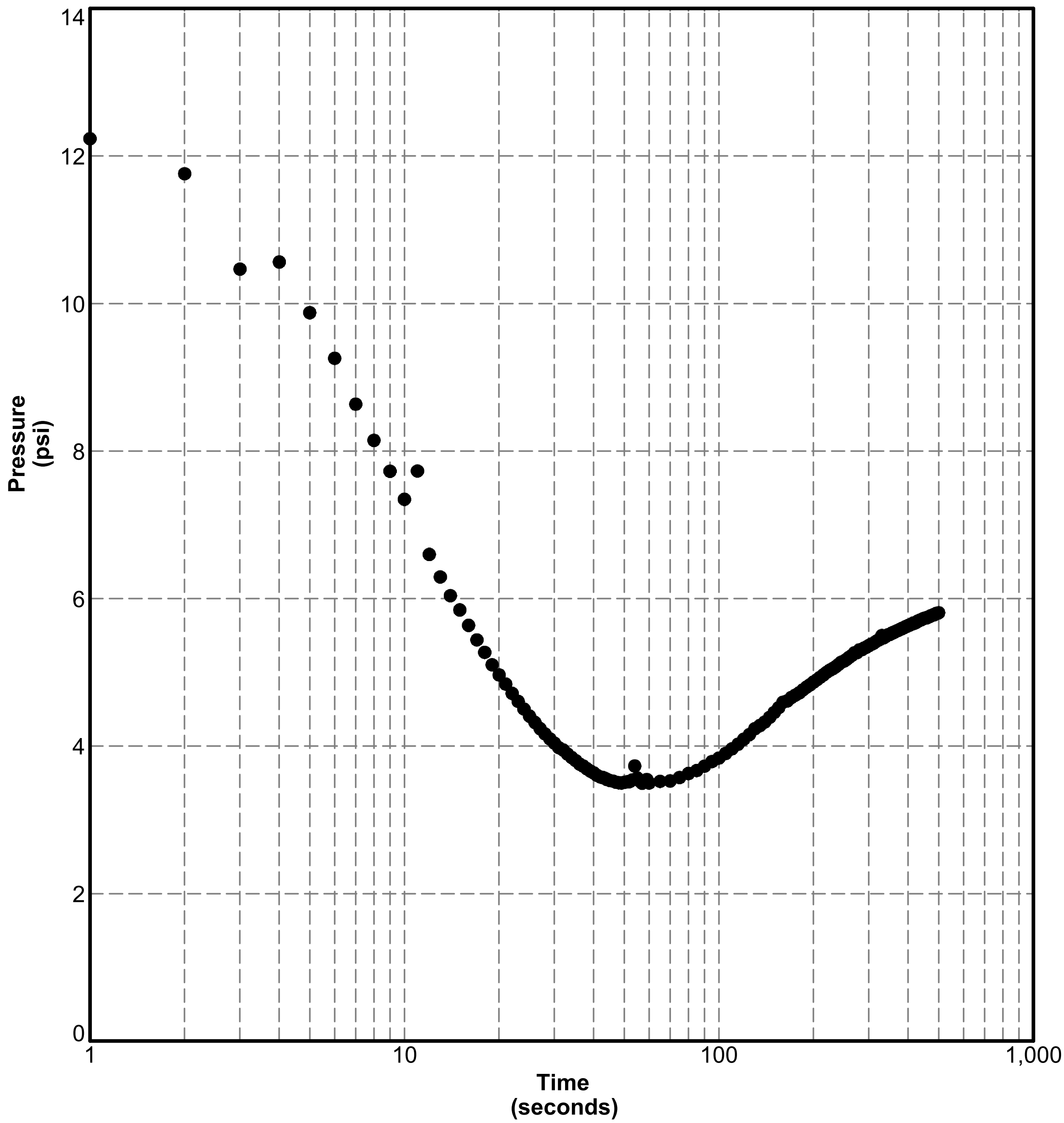
BRN-P2-24C

Electronic File Name: BRN-P2-24C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16488798.1
Easting: 1314388.9
Elevation: 41.8
Total Depth: 63.7 ft
Termination Criteria:
Test Depth: 40.8 ft



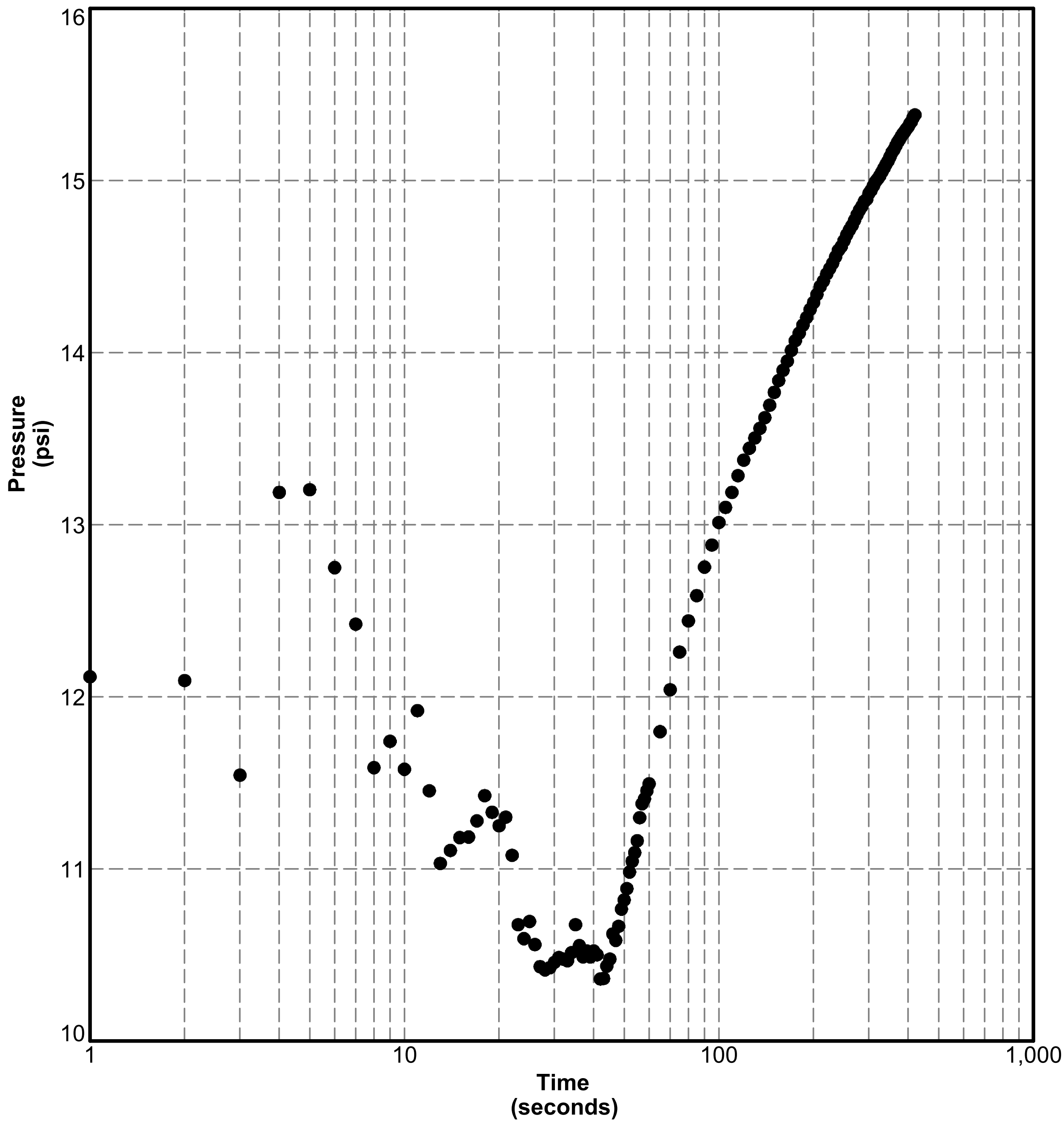
BRN-P2-24C

Electronic File Name: BRN-P2-24C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16488798.1
Easting: 1314388.9
Elevation: 41.8
Total Depth: 63.7 ft
Termination Criteria:
Test Depth: 60.4 ft



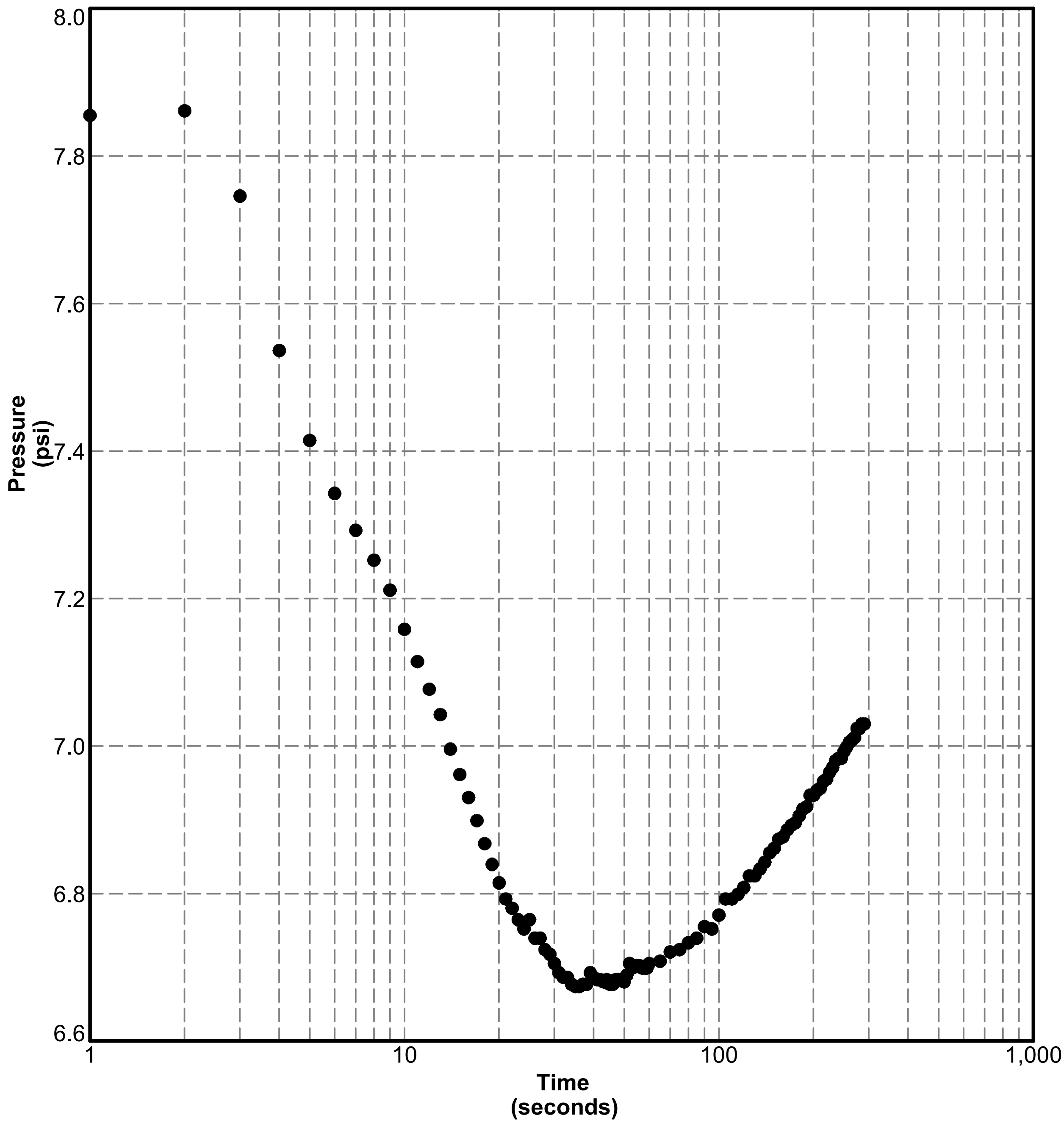
BRN-P2-24C

Electronic File Name: BRN-P2-24C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489409.2
Easting: 1314252.4
Elevation: 40.7
Total Depth: 62.3 ft
Termination Criteria:
Test Depth: 26.1 ft



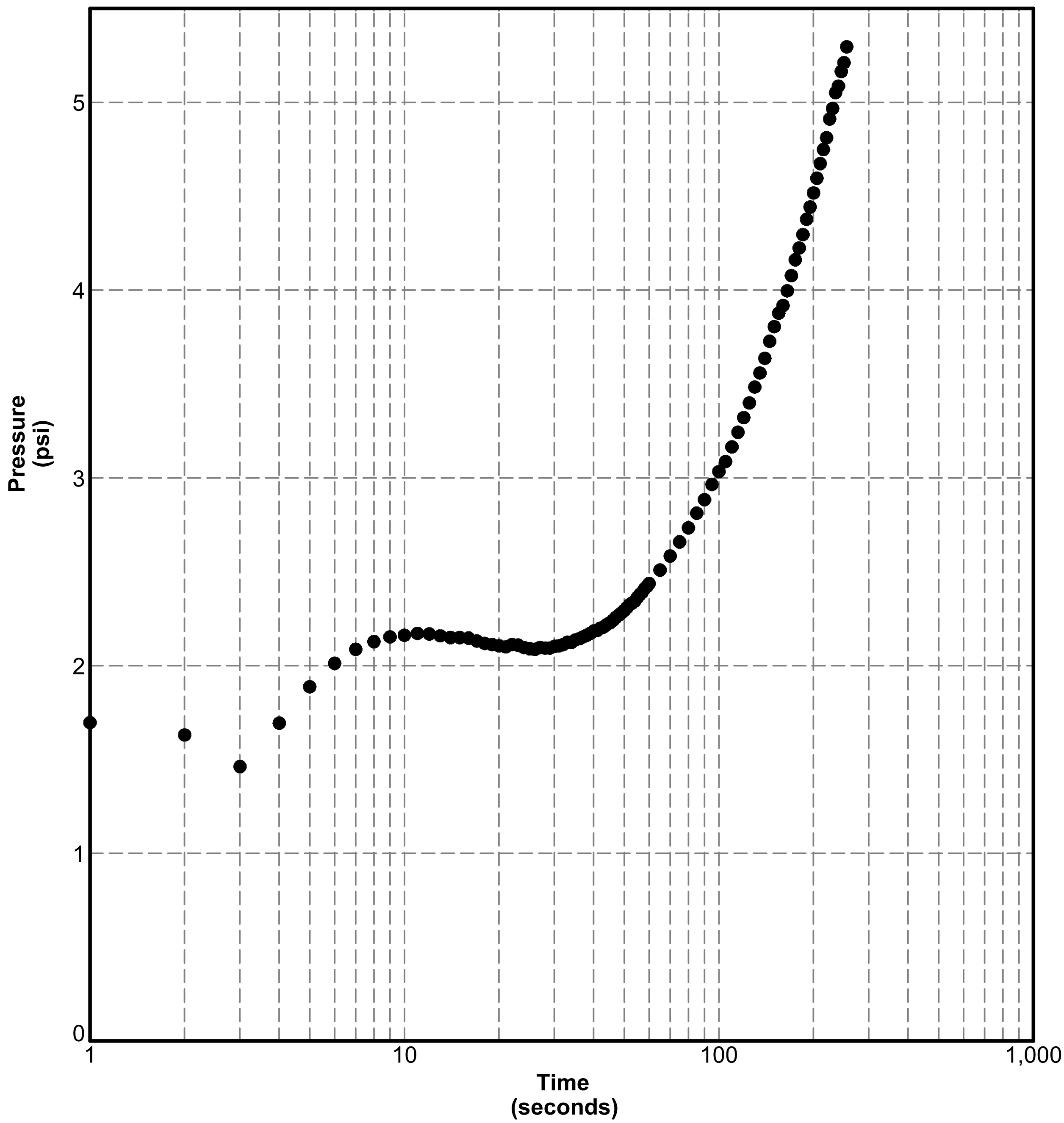
BRN-P2-25C

Electronic File Name: BRN-P2-25C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014 **Northing:** 16489409.2 **Total Depth:** 62.3 ft
Estimated Water Depth: 0 ft **Easting:** 1314252.4 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 40.7 **Test Depth:** 35.4 ft



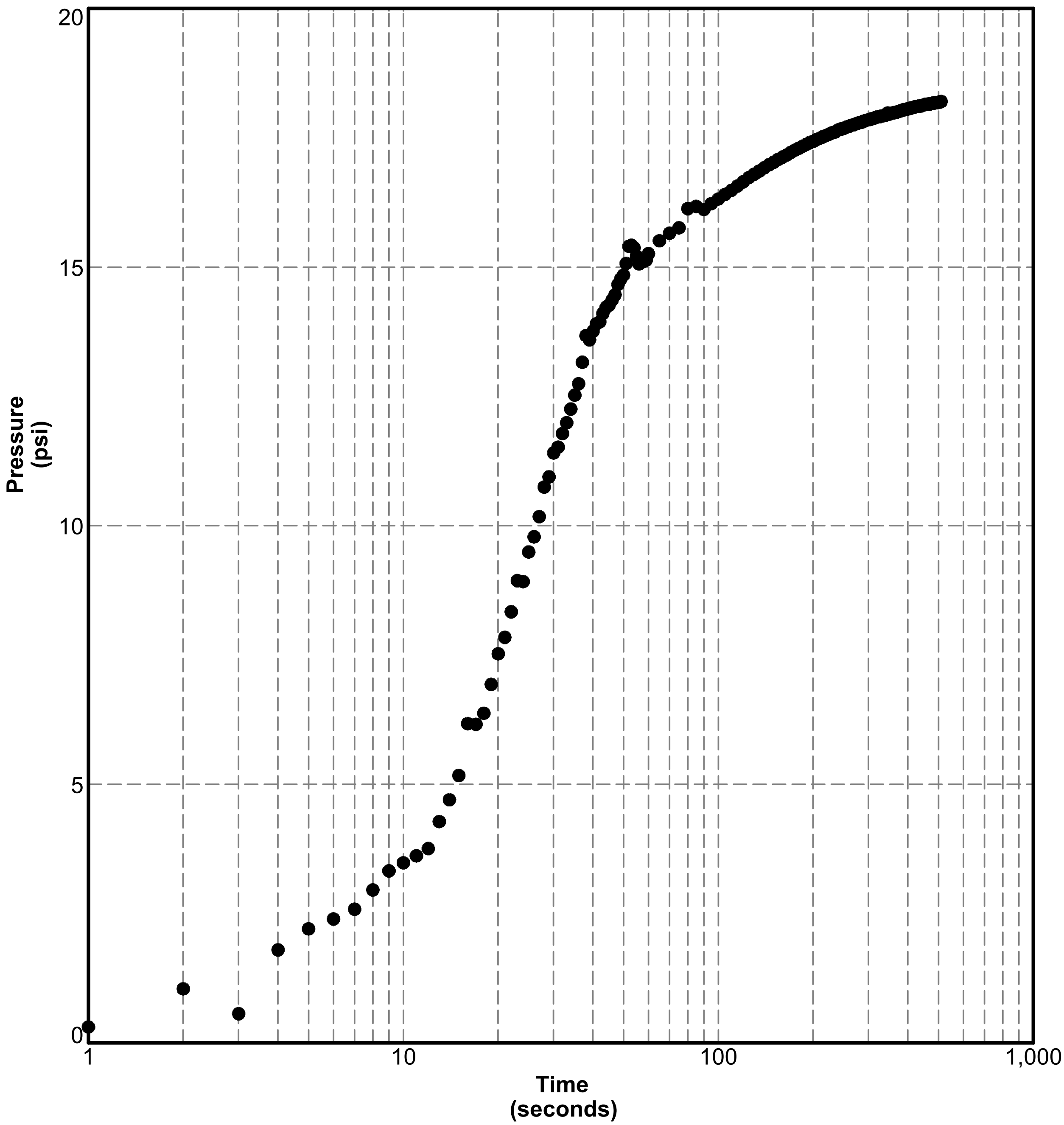
BRN-P2-25C

Electronic File Name: BRN-P2-25C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 2, 2014 **Northing:** 16489409.2 **Total Depth:** 62.3 ft
Estimated Water Depth: 0 ft **Easting:** 1314252.4 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 40.7 **Test Depth:** 62.3 ft



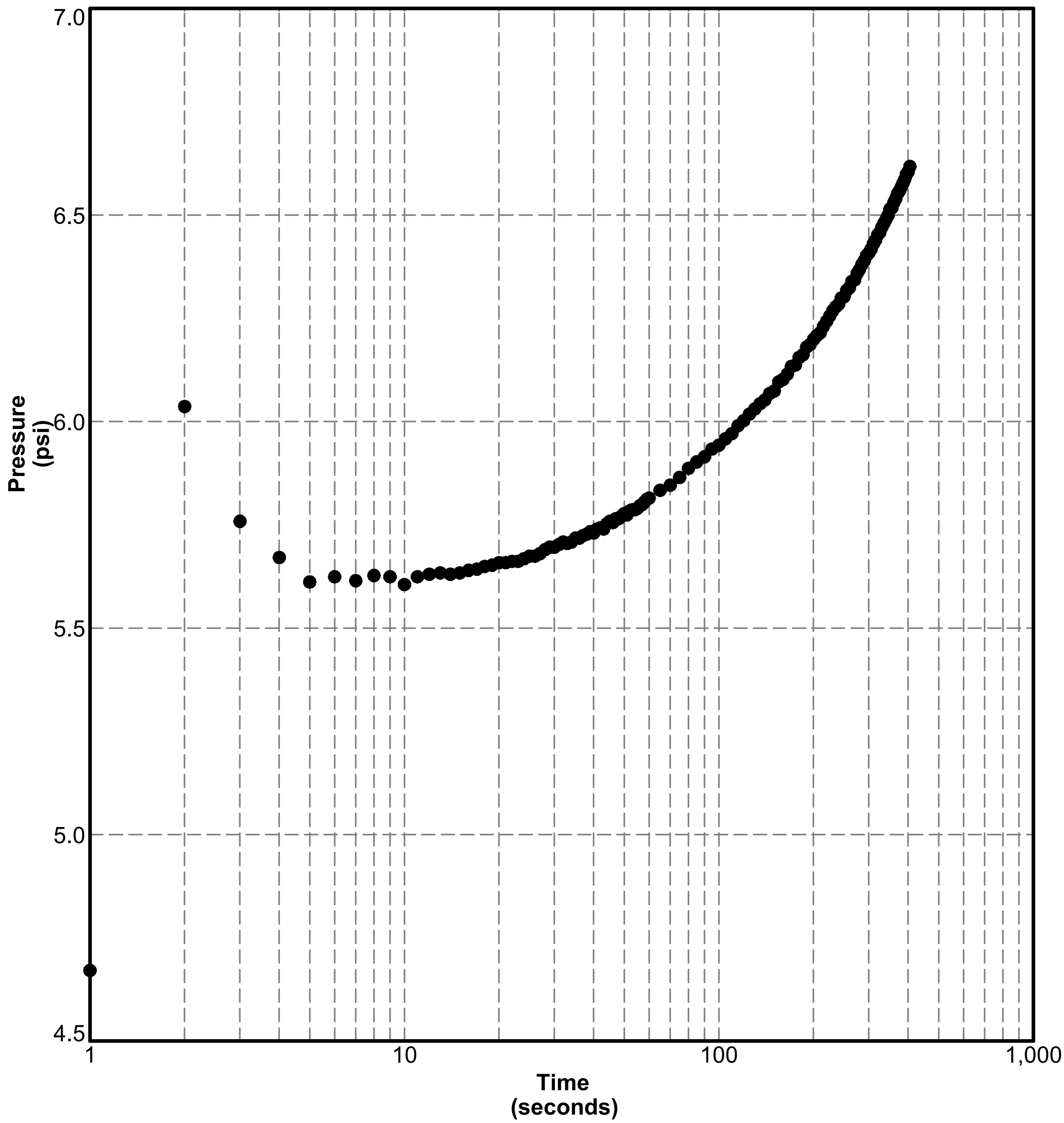
BRN-P2-25C

Electronic File Name: BRN-P2-25C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489591.3
Easting: 1314179.4
Elevation: 40.4
Total Depth: 69.4 ft
Termination Criteria:
Test Depth: 32.3 ft



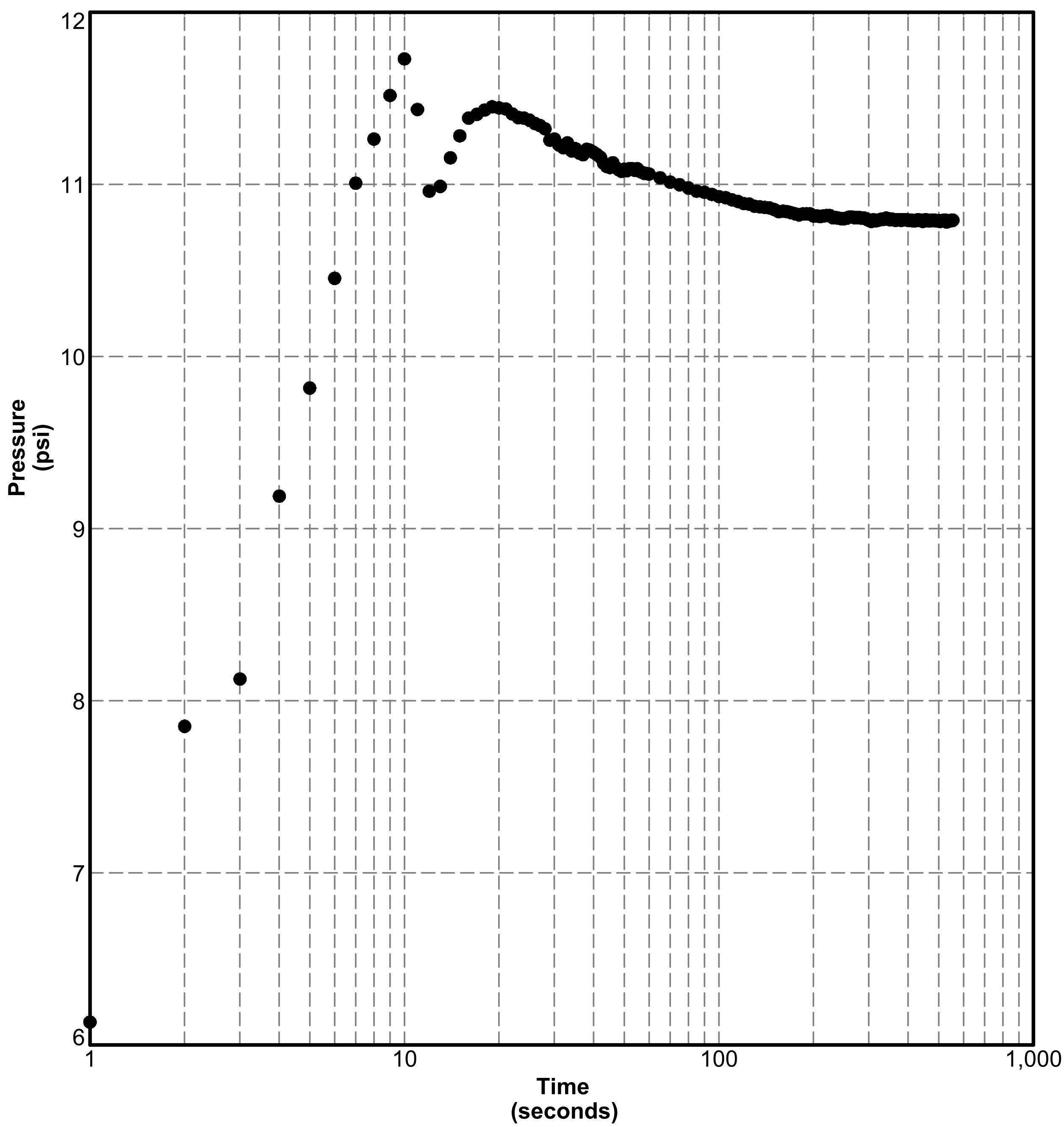
BRN-P2-26C

Electronic File Name: BRN-P2-26C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489591.3
Easting: 1314179.4
Elevation: 40.4
Total Depth: 69.4 ft
Termination Criteria:
Test Depth: 35.6 ft



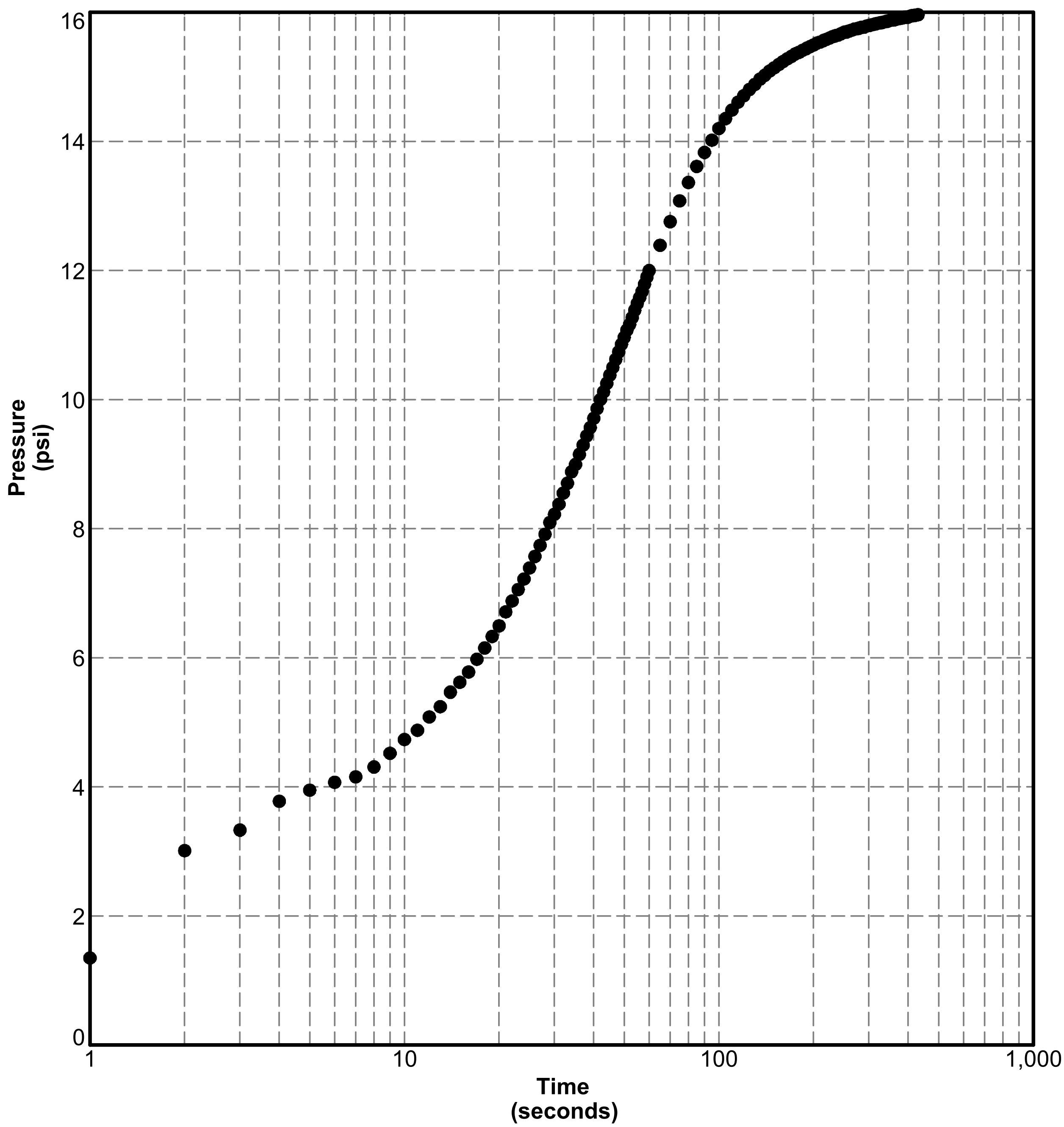
BRN-P2-26C

Electronic File Name: BRN-P2-26C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014 **Northing:** 16489591.3 **Total Depth:** 69.4 ft
Estimated Water Depth: 0 ft **Easting:** 1314179.4 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 40.4 **Test Depth:** 57.1 ft



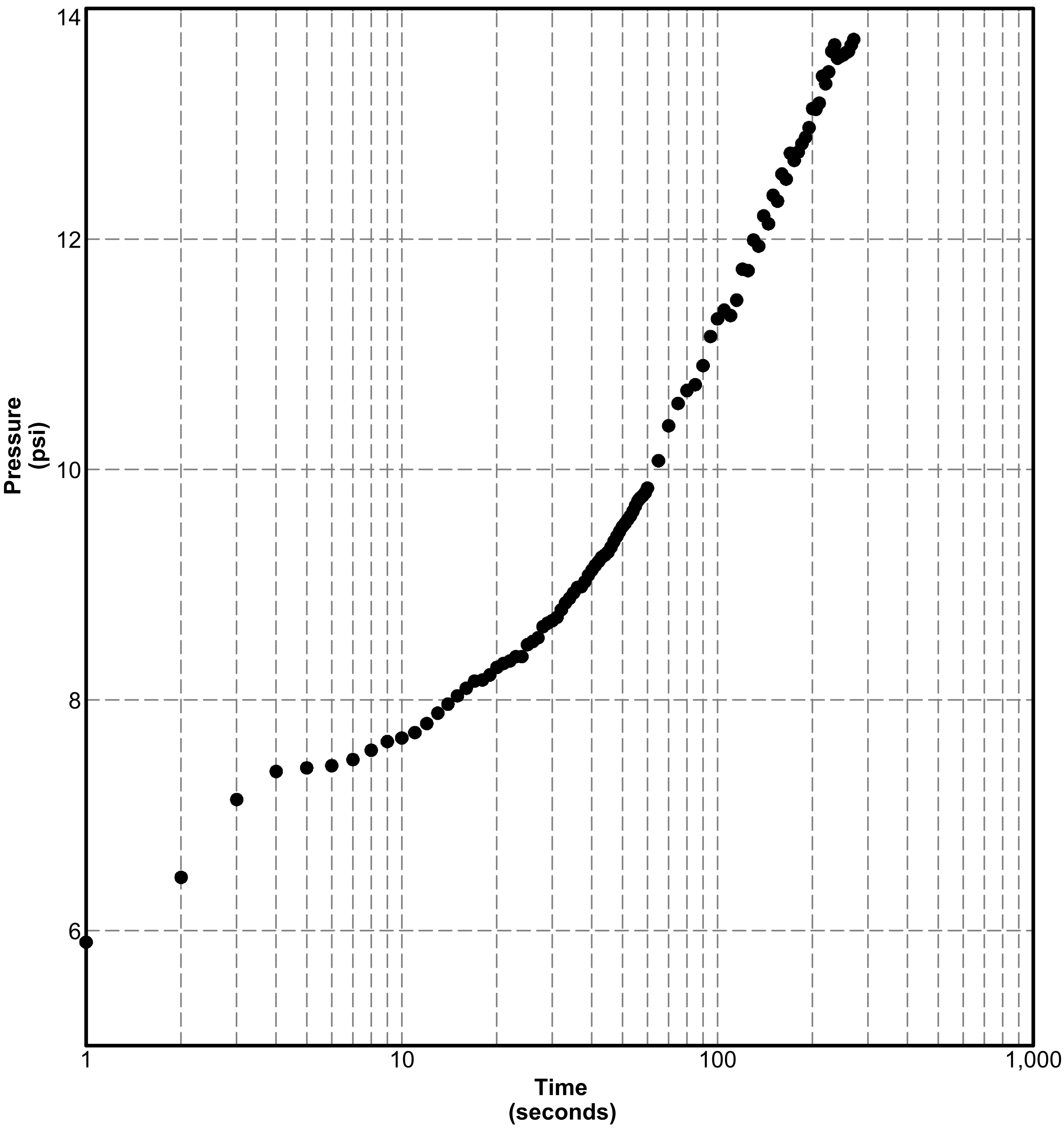
BRN-P2-26C

Electronic File Name: BRN-P2-26C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489584.4
Easting: 1314165.2
Elevation: 40.6
Total Depth: 73.5 ft
Termination Criteria:
Test Depth: 26.1 ft



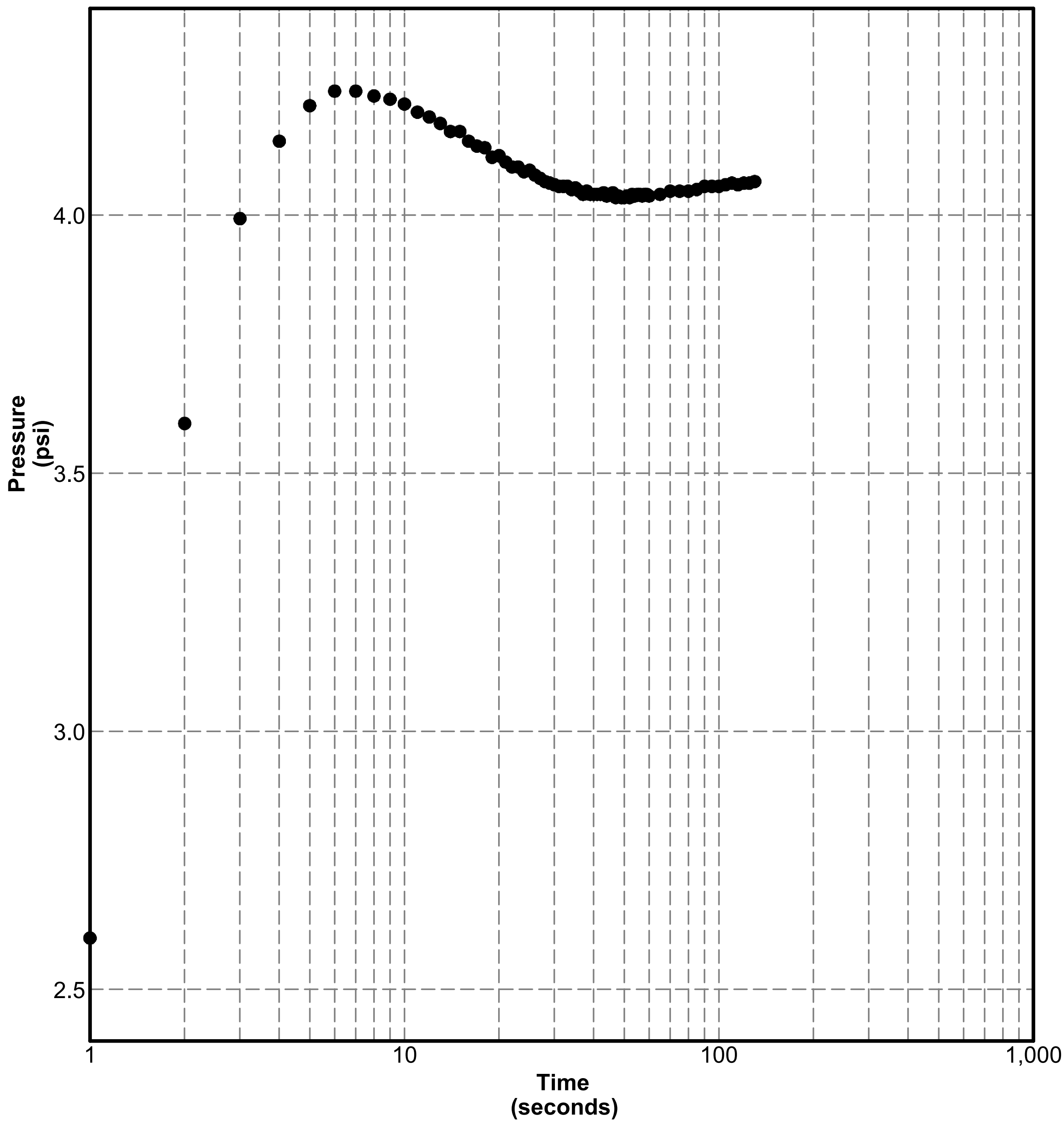
BRN-P2-27C

Electronic File Name: BRN-P2-27C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489584.4
Easting: 1314165.2
Elevation: 40.6
Total Depth: 73.5 ft
Termination Criteria:
Test Depth: 28.4 ft



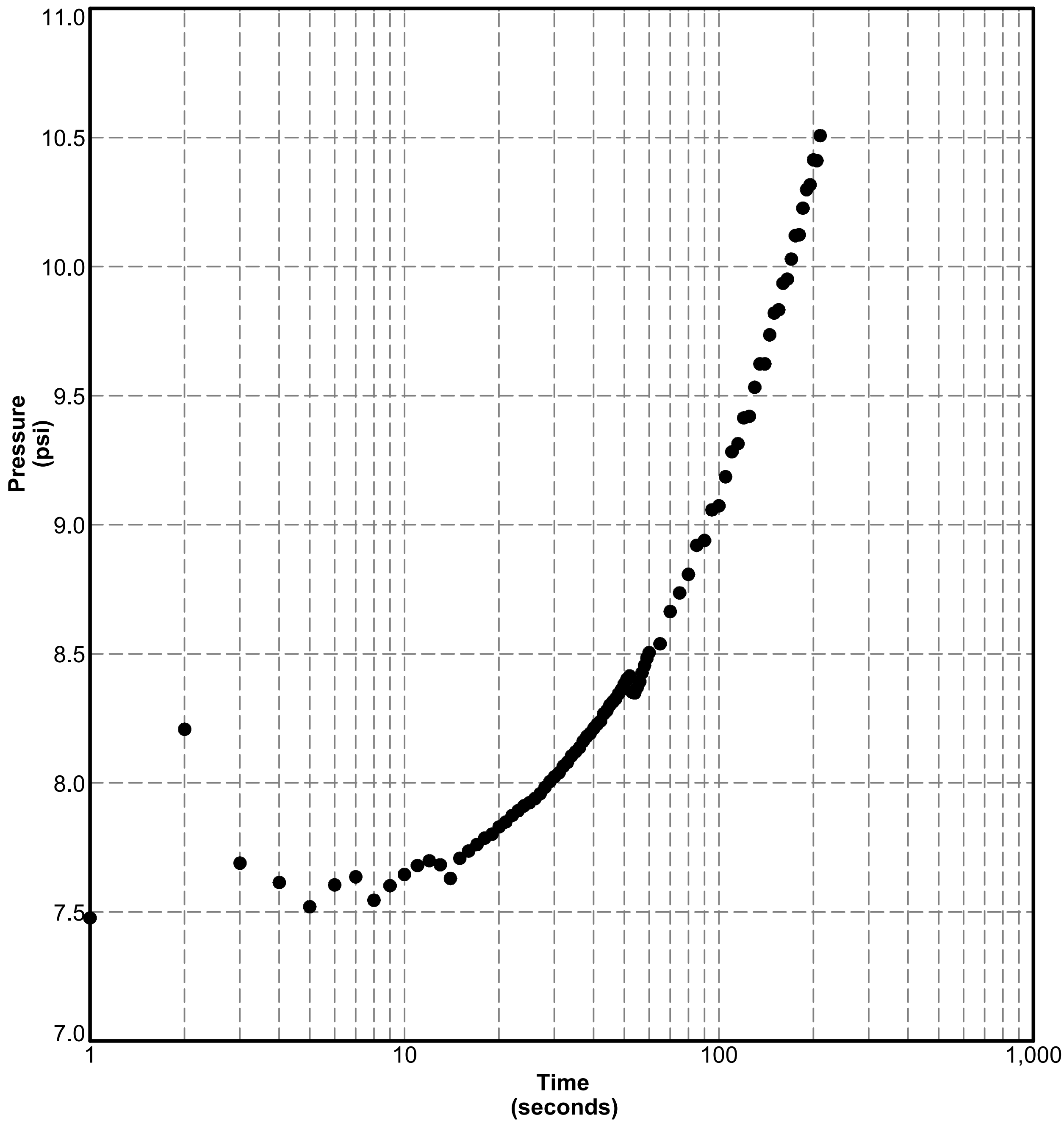
BRN-P2-27C

Electronic File Name: BRN-P2-27C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014 **Northing:** 16489584.4 **Total Depth:** 73.5 ft
Estimated Water Depth: 0 ft **Easting:** 1314165.2 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 40.6 **Test Depth:** 51.0 ft



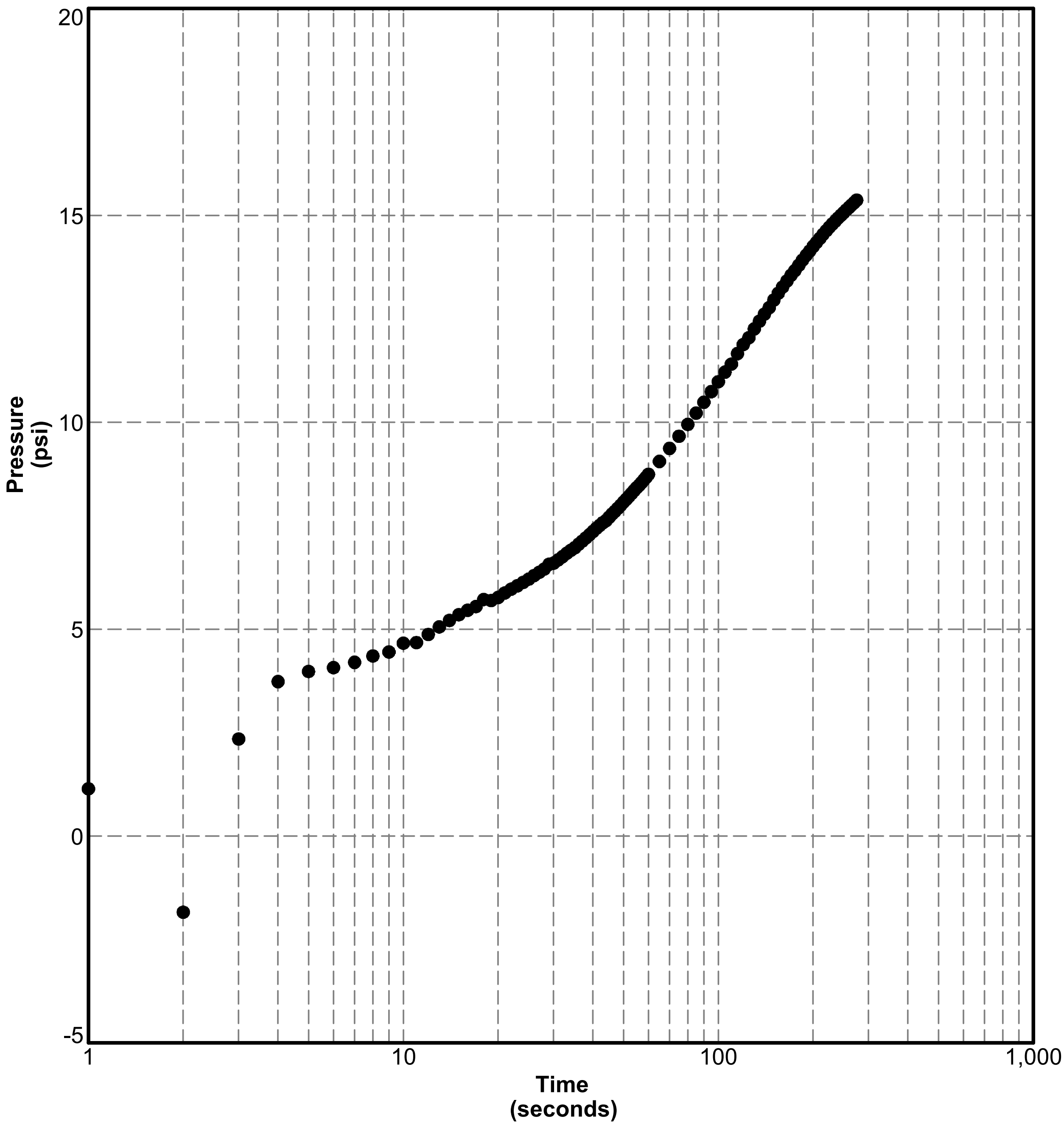
BRN-P2-27C

Electronic File Name: BRN-P2-27C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489584.4
Easting: 1314165.2
Elevation: 40.6
Total Depth: 73.5 ft
Termination Criteria:
Test Depth: 59.7 ft



BRN-P2-27C

Electronic File Name: BRN-P2-27C.dis

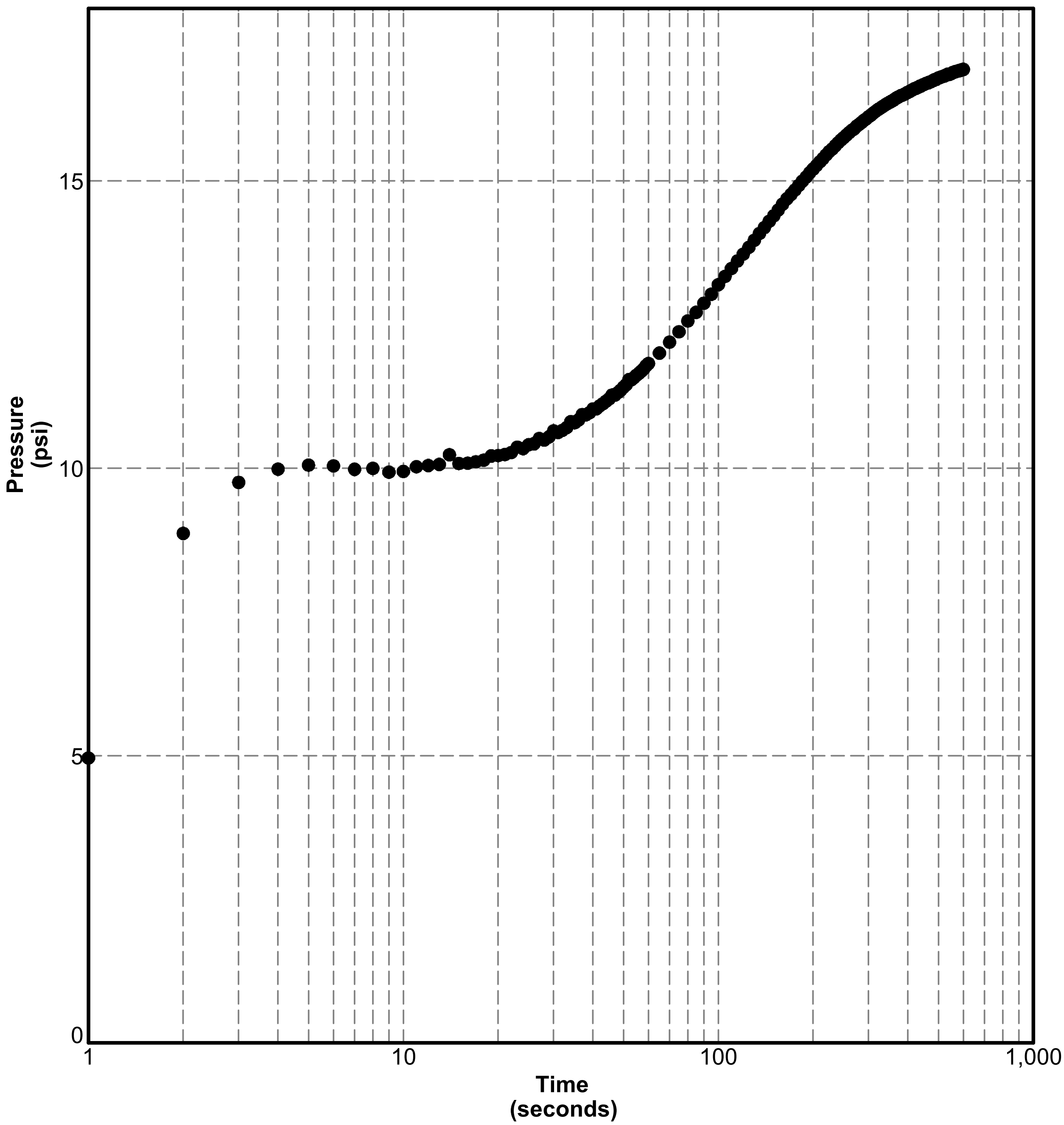
Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Aug. 1, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov

Northing: 16489584.4
Easting: 1314165.2
Elevation: 40.6

Total Depth: 73.5 ft
Termination Criteria:
Test Depth: 60.4 ft



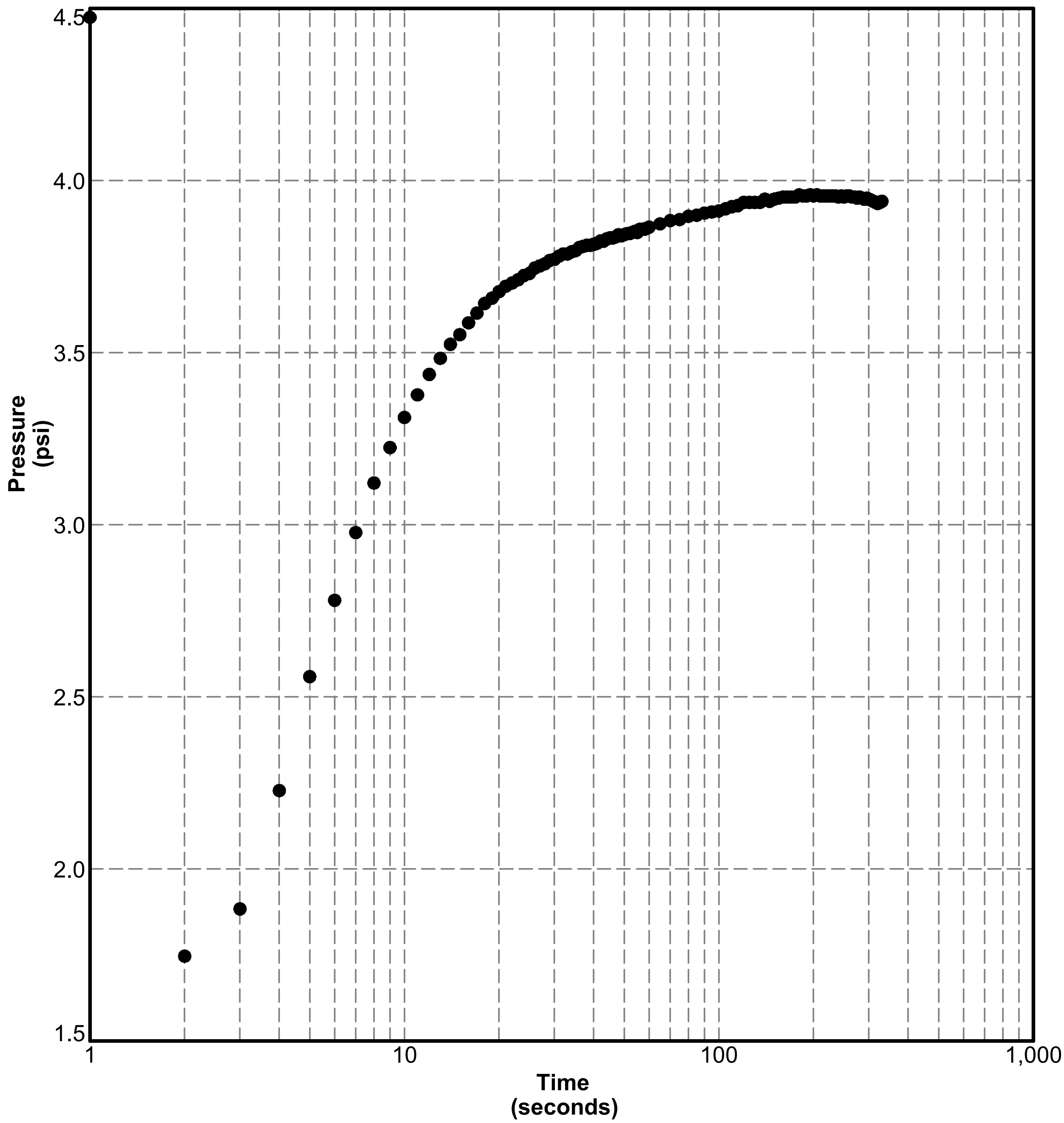
BRN-P2-27C

Electronic File Name: BRN-P2-27C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014 **Northing:** 16489571.1 **Total Depth:** 50.9 ft
Estimated Water Depth: 0 ft **Easting:** 1314138.7 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 31.5 **Test Depth:** 16.7 ft



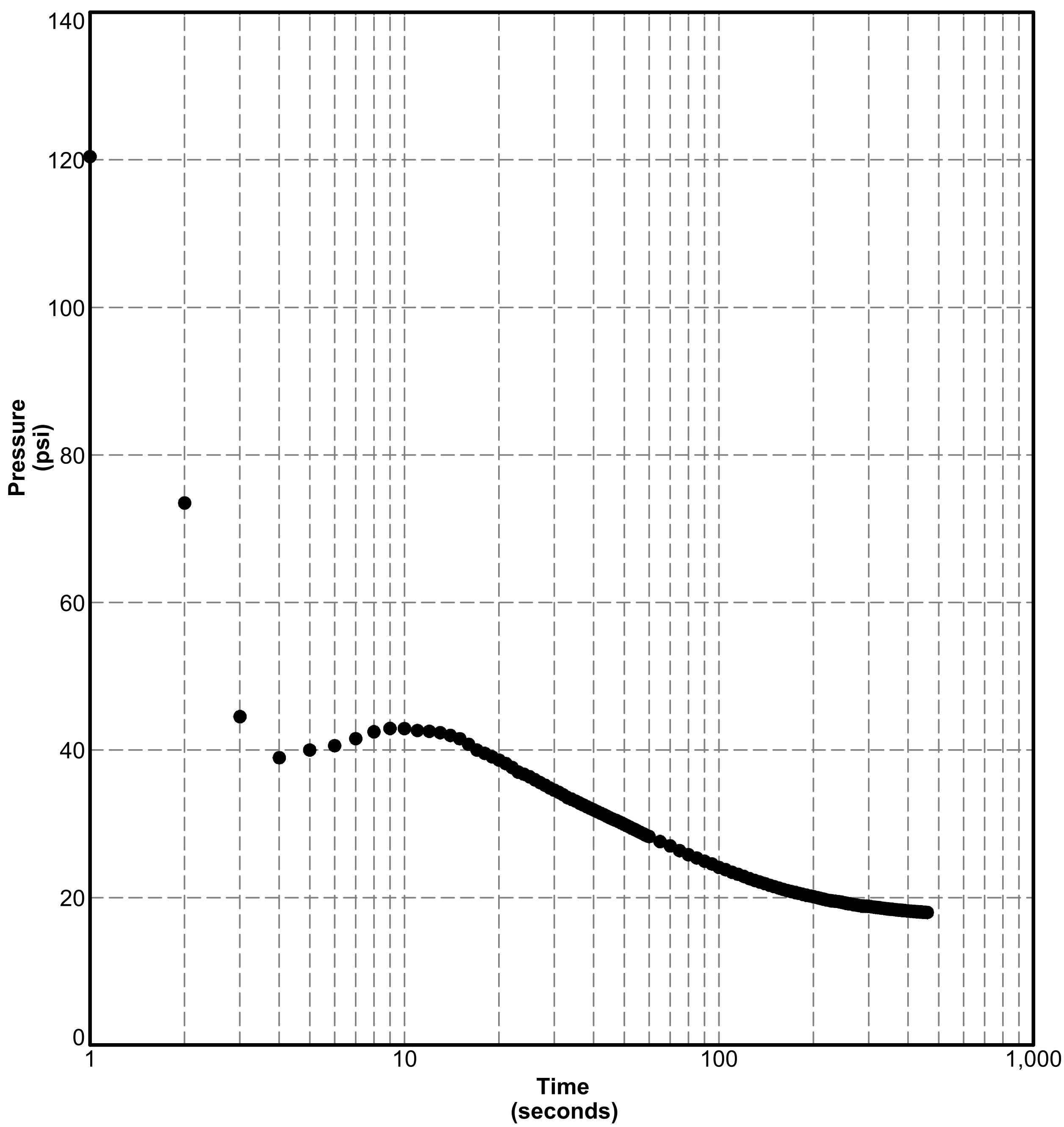
BRN-P2-28C

Electronic File Name: BRN-P2-28C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date:	Jul. 31, 2014	**Northing:**	16489571.1	**Total Depth:**	50.9 ft
Estimated Water Depth:	0 ft	**Easting:**	1314138.7	**Termination Criteria:**	
Rig/Operator:	Markov	**Elevation:**	31.5	**Test Depth:**	48.4 ft



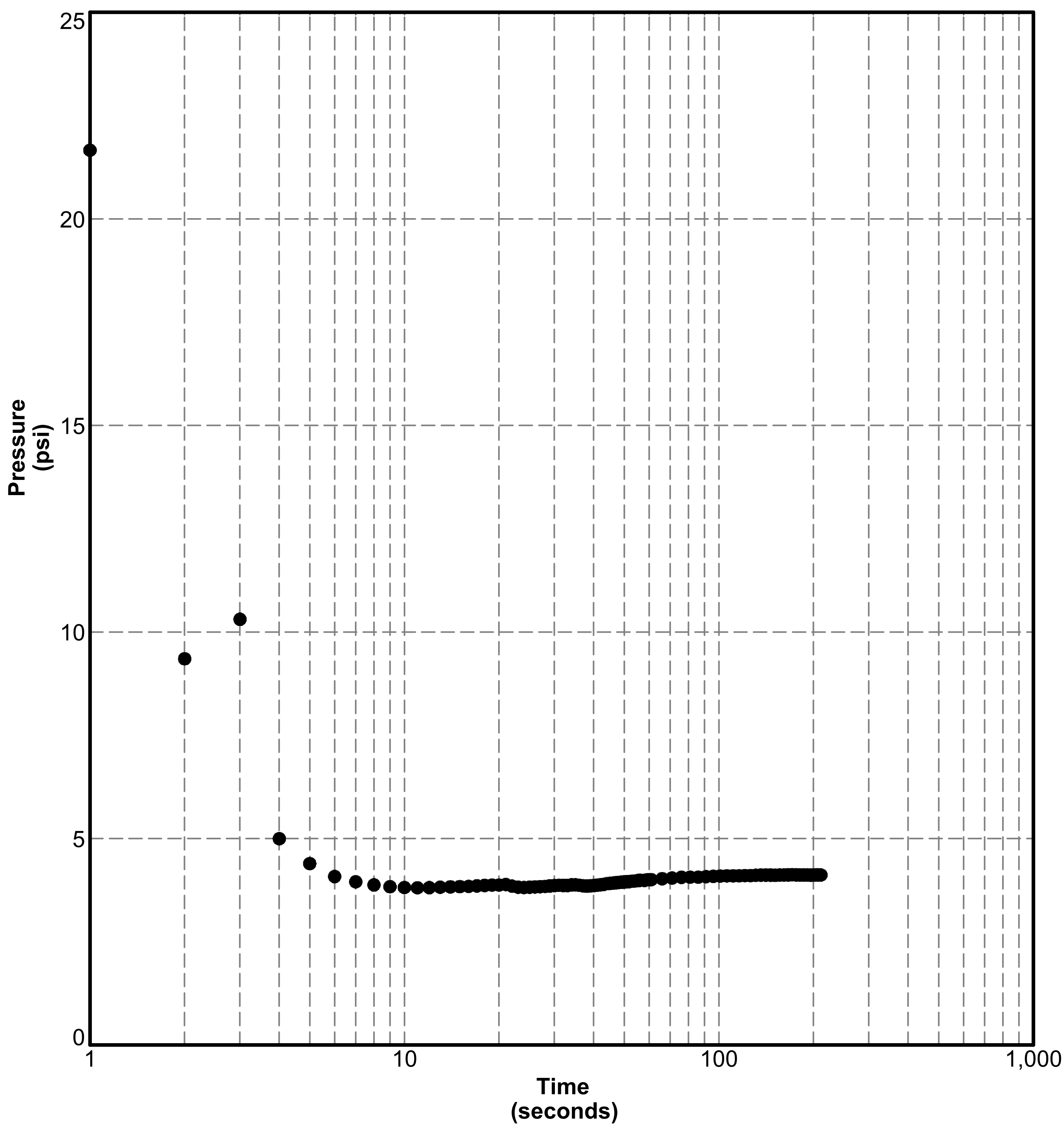
BRN-P2-28C

Electronic File Name: BRN-P2-28C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489548.2
Easting: 1314088.2
Elevation: 30.3
Total Depth: 50.7 ft
Termination Criteria:
Test Depth: 23.5 ft



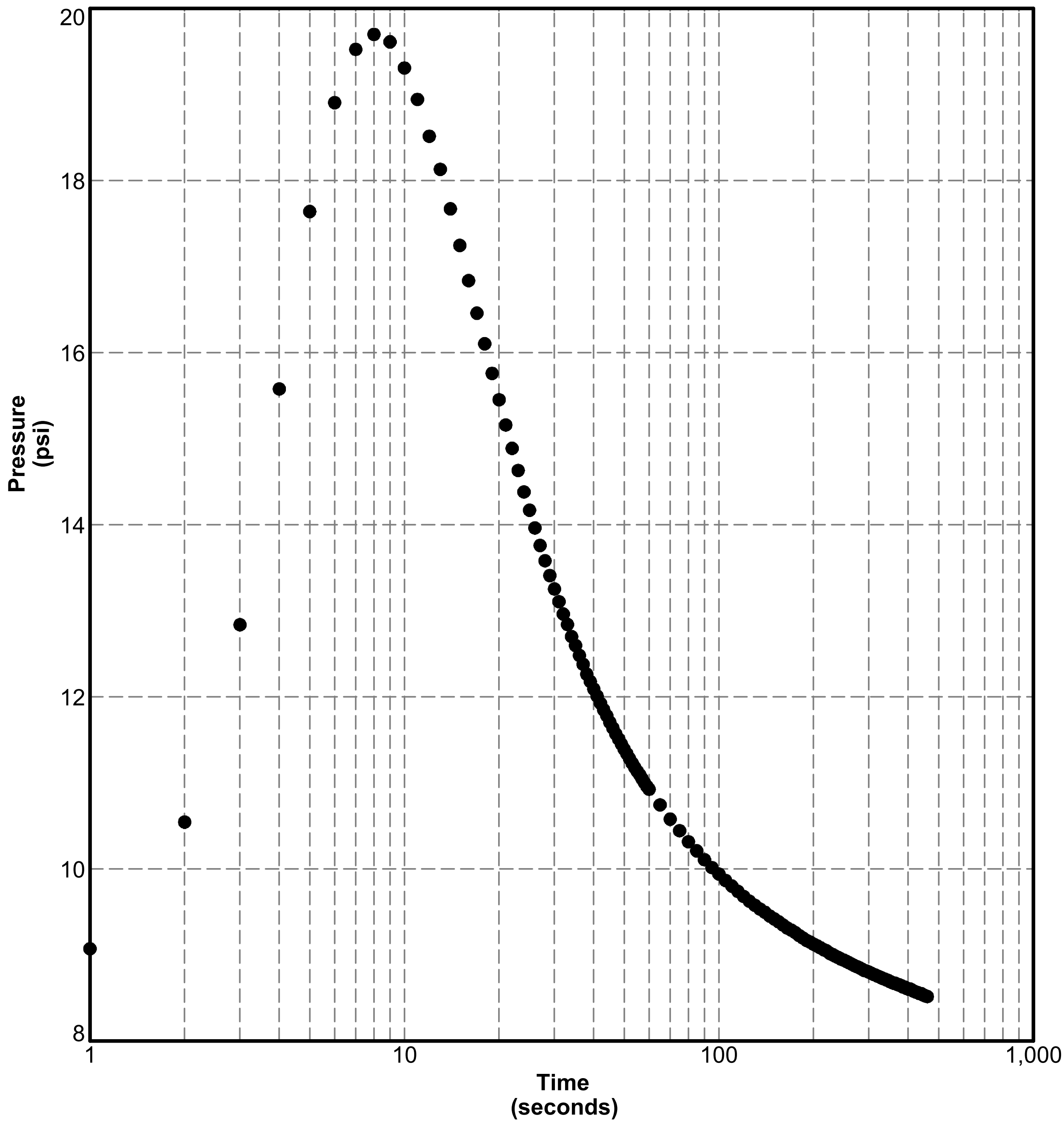
BRN-P2-29C

Electronic File Name: BRN-P2-29C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489548.2
Easting: 1314088.2
Elevation: 30.3
Total Depth: 50.7 ft
Termination Criteria:
Test Depth: 32.2 ft



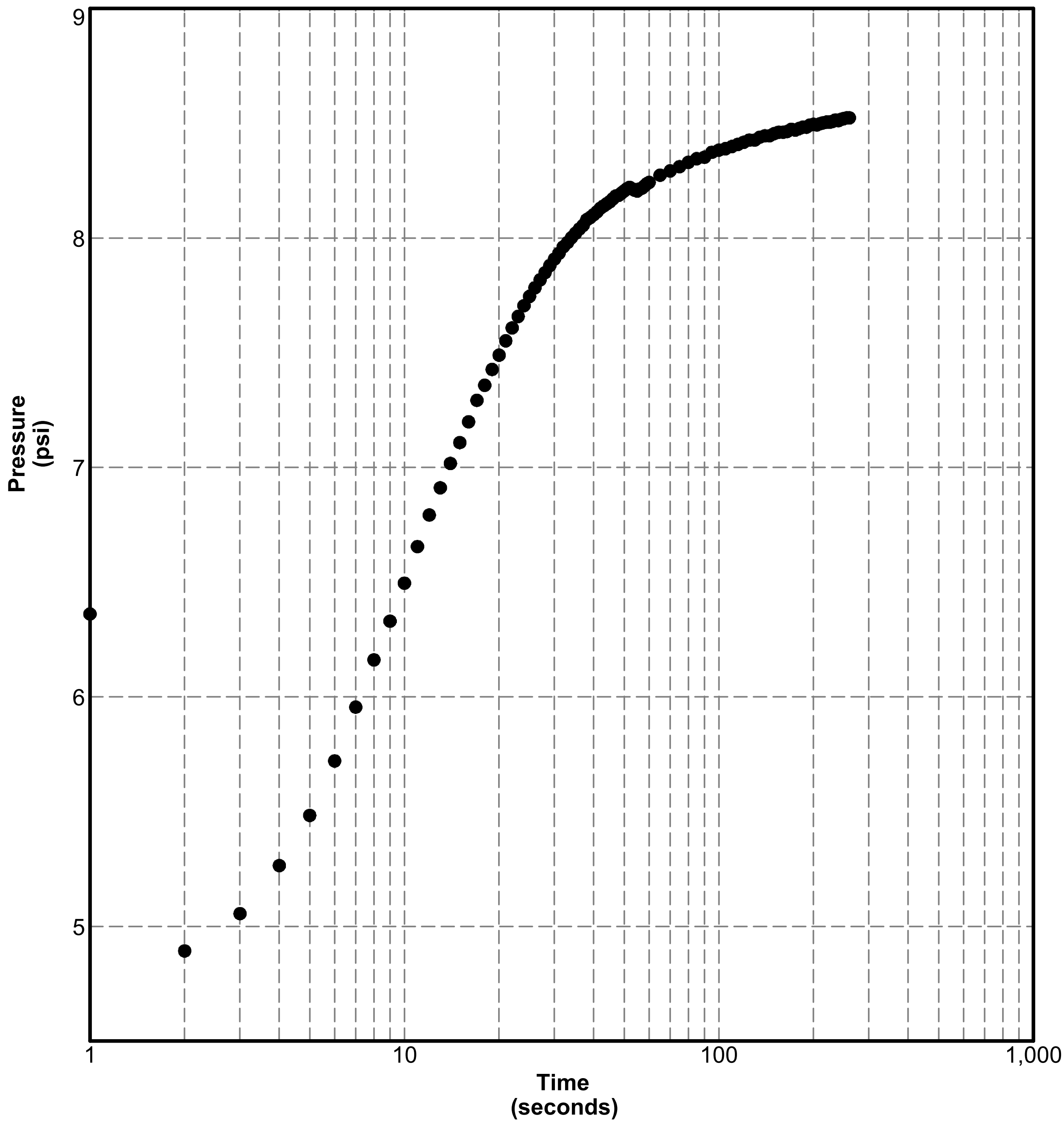
BRN-P2-29C

Electronic File Name: BRN-P2-29C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014 **Northing:** 16489548.2 **Total Depth:** 50.7 ft
Estimated Water Depth: 0 ft **Easting:** 1314088.2 **Termination Criteria:**
Rig/Operator: Markov **Elevation:** 30.3 **Test Depth:** 36.9 ft



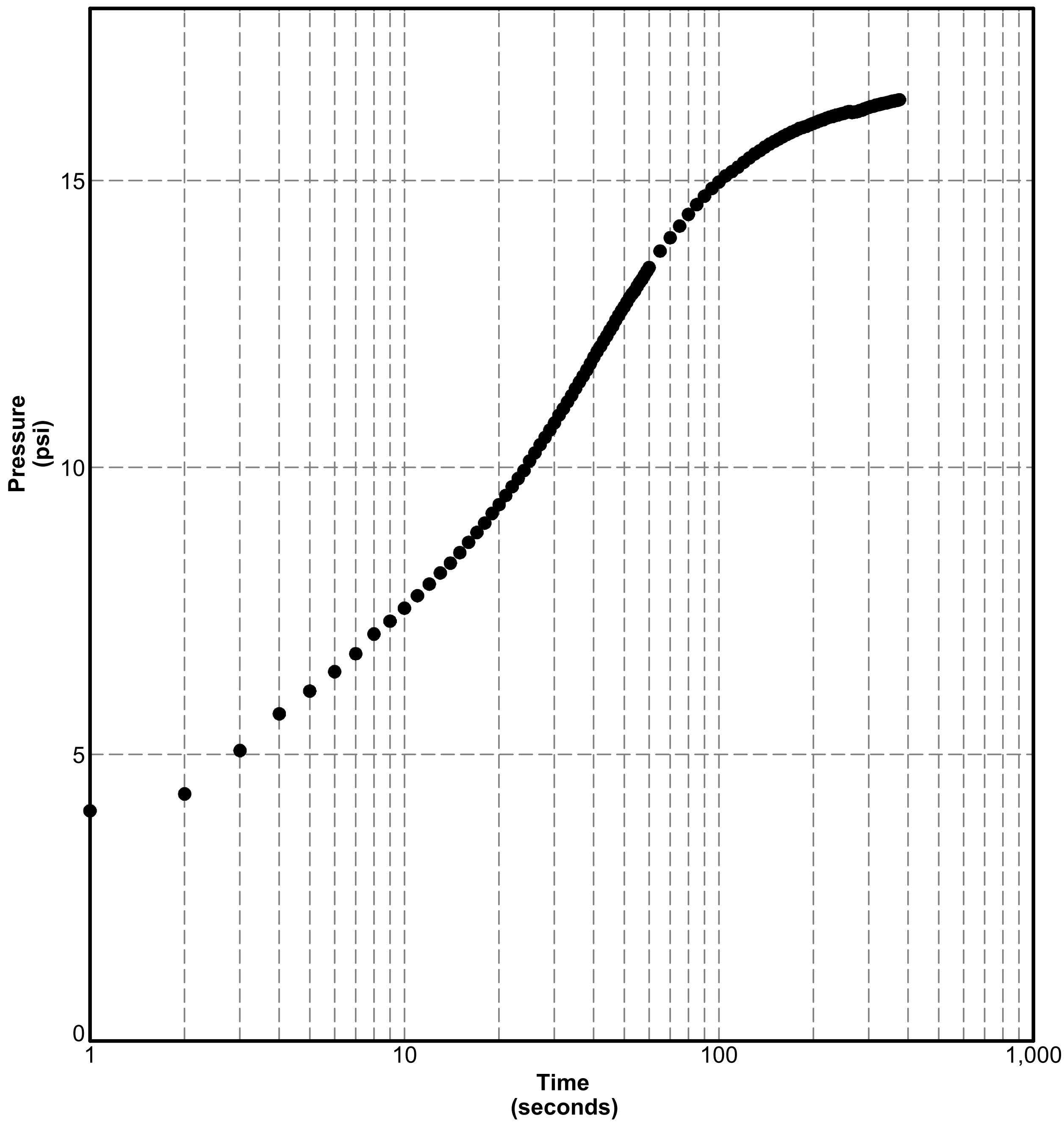
BRN-P2-29C

Electronic File Name: BRN-P2-29C.dis

Pore Pressure Dissipation

Brownsville, Tx
Project Number :IBWC

Date: Jul. 31, 2014
Estimated Water Depth: 0 ft
Rig/Operator: Markov
Northing: 16489548.2
Easting: 1314088.2
Elevation: 30.3
Total Depth: 50.7 ft
Termination Criteria:
Test Depth: 48.7 ft



BRN-P2-29C

Electronic File Name: BRN-P2-29C.dis

Appendix F: Borehole Inclinometer

DRILLING LOG	DIVISION	INSTALLATION	SHEET 1 OF 3 SHEETS

1. PROJECT IBWC (LAB data included)	10. SIZE AND TYPE OF BIT
2. LOCATION (Coordinates or Station) Brownsville, TX	11. DATUM FOR ELEVATION SHOWN (TBM or MSL)
3. DRILLING AGENCY	12. MANUFACTURER'S DESIGNATION OF DRILL
4. HOLE NO. (As shown on drawing title and file number) P3-31	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
5. NAME OF DRILLER	14. TOTAL NUMBER CORE BOXES
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED / COMPLETED
7. THICKNESS OF OVERBURDEN	17. ELEVATION TOP OF HOLE
8. DEPTH DRILLED INTO ROCK	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE 61.5	19. SIGNATURE OF INSPECTOR

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
	0.0		- 0.0' to 1.5' Clay (CL): dark grey, stiff			SPT: 4-8-7
			- 1.5' to 3.0' Silty Sand (SM): dry			SPT: 3-3-5
9			- 3.0' to 4.5' light brown lean clay with sand			SPT: 6-7-7
			- 4.5' to 6.0' Silty Sand (SM): light grey, laminated, dry			SPT: 7-4-3
11			- 6.0' to 7.5' light brown sandy silty clay			SPT: 4-3-2
			- 7.5' to 9.0' Silt (ML): some sand, light grey			SPT: 2-3-4
	10		- 9.0' to 10.5' Silt (ML) some sand, brown			SPT: 2-1-2
29			- 10.5' to 12.0' Brown sandy silty clay			SPT: 1-1-1
33			- 12.0' to 13.5' Brown lean clay			SPT: wt-wt-1
34			- 13.5' to 15.0' Brown silt			SPT: wt-wt-1
			- 15.0' to 16.5' Silty Sand (SM): very wet, dark grey			SPT: wt-wt-wt
32			- 16.5' to 18.0' Brown lean clay			SPT: wt-wt-1
33			- 18.0' to 19.5' Brown lean clay			SPT: wt-1-1
33			- 19.5' to 21.0' Brown lean clay			SPT: wt-1-1

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE | **Hole No. P3-31**

PROJECT: IBWC (LAB data included) | INSTALLATION | SHEET 2 OF 3 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			- 19.5' to 21.0' Brown lean clay *(continued)*			
			- 21.0' to 22.5' Clay (CH): soft, some organics, rotts, wood, wet			
30			- 22.5' to 24.0' Brown lean clay			SPT: 1-1-2
			- 24.0' to 25.5' Silt (ML): dark grey, organics			SPT: 1-2-2
	30		- 25.5' to 31.5' Silt (ML): laminated, organics, dark grey to black, large pieces of wood			SPT: 2-4-7
			- 31.5' to 35.0' Silt (ML) dark grey, organics, wood, laminated			
			- 35.0' to 45.0' Clay (CH): dense, stiff, tan - 35.1' to 36.0' Sparry Calcite crystals			
	40					SPT: 3-5-8

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE | **Hole No. P3-31**

PROJECT: IBWC (LAB data included) | INSTALLATION | SHEET 3 OF 3 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			- 35.0' to 45.0' Clay (CH): dense, stiff, tan *(continued)*			
25			- 45.0' to 46.5' Silt (ML) to Silty Sand (SM): wet, soft, some organics			SPT: 3-4-6
	50		- 46.5' to 55.0' Silt (ML) to Silty Sand (SM): wet, soft, few organics			SPT: 3-4-6
			- 55.0' to 60.0' Clay (CH): dense, stiff, tan			SPT: 3-5-7
26	60		- 60.0' to 61.5' Silty Sand (SM): tan, wet			SPT: 3-4-5

Hole No. P3-32

DRILLING LOG

DIVISION	
INSTALLATION	
SHEET 1 OF 4 SHEETS	
1. PROJECT	IBWC (LAB data included)
2. LOCATION (Coordinates or Station)	Brownsville, TX
3. DRILLING AGENCY	
4. HOLE NO. *(As shown on drawing title and file number)*	P3-32
5. NAME OF DRILLER	
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN	
8. DEPTH DRILLED INTO ROCK	
9. TOTAL DEPTH OF HOLE	80.0
10. SIZE AND TYPE OF BIT	
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*	
12. MANUFACTURER'S DESIGNATION OF DRILL	
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED: UNDISTURBED:
14. TOTAL NUMBER CORE BOXES	
15. ELEVATION GROUND WATER	
16. DATE HOLE	STARTED: COMPLETED:
17. ELEVATION TOP OF HOLE	
18. TOTAL CORE RECOVERY FOR BORING	%
19. SIGNATURE OF INSPECTOR	

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
	0.0		- 0.0' to 4.7' Clay (CL-CH) alternating from stiff to soft; brown.			SPT: 5-2-2 SPT: 1-2-2 SPT: 3-4-5
22			- 4.7' to 6.9' Grayish brown lean clay			
19			- 6.9' to 9.1' Brown lean clay			
18	10		- 9.1' to 11.3' Brown lean clay			
22			- 11.3' to 13.5' Brown lean clay			
25			- 13.5' to 15.7' Brown lean clay			
27			- 15.7' to 17.9' Brown fat clay			
29			- 17.9' to 20.1' Grayish brown fat clay			

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE	Hole No. P3-32
PROJECT: IBWC (LAB data included)	INSTALLATION	SHEET 2 OF 4 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
25			- 20.1' to 22.3' Dark brown fat clay			
28			- 22.3' to 24.5' Grayish brown lean clay			
31			- 24.5' to 26.7' Dark brown lean clay			
26			- 26.7' to 29.0' Dark brown lean clay			
28	30		- 29.0' to 31.2' Dark brown lean clay			
29			- 31.2' to 33.4' brown fat clay			
27			- 33.4' to 35.6' brown fat clay			
26			- 35.6' to 37.6' brown fat clay			
	40		- 37.6' to 42.0' Clay (CH): tan, softer, more stiff, moist			SPT: 2-2-3
			- 42.0' to 45.0' Clay (CH): tan, dense, stiff, Sparry calcite crystals			SPT: 2-4-5

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. P3-32
PROJECT: IBWC (LAB data included)	INSTALLATION		SHEET 3 OF 4 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			- 42.0' to 45.0' Clay (CH): tan, dense, stiff, Sparry calcite crystals *(continued)*			
28			- 45.0' to 48.0' light brown fat clay			SPT: 2-3-4
	50		- 48.0' to 51.0' Clay (CH): tan, dense, stiff			SPT: 3-4-6
			- 51.0' to 54.0' Clay (CH): tan, dense, stiff			SPT: 3-4-7
21			- 54.0' to 57.0' light brown lean clay			SPT: 3-5-7
			- 57.0' to 60.0' Clay (CH): tan, dense, stiff			SPT: 3-4-5
	60		- 60.0' to 63.0' Clay (CH): tan, dense, stiff, transitioning to (SM) tan, silty sand, wet			SPT: 3-5-9
26			- 63.0' to 66.0' light brown lean clay with sand			SPT: 2-4-5
26			- 66.0' to 69.0' light brown sandy silt			SPT: 4-6-9

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. P3-32
PROJECT IBWC (LAB data included)		INSTALLATION	SHEET 4 OF 4 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (*Drilling time, water loss, depth weathering, etc., if significant*) g
			- 66.0' to 69.0' light brown sandy silt *(continued)*			
25	70		- 69.0' to 70.0' light brown silty sand			SPT: 3-7-14
26			- 72.0' to 73.5' light brown silty sand			SPT: 3-4-7
			- 73.5' to 75.0' Fine grained sand with silt (SP-SM): wet			
			- 75.0' to 78.0' Fine grained sand with silt (SP-SM) - transition into Clay (CH)			SPT: 4-8-10
	80		- 78.0' to 80.0' Clay (CH): tan, dense, stiff			SPT: 8-9-11
	90					

Hole No. P3-33

DRILLING LOG	DIVISION	INSTALLATION	SHEET 1 OF 4 SHEETS

1. PROJECT IBWC (LAB data included)	10. SIZE AND TYPE OF BIT
2. LOCATION (Coordinates or Station) Brownsville, TX	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*
3. DRILLING AGENCY	12. MANUFACTURER'S DESIGNATION OF DRILL
4. HOLE NO. *(As shown on drawing title and file number)* P3-33	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN — DISTURBED — UNDISTURBED
5. NAME OF DRILLER	14. TOTAL NUMBER CORE BOXES
	15. ELEVATION GROUND WATER
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	16. DATE HOLE — STARTED — COMPLETED
	17. ELEVATION TOP OF HOLE
7. THICKNESS OF OVERBURDEN	18. TOTAL CORE RECOVERY FOR BORING %
8. DEPTH DRILLED INTO ROCK	19. SIGNATURE OF INSPECTOR
9. TOTAL DEPTH OF HOLE 70.0	

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
	0.0		- 0.0' to 2.0' Gravel with Silt (GM)-fill (top of the parking area)			SPT: 1-2-3
22			- 2.0' to 4.2' Brown Lean clay			
			- 4.2' to 6.4' Silty Sand (SM)			
25			- 6.4' to 8.4' Brown lean clay			
	10		- 8.4' to 10.8' Poorly graded sand with Silt (SP-SM)			
27			- 10.8' to 13.0' Brown silty clay with sand			
30			- 13.0' to 15.2' Grayish brown lean clay			
			- 15.2' to 17.2' Silt (ML): very soft and wet			
32			- 17.2' to 18.8' Brown silty clay with sand			SPT: wt-wt-wt
30			- 18.8' to 20.3' Brown silt with sand			SPT: 3-6-3

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE

Hole No. P3-33

PROJECT: IBWC (LAB data included)

INSTALLATION:

SHEET 2 OF 4 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
31			- 20.3' to 21.8' Brown silty clay with sand			SPT: 4-3-1
29			- 21.8' to 23.3' Silty Sand (SM): dark grey, very wet, very soft, more charred wood			SPT: 4-3-1
26			- 23.3' to 24.8' Brown sandy silt			SPT: 1-3-4
			- 24.8' to 26.3' Silty Sand (SM) transitioning into hard, dense, dark grey clay			SPT: 2-2-2
32			- 26.3' to 27.8' Clay (CH): dense grey clay, moist, uniform consistency			SPT: 3-3-4
37			- 27.8' to 29.3' Brown lean clay - 28.9' to 29.3' Clay (CH): dense			SPT: 3-3-4
	30		- 29.3' to 30.8' Silt (ML): very wet, some sand, fairly soft, firmer with depth, dark grey			SPT: 3-4-4
29			- 30.8' to 32.3' Brown silt			SPT: 1-1-2
29			- 32.3' to 33.8' Brown silt			SPT: 1-4-5
			- 33.8' to 35.3' Clay with silt (CL-ML): firm, dark grey, very wet, firmer with depth			
			- 35.3' to 38.3' Silt with some sand (ML) to Sandy Silt (SM)			
	40		- 38.3' to 41.3' Silt with some sand, not as wet, with sand-sized organics - Clay (CH) at bottom of sample			SPT: 4-5-6
27			- 41.3' to 44.3' Brown and tan lean clay - tan clay			SPT: 5-6-6

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE | **Hole No. P3-33**

PROJECT: IBWC (LAB data included) | INSTALLATION | SHEET 3 OF 4 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			- 44.3' to 47.3' Clay (CH): dense, tan, some light grey clay mixed			SPT: 5-7-9
22			- 47.3' to 47.7' Light brown lean clay with sand			SPT: 6-6-8
	50		- 47.7' to 50.3' Sand (SM) at base; visible mica			
			- 50.3' to 53.3' Silty Sand (SM): tan, laminated, wet, some fine-grained organics			SPT: 2-2-4
			- 53.3' to 56.3' Silty Sand (SM): very wet, Iron staining			SPT: 2-3-4
26			- 56.3' to 57.8' Light brown lean clay			SPT: 3-5-6
			- 57.8' to 59.3' Silt (ML) interbedded with Clay (CH): tan, very wet, clay has some iron staining			
	60		- 59.3' to 62.3' Clay (CH): tan, some silt, fairly soft, some iron staining, very moist			SPT: 4-3-1
			- 62.3' to 65.3' Silty Sand (SM): tan, laminated, thin clay layers, very wet, some Iron staining			SPT: 4-5-8
30			- 65.3' to 66.8' light brown fat clay			SPT: 3-5-7
			- 66.8' to 68.3' Silty Sand (SM) with Clay (CH): laminated clay and sand. Clay has conchoidal fracture, Iron staining			

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. P3-33
PROJECT IBWC (LAB data included)		INSTALLATION	SHEET 4 OF 4 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			- 68.3' to 70.0' Clay (CH): very firm, dry, tan, Iron staining			SPT: 5-7-8
	70					
	80					
	90					

DRILLING LOG	DIVISION	INSTALLATION	SHEET 1 OF 3 SHEETS

1. PROJECT IBWC (LAB data included)	10. SIZE AND TYPE OF BIT
2. LOCATION (Coordinates or Station) Brownsville, TX	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*
3. DRILLING AGENCY	12. MANUFACTURER'S DESIGNATION OF DRILL
4. HOLE NO. *(As shown on drawing title and file number)* P3-34	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
5. NAME OF DRILLER	14. TOTAL NUMBER CORE BOXES
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED / COMPLETED
7. THICKNESS OF OVERBURDEN	17. ELEVATION TOP OF HOLE
8. DEPTH DRILLED INTO ROCK	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE 60.0	19. SIGNATURE OF INSPECTOR

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
	0.0		- 0.0' to 1.5' Gravel with sitl ant Silt			SPT: 2-2-3
			- 1.5' to 3.0' Silty Sand (SM): moist, brown			SPT: 2-3-5
16			- 3.0' to 4.5' Silty Sand (SM): loose, soft, moist			SPT: 3-2-2
			- 4.5' to 6.0' Silty Sand (SM): more silt, dark brown, moist			SPT: 1-5-7
			- 6.0' to 7.5' Silt (ML): hard packed, with gravel - White Calcite crust and concretions			SPT: 7-8-10
			- 7.5' to 9.5' Rock- Crystalline Limestone			
8	10		- 9.5' to 10.5' Tan clayey gravel with sand			SPT: 9-9-7
18			- 10.5' to 12.0' Brown silty sand with gravel			SPT: 10-5-3
39			- 12.0' to 13.5' Silt with sand and some gravel (SM-ML): dark grey, some wood debris			SPT: 1-1-1
35			- 13.5' to 15.0' Brown lean clay			SPT: 0-1-1
31			- 15.0' to 16.5' Brown lean clay			SPT: 1-5-7
30			- 16.5' to 18.0' Brown lean clay			SPT: 1-1-1
29			- 18.0' to 19.5' Brown lean clay			SPT: 1-2-1
32			- 19.5' to 21.0' Brown lean clay			

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE	Hole No. P3-34
PROJECT IBWC (LAB data included)	INSTALLATION	SHEET 2 OF 3 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
			- 19.5' to 21.0' Brown lean clay *(continued)*			SPT: 1-1-2
33			- 21.0' to 22.5' Brown lean clay with sand			SPT: 2-2-3
			- 22.5' to 24.0' Clay (CH): dark grey, soft, wood debris			SPT: 2-4-5
28			- 24.0' to 25.5' light brown and brown lean clay			SPT: 2-2-3
			- 25.5' to 27.0' Silt (ML) to Silty Sand (SM): wood debris, organics			
			- 27.0' to 30.0' Silt (ML) to Silty Sand (SM): wood debris, organics			SPT: 3-2-2
24	30		- 30.0' to 33.0' Silt (ML) - transition into Clay (CH): tan			SPT: 3-4-5
32			- 33.0' to 34.5' Clay (CH): tan, dense, stiff			SPT: 2-3-5
			- 34.5' to 37.5' Clay (CH): tan, dense, stiff			SPT: 2-3-5
	40		- 37.5' to 42.0' Clay (CH): tan, dense, stiff			SPT: 2-4-6
27			- 42.0' to 43.5' light brown lean clay			SPT: 2-3-5
			- 43.5' to 45.0' Clay (CH)			

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. P3-34
PROJECT IBWC (LAB data included)		INSTALLATION	SHEET 3 OF 3 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			- 43.5' to 45.0' Clay (CH) *(continued)*			
			- 45.0' to 51.0' Clay (CH): tan, dense, stiff			SPT: 4-4-5
						SPT: 3-4-5
	50					
			- 51.0' to 54.0' Clay (CH): tan, not too stiff			SPT: 4-5-6
26			- 54.0' to 55.5' light brown lean clay			SPT: 3-4-9
			- 55.5' to 58.5' Clay (CH): tan, dense, stiff, Iron staining			
			- 57.0' to 58.5' Silty Sand (SM): tan with Iron staining			SPT: 5-9-9
			- Clay (CH)			
27			- 58.5' to 60.0' Clay (CH): tan, dense, stiff			SPT: 7-7-9
	60					

DRILLING LOG	DIVISION	INSTALLATION	SHEET 1 OF 4 SHEETS

1. PROJECT IBWC (LAB data included)	10. SIZE AND TYPE OF BIT
2. LOCATION (Coordinates or Station) Brownsville, TX	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*
3. DRILLING AGENCY	12. MANUFACTURER'S DESIGNATION OF DRILL
4. HOLE NO. *(As shown on drawing title and file number)* P3-35	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
5. NAME OF DRILLER	14. TOTAL NUMBER CORE BOXES
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED / COMPLETED
7. THICKNESS OF OVERBURDEN	17. ELEVATION TOP OF HOLE
8. DEPTH DRILLED INTO ROCK	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE 70.0	19. SIGNATURE OF INSPECTOR

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
	0.0		- 0.0' to 1.5' Clay (CL); sandy, organics			SPT: 1-8-11
			- 1.5' to 3.0' Silt (ML) and clay (CL): dry, stiff, than and dark grey with organics			SPT: 9-9-11
8			- 3.0' to 4.5' Light brown lean clay			SPT: 5-5-4
			- 4.5' to 6.0' Silt (ML); tan, dry, mottled with clay lenses			SPT: 5-3-3
			- 6.0' to 7.5' silt (ML) with sand (SM-SP), laminated, dry			SPT: 3-2-3
17			- 7.5' to 9.0' Brown silty clay with sand			SPT: 3-2-2
	10		- 9.0' to 10.5' Clayey-Silty sand (SM-SC): tan, grey, moist, slightly plastic, mottley			SPT: 1-2-3
			- 10.5' to 12.0' Clay (CL): grey, soft, mottled, moist to wet.			SPT: 2-1-2
32			- 12.0' to 13.5' Brown lean clay			SPT: 1-1-1
			- 13.5' to 15.0' Silt (ML) grey to brown, wet, yello-orange glass; wet organics			SPT: 1-1-1
30			- 15.0' to 16.5' Brown lean clay			SPT: wt-wt-wt
			- 16.5' to 19.5' Silt (CL-ML) uniform, dark grey, wet, soft, with few roots 1/16" diameter			SPT: wt-wt-wt
33			- 19.5' to 21.0' Brown silty clay with sand			SPT: wt-wt-wt

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE	Hole No. P3-35
PROJECT: IBWC (LAB data included)	INSTALLATION	SHEET 2 OF 4 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			- 19.5' to 21.0' Brown silty clay with sand *(continued)*			
			- 21.0' to 22.5' Silt (ML): dark grey, wet, very soft; slight sand, black wood at the bottom			SPT: wt-1-1
			- 22.5' to 22.8' Peat, Clay with organics			SPT: 1-1-1
			- 22.8' to 25.5' Clay (CL): dark grey, wet, silty			SPT: 1-1-2
31			- 25.5' to 27.0' Clay (CL) with silt, TRANSITION, wet, dark grey			
			- 27.0' to 28.5' Sandy Silt (ML): soft, wet, dark grey			SPT: 1-2-2
31			- 28.5' to 30.0' Brown silty clay			SPT: 2-2-2 SPT: wt-2-2
	30		- 30.0' to 31.5' Silt (ML): soft, damp, dark grey, uniform			
31			- 31.5' to 33.0' Brown lean clay			SPT: 1-2-2
			- 33.0' to 34.5' Clay (CL): silty, dark grey, moist, soft			SPT: 1-2-2
30			- 34.5' to 36.0' Clay (CL): silty, dark grey, moist			SPT: 2-2-4
			- 36.0' to 37.5' Clay (CL): silty, dark grey, moist; Wood/organics-Peat at 35.5 ft			
27			- 37.5' to 39.0' Brown lean clay			SPT: 2-4-5
	40		- 39.0' to 40.5' Clay (CH): tan, some organics, brown; Grey, weathered, mottled, dry, stiff			
			- 40.5' to 43.5' Clay (CH): tan, some organics, brown; Grey, weathered, mottled, dry, stiff			SPT: 2-5-9
22			- 43.5' to 45.0' Light brown lean clay			SPT: 3-6-9

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE

Hole No. P3-35

PROJECT: IBWC (LAB data included)

INSTALLATION

SHEET 3 OF 4 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			- 43.5' to 45.0' Light brown lean clay *(continued)*			
			- 45.0' to 46.5' Clay (CH): wet, very soft, tan, oxidized			
26			- 46.5' to 48.0' Light brown lean clay			SPT: 2-2-3
			- 48.0' to 49.5' Clay (CH): wet, very soft, tan			
27	50		- 49.5' to 51.0' Clay (CL): tan, brown, wet, very soft, silty (ML), Possibly CL-ML			SPT: 2-4-4
			- 51.0' to 52.4' clay-Silt (CL-ML): tan, orange mottles, very soft, wet			SPT: 2-2-6
26			- 52.4' to 54.0' Light brown lean clay			
			- 54.0' to 55.5' clay-Silt (CL-ML): tan, orange mottles, very soft, wet			
25			- 55.5' to 57.0' Light brown silty clay with sand			
			- 57.0' to 58.5' Silt (ML): with clay layers, tan, wet, soft, increasing sand (very fine) content			
26			- 58.5' to 60.0' Light brown lean clay			SPT: 2-3-4
	60		- 60.0' to 61.5' Clay (CL-CH): laminated, tan, brown with organics, soft to stiff, very soft			SPT: 4-5-8
26			- 61.5' to 63.0' Light brown silty, clayey sand			SPT: 2-3-5
			- 63.0' to 64.5' Sand (SP-SM): tan, very fine grained, loose, uniform, clay (CH) and bottom 0.2'			
27			- 64.5' to 66.0' Light brown fat clay			SPT: 2-6-7
			- 66.0' to 67.5' Clay (CL-CH): grey with mottles (red/orange), very stiff to hard, dry			
						SPT: 2-4-6

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE		Hole No. P3-35
PROJECT: IBWC (LAB data included)	INSTALLATION	SHEET 4 OF 4 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			- 67.5' to 70.0' Clay (CL): grey brown, mottled (orange), very stiff to hard, dry *(continued)*			
27	70		- 70.0' to 71.5' Light brown lean clay			SPT: 2-2-3
			- 71.5' to 72.0' Clay (CL-ML): brown-tan, moist, very soft, with mottles			
	80					
	90					

DRILLING LOG	DIVISION	INSTALLATION	SHEET 1 OF 3 SHEETS

1. PROJECT IBWC (LAB data included)	10. SIZE AND TYPE OF BIT
2. LOCATION (Coordinates or Station) Brownsville, TX	11. DATUM FOR ELEVATION SHOWN (TBM or MSL)
3. DRILLING AGENCY	12. MANUFACTURER'S DESIGNATION OF DRILL
4. HOLE NO. (As shown on drawing title and file number) P3-36	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
5. NAME OF DRILLER	14. TOTAL NUMBER CORE BOXES
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED / COMPLETED
7. THICKNESS OF OVERBURDEN	17. ELEVATION TOP OF HOLE
8. DEPTH DRILLED INTO ROCK	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE 60.0	19. SIGNATURE OF INSPECTOR

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
	0.0		- 0.0' to 1.5' silty clay (CL) dark gray with organics, plastic			SPT: 2-3
			- 1.5' to 3.0' silty sand (SM) grey, vfg REC 0.8'			SPT: 3-2-2
			- 3.0' to 4.5' silty sand (SM); brown., vfg rec 0.8'			SPT: 1-1-2
21			- 4.5' to 6.0' Brown lean clay with sand			SPT: 1-1-1
			- 6.0' to 6.7' silty sand (SM) rec 1.5			SPT: 2-2-2
20			- 6.7' to 7.5' Brown sandy lean clay			
			- 7.5' to 8.7' gravel/cobbles			SPT: 5-2-5
	10		- 8.7' to 10.5' gravel (lms)			
			- 10.5' to 12.0' LMS rock/ riprap, cobbles old channel			
19			- 12.0' to 12.8' Brown sandy lean clay			
			- 12.8' to 13.5' Silty sand (SM) mix with lms rock			SPT: 3-11-10
27			- 13.5' to 15.0' silty sand (SM) grey, wet, vfg, rec 0.8'			SPT: 3-11-10
29			- 15.0' to 16.5' Brown Silty sand			SPT: 3-2-2
23			- 16.5' to 18.0' silty sand (SM); grey, wet, coarse sand, rec. 0.8'			
29			- 18.0' to 19.5' Silty sand (SM); grey moist, vgf, piecesof wood and roots - silt (ML) at 19.2, moist, grey			
28			- 19.5' to 21.0' Brown silty sand			

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE

Hole No. P3-36

PROJECT: IBWC (LAB data included)

INSTALLATION

SHEET 2 OF 3 SHEETS

% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (*Drilling time, water loss, depth weathering, etc., if significant*) g
			- 19.5' to 21.0' Brown silty sand *(continued)*			SPT: 1-1-2
29			- 21.0' to 22.5' Brown lean clay			SPT: 2-3-2
27			- 22.5' to 24.0' Brown silty clay with sand			SPT: 1-2-2
26			- 24.0' to 25.0' Brown lean clay			
25			- 25.0' to 25.5' silty clay (CL) brown			SPT: 2-2-3
			- 25.5' to 30.0' tan, stiff clay (CH) with organics, rec 1.5			
25	30		- 30.0' to 31.5' light brown fat clay			SPT: 2-3-4
			- 31.5' to 35.0' tan, stiff clay (CH) with organics, rec 1.5			
			- 35.0' to 40.0' tan, stiff clay (CH)			SPT: 2-3-4
24	40		- 40.0' to 41.5' tan, stiff clay (CH)			SPT: 1-1
			- 41.5' to 45.0' Silt (ML) wet, soft, uniform, slight cohession			

DRILLING LOG (Cont Sheet)

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE	Hole No. P3-36
PROJECT: IBWC (LAB data included)	INSTALLATION	SHEET 3 OF 3 SHEETS

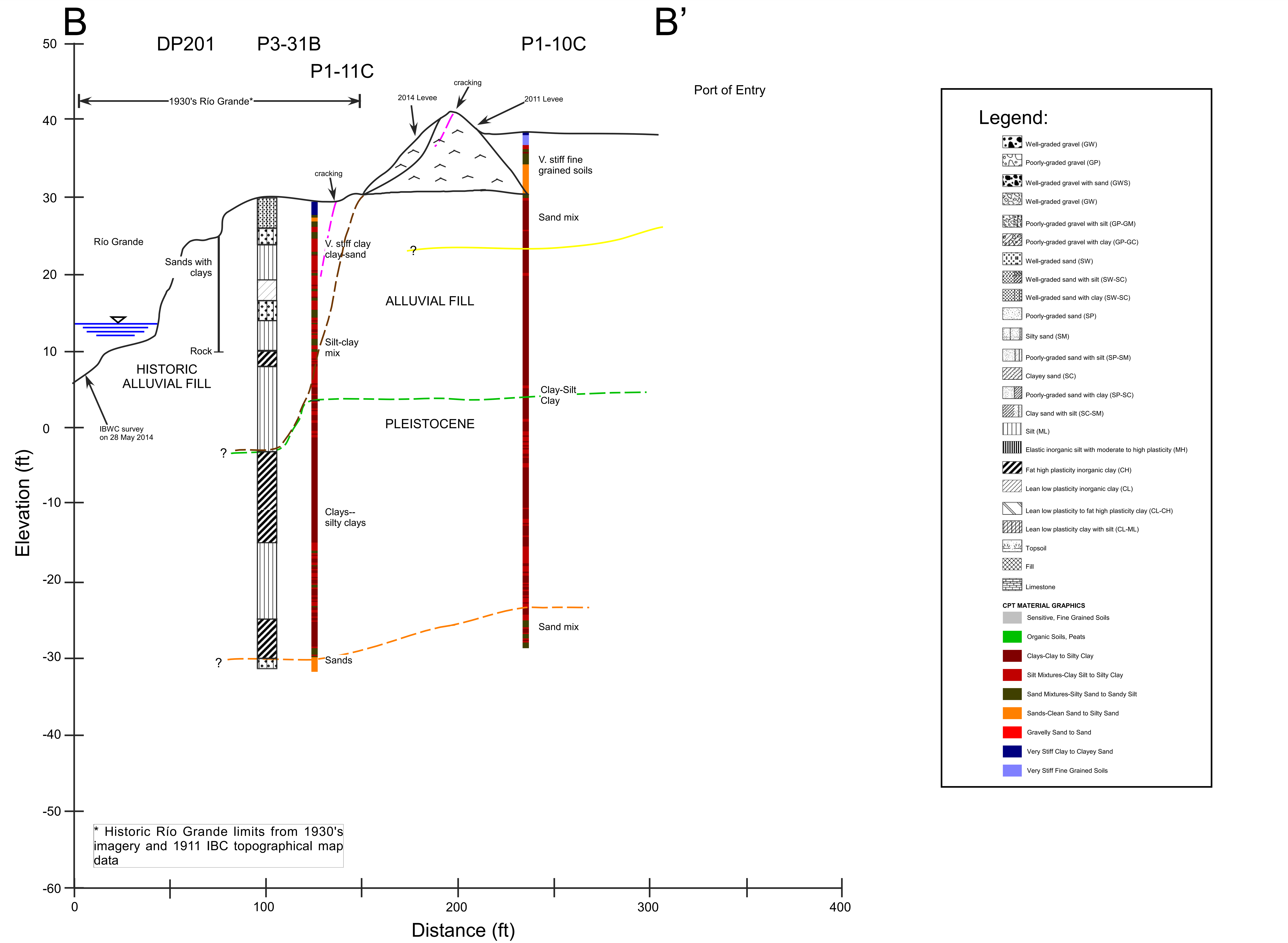
% MOISTURE CONTENT a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			- 41.5' to 45.0' Silt (ML) wet, soft, uniform, slight cohession *(continued)*			
28			- 45.0' to 46.5' light brown silty clay			
			- 46.5' to 50.0' Silt (ML): wet, soft, uniform, slight cohesion.			
	50		- 50.0' to 52.2' clayey silt (ML): brown, tan, moist			SPT: 1-4-10
22			- 52.2' to 60.0' light brown silty sand			SPT: 4-18-18
	60					

ENG FORM 1836-A JUN 67

PROJECT: IBWC (LAB data included) HOLE NO. P3-36

Appendix G: Cross-Sections


A
A'
DP201
P1-13C
B1-1984
B2-1984
CB-4A (proj)
P2-14
P1-11C
P3-31B
P2-15C
P2-16C
P3-33B
P2-18C
P2-19C
P3-35B
P2-20C
DP-202 (proj)
P2-21C
P3-36B
P2-22C
P1-03C
P1-02C
P2-24C
LEVEE
FILL
Rip Rap
Clay f. sand
Sandy silty clay brown, stiff
Green grey sandy-silt clay
Sand
V. soft
V. stiff
Stiff silt
V. stiff silty clay
Clay and silty clay
Rock
very stiff clay clayey sand
Clay and silty clay
Clay
PLEISTOCENE ALLUVIUM
Sand mix
Sands-silty sand with clay
Clay, silty sand with clay
Clay/silty clay
Silty clay/ sandy silt
well screen
Mixed sand, silt and clay
Mix clay, silt and sand
Clay/ silty clay
Sand/silt
Mix sand silty clay
Clay and silty clay
Mix clay and sand
Clay and silty clay
Refusal
Sand and clay
Lake Brown
Ressaca
POINT BAR
PLEISTOCENE
Clean sand
Sands
Fill
Mixed sand
Very stiff, fine-grained soils
Legend:
Well-graded gravel (GW)
Poorly-graded gravel (GP)
Well-graded gravel with sand (GWS)
Well-graded gravel (GW)
Poorly-graded gravel with silt (GP-GM)
Poorly-graded gravel with clay (GP-GC)
Well-graded sand (SW)
Well-graded sand with silt (SW-SC)
Well-graded sand with clay (SW-SC)
Poorly-graded sand (SP)
Silty sand (SM)
Poorly-graded sand with silt (SP-SM)
Clayey sand (SC)
Poorly-graded sand with clay (SP-SC)
Clay sand with silt (SC-SM)
Silt (ML)
Elastic inorganic silt with moderate to high plasticity (MH)
Fat high plasticity inorganic clay (CH)
Lean low plasticity inorganic clay (CL)
Lean low plasticity to fat high plasticity clay (CL-CH)
Lean low plasticity clay with silt (CL-ML)
Topsoil
Fill
Limestone
CPT MATERIAL GRAPHICS
Sensitive, Fine Grained Soils
Organic Soils, Peats
Clays-Clay to Silty Clay
Silt Mixtures-Clay Silt to Silty Clay
Sand Mixtures-Silty Sand to Sandy Silt
Sands-Clean Sand to Silty Sand
Gravelly Sand to Sand
Very Stiff Clay to Clayey Sand
Very Stiff Fine Grained Soils
SUBSURFACE PROFILE
Diagram: A-A' (A-L)
Sta. 1894+00 to 1904+85
Created by:
Joseph B. Dunbar, Ph.D
Carla Roig-Silva
IBWC
DATE
12/19/2014
U.S. Army Corps of Engineers Engineering Research and Development Center Geotechnical and Structures Laboratory
ERDC



SUBSURFACE PROFILE

Diagram: B-B' (B-111)

Sta. 1898+43

Created by:

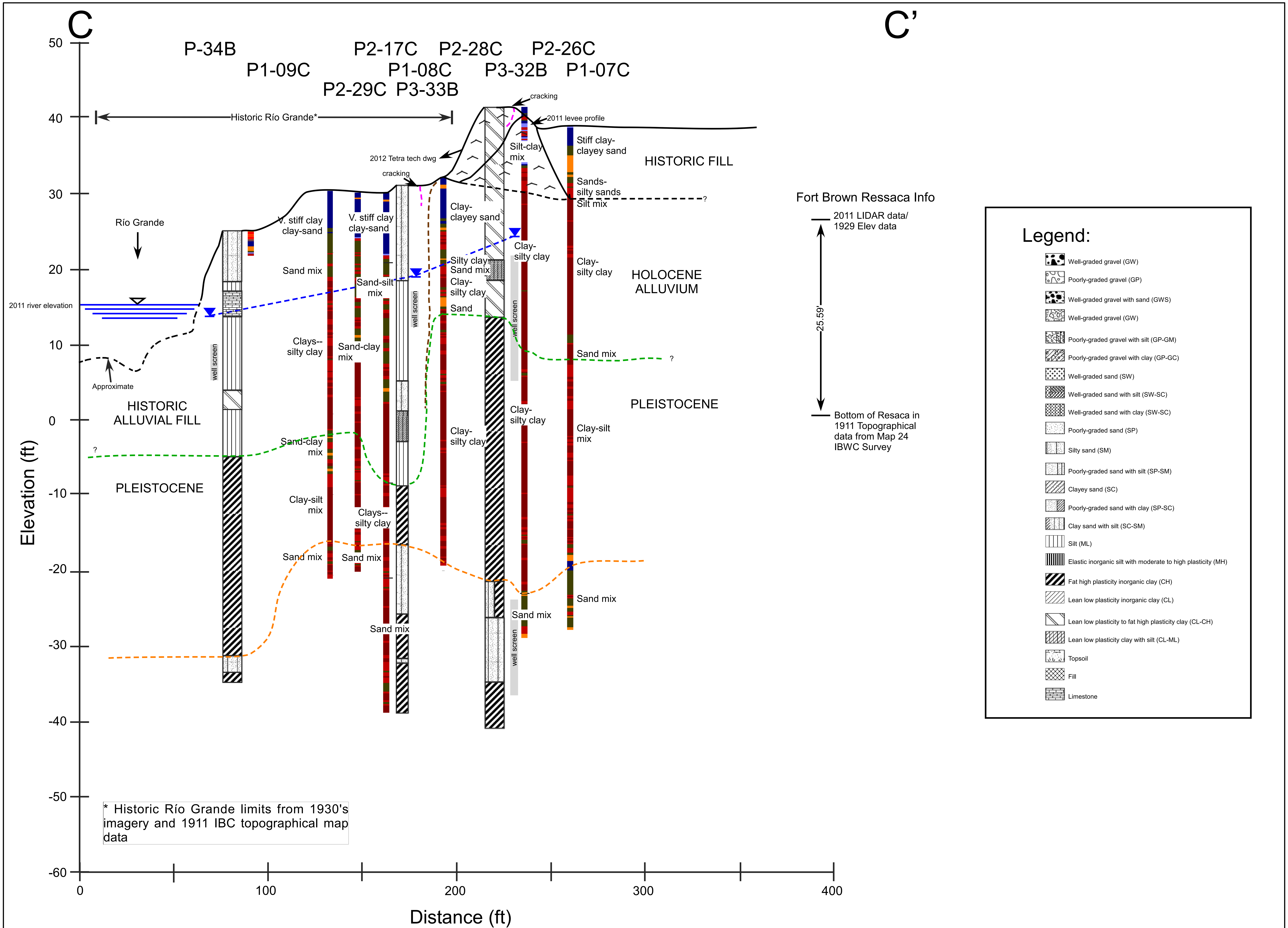
Joseph B. Dunbar, Ph.D
Carla Roig-Silva

IBWC

DATE

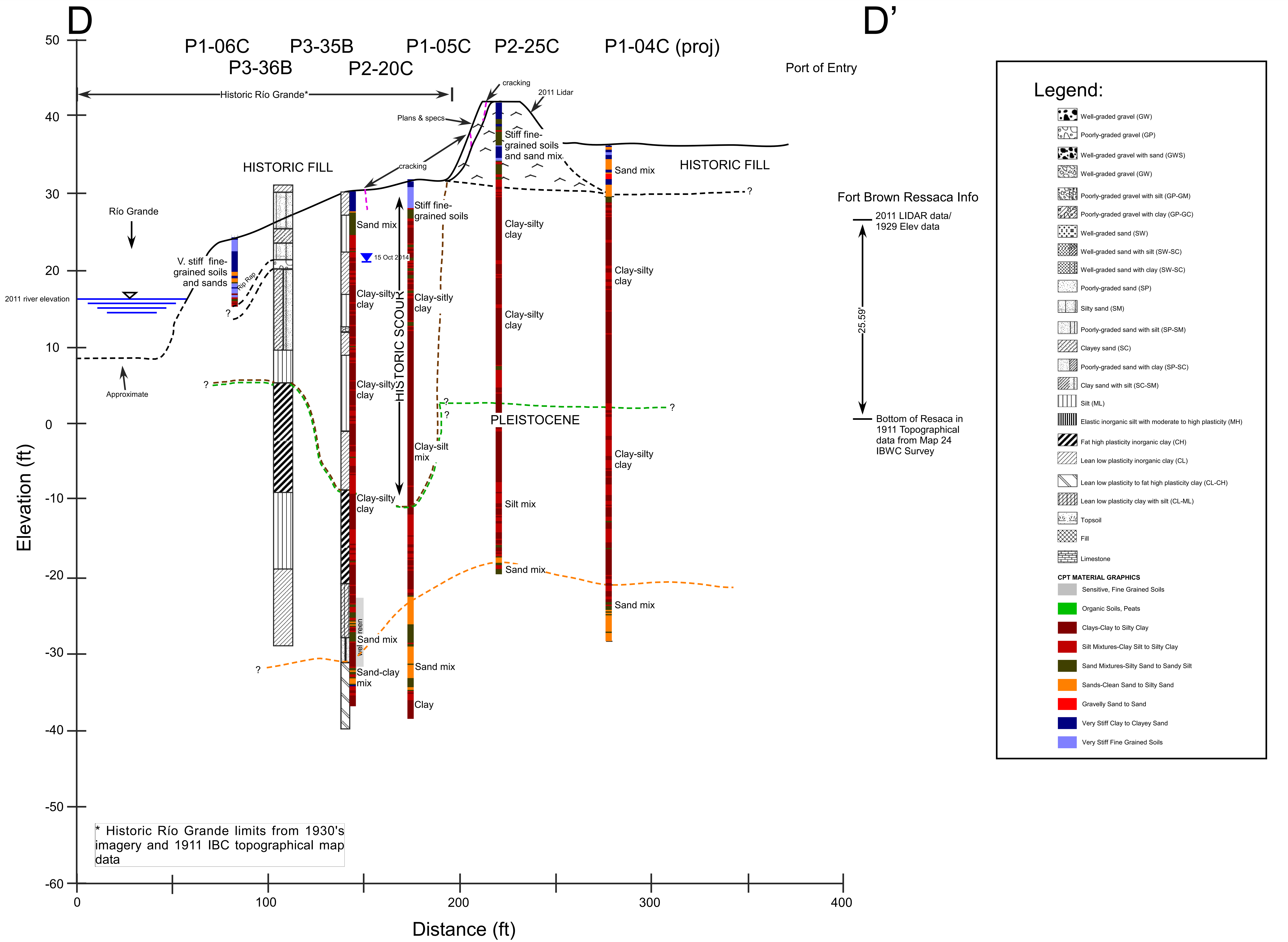
12/19/2014

U.S. Army Corps of Engineers
Engineering Research and Development Center
Geotechnical and Structures Laboratory



SUBSURFACE PROFILE

Diagram: C-C' (C-211) Sta.1900+13 Created by: Joseph B. Dunbar, Ph.D Carla Roig-Silva	IBWC DATE 12/19/2014	U.S. Army Corps of Engineers Engineering Research and Development Center Geotechnical and Structures Laboratory



SUBSURFACE PROFILE

Diagram: D-D’ (D-311)

Sta. 1902+28.5

Created by:

Joseph B. Dunbar, Ph.D
Carla Roig-Silva

IBWC

DATE 12/19/2014

U.S. Army Corps of Engineers Engineering Research and Development Center Geotechnical and Structures Laboratory

Appendix H: Survey Data

8/26/2014				9/9/2014													
			TOTAL STATION						DELTAS								
Point ID	Northing	Easting	Elevation	Point ID	Northing	Easting	Elevation	Northing	Easting	Elevation	Point ID	Northing	Easting	Elevatio	Northing	Easting	Elevation
0826_TX1A	2865310.992	650582.353	12.548	0909_TX1A	2865310.975	650582.326	12.551	-0.017	-0.027	0.003							
0826_TX1B	2865308.459	650577.368	12.197	0909_TX1B	2865308.439	650577.373	12.194	-0.02	0.005	-0.003							
0826_TX1C	2865305.821	650571.867	10.631	0909_TX1C	2865305.833	650571.87	10.628	0.012	0.003	-0.003							
0826_TX1D	2865304.163	650567.499	9.205	0909_TX1D	2865304.169	650567.494	9.199	0.006	-0.005	-0.006							
0826_TX1E	2865299.203	650558.951	8.603	0909_TX1E	2865299.184	650558.954	8.6	-0.019	0.003	-0.003							
0826_TX1F	2865297.171	650553.596	8.441	0909_TX1F	2865297.163	650553.621	8.438	-0.008	0.025	-0.003							
0826_TX1G	2865292.962	650545.382	7.76	0909_TX1G	2865292.963	650545.386	7.755	0.001	0.004	-0.005							
0826_TX2A	2865261.473	650607.885	11.972	0909_TX2A	2865261.479	650607.895	11.967	0.006	0.01	-0.005	1001_TX2A	2865261.484	650607.89	11.955	-0.011	-0.005	0.017
0826_TX2B	2865259.738	650604.4	12.332	0909_TX2B	2865259.759	650604.427	12.325	0.021	0.027	-0.007	1001_TX2B	2865259.765	650604.436	12.319	-0.027	-0.036	0.013
0826_TX2C	2865256.211	650596.269	10.279	0909_TX2C	2865256.216	650596.309	10.285	0.005	0.04	0.006	1001_TX2C	2865256.21	650596.301	10.284	0.001	-0.032	-0.005
0826_TX2D	2865254.096	650590.729	9.405	0909_TX2D	2865254.102	650590.751	9.399	0.006	0.022	-0.006	1001_TX2D	2865254.096	650590.748	9.395	0	-0.019	0.01
0826_TX2E	2865249.201	650579.525	9.2	0909_TX2E	2865249.21	650579.535	9.197	0.009	0.01	-0.003	1001_TX2E	2865249.206	650579.529	9.194	-0.005	-0.004	0.006
0826_TX2F	2865247.052	650574.468	8.9	0909_TX2F	2865247.067	650574.481	8.896	0.015	0.013	-0.004	1001_TX2F	2865247.061	650574.481	8.89	-0.009	-0.013	0.01
0826_TX2G	2865241.135	650562.056	6.94	0909_TX2G	2865241.139	650562.065	6.934	0.004	0.009	-0.006	1001_TX2G	2865241.142	650562.054	6.93	-0.007	0.002	0.01
0826_TX3A	2865205.251	650637.01	12.449	0909_TX3A	2865205.266	650637.001	12.446	0.015	-0.009	-0.003	1001_TX3A	2865205.267	650637.002	12.453	-0.016	0.008	-0.004
0826_TX3B	2865203.395	650633.728	12.552	0909_TX3B	2865203.382	650633.724	12.548	-0.013	-0.004	-0.004	1001_TX3B	2865203.391	650633.71	12.54	0.004	0.018	0.012
0826_TX3C	2865201.991	650630.42	12.33	0909_TX3C	2865201.977	650630.41	12.327	-0.014	-0.01	-0.003	1001_TX3C	2865201.985	650630.406	12.312	0.006	0.014	0.018
0826_TX3D	2865198.763	650624.023	10.167	0909_TX3D	2865198.755	650624.01	10.166	-0.008	-0.013	-0.001	1001_TX3D	2865199.161	650623.856	10.213	-0.398	0.167	-0.046
0826_TX3E	2865194.472	650618.965	9.516	0909_TX3E	2865194.467	650618.951	9.51	-0.005	-0.014	-0.006	1001_TX3E	2865194.494	650618.932	9.496	-0.022	0.033	0.02
0826_TX3F	2865189.357	650610.457	9.295	0909_TX3F	2865189.334	650610.452	9.289	-0.023	-0.005	-0.006	1001_TX3F	2865189.329	650610.437	9.283	0.028	0.02	0.012
0826_TX3G	2865183.94	650602.198	8.17	0909_TX3G	2865183.931	650602.192	8.168	-0.009	-0.006	-0.002	1001_TX3G	2865184.093	650601.046	8.046	-0.153	1.152	0.124
0826_TX3H	2865179.074	650594.662	7.154	0909_TX3H	2865179.089	650594.644	7.152	0.015	-0.018	-0.002	1001_TX3H	2865179.077	650594.644	7.145	-0.003	0.018	0.009

				GPS						DELTAS									
0826_GX1A	2865310.959	650582.353	12.554		0909_GX1A	2865310.972	650582.344	12.558	0.013	-0.009	0.004		1001_GX1A	2865310.964	650582.349	12.528	-0.005	0.004	0.026
0826_GX1B	2865308.411	650577.398	12.203		0909_GX1B	2865308.442	650577.392	12.196	0.031	-0.006	-0.007		1001_GX1B	2865308.405	650577.404	12.186	0.006	-0.006	0.017
0826_GX1C	2865305.83	650571.909	10.642		0909_GX1C	2865305.837	650571.888	10.615	0.007	-0.021	-0.027		1001_GX1C	2865305.814	650571.903	10.624	0.016	0.006	0.018
0826_GX1D	2865304.159	650567.517	9.221		0909_GX1D	2865304.179	650567.512	9.214	0.02	-0.005	-0.007		1001_GX1D	2865304.14	650567.504	9.184	0.019	0.013	0.037
0826_GX1E	2865299.195	650558.985	8.612		0909_GX1E	2865299.196	650558.973	8.599	0.001	-0.012	-0.013		1001_GX1E	2865299.188	650558.984	8.567	0.007	0.001	0.045
0826_GX1F	2865297.16	650553.639	8.46		0909_GX1F	2865297.172	650553.64	8.447	0.012	0.001	-0.013		1001_GX1F	2865297.157	650553.649	8.427	0.003	-0.01	0.033
0826_GX1G	2865292.965	650545.421	7.783		0909_GX1G	2865292.965	650545.403	7.772	0	-0.018	-0.011		1001_GX1G	2865292.967	650545.423	7.763	-0.002	-0.002	0.02
0826_GX2A	2865261.452	650607.87	12.015		0909_GX2A	2865261.466	650607.848	12.035	0.014	-0.022	0.02		1001_GX2A	2865261.503	650607.885	11.951	-0.051	-0.015	0.064
0826_GX2B	2865259.749	650604.459	12.343		0909_GX2B	2865259.754	650604.492	12.371	0.005	0.033	0.028		1001_GX2B	2865259.746	650604.461	12.309	0.003	-0.002	0.034
0826_GX2C	2865256.198	650596.33	10.3		0909_GX2C	2865256.22	650596.319	10.275	0.022	-0.011	-0.025		1001_GX2C	2865256.199	650596.332	10.281	-0.001	-0.002	0.019
0826_GX2D	2865254.095	650590.774	9.414		0909_GX2D	2865254.111	650590.765	9.396	0.016	-0.009	-0.018		1001_GX2D	2865254.074	650590.768	9.387	0.021	0.006	0.027
0826_GX2E	2865249.192	650579.553	9.215		0909_GX2E	2865249.215	650579.549	9.19	0.023	-0.004	-0.025		1001_GX2E	2865249.197	650579.565	9.175	-0.005	-0.012	0.04
0826_GX2F	2865247.043	650574.507	8.895		0909_GX2F	2865247.077	650574.498	8.896	0.034	-0.009	0.001		1001_GX2F	2865247.053	650574.509	8.875	-0.01	-0.002	0.02
0826_GX2G	2865241.135	650562.08	6.927		0909_GX2G	2865241.152	650562.082	6.93	0.017	0.002	0.003		1001_GX2G	2865241.137	650562.088	6.93	-0.002	-0.008	-0.003
0826_GX3A	2865205.256	650637.029	12.449		0909_GX3A	2865205.27	650637.012	12.455	0.014	-0.017	0.006		1001_GX3A	2865205.253	650637.025	12.439	0.003	0.004	0.01
0826_GX3B	2865203.39	650633.747	12.545		0909_GX3B	2865203.387	650633.732	12.541	-0.003	-0.015	-0.004		1001_GX3B	2865203.375	650633.736	12.524	0.015	0.011	0.021
0826_GX3C	2865201.961	650630.433	12.318		0909_GX3C	2865201.984	650630.421	12.324	0.023	-0.012	0.006		1001_GX3C	2865201.971	650630.423	12.294	-0.01	0.01	0.024
0826_GX3D	2865198.737	650624.014	10.162		0909_GX3D	2865198.76	650624.017	10.162	0.023	0.003	0		1001_GX3D	2865199.151	650623.877	10.218	-0.414	0.137	-0.056
0826_GX3E	2865194.454	650618.976	9.516		0909_GX3E	2865194.471	650618.953	9.507	0.017	-0.023	-0.009		1001_GX3E	2865194.485	650618.952	9.477	-0.031	0.024	0.039
0826_GX3F	2865189.321	650610.473	9.287		0909_GX3F	2865189.343	650610.46	9.291	0.022	-0.013	0.004		1001_GX3F	2865189.321	650610.461	9.254	0	0.012	0.033
0826_GX3G	2865183.917	650602.202	8.164		0909_GX3G	2865183.944	650602.208	8.174	0.027	0.006	0.01		1001_GX3G	2865184.08	650601.065	8.018	-0.163	1.137	0.146
0826_GX3H	2865179.072	650594.659	7.153		0909_GX3H	2865179.096	650594.656	7.156	0.024	-0.003	0.003		1001_GX3H	2865179.064	650594.67	7.15	0.008	-0.011	0.003

Appendix I: Lab Data

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

January 20, 2015
TEAM Project No. 142086
Report No. 1

U.S. Army Corps of Engineers
Building 3396, Office 1103
3909 Halls Ferry Road
Vicksburg, MS, 39180

Attn: Mr. Lucas Walshire, P.E.
Re: Laboratory Testing Services
IBWC: Brownsville Levee
BPA Number W9126G-14-A-0032-0002

Dear Mr. Walshire:

Submitted here is our report of laboratory testing services completed on soil samples received at our materials testing laboratory in Arlington, Texas, September 16 and October 16, 2014, for the above referenced project. The laboratory test program authorized December 22, 2014 was completed utilizing the following test methodologies:

Atterberg Limits	ASTM D-4318
Grain Size Analysis	ASTM D-422
Classification of Soils	ASTM D-2487
Moisture Content	ASTM D-2216
Controlled Expansion Consolidation	USACE EM 1110-2-1906, Appendix VIII
Direct Shear Test	USACE EM 1110-2-1906, Appendix IX
Unconsolidated-Undrained Triaxial	ASTM D-2850

We appreciate the opportunity to be of assistance to you with this project. Should you have any questions, or if we may be of further assistance, please call the undersigned at (817) 467-5500.

Very truly yours,
TEAM Consultants, Inc.

James Hutt
Vice President

Edward Gomez, P.E.
Project Engineer

JH/EG/ms

Attachments:
Summary of Laboratory Test Results
Compact Disk of Laboratory Test Results

SUMMARY OF LABORATORY TEST RESULTS

LABORATORY TESTING SERVICES

Brownsville Levee Geotechnical Investigation

Boring No.	Sample No.	Sample Depth (ft.)	Visual Description & Unified Soil Classification (ASTM D-2487 & D-2488)		Percent Passing Sieve #4	#10	#20	#40	#60	#80	#100	#200
BRN-P3-32b	--	4.7-6.7	Grayish brown lean clay	CL	100	99.8	99.3	99.0	98.8	98.3	97.7	90.1
	--	6.9-8.9	Brown lean clay	CL	---	---	---	---	---	---	---	96.2
	--	9.1-11.1	Brown lean clay	CL	99.5	99.4	99.3	99.1	98.9	98.8	98.7	96.6
	--	11.3-13.3	Brown lean clay	CL	---	---	---	---	---	---	---	98.7
	--	13.5-15.5	Brown lean clay	CL	100	99.6	99.5	99.3	99.2	99.2	99.1	98.5
	--	15.7-17.7	Brown fat clay	CH	---	---	---	---	---	---	---	99.5
	--	17.9-19.9	Grayish brown fat clay	CH	100	99.9	99.8	99.6	99.5	99.3	99.2	97.9
	--	20.1-22.1	Dark brown fat clay	CH	100	99.9	99.8	99.7	99.7	99.6	99.5	98.3
	--	22.3-24.3	Grayish brown lean clay	CL	100	100	100	99.9	99.9	99.8	99.8	98.3
	--	24.5-26.5	Dark brown lean clay	CL	---	---	---	---	---	---	---	---
	--	26.7-28.7	Dark brown lean clay	CL	99.3	98.9	98.6	98.2	97.8	96.9	96.3	87.9
BRN-P3-31	3	3.0-4.5	Light brown lean clay with sand	CL	100	100	99.9	99.8	99.6	98.7	97.5	78.2
	5	6.0-7.5	Light brown sandy silty clay	CL-ML	99.8	99.7	99.6	99.5	99.1	97.5	94.5	67.8
	8	10.5-12.0	Brown sandy silty clay	CL-ML	99.8	99.6	99.2	99.0	98.3	95.9	91.7	64.1
	9	12.0-13.5	Brown lean clay	CL	100	100	99.6	99.2	98.7	98.2	97.9	93.2
	10	13.5-15.0	Brown silt	ML	100	99.9	99.7	99.6	99.2	98.6	98.1	85.8
	12	16.5-18.0	Brown lean clay	CL	100	100	99.9	99.9	99.7	99.4	99.3	95.9
	13	18.0-19.5	Brown lean clay	CL	100	99.9	99.7	99.4	99.1	98.7	98.6	95.3
	14	19.5-21.0	Brown lean clay	CL	100	100	100	99.9	99.9	99.8	99.8	99.7
	16	22.5-24.0	Brown lean clay	CL	100	100	99.8	99.8	99.7	99.7	99.6	98.7
	21	45.0-46.5	Light brown lean clay	CL	100	100	99.9	99.8	99.7	99.4	99.2	87.8
	24	60.0-61.5	Light brown silty sand	SM	100	100	100	99.6	99.3	97.7	95.2	48.0
BRN-P3-32	--	29.0-31.0	Dark brown lean clay	CL	---	---	---	---	---	---	---	---
	--	31.2-33.2	Brown fat clay	CH	---	---	---	---	---	---	---	---
	--	33.4-35.4	Brown fat clay	CH	100	100	99.9	99.8	99.7	99.5	99.3	97.0
	--	35.6-37.6	Brown fat clay	CH	---	---	---	---	---	---	---	---
	7	45.0-46.5	Light brown fat clay	CH	100	100	99.9	99.8	99.8	99.7	99.7	99.0
	10	54.0-55.5	Light brown lean clay	CL	100	99.2	98.7	98.6	98.6	98.5	98.4	96.8
	13	63.0-64.5	Light brown lean clay with sand	CL	100	99.9	99.5	99.4	99.3	99.0	98.6	84.6
	14	66.0-67.5	Light brown sandy silt	ML	100	100	100	99.9	99.4	98.2	96.9	59.2
	15	69.0-70.0	Light brown silty sand	SM	100	100	99.9	99.9	99.0	93.1	86.1	35.7
	16	72.0-73.5	Light brown silty sand	SM	99.8	99.6	99.4	99.1	98.3	93.9	87.2	22.7

SUMMARY OF LABORATORY TEST RESULTS

LABORATORY TESTING SERVICES

Brownsville Levee Geotechnical Investigation

Boring No.	Sample No.	Sample Depth (ft.)	Visual Description & Unified Soil Classification (ASTM D-2487 & D-2488)		Percent Passing Sieve #4	#10	#20	#40	#60	#80	#100	#200
BRN-P3-33	--	2.0-4.0	Brown lean clay	CL	---	---	---	---	---	---	---	---
	--	6.4-8.4	Brown lean clay	CL	100	100	100	100	99.9	99.9	99.8	99.5
	--	10.8-12.8	Brown silty clay with sand	CL-ML	100	100	100	99.9	99.5	98.6	97.6	84.0
	--	13.0-15.0	Grayish brown lean clay	CL	100	100	100	100	99.9	99.6	98.9	90.4
	6	17.2-18.8	Brown silty clay with sand	CL-ML	99.9	99.8	99.8	99.6	98.4	96.1	93.3	75.6
	7	18.8-20.3	Brown silt with sand	CL-ML	97.8	97.0	96.2	95.8	95.3	94.4	93.5	80.6
	8	20.3-21.8	Brown silty clay with sand	CL-ML	99.2	98.0	97.3	97.0	96.5	95.2	93.3	78.7
	9	21.8-23.3	Brown silty sand	SM	98.7	98.7	98.6	98.5	98.2	95.2	88.9	49.0
	10	23.3-24.8	Brown sandy silt	ML	100	100	100	100	99.5	96.6	92.0	61.1
	12	26.3-27.8	Brown fat clay	CH	99.0	98.6	98.4	98.4	98.4	98.4	98.3	98.2
	13	27.8-29.3	Brown lean clay	CL	100	99.8	99.6	99.5	99.3	98.6	97.8	91.3
	15	30.8-32.3	Brown silt	ML	100	100	99.8	99.5	99.2	98.9	98.6	95.3
	16	32.3-33.8	Brown silt	ML	100	99.9	99.8	99.5	98.8	98.2	97.6	88.2
	20	41.3-42.8	Brown & tan lean clay	CL	100	100	99.7	99.5	99.5	99.4	99.2	97.6
	22	47.3-48.8	Light brown lean clay with sand	CL	100	100	99.9	99.6	99.1	98.3	97.6	78.2
	25	56.3-57.8	Light brown lean clay	CL	100	100	100	100	99.9	99.8	99.8	98.3
	28	65.3-66.8	Light brown fat clay	CH	100	99.8	99.7	99.7	99.6	99.6	99.5	98.9
BRN-P3-34W	3	3.0-4.5	Brown silty sand	SM	99.9	99.6	99.4	99.4	98.1	93.1	83.9	42.8
	7	9.5-10.5	Tan clayey gravel with sand	GC	61.1	53.9	48.2	44.6	42.5	40.9	39.6	34.1
	8	10.5-12.0	Brown silty sand with gravel	SM	74.5	68.3	64.8	62.7	60.1	51.8	45.3	24.3
	9	12.0-13.5	Brown sandy, silty clay	CL-ML	99.0	98.6	98.2	97.9	97.1	95.3	93.2	69.3
	10	13.5-15.0	Brown lean clay	CL	100	100	99.7	99.1	99.0	98.9	98.8	97.7
	11	15.0-16.5	Brown lean clay	CL	100	99.6	99.4	99.2	98.7	98.0	97.4	88.8
	12	16.5-18.0	Brown lean clay	CL	100	99.8	99.7	99.6	99.2	98.7	98.3	90.4
	13	18.0-19.5	Brown lean clay	CL	100	99.9	99.7	99.6	99.3	98.7	98.2	90.3
	14	19.5-21.0	Brown lean clay	CL	100	99.7	99.6	99.6	99.5	99.4	97.4	98.9
	15	21.0-22.5	Brown lean clay with sand	CL	99.9	99.6	99.4	99.3	99.3	99.2	99.2	81.2
	17	24.0-25.5	Brown sandy, silty clay	CL-ML	100	99.9	99.9	99.8	99.5	98.3	96.1	59.4
	19	30.0-31.5	Light brown & brown lean clay	CL	100	99.9	99.7	99.6	99.2	98.5	97.9	90.8
	20	33.0-34.5	Light brown fat clay	CH	100	100	99.8	99.8	99.8	99.7	99.6	99.0
	23	42.0-43.5	Light brown lean clay	CL	100	99.9	99.8	99.8	99.8	99.8	99.8	98.6
	27	54.0-55.5	Light brown lean clay	CL	100	100	100	100	99.9	99.8	99.8	99.5
	29	58.5-60.0	Light brown fat clay	CH	100	100	100	100	100	99.9	99.6	98.7

SUMMARY OF LABORATORY TEST RESULTS

LABORATORY TESTING SERVICES
Brownsville Levee Geotechnical Investigation

Boring No.	Sample No.	Sample Depth (ft.)	Visual Description & Unified Soil Classification (ASTM D-2487 & D-2488)		Percent Passing Sieve #4	#10	#20	#40	#60	#80	#100	#200
BRN-P3-35W	3	3.0-4.5	Light brown lean clay	CL	100.0	100.0	99.9	99.9	99.6	99.0	98.2	91.4
	6	7.5-9.0	Brown silty clay with sand	CL-ML	100.0	99.7	99.3	98.9	98.2	97.3	96.3	84.1
	9	12.0-13.5	Brown lean clay	CL	100.0	100.0	99.8	99.6	99.1	98.5	98.2	92.3
	11	15.0-16.5	Brown lean clay	CL	100.0	99.9	99.8	99.6	98.7	97.7	96.9	85.9
	14	19.5-21.0	Brown silty clay with sand	CL-ML	100.0	100.0	99.9	99.5	98.6	96.9	94.8	77.4
	18	25.5-27.0	Brown lean clay	CL	100.0	100.0	99.9	99.9	99.9	99.9	99.9	99.0
	20	28.5-30.0	Brown silty clay	CL-ML	100.0	99.9	99.9	99.7	99.6	99.4	99.4	97.7
	22	31.5-33.0	Brown lean clay	CL	100.0	100.0	100.0	99.8	99.6	99.5	99.5	97.6
	24	34.5-36.0	Brown lean clay	CL	100.0	99.7	99.4	99.3	99.2	99.1	99.1	97.9
	25	37.5-39.0	Brown lean clay	CL	100.0	100.0	99.9	99.8	99.1	98.4	97.7	85.8
	28	43.5-45.0	Light brown lean clay	CL	100.0	100.0	100.0	100.0	99.8	99.7	99.6	98.1
	29	46.5-48.0	Light brown lean clay	CL	100.0	100.0	100.0	100.0	100.0	100.0	99.9	99.5
	30	49.5-51.0	Light brown lean clay	CL	100.0	100.0	99.9	99.7	99.5	99.2	99.0	95.5
	31	52.5-54.0	Light brown lean clay	CL	100.0	100.0	100.0	100.0	99.8	99.5	99.3	94.1
	32	55.5-57.0	Light brown silty clay with sand	CL-ML	100.0	100.0	100.0	99.9	99.5	98.9	98.3	83.5
	33	58.5-60.0	Light brown lean clay	CL	100.0	100.0	100.0	99.9	99.9	99.8	99.7	99.3
	34	61.5-63.0	Light brown silty, clayey sand	SC-SM	100.0	99.9	99.8	99.7	99.4	97.0	89.3	37.8
	35	64.5-66.0	Light brown fat clay	CH	100.0	100.0	100.0	100.0	99.8	99.7	99.7	98.9
	37	70.0-71.5	Light brown lean clay	CL	100.0	100.0	99.9	99.8	99.8	99.7	99.6	91.4
BRN-P3-36	4	4.5-6.0	Brown lean clay with sand	CL	100.0	99.9	99.8	99.6	98.8	97.3	94.4	73.5
	6	6.65-7.5	Brown sandy lean clay	CL	100.0	99.8	99.7	99.7	99.5	98.1	93.0	63.4
	9	12.0-12.75	Brown sandy lean clay	CL	93.7	89.6	87.3	85.7	84.4	81.4	77.8	55.2
	11	13.5-15.0	Brown sandy lean clay	CL	88.8	83.2	77.4	74.1	72.3	70.8	69.3	53.5
	12	15.0-16.5	Brown silty sand	SM	100.0	99.8	99.6	99.4	99.3	98.7	94.8	47.0
	13	16.5-18.0	Brown silty sand with gravel	SM	81.1	79.1	78.0	77.0	76.3	75.6	74.2	37.0
	14	18.0-19.5	Brown silty sand	SM	99.4	98.9	98.3	98.1	97.7	90.1	79.0	44.1
	15	19.5-21.0	Brown silty sand	SM	98.9	98.1	96.6	96.3	95.7	84.0	66.8	21.1
	16	21.0-22.5	Brown lean clay	CL	100.0	100.0	99.9	99.9	99.9	99.6	99.2	91.9
	17	22.5-24.0	Brown silty clay with sand	CL-ML	100.0	100.0	100.0	100.0	100.0	99.4	98.5	84.3
	18	22.5-24.0	Brown lean clay	CL	100.0	100.0	100.0	100.0	100.0	99.8	99.5	95.7
	19	24.0-25.0	Brown lean clay	CL	100.0	100.0	100.0	100.0	100.0	99.8	99.5	93.3
	20	25.0-25.5	Brown lean clay	CL	100.0	100.0	100.0	100.0	100.0	100.0	99.9	98.7
	21	30.0-31.5	Light brown fat clay	CH	99.9	99.8	99.7	99.6	99.6	99.5	99.4	96.3
	23	40.0-41.5	Light brown lean clay	CL	100.0	100.0	100.0	100.0	100.0	100.0	99.9	97.6
	25	45.0-46.5	Light brown silty clay	CL-ML	100.0	100.0	100.0	100.0	99.6	99.2	98.8	85.3
	29	"Last"	Light brown silty sand	SM	100.0	100.0	100.0	99.9	99.2	95.9	89.8	32.9

SUMMARY OF LABORATORY TEST RESULTS

LABORATORY TESTING SERVICES

Brownsville Levee Geotechnical Investigation

Boring No.	Sample No.	Sample Depth (ft.)	Visual Description & Unified Soil Classification (ASTM D-2487 & D-2488)		Moisture Content (%)	Unit Dry Weight (pcf)	Atterberg Limits LL	PL	PI	Remarks
BRN-P3-32b	--	4.7-6.7	Grayish brown lean clay	CL	22.4	102.9	48	19	29	(2)
	--	6.9-8.9	Brown lean clay	CL	18.9	103.0	44	21	23	(1)
	--	9.1-11.1	Brown lean clay	CL	18.4	108.7	47	19	28	
	--	11.3-13.3	Brown lean clay	CL	22.1	100.5	37	20	17	(1)
	--	13.5-15.5	Brown lean clay	CL	24.7	99.6	49	22	27	
	--	15.7-17.7	Brown fat clay	CH	27.2	94.4	56	24	32	(1)
	--	17.9-19.9	Grayish brown fat clay	CH	28.9	91.4	57	24	33	(2)
	--	20.1-22.1	Dark brown fat clay	CH	24.9	---	50	21	29	
	--	22.3-24.3	Grayish brown lean clay	CL	28.1	93.1	47	21	26	(2)
	--	24.5-26.5	Dark brown lean clay	CL	31.0	91.2	44	21	23	
	--	26.7-28.7	Dark brown lean clay	CL	26.2	97.4	39	21	18	
BRN-P3-31	3	3.0-4.5	Light brown lean clay with sand	CL	8.6	---	26	18	8	
	5	6.0-7.5	Light brown sandy silty clay	CL-ML	10.8	---	24	19	5	
	8	10.5-12.0	Brown sandy silty clay	CL-ML	29.1	---	26	20	6	
	9	12.0-13.5	Brown lean clay	CL	33.1	---	30	22	8	
	10	13.5-15.0	Brown silt	ML	34.2	---	Non-Plastic			
	12	16.5-18.0	Brown lean clay	CL	31.6	---	30	22	8	
	13	18.0-19.5	Brown lean clay	CL	32.5	---	36	23	13	
	14	19.5-21.0	Brown lean clay	CL	32.8	---	41	22	19	
	16	22.5-24.0	Brown lean clay	CL	30.2	---	41	22	19	
	21	45.0-46.5	Light brown lean clay	CL	25.2	---	28	18	10	
	24	60.0-61.5	Light brown silty sand	SM	25.6	---	Non-Plastic			
BRN-P3-32	--	29.0-31.0	Dark brown lean clay	CL	27.8	95.7	40	19	21	
	--	31.2-33.2	Brown fat clay	CH	29.2	95.5	69	25	44	
	--	33.4-35.4	Brown fat clay	CH	26.6	---	55	22	33	
	--	35.6-37.6	Brown fat clay	CH	25.7	98.3	55	22	33	
	7	45.0-46.5	Light brown fat clay	CH	28.0	---	71	24	47	
	10	54.0-55.5	Light brown lean clay	CL	21.4	---	41	18	23	
	13	63.0-64.5	Light brown lean clay with sand	CL	26.3	---	36	16	20	
	14	66.0-67.5	Light brown sandy silt	ML	26.2	---	--	--	--	
	15	69.0-70.0	Light brown silty sand	SM	25.2	---	--	--	--	
	16	72.0-73.5	Light brown silty sand	SM	25.7	---	--	--	--	
		Notes:	1) See attached lab data sheets for report of Consolidation Test							
			2) See attached lab data sheets for report of Direct Shear Test							
			3) See attached graphical presentation of Hydrometer analysis.							

SUMMARY OF LABORATORY TEST RESULTS

LABORATORY TESTING SERVICES
Brownsville Levee Geotechnical Investigation

Boring No.	Sample No.	Sample Depth (ft.)	Visual Description & Unified Soil Classification (ASTM D-2487 & D-2488)		Moisture Content (%)	Unit Dry Weight (pcf)	Atterberg Limits LL	PL	PI	Remarks
BRN-P3-33	--	2.0-4.0	Brown lean clay	CL	22.2	98.3	45	21	24	(1)
	--	6.4-8.4	Brown lean clay	CL	24.7	97.9	36	22	14	(2)
	--	10.8-12.8	Brown silty clay with sand	CL-ML	26.6	96.2	27	22	5	
	--	13.0-15.0	Grayish brown lean clay	CL	29.7	90.1	33	22	11	(2)(3)
	6	17.2-18.8	Brown silty clay with sand	CL-ML	31.8	---	25	21	4	
	7	18.8-20.3	Brown silt with sand	CL-ML	29.7	---	29	22	7	
	8	20.3-21.8	Brown silty clay with sand	CL-ML	31.4	---	27	21	6	
	9	21.8-23.3	Brown silty sand	SM	29.4	---	Non-Plastic			
	10	23.3-24.8	Brown sandy silt	ML	25.7	---	Non-Plastic			
	12	26.3-27.8	Brown fat clay	CH	31.6	---	63	26	37	
	13	27.8-29.3	Brown lean clay	CL	37.4	---	49	20	29	
	15	30.8-32.3	Brown silt	ML	28.9	---	--	--	--	
	16	32.3-33.8	Brown silt	ML	29.0	---	--	--	--	
	20	41.3-42.8	Brown & tan lean clay	CL	26.9	---	49	22	27	
	22	47.3-48.8	Light brown lean clay with sand	CL	22.4	---	31	18	13	
	25	56.3-57.8	Light brown lean clay	CL	26.0	---	36	19	17	
	28	65.3-66.8	Light brown fat clay	CH	29.6	---	53	25	28	
BRN-P3-34W	3	3.0-4.5	Brown silty sand	SM	15.6	---	Non-Plastic			
	7	9.5-10.5	Tan clayey gravel with sand	GC	7.5	---	26	15	11	
	8	10.5-12.0	Brown silty sand with gravel	SM	18.0	---	Non-Plastic			
	9	12.0-13.5	Brown sandy, silty clay	CL-ML	38.9	---	27	20	7	
	10	13.5-15.0	Brown lean clay	CL	35.2	---	39	23	16	
	11	15.0-16.5	Brown lean clay	CL	31.0	---	31	23	8	
	12	16.5-18.0	Brown lean clay	CL	29.8	---	34	22	12	
	13	18.0-19.5	Brown lean clay	CL	29.4	---	38	21	17	
	14	19.5-21.0	Brown lean clay	CL	32.4	---	39	23	16	
	15	21.0-22.5	Brown lean clay with sand	CL	33.2	---	45	22	23	
	17	24.0-25.5	Brown sandy, silty clay	CL-ML	28.2	---	24	20	4	
	19	30.0-31.5	Light brown & brown lean clay	CL	24.3	---	33	19	14	
	20	33.0-34.5	Light brown fat clay	CH	31.5	---	75	24	51	
	23	42.0-43.5	Light brown lean clay	CL	27.4	---	33	17	16	
	27	54.0-55.5	Light brown lean clay	CL	26.4	---	41	20	21	
	29	58.5-60.0	Light brown fat clay	CH	27.1	---	65	24	41	

Notes:
1) See attached lab data sheets for report of Consolidation Test
2) See attached lab data sheets for report of Direct Shear Test
3) See attached graphical presentation of Hydrometer analysis.

SUMMARY OF LABORATORY TEST RESULTS

LABORATORY TESTING SERVICES

Brownsville Levee Geotechnical Investigation

Boring No.	Sample No.	Sample Depth (ft.)	Visual Description & Unified Soil Classification (ASTM D-2487 & D-2488)		Moisture Content (%)	Unit Dry Weight (pcf)	Atterberg Limits LL	PL	PI	Remarks
BRN-P3-35W	3	3.0-4.5	Light brown lean clay	CL	8.0	---	31	22	9	
	6	7.5-9.0	Brown silty clay with sand	CL-ML	16.9	---	27	21	6	
	9	12.0-13.5	Brown lean clay	CL	31.9	---	32	22	10	
	11	15.0-16.5	Brown lean clay	CL	30.4	---	32	21	11	
	14	19.5-21.0	Brown silty clay with sand	CL-ML	32.5	---	27	21	6	
	18	25.5-27.0	Brown lean clay	CL	30.9	---	35	24	11	
	20	28.5-30.0	Brown silty clay	CL-ML	31.3	---	30	25	5	
	22	31.5-33.0	Brown lean clay	CL	30.7	---	35	21	14	
	24	34.5-36.0	Brown lean clay	CL	30.1	---	40	24	16	
	25	37.5-39.0	Brown lean clay	CL	27.0	---	31	21	10	
	28	43.5-45.0	Light brown lean clay	CL	21.5	---	47	18	29	
	29	46.5-48.0	Light brown lean clay	CL	26.0	---	32	20	12	
	30	49.5-51.0	Light brown lean clay	CL	26.7	---	30	18	12	
	31	52.5-54.0	Light brown lean clay	CL	26.3	---	30	19	11	
	32	55.5-57.0	Light brown silty clay with sand	CL-ML	24.7	---	25	19	6	
	33	58.5-60.0	Light brown lean clay	CL	25.7	---	40	20	20	
	34	61.5-63.0	Light brown silty, clayey sand	SC-SM	26.3	---	22	17	5	
	35	64.5-66.0	Light brown fat clay	CH	26.9	---	60	24	36	
	37	70.0-71.5	Light brown lean clay	CL	27.2	---	29	19	10	
BRN-P3-36	4	4.5-6.0	Brown lean clay with sand	CL	21.4	---	35	17	18	
	6	6.65-7.5	Brown sandy lean clay	CL	19.7	---	29	18	11	
	9	12.0-12.75	Brown sandy lean clay	CL	18.9	---	27	18	9	
	11	13.5-15.0	Brown sandy lean clay	CL	26.5	---	32	22	10	
	12	15.0-16.5	Brown silty sand	SM	28.5	---	Non-Plastic			
	13	16.5-18.0	Brown silty sand with gravel	SM	23.4	---	Non-Plastic			
	14	18.0-19.5	Brown silty sand	SM	28.8	---	Non-Plastic			
	15	19.5-21.0	Brown silty sand	SM	27.5	---	Non-Plastic			
	16	21.0-22.5	Brown lean clay	CL	28.6	---	31	22	9	
	17	22.5-24.0	Brown silty clay with sand	CL-ML	26.6	---	26	20	6	
	18	22.5-24.0	Brown lean clay	CL	29.3	---	37	22	15	
	19	24.0-25.0	Brown lean clay	CL	26.1	---	31	21	10	
	20	25.0-25.5	Brown lean clay	CL	24.8	---	41	17	24	
	21	30.0-31.5	Light brown fat clay	CH	25.3	---	62	21	41	
	23	40.0-41.5	Light brown lean clay	CL	24.2	---	49	18	31	
	25	45.0-46.5	Light brown silty clay	CL-ML	28.1	---	24	19	5	
	29	"Last"	Light brown silty sand	SM	22.1	---	Non-Plastic			

SUMMARY OF LABORATORY TEST RESULTS

LABORATORY TESTING SERVICES

Brownsville Levee Geotechnical Investigation

Boring No.	Sample No.	Sample Depth (ft.)	Visual Description & Unified Soil Classification (ASTM D-2487 & D-2488)		Moisture Content (%)	Unit Dry Weight (pcf)	Confining Pressure (tsf)	Q (tsf)	Strain @ Failure (%)	Type Failure
BRN-P3-32b	--	4.7-6.7	Grayish brown lean clay	CL	22.4	--	--	--	--	
	--	6.9-8.9	Brown lean clay	CL	18.9	--	--	--	--	
	--	9.1-11.1	Brown lean clay	CL	18.4	108.7	0.63	5.16	5.5	Vertical
	--	11.3-13.3	Brown lean clay	CL	22.1	--	--	--	--	
	--	13.5-15.5	Brown lean clay	CL	24.7	99.6	0.91	2.32	7.2	60° Angular
	--	15.7-17.7	Brown fat clay	CH	27.2	--	--	--	--	
	--	17.9-19.9	Grayish brown fat clay	CH	28.9	--	--	--	--	
	--	20.1-22.1	Dark brown fat clay	CH	24.9	--	--	--	--	
	--	22.3-24.3	Grayish brown lean clay	CL	28.1	--	--	--	--	
	--	24.5-26.5	Dark brown lean clay	CL	31.0	91.2	1.59	0.82	15.0	Internal
	--	26.7-28.7	Dark brown lean clay	CL	26.2	97.4	1.73	1.32	15.0	Internal
BRN-P3-31	3	3.0-4.5	Light brown lean clay with sand	CL	8.6	--	--	--	--	
	5	6.0-7.5	Light brown sandy silty clay	CL-ML	10.8	--	--	--	--	
	8	10.5-12.0	Brown sandy silty clay	CL-ML	29.1	--	--	--	--	
	9	12.0-13.5	Brown lean clay	CL	33.1	--	--	--	--	
	10	13.5-15.0	Brown silt	ML	34.2	--	--	--	--	
	12	16.5-18.0	Brown lean clay	CL	31.6	--	--	--	--	
	13	18.0-19.5	Brown lean clay	CL	32.5	--	--	--	--	
	14	19.5-21.0	Brown lean clay	CL	32.8	--	--	--	--	
	16	22.5-24.0	Brown lean clay	CL	30.2	--	--	--	--	
	21	45.0-46.5	Light brown lean clay	CL	25.2	--	--	--	--	
	24	60.0-61.5	Light brown silty sand	SM	25.6	--	--	--	--	
BRN-P3-32	--	29.0-31.0	Dark brown lean clay	CL	27.8	95.7	1.88	1.18	15.0	Internal
	--	31.2-33.2	Brown fat clay	CH	29.2	95.5	2.01	1.69	7.2	55° Angular, Slickensided
	--	33.4-35.4	Brown fat clay	CH	26.6	--	--	--	--	
	--	35.6-37.6	Brown fat clay	CH	25.7	98.3	2.29	1.17	15.0	Internal
	7	45.0-46.5	Light brown fat clay	CH	28.0	--	--	--	--	
	10	54.0-55.5	Light brown lean clay	CL	21.4	--	--	--	--	
	13	63.0-64.5	Light brown lean clay with sand	CL	26.3	--	--	--	--	
	14	66.0-67.5	Light brown sandy silt	ML	26.2	--	--	--	--	
	15	69.0-70.0	Light brown silty sand	SM	25.2	--	--	--	--	
	16	72.0-73.5	Light brown silty sand	SM	25.7	--	--	--	--	

SUMMARY OF LABORATORY TEST RESULTS

LABORATORY TESTING SERVICES

Brownsville Levee Geotechnical Investigation

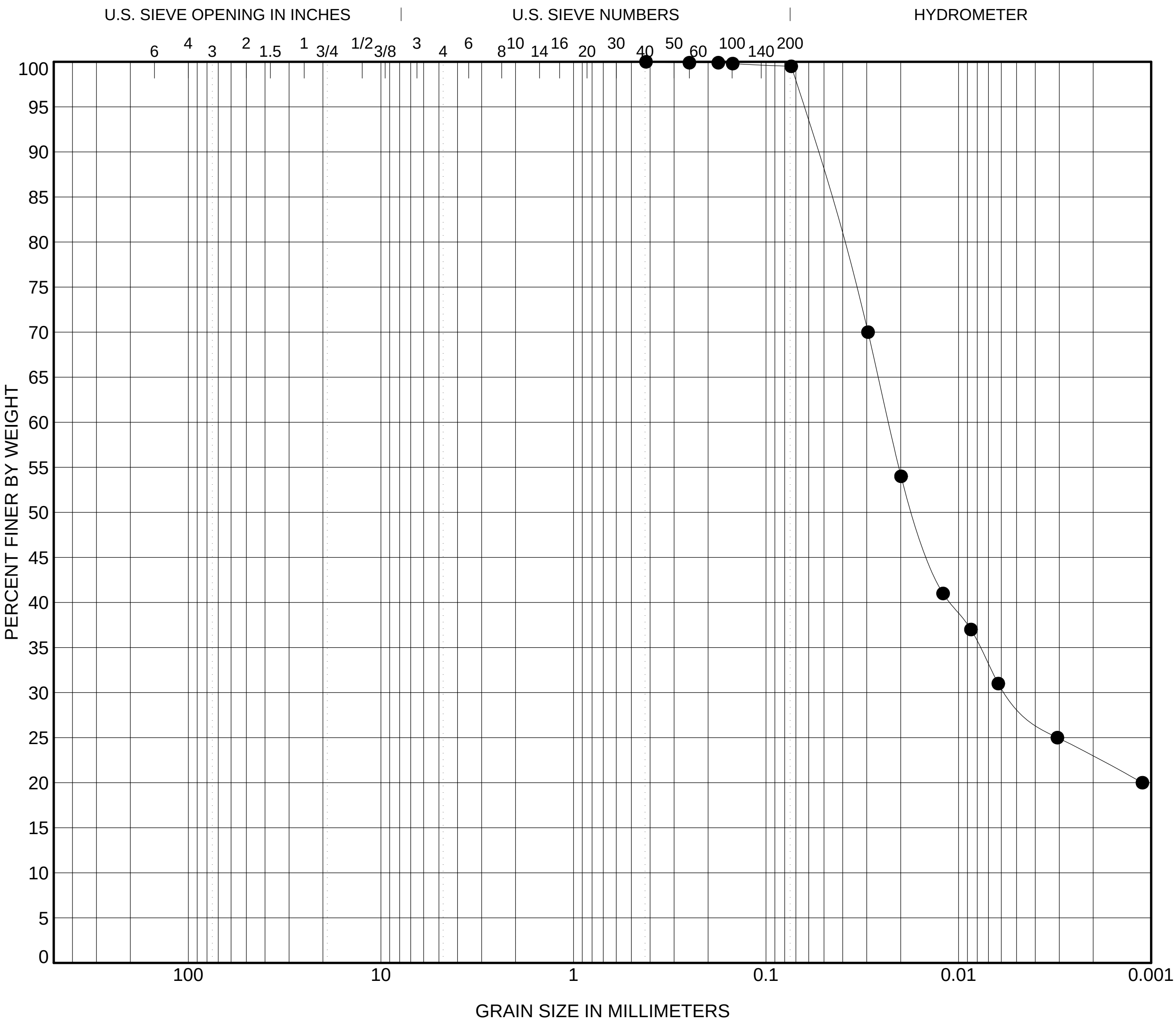
Boring No.	Sample No.	Sample Depth (ft.)	Visual Description & Unified Soil Classification (ASTM D-2487 & D-2488)		Moisture Content (%)	Unit Dry Weight (pcf)	Confining Pressure (tsf)	Q (tsf)	Strain @ Failure (%)	Type Failure
BRN-P3-33	--	2.0-4.0	Brown lean clay	CL	22.2	--	--	--	--	
	--	6.4-8.4	Brown lean clay	CL	24.7	97.9	0.46	0.91	15.0	Internal
	--	10.8-12.8	Brown silty clay with sand	CL-ML	26.6	96.2	0.74	0.54	15.0	Internal
	--	13.0-15.0	Grayish brown lean clay	CL	29.7	--	--	--	--	
	6	17.2-18.8	Brown silty clay with sand	CL-ML	31.8	--	--	--	--	
	7	18.8-20.3	Brown silt with sand	CL-ML	29.7	--	--	--	--	
	8	20.3-21.8	Brown silty clay with sand	CL-ML	31.4	--	--	--	--	
	9	21.8-23.3	Brown silty sand	SM	29.4	--	--	--	--	
	10	23.3-24.8	Brown sandy silt	ML	25.7	--	--	--	--	
	12	26.3-27.8	Brown fat clay	CH	31.6	--	--	--	--	
	13	27.8-29.3	Brown lean clay	CL	37.4	--	--	--	--	
	15	30.8-32.3	Brown silt	ML	28.9	--	--	--	--	
	16	32.3-33.8	Brown silt	ML	29.0	--	--	--	--	
	20	41.3-42.8	Brown & tan lean clay	CL	26.9	--	--	--	--	
	22	47.3-48.8	Light brown lean clay with sand	CL	22.4	--	--	--	--	
	25	56.3-57.8	Light brown lean clay	CL	26.0	--	--	--	--	
	28	65.3-66.8	Light brown fat clay	CH	29.6	--	--	--	--	
BRN-P3-34W	3	3.0-4.5	Brown silty sand	SM	15.6	--	--	--	--	
	7	9.5-10.5	Tan clayey gravel with sand	GC	7.5	--	--	--	--	
	8	10.5-12.0	Brown silty sand with gravel	SM	18.0	--	--	--	--	
	9	12.0-13.5	Brown sandy, silty clay	CL-ML	38.9	--	--	--	--	
	10	13.5-15.0	Brown lean clay	CL	35.2	--	--	--	--	
	11	15.0-16.5	Brown lean clay	CL	31.0	--	--	--	--	
	12	16.5-18.0	Brown lean clay	CL	29.8	--	--	--	--	
	13	18.0-19.5	Brown lean clay	CL	29.4	--	--	--	--	
	14	19.5-21.0	Brown lean clay	CL	32.4	--	--	--	--	
	15	21.0-22.5	Brown lean clay with sand	CL	33.2	--	--	--	--	
	17	24.0-25.5	Brown sandy, silty clay	CL-ML	28.2	--	--	--	--	
	19	30.0-31.5	Light brown & brown lean clay	CL	24.3	--	--	--	--	
	20	33.0-34.5	Light brown fat clay	CH	31.5	--	--	--	--	
	23	42.0-43.5	Light brown lean clay	CL	27.4	--	--	--	--	
	27	54.0-55.5	Light brown lean clay	CL	26.4	--	--	--	--	
	29	58.5-60.0	Light brown fat clay	CH	27.1	--	--	--	--	

SUMMARY OF LABORATORY TEST RESULTS

LABORATORY TESTING SERVICES

Brownsville Levee Geotechnical Investigation

Boring No.	Sample No.	Sample Depth (ft.)	Visual Description & Unified Soil Classification (ASTM D-2487 & D-2488)		Moisture Content (%)	Unit Dry Weight (pcf)	Confining Pressure (tsf)	Q (tsf)	Strain @ Failure (%)	Type Failure
BRN-P3-35W	3	3.0-4.5	Light brown lean clay	CL	8.0	---	--	--	--	
	6	7.5-9.0	Brown silty clay with sand	CL-ML	16.9	---	--	--	--	
	9	12.0-13.5	Brown lean clay	CL	31.9	---	--	--	--	
	11	15.0-16.5	Brown lean clay	CL	30.4	---	--	--	--	
	14	19.5-21.0	Brown silty clay with sand	CL-ML	32.5	---	--	--	--	
	18	25.5-27.0	Brown lean clay	CL	30.9	---	--	--	--	
	20	28.5-30.0	Brown silty clay	CL-ML	31.3	---	--	--	--	
	22	31.5-33.0	Brown lean clay	CL	30.7	---	--	--	--	
	24	34.5-36.0	Brown lean clay	CL	30.1	---	--	--	--	
	25	37.5-39.0	Brown lean clay	CL	27.0	---	--	--	--	
	28	43.5-45.0	Light brown lean clay	CL	21.5	---	--	--	--	
	29	46.5-48.0	Light brown lean clay	CL	26.0	---	--	--	--	
	30	49.5-51.0	Light brown lean clay	CL	26.7	---	--	--	--	
	31	52.5-54.0	Light brown lean clay	CL	26.3	---	--	--	--	
	32	55.5-57.0	Light brown silty clay with sand	CL-ML	24.7	---	--	--	--	
	33	58.5-60.0	Light brown lean clay	CL	25.7	---	--	--	--	
	34	61.5-63.0	Light brown silty, clayey sand	SC-SM	26.3	---	--	--	--	
	35	64.5-66.0	Light brown fat clay	CH	26.9	---	--	--	--	
	37	70.0-71.5	Light brown lean clay	CL	27.2	---	--	--	--	
BRN-P3-36	4	4.5-6.0	Brown lean clay with sand	CL	21.4	---	--	--	--	
	6	6.65-7.5	Brown sandy lean clay	CL	19.7	---	--	--	--	
	9	12.0-12.75	Brown sandy lean clay	CL	18.9	---	--	--	--	
	11	13.5-15.0	Brown sandy lean clay	CL	26.5	---	--	--	--	
	12	15.0-16.5	Brown silty sand	SM	28.5	---	--	--	--	
	13	16.5-18.0	Brown silty sand with gravel	SM	23.4	---	--	--	--	
	14	18.0-19.5	Brown silty sand	SM	28.8	---	--	--	--	
	15	19.5-21.0	Brown silty sand	SM	27.5	---	--	--	--	
	16	21.0-22.5	Brown lean clay	CL	28.6	---	--	--	--	
	17	22.5-24.0	Brown silty clay with sand	CL-ML	26.6	---	--	--	--	
	18	22.5-24.0	Brown lean clay	CL	29.3	---	--	--	--	
	19	24.0-25.0	Brown lean clay	CL	26.1	---	--	--	--	
	20	25.0-25.5	Brown lean clay	CL	24.8	---	--	--	--	
	21	30.0-31.5	Light brown fat clay	CH	25.3	---	--	--	--	
	23	40.0-41.5	Light brown lean clay	CL	24.2	---	--	--	--	
	25	45.0-46.5	Light brown silty clay	CL-ML	28.1	---	--	--	--	
	29	"Last"	Light brown silty sand	SM	22.1	---	--	--	--	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

							CC	CU
●								

	Specimen Identification	D90	D50	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**P3-33 6.4-8.4**	**0.055**	**0.017**	**0.006**		**0.0**	**0.5**	**70.4**	**29.1**

TEAM Consultants, Inc.
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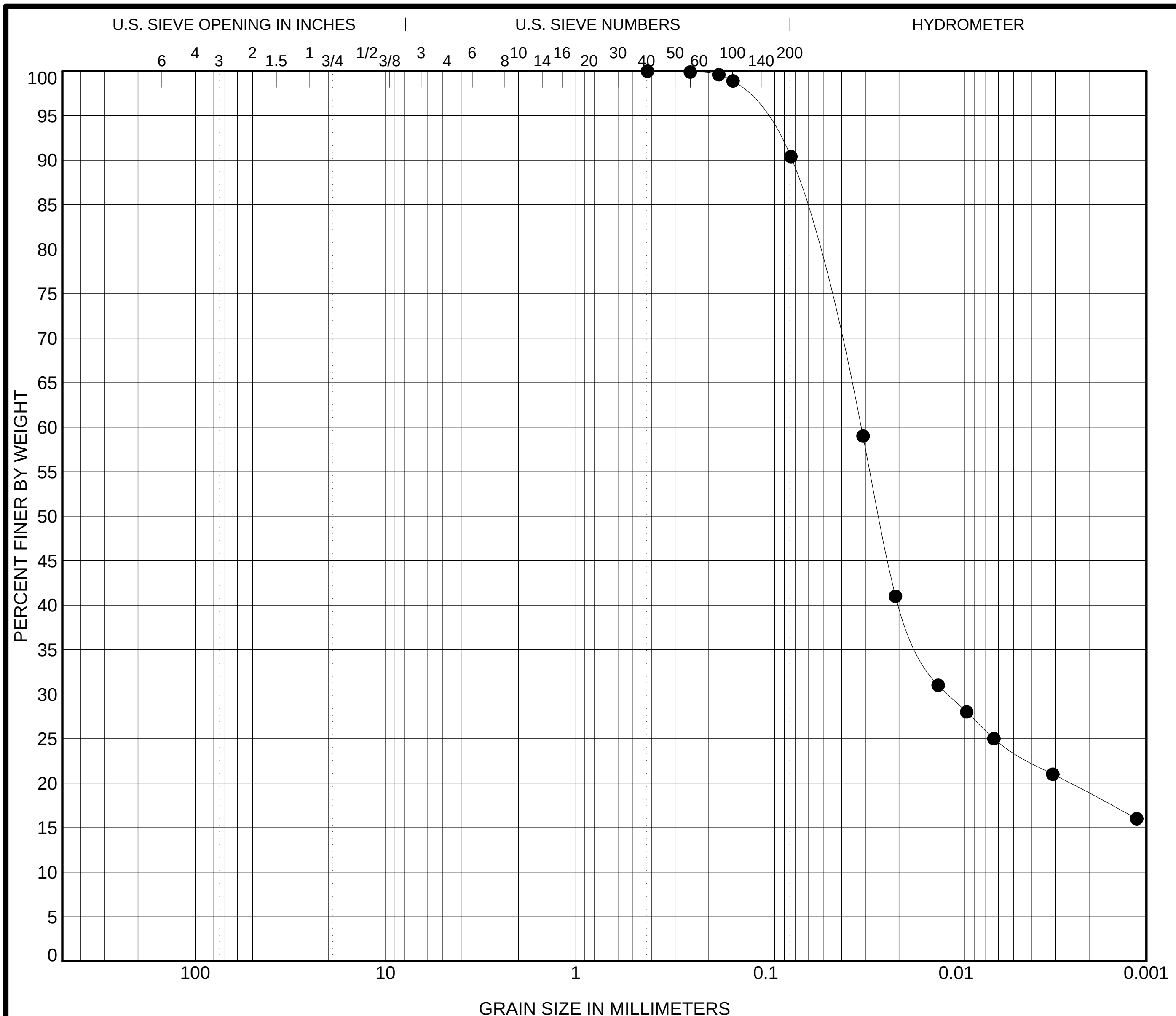
Fax: (817) 468-9920

GRAIN SIZE DISTRIBUTION

PROJECT: Brownsville Levee Subsurface Investigation

LOCATION: Brownsville, Texas

TEAM Job Number: 142086



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

						CC	CU
●							

	Specimen Identification	D90	D50	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**P3-33 13-15**	**0.073**	**0.025**	**0.011**		**0.0**	**9.6**	**66.7**	**23.7**

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GRAIN SIZE DISTRIBUTION

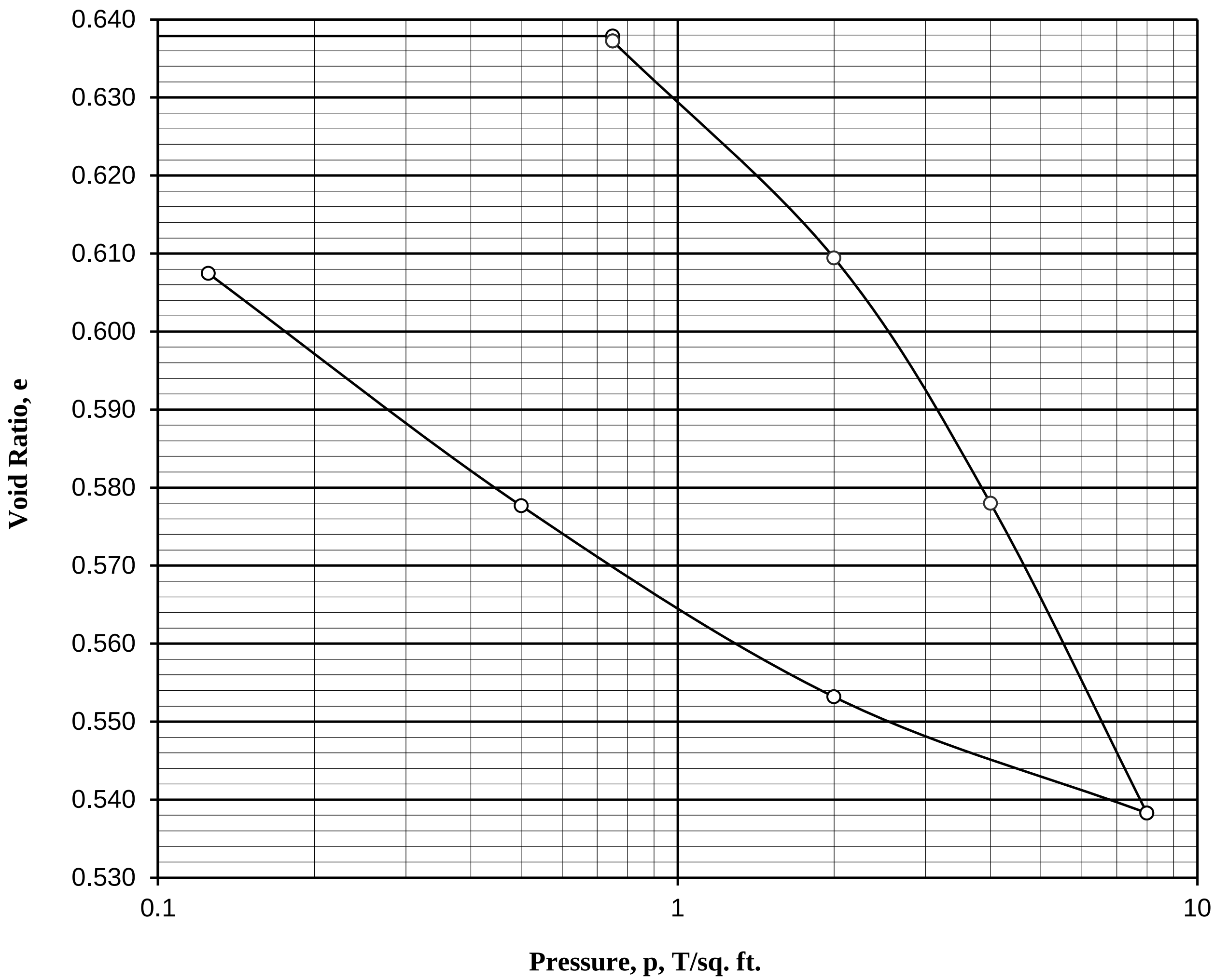
PROJECT: Brownsville Levee Subsurface Investigation

LOCATION: Brownsville, Texas

TEAM Job Number: 142086

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing



Type of specimen: Undisturbed				Before Test		After Test	
Diam. 2.50 in.		Ht. 0.495 in.		Water Content, w_o	18.90%	W_f	22.19%
Overburden Pressure, P_o			T/sq. ft.	Void Ratio, e_o	0.6379	e_f	0.6074
Preconsol. Pressure, P_c			T/sq. ft.	Saturation, S_o	80.1%	S_f	98.7%
Compression Index, C_c				Dry Density, γd	103.0 lb/ft^3		
Classification	Brown lean clay						
LL	44	G_s	2.703	Project	Brownsville Levee Repair		
PL	21				Team Project No.: 142086		
Remarks							
				Boring No:	BRN-P3-32b	Sample No.:	---
				Depth:	6.9-8.9	Date:	12/19/14
				CONSOLIDATION TEST REPORT			

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST
(Specimen Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086
Boring No.: BRN-P3-32b Sample No.: --- Depth: 6.9-8.9 Date: 12/19/14

Classification Brown lean clay

				Before Test		After Test	
				Specimen	Trimmings	Specimen	
Tare No.				Ring and Plates	460	424	
Weight in grams	Tare plus wet soil			289.94	780.9	115.62	
	Tare plus dry soil			277.52	714.4	101.04	
	Water	W_W	W_{WO}	12.42	66.45	W_{wf}	14.58
	Tare			211.81	362.7	35.33	
	Dry soil	W_S		65.71	351.66	65.71	
Water Content		w	W_O	18.90%	18.90%	W_f	22.19%

Consolidometer No.:	1	Area of specimen, A, (sq. cm.)	31.67
Weight of ring, g	N/A	Height of specimen, H, (in.)	0.495
Weight of plates, g	N/A	Specific Gravity of solids, (Gs)	2.703

Height of solids, $H_s = \frac{W_S}{A \times G_s \times \gamma_W} = \frac{65.71}{31.67 \times 2.70 \times 1 \times 2.54} = 0.3022$ in.

Original height of water, $H_{WO} = \frac{W_{WO}}{A \times \gamma_W} = \frac{12.42}{31.67 \times 1 \times 2.54} = 0.1544$ in.

Final height of water, $H_{Wf} = \frac{W_{Wf}}{A \times \gamma_W} = \frac{14.58}{31.67 \times 1 \times 2.54} = 0.1812$ in.

Net change in height of specimen at end of test, $\Delta H =$ -0.00920 in.

Height of specimen at end of test, $H_f = H - \Delta H =$ 0.4858 in.

Void ratio before test, $e_O = = \frac{H - H_S}{H_S} = \frac{0.495 - 0.3022}{0.3022} = 0.6379$

Void ratio after test, $e_f = \frac{H_f - H_S}{H_S} = \frac{0.4858 - 0.3022}{0.3022} = 0.6074$

Degree of saturation before test, $S_O = \frac{H_{WO}}{H - H_S} = \frac{0.1544}{0.4950 - 0.3022} = 80.1\%$

Degree of saturation after test, $S_f = \frac{H_{wf}}{H_f - H_S} = \frac{0.1812}{0.4858 - 0.3022} = 98.7\%$

Dry density before test, $\gamma_d = \frac{W_S}{H \times A} = \frac{65.71 \times 62.4}{0.495 \times 31.67 \times 2.54} = 103.0$ lb./cu.ft.

Remarks ______

Technician James Hutt Computed by James Hutt Checked by James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-32b Sample No.: --- Depth: 6.9-8.9 Consol.No.: 1

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C
12/19	0.25	9:40	0	2007	20	12/21	4	8:45	0	2122	
12/19	0.5	9:41	1	2013		12/21	4	8:45	0.05	2166	
12/19	0.75	9:48	8	2023		12/21	4	8:45	0.1	2171.5	
12/19	0.75	18:15	515	2020		12/21	4	8:45	0.2	2177	
12/20	0.75	9:10	1410	2021	20	12/21	4	8:45	0.33	2182	
						12/21	4	8:45	0.5	2186	
						12/21	4	8:45	0.75	2190	
						12/21	4	8:46	1	2193	
12/20	2	9:15	0	2021	20	12/21	4	8:47	2	2200	
12/20	2	9:15	0.05	2076		12/21	4	8:49	4	2208	
12/20	2	9:15	0.1	2080		12/21	4	8:53	8	2212	
12/20	2	9:15	0.2	2084		12/21	4	9:00	15	2215	
12/20	2	9:15	0.33	2087		12/21	4	9:15	30	2218.2	
12/20	2	9:15	0.5	2088.8		12/21	4	9:45	60	2221.5	
12/20	2	9:15	0.75	2091		12/21	4	10:25	100	2223.8	
12/20	2	9:16	1	2092.2		12/21	4	12:10	205	2227	
12/20	2	9:17	2	2097.2		12/21	4	14:00	315	2229	
12/20	2	9:19	4	2100.5		12/21	4	16:00	435	2230.8	
12/20	2	9:23	8	2103.5		12/22	4	8:00	1395	2235	
12/20	2	9:31	16	2106.5							
12/20	2	9:45	30	2108.8							
12/20	2	10:15	60	2111							
12/20	2	10:55	100	2113.2							
12/20	2	12:50	215	2116							
12/20	2	14:25	310	2117.2							
12/20	2	17:45	510	2119	20						
12/21	2	8:55	1420	2122							

Technician James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-32b Sample No.: --- Depth: 6.9-8.9 Consol.No.: 1

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C
12/22	8	8:25	0	2235	20						
12/22	8	8:25	0.05	2293							
12/22	8	8:25	0.1	2298							
12/22	8	8:25	0.2	2305							
12/22	8	8:25	0.33	2309							
12/22	8	8:25	0.5	2312.5							
12/22	8	8:25	0.75	2316.5							
12/22	8	8:26	1	2319							
12/22	8	8:27	2	2326							
12/22	8	8:29	4	2332							
12/22	8	8:33	8	2338.5							
12/22	8	8:41	16	2345							
12/22	8	8:55	30	2350							
12/22	8	9:25	60	2355							
12/22	8	10:07	102	2359							
12/22	8	11:45	200	2365							
12/22	8	13:25	300	2368							
12/22	8	18:00	575	2373.5							
12/23	8	6:20	1315	2377	21						

Technician James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-32b Sample No.: --- Depth: 6.9-8.9 Consol.No.: 1

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C
						REBOUND LOADS					
						12/23	2	6:20	Rebound	2377	21
						12/24	2	8:00	1540	2297	
						12/24	0.5	8:00	Rebound	2297	20
						12/25	0.5	8:40	1480	2200	
						12/25	0.125	8:40	Rebound	2200	20
						12/29	0.125	8:00	5720	2099	19
						Machine Deflection Readings					
							0.25			2007	
							0.75			2019	
							2			2036	
							4			2054	
							8			2076	
							2			2041	
							1			2018	
							0.125			2007	

Technician James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Computation of Void Ratio)

PROJECT Brownsville Levee Repair TEAM Job No.: 142086 DATE: 12/19/14

BORING NO. BRN-P3-32b SAMPLE NO. --- DEPTH 6.9-8.9 CONSOLIDOMETER NO. 1

Pressure, P T./sq.ft.	Date Increment Applied	Time in Min. Increment Effective	Dial Reading 10^{-4} in.	Correction 10^{-4} in.	Change in Height, ΔH 10^{-4} in.	Height of Voids, H_V 10^{-4} in.	Void Ratio, e
0.1	12/19	Zero Point	2000	2000	0	1928	0.6379
0.75	12/19	Initial Load	2019	2019	0	1928	0.6379
0.75	12/19	1410	2021	2019	-2	1926	0.6372
2	12/20	1420	2122	2036	-86	1842	0.6094
4	12/21	1395	2235	2054	-181	1747	0.5780
8	12/22	1315	2377	2076	-301	1627	0.5383
2	12/23	1540	2297	2041	-256	1672	0.5532
0.5	12/24	1480	2200	2018	-182	1746	0.5777
0.125	12/25	5720	2099	2007	-92	1836	0.6074

Note:

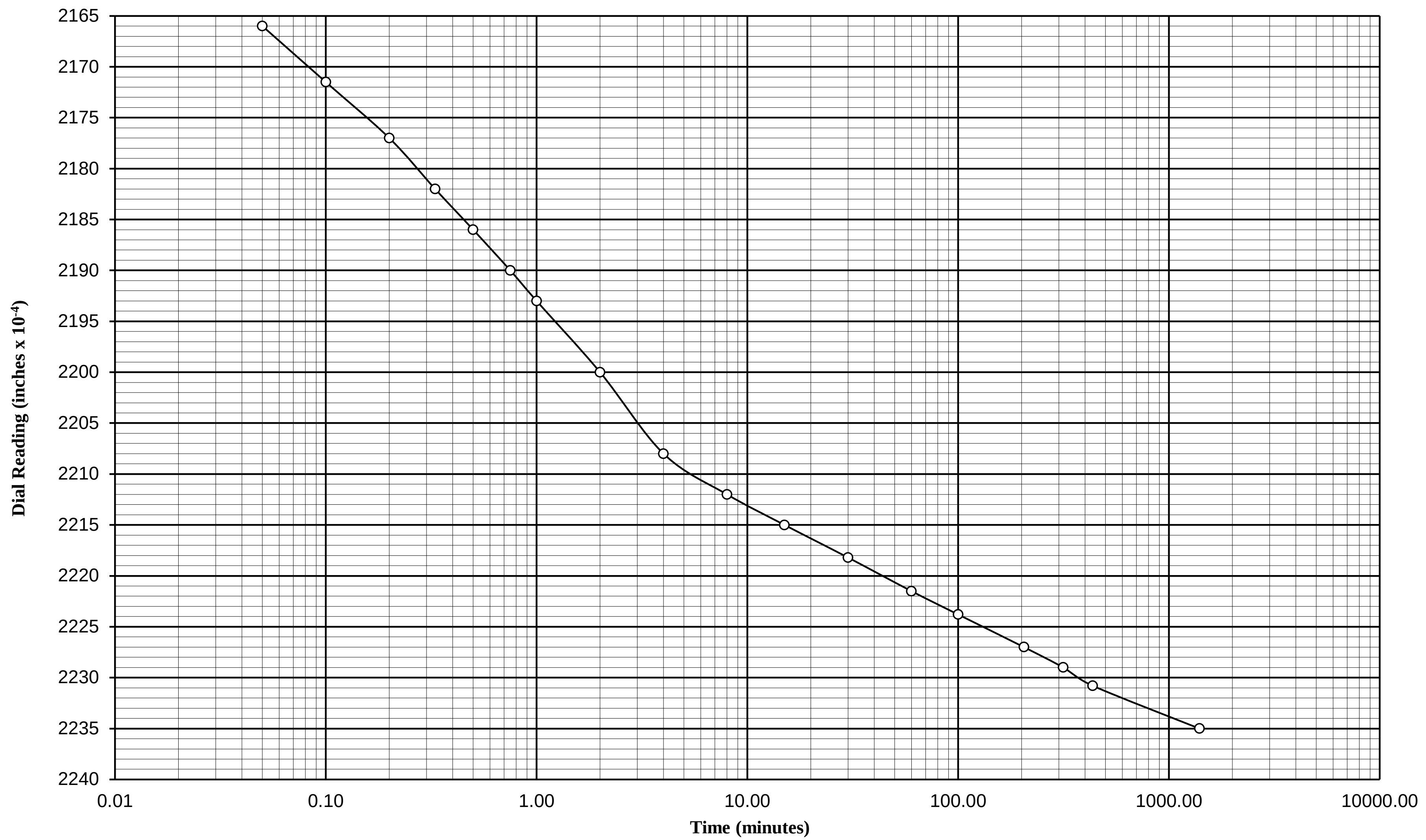
Height of voids, $H_V = (H - H_S) - \Delta H$ $H_S =$ 0.3022

Void Ratio, $e = \frac{H_V}{H_S}$

Technician James Hutt Computed by James Hutt Checked by James Hutt

CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	28.2×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.20885	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	0.45	Date:	12/19/2014
DEPTH:	6.9-8.9	Load (tsf):	2	Remarks	
SAMPLE:	---	Thickness (inches)	0.495		



CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	25.6 x 10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.21855	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	0.48	Date:	12/19/2014
DEPTH:	6.9-8.9	Load (tsf):	4	Remarks	
SAMPLE:	---	Thickness (inches)	0.495		

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CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	8.36×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.23235	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	1.4	Date:	12/19/2014
DEPTH:	6.9-8.9	Load (tsf):	8	Remarks	
SAMPLE:	---	Thickness (inches)	0.495		

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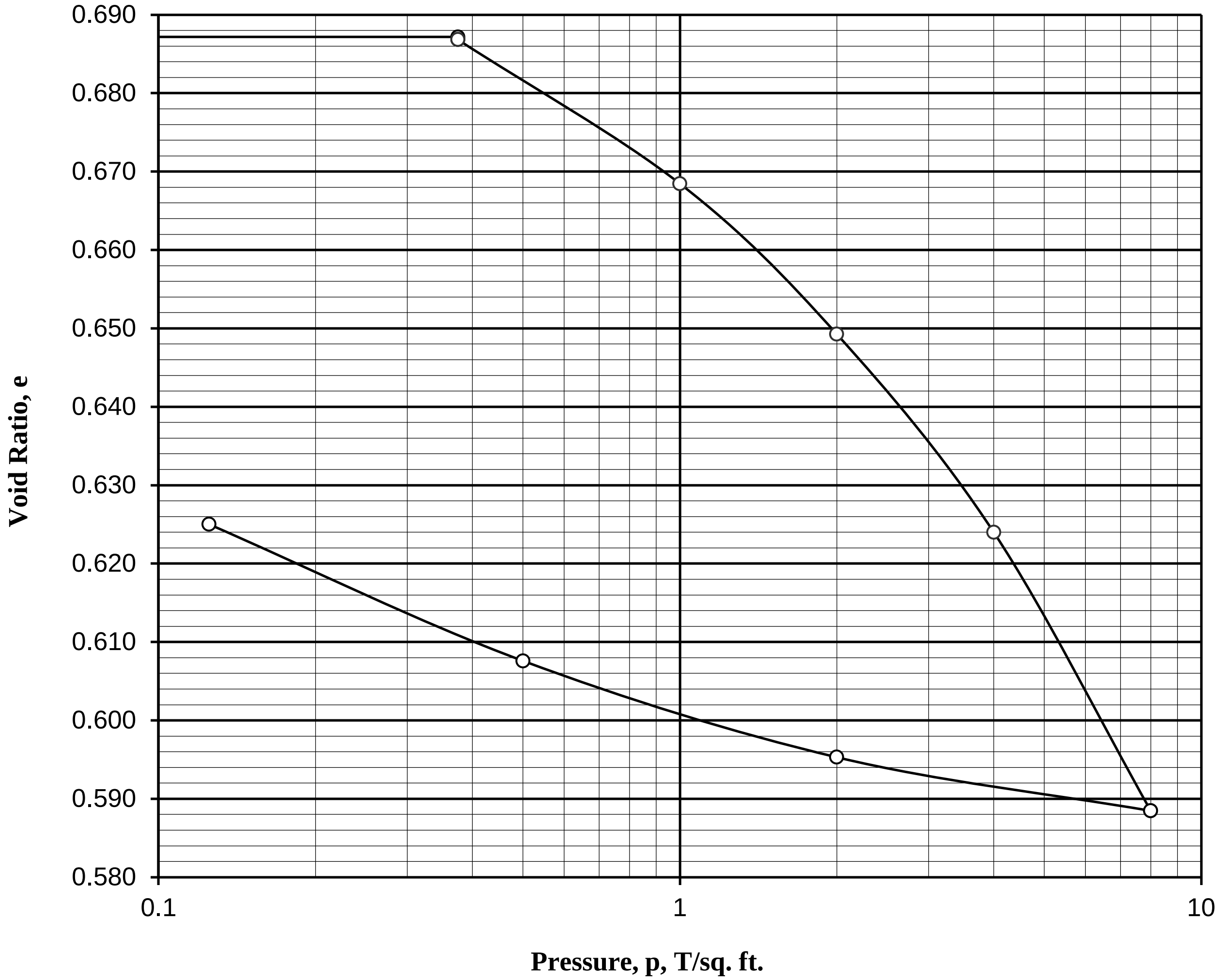
Geotechnical, Environmental, Construction Materials Testing



Type of specimen: Undisturbed	Before Test		After Test
Diam. 2.50 in. / Ht. 0.494 in.	Water Content, w_o	22.06%	W_f 22.71%
Overburden Pressure, P_o T/sq. ft.	Void Ratio, e_o	0.6872	e_f 0.6250
Preconsol. Pressure, P_c T/sq. ft.	Saturation, S_o	87.2%	S_f 98.7%
Compression Index, C_c	Dry Density, γd	100.5 lb/ft^3	
Classification Brown lean clay			
LL 37 / G_s 2.716	Project	Brownsville Levee Repair	
PL 20		Team Project No.: 142086	
Remarks			
	Boring No: BRN-P3-32b	Sample No.: ---	
	Depth: 11.3-13.3'	Date: 12/19/14	
	CONSOLIDATION TEST REPORT		

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST
(Specimen Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086
Boring No.: BRN-P3-32b Sample No.: --- Depth: 11.3-13.3' Date: 12/19/14

Classification Brown lean clay

			Before Test			After Test	
			Specimen		Trimmings	Specimen	
Tare No.			Ring and Plates		472	450	
Weight in grams	Tare plus wet soil		289.89		761.9	113.70	
	Tare plus dry soil		275.78		691.41	99.17	
	Water	W_W	W_{WO}	14.11	70.51	W_{wf}	14.53
	Tare		211.81		371.7	35.20	
	Dry soil	W_S	63.97		319.68	63.97	
Water Content		w	W_O	22.06%	22.06%	W_f	22.71%

Consolidometer No.:	2	Area of specimen, A, (sq. cm.)	31.67
Weight of ring, g	N/A	Height of specimen, H, (in.)	0.494
Weight of plates, g	N/A	Specific Gravity of solids, (Gs)	2.716

Height of solids, $H_s = \dfrac{W_S}{A \times G_s \times \gamma_W} = \dfrac{63.97}{31.67 \times 2.72 \times 1 \times 2.54} = 0.2928$ in.

Original height of water, $H_{WO} = \dfrac{W_{WO}}{A \times \gamma_W} = \dfrac{14.11}{31.67 \times 1 \times 2.54} = 0.1754$ in.

Final height of water, $H_{Wf} = \dfrac{W_{Wf}}{A \times \gamma_W} = \dfrac{14.53}{31.67 \times 1 \times 2.54} = 0.1806$ in.

Net change in height of specimen at end of test, $\Delta H =$ -0.01820 in.

Height of specimen at end of test, $H_f = H - \Delta H =$ 0.4758 in.

Void ratio before test, $e_O = = \dfrac{H - H_S}{H_S} = \dfrac{0.494 - 0.2928}{0.2928} = 0.6872$

Void ratio after test, $e_f = \dfrac{H_f - H_S}{H_S} = \dfrac{0.4758 - 0.2928}{0.2928} = 0.6250$

Degree of saturation before test, $S_O = \dfrac{H_{WO}}{H - H_S} = \dfrac{0.1754}{0.4940 - 0.2928} = 87.2\%$

Degree of saturation after test, $S_f = \dfrac{H_{wf}}{H_f - H_S} = \dfrac{0.1806}{0.4758 - 0.2928} = 98.7\%$

Dry density before test, $\gamma_d = \dfrac{W_S}{H \times A} = \dfrac{63.97 \times 62.4}{0.494 \times 31.67 \times 2.54} = 100.5$ lb./cu.ft.

Remarks ______________________

Technician James Hutt Computed by James Hutt Checked by James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-32b Sample No.: --- Depth: 1.3-13.3 Consol.No.: 2

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. °C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. °C
12/19	0.25	10:05	0	2008	20	12/21	2	8:50	0	2084.8	
12/19	0.375	10:07	2	2011		12/21	2	8:50	0.05	2116.5	
12/19	0.375	18:15	490	2012		12/21	2	8:50	0.1	2119.5	
12/20	0.375	9:10	1385	2013	20	12/21	2	8:50	0.2	2123	
						12/21	2	8:50	0.33	2125.2	
						12/21	2	8:50	0.5	2127	
						12/21	2	8:50	0.75	2129	
						12/21	2	8:51	1	2130	
12/20	1	9:20	0	2013	20	12/21	2	8:52	2	2132.5	
12/20	1	9:20	0.05	2052		12/21	2	8:54	4	2135	
12/20	1	9:20	0.1	2055		12/21	2	8:58	8	2137.2	
12/20	1	9:20	0.2	2058		12/21	2	9:05	15	2139.5	
12/20	1	9:20	0.33	2060		12/21	2	9:20	30	2142	
12/20	1	9:20	0.5	2061.5		12/21	2	9:50	60	2144.5	
12/20	1	9:20	0.75	2063		12/21	2	10:30	100	2146.2	
12/20	1	9:21	1	2064		12/21	2	12:10	200	2149	
12/20	1	9:22	2	2066.5		12/21	2	14:00	310	2150.8	
12/20	1	9:24	4	2069		12/21	2	16:00	430	2151.8	
12/20	1	9:28	8	2071		12/22	2	8:00	1390	2155	
12/20	1	9:35	15	2073							
12/20	1	9:52	32	2075							
12/20	1	10:20	60	2077							
12/20	1	11:00	100	2078.5							
12/20	1	12:50	210	2080.5							
12/20	1	14:25	305	2081.5							
12/20	1	17:45	505	2082.8							
12/21	1	8:50	1410	2084.8							

Technician James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-32b Sample No.: --- Depth: 1.3-13.3 Consol.No.: 2

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. °C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. °C
12/22	4	8:30	0	2155	20	12/23	8	6:30	0	2243	21
12/22	4	8:30	0.05	2192		12/23	8	6:30	0.05	2291	
12/22	4	8:30	0.1	2196.5		12/23	8	6:30	0.1	2298	
12/22	4	8:30	0.2	2200		12/23	8	6:30	0.2	2304	
12/22	4	8:30	0.33	2203.5		12/23	8	6:30	0.33	2309.5	
12/22	4	8:30	0.5	2206		12/23	8	6:30	0.5	2313	
12/22	4	8:30	0.75	2208		12/23	8	6:30	0.75	2316.5	
12/22	4	8:31	1	2209.5		12/23	8	6:31	1	2319	
12/22	4	8:32	2	2213		12/23	8	6:32	2	2324	
12/22	4	8:34	4	2216.5		12/23	8	6:34	4	2329.2	
12/22	4	8:38	8	2220		12/23	8	6:38	8	2333.5	
12/22	4	8:45	15	2223		12/23	8	6:45	15	2338	
12/22	4	9:00	30	2226.2		12/23	8	7:00	30	2342	
12/22	4	9:30	60	2229.5		12/23	8	7:30	60	2347	
12/22	4	10:10	100	2232		12/23	8	8:10	100	2350.5	
12/22	4	11:50	200	2235.2		12/23	8	9:55	205	2354	
12/22	4	13:30	300	2237.5		12/23	8	14:35	485	2359	
12/22	4	18:00	570	2240.8		12/23	8	17:10	640	2360	
12/23	4	6:20	1310	2243	21	12/24	8	8:00	1530	2363	20

Technician James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-32b Sample No.: --- Depth: 1.3-13.3 Consol.No.: 2

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C
						REBOUND LOADS					
						12/24	2	8:00	Rebound	2363.0	20
						12/25	2	8:40	1480	2316	
						12/25	0.5	8:40	Rebound	2316	20
						12/26	0.5	8:40	1440	2254	
						12/26	0.125	8:40	Rebound	2254	20
						12/29	0.125	8:00	4280	2187	19
						Machine Deflection Readings					
							0.25			2008	
							0.375			2012	
							1			2030	
							2			2044	
							4			2058	
							8			2074	
							2			2047	
							1			2021	
							0.125			2005	

Technician James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Computation of Void Ratio)

PROJECT Brownsville Levee Repair TEAM Job No.: 142086 DATE: 12/19/14

BORING NO. BRN-P3-32b SAMPLE NO. --- DEPTH 11.3-13.3' CONSOLIDOMETER NO. 2

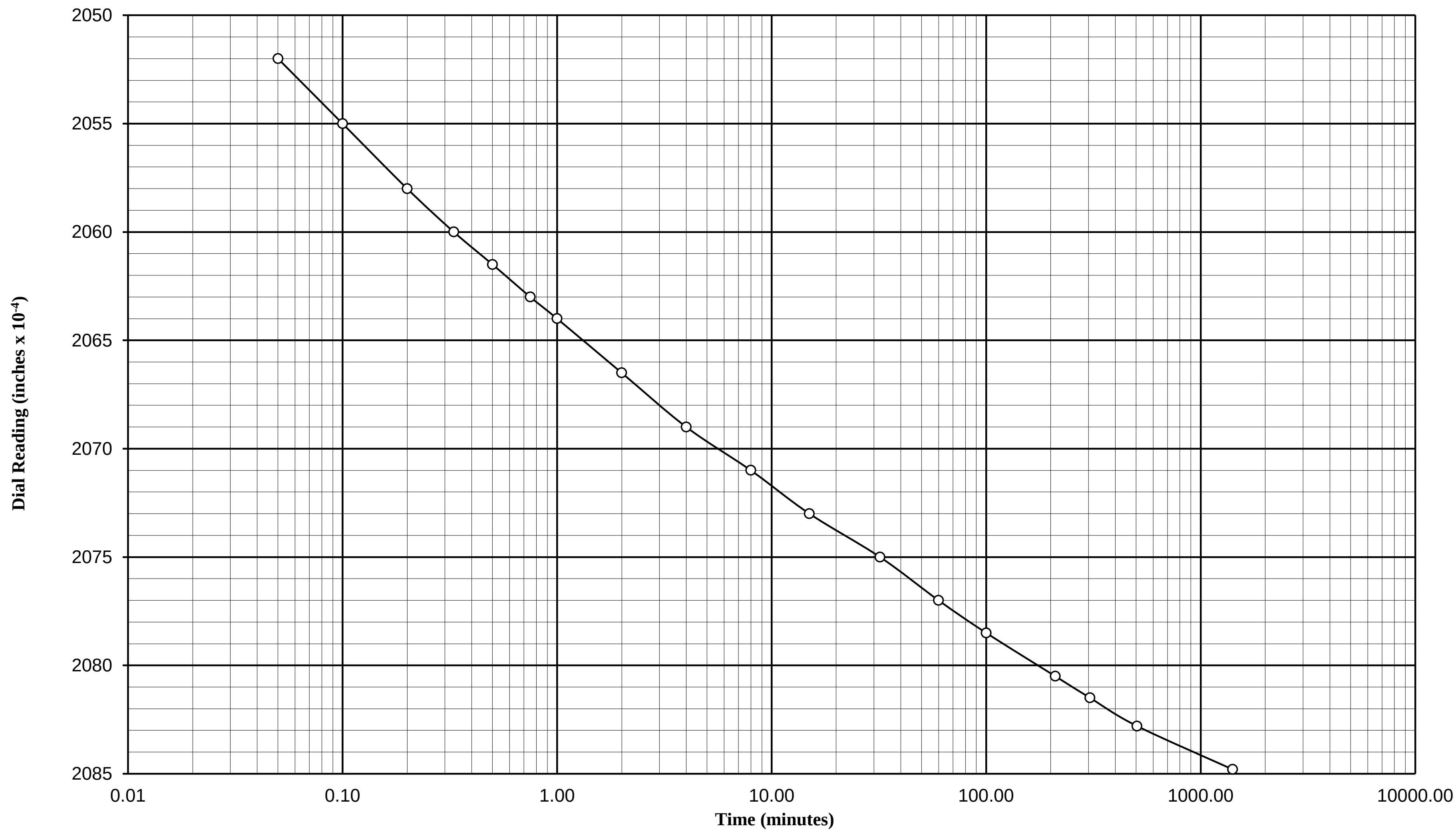
Pressure, P T./sq.ft.	Date Increment Applied	Time in Min. Increment Effective	Dial Reading 10^{-4} in.	Correction 10^{-4} in.	Change in Height, ΔH 10^{-4} in.	Height of Voids, H_V 10^{-4} in.	Void Ratio, e
0.1	12/19	Zero Point	2000	2000	0	2012	0.6872
0.4	12/19	Initial Load	2012	2012	0	2012	0.6872
0.4	12/19	1385	2013	2012	-1	2011	0.6868
1	12/20	1410	2084.8	2030	-54.8	1957	0.6685
2	12/21	1390	2155	2044	-111	1901	0.6493
4	12/22	1310	2243	2058	-185	1827	0.6240
8	12/23	1530	2363	2074	-289	1723	0.5885
2	12/24	1480	2316	2047	-269	1743	0.5953
1	12/25	1440	2254	2021	-233	1779	0.6076
0.125	12/26	4280	2187	2005	-182	1830	0.6250

Note:

Height of voids, $H_V = (H - H_S) - \Delta H$ $H_S =$ 0.2928

Void Ratio, $e = \frac{H_V}{H_S}$

Technician James Hutt Computed by James Hutt Checked by James Hutt



CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	22.0×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.20620	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	0.58	Date:	12/19/2014
DEPTH:	11.3-13.3'	Load (tsf):	1	Remarks	
SAMPLE:	---	Thickness (inches)	0.494		

TEAM Consultants, Inc.

CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	19.2×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.21283	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	0.65	Date:	12/19/2014
DEPTH:	11.3-13.3'	Load (tsf):	2	Remarks	
SAMPLE:	---	Thickness (inches)	0.494		

TEAM Consultants, Inc.

CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	8.09×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.22118	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	1.5	Date:	12/19/2014
DEPTH:	11.3-13.3'	Load (tsf):	4	Remarks	
SAMPLE:	---	Thickness (inches)	0.494		

TEAM Consultants, Inc.

CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	16.5×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.23160	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	0.71	Date:	12/19/2014
DEPTH:	11.3-13.3'	Load (tsf):	8	Remarks	
SAMPLE:	---	Thickness (inches)	0.494		

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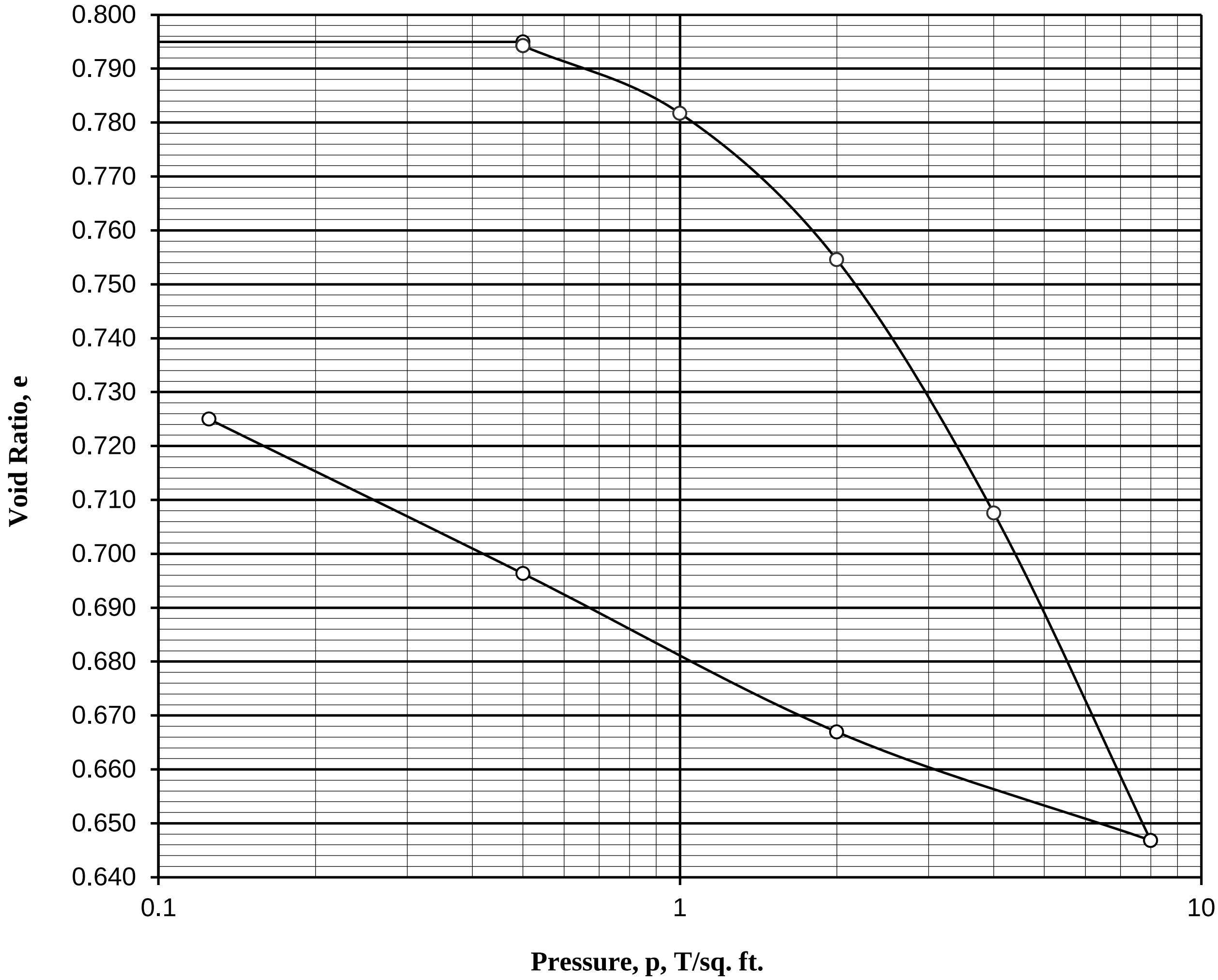
Geotechnical, Environmental, Construction Materials Testing



Type of specimen: Undisturbed		Before Test		After Test	
Diam. 2.50 in.	Ht. 0.495 in.	Water Content, w_o	27.19%	W_f	26.61%
Overburden Pressure, P_o T/sq. ft.		Void Ratio, e_o	0.7950	e_f	0.7250
Preconsol. Pressure, P_c T/sq. ft.		Saturation, S_o	92.9%	S_f	99.7%
Compression Index, C_c		Dry Density, γd	94.4 lb/ft^3		
Classification Brown fat clay					

LL 56	G_s 2.716	Project	Brownsville Levee Repair
PL 24			Team Project No.: 142086
Remarks			
		Boring No: BRN-P3-32b	Sample No.: ---
		Depth: 15.7-17.7	Date: 12/19/14
		CONSOLIDATION TEST REPORT	

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST
(Specimen Data)

Project: Brownsville Levee Repair   TEAM Job No.: 142086
Boring No.: BRN-P3-32b   Sample No.: ---   Depth: 15.7-17.7   Date: 12/19/14

Classification   Brown fat clay

			Before Test			After Test	
			Specimen		Trimmings	Specimen	
Tare No.			Ring and Plates		478	412	
Weight in grams	Tare plus wet soil		288.44		768.9	111.80	
	Tare plus dry soil		272.06		681.54	95.77	
	Water	W_W	W_{WO}	16.38	87.36	W_{wf}	16.03
	Tare		211.81		360.2	35.52	
	Dry soil	W_S	60.25		321.35	60.25	
Water Content		w	W_O	27.19%	27.19%	W_f	26.61%

Consolidometer No.:	3	Area of specimen, A, (sq. cm.)	31.67
Weight of ring, g	N/A	Height of specimen, H, (in.)	0.495
Weight of plates, g	N/A	Specific Gravity of solids, (Gs)	2.716

Height of solids, $H_s = \dfrac{W_S}{A \times G_s \times \gamma_W} = \dfrac{60.25}{31.67 \times 2.72 \times 1 \times 2.54} = 0.2758$ in.

Original height of water, $H_{WO} = \dfrac{W_{WO}}{A \times \gamma_W} = \dfrac{16.38}{31.67 \times 1 \times 2.54} = 0.2036$ in.

Final height of water, $H_{Wf} = \dfrac{W_{Wf}}{A \times \gamma_W} = \dfrac{16.03}{31.67 \times 1 \times 2.54} = 0.1993$ in.

Net change in height of specimen at end of test, $\Delta H =$ -0.01930 in.

Height of specimen at end of test, $H_f = H - \Delta H =$ 0.4757 in.

Void ratio before test, $e_O = = \dfrac{H - H_S}{H_S} = \dfrac{0.495 - 0.2758}{0.2758} = 0.7950$

Void ratio after test, $e_f = \dfrac{H_f - H_S}{H_S} = \dfrac{0.4757 - 0.2758}{0.2758} = 0.7250$

Degree of saturation before test, $S_O = \dfrac{H_{WO}}{H - H_S} = \dfrac{0.2036}{0.4950 - 0.2758} = 92.9\%$

Degree of saturation after test, $S_f = \dfrac{H_{wf}}{H_f - H_S} = \dfrac{0.1993}{0.4757 - 0.2758} = 99.7\%$

Dry density before test, $\gamma_d = \dfrac{W_S}{H \times A} = \dfrac{60.25 \times 62.4}{0.495 \times 31.67 \times 2.54} = 94.4$ lb./cu.ft.

Remarks ______

Technician James Hutt   Computed by James Hutt   Checked by James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-32b Sample No.: --- Depth: 15.7-17.7 Consol.No.: 3

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{o}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{o}C
12/19	0.25	10:50	0	2008	20	12/21	2	8:55	0	2064.5	
12/19	0.5	10:51	1	2017		12/21	2	8:55	0.05	2092.5	
12/19	0.5	18:15	445	2020		12/21	2	8:55	0.1	2096.5	
12/20	0.5	9:10	1340	2021	20	12/21	2	8:55	0.2	2100.5	
						12/21	2	8:55	0.33	2103.5	
						12/21	2	8:55	0.5	2106	
						12/21	2	8:55	0.75	2108.8	
						12/21	2	8:56	1	2110.8	
12/20	1	9:25	0	2021	20	12/21	2	8:57	2	2115.8	
12/20	1	9:25	0.05	2037		12/21	2	8:59	4	2121.2	
12/20	1	9:25	0.1	2038.5		12/21	2	9:03	8	2126.8	
12/20	1	9:25	0.2	2040		12/21	2	9:10	15	2131.5	
12/20	1	9:25	0.33	2041.2		12/21	2	9:25	30	2136	
12/20	1	9:25	0.5	2042.2		12/21	2	9:55	60	2139.2	
12/20	1	9:25	0.75	2043.5		12/21	2	10:35	100	2142	
12/20	1	9:26	1	2044.2		12/21	2	12:20	205	2145.2	
12/20	1	9:27	2	2046.2		12/21	2	14:00	305	2147.5	
12/20	1	9:29	4	2048.8		12/21	2	16:00	425	2149	
12/20	1	9:33	8	2051.2		12/22	2	8:00	1385	2153.5	
12/20	1	9:40	15	2053.2							
12/20	1	9:55	30	2055.2							
12/20	1	10:25	60	2057.2							
12/20	1	11:10	105	2059							
12/20	1	12:50	205	2060.8							
12/20	1	14:25	300	2061.5							
12/20	1	17:45	500	2062.8							
12/21	1	8:55	1410	2064.5							

Technician James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-32b Sample No.: --- Depth: 15.7-17.7 Consol.No.: 3

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C
12/22	4	8:35	0	2153.5	20	12/23	8	6:35	0	2299	21
12/22	4	8:35	0.05	2194		12/23	8	6:35	0.05	2343	
12/22	4	8:35	0.1	2199		12/23	8	6:35	0.1	2349	
12/22	4	8:35	0.2	2204		12/23	8	6:35	0.2	2356	
12/22	4	8:35	0.33	2208		12/23	8	6:35	0.33	2362	
12/22	4	8:35	0.5	2212		12/23	8	6:35	0.5	2366	
12/22	4	8:35	0.75	2216		12/23	8	6:35	0.75	2372	
12/22	4	8:36	1	2220		12/23	8	6:36	1	2376	
12/22	4	8:37	2	2230		12/23	8	6:37	2	2390	
12/22	4	8:39	4	2241		12/23	8	6:39	4	2406	
12/22	4	8:43	8	2252.5		12/23	8	6:43	8	2425	
12/22	4	8:50	15	2262		12/23	8	6:50	15	2440	
12/22	4	9:05	30	2270.5		12/23	8	7:07	32	2453	
12/22	4	9:37	62	2277.5		12/23	8	7:35	60	2463	
12/22	4	10:15	100	2281.5		12/23	8	8:15	100	2468	
12/22	4	11:55	200	2287		12/23	8	10:00	205	2474.5	
12/22	4	13:35	300	2290.2		12/23	8	14:35	480	2480.8	
12/22	4	18:00	565	2295		12/23	8	17:10	635	2482.5	
12/23	8	6:20	1305	2299	21	12/24	8	8:00	1525	2486.5	20

Technician James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-32b Sample No.: --- Depth: 15.7-17.7 Consol.No.: 3

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C
						REBOUND LOADS					
						12/24	2	8:00	Rebound	2486.5	20
						12/25	2	8:40	1480	2399	
						12/25	0.5	8:40	Rebound	2399	20
						12/26	0.5	8:40	1440	2294	
						12/26	0.125	8:40	Rebound	2294	20
						12/29	0.125	8:00	4280	2199	19
						Machine Deflection Readings					
							0.25			2008	
							0.5			2019	
							1			2028	
							2			2042	
							4			2058	
							8			2078	
							2			2046	
							1			2022	
							0.125			2006	

Technician James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Computation of Void Ratio)

PROJECT Brownsville Levee Repair TEAM Job No.: 142086 DATE: 12/19/14

BORING NO. BRN-P3-32b SAMPLE NO. --- DEPTH 15.7-17.7 CONSOLIDOMETER NO. 3

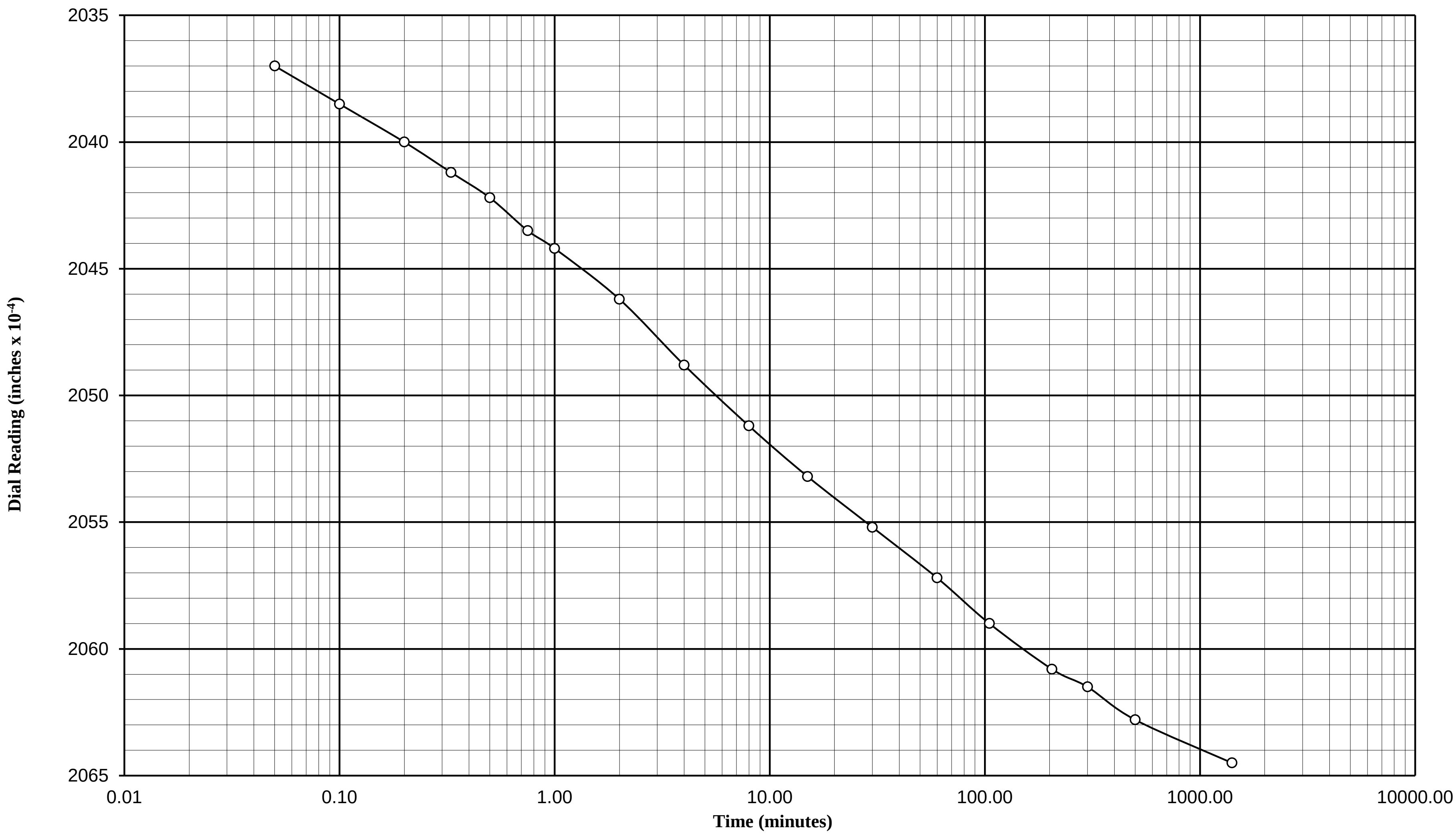
Pressure, P T./sq.ft.	Date Increment Applied	Time in Min. Increment Effective	Dial Reading 10^{-4} in.	Correction 10^{-4} in.	Change in Height, ΔH 10^{-4} in.	Height of Voids, H_V 10^{-4} in.	Void Ratio, e
0.1	12/19	Zero Point	2000	2000	0	2192	0.7950
0.5	12/19	Initial Load	2019	2019	0	2192	0.7950
0.5	12/19	1340	2021	2019	-2	2190	0.7942
1	12/20	1410	2064.5	2028	-36.5	2156	0.7817
2	12/21	1385	2153.5	2042	-111.5	2081	0.7545
4	12/22	1305	2299	2058	-241	1951	0.7076
8	12/23	1525	2486.5	2078	-408.5	1784	0.6468
2	12/24	1480	2399	2046	-353	1839	0.6670
1	12/25	1440	2294	2022	-272	1920	0.6963
0.125	12/26	4280	2199	2006	-193	1999	0.7250

Note:

Height of voids, $H_V = (H - H_S) - \Delta H$ $H_S =$ 0.2758

Void Ratio, $e = \frac{H_V}{H_S}$

Technician James Hutt Computed by James Hutt Checked by James Hutt



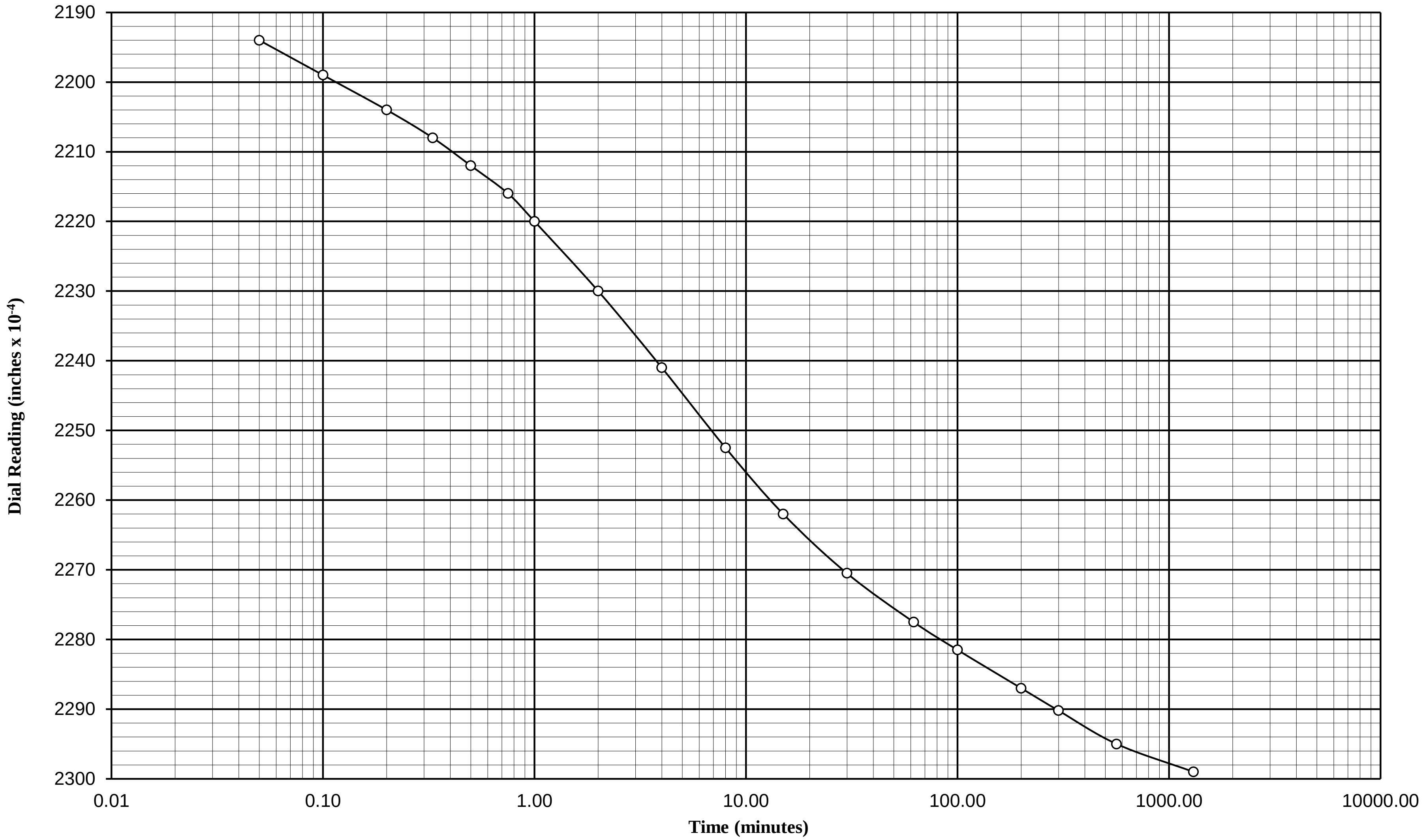
CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	4.95×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.20473	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	2.6	Date:	12/19/2014
DEPTH:	15.7-17.7	Load (tsf):	1	Remarks	
SAMPLE:	---	Thickness (inches)	0.495		

TEAM Consultants, Inc.

CONSOLIDATION TEST - DIAL READING TIME CURVE

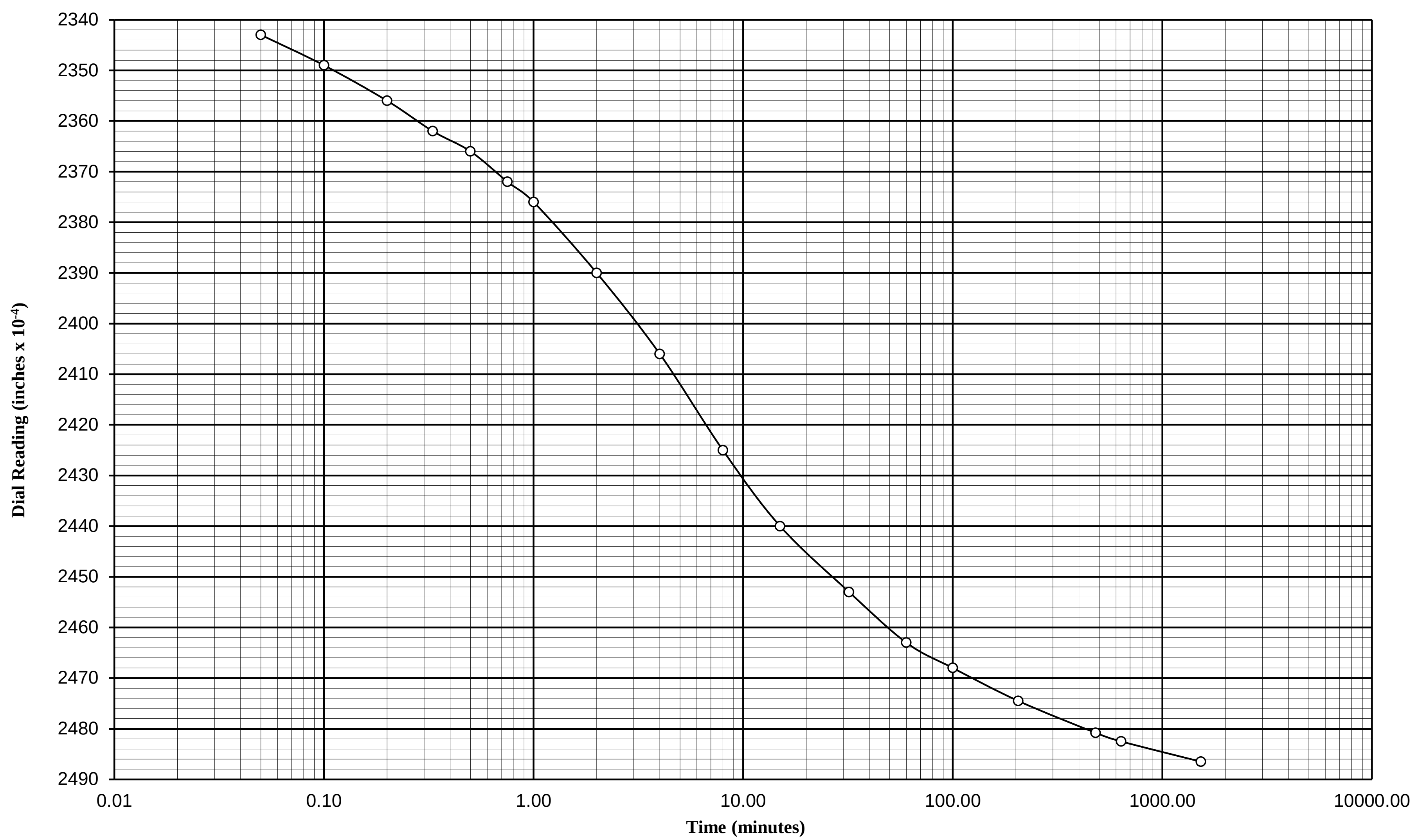
PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	8.40×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.21138	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	1.5	Date:	12/19/2014
DEPTH:	15.7-17.7	Load (tsf):	2	Remarks	
SAMPLE:	---	Thickness (inches)	0.495		



CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	4.83×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.22333	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	2.5	Date:	12/19/2014
DEPTH:	15.7-17.7	Load (tsf):	4	Remarks	
SAMPLE:	---	Thickness (inches)	0.495		

TEAM Consultants, Inc.



CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	3.78×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.23995	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	3.0	Date:	12/19/2014
DEPTH:	15.7-17.7	Load (tsf):	8	Remarks	
SAMPLE:	---	Thickness (inches)	0.495		

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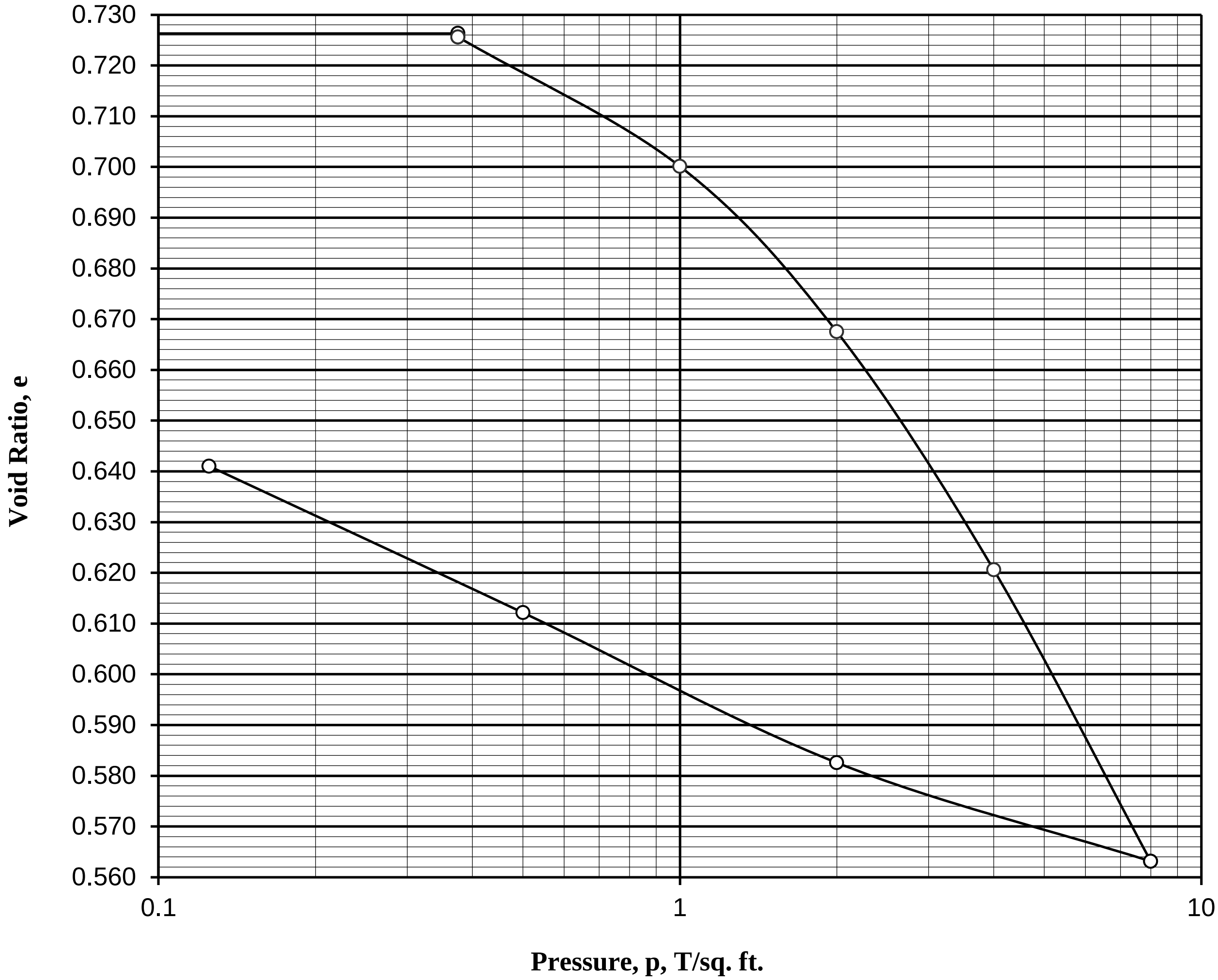
Geotechnical, Environmental, Construction Materials Testing



Type of specimen: Undisturbed		Before Test		After Test	
Diam. 2.50 in.	Ht. 0.502 in.	Water Content, w_o	22.16%	W_f	23.43%
Overburden Pressure, P_o T/sq. ft.		Void Ratio, e_o	0.7263	e_f	0.6410
Preconsol. Pressure, P_c T/sq. ft.		Saturation, S_o	83.0%	S_f	99.4%
Compression Index, C_c		Dry Density, γd	98.3 lb/ft^3		
Classification Brown lean clay					
LL 45	G_s 2.719	Project	Brownsville Levee Repair		
PL 21					
Remarks			Team Project No.: 142086		
		Boring No:	BRN-P3-33	Sample No.:	---
		Depth:	2-4'	Date:	12/19/14
		CONSOLIDATION TEST REPORT			

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST
(Specimen Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086
Boring No.: BRN-P3-33 Sample No.: --- Depth: 2-4' Date: 12/19/14

Classification Brown lean clay

				Before Test		After Test	
				Specimen	Trimmings	Specimen	
Tare No.				Ring and Plates	463	439	
Weight in grams	Tare plus wet soil			188.31	770.1	113.75	
	Tare plus dry soil			174.21	700.15	98.85	
	Water	W_W	W_{WO}	14.10	69.95	W_{wf}	14.90
	Tare			110.61	384.6	35.25	
	Dry soil	W_S		63.60	315.59	63.60	
Water Content		w	W_O	22.16%	22.16%	W_f	23.43%

Consolidometer No.:	5	Area of specimen, A, (sq. cm.)	31.67
Weight of ring, g	N/A	Height of specimen, H, (in.)	0.502
Weight of plates, g	N/A	Specific Gravity of solids, (Gs)	2.719

Height of solids, $H_s = \frac{W_S}{A \times G_s \times \gamma_W} = \frac{63.60}{31.67 \times 2.72 \times 1 \times 2.54} = 0.2908$ in.

Original height of water, $H_{WO} = \frac{W_{WO}}{A \times \gamma_W} = \frac{14.10}{31.67 \times 1 \times 2.54} = 0.1752$ in.

Final height of water, $H_{Wf} = \frac{W_{Wf}}{A \times \gamma_W} = \frac{14.90}{31.67 \times 1 \times 2.54} = 0.1852$ in.

Net change in height of specimen at end of test, ΔH = -0.02480 in.

Height of specimen at end of test, $H_f = H - \Delta H =$ 0.4772 in.

Void ratio before test, $e_O = = \frac{H - H_S}{H_S} = \frac{0.502 - 0.2908}{0.2908} = 0.7263$

Void ratio after test, $e_f = \frac{H_f - H_S}{H_S} = \frac{0.4772 - 0.2908}{0.2908} = 0.6410$

Degree of saturation before test, $S_O = \frac{H_{WO}}{H - H_S} = \frac{0.1752}{0.5020 - 0.2908} = 83.0\%$

Degree of saturation after test, $S_f = \frac{H_{wf}}{H_f - H_S} = \frac{0.1852}{0.4772 - 0.2908} = 99.4\%$

Dry density before test, $\gamma_d = \frac{W_S}{H \times A} = \frac{63.60 \times 62.4}{0.502 \times 31.67 \times 2.54} = 98.3$ lb./cu.ft.

Remarks

Technician James Hutt Computed by James Hutt Checked by James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-33 Sample No.: --- Depth: 2-4' Consol.No.: 5

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C
12/19	0.25	11:10	0	2002	20	12/21	2	9:00	0	2090.2	
12/19	0.375	11:15	5	2009		12/21	2	9:00	0.05	2121	
12/19	0.375	18:15	425	2008		12/21	2	9:00	0.1	2125.5	
12/20	0.375	9:10	1320	2006	20	12/21	2	9:00	0.2	2129.5	
						12/21	2	9:00	0.33	2133.5	
						12/21	2	9:00	0.5	2136.5	
						12/21	2	9:00	0.75	2139.5	
						12/21	2	9:01	1	2142	
12/20	1	9:30	0	2006	20	12/21	2	9:02	2	2148	
12/20	1	9:30	0.05	2039		12/21	2	9:04	4	2154.5	
12/20	1	9:30	0.1	2042		12/21	2	9:08	8	2161.5	
12/20	1	9:30	0.2	2044.5		12/21	2	9:15	15	2167.8	
12/20	1	9:30	0.33	2046.8		12/21	2	9:30	30	2173.8	
12/20	1	9:30	0.5	2048.5		12/21	2	10:00	60	2178.8	
12/20	1	9:30	0.75	2051		12/21	2	10:45	105	2182.5	
12/20	1	9:31	1	2052.5		12/21	2	12:20	200	2186	
12/20	1	9:32	2	2057.5		12/21	2	14:00	300	2189	
12/20	1	9:34	4	2062.5		12/21	2	16:00	420	2191	
12/20	1	9:38	8	2067.5		12/22	2	8:00	1380	2195	
12/20	1	9:45	15	2072							
12/20	1	10:03	33	2076							
12/20	1	10:30	60	2079							
12/20	1	11:10	100	2081.2							
12/20	1	12:50	200	2084							
12/20	1	14:35	305	2086							
12/20	1	17:45	495	2087.2	20						
12/21	1	9:00	1410	2090.2							

Technician James Hutt

TEAM Consultants, Inc.

Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair TEAM Job No.: 142086

Boring No.: BRN-P3-33 Sample No.: --- Depth: 2-4' Consol.No.: 5

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. °C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. °C
12/22	4	8:40	0	2195	20	12/23	8	6:40	0	2343.5	21
12/22	4	8:40	0.05	2243		12/23	8	6:40	0.05	2400	
12/22	4	8:40	0.1	2249		12/23	8	6:40	0.1	2406	
12/22	4	8:40	0.2	2254		12/23	8	6:40	0.2	2412	
12/22	4	8:40	0.33	2258.5		12/23	8	6:40	0.33	2417	
12/22	4	8:40	0.5	2262.2		12/23	8	6:40	0.5	2421.5	
12/22	4	8:40	0.75	2266.5		12/23	8	6:40	0.75	2426.5	
12/22	4	8:41	1	2270		12/23	8	6:41	1	2430	
12/22	4	8:42	2	2278.5		12/23	8	6:42	2	2440.5	
12/22	4	8:44	4	2288		12/23	8	6:44	4	2453	
12/22	4	8:48	8	2297.5		12/23	8	6:48	8	2466.5	
12/22	4	8:55	15	2307		12/23	8	6:56	16	2481	
12/22	4	9:11	31	2316		12/23	8	7:15	35	2495	
12/22	4	9:40	60	2322.5		12/23	8	7:40	60	2502	
12/22	4	10:22	102	2327.5		12/23	8	8:20	100	2507.5	
12/22	4	12:00	200	2333		12/23	8	10:00	200	2513.5	
12/22	4	13:55	315	2336.5		12/23	8	14:35	475	2519.8	
12/22	4	18:00	560	2340.5		12/23	8	17:10	630	2522	
12/23	4	6:20	1300	2343.5	21	12/24	8	8:00	1520	2525.5	20

Technician James Hutt

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Geotechnical, Environmental, Construction Materials Testing

CONSOLIDATION TEST

(Time - Consolidation Data)

Project: Brownsville Levee Repair   TEAM Job No.: 142086

Boring No.: BRN-P3-33   Sample No.: ---   Depth: 2-4'   Consol.No.: 5

Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}C
						REBOUND LOADS					
						12/24	2	8:00	Rebound	2525.5	20
						12/25	2	8:40	1480	2446	
						12/25	0.5	8:40	Rebound	2446	20
						12/26	0.5	8:40	1440	2343	
						12/26	0.125	8:40	Rebound	2343	20
						12/29	0.125	8:00	4280	2251	19
						Machine Deflection Readings					
							0.25			2002	
							0.375			2004	
							1			2014	
							2			2024	
							4			2036	
							8			2051	
							2			2028	
							1			2011	
							0.125			2003	

Technician James Hutt

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CONSOLIDATION TEST

(Computation of Void Ratio)

PROJECT Brownsville Levee Repair TEAM Job No.: 142086 DATE: 12/19/14

BORING NO. BRN-P3-33 SAMPLE NO. --- DEPTH 2-4' CONSOLIDOMETER NO. 5

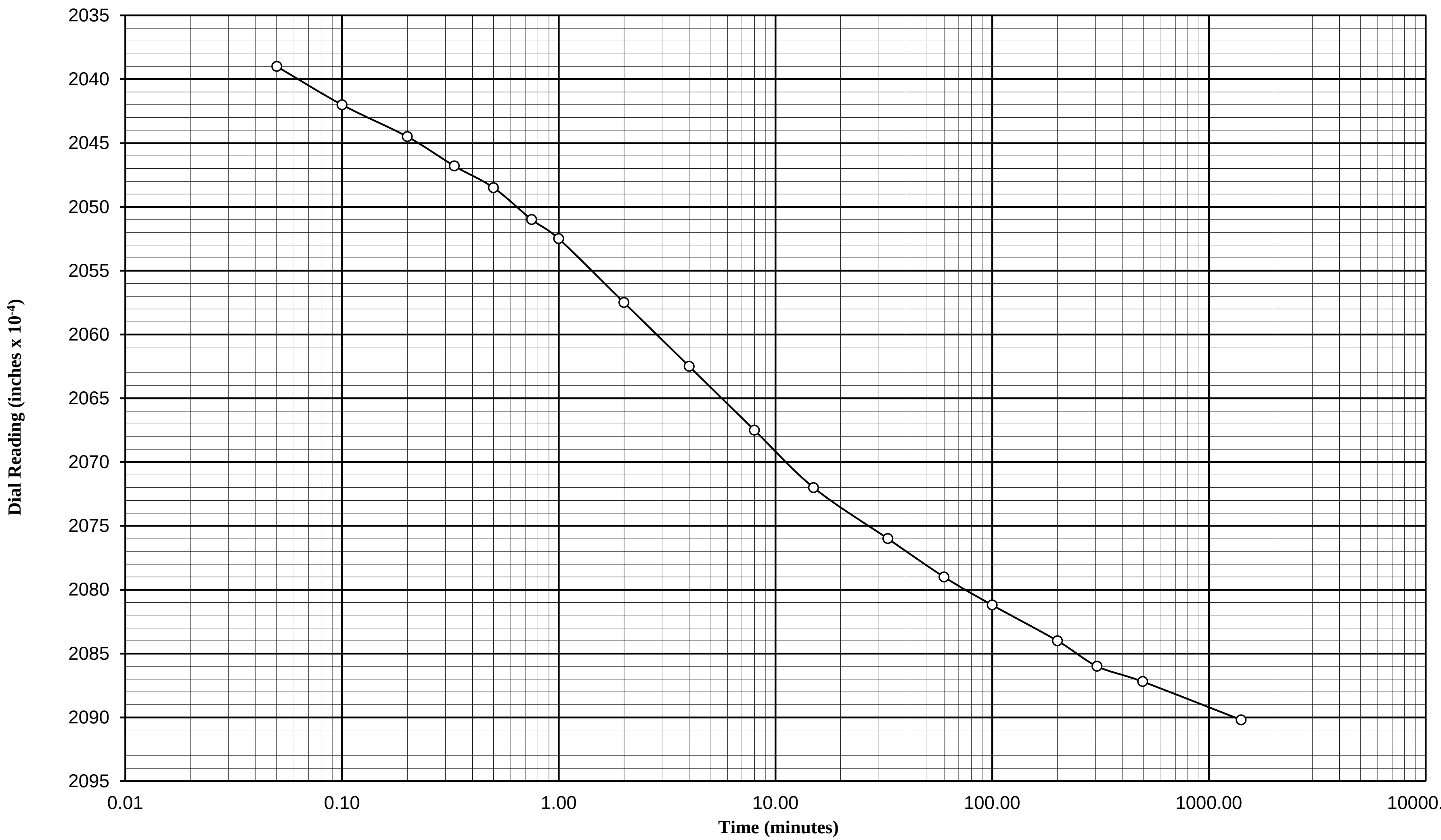
Pressure, P T./sq.ft.	Date Increment Applied	Time in Min. Increment Effective	Dial Reading 10^{-4} in.	Correction 10^{-4} in.	Change in Height, ΔH 10^{-4} in.	Height of Voids, H_V 10^{-4} in.	Void Ratio, e
0.1	12/19	Zero Point	2000	2000	0	2112	0.7263
0.375	12/19	Initial Load	2004	2004	0	2112	0.7263
0.375	12/19	1320	2006	2004	-2	2110	0.7256
1	12/20	1410	2090.2	2014	-76.2	2036	0.7001
2	12/21	1380	2195	2024	-171	1941	0.6675
4	12/22	1300	2343.5	2036	-307.5	1805	0.6206
8	12/23	1520	2525.5	2051	-474.5	1638	0.5631
2	12/24	1480	2446	2028	-418	1694	0.5826
1	12/25	1440	2343	2011	-332	1780	0.6121
0.125	12/26	4280	2251	2003	-248	1864	0.6410

Note:

Height of voids, $H_V = (H - H_S) - \Delta H$ $H_S =$ 0.2908

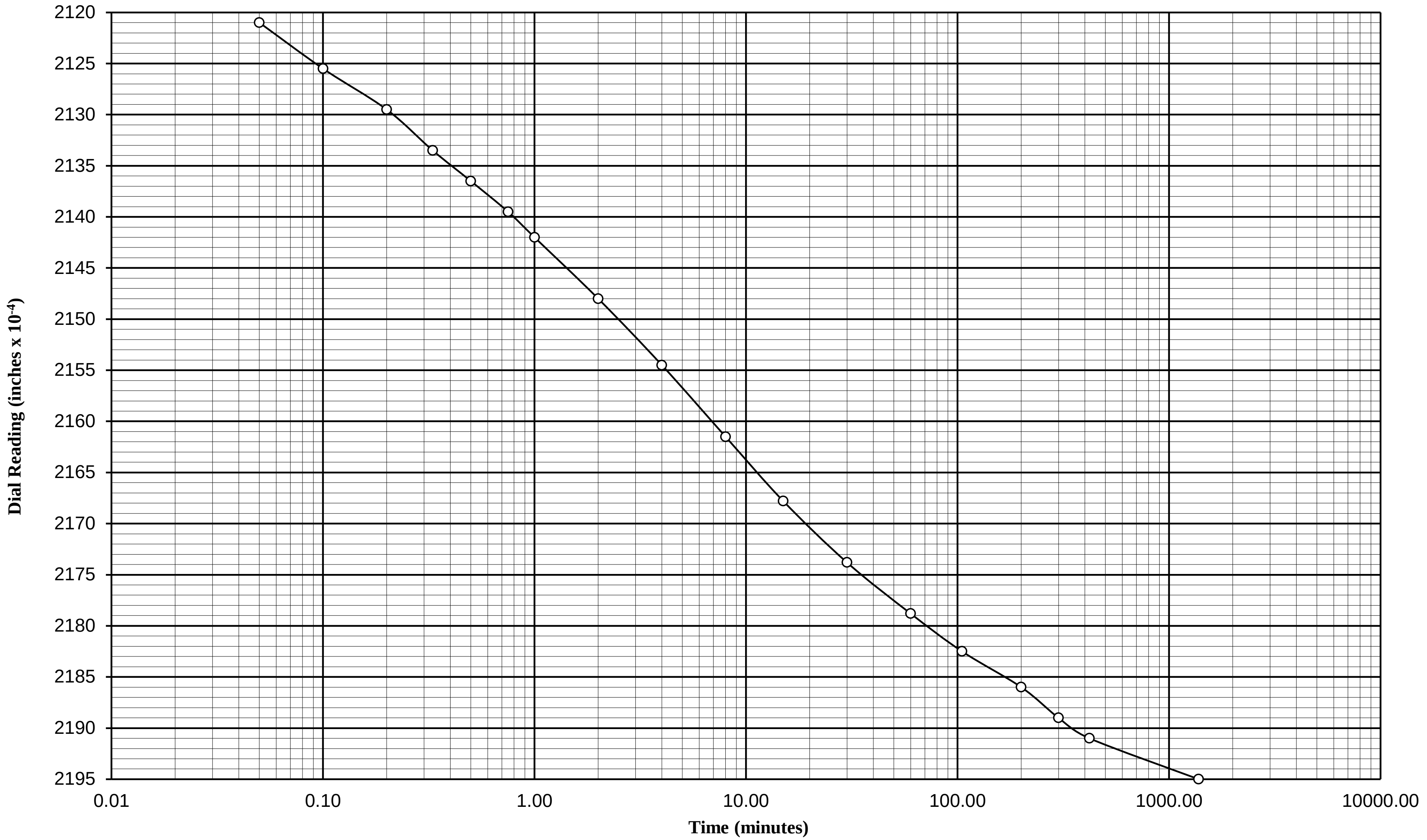
Void Ratio, $e = \frac{H_V}{H_S}$

Technician James Hutt Computed by James Hutt Checked by James Hutt



CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	6.90×10^{-4} (cm²/sec)		
		d_{50} (inches):	0.20570	TEAM Project No.:	142086
BORING NO.:	BRN-P3-33	t_{50} (min):	1.9	Date:	12/19/2014
DEPTH:	2-4'	Load (tsf):	1	Remarks	
SAMPLE:	---	Thickness (inches)	0.502		



CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	6.04×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.21485	TEAM Project No.:	142086
BORING NO.:	BRN-P3-33	t_{50} (min):	2.1	Date:	12/19/2014
DEPTH:	2-4'	Load (tsf):	2	Remarks	
SAMPLE:	---	Thickness (inches)	0.502		

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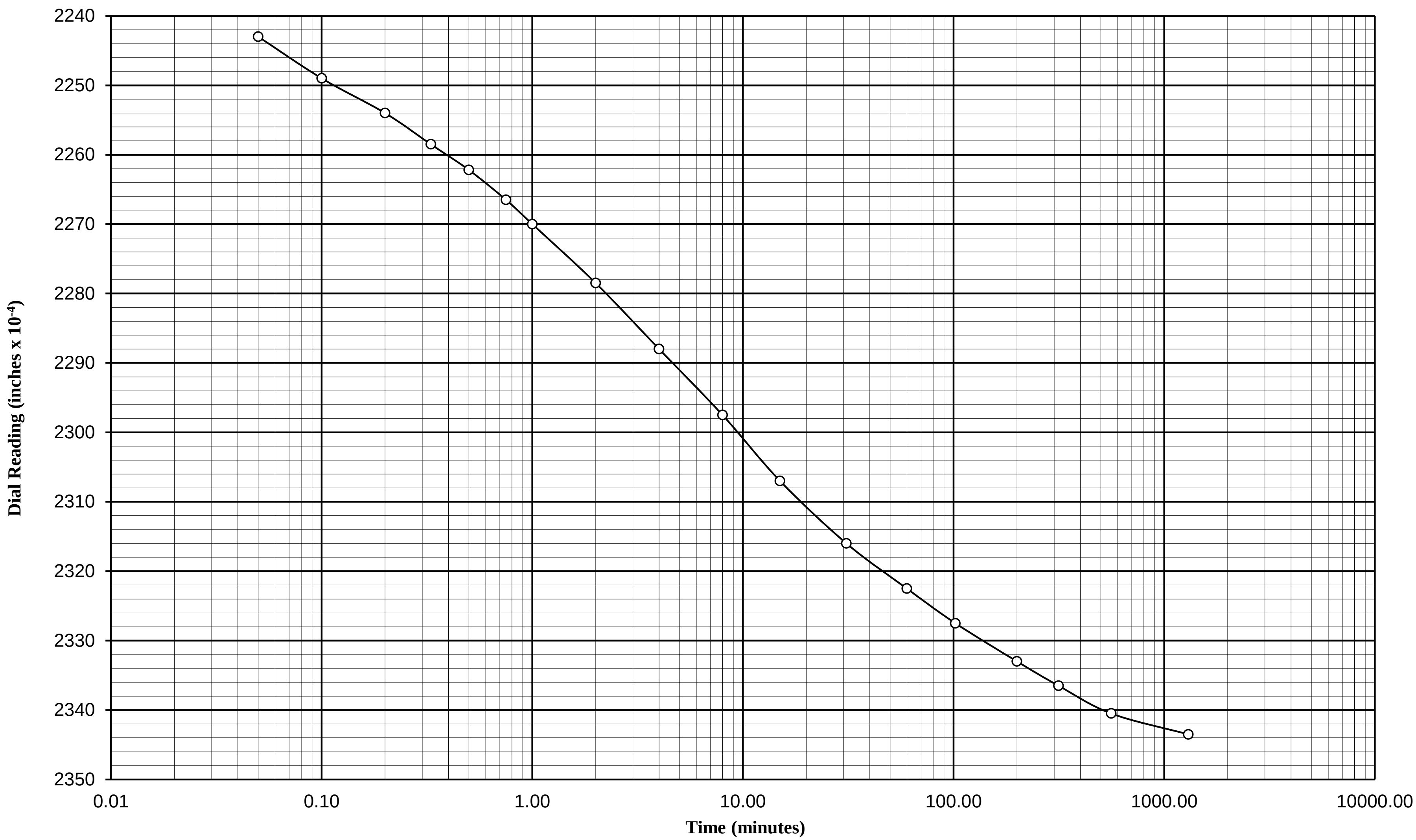

CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	4.83×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.22820	TEAM Project No.:	142086
BORING NO.:	BRN-P3-33	t_{50} (min):	2.5	Date:	12/19/2014
DEPTH:	2-4'	Load (tsf):	4	Remarks	
SAMPLE:	---	Thickness (inches)	0.502		

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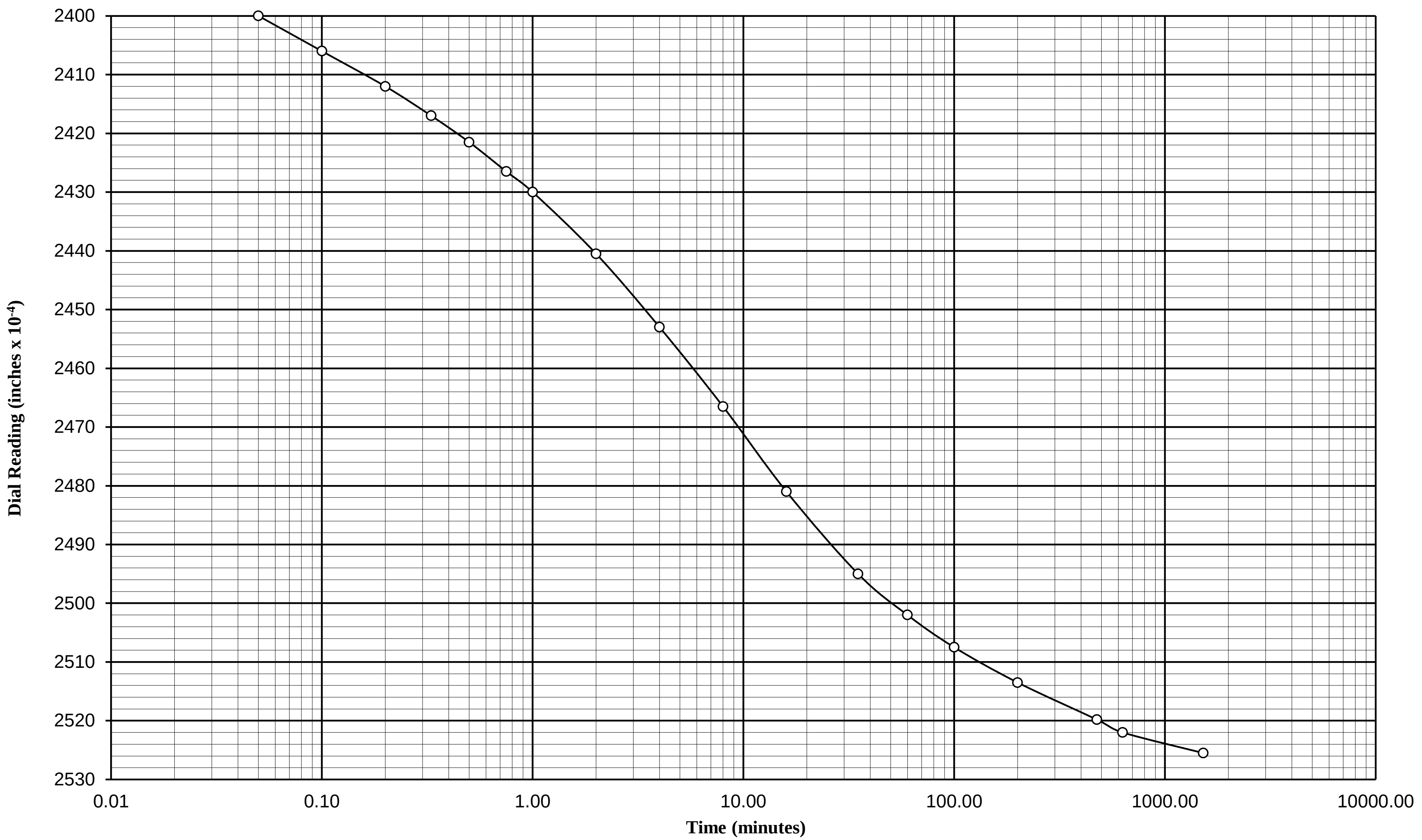


CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	3.33×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.24500	TEAM Project No.:	142086
BORING NO.:	BRN-P3-33	t_{50} (min):	3.4	Date:	12/19/2014
DEPTH:	2-4'	Load (tsf):	8	Remarks	
SAMPLE:	---	Thickness (inches)	0.502		

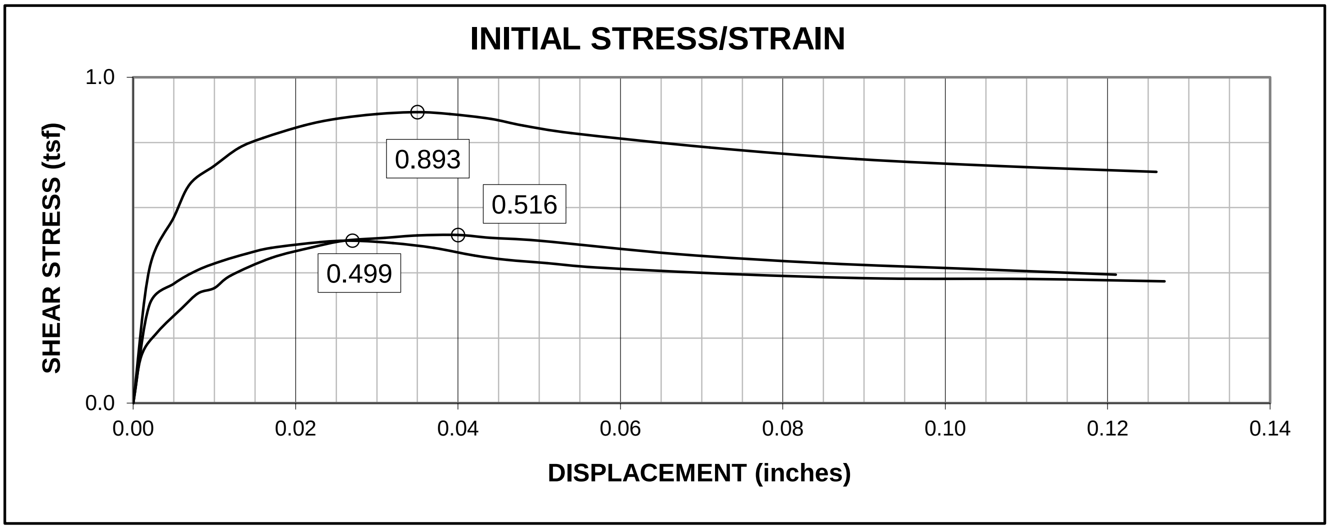
TEAM Consultants, Inc.

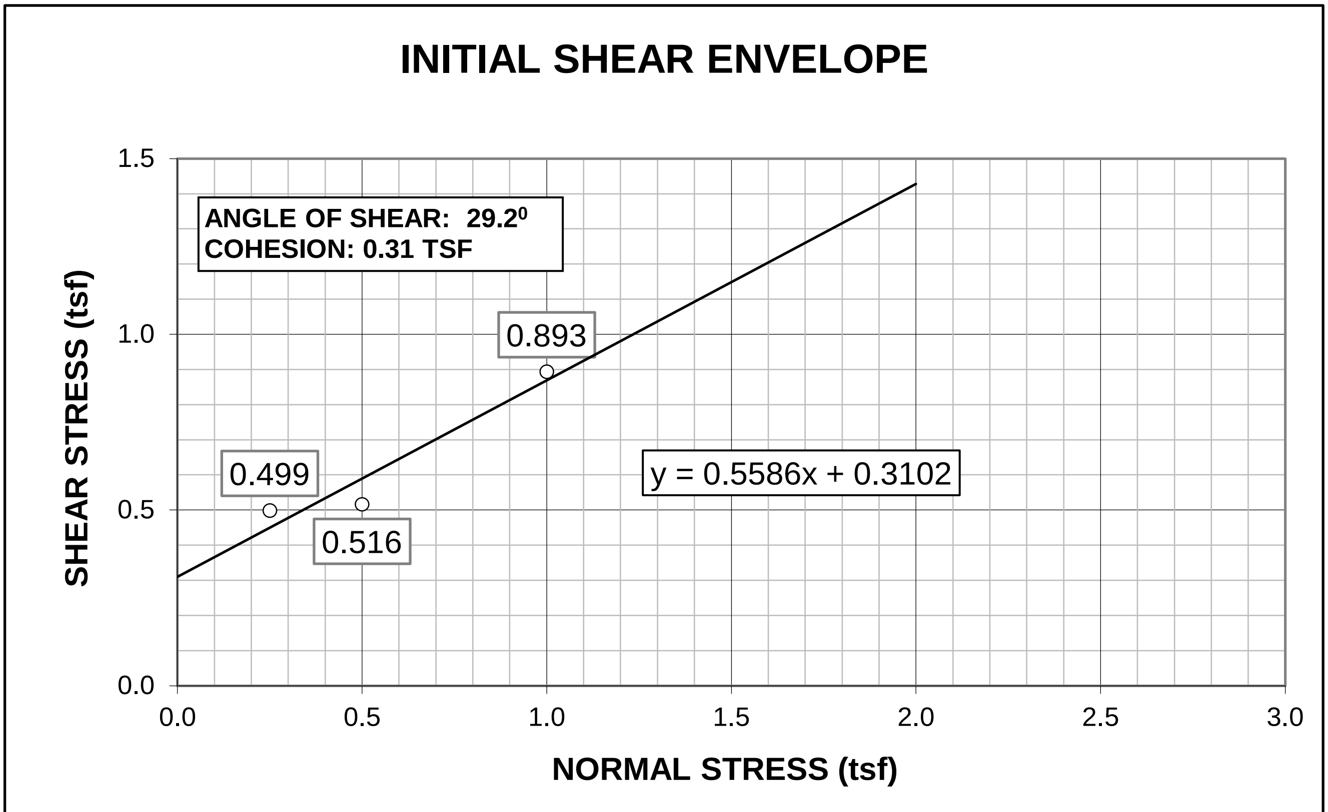
PROJECT: Brownsville Levee **DATE:** 12/30/2014 **JOB NO.:** 142086

SAMPLE: P-3 32b, 4.7-6.7 **DESCRIPTION:** Grayish brown lean clay

TYPE OF TEST: Consolidated-Drained (Initial Shear) Normal loading: 0.25 , 0.5 & 1 tsf

NORMAL STRESS(tsf)	MOISTURE CONTENT(%) INITIAL	MOISTURE CONTENT(%) FINAL	UNIT WEIGHT (pcf)	MAXIMUM SHEAR STRESS (tsf)
0.25	22.4	24.6	102.1	0.499
0.5	22.4	23.1	102.8	0.516
1.0	22.4	21.8	103.9	0.893





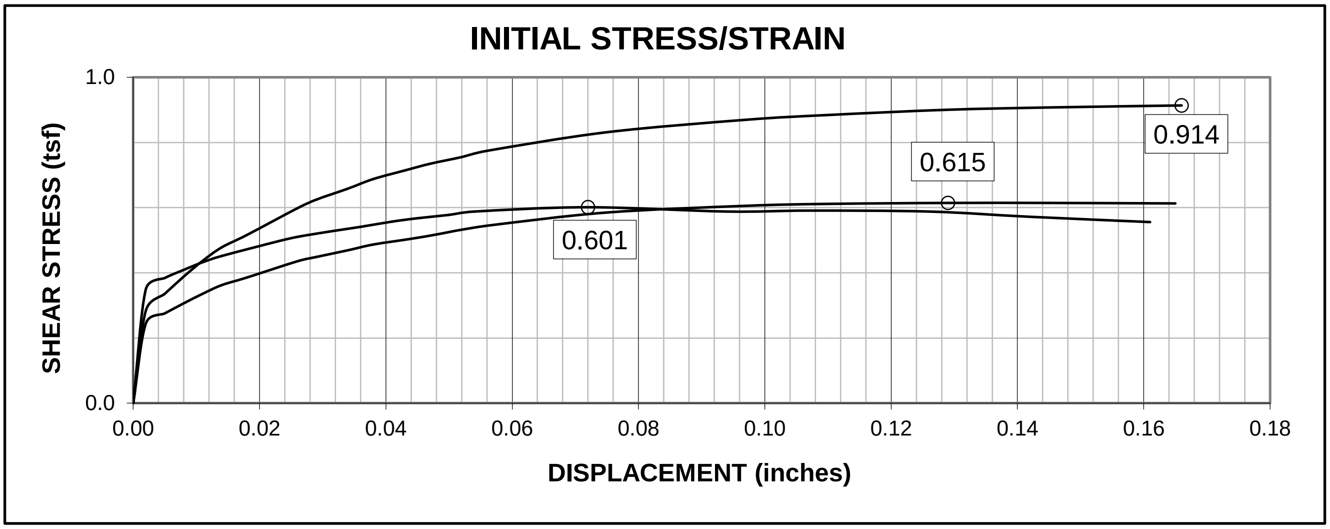
CONSOLIDATED DRAINED DIRECT SHEAR TEST

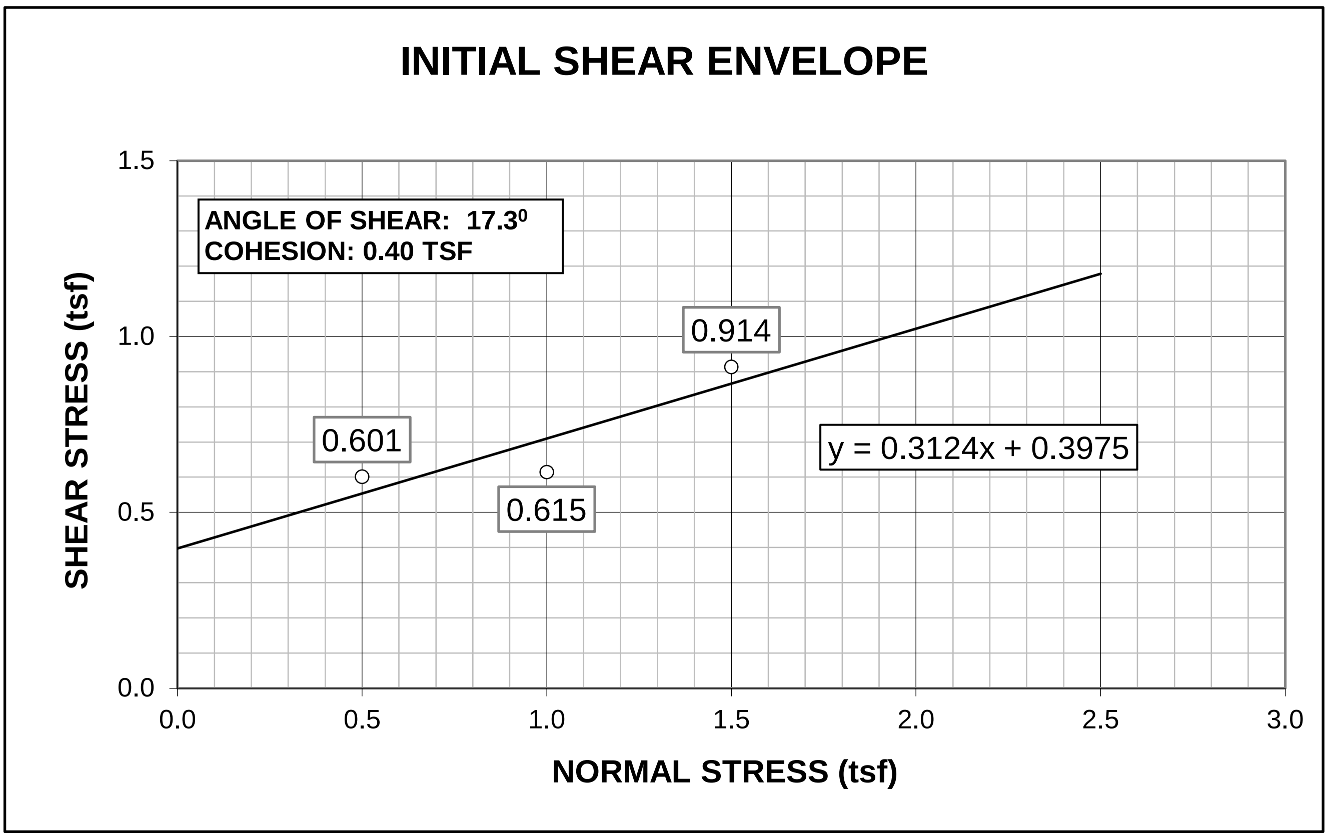
PROJECT: Brownsville Levee **DATE:** 1/8/2015 **JOB NO.:** 142086

SAMPLE: P-3 32b, 17.9-19.9 **DESCRIPTION:** Grayish brown fat clay

TYPE OF TEST: Consolidated-Drained (Initial Shear) Normal loading: 0.5 , 1.0 & 1.5 tsf

NORMAL STRESS(tsf)	MOISTURE CONTENT(%) INITIAL	FINAL	UNIT WEIGHT (pcf)	MAXIMUM SHEAR STRESS (tsf)
0.5	28.9	29.6	91.9	0.601
1.0	28.9	29.0	90.8	0.615
1.5	28.9	29.0	91.4	0.914

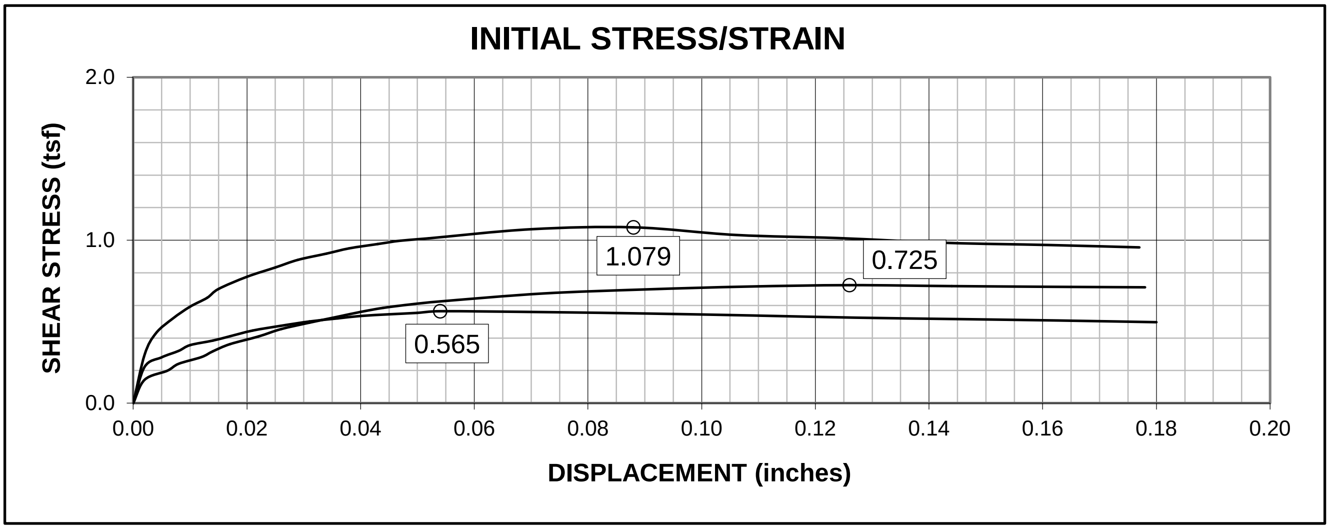


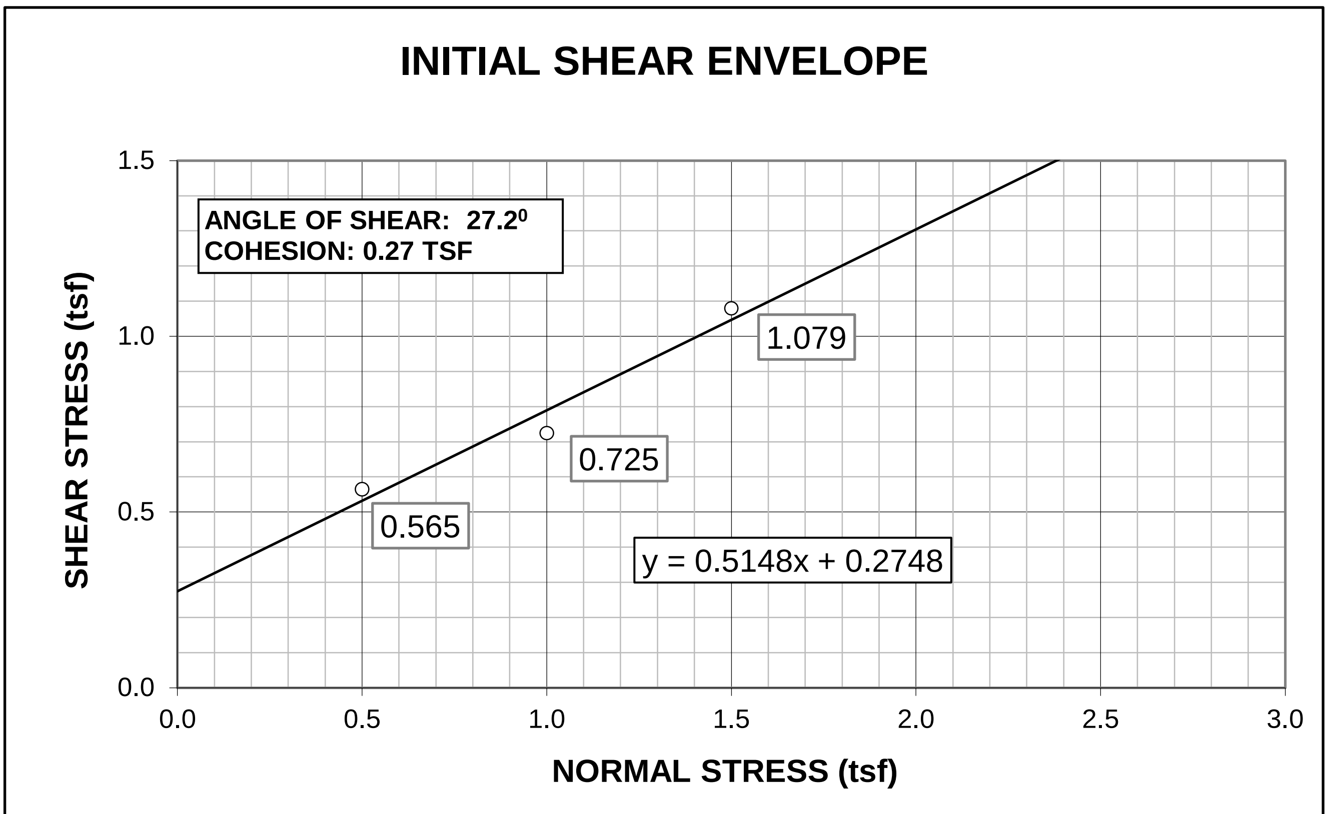


CONSOLIDATED DRAINED DIRECT SHEAR TEST

PROJECT:	Brownsville Levee	**DATE:**	1/5/2015	**JOB NO.:**	142086
SAMPLE:	P-3 32b, 22.3-24.3	**DESCRIPTION:**	Grayish brown lean clay		
TYPE OF TEST:	Consolidated-Drained (Initial Shear)	Normal loading:	0.5 , 1.0 & 1.5 tsf		

NORMAL STRESS(tsf)	MOISTURE CONTENT(%) INITIAL	FINAL	UNIT WEIGHT (pcf)	MAXIMUM SHEAR STRESS (tsf)
0.5	28.1	29.5	92.0	0.565
1.0	28.1	29.9	89.4	0.725
1.5	28.1	23.8	97.9	1.079





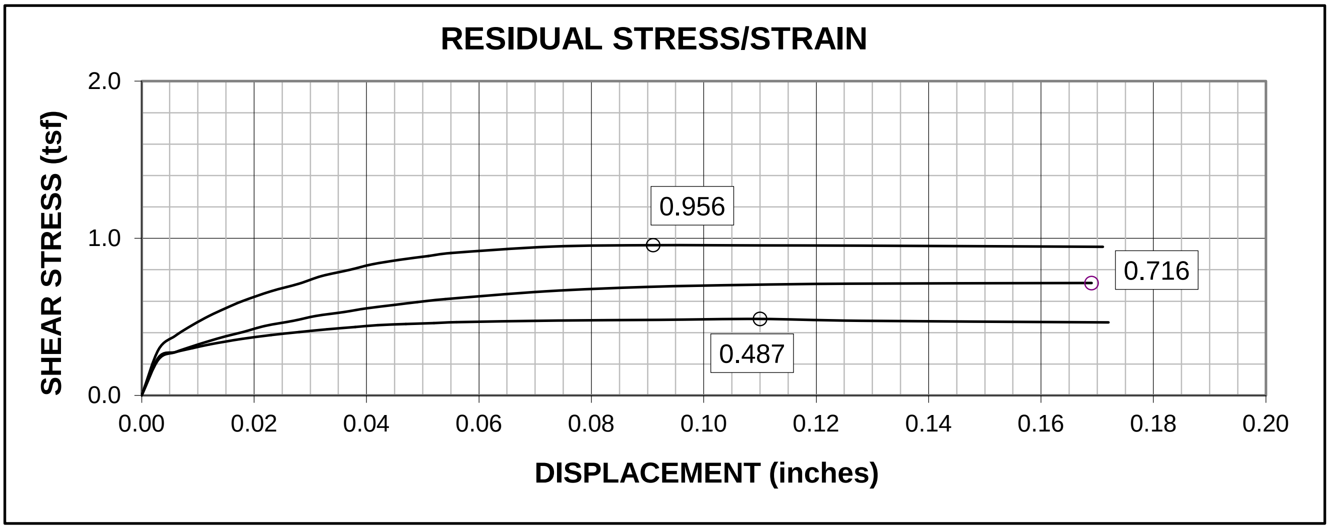
CONSOLIDATED DRAINED DIRECT SHEAR TEST

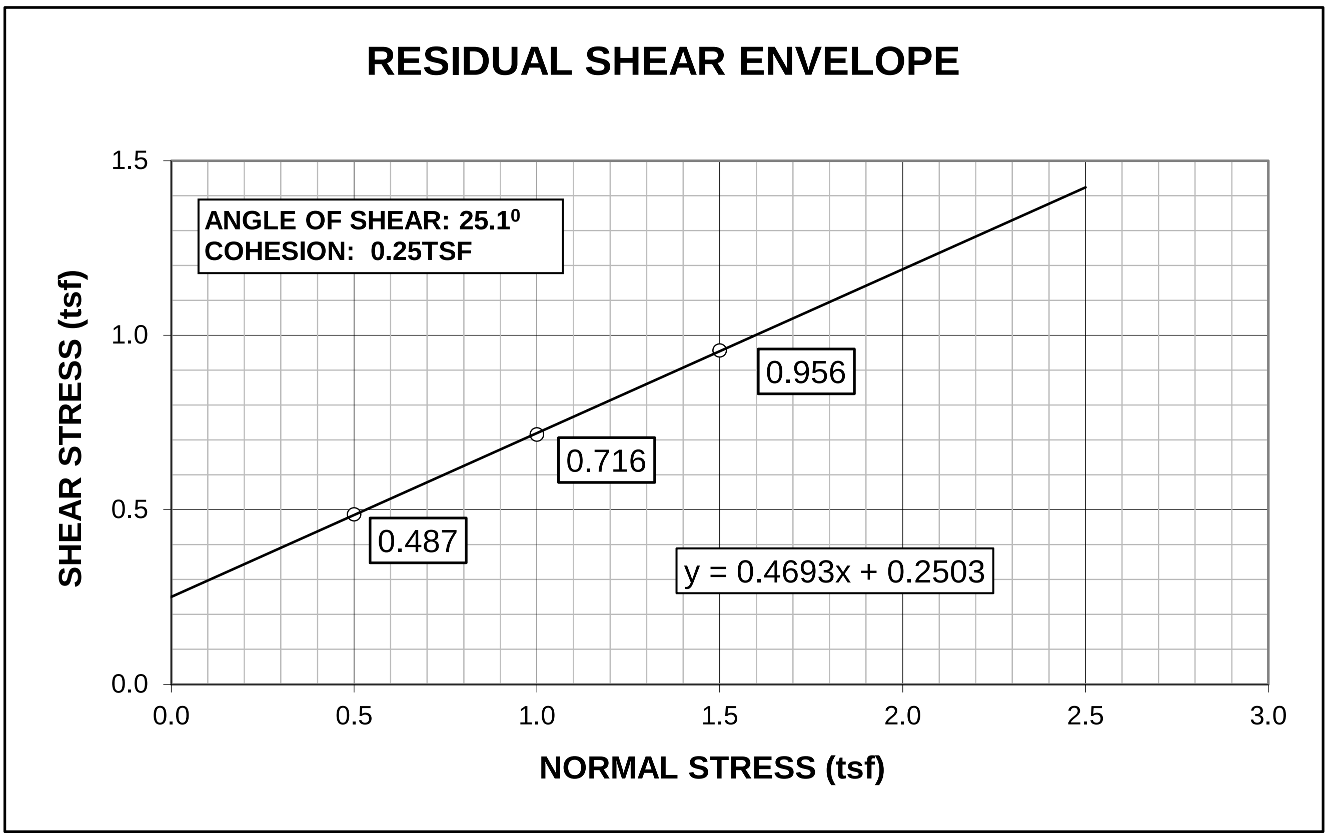
PROJECT: Brownsville Levee **DATE:** 1/5/2015 **JOB NO.:** 142086

SAMPLE: P-3 32b, 22.3-24.3 **DESCRIPTION:** Grayish brown lean clay

TYPE OF TEST: Consolidated-Drained (Residual Shear) Normal loading: 0.5 , 1.0 & 1.5 tsf

NORMAL STRESS(tsf)	MOISTURE CONTENT(%) INITIAL	FINAL	UNIT WEIGHT (pcf)	MAXIMUM SHEAR STRESS (tsf)
0.5	28.1	29.5	92.0	0.487
1.0	28.1	29.9	89.4	0.716
1.5	28.1	23.8	97.9	0.956





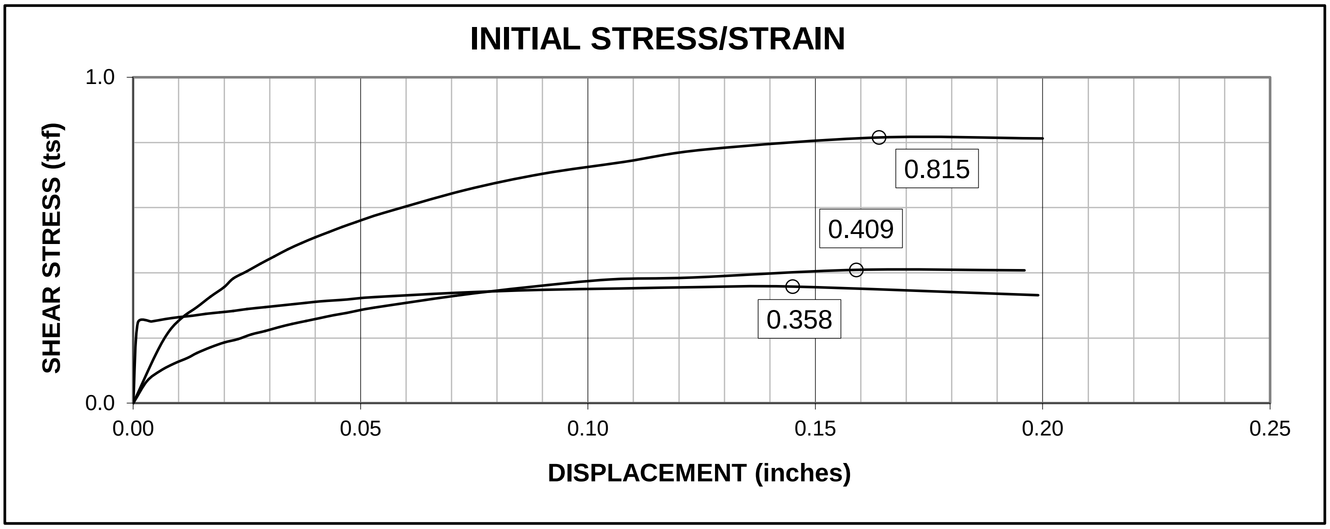
CONSOLIDATED DRAINED DIRECT SHEAR TEST

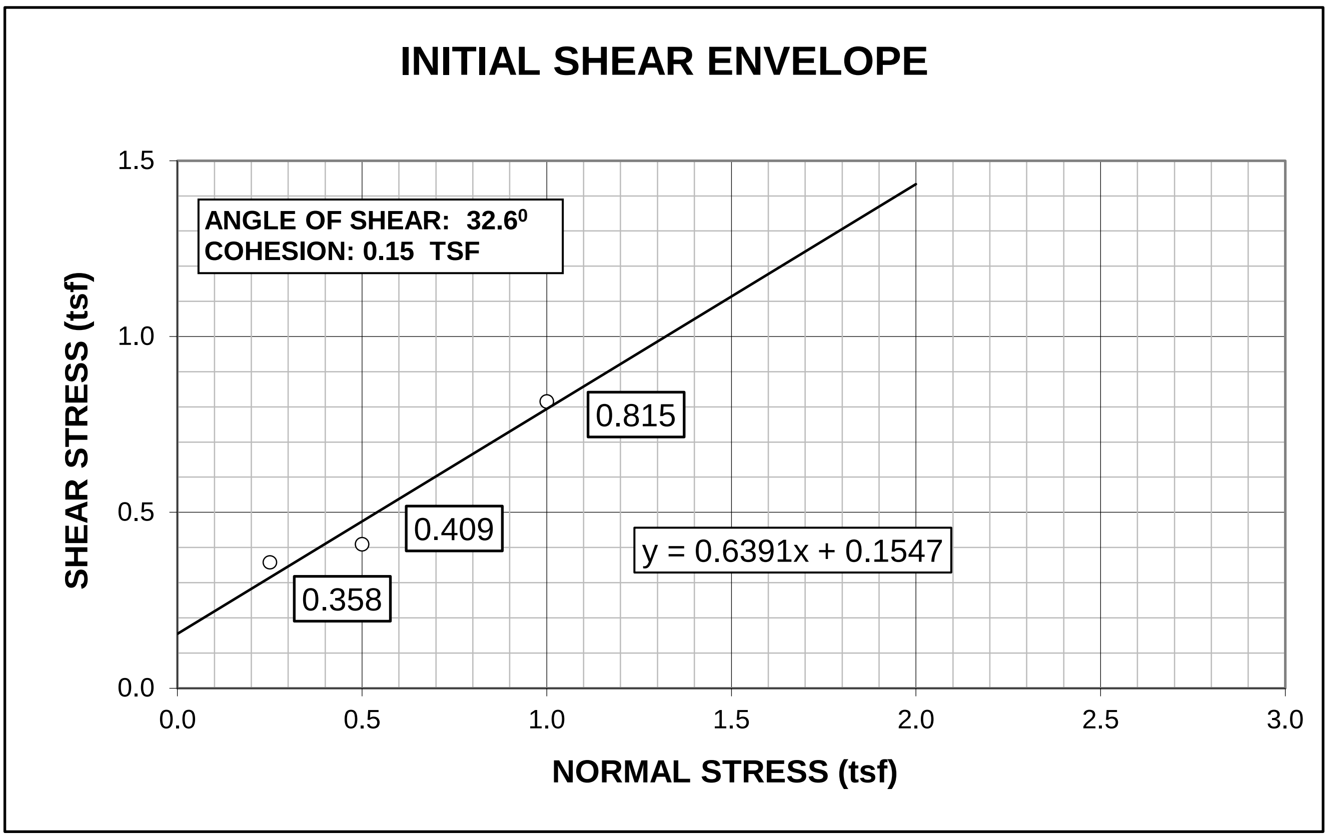
PROJECT: Brownsville Levee **DATE:** 12/19/2014 **JOB NO.:** 142086

SAMPLE: P-3 33, 13-15 **DESCRIPTION:** Grayish brown lean clay

TYPE OF TEST: Consolidated-Drained (Initial Shear) Normal loading: 0.25 , 0.5 & 1 tsf

NORMAL STRESS(tsf)	MOISTURE CONTENT(%) INITIAL	MOISTURE CONTENT(%) FINAL	UNIT WEIGHT (pcf)	MAXIMUM SHEAR STRESS (tsf)
0.25	29.7	28.2	89.3	0.358
0.5	29.7	25.5	90.5	0.409
1.0	29.7	23.2	90.6	0.815





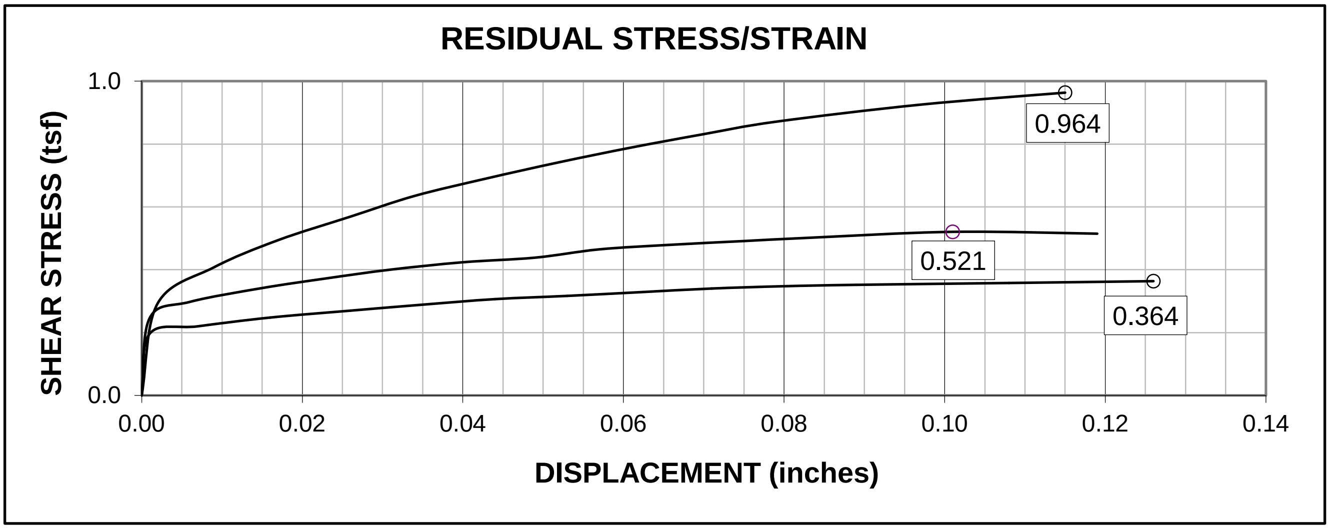
CONSOLIDATED DRAINED DIRECT SHEAR TEST

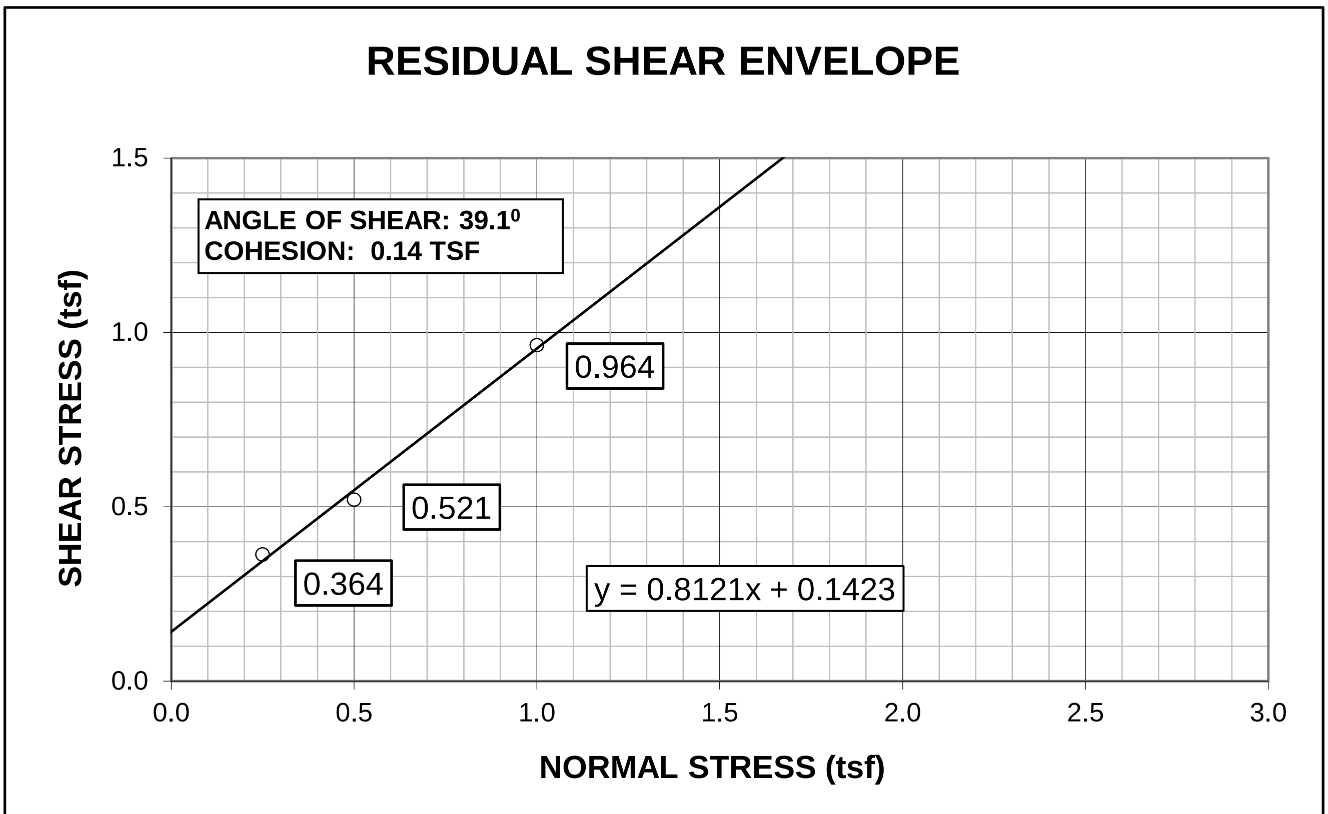
PROJECT: Brownsville Levee **DATE:** 12/19/2014 **JOB NO.:** 142086

SAMPLE: P-3 33, 13-15 **DESCRIPTION:** Grayish brown lean clay

TYPE OF TEST: Consolidated-Drained (Residual Shear) Normal loading: 0.25 , 0.5 & 1 tsf

NORMAL STRESS(tsf)	MOISTURE CONTENT(%) INITIAL	MOISTURE CONTENT(%) FINAL	UNIT WEIGHT (pcf)	MAXIMUM SHEAR STRESS (tsf)
0.25	29.7	28.2	89.3	0.364
0.5	29.7	25.5	90.5	0.521
1.0	29.7	23.2	90.6	0.964





CONSOLIDATED DRAINED DIRECT SHEAR TEST

TRIAXIAL TEST: UNCONFINED (ASTM D-2166) OR UNCONSOLIDATED-UNDRAINED (ASTM D-2850)

Project: USACE-Brownsville Levee | **Hole :** P3-32b | **Sample :** | **Depth:** 9.1-11.1

TEAM Project No.: 142086 | **Date:** 1/6/15 | **Material:** Brown lean clay

Height 1: 5.846 " **Dia.1:** 2.855 "

Height 2: 5.846 " **Dia.2:** 2.867 " **Area:** 6.447 In²

Height 3: 5.852 " **Dia.3:** 2.873 "

Young's Modulus for Membrane (tsf) 11.56

Weight g: 1273.7 **Strain Rate:** 0.060 (Inches/Minute)

Wet γ (pcf): 128.7 **Strain Rate:** 1.03 (%/Minute)

Dry γ (pcf): 108.7 **Length/Diameter Ratio:** 2.041

Test Type: Unconfined Compression

or UU Triaxial @ 8.8 psi X

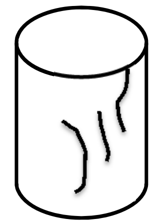
Proving Ring Constant: 1

Moisture Content (ASTM D 2216)

Before (cuttings) X **After**

Can-Dish No.:	673
Wet Wt. (Sple+Can):	338.7
Dry Wt. (Sple+Can):	307.2
Wt. of Can:	135.9
Wt. of Dry Soil:	171.3
Wt. of Water:	31.5
% Moisture:	18.4

GRAPHICAL DESCRIPTION OF FAILURE



Vertical

Confining Pressure (psi)	Dial Deflection	% Strain	Corrected Area (IN^2)	Load Dial Readings	Load Lbs	Deviator Stress (TSF) UNCORRECTED	Deviator Stress (TSF) CORRECTED*	Shearing Strength (cohesion)
8.8								
	0.020	0.342	6.469	86.8	86.8	0.966	0.965	0.483
	0.040	0.684	6.491	141.0	141.0	1.564	1.563	0.782
	0.060	1.026	6.514	183.3	183.3	2.027	2.025	1.012
	0.080	1.368	6.536	220.3	220.3	2.427	2.424	1.212
	0.100	1.710	6.559	254.8	254.8	2.797	2.794	1.397
	0.120	2.052	6.582	287.1	287.1	3.141	3.137	1.569
	0.140	2.394	6.605	319.0	319.0	3.478	3.473	1.737
	0.160	2.736	6.628	348.7	348.7	3.789	3.783	1.892
	0.180	3.078	6.651	377.1	377.1	4.082	4.076	2.038
	0.200	3.420	6.675	402.7	402.7	4.344	4.337	2.169
	0.220	3.762	6.699	424.9	424.9	4.568	4.560	2.280
	0.240	4.104	6.723	445.5	445.5	4.771	4.763	2.382
	0.260	4.446	6.747	462.9	462.9	4.941	4.932	2.466
	0.280	4.788	6.771	476.7	476.7	5.070	5.061	2.530
	0.300	5.130	6.795	487.0	487.0	5.160	5.151	2.575
	0.320	5.472	6.820	489.8	489.8	5.171	5.160	2.580
	0.340	5.814	6.845	473.7	473.7	4.983	4.972	2.486
	0.360	6.156	6.870	429.4	429.4	4.500	4.489	2.244
	0.380	6.498	6.895	400.1	400.1	4.178	4.166	2.083
	0.400	6.840	6.920	378.5	378.5	3.938	3.925	1.962

Strain (Inches/Inch) @ 50% Maximum Stress = 0.01512

Deformation @ 50% Maximum Stress (Inches)= 0.0884

Maximum Compressive Strength (TSF)= 5.16

% Strain @ Maximum Strength = 5.47%

Tested by: J. Young

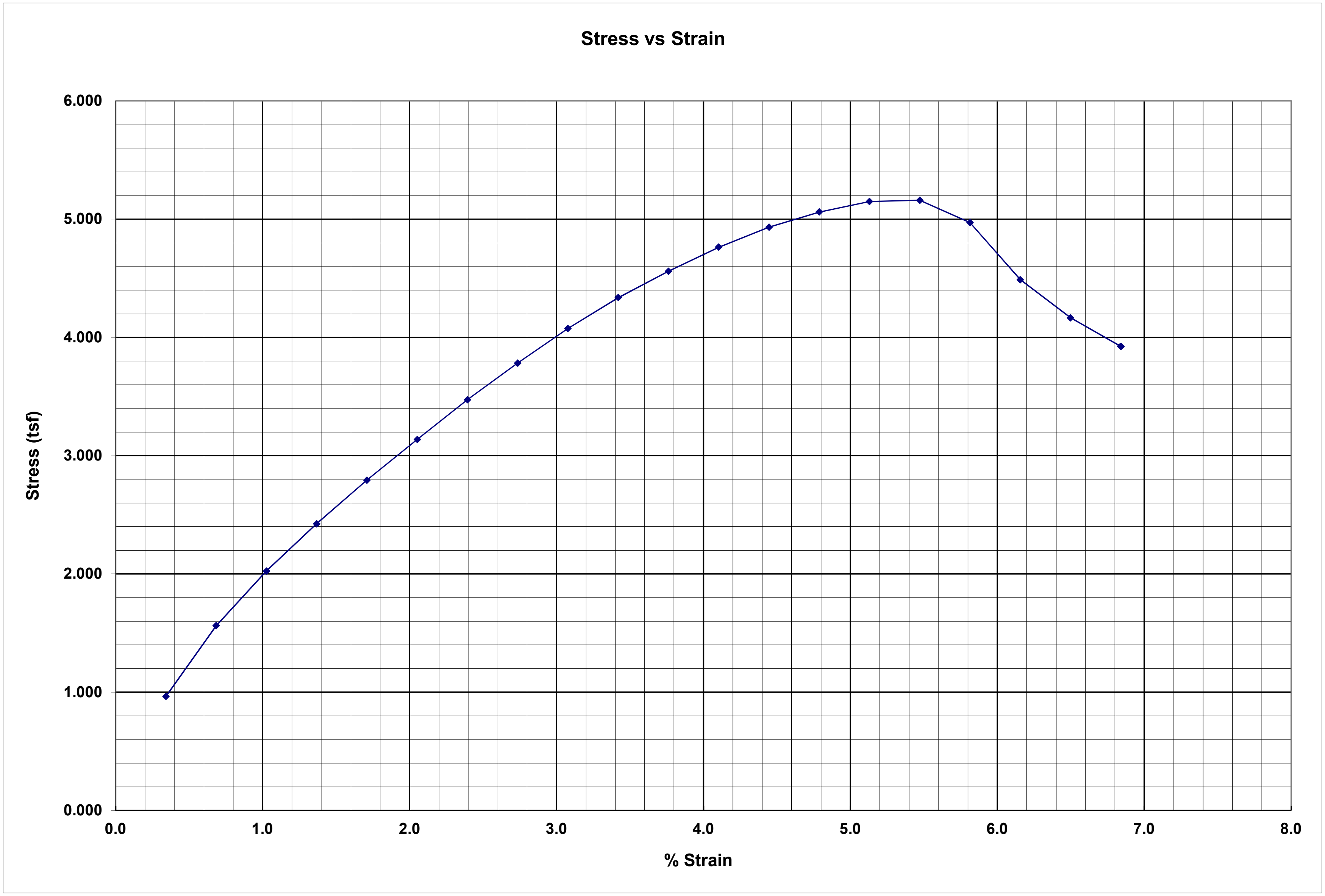
* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

TRIAXIAL COMPRESSION TEST STRESS/ STRAIN CURVE

Project: USACE-Brownsville Levee **Hole No.:** P3-32b **Sample No.:** 0 **Depth:** 9.1-11.1

TEAM Project No.: 142086 **Material:** Brown lean clay

Date: 1/6/15



TEAM Consultants, Inc.

TRIAXIAL TEST: UNCONFINED (ASTM D-2166) OR UNCONSOLIDATED-UNDRAINED (ASTM D-2850)

Project: USACE-Brownsville Levee
TEAM Project No.: 142086 **Date:** 1/6/15
Hole : P3-32b **Sample :** **Depth:** 13.5-15.5
Material: Brown lean clay

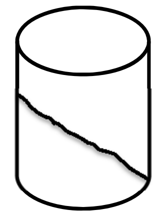
Height 1:	5.847 "	Dia.1:	2.883 "		
Height 2:	5.839 "	Dia.2:	2.862 "	Area:	6.475 In²
Height 3:	5.844 "	Dia.3:	2.869 "		
Young's Modulus for Membrane (tsf)	11.56				
Weight g:	1232.9	Strain Rate:	0.060	(Inches/Minute)	
Wet γ (pcf):	124.1	Strain Rate:	1.03	(%/Minute)	
Dry γ (pcf):	99.6	Length/Diameter Ratio:	2.035		
Test Type:	Unconfined Compression				
or	UU Triaxial @	12.6	psi	X	
Proving Ring Constant:	1				

Moisture Content (ASTM D 2216)

Before (cuttings) X After

Can-Dish No.:	677
Wet Wt. (Sple+Can):	432.2
Dry Wt. (Sple+Can):	372.7
Wt. of Can:	131.7
Wt. of Dry Soil:	241
Wt. of Water:	59.5
% Moisture:	24.7

GRAPHICAL DESCRIPTION OF FAILURE



Angular 60°

Confining Pressure (psi)	Dial Deflection	% Strain	Corrected Area (IN^2)	Load Dial Readings	Load Lbs	Deviator Stress (TSF) UNCORRECTED	Deviator Stress (TSF) CORRECTED*	Shearing Strength (cohesion)
12.6								
	0.020	0.342	6.497	60.7	60.7	0.673	0.672	0.336
	0.040	0.685	6.520	91.5	91.5	1.011	1.009	0.505
	0.060	1.027	6.542	113.7	113.7	1.252	1.250	0.625
	0.080	1.369	6.565	132.3	132.3	1.451	1.448	0.724
	0.100	1.711	6.588	147.9	147.9	1.617	1.613	0.807
	0.120	2.054	6.611	160.6	160.6	1.750	1.746	0.873
	0.140	2.396	6.634	171.2	171.2	1.858	1.854	0.927
	0.160	2.738	6.658	180.3	180.3	1.950	1.944	0.972
	0.180	3.080	6.681	187.8	187.8	2.024	2.018	1.009
	0.200	3.423	6.705	194.4	194.4	2.087	2.081	1.040
	0.220	3.765	6.729	200.2	200.2	2.142	2.135	1.068
	0.240	4.107	6.753	205.0	205.0	2.185	2.178	1.089
	0.260	4.450	6.777	209.2	209.2	2.223	2.215	1.107
	0.280	4.792	6.801	213.4	213.4	2.259	2.250	1.125
	0.300	5.134	6.826	216.3	216.3	2.282	2.272	1.136
	0.320	5.476	6.850	216.6	216.6	2.277	2.266	1.133
	0.340	5.819	6.875	218.7	218.7	2.291	2.280	1.140
	0.360	6.161	6.900	221.0	221.0	2.307	2.295	1.147
	0.380	6.503	6.926	223.2	223.2	2.320	2.308	1.154
	0.400	6.845	6.951	225.0	225.0	2.331	2.318	1.159
	0.420	7.188	6.977	226.1	226.1	2.334	2.320	1.160
	0.440	7.530	7.003	226.6	226.6	2.330	2.315	1.158
	0.460	7.872	7.029	227.3	227.3	2.329	2.314	1.157
	0.480	8.214	7.055	227.3	227.3	2.320	2.304	1.152
	0.500	8.557	7.081	227.8	227.8	2.316	2.300	1.150
	0.550	9.412	7.148	225.2	225.2	2.268	2.250	1.125
	0.600	10.268	7.216	222.0	222.0	2.215	2.195	1.097
	0.650	11.124	7.286	212.6	212.6	2.101	2.080	1.040
	0.700	11.979	7.357	193.3	193.3	1.892	1.869	0.934

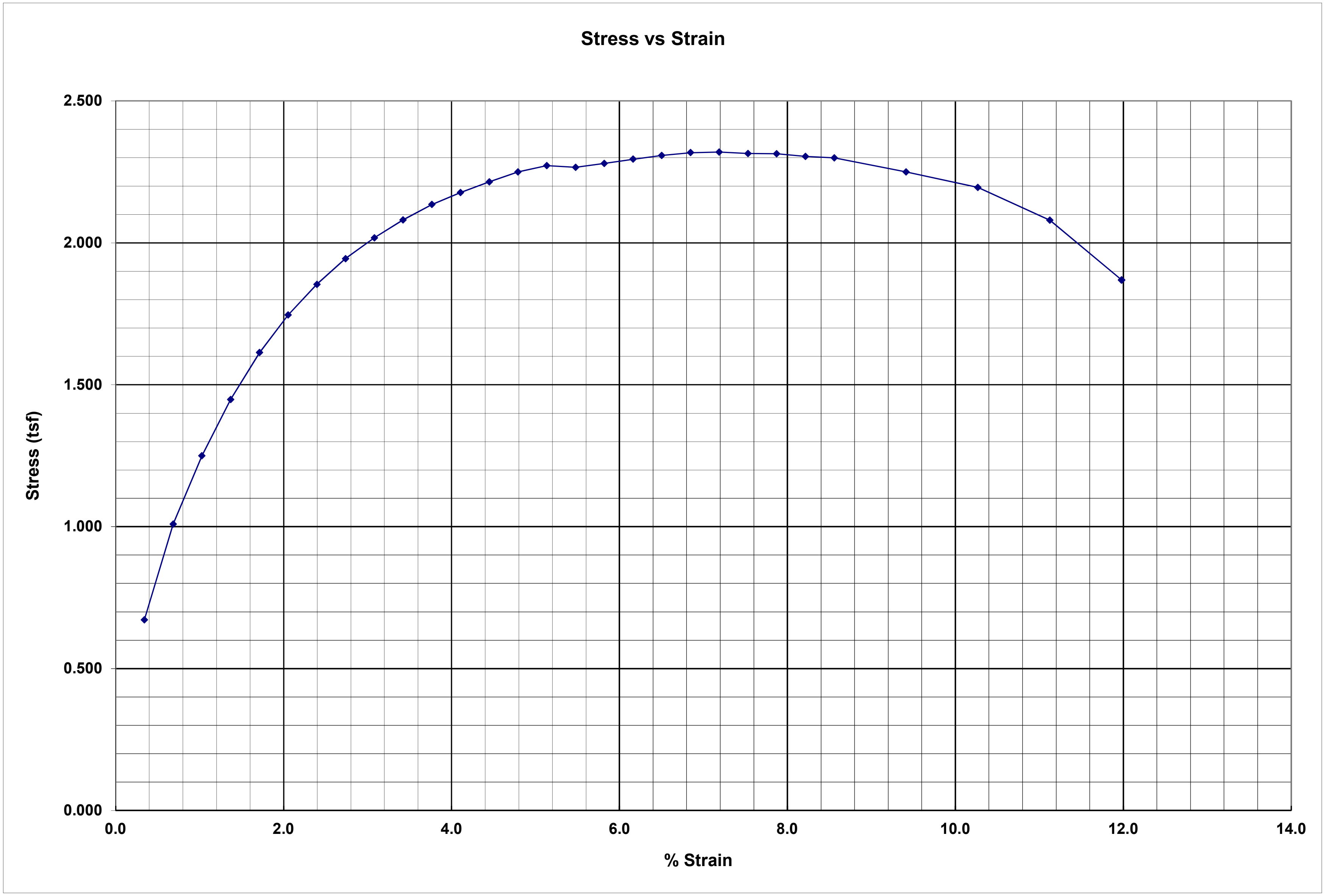
Strain (Inches/Inch) @ 50% Maximum Stress = 0.00899
Deformation @ 50% Maximum Stress (Inches)= 0.0525
Maximum Compressive Strength (TSF)= 2.32
% Strain @ Maximum Strength = 7.19%

Tested by: J. Young

* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

TRIAXIAL COMPRESSION TEST STRESS/ STRAIN CURVE

Project:	USACE-Brownsville Levee	**Hole No.:**	P3-32b	**Sample No.:**	0	**Depth:**	13.5-15.5
TEAM Project No.:	142086	**Material:**	Brown lean clay				
Date:	1/6/15						



TEAM Consultants, Inc.

TRIAXIAL TEST: UNCONFINED (ASTM D-2166) OR UNCONSOLIDATED-UNDRAINED (ASTM D-2850)

Project: USACE-Brownsville Levee
TEAM Project No.: 142086 **Date:** 1/6/15
Hole : P3-32b **Sample :** **Depth:** 24.5-26.5
Material: Dark brown lean clay

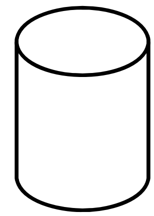
Height 1: 5.830 " **Dia.1:** 2.856 "
Height 2: 5.806 " **Dia.2:** 2.844 " **Area:** 6.402 In²
Height 3: 5.846 " **Dia.3:** 2.865 "
Young's Modulus for Membrane (tsf) 11.56
Weight g: 1170.4 **Strain Rate:** 0.030 (Inches/Minute)
Wet γ (pcf): 119.5 **Strain Rate:** 0.51 (%/Minute)
Dry γ (pcf): 91.2 **Length/Diameter Ratio:** 2.041
Test Type: Unconfined Compression
or UU Triaxial @ 22.1 psi X
Proving Ring Constant: 1

Moisture Content (ASTM D 2216)

Before (cuttings) X **After**

Can-Dish No.:	675
Wet Wt. (Sple+Can):	410.2
Dry Wt. (Sple+Can):	345.7
Wt. of Can:	137.6
Wt. of Dry Soil:	208.1
Wt. of Water:	64.5
% Moisture:	31.0

GRAPHICAL DESCRIPTION OF FAILURE



Internal

Confining Pressure (psi)	Dial Deflection	% Strain	Corrected Area (IN^2)	Load Dial Readings	Load Lbs	Deviator Stress (TSF) UNCORRECTED	Deviator Stress (TSF) CORRECTED*	Shearing Strength (cohesion)
22.1								
	0.020	0.343	6.424	11.3	11.3	0.127	0.126	0.063
	0.040	0.686	6.446	17.6	17.6	0.197	0.196	0.098
	0.060	1.030	6.468	23.5	23.5	0.261	0.259	0.130
	0.080	1.373	6.491	28.8	28.8	0.320	0.317	0.159
	0.100	1.716	6.514	33.7	33.7	0.373	0.369	0.185
	0.120	2.059	6.536	38.2	38.2	0.420	0.416	0.208
	0.140	2.402	6.559	42.3	42.3	0.464	0.460	0.230
	0.160	2.746	6.583	46.0	46.0	0.503	0.498	0.249
	0.180	3.089	6.606	49.4	49.4	0.538	0.532	0.266
	0.200	3.432	6.629	52.3	52.3	0.568	0.561	0.281
	0.220	3.775	6.653	55.0	55.0	0.596	0.588	0.294
	0.240	4.119	6.677	57.8	57.8	0.623	0.615	0.308
	0.260	4.462	6.701	60.1	60.1	0.646	0.637	0.319
	0.280	4.805	6.725	62.2	62.2	0.666	0.657	0.329
	0.300	5.148	6.749	64.1	64.1	0.684	0.674	0.337
	0.320	5.491	6.774	65.8	65.8	0.699	0.688	0.344
	0.340	5.835	6.798	67.4	67.4	0.714	0.703	0.352
	0.360	6.178	6.823	69.0	69.0	0.728	0.716	0.358
	0.380	6.521	6.848	70.4	70.4	0.740	0.727	0.364
	0.400	6.864	6.874	71.6	71.6	0.750	0.737	0.368
	0.420	7.207	6.899	72.8	72.8	0.760	0.746	0.373
	0.440	7.551	6.925	73.7	73.7	0.767	0.752	0.376
	0.460	7.894	6.950	74.8	74.8	0.775	0.760	0.380
	0.480	8.237	6.976	75.9	75.9	0.783	0.767	0.384
	0.500	8.580	7.003	76.8	76.8	0.790	0.773	0.387
	0.550	9.438	7.069	79.1	79.1	0.806	0.787	0.394
	0.600	10.296	7.137	80.9	80.9	0.817	0.797	0.398
	0.650	11.154	7.206	82.6	82.6	0.826	0.804	0.402
	0.700	12.012	7.276	84.5	84.5	0.836	0.813	0.406
	0.750	12.870	7.347	86.0	86.0	0.843	0.818	0.409
	0.800	13.728	7.421	87.5	87.5	0.849	0.823	0.411
	0.850	14.586	7.495	88.6	88.6	0.851	0.823	0.411
	0.870	14.930	7.525	89.2	89.2	0.854	0.825	0.412

Strain (Inches/Inch) @ 50% Maximum Stress = 0.02029
Deformation @ 50% Maximum Stress (Inches)= 0.1178
Maximum Compressive Strength (TSF)= 0.82
% Strain @ Maximum Strength = 14.93%

Tested by: J. Young

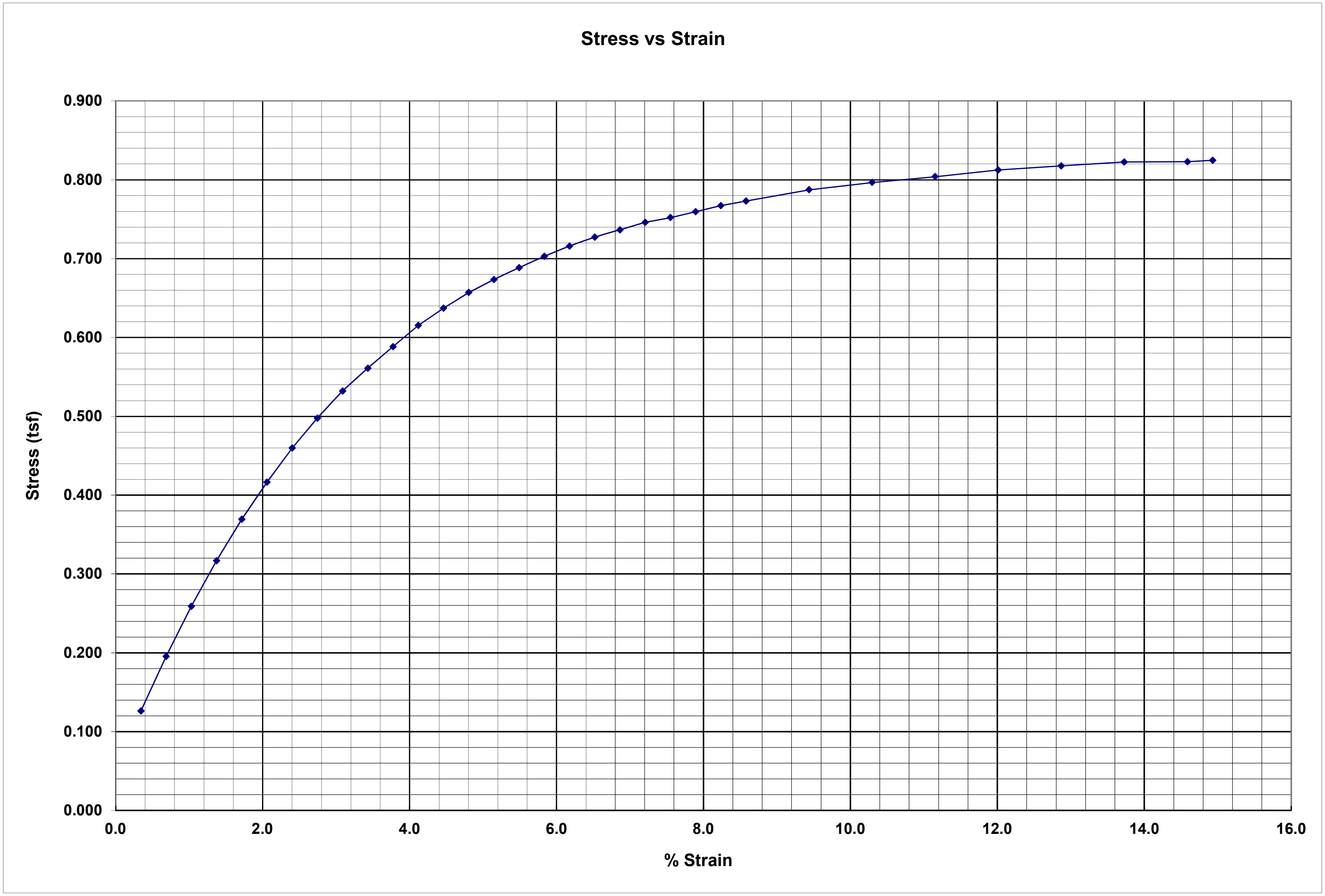
* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

TRIAXIAL COMPRESSION TEST STRESS/ STRAIN CURVE

Project: USACE-Brownsville Levee **Hole No.:** P3-32b **Sample No.:** 0 **Depth:** 24.5-26.5

TEAM Project No.: 142086 **Material:** Dark brown lean clay

Date: 1/6/15



TEAM Consultants, Inc.

TRIAXIAL TEST: UNCONFINED (ASTM D-2166) OR UNCONSOLIDATED-UNDRAINED (ASTM D-2850)

Project: USACE-Brownsville Levee **Hole :** P3-32b **Sample :** **Depth:** 26.7-28.7

TEAM Project No.: 142086 **Date:** 1/6/15 **Material:** Dark brown lean clay

Height 1:	5.854 "	Dia.1:	2.838 "
Height 2:	5.842 "	Dia.2:	2.856 " Area: 6.421 In²
Height 3:	5.841 "	Dia.3:	2.884 "

Young's Modulus for Membrane (tsf) 11.56

Weight g: 1211.7 Strain Rate: 0.060 (Inches/Minute)

Wet γ (pcf): 123.0 Strain Rate: 1.03 (%/Minute)

Dry γ (pcf): 97.4 Length/Diameter Ratio: 2.044

Test Type: Unconfined Compression

or UU Triaxial @ 24.0 psi X

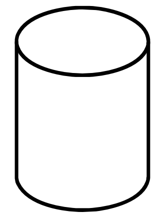
Proving Ring Constant: 1

Moisture Content (ASTM D 2216)

Before (cuttings) X After

Can-Dish No.:	687
Wet Wt. (Sple+Can):	421.2
Dry Wt. (Sple+Can):	363.5
Wt. of Can:	143.5
Wt. of Dry Soil:	220
Wt. of Water:	57.7
% Moisture:	26.2

GRAPHICAL DESCRIPTION OF FAILURE



Internal

Confining Pressure (psi)	Dial Deflection	% Strain	Corrected Area (IN²)	Load Dial Readings	Load Lbs	Deviator Stress (TSF) UNCORRECTED	Deviator Stress (TSF) CORRECTED*	Shearing Strength (cohesion)
24.0								
	0.020	0.342	6.443	14.3	14.3	0.159	0.159	0.079
	0.040	0.684	6.465	23.8	23.8	0.265	0.263	0.132
	0.060	1.026	6.488	33.3	33.3	0.369	0.367	0.184
	0.080	1.369	6.510	42.6	42.6	0.471	0.469	0.234
	0.100	1.711	6.533	51.5	51.5	0.568	0.564	0.282
	0.120	2.053	6.556	59.9	59.9	0.658	0.654	0.327
	0.140	2.395	6.579	67.1	67.1	0.735	0.730	0.365
	0.160	2.737	6.602	73.9	73.9	0.806	0.801	0.400
	0.180	3.079	6.625	79.9	79.9	0.868	0.862	0.431
	0.200	3.421	6.649	85.2	85.2	0.923	0.916	0.458
	0.220	3.763	6.672	90.1	90.1	0.973	0.965	0.483
	0.240	4.106	6.696	94.4	94.4	1.015	1.007	0.504
	0.260	4.448	6.720	98.3	98.3	1.053	1.044	0.522
	0.280	4.790	6.744	101.3	101.3	1.082	1.072	0.536
	0.300	5.132	6.769	104.1	104.1	1.107	1.097	0.549
	0.320	5.474	6.793	106.7	106.7	1.131	1.120	0.560
	0.340	5.816	6.818	109.0	109.0	1.151	1.140	0.570
	0.360	6.158	6.843	111.3	111.3	1.171	1.159	0.580
	0.380	6.501	6.868	113.3	113.3	1.188	1.175	0.588
	0.400	6.843	6.893	115.1	115.1	1.203	1.189	0.595
	0.420	7.185	6.918	116.7	116.7	1.214	1.200	0.600
	0.440	7.527	6.944	118.3	118.3	1.227	1.213	0.606
	0.460	7.869	6.970	120.0	120.0	1.240	1.225	0.612
	0.480	8.211	6.996	121.7	121.7	1.253	1.237	0.618
	0.500	8.553	7.022	123.2	123.2	1.264	1.247	0.624
	0.550	9.409	7.088	126.2	126.2	1.282	1.263	0.632
	0.600	10.264	7.156	128.5	128.5	1.293	1.273	0.636
	0.650	11.119	7.225	131.4	131.4	1.309	1.288	0.644
	0.700	11.975	7.295	133.5	133.5	1.318	1.295	0.647
	0.750	12.830	7.366	136.3	136.3	1.332	1.307	0.654
	0.800	13.685	7.439	137.8	137.8	1.334	1.307	0.654
	0.850	14.541	7.514	140.3	140.3	1.344	1.316	0.658
	0.870	14.883	7.544	140.9	140.9	1.345	1.316	0.658

Strain (Inches/Inch) @ 50% Maximum Stress = 0.02069

Deformation @ 50% Maximum Stress (Inches)= 0.1209 **Tested by:** J. Young

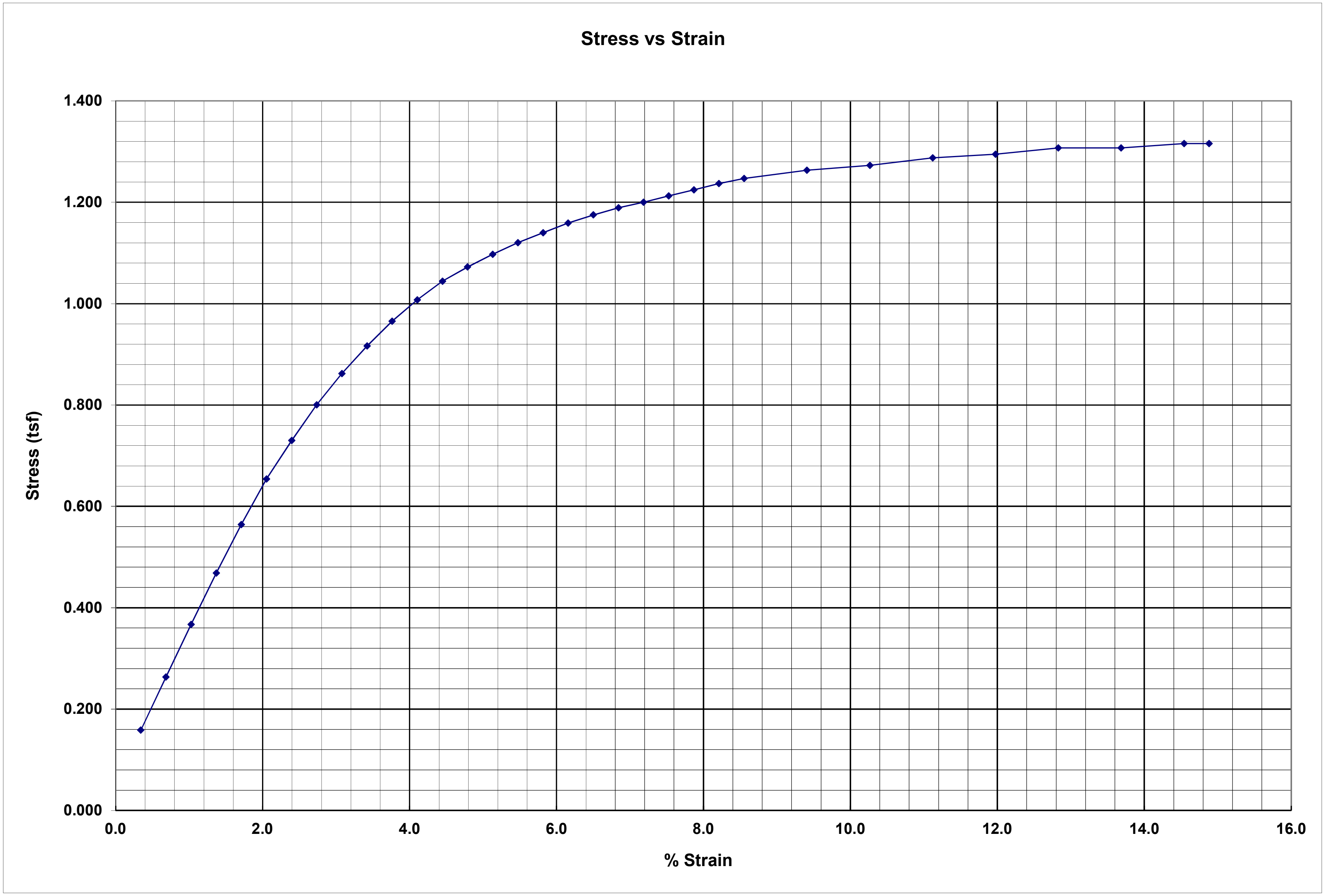
Maximum Compressive Strength (TSF)= 1.32

% Strain @ Maximum Strength = 14.54%

* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

TRIAXIAL COMPRESSION TEST STRESS/ STRAIN CURVE

Project:	USACE-Brownsville Levee	**Hole No.:**	P3-32b
Sample No.:	0	**Depth:**	26.7-28.7
TEAM Project No.:	142086	**Material:**	Dark brown lean clay
Date:	1/6/15		



TRIAXIAL TEST: UNCONFINED (ASTM D-2166) OR UNCONSOLIDATED-UNDRAINED (ASTM D-2850)

Project: USACE-Brownsville Levee
TEAM Project No.: 142086 **Date:** 1/6/15
Hole : P3-32 **Sample :** **Depth:** 29-31
Material: Dark brown lean clay

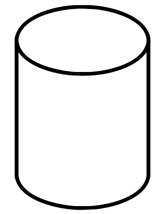
Height 1: 5.854 " **Dia.1:** 2.848 "
Height 2: 5.834 " **Dia.2:** 2.844 " **Area:** 6.384 In^2
Height 3: 5.847 " **Dia.3:** 2.861 "
Young's Modulus for Membrane (tsf) 11.56
Weight g: 1198.5 **Strain Rate:** 0.060 (Inches/Minute)
Wet γ (pcf): 122.4 **Strain Rate:** 1.03 (%/Minute)
Dry γ (pcf): 95.7 **Length/Diameter Ratio:** 2.050
Test Type: Unconfined Compression
or UU Triaxial @ 26.0 psi X
Proving Ring Constant: 1

Moisture Content (ASTM D 2216)

Before (cuttings) X **After**

Can-Dish No.:	678
Wet Wt. (Sple+Can):	336.6
Dry Wt. (Sple+Can):	291.1
Wt. of Can:	127.4
Wt. of Dry Soil:	163.7
Wt. of Water:	45.5
% Moisture:	27.8

GRAPHICAL DESCRIPTION OF FAILURE



Internal

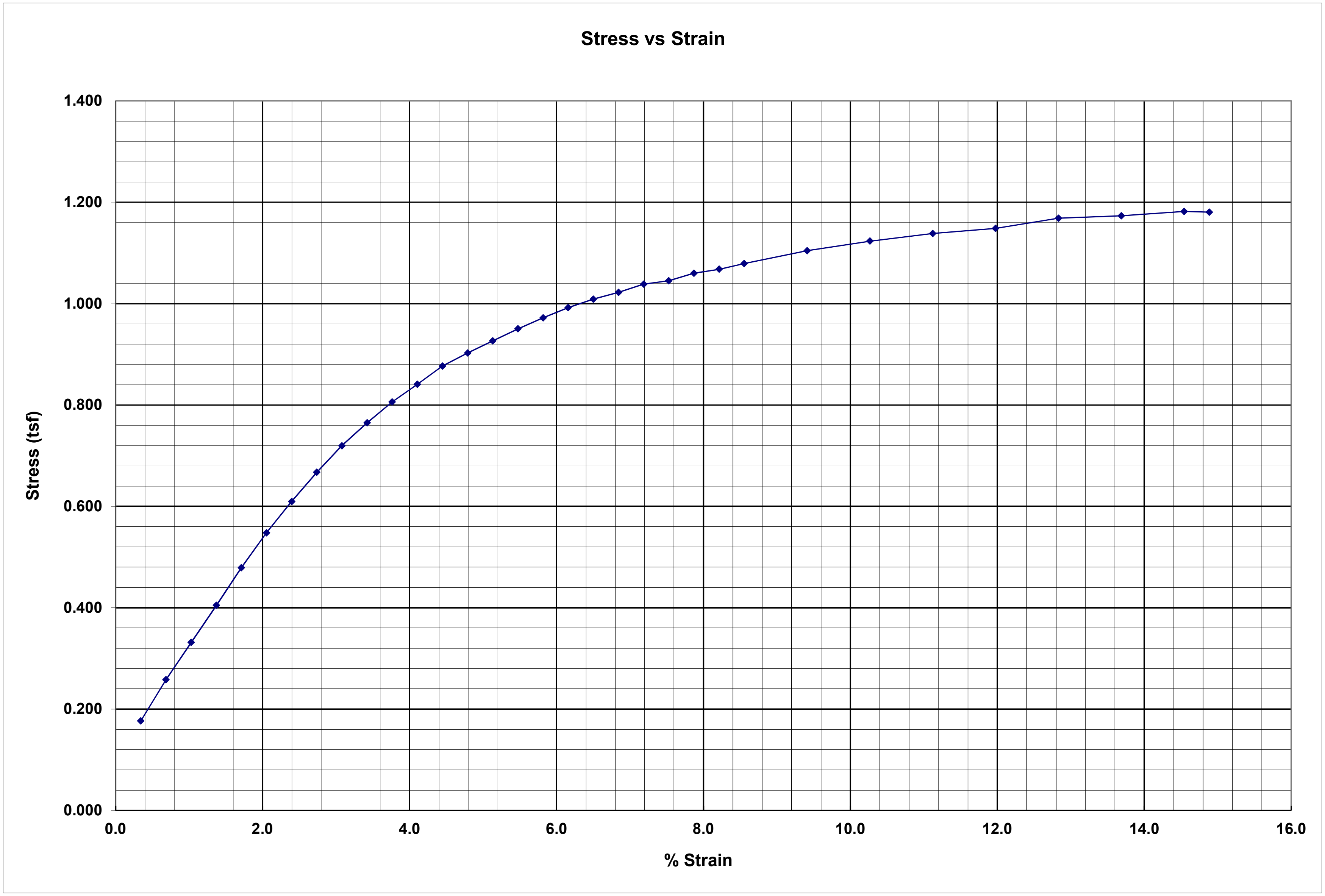
Confining Pressure (psi)	Dial Deflection	% Strain	Corrected Area (IN^2)	Load Dial Readings	Load Lbs	Deviator Stress (TSF) UNCORRECTED	Deviator Stress (TSF) CORRECTED*	Shearing Strength (cohesion)
26.0								
	0.020	0.342	6.406	15.8	15.8	0.177	0.177	0.088
	0.040	0.684	6.428	23.1	23.1	0.259	0.258	0.129
	0.060	1.027	6.450	29.9	29.9	0.334	0.332	0.166
	0.080	1.369	6.472	36.6	36.6	0.408	0.405	0.202
	0.100	1.711	6.495	43.5	43.5	0.483	0.479	0.240
	0.120	2.053	6.518	50.0	50.0	0.552	0.548	0.274
	0.140	2.395	6.541	55.8	55.8	0.614	0.610	0.305
	0.160	2.737	6.564	61.3	61.3	0.673	0.667	0.334
	0.180	3.080	6.587	66.4	66.4	0.726	0.720	0.360
	0.200	3.422	6.610	70.8	70.8	0.771	0.765	0.382
	0.220	3.764	6.634	75.0	75.0	0.814	0.806	0.403
	0.240	4.106	6.657	78.5	78.5	0.849	0.841	0.420
	0.260	4.448	6.681	82.2	82.2	0.886	0.877	0.438
	0.280	4.790	6.705	84.9	84.9	0.912	0.903	0.451
	0.300	5.133	6.729	87.5	87.5	0.937	0.927	0.463
	0.320	5.475	6.754	90.1	90.1	0.961	0.950	0.475
	0.340	5.817	6.778	92.6	92.6	0.984	0.972	0.486
	0.360	6.159	6.803	94.9	94.9	1.004	0.992	0.496
	0.380	6.501	6.828	96.9	96.9	1.022	1.009	0.505
	0.400	6.843	6.853	98.6	98.6	1.036	1.022	0.511
	0.420	7.186	6.878	100.6	100.6	1.053	1.039	0.519
	0.440	7.528	6.904	101.6	101.6	1.060	1.045	0.523
	0.460	7.870	6.929	103.5	103.5	1.075	1.060	0.530
	0.480	8.212	6.955	104.7	104.7	1.084	1.068	0.534
	0.500	8.554	6.981	106.2	106.2	1.096	1.079	0.540
	0.550	9.410	7.047	109.9	109.9	1.123	1.105	0.552
	0.600	10.265	7.114	113.0	113.0	1.143	1.123	0.562
	0.650	11.121	7.183	115.7	115.7	1.160	1.139	0.569
	0.700	11.976	7.252	118.0	118.0	1.172	1.149	0.574
	0.750	12.831	7.324	121.4	121.4	1.194	1.169	0.584
	0.800	13.687	7.396	123.2	123.2	1.200	1.173	0.587
	0.850	14.542	7.470	125.5	125.5	1.210	1.182	0.591
	0.870	14.885	7.500	126.0	126.0	1.210	1.181	0.590

Strain (Inches/Inch) @ 50% Maximum Stress = 0.02291
Deformation @ 50% Maximum Stress (Inches)= 0.1337 **Tested by:** J. Young
Maximum Compressive Strength (TSF)= 1.18
% Strain @ Maximum Strength = 14.54%

* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

TRIAXIAL COMPRESSION TEST STRESS/ STRAIN CURVE

Project:	USACE-Brownsville Levee	**Hole No.:**	P3-32	**Sample No.:**	0	**Depth:**	29-31
TEAM Project No.:	142086	**Material:**	Dark brown lean clay				
Date:	1/6/15						



TEAM Consultants, Inc.

TRIAXIAL TEST: UNCONFINED (ASTM D-2166) OR UNCONSOLIDATED-UNDRAINED (ASTM D-2850)

Project: USACE-Brownsville Levee | **Hole :** P3-32 | **Sample :** | **Depth:** 31.2-33.2

TEAM Project No.: 142086 | **Date:** 1/6/15 | **Material:** Brown fat clay

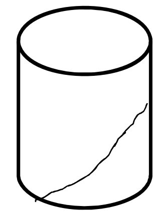
Height 1: 5.860 " **Dia.1:** 2.876 "
Height 2: 5.836 " **Dia.2:** 2.860 " **Area:** 6.456 In²
Height 3: 5.830 " **Dia.3:** 2.865 "
Young's Modulus for Membrane (tsf) 11.56
Weight g: 1221.5 **Strain Rate:** 0.060 (Inches/Minute)
Wet γ (pcf): 123.4 **Strain Rate:** 1.03 (%/Minute)
Dry γ (pcf): 95.5 **Length/Diameter Ratio:** 2.038
Test Type: Unconfined Compression
or UU Triaxial @ 27.5 psi X
Proving Ring Constant: 1

Moisture Content (ASTM D 2216)

Before (cuttings) X **After**

Can-Dish No.:	498
Wet Wt. (Sple+Can):	354
Dry Wt. (Sple+Can):	306.9
Wt. of Can:	145.4
Wt. of Dry Soil:	161.5
Wt. of Water:	47.1
% Moisture:	29.2

GRAPHICAL DESCRIPTION OF FAILURE



Angular 55° (slickensided)

Confining Pressure (psi)	Dial Deflection	% Strain	Corrected Area (IN^2)	Load Dial Readings	Load Lbs	Deviator Stress (TSF) UNCORRECTED	Deviator Stress (TSF) CORRECTED*	Shearing Strength (cohesion)
27.5								
	0.020	0.342	6.478	60.2	60.2	0.670	0.669	0.334
	0.040	0.685	6.500	88.0	88.0	0.975	0.973	0.487
	0.060	1.027	6.523	104.7	104.7	1.156	1.154	0.577
	0.080	1.369	6.545	115.7	115.7	1.273	1.271	0.635
	0.100	1.712	6.568	123.9	123.9	1.358	1.355	0.677
	0.120	2.054	6.591	130.0	130.0	1.420	1.416	0.708
	0.140	2.396	6.614	135.2	135.2	1.472	1.467	0.734
	0.160	2.739	6.638	139.5	139.5	1.513	1.508	0.754
	0.180	3.081	6.661	143.3	143.3	1.549	1.543	0.772
	0.200	3.423	6.685	146.4	146.4	1.577	1.570	0.785
	0.220	3.766	6.708	149.2	149.2	1.601	1.594	0.797
	0.240	4.108	6.732	151.6	151.6	1.621	1.614	0.807
	0.260	4.451	6.756	153.6	153.6	1.637	1.628	0.814
	0.280	4.793	6.781	155.6	155.6	1.652	1.643	0.821
	0.300	5.135	6.805	157.4	157.4	1.666	1.656	0.828
	0.320	5.478	6.830	159.1	159.1	1.678	1.667	0.833
	0.340	5.820	6.855	160.3	160.3	1.684	1.673	0.837
	0.360	6.162	6.880	161.6	161.6	1.691	1.679	0.840
	0.380	6.505	6.905	162.6	162.6	1.696	1.683	0.842
	0.400	6.847	6.930	163.7	163.7	1.701	1.688	0.844
	0.420	7.189	6.956	164.6	164.6	1.704	1.690	0.845
	0.440	7.532	6.982	165.2	165.2	1.704	1.690	0.845
	0.460	7.874	7.008	165.9	165.9	1.704	1.689	0.845
	0.480	8.216	7.034	166.2	166.2	1.701	1.685	0.843
	0.500	8.559	7.060	166.8	166.8	1.701	1.684	0.842
	0.520	8.901	7.087	166.6	166.6	1.693	1.676	0.838
	0.540	9.243	7.113	166.3	166.3	1.684	1.666	0.833
	0.560	9.586	7.140	165.9	165.9	1.673	1.654	0.827
	0.580	9.928	7.167	165.1	165.1	1.659	1.639	0.820
	0.600	10.270	7.195	163.7	163.7	1.638	1.619	0.809
	0.620	10.613	7.222	161.9	161.9	1.614	1.593	0.797
	0.640	10.955	7.250	159.9	159.9	1.588	1.567	0.783
	0.660	11.298	7.278	159.0	159.0	1.573	1.551	0.775

Strain (Inches/Inch) @ 50% Maximum Stress = 0.00541

Deformation @ 50% Maximum Stress (Inches)= 0.0315 **Tested by:**

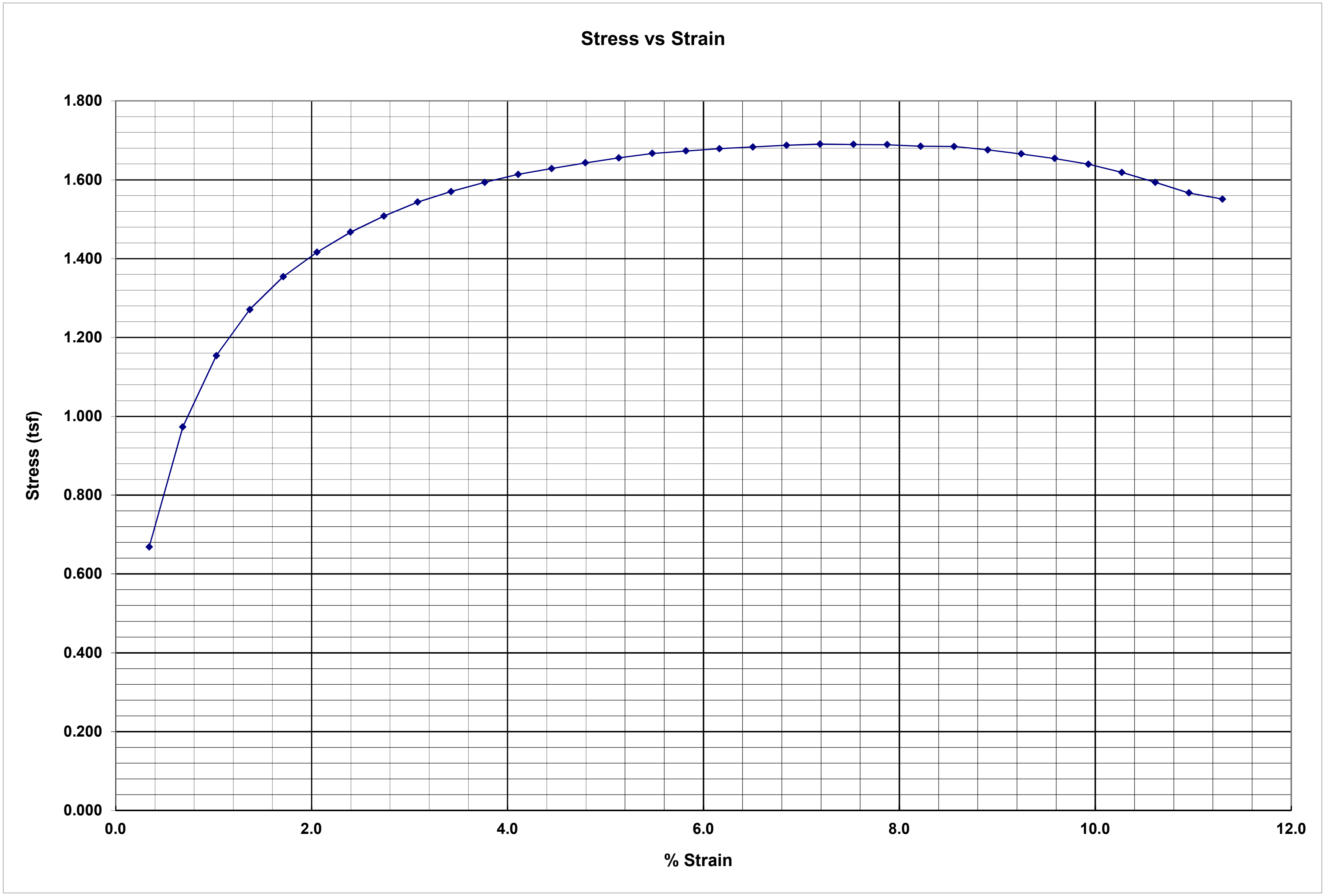
Maximum Compressive Strength (TSF)= 1.69

% Strain @ Maximum Strength = 7.19%

* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

TRIAXIAL COMPRESSION TEST STRESS/ STRAIN CURVE

Project:	USACE-Brownsville Levee	**Hole No.:**	P3-32	**Sample No.:**	0	**Depth:**	31.2-33.2
TEAM Project No.:	142086	**Material:**	Brown fat clay				
Date:	1/6/15						



TEAM Consultants, Inc.

TRIAXIAL TEST: UNCONFINED (ASTM D-2166) OR UNCONSOLIDATED-UNDRAINED (ASTM D-2850)

Project: USACE-Brownsville Levee
TEAM Project No.: 142086 **Date:** 1/6/15
Hole : P3-32 **Sample :** **Depth:** 35.6-37.6
Material: Brown fat clay

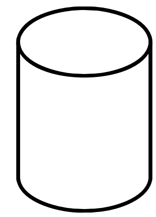
Height 1: 5.840 " **Dia.1:** 2.862 "
Height 2: 5.849 " **Dia.2:** 2.875 " **Area:** 6.471 In²
Height 3: 5.849 " **Dia.3:** 2.874 "
Young's Modulus for Membrane (tsf) 11.56
Weight g: 1227.4 **Strain Rate:** 0.060 (Inches/Minute)
Wet γ (pcf): 123.6 **Strain Rate:** 1.03 (%/Minute)
Dry γ (pcf): 98.3 **Length/Diameter Ratio:** 2.037
Test Type: Unconfined Compression
or UU Triaxial @ 31.8 psi X
Proving Ring Constant: 1

Moisture Content (ASTM D 2216)

Before (cuttings) X **After**

Can-Dish No.:	689
Wet Wt. (Sple+Can):	364
Dry Wt. (Sple+Can):	318.2
Wt. of Can:	140.1
Wt. of Dry Soil:	178.1
Wt. of Water:	45.8
% Moisture:	25.7

GRAPHICAL DESCRIPTION OF FAILURE



Internal

Confining Pressure (psi)	Dial Deflection	% Strain	Corrected Area (IN^2)	Load Dial Readings	Load Lbs	Deviator Stress (TSF) UNCORRECTED	Deviator Stress (TSF) CORRECTED*	Shearing Strength (cohesion)
31.8								
	0.020	0.342	6.493	32.5	32.5	0.360	0.360	0.180
	0.040	0.684	6.515	49.5	49.5	0.547	0.546	0.273
	0.060	1.026	6.538	61.5	61.5	0.677	0.675	0.338
	0.080	1.368	6.561	70.1	70.1	0.769	0.766	0.383
	0.100	1.711	6.583	76.8	76.8	0.840	0.837	0.418
	0.120	2.053	6.606	82.0	82.0	0.894	0.890	0.445
	0.140	2.395	6.630	86.3	86.3	0.937	0.933	0.466
	0.160	2.737	6.653	89.7	89.7	0.971	0.965	0.483
	0.180	3.079	6.676	92.7	92.7	1.000	0.994	0.497
	0.200	3.421	6.700	95.2	95.2	1.023	1.016	0.508
	0.220	3.763	6.724	97.3	97.3	1.042	1.035	0.518
	0.240	4.105	6.748	99.3	99.3	1.060	1.052	0.526
	0.260	4.447	6.772	101.2	101.2	1.076	1.067	0.534
	0.280	4.790	6.796	102.9	102.9	1.090	1.081	0.540
	0.300	5.132	6.821	104.4	104.4	1.102	1.092	0.546
	0.320	5.474	6.845	105.8	105.8	1.113	1.102	0.551
	0.340	5.816	6.870	107.0	107.0	1.121	1.110	0.555
	0.360	6.158	6.895	108.2	108.2	1.130	1.118	0.559
	0.380	6.500	6.921	109.3	109.3	1.137	1.125	0.562
	0.400	6.842	6.946	110.2	110.2	1.143	1.129	0.565
	0.420	7.184	6.972	111.1	111.1	1.148	1.134	0.567
	0.440	7.527	6.997	112.1	112.1	1.153	1.139	0.569
	0.460	7.869	7.023	112.8	112.8	1.157	1.141	0.571
	0.480	8.211	7.050	113.7	113.7	1.162	1.146	0.573
	0.500	8.553	7.076	114.7	114.7	1.167	1.150	0.575
	0.550	9.408	7.143	116.5	116.5	1.174	1.156	0.578
	0.600	10.263	7.211	118.2	118.2	1.180	1.160	0.580
	0.650	11.119	7.280	119.9	119.9	1.186	1.164	0.582
	0.700	11.974	7.351	121.3	121.3	1.188	1.165	0.582
	0.750	12.829	7.423	122.6	122.6	1.190	1.165	0.582
	0.800	13.685	7.497	124.2	124.2	1.193	1.166	0.583
	0.850	14.540	7.572	125.7	125.7	1.195	1.167	0.584
	0.900	15.395	7.648	126.6	126.6	1.192	1.162	0.581

Strain (Inches/Inch) @ 50% Maximum Stress = 0.00784
Deformation @ 50% Maximum Stress (Inches)= 0.0459
Maximum Compressive Strength (TSF)= 1.17
% Strain @ Maximum Strength = 14.54%

Tested by: J. Young

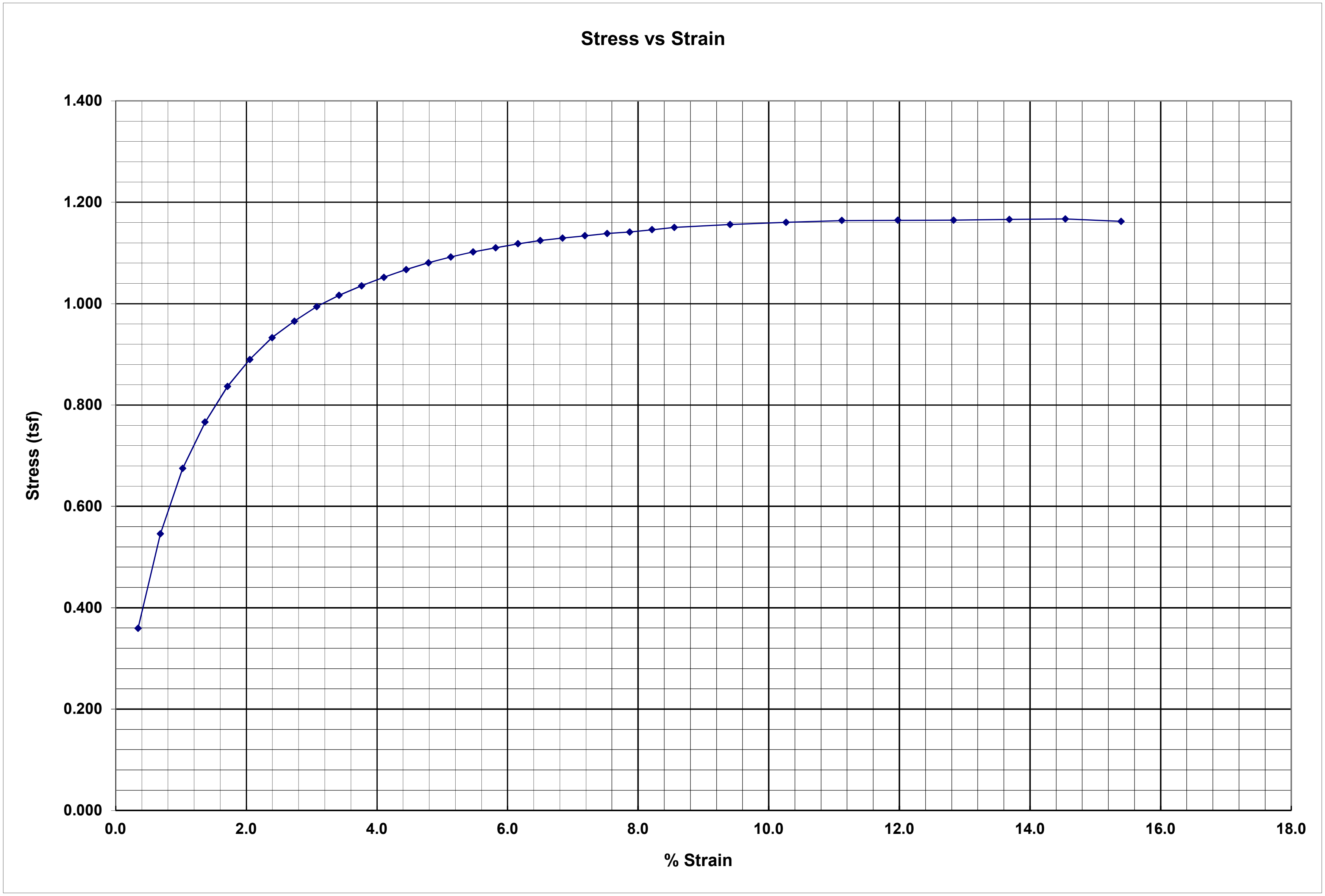
* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

TRIAXIAL COMPRESSION TEST STRESS/ STRAIN CURVE

Project: USACE-Brownsville Levee **Hole No.:** P3-32 **Sample No.:** 0 **Depth:** 35.6-37.6

TEAM Project No.: 142086 **Material:** Brown fat clay

Date: 1/6/15



TRIAXIAL TEST: UNCONFINED (ASTM D-2166) OR UNCONSOLIDATED-UNDRAINED (ASTM D-2850)

Project: USACE-Brownsville Levee **Hole :** P3-33 **Sample :** **Depth:** 6.4-8.4

TEAM Project No.: 142086 **Date:** 1/6/15 **Material:** Brown lean clay

Height 1: 5.840 " **Dia.1:** 2.858 "

Height 2: 5.846 " **Dia.2:** 2.849 " **Area:** 6.351 In²

Height 3: 5.842 " **Dia.3:** 2.824 "

Young's Modulus for Membrane (tsf) 11.56

Weight g: 1188.3 **Strain Rate:** 0.060 (Inches/Minute)

Wet γ (pcf): 122.0 **Strain Rate:** 1.03 (%/Minute)

Dry γ (pcf): 97.9 **Length/Diameter Ratio:** 2.055

Test Type: Unconfined Compression

or UU Triaxial @ 6.4 psi X

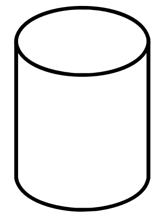
Proving Ring Constant: 1

Moisture Content (ASTM D 2216)

Before (cuttings) X **After**

Can-Dish No.:	464
Wet Wt. (Sple+Can):	357.5
Dry Wt. (Sple+Can):	315.4
Wt. of Can:	144.7
Wt. of Dry Soil:	170.7
Wt. of Water:	42.1
% Moisture:	24.7

GRAPHICAL DESCRIPTION OF FAILURE



Internal

Confining Pressure (psi)	Dial Deflection	% Strain	Corrected Area (IN^2)	Load Dial Readings	Load Lbs	Deviator Stress (TSF) UNCORRECTED	Deviator Stress (TSF) CORRECTED*	Shearing Strength (cohesion)
6.4								
	0.020	0.342	6.373	7.1	7.1	0.080	0.079	0.040
	0.040	0.685	6.395	11.5	11.5	0.129	0.128	0.064
	0.060	1.027	6.417	15.9	15.9	0.179	0.177	0.088
	0.080	1.369	6.439	20.1	20.1	0.225	0.222	0.111
	0.100	1.712	6.462	23.8	23.8	0.265	0.261	0.131
	0.120	2.054	6.484	27.3	27.3	0.303	0.299	0.149
	0.140	2.396	6.507	31.1	31.1	0.344	0.340	0.170
	0.160	2.738	6.530	34.6	34.6	0.382	0.377	0.188
	0.180	3.081	6.553	38.2	38.2	0.419	0.413	0.207
	0.200	3.423	6.576	41.5	41.5	0.455	0.448	0.224
	0.220	3.765	6.600	44.8	44.8	0.488	0.481	0.241
	0.240	4.108	6.623	48.3	48.3	0.525	0.517	0.258
	0.260	4.450	6.647	51.7	51.7	0.560	0.551	0.275
	0.280	4.792	6.671	54.4	54.4	0.587	0.578	0.289
	0.300	5.135	6.695	57.0	57.0	0.613	0.603	0.302
	0.320	5.477	6.719	59.5	59.5	0.637	0.627	0.313
	0.340	5.819	6.743	61.9	61.9	0.661	0.650	0.325
	0.360	6.162	6.768	64.2	64.2	0.683	0.671	0.336
	0.380	6.504	6.793	66.2	66.2	0.702	0.689	0.345
	0.400	6.846	6.818	68.4	68.4	0.722	0.709	0.354
	0.420	7.188	6.843	70.1	70.1	0.737	0.723	0.362
	0.440	7.531	6.868	72.0	72.0	0.755	0.741	0.370
	0.460	7.873	6.894	73.7	73.7	0.770	0.755	0.377
	0.480	8.215	6.920	75.4	75.4	0.785	0.769	0.384
	0.500	8.558	6.945	77.1	77.1	0.799	0.783	0.391
	0.550	9.414	7.011	80.2	80.2	0.823	0.805	0.403
	0.600	10.269	7.078	83.2	83.2	0.847	0.827	0.413
	0.650	11.125	7.146	86.3	86.3	0.870	0.848	0.424
	0.700	11.981	7.216	89.2	89.2	0.890	0.867	0.433
	0.750	12.837	7.286	91.8	91.8	0.907	0.882	0.441
	0.800	13.692	7.359	94.1	94.1	0.921	0.894	0.447
	0.850	14.548	7.432	96.4	96.4	0.934	0.906	0.453
	0.870	14.890	7.462	97.2	97.2	0.938	0.909	0.454

Strain (Inches/Inch) @ 50% Maximum Stress = 0.03488

Deformation @ 50% Maximum Stress (Inches)= 0.2039

Maximum Compressive Strength (TSF)= 0.91

% Strain @ Maximum Strength = 14.89%

Tested by: J. Young

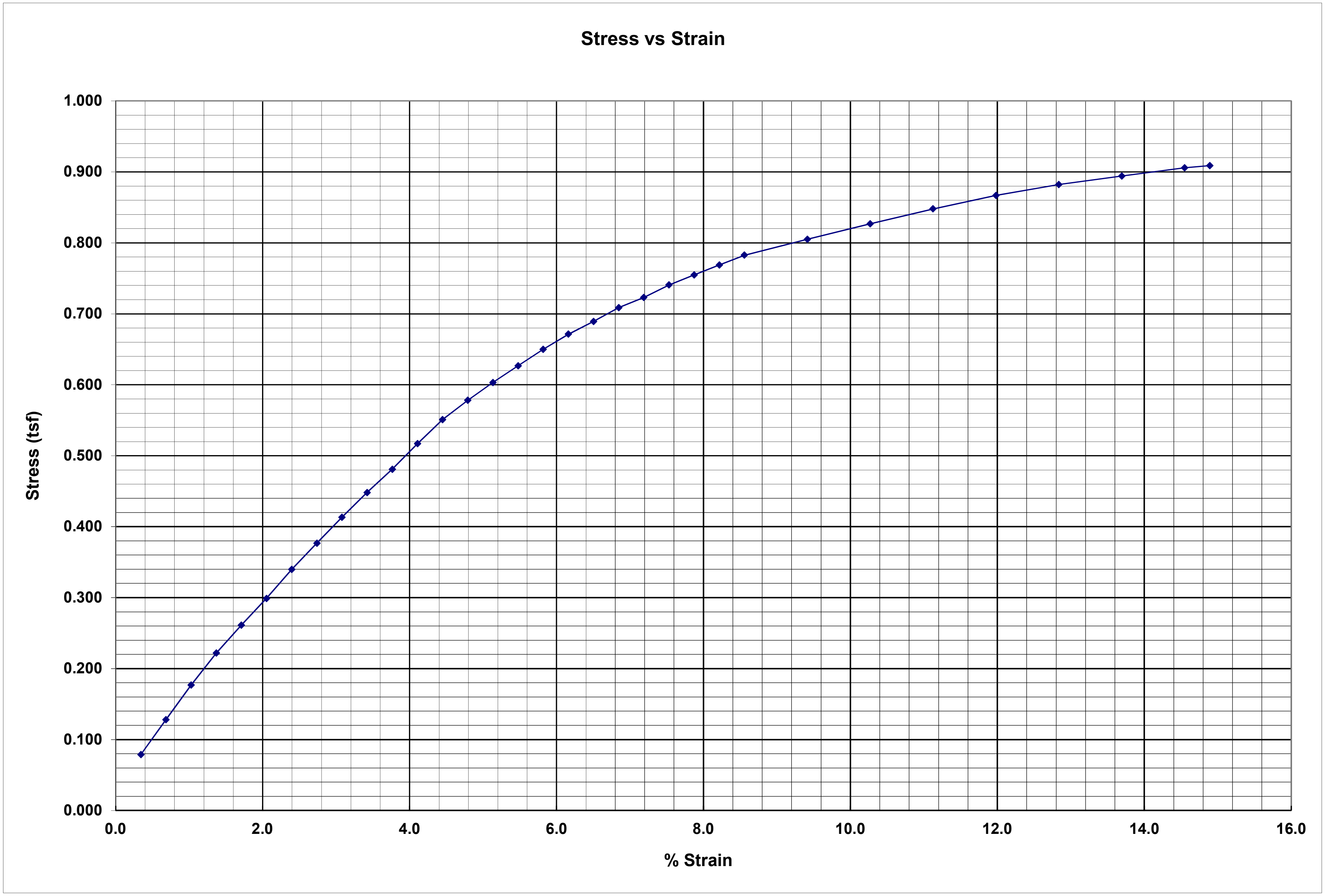
* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

TRIAXIAL COMPRESSION TEST STRESS/ STRAIN CURVE

Project: USACE-Brownsville Levee **Hole No.:** P3-33 **Sample No.:** 0 **Depth:** 6.4-8.4

TEAM Project No.: 142086 **Material:** Brown lean clay

Date: 1/6/15



TEAM Consultants, Inc.

TRIAXIAL TEST: UNCONFINED (ASTM D-2166) OR UNCONSOLIDATED-UNDRAINED (ASTM D-2850)

Project: USACE-Brownsville Levee **Hole :** P3-33 **Sample :** **Depth:** 10.8-12.8

TEAM Project No.: 142086 **Date:** 1/6/15 **Material:** Brown silty clay with sand

Height 1: 5.830 " **Dia.1:** 2.783 "

Height 2: 5.816 " **Dia.2:** 2.757 " **Area:** 6.028 In²

Height 3: 5.840 " **Dia.3:** 2.771 "

Young's Modulus for Membrane (tsf) 11.56

Weight g: 1123.7 **Strain Rate:** 0.060 (Inches/Minute)

Wet γ (pcf): 121.8 **Strain Rate:** 1.03 (%/Minute)

Dry γ (pcf): 96.2 **Length/Diameter Ratio:** 2.104

Test Type: Unconfined Compression

or UU Triaxial @ 10.2 psi X

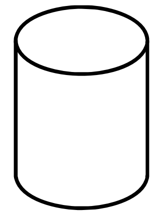
Proving Ring Constant: 1

Moisture Content (ASTM D 2216)

Before (cuttings) X **After**

Can-Dish No.:	457
Wet Wt. (Sple+Can):	399.2
Dry Wt. (Sple+Can):	345.8
Wt. of Can:	145.2
Wt. of Dry Soil:	200.6
Wt. of Water:	53.4
% Moisture:	26.6

GRAPHICAL DESCRIPTION OF FAILURE



Internal

Confining Pressure (psi)	Dial Deflection	% Strain	Corrected Area (IN^2)	Load Dial Readings	Load Lbs	Deviator Stress (TSF) UNCORRECTED	Deviator Stress (TSF) CORRECTED*	Shearing Strength (cohesion)
10.2								
	0.020	0.343	6.048	3.7	3.7	0.044	0.043	0.022
	0.040	0.686	6.069	5.1	5.1	0.060	0.059	0.029
	0.060	1.029	6.090	6.4	6.4	0.076	0.074	0.037
	0.080	1.373	6.112	7.7	7.7	0.090	0.088	0.044
	0.100	1.716	6.133	8.7	8.7	0.103	0.099	0.050
	0.120	2.059	6.154	10.1	10.1	0.118	0.114	0.057
	0.140	2.402	6.176	11.5	11.5	0.134	0.129	0.065
	0.160	2.745	6.198	12.7	12.7	0.148	0.142	0.071
	0.180	3.088	6.220	14.1	14.1	0.163	0.157	0.079
	0.200	3.431	6.242	15.6	15.6	0.180	0.173	0.087
	0.220	3.774	6.264	17.0	17.0	0.196	0.188	0.094
	0.240	4.118	6.287	18.2	18.2	0.209	0.201	0.100
	0.260	4.461	6.309	19.5	19.5	0.222	0.213	0.107
	0.280	4.804	6.332	20.8	20.8	0.237	0.227	0.114
	0.300	5.147	6.355	22.4	22.4	0.254	0.243	0.122
	0.320	5.490	6.378	23.6	23.6	0.267	0.256	0.128
	0.340	5.833	6.401	25.1	25.1	0.283	0.271	0.136
	0.360	6.176	6.425	26.8	26.8	0.301	0.288	0.144
	0.380	6.520	6.448	28.2	28.2	0.315	0.302	0.151
	0.400	6.863	6.472	29.6	29.6	0.329	0.315	0.158
	0.420	7.206	6.496	30.8	30.8	0.342	0.327	0.164
	0.440	7.549	6.520	31.9	31.9	0.352	0.337	0.169
	0.460	7.892	6.544	32.8	32.8	0.361	0.345	0.173
	0.480	8.235	6.569	34.0	34.0	0.373	0.357	0.178
	0.500	8.578	6.593	35.3	35.3	0.385	0.368	0.184
	0.550	9.436	6.656	38.9	38.9	0.421	0.402	0.201
	0.600	10.294	6.719	41.8	41.8	0.448	0.428	0.214
	0.650	11.152	6.784	43.8	43.8	0.465	0.443	0.221
	0.700	12.010	6.850	47.1	47.1	0.495	0.471	0.235
	0.750	12.867	6.918	50.7	50.7	0.528	0.502	0.251
	0.800	13.725	6.987	53.3	53.3	0.550	0.522	0.261
	0.850	14.583	7.057	55.6	55.6	0.568	0.539	0.269
	0.870	14.926	7.085	56.1	56.1	0.570	0.540	0.270

Strain (Inches/Inch) @ 50% Maximum Stress = 0.05812

Deformation @ 50% Maximum Stress (Inches)= 0.3380 **Tested by:** J. Young

Maximum Compressive Strength (TSF)= 0.54

% Strain @ Maximum Strength = 14.93%

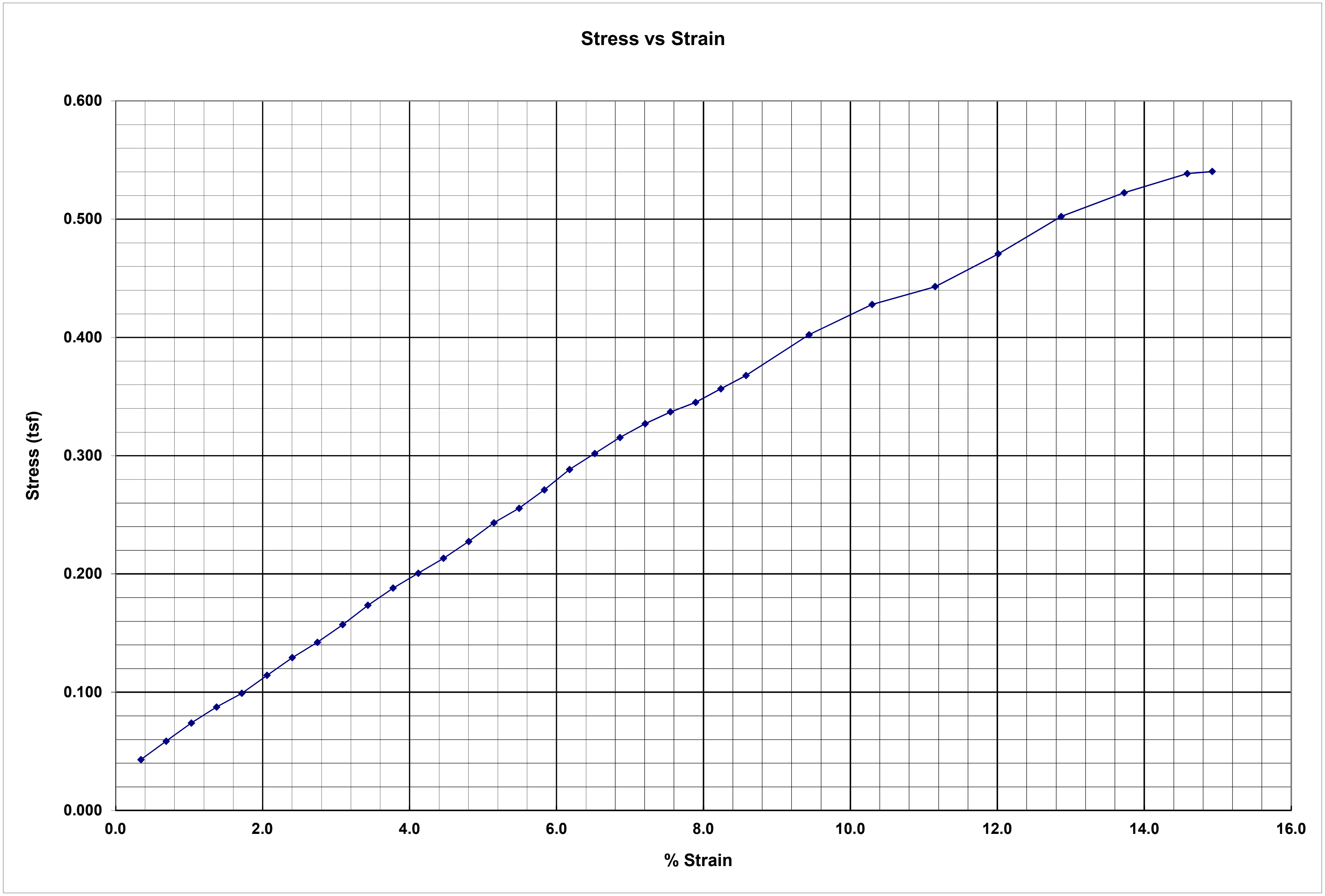
* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

TRIAXIAL COMPRESSION TEST STRESS/ STRAIN CURVE

Project: USACE-Brownsville Levee **Hole No.:** P3-33 **Sample No.:** 0 **Depth:** 10.8-12.8

TEAM Project No.: 142086 **Material:** Brown silty clay with sand

Date: 1/6/15



TEAM Consultants, Inc.

Appendix J: Recorded Communications 5 May 2014

From: Jose Nunez [mailto:Jose.Nunez@ibwc.gov]
Sent: Tuesday, May 06, 2014 3:13 PM
To: Dunbar, Joseph ERD
Cc: Isela CANAVA; Ramon Navarro
Subject: [EXTERNAL] Shifting Floodplain Embankment, Brownsville, Texas

Joe:

Reference is made to our telephone conversation this afternoon. The email described below from Mr. Ramon Navarro, our Construction Engineer in the Lower Rio Grande Valley, describes the geotechnical challenges that we are facing in the Brownsville, Texas, area.

The Upper Brownsville Levee Rehabilitation Project that is being affected by the cracks has the following coordinates:

	Northing	Easting
Beginning Coordinates	16,520,066.9	1,275,910.14
Ending Coordinates	16,489,164.3	1,314,326.25

Once we obtain the approval from our Contracting Office, we will forward to your attention a copy of the Scope of Work for the Geotechnical Investigations that we would like to procure the services of the USACE and any supporting documents (Geotechnical Report, Construction Plans, etc.) that you might need to perform this task(s). If you have any question, please give me a call. Regards,

José A. Nuñez, P.E.
Acting Principal Engineer
IBWC, U.S. Section
Headquarters
(915) 832-4710 <tel:9158324710>
(915) 433-0680 <tel:9154330680> Cell

>>> Ramon Navarro 3/31/2014 7:16 PM >>>

All,

An area on the USIBWC Upper Brownsville Levee Rehabilitation Project , from the East side of Sta. 1904+85 to the riprap area at Sta. 1898+00, has started to subside and there are cracks starting from the river bank east of Sta. 1904+50 that traverse up to the top of the levee. These cracks terminate at the CBP Fence Foundation (See Attached Photo Log Exhibits 1, 2, 3, 4, & 28).

The cracking continues down this foundation wall, back onto the top of levee over to the riprap area at Sta. 1898+00. There are indications that the cracking may continue under the riprap area towards the North Bound P.O.E. Bridge Abutment. (See Attached Photo Log Exhibits 5 through 19).

There is a second set of cracks that originate at the Rio Grande River Bank from Sta. 1896+50 heading East to Station 1903+50 (See Attached Photo Log Exhibits 20 through 27).

Due to the thick vegetation cover, photographing the cracks that start from the river bank was not possible.

The material between these cracks appears to be moving towards the Rio Grande River. There are indications that the cracks are growing in width and signs of up to three inches of torsional subsidence is visible. It appears the cracks are increasing in width by one inch per day. This is based on initial measurements that were taken on 3/29/14 (date when USIBWC was notified of this issue) through today.

It was mentioned by USIBWC Operations and Maintenance Representative , Joel Saldivar, the water levels in the Rio Grande had dropped significantly in the past month in this area. The effects of the water elevation drop may have attributed to soil subsidence along this levee reach culminating at this location. It was also noted that Anzalduas Dam is planning on releasing water later this week upstream from this area. This release has the potential to further impact the soil subsidence in this area.

A meeting was held today with USIBWC and DHS, both parties agreed the protection of life / limb and the protection of the area from further damage was necessary. It was agreed to by all parties the best course of action was to barricade the access roads in this levee section until a remedy has been determined to fix the problem.

The floodplain embankment drops into the Rio Grande River at a 90 Degree Angle in this location. Due to this, the USIBWC was unable to assess any River Bank Subsidence. The DHS representatives present at the aforementioned meeting offered to allow a USIBWC Representative to board a DHS/CBP patrol boat to better assess the river bank condition in this area. DHS will notify the USIBWC by Close of Business tomorrow if this is a possible option.

The Contractor on this project completed construction operations within this area in late October 2013 and will not be impacted by this levee section closure.

I have attached the following documents for your reference:

1. Photo Log showing current Area Conditions.

2. Plan Sheet with numerical Photo Log Call Outs.

3. Geotechnical Boring Logs for the Area in Question.

4. Proposed Cross Sections of Completed Work in this Area.

5. Meeting Minutes from USIBWC / DHS Meeting.

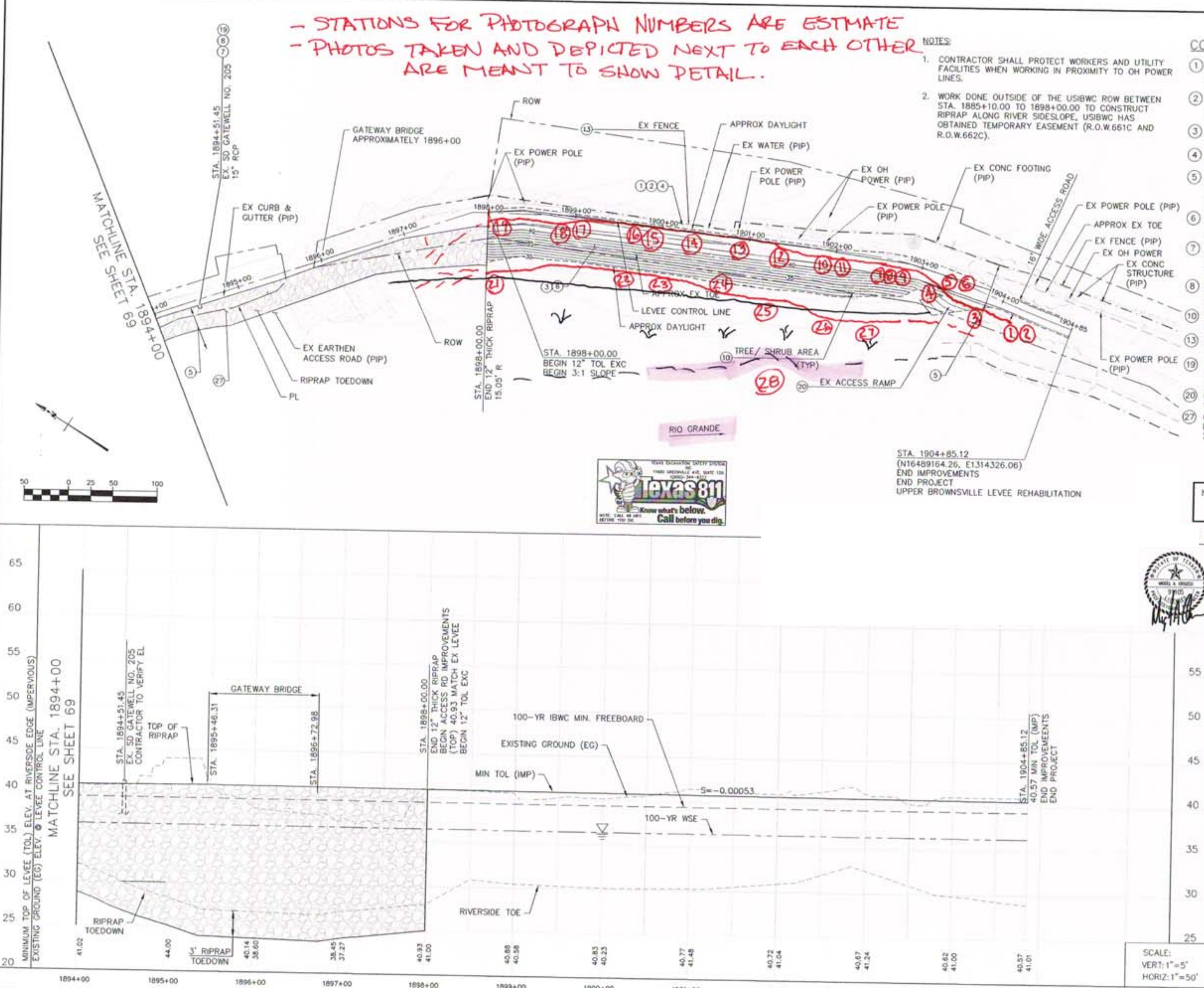
Please let me know if you have further questions regarding this matter.

Thank you,

Ramon F. Navarro, C.F.M.

C.O.R. / Civil Engineer
IBWC, U.S. Section
Headquarters
(956) 564-2991 (cell)
(956) 373-9776 (fax)
ramon.navarro@ibwc.gov

"Excellence through Teamwork"



NOTES:

1. CONTRACTOR SHALL PROTECT WORKERS AND UTILITY FACILITIES WHEN WORKING IN PROXIMITY TO OH POWER LINES.
2. WORK DONE OUTSIDE OF THE USIBWC ROW BETWEEN STA. 1885+10.00 TO 1898+00.00 TO CONSTRUCT RIPRAP ALONG RIVER SIDESLOPE, USIBWC HAS OBTAINED TEMPORARY EASEMENT (R.O.W.661C AND R.O.W.662C).

CONSTRUCTION NOTES

(1) CONSTRUCT TOL IMPROVEMENTS PER PLAN AND PROFILE HEREIN, TYPICAL SECTIONS ON SHEET 74 THRU 76, AND SECTIONS ON SHEETS 98 THRU 246.

(2) CONSTRUCT FLEX BASE ON TOP OF LEVEE PER PLAN HEREIN, TYPICAL DETAILS ON SHEET 74 AND 75 AND SPEC NO 50 20 00.

(3) GRADE LEVEE SIDESLOPES PER TYPICAL DETAILS ON SHEET 74 AND 75 AND SPEC NO 35 41 00.

(4) EXC 12-INCH DEPTH OF EX TOL PER TYPICAL SECTIONS ON SHEETS 74 AND 75 AND SPEC NO 35 41 00.

(5) USACE/FEMA NO VEGETATION ZONE, REMOVE ALL VEGETATION WITH A STEM OR TRUNK GREATER THAN 0.5-INCH THICK IN DIAMETER. SEE SPEC NO 31 11 00 & DETAILS B & D ON SHEET 77 FOR VEGETATION REMOVAL.

(6) INSTALL SEEDING FOR EROSION CONTROL WITH SEED MIXTURE PER PLAN HEREIN, TYPICAL SECTION ON SHEET 74 AND 75 AND SPEC NO 31 92 00.

(7) REMOVE SEDIMENT AND CLEAN INLET/OUTLET STRUCTURES AND PIPES TO RE-ESTABLISH FLOW PATH. CONTRACTOR TO PAINT GATEWELL STRUCTURE AND COVER PER DETAIL ON SHEET 80.

(8) PROTECT IN PLACE EX GATEWELL AND SLUICE GATE. PAINT EX GATEWELL COVER PER PLAN HEREIN AND PER SPEC NO 60 98 00.

(10) REMOVE EX TREES OR SHRUBS WITHIN USACE/FEMA NO VEGETATION ZONE PER DETAILS B & D ON SHEET 77.

(13) REMOVE INTERFERING PORTION OF FENCE OR GATE AND REPLACE NEW IN KIND. SEE DETAIL K ON SHEET 86.

(19) CONTRACTOR TO PROVIDE ACCESS TO EX GATEWELL TO ALLOW USIBWC TO INSPECT & DETERMINE CONDITION OF STRUCTURE & GATEWELL PER SPEC NO. 60 98 00.

(20) EXISTING ACCESS RAMP TO BE RECONSTRUCTED PER SHEETS 89-97 AND DETAIL N ON SHEET 88.

(27) CONSTRUCT RIPRAP PER PLAN AND PROFILE HEREIN, DETAIL ON SHEET 76 AND CROSS SECTIONS ON SHEETS 243-245.

NOTE: LOCATION OF EXISTING UTILITIES ARE APPROXIMATE AND SHALL BE FIELD VERIFIED BY CONTRACTOR.

QUANTITY TABLE (REFER TO GENERAL NOTE #46 SHT 2)

DIV	ITEM	DESC CODE	DESCRIPTION	UNIT	EST.	FINAL
01	57	00	CONS. EXITS/ENTRANCES	EA	0	
01	57	00	TEMP. SED. CONTROL SILT FENCE	LF	1,790	
02	91	00	REMOVE & REPLACE EX. FENCE	LF	0	
02	91	00	REMOVE & REPLACE EX. WIRE GATE	EA	0	
02	91	00	REMOVE & REPLACE EX. 9' HT GATE	EA	0	
31	11	00	PREPARING RIGHT OF WAY	STA	11	
31	92	00	VEG. FOR EROSION CONTROL (TEMP.)	SY	1,674	
31	92	00	VEG. FOR EROSION CONTROL (PERM.)	SY	1,674	
35	41	00	EXCAVATION	CY	158	
35	41	00	COMPACTED FILL	CY	157	
35	41	00	TOP SOIL	CY	0	
35	41	00	MAINTENANCE RAMP	EA	1	
35	42	00	RIPRAP	CY	870	
35	43	00	TOE DRAINS	LF	0	
50	20	00	8-INCH FLEX BASE	SY	1,339	
60	78	00	SLUICE GATE REPLACEMENT	EA	0	
90	79	00	REMOVE & REPLACE VEHICLE GATE	EA	0	
60	79	00	ABANDON AND PLUG STRUCTURE	EA	0	

PLANS PREPARED BY:
TETRA TECH
17885 VON KARMAN AVE., SUITE 500
IRVINE, CA 92614 (949) 809-5000

TEXAS REGISTRATION NO. F-3924

REV.	DESCRIPTION	BY	DATE

INTERNATIONAL BOUNDARY AND WATER COMMISSION
UNITED STATES SECTION
UPPER BROWNSVILLE LEVEE REHABILITATION
CAMERON COUNTY, TEXAS
PLAN & PROFILE

SCALE: VERT: 1"=5' HORIZ: 1"=50'

STA. 1894+00.00 TO STA. 1904+86.00

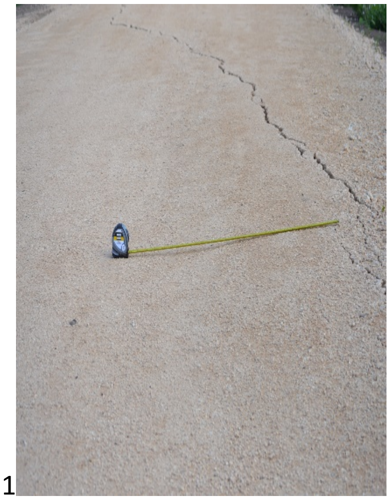
DESIGNED EM FILE
DRAWN EM RECOMMENDED IGP
CHECKED YHC JL AWG APPROVED MAO

May 29, 2012

1. 
2 
3 
4 
5 
6 
7 
8 
9 


10


10A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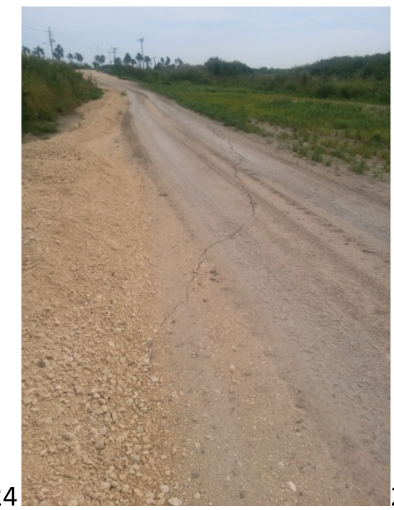
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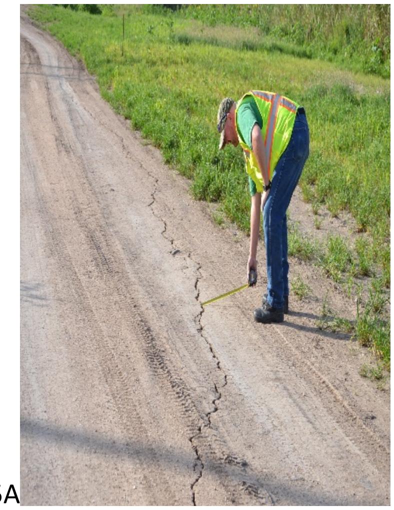

25A

25B

25C

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27

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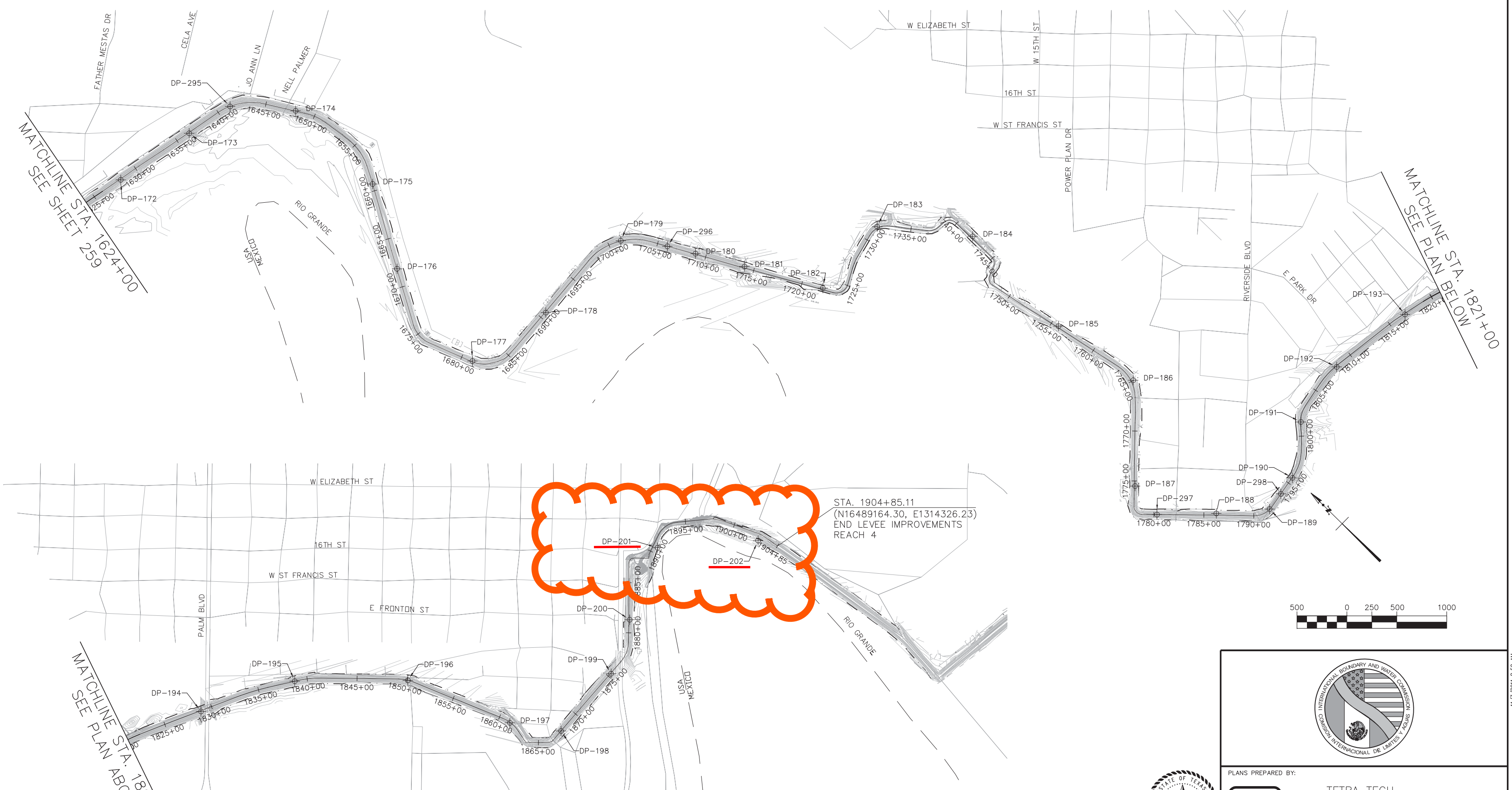

PT Name	Station	Offset	Northing	Easting	Elevation	HoleDepth
DP-172	1628+13.54	9.830'	16502532.49	1299805.79	47.53	50
DP-173	1636+42.58	8.924'	16502377.24	1300620.08	47.54	50
DP-174	1648+04.41	-2.222'	16501782.39	1301532.65	46.90	50
DP-175	1659+28.89	-1.202'	16500719.75	1301560.48	45.24	49.8
DP-176	1668+12.04	-1.592'	16499946.91	1301133.13	44.90	50
DP-177	1681+14.17	-7.112'	16498759.93	1301014.37	44.32	50
DP-178	1690+68.45	7.844'	16498583.21	1301877.45	44.56	50
DP-179	1701+26.10	-2.301'	16498560.82	1302916.14	43.74	50
DP-180	1708+93.09	-5.692'	16497940.04	1303353.96	44.05	50
DP-181	1714+01.12	-15.169'	16497502.21	1303610.05	44.04	50
DP-182	1722+14.23	-15.276'	16496794.55	1304012.00	44.85	50
DP-183	1731+62.47	-13.579'	16496843.69	1304830.71	44.71	50
DP-184	1742+06.23	-19.942'	16496099.62	1305438.98	45.24	50
DP-185	1755+75.77	-15.122'	16494856.54	1305412.17	44.55	50
DP-186	1764+96.00	-22.600'	16493944.78	1305558.05	44.22	50
DP-187	1775+50.42	-8.950'	16493183.00	1304823.20	44.10	50
DP-188	1786+24.55	-17.321'	16492413.76	1305204.45	43.69	50
DP-189	1791+77.93	-2.064'	16492067.25	1305612.97	43.21	50
DP-190	1795+65.76	3.334'	16492126.18	1305995.87	43.29	50
DP-191	1801+42.56	4.303'	16492461.60	1306450.76	43.91	50
DP-192	1808+18.69	11.145'	16492597.75	1307093.50	44.00	50
DP-193	1816+82.58	8.602'	16492486.04	1307948.82	43.91	50

PT Name	Station	Offset	Northing	Easting	Elevation	HoleDepth
DP-194	1828+84.91	8.244'	16492075.90	1309072.41	42.96	50
DP-195	1838+73.63	-0.806'	16491626.62	1309950.56	43.15	50
DP-196	1850+10.17	2.002'	16490831.54	1310757.04	43.39	50
DP-197	1861+19.89	4.600'	16489807.99	1311179.24	42.89	50
DP-198	1867+76.50	0.694'	16489388.51	1311484.29	42.78	50
DP-199	1875+36.47	5.148'	16489439.16	1312242.57	43.38	50
DP-200	1881+30.85	7.923'	16489689.54	1312756.49	42.58	50
DP-201	1890+61.31	47.587'	16490001.56	1313463.13	42.96	50
DP-202	1902+85.66	65.098'	16489339.93	1314225.65	39.68	50
DP-295	1641+33.56	1.701'	16502277.22	1301099.20	47.37	50
DP-296	1706+02.26	-0.573'	16498191.04	1303206.97	44.29	50
DP-297	1780+30.57	0.332'	16492826.01	1304776.35	44.10	50
DP-298	1793+67.77	0.358'	16492094.74	1305800.37	43.33	50

11/3/2011 8:52 AM

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TEXAS REGISTRATION NO. F-3924

Raba Kistner

INTERNATIONAL BOUNDARY AND WATER COMMISSION
UNITED STATES SECTION
UPPER BROWNSVILLE LEVEE REHABILITATION
CAMERON COUNTY, TEXAS

GEOTECHNICAL BORING LOCATION (2)

STA. 1624+00.00 TO STA. 1904+85.11

DESIGNED	EM	FILE	
DRAWN	EM	RECOMMENDED	IGP
CHECKED	YHC, JL, AWG	APPROVED	MAO

P:\WATR\T27062 DONNA TO BROWNSVILLE\08 DESIGN\SUB-REACH #4\T27062C260.DWG

LOG OF BORING NO. DP-201

The Lower Rio Grande Flood Control Project
Cameron County, Texas

Raba Kistner — TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger & Mud Rotary
LOCATION: N 16490001.56170; E 1313463.12860
SURFACE ELEVATION: 42.96 ft

SHEAR STRENGTH, TONS/FT²: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 — PLASTIC LIMIT / WATER CONTENT / LIQUID LIMIT: 10 20 30 40 50 60 70 80

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WIEGHT, pcf	PLASTICITY INDEX	% -200
0	FILL: CLAY, sandy, lean, very stiff, light brown	16		12	
2	CLAY, fat, stiff to very stiff, brown	14			97
		13			
		22		30	
		12			96
		15			
15	CLAY, lean, with sand, very stiff, light brown	15			82
20	- with orange ferrous stains below a depth of about 20 feet	16		12	
25	CLAY, lean, stiff, grayish-brown	9			96
30		9		9	
	Driller's Note: Converted to mud rotary drilling method at a depth of about 31-1/2 feet				
35	SAND, silty, medium dense, brown	23			15

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft **DATE DRILLED:** 12/3/2008
DEPTH TO WATER: 30 ft **DATE MEASURED:** 12/3/2008
PROJ. No.: AMA08-115-00 **FIGURE:** A-202a

LOG OF BORING NO. DP-201

The Lower Rio Grande Flood Control Project
Cameron County, Texas

Raba Kistner — TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger & Mud Rotary
LOCATION: N 16490001.56170; E 1313463.12860
SURFACE ELEVATION: 42.96 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WIEGHT, pcf	PLASTICITY INDEX	% -200
40	SAND, silty, medium dense, brown *(continued)*	25			15
49		14			
50	Boring terminated at a depth of about 50 feet.				

NOTES:
During the drilling operations, groundwater was observed at a depth of about 30 feet. Within 15 minutes of encountering groundwater, groundwater was measured at about 30 feet. Upon completion of the drilling operations, the boring caved-in to a depth of about 48.5 feet.
Backfilled with cement/bentonite grout.
Elevation and coordinates provided by Client.

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft **DATE DRILLED:** 12/3/2008
DEPTH TO WATER: 30 ft **DATE MEASURED:** 12/3/2008
PROJ. No.: AMA08-115-00 **FIGURE:** A-202b

LOG OF BORING NO. DP-202

The Lower Rio Grande Flood Control Project
Cameron County, Texas

Raba Kistner — TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger & Mud Rotary
LOCATION: N 16489339.93270; E 1314225.64544
SURFACE ELEVATION: 39.68 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WIEGHT, pcf	PLASTICITY INDEX	% -200
0	FILL: CLAY, fat, with sand, firm to stiff, light brown to brown, with calcareous nodules	6			85
		7		28	
4	- sand content decreases with depth	9			95
6	CLAY, sandy, lean, stiff to firm, brown, with calcareous nodules and black ferrous stains	15		13	
		4			
10	SAND, clayey, medium dense, light brown	13			39
15	CLAY, fat, stiff to very stiff, brown to light brown, with black ferrous stains	8		32	
20		9			89
25		10		42	
30	- becomes reddish-brown in color below a depth of about 30 feet		101	40	99
	Driller's Note: Converted to mud rotary drilling method at a depth of about 32 feet				
35	- firm 35 to 37 feet		95		99

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft **DATE DRILLED:** 12/3/2008
DEPTH TO WATER: 22 ft **DATE MEASURED:** 12/3/2008
PROJ. No.: AMA08-115-00 **FIGURE:** A-203a

LOG OF BORING NO. DP-202

The Lower Rio Grande Flood Control Project
Cameron County, Texas

Raba Kistner — TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger & Mud Rotary
LOCATION: N 16489339.93270; E 1314225.64544
SURFACE ELEVATION: 39.68 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WIEGHT, pcf	PLASTICITY INDEX	% -200
40	CLAY, fat, stiff to very stiff, brown to light brown, with black ferrous stains *(continued)*				
49					97
50	Boring terminated at a depth of about 50 feet.				

NOTES:
During the drilling operations, groundwater was observed at a depth of about 22 feet. Within 15 minutes of encountering groundwater, groundwater was measured at about 22 feet. Upon completion of the drilling operations, the boring caved-in to a depth of about 32 feet.
Backfilled with cement/bentonite grout.
Elevation and coordinates provided by Client.

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft **DATE DRILLED:** 12/3/2008
DEPTH TO WATER: 22 ft **DATE MEASURED:** 12/3/2008
PROJ. No.: AMA08-115-00 **FIGURE:** A-203b

INTERNATIONAL BOUNDARY AND WATER COMMISSION · COMISION INTERNACIONAL DE LIMITES Y AGUAS

PLANS PREPARED BY:

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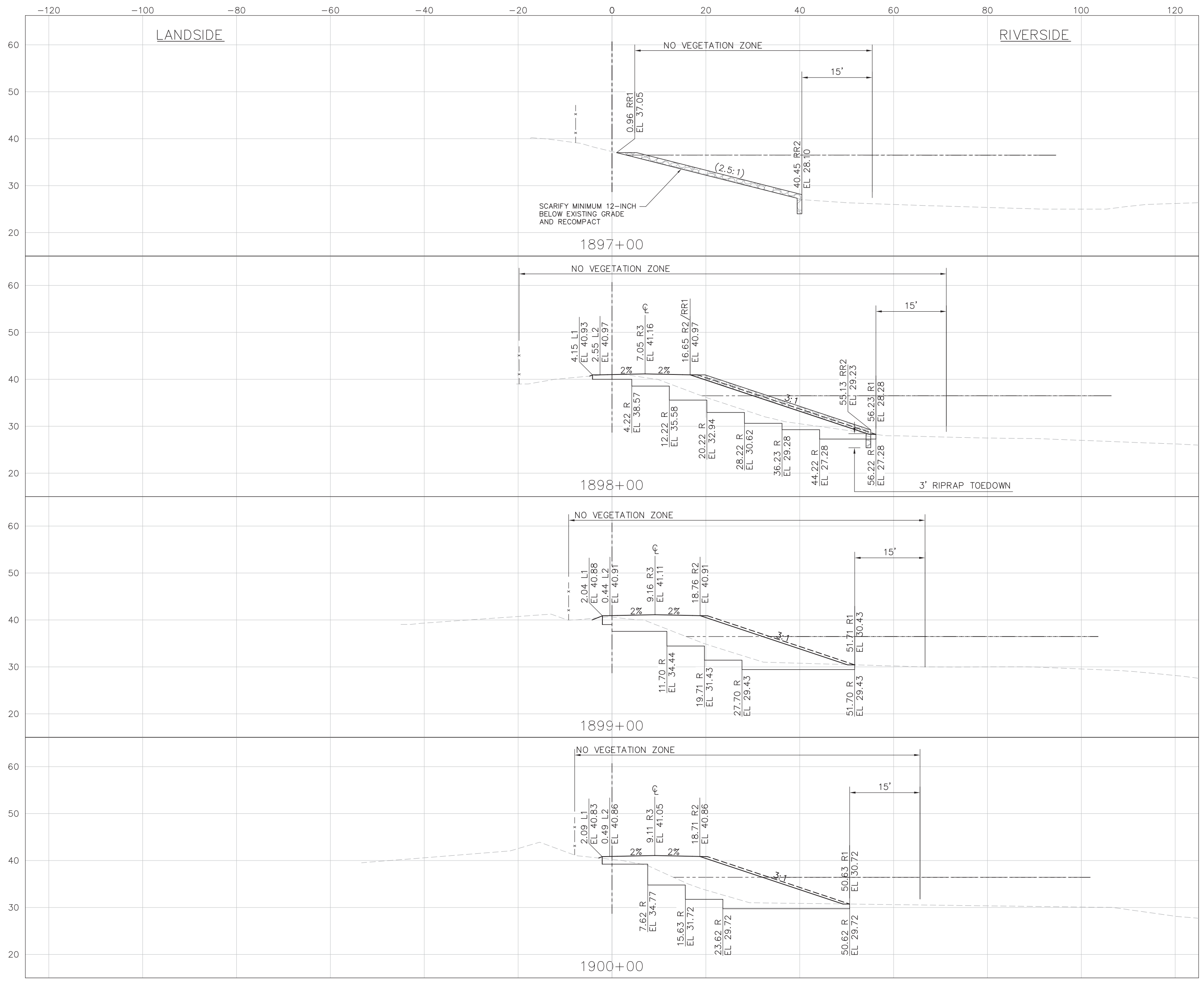
FAX: (949) 809-5010

Raba Kistner

INTERNATIONAL BOUNDARY AND WATER COMMISSION
UNITED STATES SECTION
UPPER BROWNSVILLE LEVEE REHABILITATION
CAMERON COUNTY, TEXAS

GEOTECHNICAL BORING LOG (25)

DESIGNED	EM	FILE	
DRAWN	EM	RECOMMENDED	IGP
CHECKED	YHC, JL, AWG	APPROVED	MAO



NOTES:

1. SEE SHEET 2 FOR GENERAL NOTES.
2. SEE SHEET 3 FOR ABBREVIATIONS.
3. SEE SHEET 74 FOR BENCHING AND EXCAVATION DETAILS.
4. SEE SHEET 76 FOR RIPRAP DETAILS.
5. SEE SHEET 74 THRU 76 FOR TYPICAL SECTIONS.
6. SEE SHEET 86 FOR TOE DRAIN DETAIL.
7. FLEX BASE ON LEVEE NOT SHOWN.
8. FEMA/USACE NO VEGETATION ZONE, REMOVE ALL VEGETATION WITH A STEM GREATER THAN 1/2-INCH THICK IN DIAMETER.
9. BENCHING OFFSET AND ELEVATION SHOWN ON CROSS SECTIONS ARE FOR REFERENCE ONLY. CONTRACTOR MAY ADJUST BENCHING OFFSETS AND ELEVATIONS FOR CONTRACTIBILITY WHILE ADHERING TO TYPICAL CROSS SECTIONS OR PER USIBWC COR DIRECTION FOR CONFORMANCE COMPLIANCE.
10. OFFSET DIMENSIONS TO ROW AS SHOWN ARE APPROXIMATE, FOR GENERAL REFERENCE ONLY, AND SHOULD NOT BE UTILIZED TO DETERMINE ACTUAL ROW LIMITS.

LEGEND

- TOP SOIL
- IMPERVIOUS LEVEE
- EX GROUND
- EXCAVATION LIMIT
- 100-WSE
- CONTROL LINE
- BORDER FENCE
- EX FENCE
- ACCESS ROAD CENTERLINE
- TOE DRAIN
- RIPRAP

SCALE: 1"=10' (HOR)
1"=10' (VERT)

PLANS PREPARED BY:

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FAX: (949) 809-5010

TEXAS REGISTRATION NO. F-3924

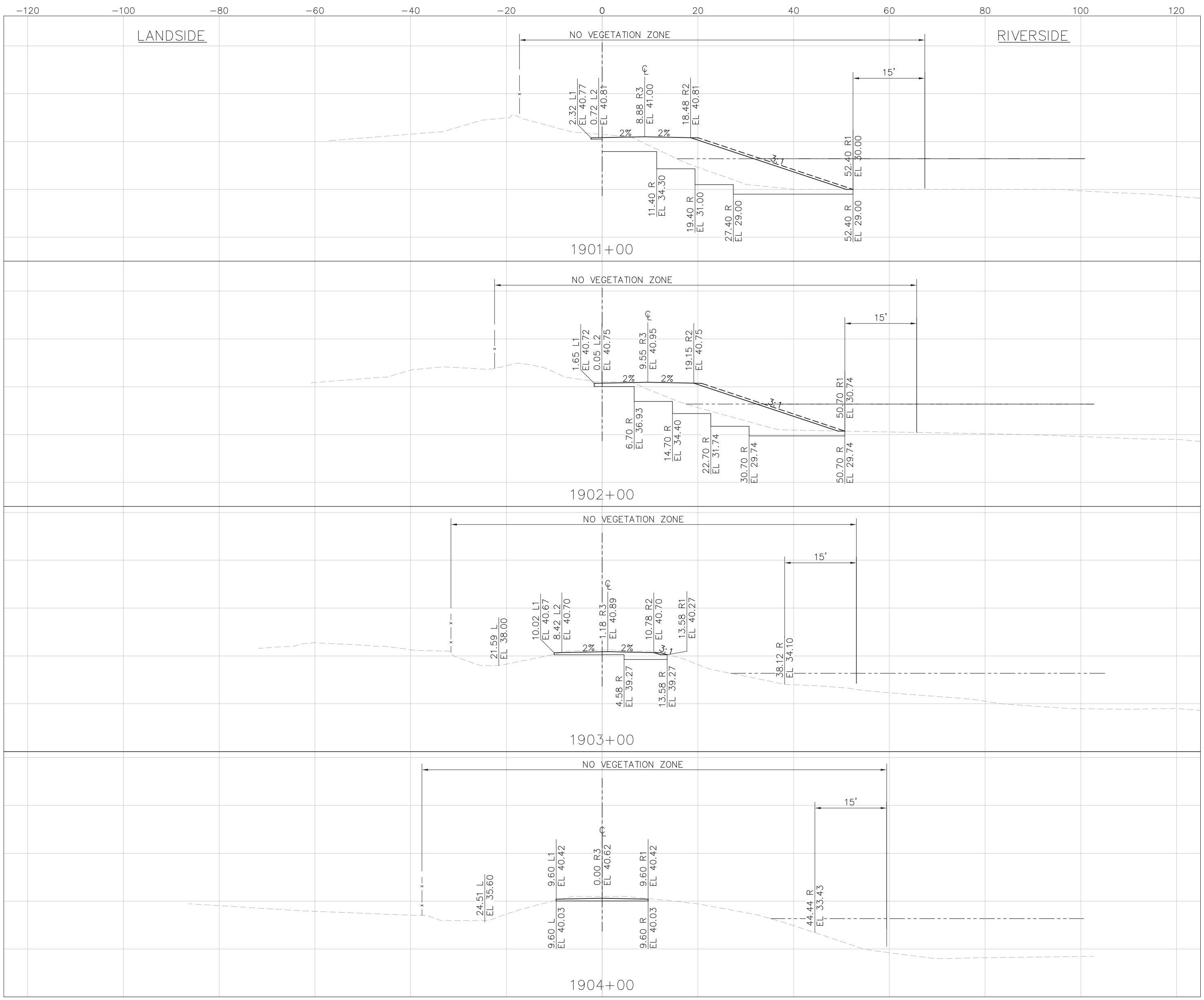
INTERNATIONAL BOUNDARY AND WATER COMMISSION
UNITED STATES SECTION
UPPER BROWNSVILLE LEVEE REHABILITATION
CAMERON COUNTY, TEXAS

IMPERVIOUS LEVEE/
EMBANKMENT CROSS SECTIONS (148)
STA. 1897+00 TO STA. 1900+00

DESIGNED	EM	FILE	
DRAWN	EM	RECOMMENDED	IGP
CHECKED	YHC, JL, AWG	APPROVED	MAO

SHEET 245 OF 299

11/3/2011 10:43 AM



NOTES:

1. SEE SHEET 2 FOR GENERAL NOTES.
2. SEE SHEET 3 FOR ABBREVIATIONS.
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9. OFFSET DIMENSIONS TO ROW AS SHOWN ARE APPROXIMATE, FOR GENERAL REFERENCE ON AND SHOULD NOT BE UTILIZED TO DETERMIN ACTUAL ROW LIMITS.

LEGEND

- TOP SOIL
- IMPERVIOUS LEVEE
- EX GROUND
- EXCAVATION LIMIT
- 100-WSE
- CONTROL LINE
- BORDER FENCE
- EX FENCE
- ACCESS ROAD CENTERLINE
- TOE DRAIN

SCALE: 1"=10' (HOR)
1"=10' (VERT)

10 0 5 10 20

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TEXAS REGISTRATION NO. F-3924

INTERNATIONAL BOUNDARY AND WATER COMMISSION
UNITED STATES SECTION
UPPER BROWNSVILLE LEVEE REHABILITATION
CAMERON COUNTY, TEXAS

IMPERVIOUS LEVEE/
EMBANKMENT CROSS SECTIONS (149)
STA. 1901+00 TO STA. 1904+00

DESIGNED	EM	FILE	
DRAWN	EM	RECOMMENDED	IGP
CHECKED	YHC, JL, AWG	APPROVED	MAO

SHEET 246 OF 299

5/31/2012 1:00 PM
P:\WATR\127062 DONNA TO BROWNSVILLE\08 DESIGN\SUB-REACH #4\127062C246.DWG

MEMORANDUM OF EMBANKMENT FAILURE MEETING FOR INFORMATION

Name & Initials	Organization	Role	In Attendance	Distribution of Minutes
Ramon Navarro (RN)	IBWC - GOV	COR / Civil Engineer	X	X
Joel Saldivar (JS)	IBWC - GOV	Engineering Technician	X	X
Steve Rouse (SR)	VSC	CM Inspector	X	X
Morgan Greenfield (MG)	VSC	FEM	X	X
Emilio Garza (EG)	LECON	Safety Officer	X	X
Juan Salazar	CBP - GOV	Border Patrol	X	X
Amador Carbajal	CBP - GOV	Border Patrol	X	X

CONTRACT INFORMATION

Project	Upper Brownsville Levee Rehabilitation (UBL)	**Project Number**	IBM13C0001
Owner	IBWC	**CO** **COR**	Ruben Pino Jr. Ramon Navarro
Contractor:	Lloyd Engineering and Construction (LECON, INC.)	**GM** **CQCSM** Alt. CQCSM	Daniel LLoyd, PE Brian Tiehen, George Heines
Notice to Proceed	May 30, 2013	**On Site Mobilization**	June 17, 2013 to June 28, 2013
Contract Completion	Sept 23, 2014	**Duration** **Days Remaining** **% Time used** **Start of Red Zone Date (80% Completion)**	485 DAYS 180 DAYS (Day 305) 63.29% used 22 June 2014
VSC CM	Kevin Salcido, PE Project Manager	**Project Inspector**	Steve Rouse Staff Engineer Alberto Urueta (Alt CI)

SUBJECT: IBM13C0001- UBL Emergency Shifting Embankment Meeting
Time: 12:00 PM
LOCATION: Sta. 1904+85 to Sta. 1897+00 (Gateway Bridge)

INTRODUCTIONS

1. Sign in sheet – See attached (Top of Page)

NEW BUSINESS

ITEM	DESCRIPTION	STATUS	RESPONSIBILITY Company	RESPONSIBILITY Person
1.	A meeting was called to determine if the access road was usable from the top of the levee at Sta. 1904+85 to the bridge at Sta. 1895+00, to allow traffic from all parties to continue use of said access road. The area from the East side of Sta. 1904+85 to the Riprap at Sta. 1898+00 has started to subside and there is a crack starting from the river bank east of Sta. 1904+50, across the access ramp, to upon the top of the levee that travels to the CBP fence foundation. The crack continues down this foundation wall, back onto the top of levee up to the riprap at Sta. 1898+00. There are indications that the crack may continue under the riprap towards the North Bound East Gate Bridge abutment. There is a second crack at the toe of the levee from Sta. 1903+00 heading West to Sta. 1896+50.	OPEN	LECON IBWC VSC	EG RN/JS SR/MG

The material between these two cracks appears to be moving towards the Rio Grande River. There are indications that the cracks are growing in width and signs of up to two inches of vertical subsidence is visible.

The USIBWC Representatives, CI for Vista Sciences, along with LECON Inc.'s SSHO, feel that it is a safety issue and requested this meeting to stop any and all traffic from crossing this area. The CBP Representatives concurred with this request and agreed to send an email to USIBWC agreeing to the traffic shutdown of this area.

The disturbance and damage has not been fully realized and there are too many unknowns to make an honest evaluation at this time. In the interest of protecting life, limb and equipment, both agreed to barricade the access road from Sta. 1895+00 near the bridge support column to the top of levee at Sta. 1904+85 (the end of Reach 4).

ACTION ITEMS

ITEM	DESCRIPTION	STATUS	RESPONSIBILITY Company	Person
1.	LECON has placed barricades on the access roads at both top of levee and at the bottom of levee at Sta. 1904+85 to block vehicular traffic from access to this area from the East.		LECON	**EG**
2.	LECON has placed a barricade at Sta. 1895+00 under the bridge to block access from the west.		LECON	**EG**
3.	IBWC COR has determined that mowing activities in this area need to be stopped until further notice.		LECON/ IBWC	**EG/RN**

RESOLVED/ UNRESOLVED ISSUES (NEW)

RESOLVED

IBWC has granted permission to barricade the access roads from both directions due to safety issues and damage control.

UNRESOLVED

IBWC may need to perform a geotechnical investigation on the current soil condition under the levee section from **Sta. 1904+85 to the bridge at Sta. 1895+00,** in order to determine the best direction in design, to alleviate the problem. This may have to be a joint effort from the Designer of Record, the Geotechnical Engineering Section, and the O&M Branch.

It was pointed out the Sub-Contractor on this project, Affolter Construction Inc., has had experience with similar mass failures of this nature and can offer a remedy to fix the problem.

CONCLUSIONS

After all parties had walked the area in question, and conducted discussions on the possible cause of the area subsidence, it was decided that the protection of life and limb was a factor and that the protection of the area from further damage was necessary. It was agreed to by all parties that the best course of action is to barricade the access roads until a remedy has been determined to fix the problem and the solution implemented. This will protect the safety of all who would normally use this access area. There is to be no mowing of the area until the extent of the damage has been fully assessed. This will protect the safety of the O&M personnel and equipment as well.

Public Notices for Release of Documents and Public Meetings

Workplan Document Release and Public Meeting - January 22,23,24,26, and 27, 2013

AFFIDAVIT OF PUBLICATION

IN THE MATTER OF
Public Notice

STATE OF HAWAII }
} SS.
City and County of Honolulu }

Doc. Date: JAN 2 4 2013 **# Pages:** 1

Notary Name: Patricia K. Reese First Judicial **Circuit**

Doc. Description: Affidavit of Publication

Notary Signature JAN 2 4 2013 Date

PATRICIA K. REESE NOTARY PUBLIC (Stamp or Seal) Comm. No. 86-467 STATE OF HAWAII

Rose Rosales being duly sworn, deposes and says that she is a clerk, duly authorized to execute this affidavit of Oahu Publications, Inc. publisher of The Honolulu Star-Advertiser and MidWeek, that said newspapers are newspapers of general circulation in the State of Hawaii, and that the attached notice is true notice as was published in the aforementioned newspapers as follows:

Honolulu Star-Advertiser 3 times on:

01/22, 01/23, 01/24/2013

Midweek Wed. 0 times on:

_______ times on:

And that affiant is not a party to or in any way interested in the above entitled matter.

Rose Rosales

Subscribed to and sworn before me this 24th day

of January A.D. 2013

Patricia K. Reese, Notary Public of the First Judicial Circuit, State of Hawaii

My commission expires: Oct 07 2014

Ad # 0000486856 LN:__________

PATRICIA K. REESE NOTARY PUBLIC Comm. No. 86-467 STATE OF HAWAII

PUBLIC NOTICE

Notice of Availability for the Draft Supplemental Marine Resources Sampling and Analysis Plan and Public Meeting for Makua Military Reservation Oahu, Hawaii

The Army has published the draft supplemental marine resources sampling and analysis plan for Makua Military Reservation, Oahu, Hawaii. The Army has prepared this plan to evaluate if limu (seaweed) and other marine resources (such as octopus and sea cucumber) near Makua Beach are impacted with constituents associated with training activities at Makua Military Reservation and may pose a human health risk to area residents that rely on marine resources for subsistence. This sampling and analysis plan presents the purpose, scope of work, strategy, and methodology that will be used to sample and analyze edible seaweed (limu), octopus (tako or He'e), and sea cucumber ("loli").

Public comments must be received or postmarked by March 22, 2013. The Army will take all public comments into consideration before finalizing the sampling and analysis plan.

A public meeting will be held on Feb. 20, 2013 with an informal information session from 6:30-7 p.m. with Army subject matter experts available to answer specific questions about the study, followed by a facilitated public comment session from 7-9:30 at: Nanakuli High School, 89-980 Nanakuli Ave., Nanakuli, HI 96792.

The plan is available in printed form at the Waianae Public Library, 85-625 Farrington Hwy, Waianae, and at the Kapolei Public Library, 1020 Manawai St., Kapolei, and can be accessed for reading or download at: www.garrison.hawaii.army.mil/makua. For further information, please call, (808) 656-3089 or email, usaghi.pao.comrel@us.army.mil .

Public comments may be submitted online by email or by mail to: U.S. Army Garrison – Hawaii, Environmental Division, Attn: Marine Study, 948 Santos Dumont Ave., Schofield Barracks, HI 96857.
(SA486856 1/22, 1/23, 1/24/13)a

Workplan Document Release and Public Meeting - January 22,23,24,26, and 27, 2013

AFFIDAVIT OF PUBLICATION

IN THE MATTER OF
Public Notice

STATE OF HAWAII } SS.
City and County of Honolulu }

Doc. Date: JAN 2 8 2013 **# Pages:** 1
Notary Name: Patricia K. Reese First Judicial **Circuit**
Doc. Description: Affidavit of Publication

Notary Signature JAN 2 8 2013 Date

PATRICIA K. REESE NOTARY PUBLIC (Stamp or Seal) Comm. No. 86-467 STATE OF HAWAII

Rose Rosales being duly sworn, deposes and says that she is a clerk, duly authorized to execute this affidavit of Oahu Publications, Inc. publisher of The Honolulu Star-Advertiser and MidWeek, that said newspapers are newspapers of general circulation in the State of Hawaii, and that the attached notice is true notice as was published in the aforementioned newspapers as follows:

Honolulu Star-Advertiser 2 times on:

01/26, 01/27/2013

Midweek Wed. 0 times on:

_______ times on:

And that affiant is not a party to or in any way interested in the above entitled matter.

Rose Rosales

Subscribed to and sworn before me this 28th day

of Jan A.D. 20 13

Patricia K. Reese, Notary Public of the First Judicial Circuit, State of Hawaii

My commission expires: Oct 07 2014

Ad # 0000488378

LN:__________

The Army has published the draft supplemental marine resources sampling and analysis plan for Makua Military Reservation, Oahu, Hawaii. The Army has prepared this plan to evaluate if limu (seaweed) and other marine resources (such as octopus and sea cucumber) near Makua Beach are impacted with constituents associated with training activities at Makua Military Reservation and may pose a human health risk to area residents that rely on marine resources for subsistence. This sampling and analysis plan presents the purpose, scope of work, strategy, and methodology that will be used to sample and analyze edible seaweed (limu), octopus (tako or He'e), and sea cucumber ("loli").

Public comments must be received or postmarked by March 22, 2013. The Army will take all public comments into consideration before finalizing the sampling and analysis plan.

A public meeting will be held on Feb. 20, 2013 with an informal information session from 6:30-7 p.m. with Army subject matter experts available to answer specific questions about the study, followed by a facilitated public comment session from 7-9:30 at: Nanakuli High School, 89-980 Nanakuli Ave., Nanakuli, HI 96792.

The plan is available in printed form at the Waianae Public Library, 85-625 Farrington Hwy, Waianae, and at the Kapolei Public Library, 1020 Manawai St., Kapolei, and can be accessed for reading or download at: www.garrison.hawaii.army.mil/makua. For further information, please call, (808) 656-3089 or email, usaghi.pao.comrel@us.army.mil.

Public comments may be submitted online by email or by mail to: U.S. Army Garrison – Hawaii, Environmental Division, Attn: Marine Study, 948 Santos Dumont Ave., Schofield Barracks, HI 96857.
(SA488378 1/26, 1/27/13)

PATRICIA K. REESE NOTARY PUBLIC Comm. No. 86-467 STATE OF HAWAII

Study Report Document Release and Public Meeting - February 1,2, and 3, 2015

AFFIDAVIT OF PUBLICATION

IN THE MATTER OF
Public Notice

STATE OF HAWAII } SS.
City and County of Honolulu

Doc. Date: FEB - 2 2015	**# Pages:** 1
Notary Name: Patricia K. Reese	First Judicial **Circuit**
Doc. Description: Affidavit of Publication	
Notary Signature	FEB - 2 2015 Date

PATRICIA K. REESE NOTARY PUBLIC Comm. No. 86-467 STATE OF HAWAII

Lisa Kaukani being duly sworn, deposes and says that she is a clerk, duly authorized to execute this affidavit of Oahu Publications, Inc. publisher of The Honolulu Star-Advertiser, MidWeek, The Garden Island, West Hawaii Today, and Hawaii Tribune-Herald, that said newspapers are newspapers of general circulation in the State of Hawaii, and that the attached notice is true notice as was published in the aforementioned newspapers as follows:

Honolulu Star-Advertiser 2 times on:
02/01, 02/02/2015
MidWeek 0 times on:
The Garden Island 0 times on:
Hawaii Tribune-Herald 0 times on:
West Hawaii Today 0 times on:
Other Publications: 0 times on:

And that affiant is not a party to or in any way interested in the above entitled matter.

Lisa Kaukani

Subscribed to and sworn before me this 2ND day of February A.D. 2015

Patricia K. Reese, Notary Public of the First Judicial Circuit, State of Hawaii
My commission expires: Oct 07, 2018

Ad # 0000716171

PATRICIA K. REESE NOTARY PUBLIC Comm. No. 86-467 STATE OF HAWAII

SP.NO.: ____________ L.N.

PUBLIC NOTICE

Notice of Availability for the Supplemental Marine Resources Study

Makua Military Reservation, Oahu, Hawaii

The Army has published the supplemental marine resources study for Makua Military Reservation (MMR), Oahu, Hawaii. The Army has prepared this study t determine whether military activities at MMR have contributed or will contribute t contamination of the marine resources near Makua, and whether Army trainin activities at MMR pose a health risk to area residents who rely on these marin resources for food or other purposes. The study presents the sampling methods laboratory results, and associated risk assessment of the marine resources analyzed: edible seaweed (limu), octopus (tako or He'e), and sea cucumber (loli).

The public is invited to provide comments on the study in writing and/or during a public meeting. Written comments must be received or postmarked by April 3 2015, and can be submitted via email to: usaghi.pao.comrel@us.army.mil; or via mail to: U.S. Army Garrison-Hawaii, Environmental Division, Attn: Marine Study - Public Comments, 948 Santos Dumont Ave., Schofield Barracks, HI 96857.

Comments can be provided in person at a public meeting March 5, 2015. The meeting will begin with an informal information session from 6:30-7 p.m. with Army subject matter experts available to answer specific questions about the study, followed by a facilitated public comment session from 7-9:30 p.m. at:

Waianae High School Cafetorium

85-251 Farrington Hwy., Waianae, HI 96792

The Army will take all public comments into consideration before finalizing the study.

The Supplemental Marine Resources Study is available in printed form at the following public libraries: Waianae, 85-625 Farrington Hwy.; Kapolei, 1020 Manawai St.; Wahiawa, 820 California Ave.; Waialua, 67-068 Kealohanui St. The study is available for reading or download at: www.garrison.hawaii.army.mil/makua; click on "2013 MR Study." If you would like a printed copy mailed to you, please call (808) 656-3089 or email usaghi.pao.comrel@us.army.mil.
(SA716171 2/1, 2/2/15)

Study Report Document Release and Public Meeting - February 1,2, and 3, 2015

AFFIDAVIT OF PUBLICATION

IN THE MATTER OF
Public Noice

STATE OF HAWAII } SS.
City and County of Honolulu

Doc. Date: FEB - 3 2015 **# Pages:** 1
Notary Name: Patricia K. Reese First Judicial **Circuit**
Doc. Description: Affidavit of Publication

Notary Signature FEB - 3 2015 Date

PATRICIA K. REESE NOTARY PUBLIC Comm. No. 88-467 STATE OF HAWAII

Lisa Kaukani being duly sworn, deposes and says that she is a clerk, duly authorized to execute this affidavit of Oahu Publications, Inc. publisher of The Honolulu Star-Advertiser, MidWeek, The Garden Island, West Hawaii Today, and Hawaii Tribune-Herald, that said newspapers are newspapers of general circulation in the State of Hawaii, and that the attached notice is true notice as was published in the aforementioned newspapers as follows:

Honolulu Star-Advertiser	1	times on:
02/03/2015		
MidWeek	0	times on:
The Garden Island	0	times on:
Hawaii Tribune-Herald	0	times on:
West Hawaii Today	0	times on:
Other Publications:	0	times on:

And that affiant is not a party to or in any way interested in the above entitled matter.

Lisa Kaukani

Subscribed to and sworn before me this 3rd day of Feb A.D. 20 15

Patricia K. Reese, Notary Public of the First Judicial Circuit, State of Hawaii
My commission expires: Oct 07, 2018

Ad # 0000716457

PATRICIA K. REESE NOTARY PUBLIC Comm. No. 88-467 STATE OF HAWAII

SP.NO.: ______ L.N.

PUBLIC NOTICE

Notice of Availability for the Supplemental Marine Resources Study

Makua Military Reservation, Oahu, Hawaii

The Army has published the supplemental marine resources study for Makua Military Reservation (MMR), Oahu, Hawaii. The Army has prepared this study to determine whether military activities at MMR have contributed or will contribute to contamination of the marine resources near Makua, and whether Army training activities at MMR pose a health risk to area residents who rely on these marine resources for food or other purposes. The study presents the sampling methods, laboratory results, and associated risk assessment of the marine resources analyzed: edible seaweed (limu), octopus (tako or He'e), and sea cucumber (loli).

The public is invited to provide comments on the study in writing and/or during a public meeting. Written comments must be received or postmarked by April 3, 2015, and can be submitted via email to: usaghi.pao.comrel@us.army.mil; or via mail to: U.S. Army Garrison-Hawaii, Environmental Division, Attn: Marine Study - Public Comments, 948 Santos Dumont Ave., Schofield Barracks, HI 96857.

Comments can be provided in person at a public meeting March 5, 2015. The meeting will begin with an informal information session from 6:30-7 p.m. with Army subject matter experts available to answer specific questions about the study, followed by a facilitated public comment session from 7-9:30 p.m. at:

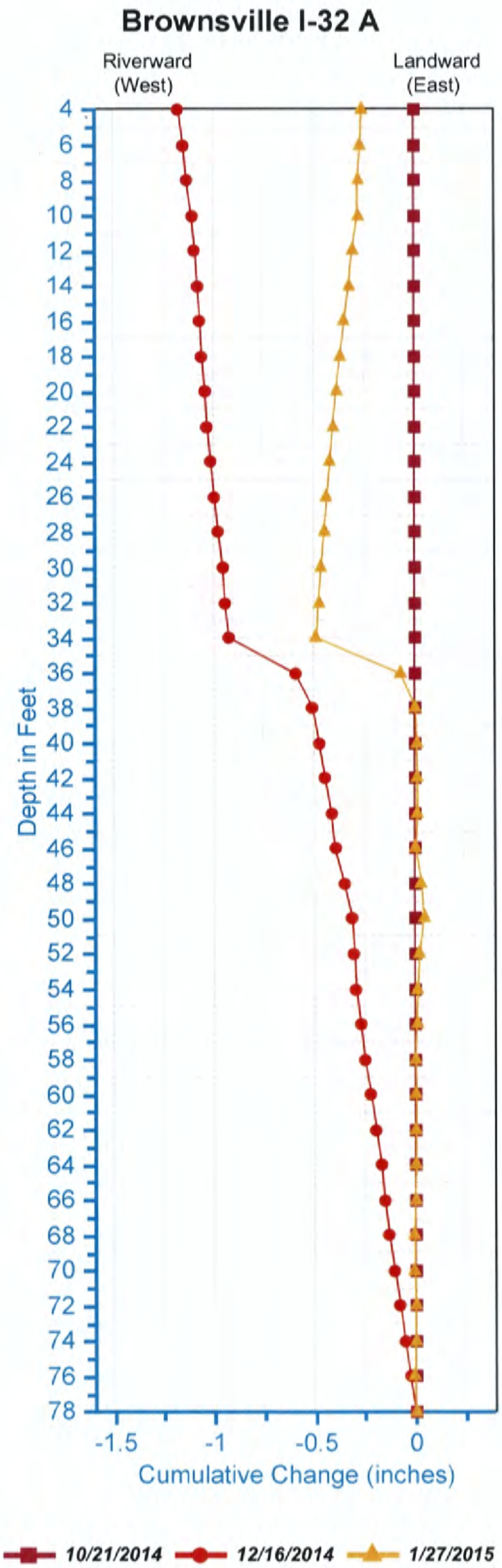
Waianae High School Cafetorium

85-251 Farrington Hwy., Waianae, HI 96792

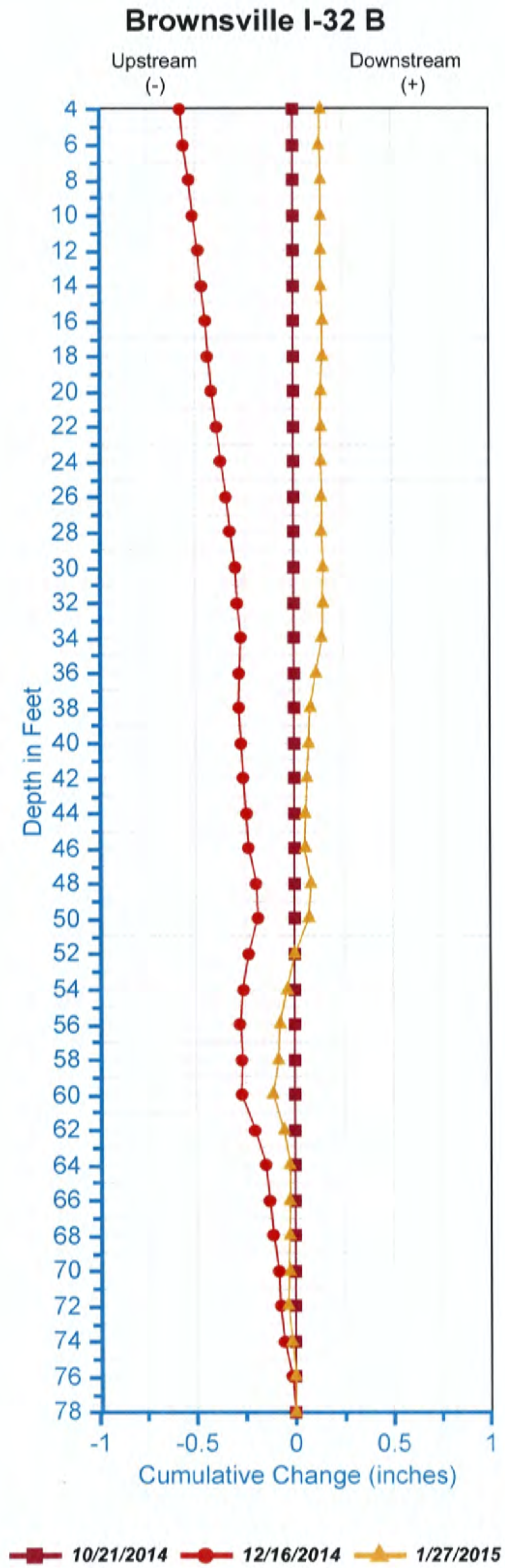
The Army will take all public comments into consideration before finalizing the study.

The Supplemental Marine Resources Study is available in printed form at the following public libraries: Waianae, 85-625 Farrington Hwy.; Kapolei, 1020 Manawai St.; Wahiawa, 820 California Ave.; Waialua, 67-068 Kealohanui St. The study is available for reading or download at: www.garrison.hawaii.army.mil/makua; click on "2013 MR Study." If you would like a printed copy mailed to you, please call (808) 656-3089 or email usaghi.pao.comrel@us.army.mil.
(SA716457 2/3/15)

Appendix K: Inclinometer Data



Brownsville I-32 A
Riverward (West)
Landward (East)
Depth in Feet
Cumulative Change (inches)
10/21/2014
12/16/2014
1/27/2015



Brownsville I-32 B
Upstream (-)
Downstream (+)
Depth in Feet
Cumulative Change (inches)
10/21/2014
12/16/2014
1/27/2015

Site: BRIBWC
Installation: I32
Survey Date: 10/21/2014 12:22:46 PM
A0 Direction: 0
Description: Crest of Levee
Survey Type: Digitilt
Num Passes: 2
Depth Units: Feet

Checksum Analysis for Survey:

A Mean:	-62.3	B Mean:	24.5
A Std.Dev:	1.8	B Std.Dev:	28.3

Depth	A0	A180	A Sum	B0	B180	B Sum
4.0	-754	691	-63	195	-180	15
6.0	-672	609	-63	182	-168	14
8.0	-596	528	-68	145	-126	19
10.0	-573	514	-59	114	-96	18
12.0	-551	489	-62	152	-132	20
14.0	-508	443	-65	170	-151	19
16.0	-465	403	-62	200	-180	20
18.0	-422	356	-66	251	-232	19
20.0	-463	403	-60	250	-208	42
22.0	-407	345	-62	187	-160	27
24.0	-343	279	-64	103	-72	31
26.0	-266	205	-61	11	12	23
28.0	-195	131	-64	-46	74	28
30.0	-167	102	-65	-105	126	21
32.0	-143	83	-60	-40	58	18
34.0	-155	90	-65	10	13	23
36.0	-179	118	-61	66	-45	21
38.0	-143	81	-62	146	-130	16
40.0	-68	7	-61	134	-114	20
42.0	-24	-37	-61	168	-149	19
44.0	-34	-28	-62	194	-175	19
46.0	-159	99	-60	170	-151	19
48.0	-162	99	-63	312	-289	23
50.0	-133	71	-62	314	-199	115
52.0	-157	97	-60	217	-131	86
54.0	-232	170	-62	151	-90	61
56.0	-272	211	-61	112	-70	42
58.0	-248	185	-63	86	-20	66
60.0	-315	254	-61	47	-112	-65
62.0	-323	261	-62	129	-173	-44
64.0	-354	291	-63	195	-178	17
66.0	-416	355	-61	201	-178	23

68.0	-406	344	-62	214	-196	18
70.0	-292	231	-61	139	-97	42
72.0	-241	179	-62	266	-253	13
74.0	-295	231	-64	601	-580	21
76.0	-349	286	-63	816	-798	18

Site:	BRIBWC		
Installation:		I32	
Survey Date:		12/16/2014 12:22:46 PM	
A0 Direction:		0	
Description:		Crest of Levee	
Survey Type:		Digitilt	
Num Passes:		2	
Depth Units:		Feet	

Checksum Analysis for Survey:

A Mean:	-26.3		B Mean:	-43.7
A Std.Dev:	4.1		B Std.Dev:	6.6

Depth	A0	A180	A Sum	B0	B180	B Sum
4.0	-754	731	-23	151	-196	-45
6.0	-667	638	-29	130	-178	-48
8.0	-599	569	-30	94	-138	-44
10.0	-569	546	-23	65	-106	-41
12.0	-544	516	-28	101	-154	-53
14.0	-498	474	-24	122	-165	-43
16.0	-456	429	-27	157	-197	-40
18.0	-414	387	-27	206	-249	-43
20.0	-453	432	-21	183	-237	-54
22.0	-402	379	-23	126	-181	-55
24.0	-336	316	-20	44	-88	-44
26.0	-262	240	-22	-41	-7	-48
28.0	-192	171	-21	-104	56	-48
30.0	-159	130	-29	-145	104	-41
32.0	-143	117	-26	-83	44	-39
34.0	-412	383	-29	-20	-27	-47
36.0	-231	209	-22	41	-80	-39
38.0	-151	125	-26	108	-146	-38
40.0	-74	51	-23	93	-136	-43
42.0	-31	7	-24	126	-169	-43
44.0	-33	4	-29	153	-196	-43
46.0	-181	155	-26	107	-145	-38
48.0	-178	151	-27	282	-311	-29
50.0	-122	99	-23	271	-320	-49
52.0	-152	117	-35	172	-226	-54
54.0	-235	202	-33	107	-152	-45
56.0	-275	242	-33	62	-105	-43
58.0	-255	228	-27	39	-76	-37
60.0	-318	294	-24	0	-44	-44
62.0	-327	298	-29	87	-126	-39
64.0	-354	318	-36	152	-188	-36
66.0	-419	388	-31	156	-193	-37

68.0	-410	387	-23	171	-197	-26
70.0	-293	275	-18	84	-137	-53
72.0	-244	216	-28	219	-267	-48
74.0	-297	268	-29	530	-585	-55
76.0	-353	328	-25	771	-816	-45

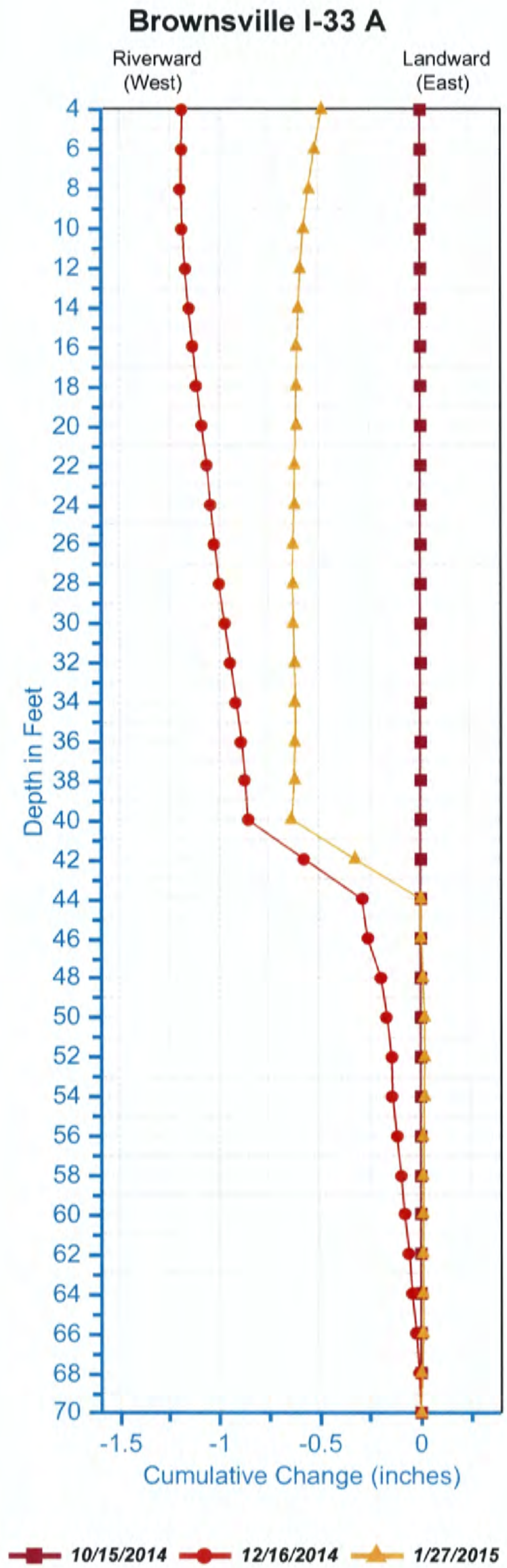
Site:	BRIBWC	
Installation:		I32
Survey Date:		1/27/2015 10:08:19 AM
A0 Direction:		0
Description:		Crest of Levee
Survey Type:		Digitilt
Num Passes:		2
Depth Units:		Feet

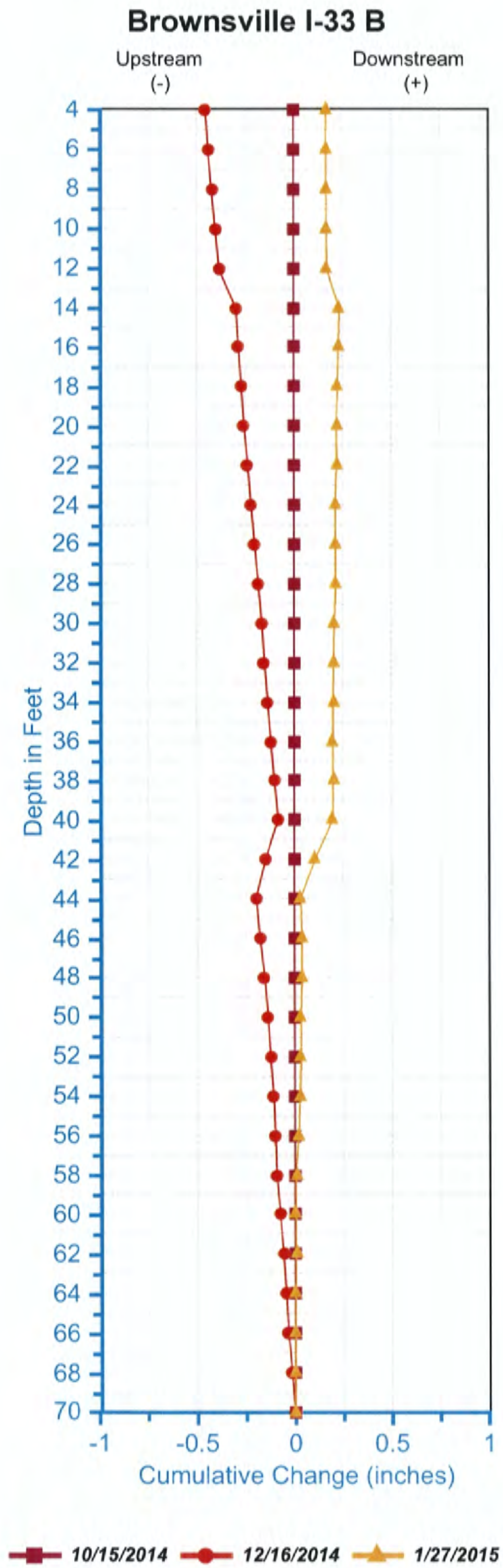
Checksum Analysis for Survey:

A Mean:	-69.5		B Mean:	18.3
A Std.Dev:	4.9		B Std.Dev:	5.5

Depth	A0	A180	A Sum	B0	B180	B Sum
4.0	-753	681	-72	198	-179	19
6.0	-664	594	-70	180	-161	19
8.0	-597	525	-72	146	-124	22
10.0	-543	500	-43	121	-95	26
12.0	-540	470	-70	148	-132	16
14.0	-494	420	-74	166	-144	22
16.0	-450	381	-69	201	-181	20
18.0	-410	337	-73	252	-243	9
20.0	-452	384	-68	241	-221	20
22.0	-398	330	-68	185	-159	26
24.0	-335	263	-72	91	-72	19
26.0	-258	189	-69	10	7	17
28.0	-187	114	-73	-62	69	7
30.0	-156	88	-68	-104	121	17
32.0	-139	70	-69	-41	56	15
34.0	-508	436	-72	38	-19	19
36.0	-245	176	-69	86	-71	15
38.0	-152	78	-74	150	-137	13
40.0	-76	7	-69	140	-123	17
42.0	-29	-39	-68	171	-155	16
44.0	-31	-42	-73	201	-181	20
46.0	-186	117	-69	144	-124	20
48.0	-176	105	-71	314	-305	9
50.0	-120	52	-68	328	-302	26
52.0	-153	84	-69	217	-202	15
54.0	-232	160	-72	160	-139	21
56.0	-271	202	-69	104	-89	15
58.0	-257	183	-74	85	-68	17
60.0	-316	250	-66	46	-23	23
62.0	-326	256	-70	131	-117	14
64.0	-354	283	-71	196	-178	18
66.0	-419	349	-70	202	-184	18

68.0	-411	340	-71	215	-201	14
70.0	-297	231	-66	139	-102	37
72.0	-244	175	-69	257	-232	25
74.0	-297	225	-72	584	-565	19
76.0	-354	284	-70	814	-803	11


Brownsville I-33 A
Riverward (West)
Landward (East)
Depth in Feet
Cumulative Change (inches)
10/15/2014
12/16/2014
1/27/2015


Brownsville I-33 B
Upstream (-)
Downstream (+)
Depth in Feet
Cumulative Change (inches)
10/15/2014
12/16/2014
1/27/2015

Site:	BRIBWC
Installation:	I-33
Survey Date:	10/15/2014 10:45:00 AM
A0 Direction:	0
Description:	Mid-Slope
Survey Type:	Digitilt
Num Passes:	2
Depth Units:	Feet

Checksum Analysis for Survey:

A Mean:	-62.7	B Mean:	-23.3
A Std.Dev:	2.4	B Std.Dev:	18.5

Depth	A0	A180	A Sum	B0	B180	B Sum
4.0	-668	603	-65	445	-462	-17
6.0	-754	690	-64	424	-444	-20
8.0	-735	673	-62	449	-467	-18
10.0	-650	589	-61	408	-429	-21
12.0	-621	561	-60	424	-551	-127
14.0	-629	565	-64	473	-496	-23
16.0	-639	577	-62	514	-532	-18
18.0	-604	544	-60	561	-579	-18
20.0	-571	507	-64	527	-550	-23
22.0	-558	499	-59	599	-619	-20
24.0	-608	542	-66	618	-640	-22
26.0	-616	553	-63	595	-616	-21
28.0	-669	607	-62	669	-683	-14
30.0	-570	507	-63	725	-744	-19
32.0	-502	440	-62	750	-767	-17
34.0	-481	416	-65	726	-746	-20
36.0	-506	443	-63	693	-710	-17
38.0	-610	548	-62	636	-658	-22
40.0	-656	592	-64	591	-615	-24
42.0	-680	620	-60	615	-633	-18
44.0	-686	621	-65	669	-689	-20
46.0	-636	570	-66	795	-818	-23
48.0	-605	542	-63	934	-951	-17
50.0	-573	511	-62	1117	-1142	-25
52.0	-718	657	-61	1264	-1289	-25
54.0	-818	747	-71	1241	-1257	-16
56.0	-855	793	-62	1041	-1059	-18
58.0	-912	850	-62	952	-977	-25
60.0	-851	790	-61	960	-983	-23
62.0	-877	819	-58	994	-1013	-19
64.0	-910	845	-65	981	-1003	-22
66.0	-957	896	-61	992	-1013	-21

68.0	-1075	1014	-61	1036	-1053	-17

Site:	BRIBWC
Installation:	I-33
Survey Date:	12/16/2014 10:45:00 AM
A0 Direction:	0
Description:	Mid-Slope
Survey Type:	Digitilt
Num Passes:	2
Depth Units:	Feet

Checksum Analysis for Survey:

	A Mean:	-36.8	B Mean:	-37.6
	A Std.Dev:	9.8	B Std.Dev:	4.1

Depth	A0	A180	A Sum	B0	B180	B Sum
4.0	-649	613	-36	421	-463	-42
6.0	-737	705	-32	399	-439	-40
8.0	-724	695	-29	423	-463	-40
10.0	-645	616	-29	379	-425	-46
12.0	-623	590	-33	398	-434	-36
14.0	-631	595	-36	454	-494	-40
16.0	-642	610	-32	491	-532	-41
18.0	-611	576	-35	539	-576	-37
20.0	-573	552	-21	503	-543	-40
22.0	-561	526	-35	578	-614	-36
24.0	-611	576	-35	595	-632	-37
26.0	-618	584	-34	571	-608	-37
28.0	-678	646	-32	643	-679	-36
30.0	-576	551	-25	706	-742	-36
32.0	-512	479	-33	727	-763	-36
34.0	-487	454	-33	703	-742	-39
36.0	-505	473	-32	666	-709	-43
38.0	-607	571	-36	612	-650	-38
40.0	-877	846	-31	635	-676	-41
42.0	-906	865	-41	642	-687	-45
44.0	-693	660	-33	644	-682	-38
46.0	-697	618	-79	773	-802	-29
48.0	-617	582	-35	912	-946	-34
50.0	-580	540	-40	1106	-1131	-25
52.0	-719	665	-54	1246	-1280	-34
54.0	-822	776	-46	1224	-1261	-37
56.0	-859	825	-34	1024	-1064	-40
58.0	-914	873	-41	931	-970	-39
60.0	-856	825	-31	940	-975	-35
62.0	-882	837	-45	973	-1010	-37
64.0	-916	876	-40	965	-1002	-37
66.0	-957	917	-40	968	-1007	-39

68.0	-1074	1028	-46	1014	-1046	-32

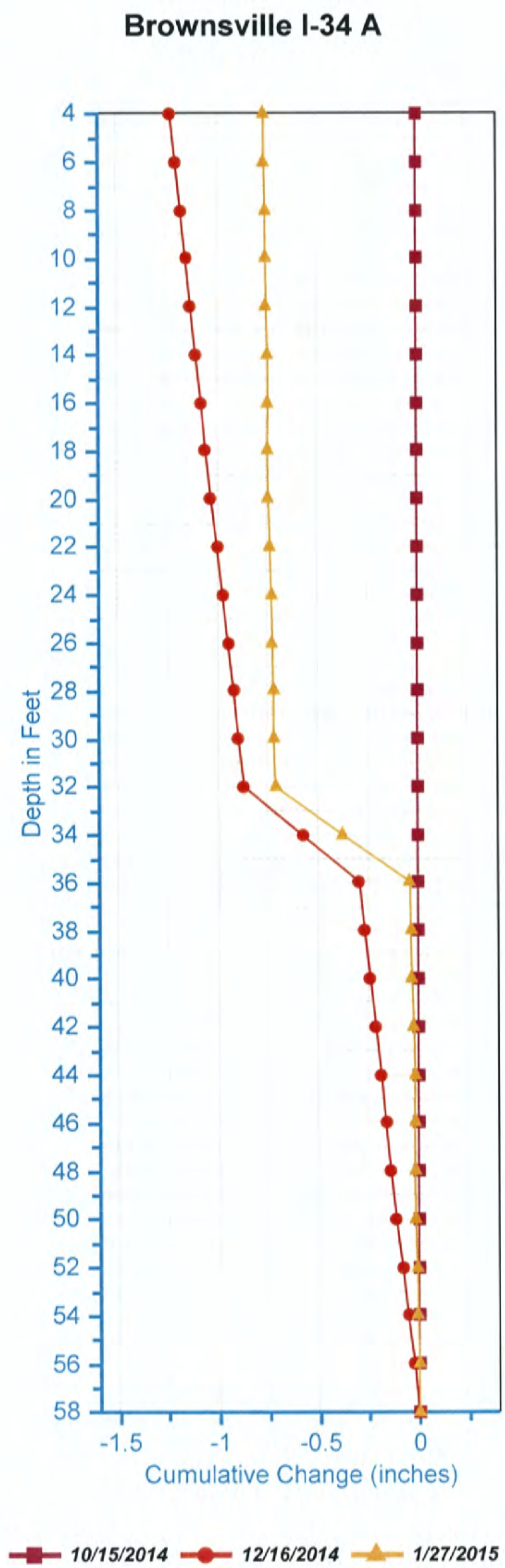
Site:	BRIBWC	
Installation:		I-33
Survey Date:		1/27/2015 11:44:51 AM
A0 Direction:		0
Description:		Mid-Slope
Survey Type:		Digitilt
Num Passes:		2
Depth Units:		Feet

Checksum Analysis for Survey:

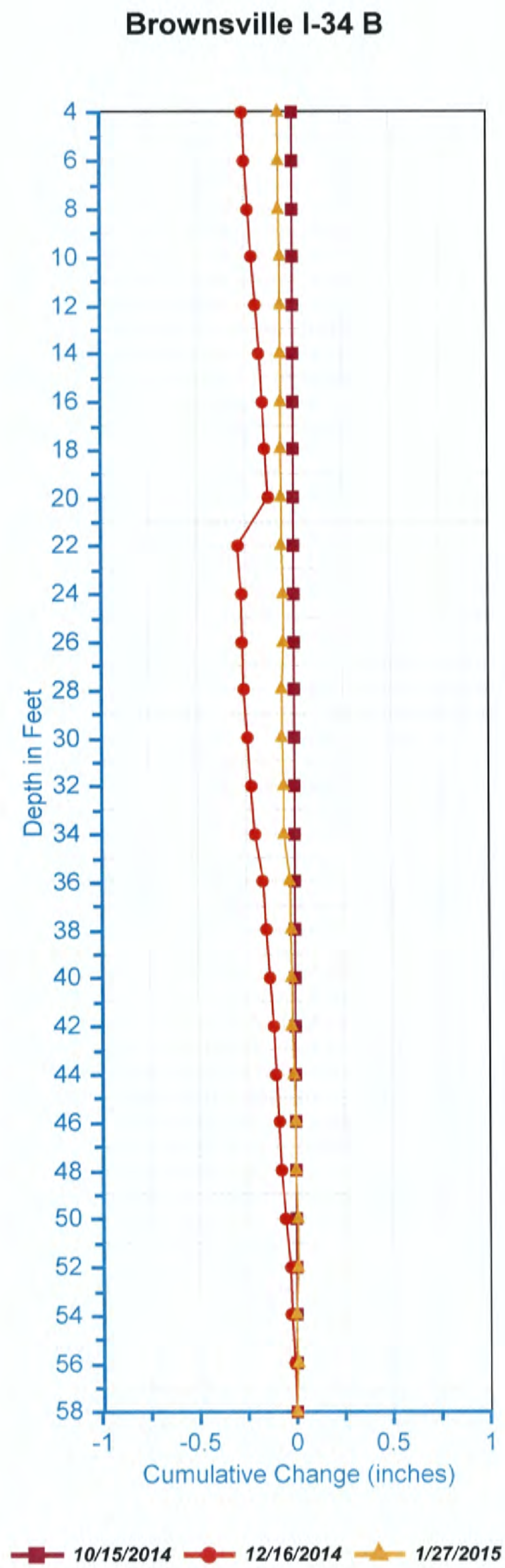
A Mean:	-48.2		B Mean:	-0.7
A Std.Dev:	2.5		B Std.Dev:	5.5

Depth	A0	A180	A Sum	B0	B180	B Sum
4.0	-630	580	-50	456	-456	0
6.0	-721	673	-48	431	-435	-4
8.0	-706	660	-46	456	-459	-3
10.0	-632	582	-50	416	-416	0
12.0	-607	562	-45	430	-437	-7
14.0	-616	566	-50	488	-488	0
16.0	-628	579	-49	525	-528	-3
18.0	-595	549	-46	574	-579	-5
20.0	-561	513	-48	536	-538	-2
22.0	-548	501	-47	610	-622	-12
24.0	-596	545	-51	626	-630	-4
26.0	-605	556	-49	604	-606	-2
28.0	-667	619	-48	681	-682	-1
30.0	-566	519	-47	738	-738	0
32.0	-499	452	-47	758	-764	-6
34.0	-473	423	-50	739	-737	2
36.0	-492	446	-46	701	-699	2
38.0	-594	547	-47	648	-648	0
40.0	-904	861	-43	686	-674	12
42.0	-941	898	-43	691	-683	8
44.0	-679	629	-50	677	-671	6
46.0	-634	586	-48	807	-804	3
48.0	-605	558	-47	942	-954	-12
50.0	-567	518	-49	1127	-1132	-5
52.0	-710	661	-49	1281	-1280	1
54.0	-809	753	-56	1261	-1247	14
56.0	-846	801	-45	1064	-1062	2
58.0	-902	856	-46	966	-964	2
60.0	-844	797	-47	970	-972	-2
62.0	-873	823	-50	1003	-1006	-3
64.0	-903	853	-50	997	-992	5
66.0	-947	895	-52	999	-1003	-4

68.0	-1063	1013	-50	1044	-1049	-5



Brownsville I-34 A
Depth in Feet
Cumulative Change (inches)
-1.5
-1
-0.5
0
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
10/15/2014
12/16/2014
1/27/2015



Brownsville I-34 B
Depth in Feet
Cumulative Change (inches)
-1
-0.5
0
0.5
1
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
10/15/2014
12/16/2014
1/27/2015

Site:	BRIBWC		
Installation:		I-34	
Survey Date:		10/15/2014 10:00:00 AM	
A0 Direction:		0	
Description:		Below Toe of Levee	
Survey Type:		Digitilt	
Num Passes:		2	
Depth Units:		Feet	

Checksum Analysis for Survey:

A Mean:	-64.1		B Mean:	-16.4
A Std.Dev:	2.2		B Std.Dev:	5.9

Depth	A0	A180	A Sum	B0	B180	B Sum
4.0	92	-157	-65	-766	750	-16
6.0	139	-202	-63	-760	745	-15
8.0	207	-274	-67	-737	726	-11
10.0	282	-344	-62	-853	832	-21
12.0	309	-371	-62	-825	808	-17
14.0	365	-431	-66	-861	844	-17
16.0	437	-501	-64	-876	858	-18
18.0	449	-512	-63	-828	814	-14
20.0	454	-519	-65	-886	869	-17
22.0	458	-514	-56	-866	848	-18
24.0	406	-470	-64	-834	811	-23
26.0	397	-460	-63	-798	781	-17
28.0	406	-471	-65	-802	788	-14
30.0	428	-494	-66	-930	907	-23
32.0	431	-493	-62	-919	907	-12
34.0	396	-464	-68	-919	904	-15
36.0	414	-479	-65	-858	848	-10
38.0	406	-470	-64	-734	733	-1
40.0	443	-510	-67	-769	743	-26
42.0	450	-515	-65	-670	653	-17
44.0	418	-482	-64	-527	506	-21
46.0	370	-435	-65	-453	441	-12
48.0	368	-434	-66	-422	413	-9
50.0	437	-501	-64	-509	477	-32
52.0	563	-625	-62	-626	616	-10
54.0	662	-727	-65	-730	710	-20
56.0	742	-806	-64	-791	775	-16

Site:	BRIBWC	
Installation:		I-34
Survey Date:		12/16/2014 10:00:00 AM
A0 Direction:		0
Description:		Below Toe of Levee
Survey Type:		Digitilt
Num Passes:		2
Depth Units:		Feet

Checksum Analysis for Survey:

	A Mean:	-30.2	B Mean:	-54.7
	A Std.Dev:	5.6	B Std.Dev:	56.7

Depth	A0	A180	A Sum	B0	B180	B Sum
4.0	89	-118	-29	-790	744	-46
6.0	132	-168	-36	-786	745	-41
8.0	203	-235	-32	-766	726	-40
10.0	278	-308	-30	-887	834	-53
12.0	302	-336	-34	-853	809	-44
14.0	361	-391	-30	-887	843	-44
16.0	436	-467	-31	-900	857	-43
18.0	443	-474	-31	-854	811	-43
20.0	446	-479	-33	-916	573	-343
22.0	448	-477	-29	-894	850	-44
24.0	400	-431	-31	-849	808	-41
26.0	390	-422	-32	-815	772	-43
28.0	403	-434	-31	-830	790	-40
30.0	420	-454	-34	-957	908	-49
32.0	218	-225	-7	-955	906	-49
34.0	173	-209	-36	-966	927	-39
36.0	408	-442	-34	-886	845	-41
38.0	400	-431	-31	-767	730	-37
40.0	437	-467	-30	-796	746	-50
42.0	446	-477	-31	-694	653	-41
44.0	417	-448	-31	-545	506	-39
46.0	367	-398	-31	-478	436	-42
48.0	365	-393	-28	-456	416	-40
50.0	431	-455	-24	-531	494	-37
52.0	556	-588	-32	-653	597	-56
54.0	657	-678	-21	-755	710	-45
56.0	738	-774	-36	-815	767	-48

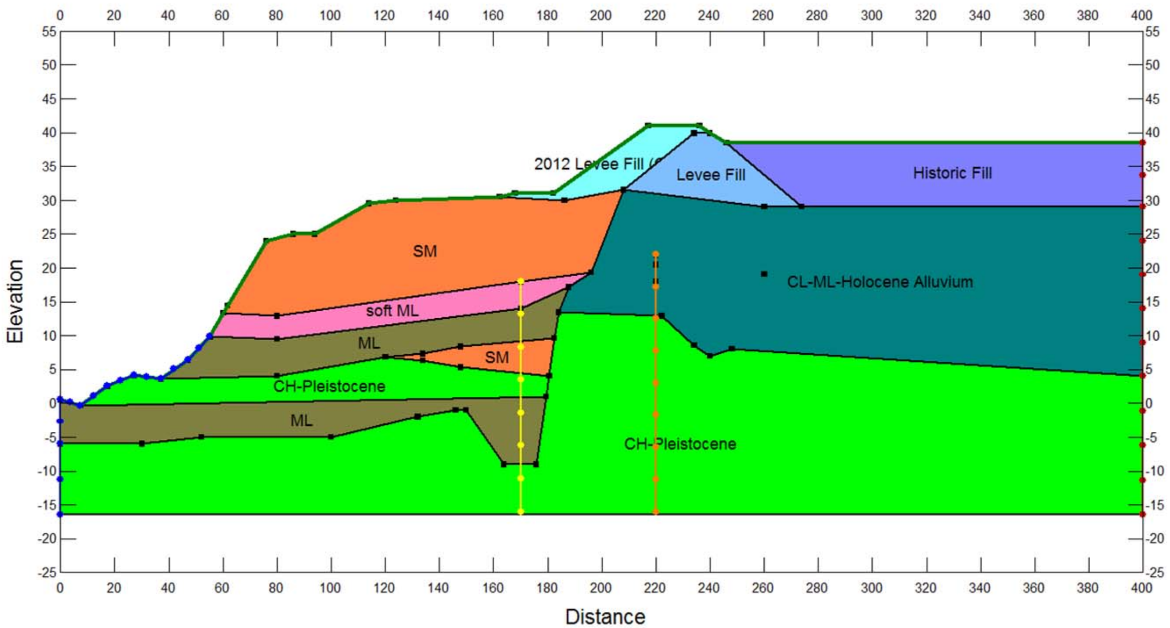
Site:	BRIBWC	
Installation:		I-34
Survey Date:		1/27/2015 1:21:59 PM
A0 Direction:		0
Description:		Below Toe of Levee
Survey Type:		Digitilt
Num Passes:		2
Depth Units:		Feet

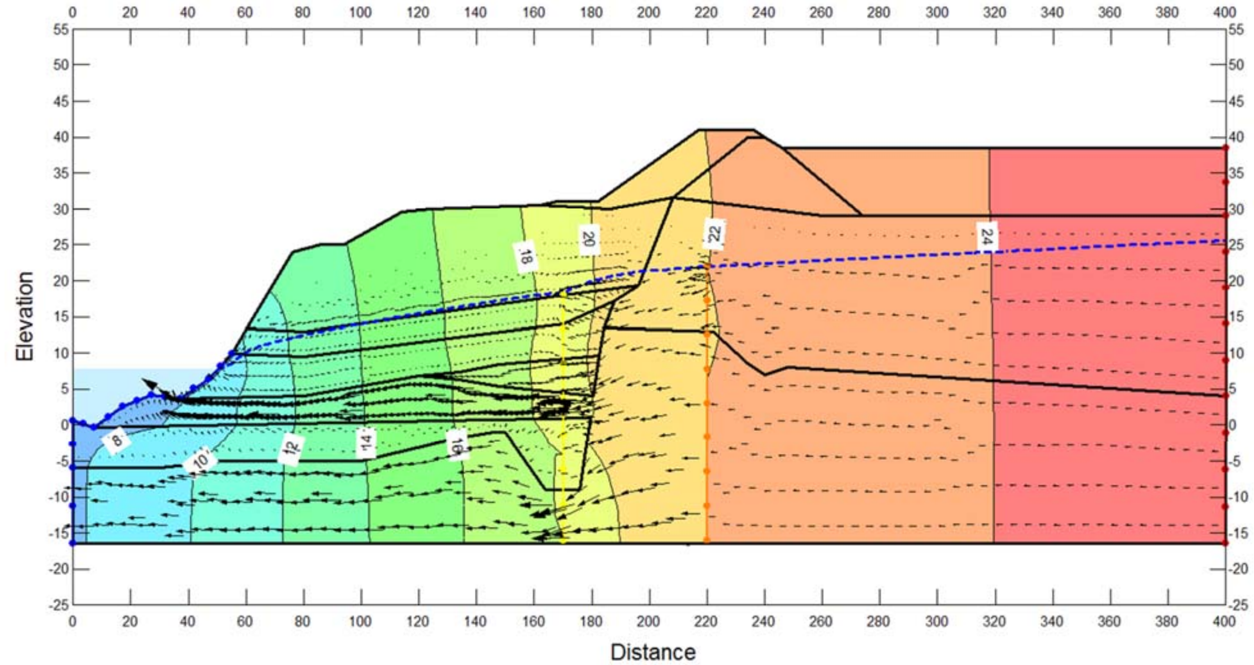
Checksum Analysis for Survey:

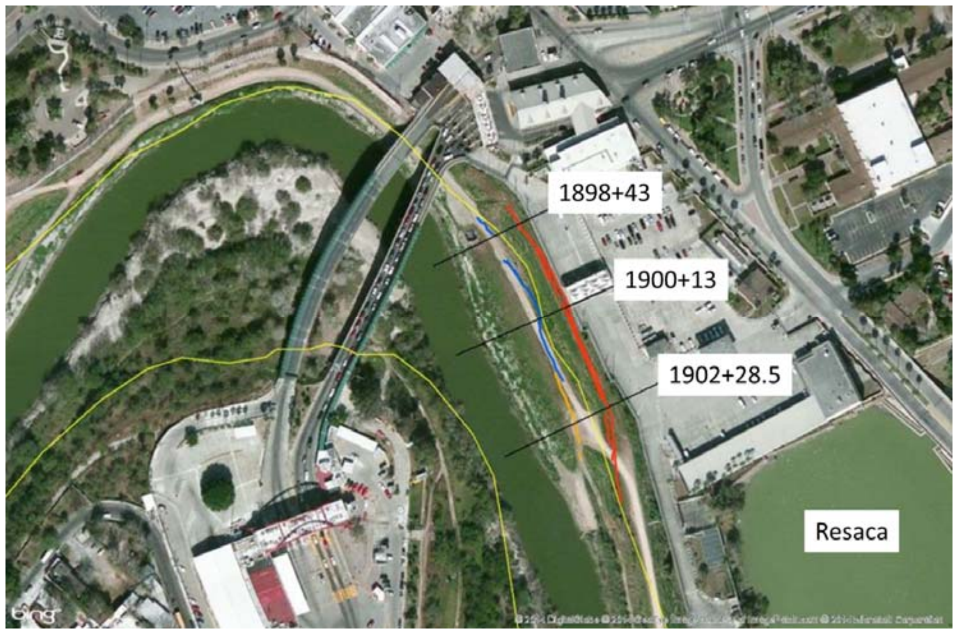
A Mean:	-107.7	B Mean:	-54.5
A Std.Dev:	3.7	B Std.Dev:	4.5

Depth	A0	A180	A Sum	B0	B180	B Sum
4.0	68	-178	-110	-786	735	-51
6.0	114	-220	-106	-782	727	-55
8.0	185	-294	-109	-762	705	-57
10.0	257	-363	-106	-870	819	-51
12.0	282	-389	-107	-843	791	-52
14.0	343	-451	-108	-877	828	-49
16.0	417	-527	-110	-895	837	-58
18.0	425	-530	-105	-849	797	-52
20.0	426	-535	-109	-905	852	-53
22.0	428	-533	-105	-889	838	-51
24.0	380	-487	-107	-847	797	-50
26.0	372	-479	-107	-811	753	-58
28.0	383	-491	-108	-828	765	-63
30.0	400	-510	-110	-953	892	-61
32.0	130	-228	-98	-945	886	-59
34.0	91	-213	-122	-967	907	-60
36.0	390	-498	-108	-883	824	-59
38.0	380	-486	-106	-767	710	-57
40.0	416	-522	-106	-783	733	-50
42.0	426	-532	-106	-694	644	-50
44.0	399	-505	-106	-545	497	-48
46.0	349	-458	-109	-475	416	-59
48.0	345	-452	-107	-455	398	-57
50.0	409	-515	-106	-520	468	-52
52.0	536	-645	-109	-635	590	-45
54.0	637	-746	-109	-753	695	-58
56.0	720	-829	-109	-807	751	-56

Appendix L: Model Plates







Boundary Conditions	type	magnitude (ft)
P3-32	head	22
P3-33	head	18
River	head	7.77
Protected side	head	25.59

material	K_{sat} (ft/s)	n	m_v (1/psf)	ratio
CH Pleistocene	3.30E-08	0.44	3.60E-06	0.2
CL-Holocene	3.30E-08	0.43	2.50E-06	0.2
SM	3.30E-07	0.3	5.00E-06	0.2
ML	1.00E-07	0.43	1.00E-05	0.2
2012 Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Historic Fill	3.30E-08	0.4	3.74E-06	0.2
soft ML	1.00E-07	0.45	1.00E-05	1

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

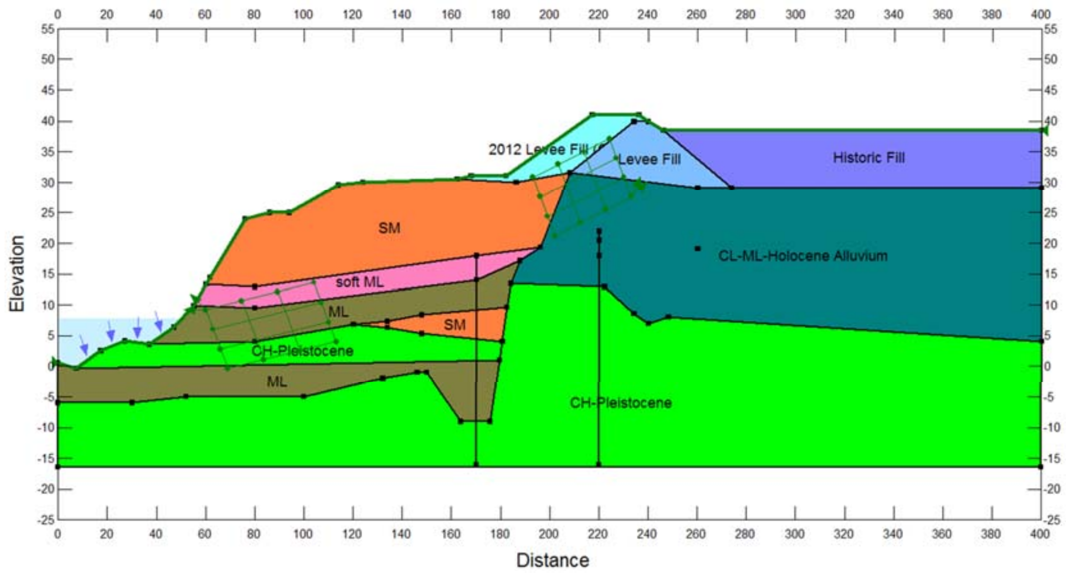
IBWC-BROWNSVILLE LEVEE

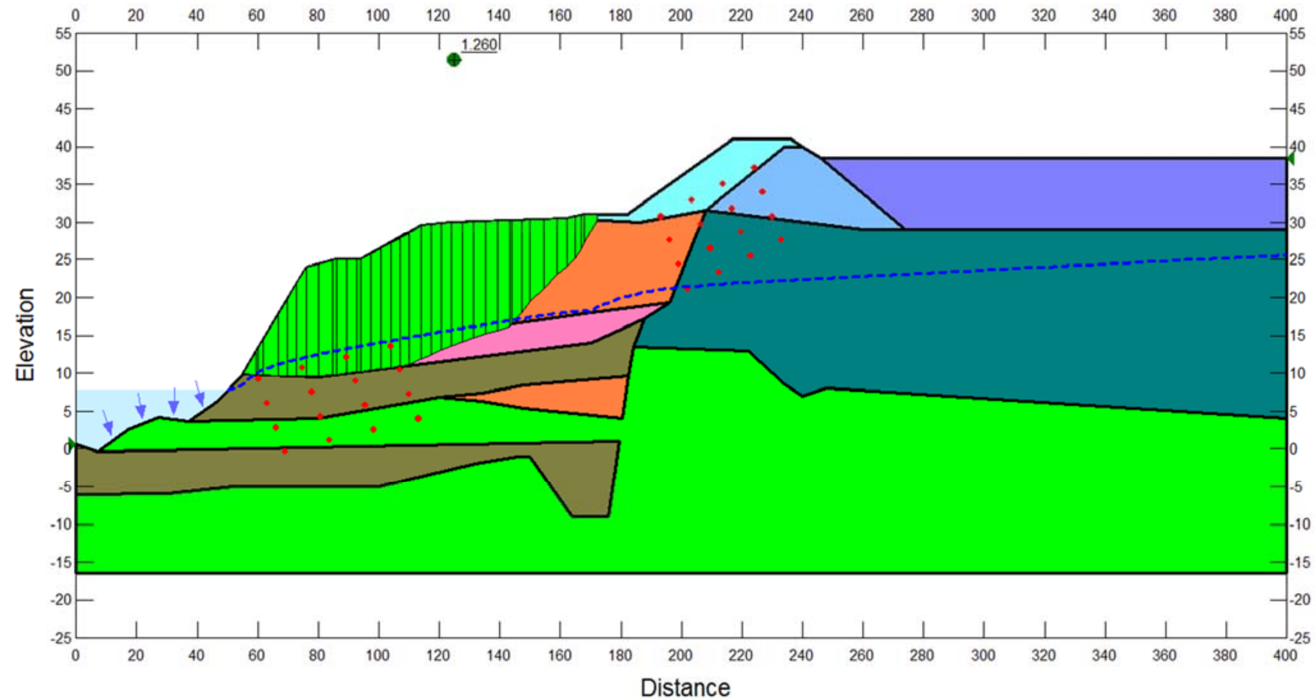
STEADY STATE SEEPAGE, SATURATED MODEL

STEADY STATE SEEPAGE (WSE 7.77 FT)

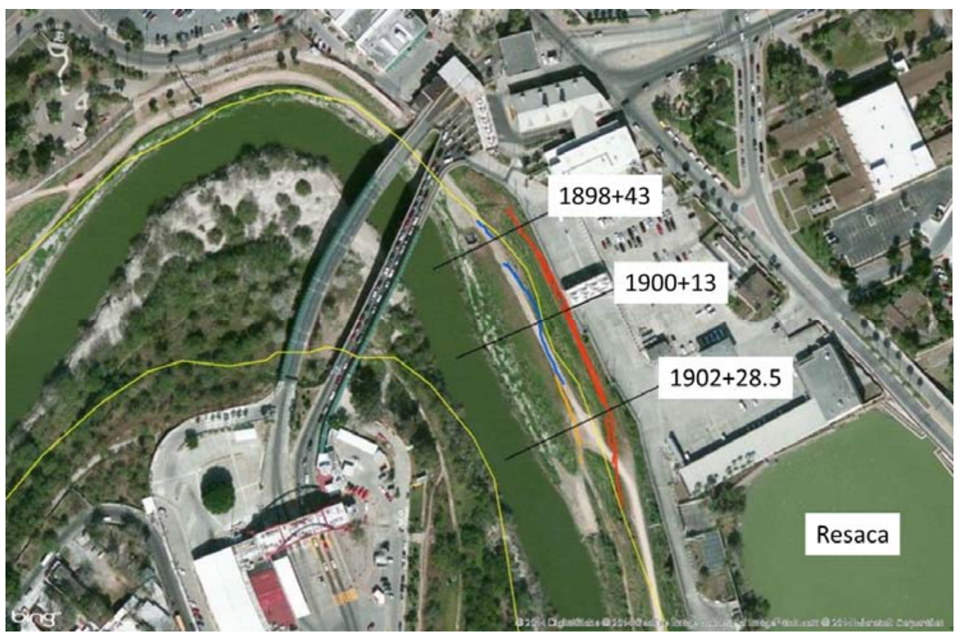
STATION 1900+13

FEB-2015 **PLATE - 1**





Minimum factor of safety (FoS): 1.26



material	unit weight (pcf)	c (psf)	phi (deg)
CH Pleistocene	121.98	200.00	24.00
CL-Holocene	123.37	800.00	17.30
SM	117.00	0.00	32.00
ML	119.38	300.00	32.60
2012 Levee Fill	127.34	620.00	29.20
Levee Fill	127.34	620.00	29.20
Historic Fill	127.34	200.00	24.00
soft ML	125.98	150*	0.00

*varied to explore impact of S_U, actual range should fall between 150-500 psf

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

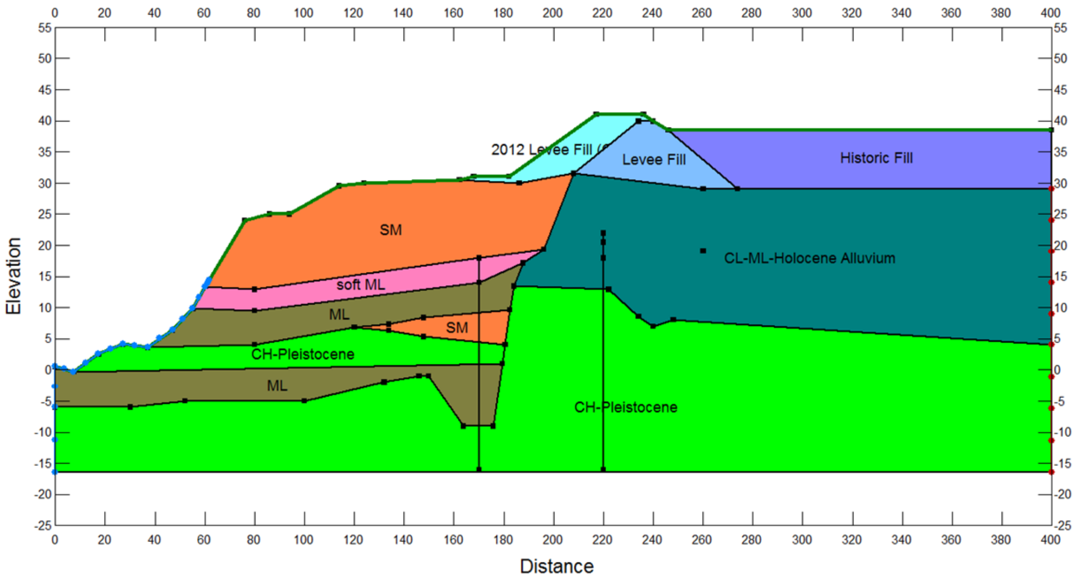
IBWC-BROWNSVILLE LEVEE

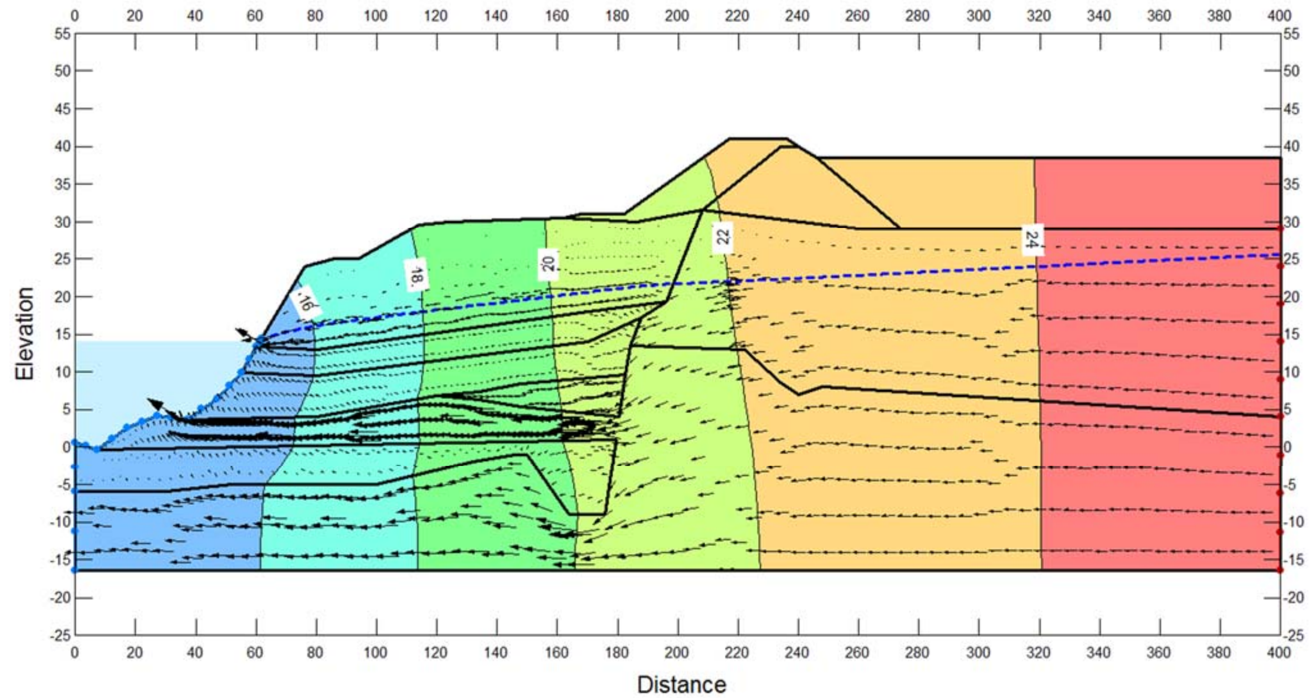
STABILITY MODEL

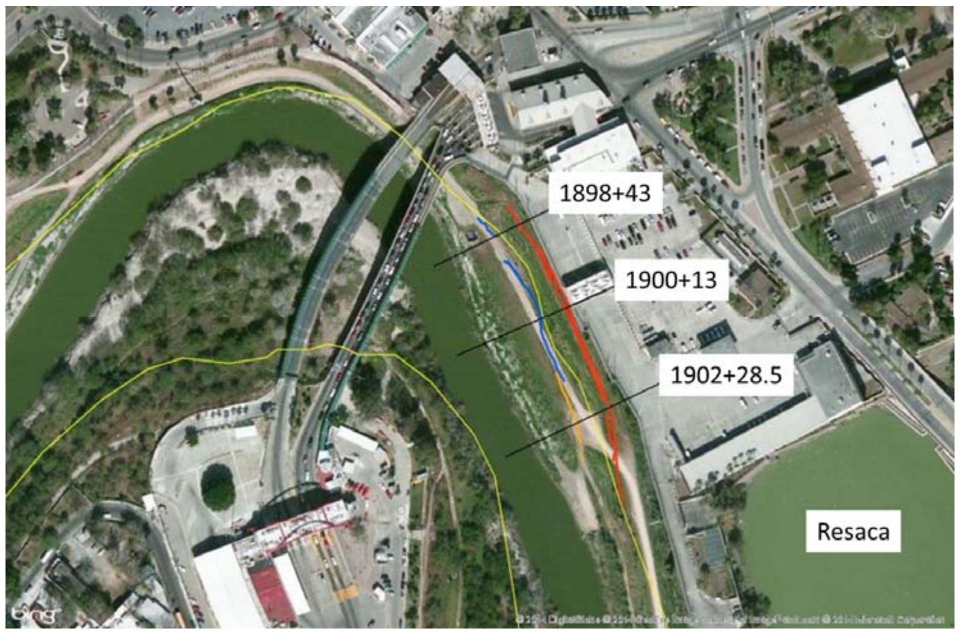
STEADY STATE FOS(WSE 7.77 FT)

STATION 1900+13

FEB-2015 **PLATE - 2**







material	K_{sat} (ft/s)	n	m_v (1/psf)	ratio
CH Pleistocene	3.30E-08	0.44	3.60E-06	0.2
CL-Holocene	3.30E-08	0.43	2.50E-06	0.2
SM	3.30E-07	0.3	5.00E-06	0.2
ML	1.00E-07	0.43	1.00E-05	0.2
2012 Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Historic Fill	3.30E-08	0.4	3.74E-06	0.2
soft ML	1.00E-07	0.45	1.00E-05	1

Boundary Conditions	type	magnitude (ft)
P3-32	head	22
P3-33	head	18
River	head	14.31
Protected side	head	25.59

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

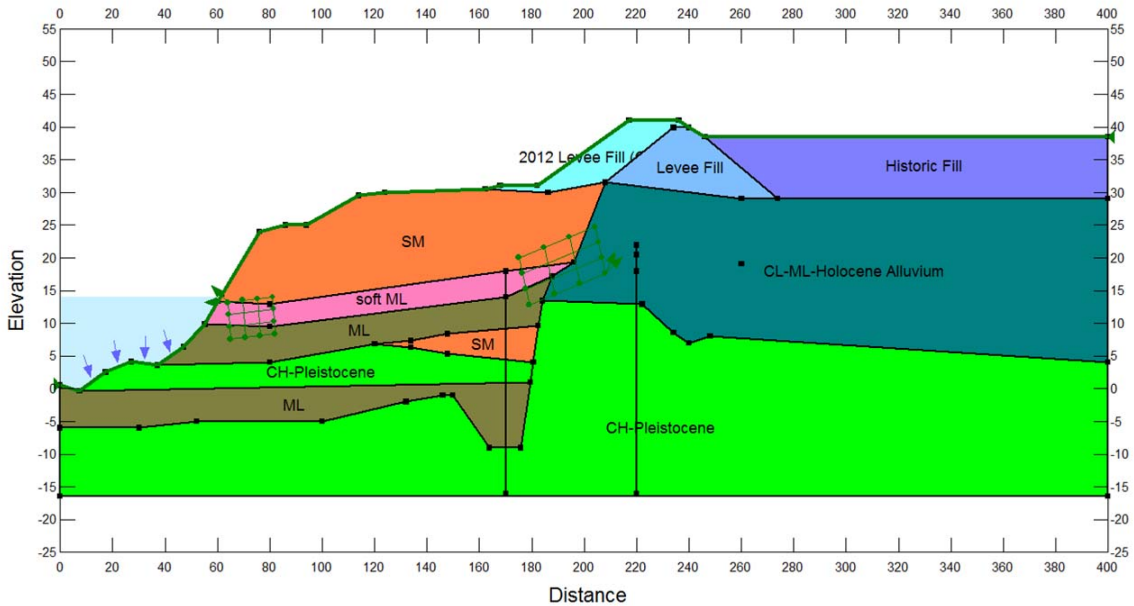
IBWC-BROWNSVILLE LEVEE

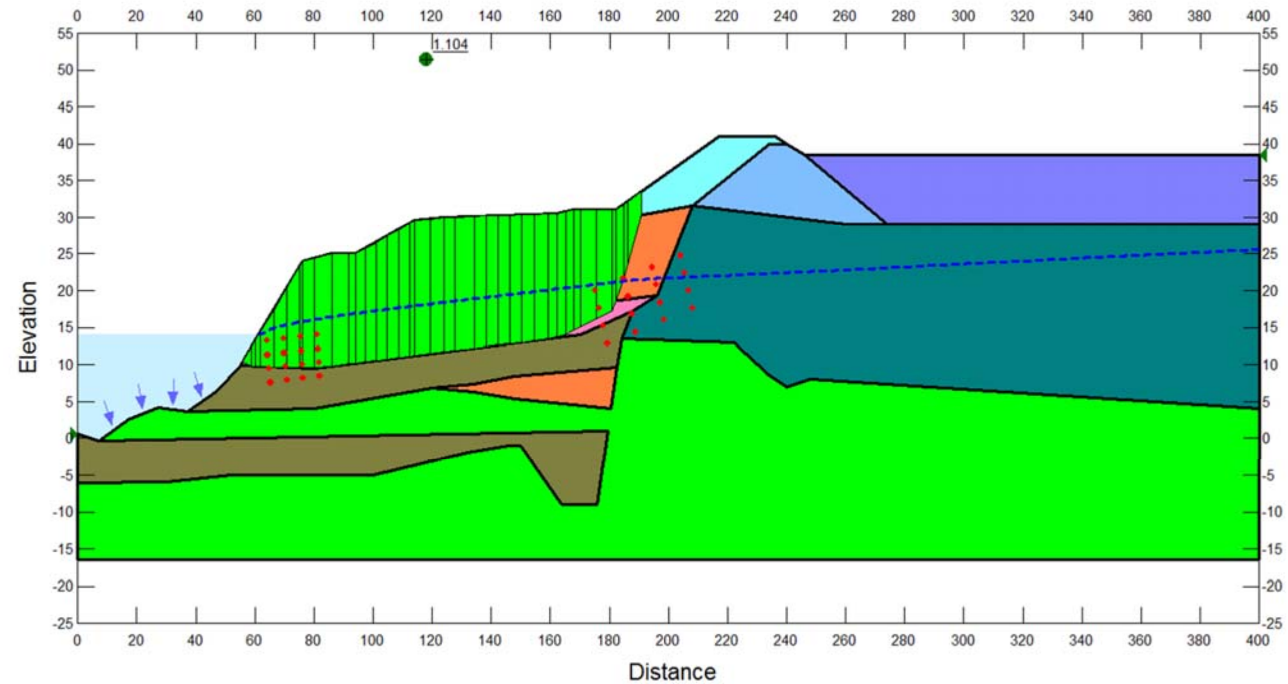
STEADY STATE SEEPAGE, SATURATED MODEL

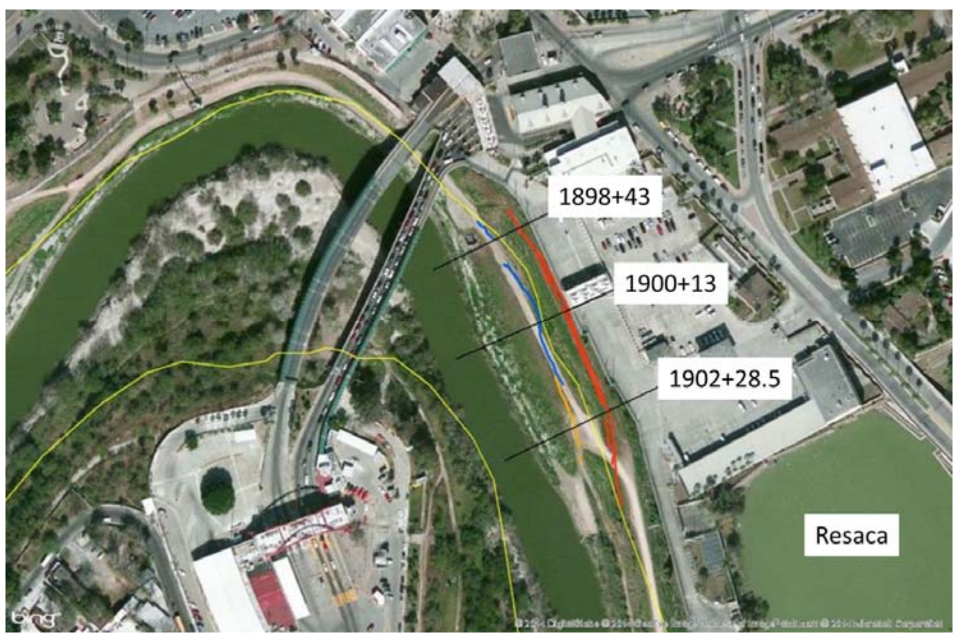
STEADY STATE SEEPAGE (WSE 14.31 FT)

STATION 1900+13

FEB-2015 **PLATE - 3**







Minimum factor of safety (FoS): 1.10

material	unit weight (pcf)	c (psf)	phi (deg)
CH Pleistocene	121.98	200.00	24.00
CL-Holocene	123.37	800.00	17.30
SM	117.00	0.00	32.00
ML	119.38	300.00	32.60
2012 Levee Fill	127.34	620.00	29.20
Levee Fill	127.34	620.00	29.20
Historic Fill	127.34	200.00	24.00
soft ML	125.98	150*	0.00

*varied to explore impact of S_U, actual range should fall between 150-500 psf

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

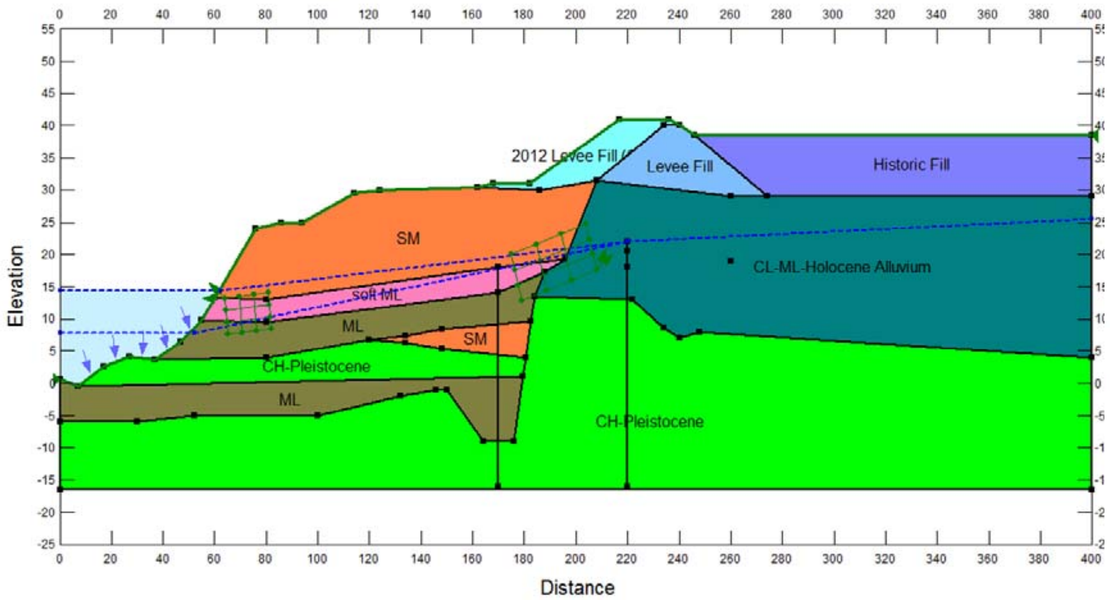
IBWC-BROWNSVILLE LEVEE

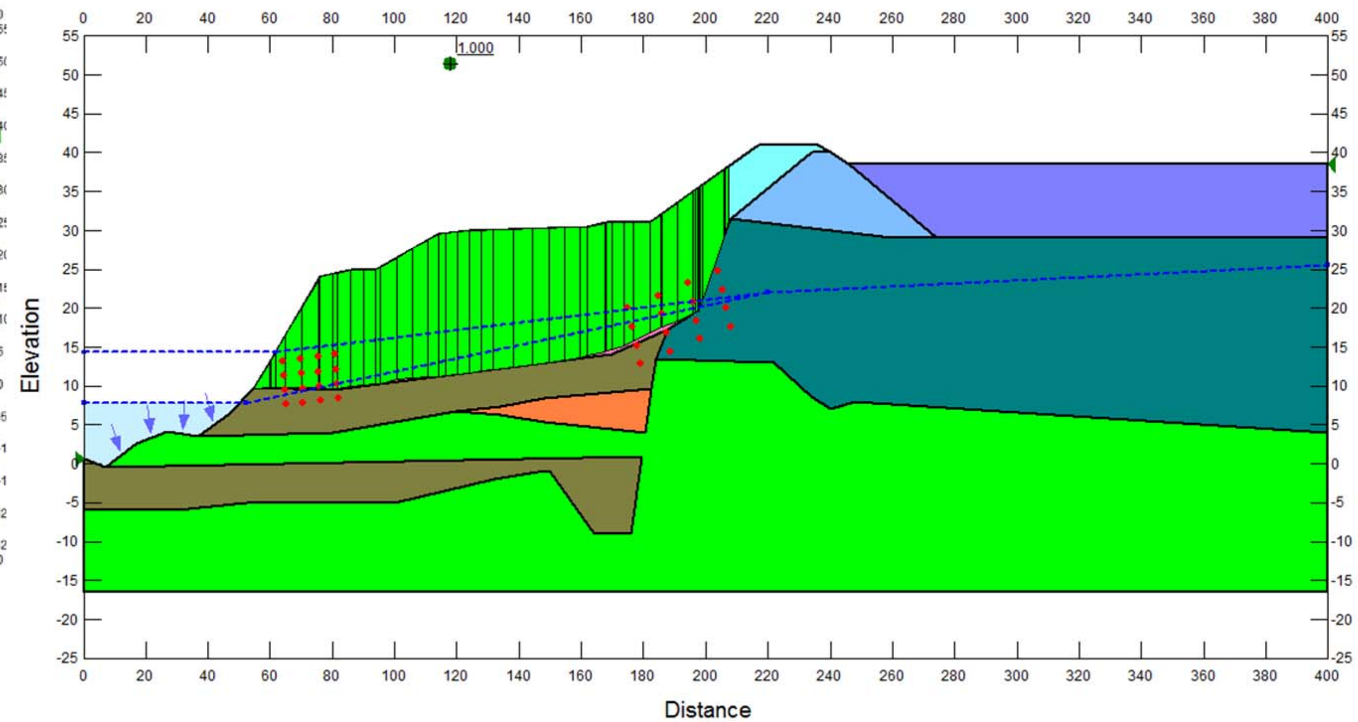
STABILITY MODEL

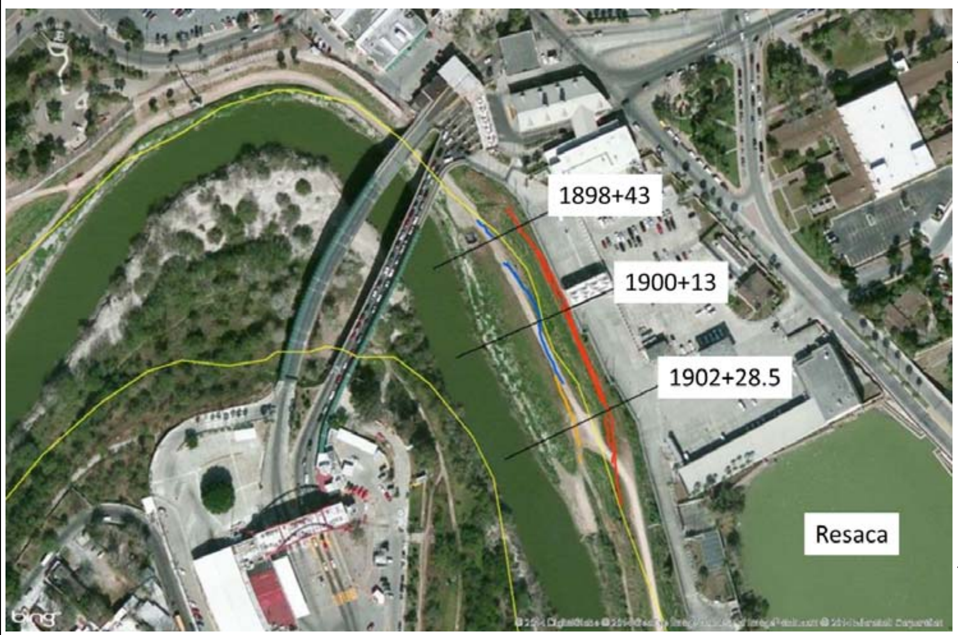
STEADY STATE FOS(WSE 14.31 FT)

STATION 1900+13

FEB-2015 **PLATE - 4**







material	unit weight (pcf)	c' (psf)	phi' (deg)	total stress c (psf)	total stress phi (deg)
CH Pleistocene	121.98	200.00	24.00	2320.00	0.00
CL-Holocene	123.37	800.00	17.30	400.00	0.00
SM	117.00	0.00	32.00	0.00	32.00
ML	119.38	300.00	32.60	0.00	29.00
2012 Levee Fill	127.34	620.00	29.20	5000.00	0.00
Levee Fill	127.34	620.00	29.20	5000.00	0.00
Historic Fill	127.34	200.00	24.00	400.00	15.00
soft ML	125.98	150*	0.00	168.00	0.00

*varied to explore impact of S_U, actual range should fall between 150-500 psf

Minimum factor of safety (FoS): 1.00

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

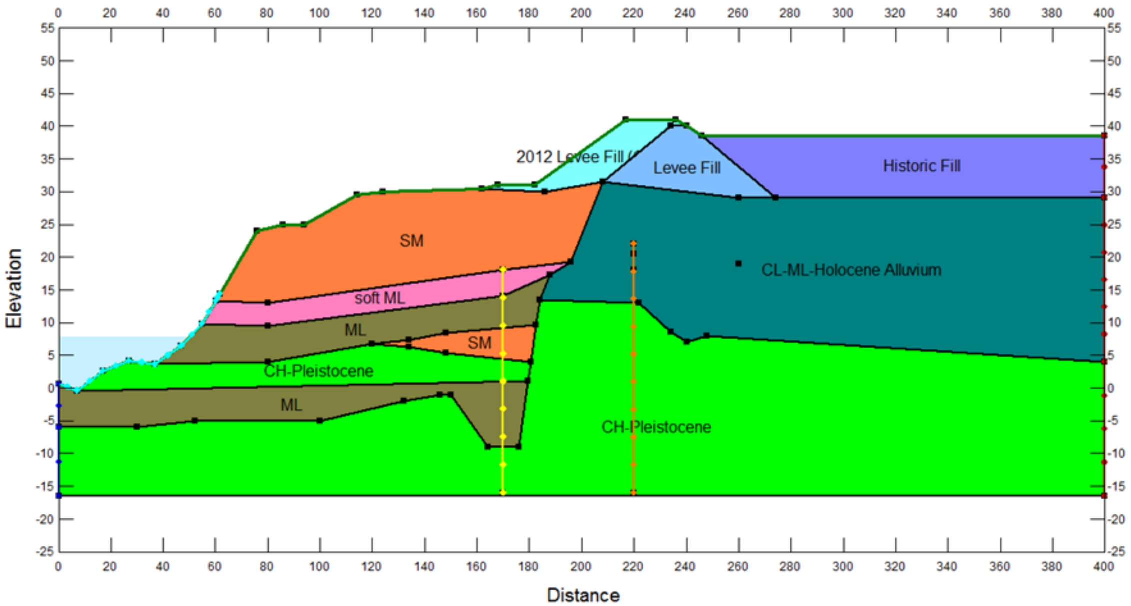
IBWC-BROWNSVILLE LEVEE

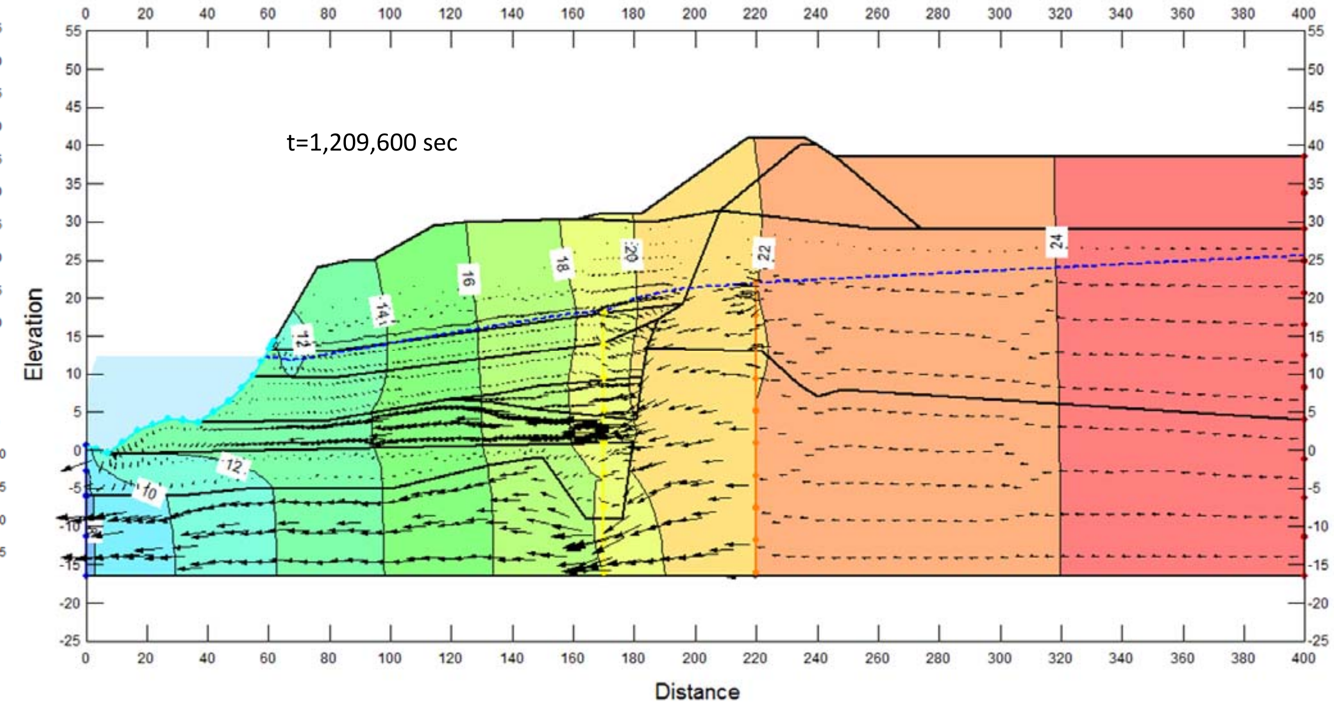
RAPID DRAWDOWN STABILITY

RAPID DRAWDOWN (WSE 7.77 & 14.31 FT)

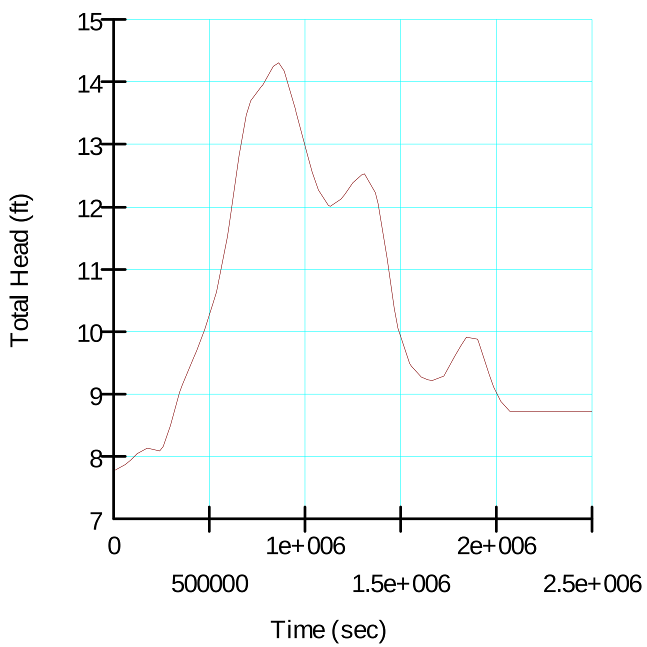
STATION 1900+13

FEB-2015 **PLATE - 5**





IBWC: Hydrograph



material	K_{sat} (ft/s)	n	m_v (1/psf)	ratio
CH Pleistocene	3.30E-08	0.44	3.60E-06	0.2
CL-Holocene	3.30E-08	0.43	2.50E-06	0.2
SM	3.30E-07	0.3	5.00E-06	0.2
ML	1.00E-07	0.43	1.00E-05	0.2
2012 Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Historic Fill	3.30E-08	0.4	3.74E-06	0.2
soft ML	1.00E-07	0.45	1.00E-05	1

Boundary Conditions	type	magnitude (ft)
P3-32	head	22
P3-33	head	18
River*	head	7.77
Protected side	head	25.59

*function above channel surface, see plot lower left corner (light blue)

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

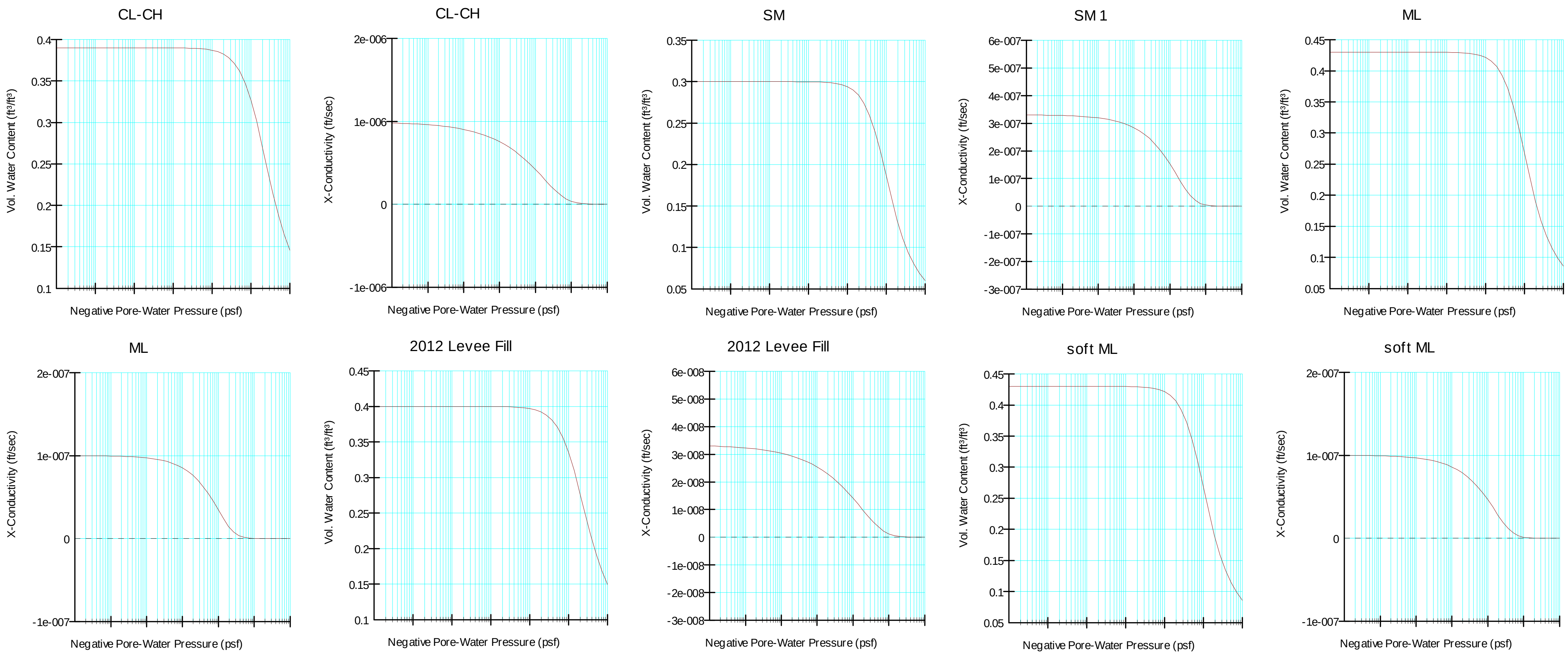
IBWC-BROWNSVILLE LEVEE

HYDROGRAPH, SATURATED MODEL

STEADY STATE SEEPAGE (WSE 14.31 FT)

STATION 1900+13

FEB-2015 **PLATE - 6**



U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

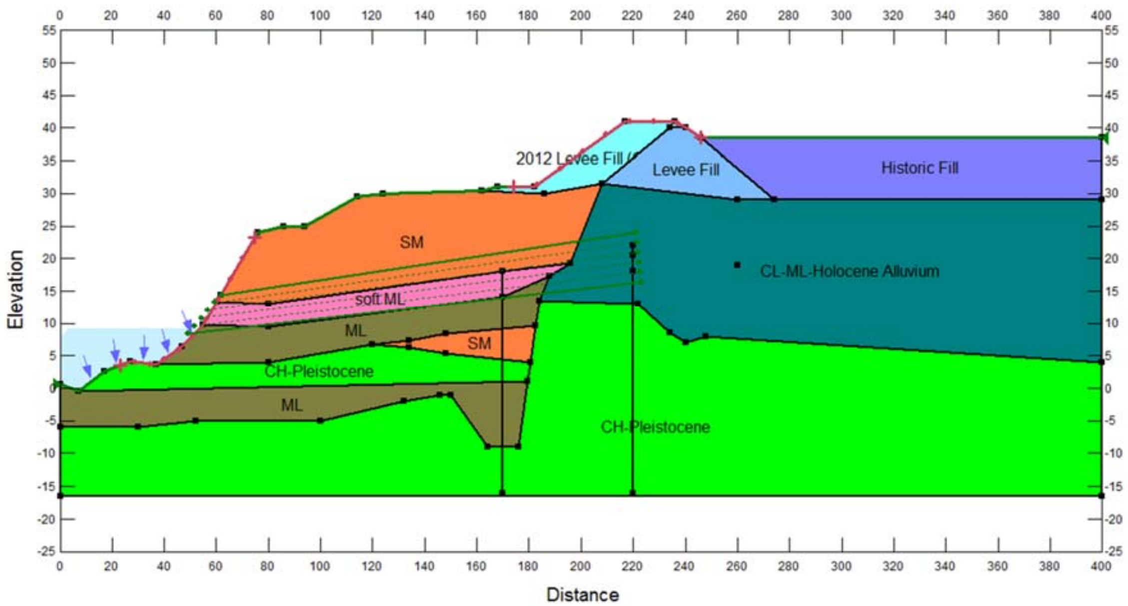
IBWC-BROWNSVILLE LEVEE

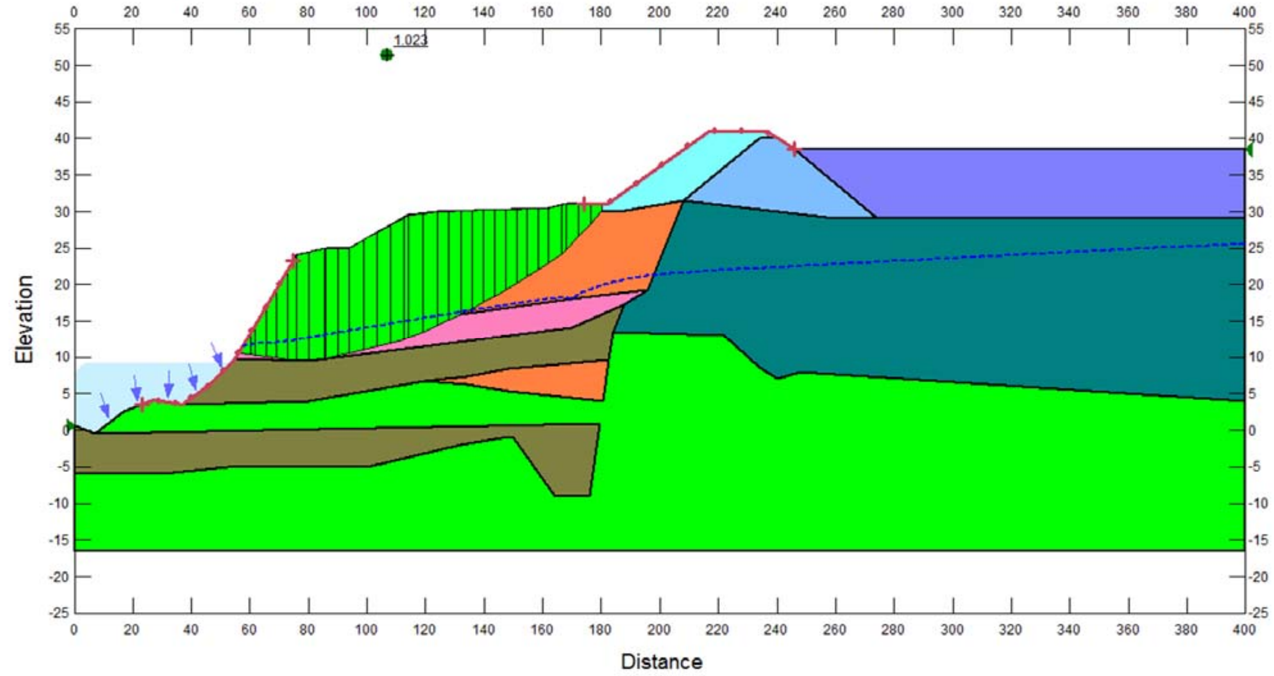
TRANSIENT HYDRAULIC PROPETIES

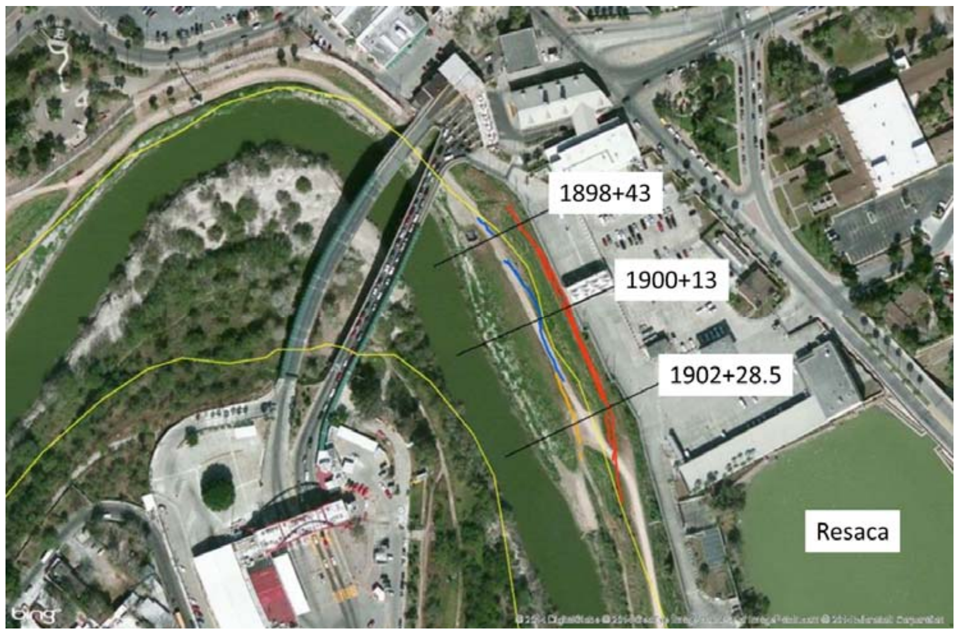
SWCC AND HCF'S

STATION 1900+13

FEB-2015 **PLATE - 7**







Minimum factor of safety (FoS): 1.02

material	unit weight (pcf)	c (psf)	phi (deg)
CH Pleistocene	121.98	200.00	24.00
CL-Holocene	123.37	800.00	17.30
SM	117.00	0.00	32.00
ML	119.38	300.00	32.60
2012 Levee Fill	127.34	620.00	29.20
Levee Fill	127.34	620.00	29.20
Historic Fill	127.34	200.00	24.00
soft ML	125.98	150*	0.00

*varied to explore impact of S_U, actual range should fall between 150-500 psf

U.S. ARMY CORPS OF ENGINEERS

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IBWC-BROWNSVILLE LEVEE

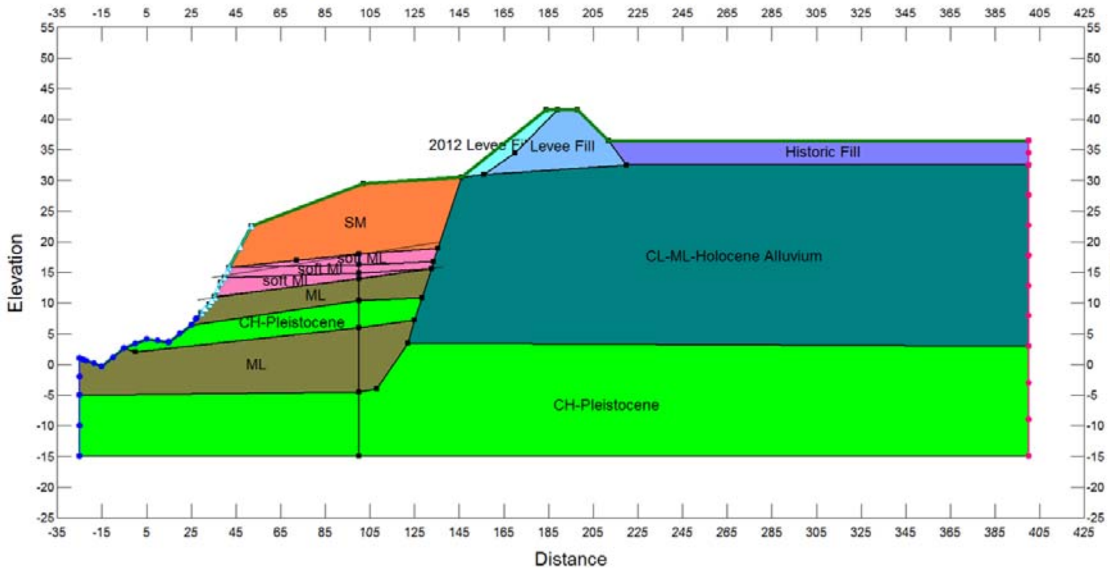
STABILITY MODEL

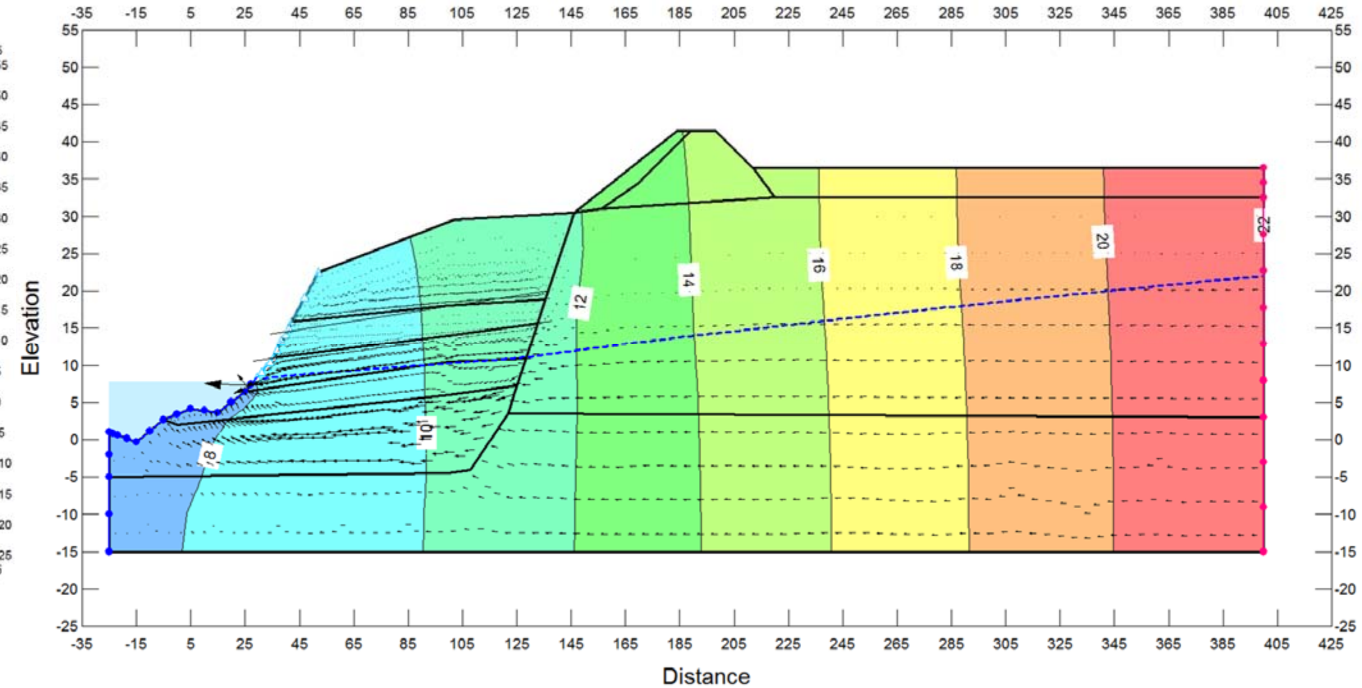
TRANSIENT FOS (HYDROGRAPH)

STATION 1900+13

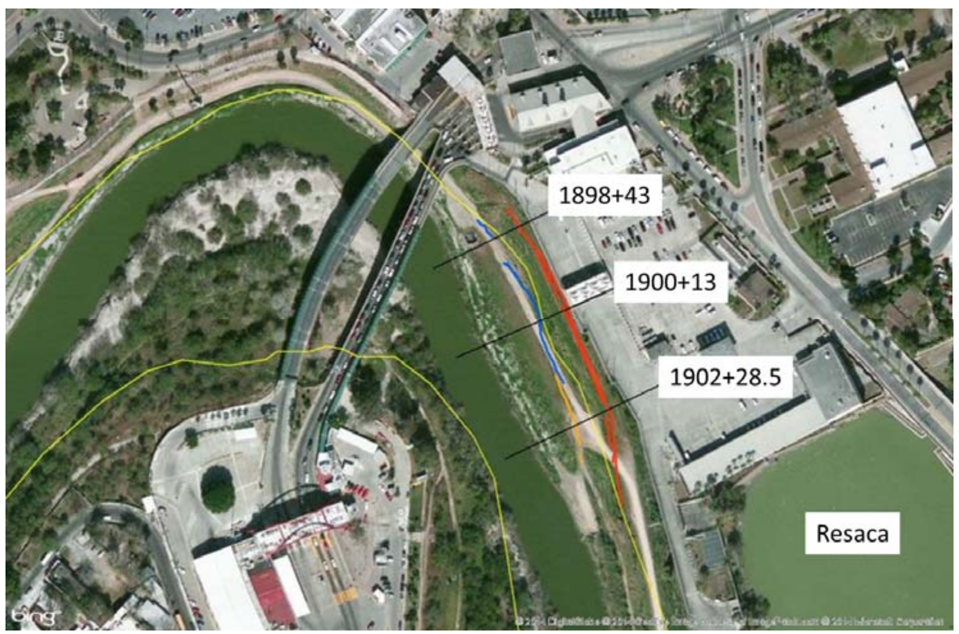
FEB-2015

PLATE - 8





Boundary Conditions	type	magnitude (ft)
River	head	7.77
Protected side	head	22.00



material	K_{sat} (ft/s)	n	m_v (1/psf)	ratio
CH Pleistocene	3.30E-08	0.44	3.60E-06	0.2
CL-Holocene	3.30E-08	0.43	2.50E-06	0.2
SM	3.30E-07	0.3	5.00E-06	0.2
ML	1.00E-07	0.43	1.00E-05	0.2
2012 Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Historic Fill	3.30E-08	0.4	3.74E-06	0.2
soft ML	1.00E-07	0.45	1.00E-05	1

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

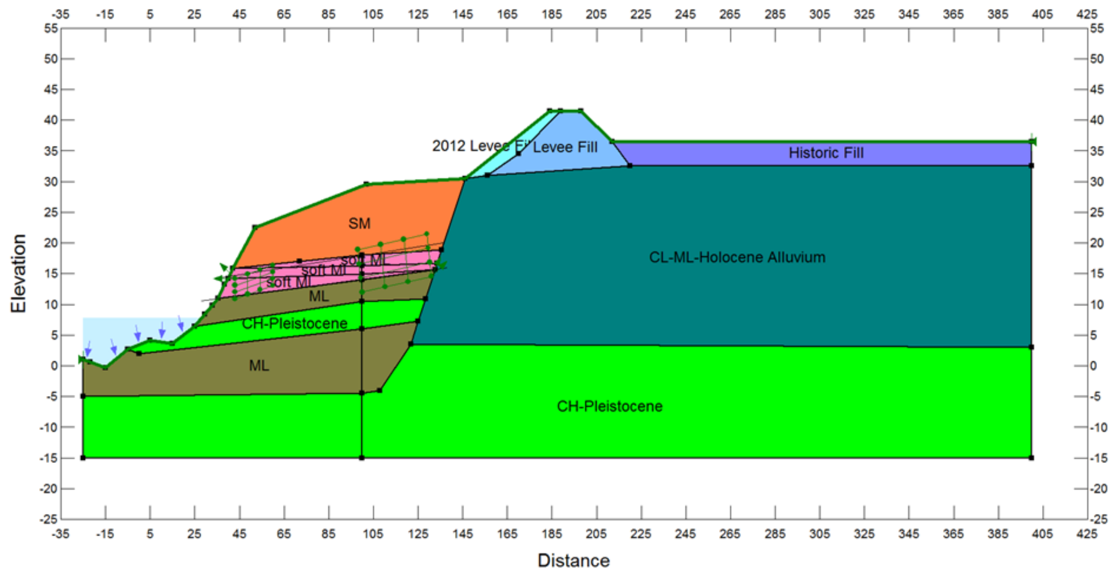
IBWC-BROWNSVILLE LEVEE

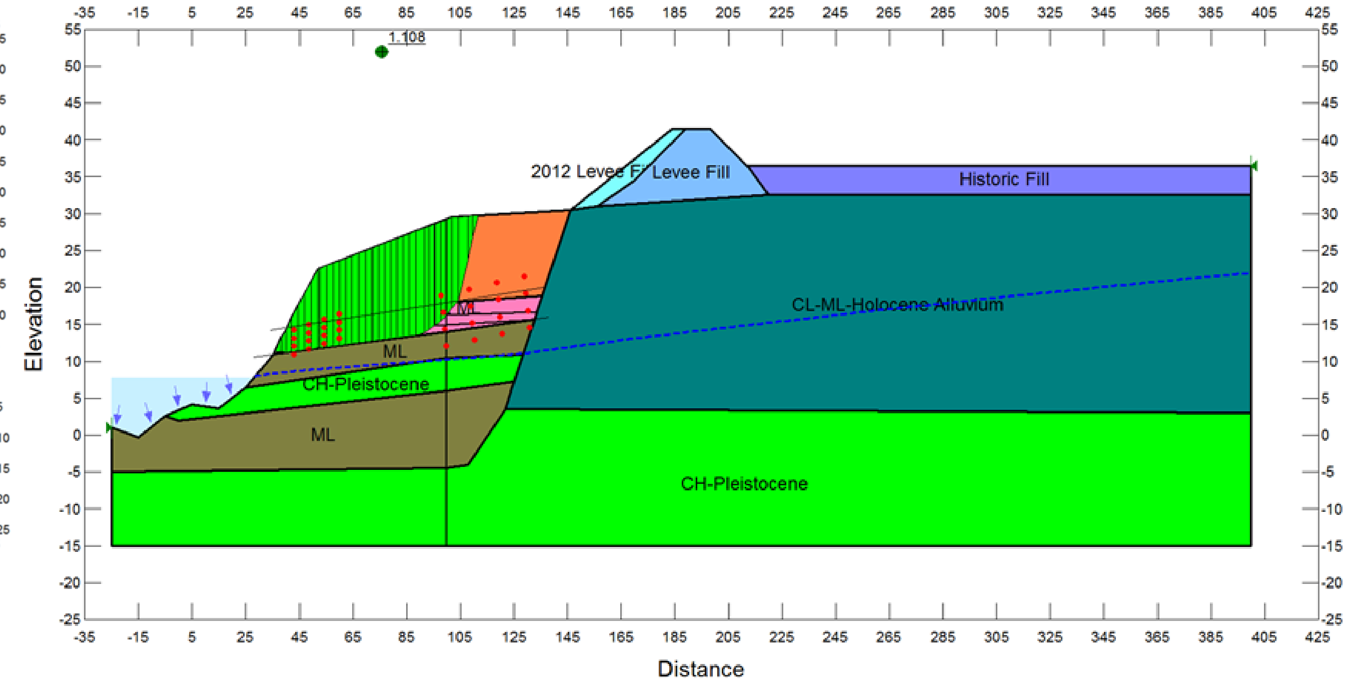
STEADY STATE SEEPAGE, SATURATED MODEL

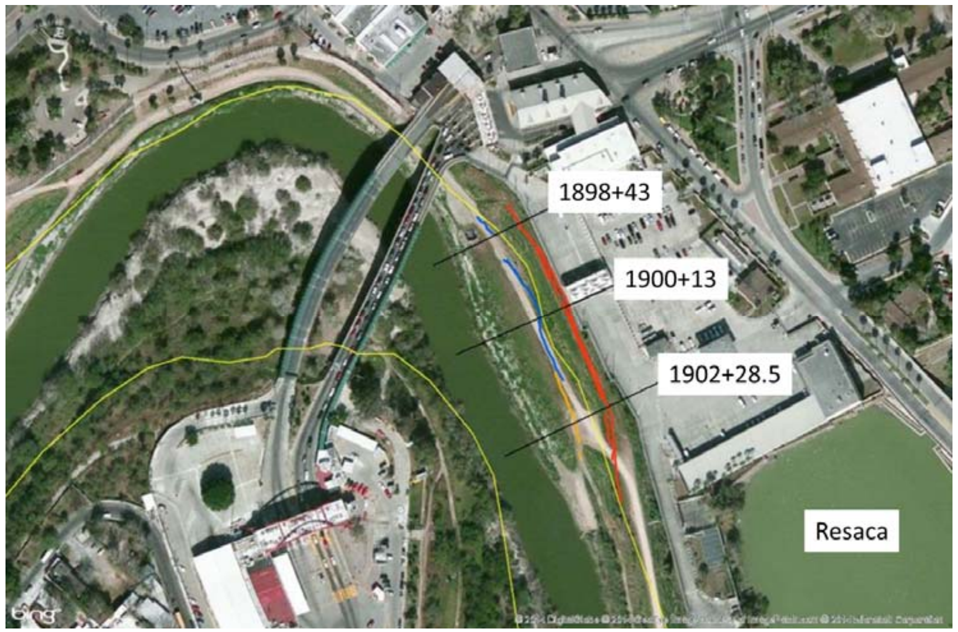
STEADY STATE SEEPAGE (WSE 7.77 FT)

STATION 1898+43

FEB-2015 **PLATE - 9**







Minimum factor of safety (FoS): 1.11

material	unit weight (pcf)	c (psf)	phi (deg)
CH Pleistocene	121.98	200.00	24.00
CL-Holocene	123.37	800.00	17.30
SM	117.00	0.00	32.00
ML	119.38	300.00	32.60
2012 Levee Fill	127.34	620.00	29.20
Levee Fill	127.34	620.00	29.20
Historic Fill	127.34	200.00	24.00
soft ML	125.98	200	0.00

U.S. ARMY CORPS OF ENGINEERS

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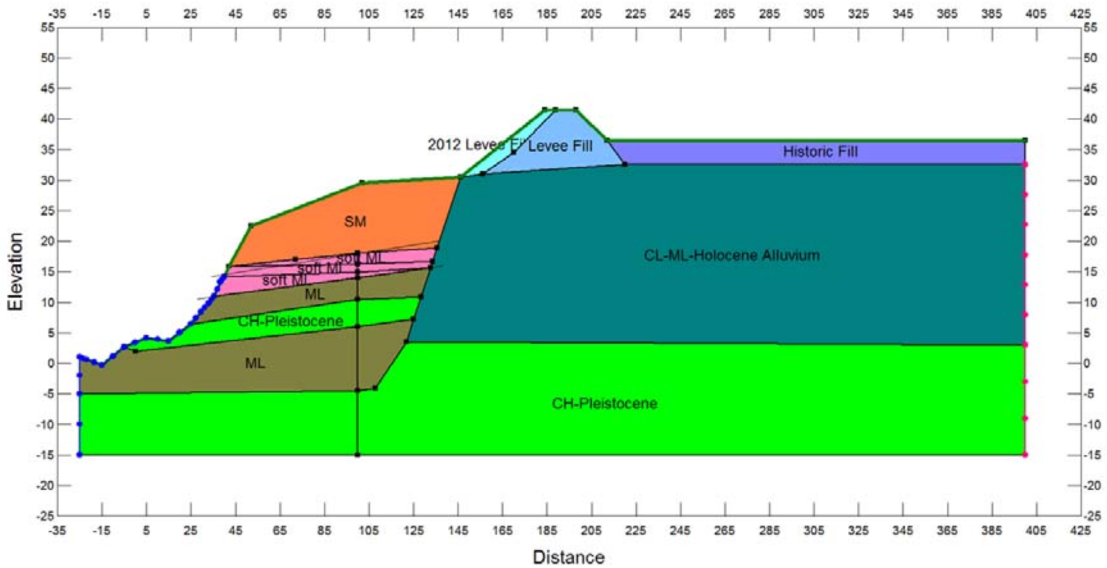
IBWC-BROWNSVILLE LEVEE

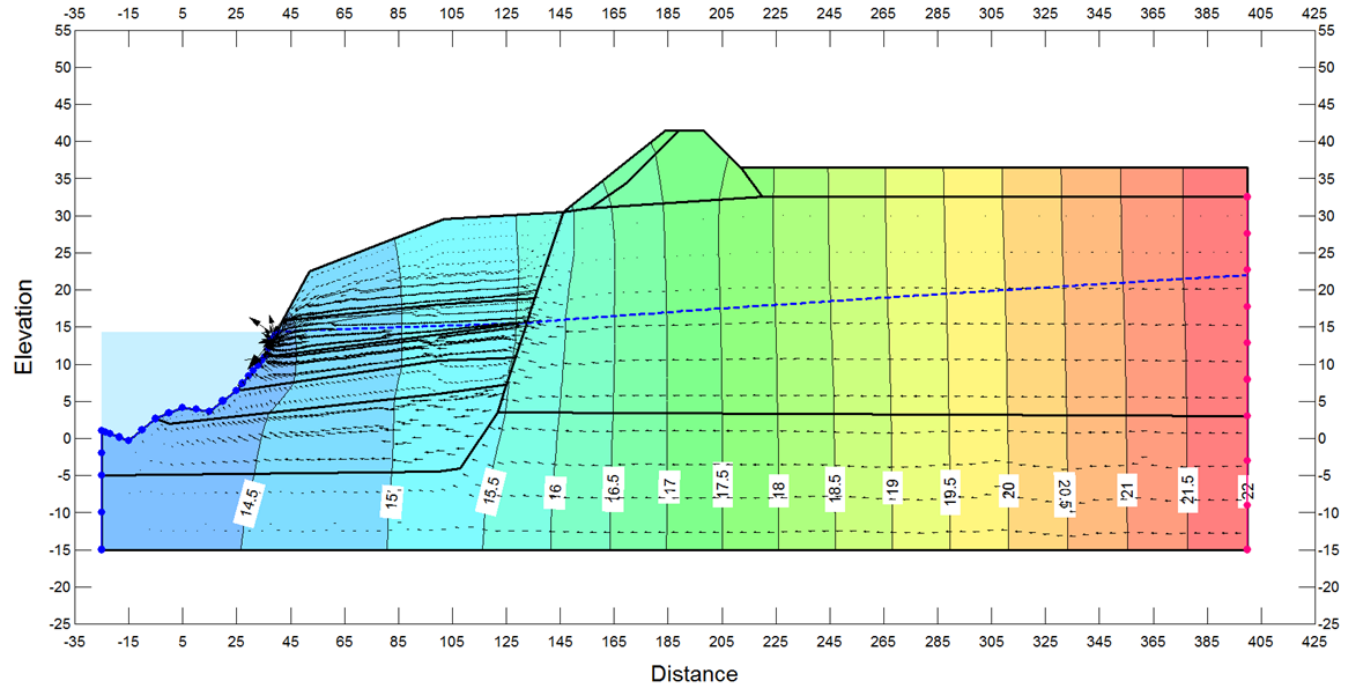
STABILITY MODEL

STEADY STATE FOS(WSE 7.77 FT)

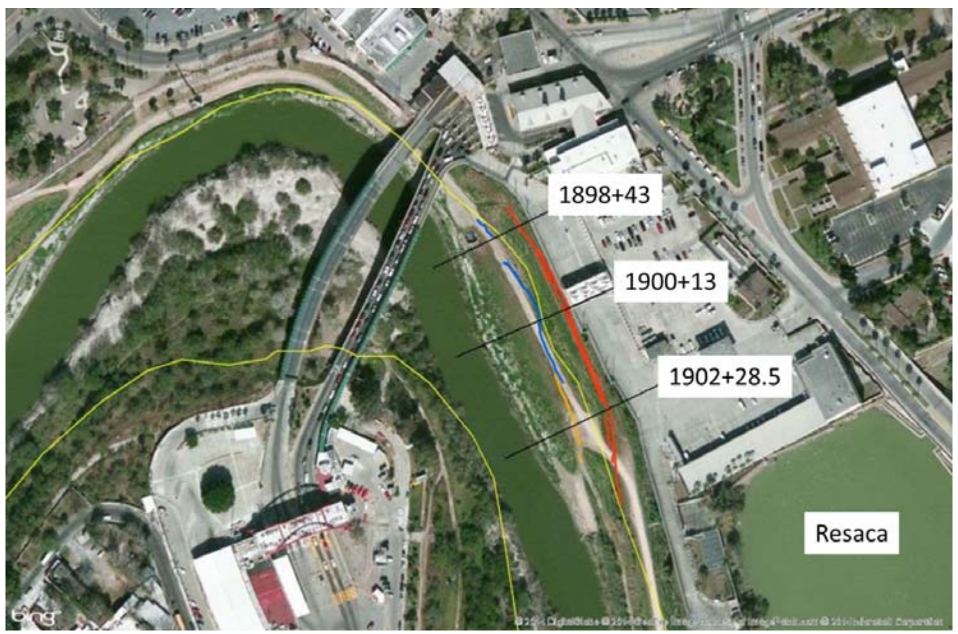
STATION 1898+43

FEB-2015 **PLATE - 10**





Boundary Conditions	type	magnitude (ft)
River	head	14.31
Protected side	head	22.00



material	K_{sat} (ft/s)	n	m_v (1/psf)	ratio
CH Pleistocene	3.30E-08	0.44	3.60E-06	0.2
CL-Holocene	3.30E-08	0.43	2.50E-06	0.2
SM	3.30E-07	0.3	5.00E-06	0.2
ML	1.00E-07	0.43	1.00E-05	0.2
2012 Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Historic Fill	3.30E-08	0.4	3.74E-06	0.2
soft ML	1.00E-07	0.45	1.00E-05	1

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

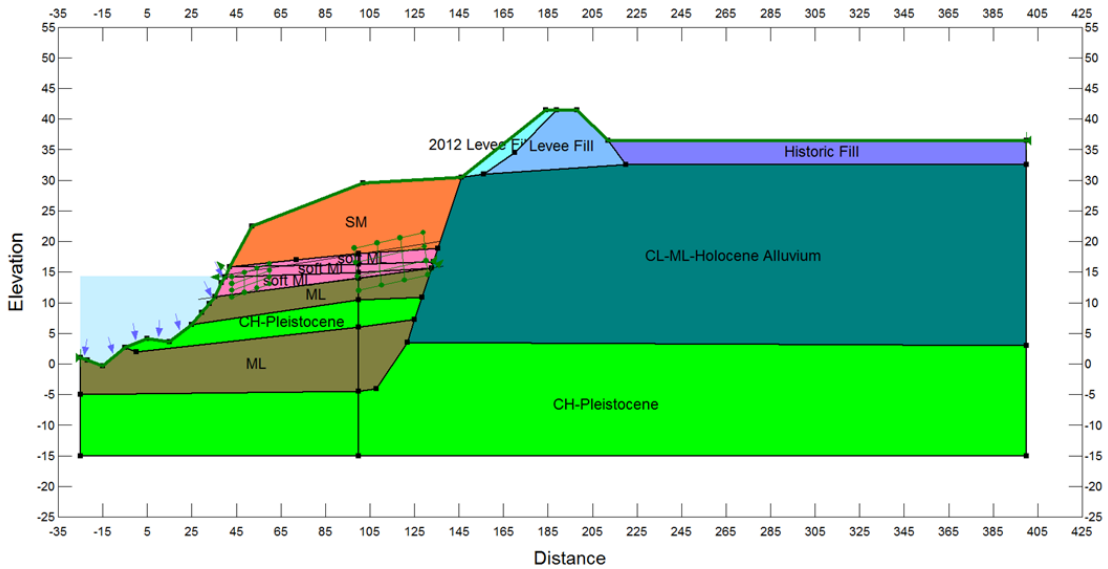
IBWC-BROWNSVILLE LEVEE

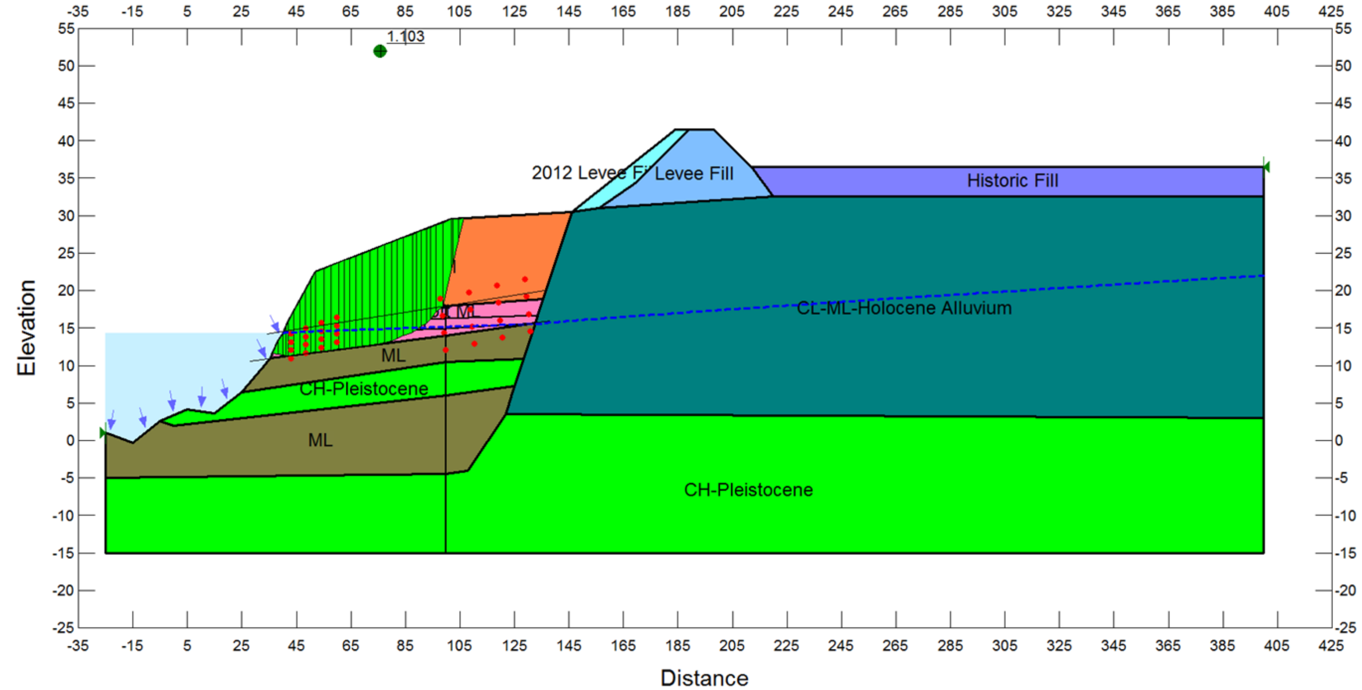
STEADY STATE SEEPAGE, SATURATED MODEL

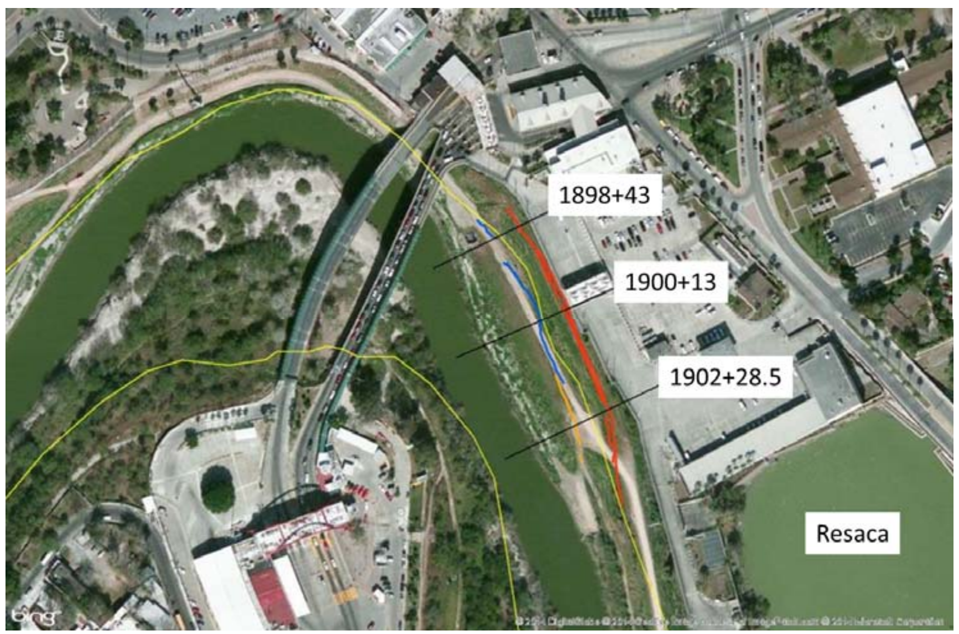
STEADY STATE SEEPAGE (WSE 14.31 FT)

STATION 1898+43

FEB-2015 **PLATE - 11**







Minimum factor of safety (FoS): 1.10

material	unit weight (pcf)	c (psf)	phi (deg)
CH Pleistocene	121.98	200.00	24.00
CL-Holocene	123.37	800.00	17.30
SM	117.00	0.00	32.00
ML	119.38	300.00	32.60
2012 Levee Fill	127.34	620.00	29.20
Levee Fill	127.34	620.00	29.20
Historic Fill	127.34	200.00	24.00
soft ML	125.98	200	0.00

U.S. ARMY CORPS OF ENGINEERS

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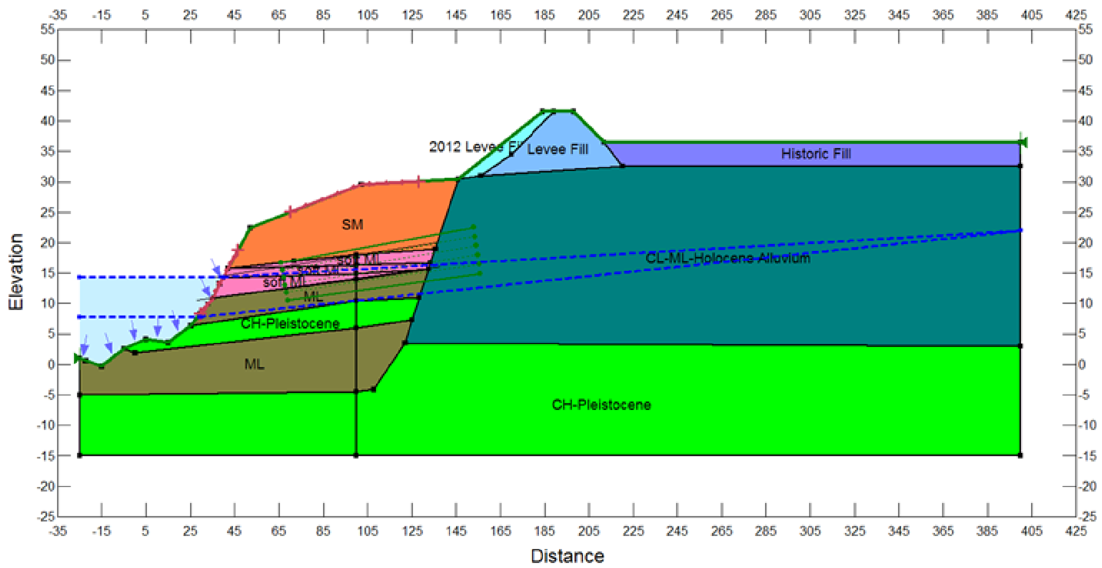
IBWC-BROWNSVILLE LEVEE

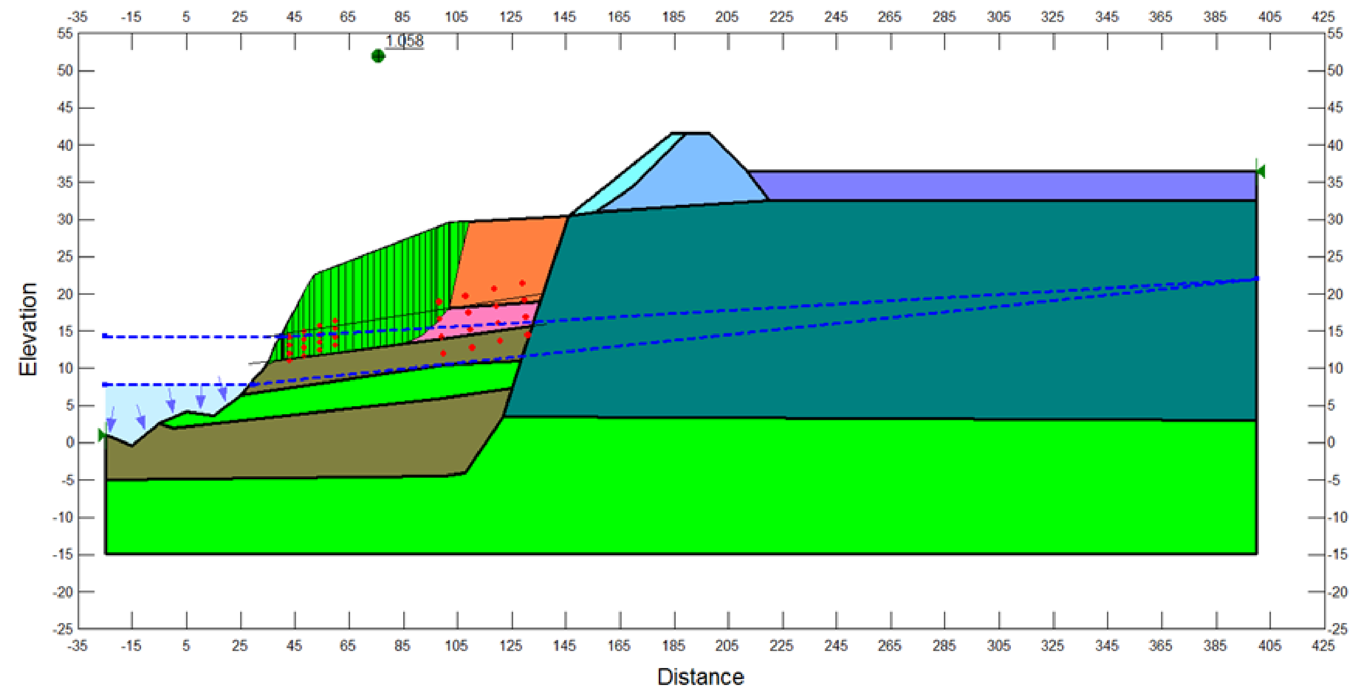
STABILITY MODEL

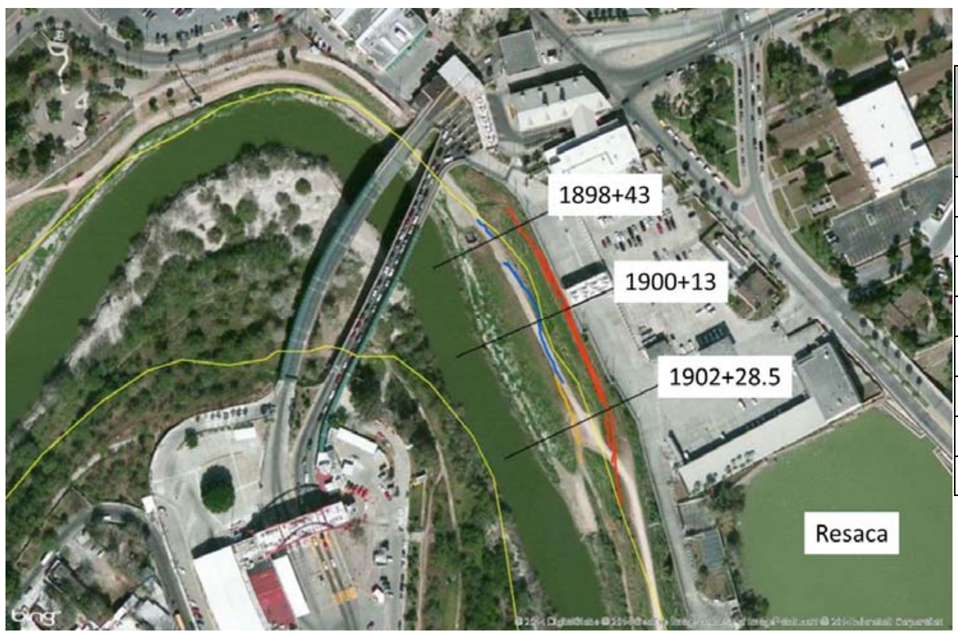
STEADY STATE FOS(WSE 14.31 FT)

STATION 1898+43

FEB-2015 **PLATE - 12**







material	unit weight (pcf)	c' (psf)	phi' (deg)	total stress c (psf)	total stress phi (deg)
CH Pleistocene	121.98	200.00	24.00	2320.00	0.00
CL-Holocene	123.37	800.00	17.30	400.00	0.00
SM	117.00	0.00	32.00	0.00	32.00
ML	119.38	300.00	32.60	0.00	29.00
2012 Levee Fill	127.34	620.00	29.20	5000.00	0.00
Levee Fill	127.34	620.00	29.20	5000.00	0.00
Historic Fill	127.34	200.00	24.00	400.00	15.00
soft ML	125.98	200.00	0.00	200.00	0.00

Minimum factor of safety (FoS): 1.06

U.S. ARMY CORPS OF ENGINEERS

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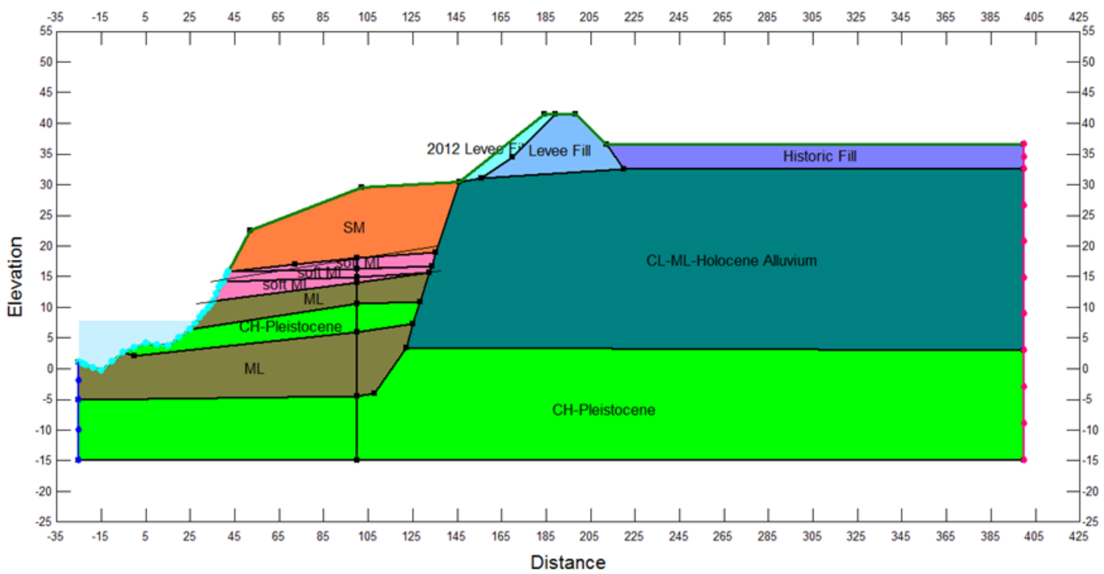
IBWC-BROWNSVILLE LEVEE

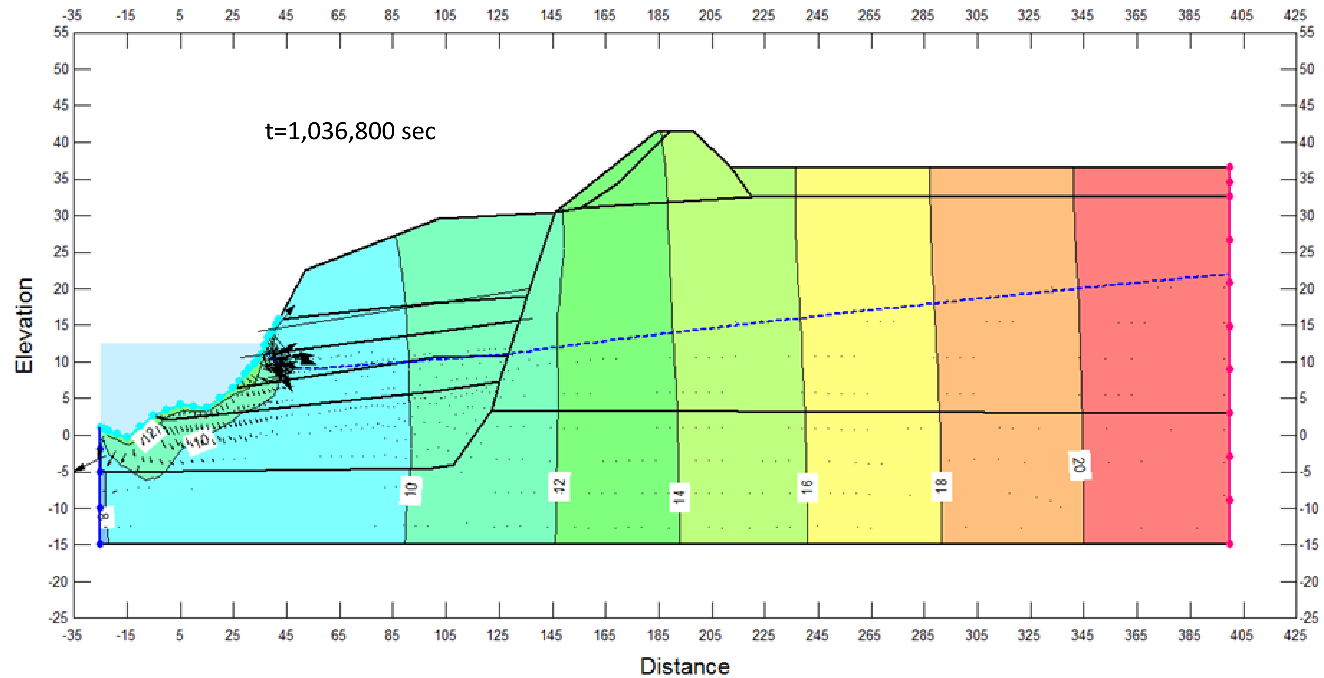
RAPID DRAWDOWN STABILITY

RAPID DRAWDOWN (WSE 7.77 & 14.31 FT)

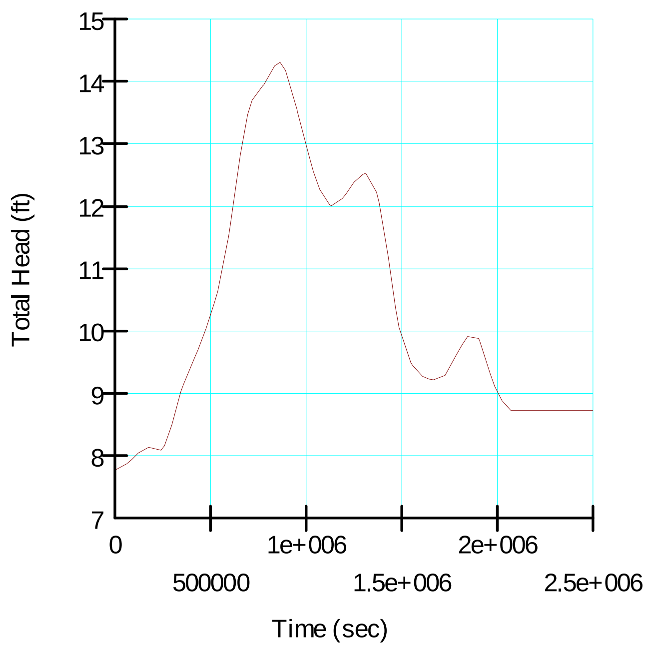
STATION 1898+43

FEB-2015 **PLATE - 13**





IBWC: Hydrograph



material	K_{sat} (ft/s)	n	m_v (1/psf)	ratio
CH Pleistocene	3.30E-08	0.44	3.60E-06	0.2
CL-Holocene	3.30E-08	0.43	2.50E-06	0.2
SM	3.30E-07	0.3	5.00E-06	0.2
ML	1.00E-07	0.43	1.00E-05	0.2
2012 Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Historic Fill	3.30E-08	0.4	3.74E-06	0.2
soft ML	1.00E-07	0.45	1.00E-05	1

Boundary Conditions	type	magnitude (ft)
River*	head	7.77
Protected side	head	25.59

*function above channel surface, see plot lower left corner (light blue)

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

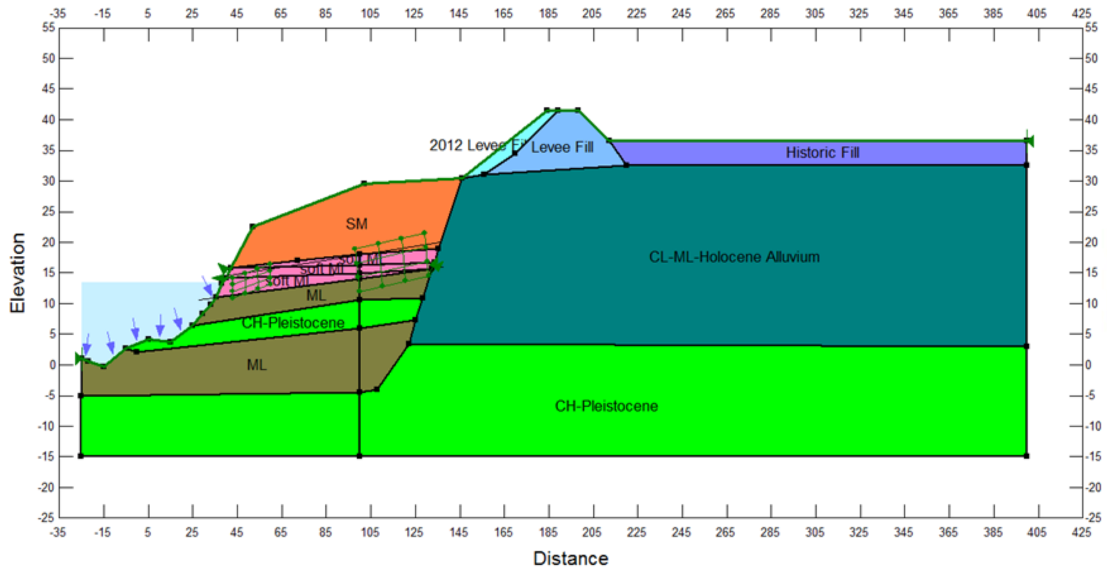
IBWC-BROWNSVILLE LEVEE

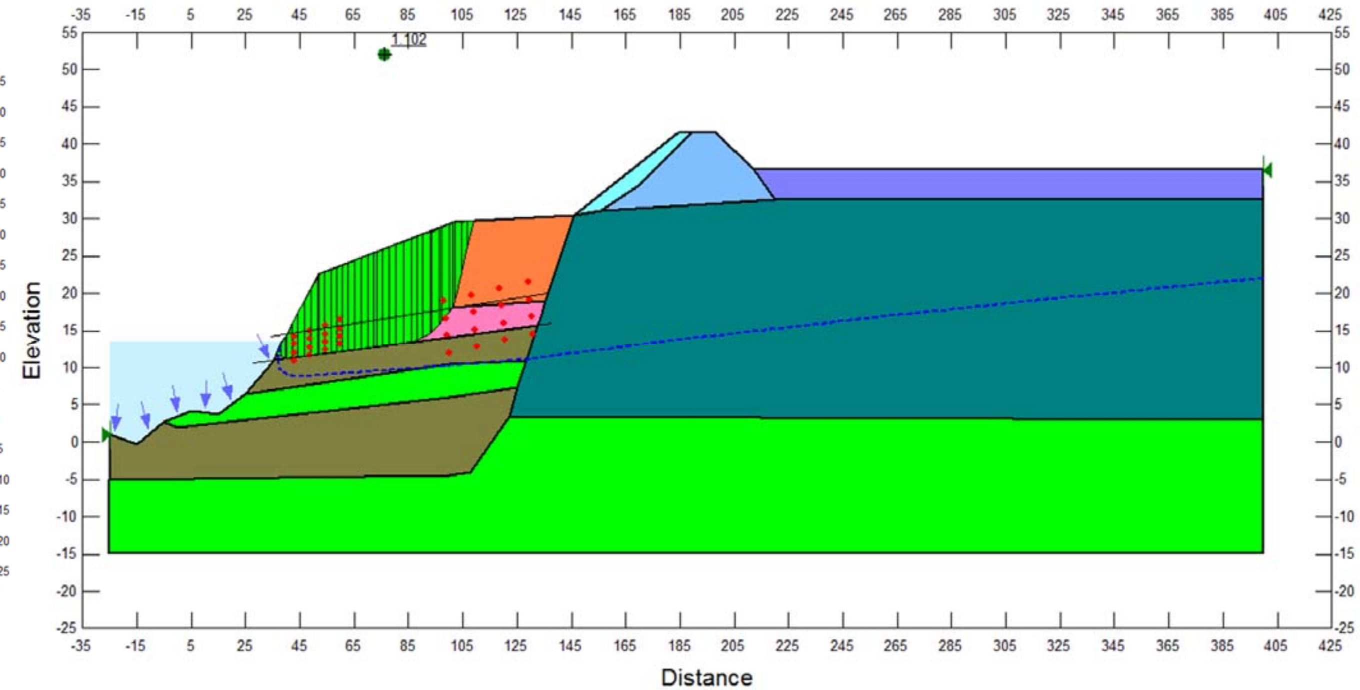
HYDROGRAPH, SATURATED MODEL

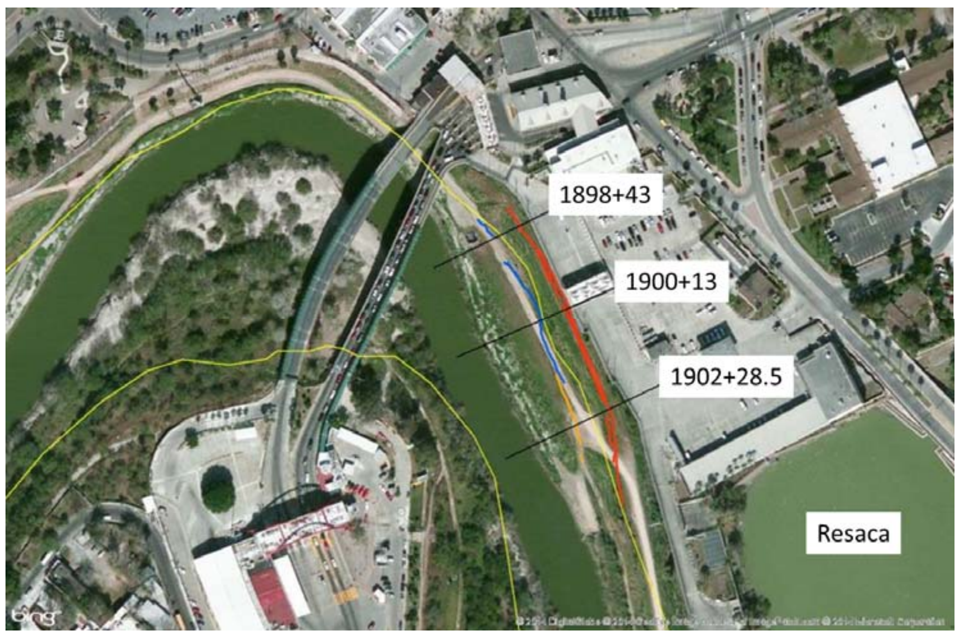
STEADY STATE SEEPAGE (WSE 14.31 FT)

STATION 1898+43

FEB-2015 **PLATE - 14**







Minimum factor of safety (FoS): 1.10

material	unit weight (pcf)	c (psf)	phi (deg)
CH Pleistocene	121.98	200.00	24.00
CL-Holocene	123.37	800.00	17.30
SM	117.00	0.00	32.00
ML	119.38	300.00	32.60
2012 Levee Fill	127.34	620.00	29.20
Levee Fill	127.34	620.00	29.20
Historic Fill	127.34	200.00	24.00
soft ML	125.98	200.00	0.00

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

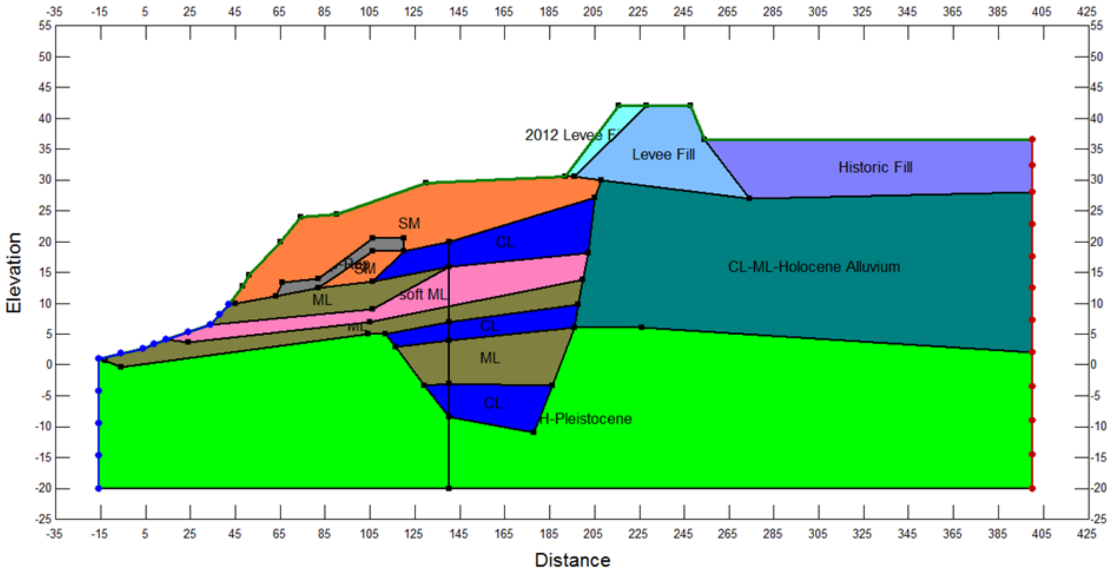
IBWC-BROWNSVILLE LEVEE

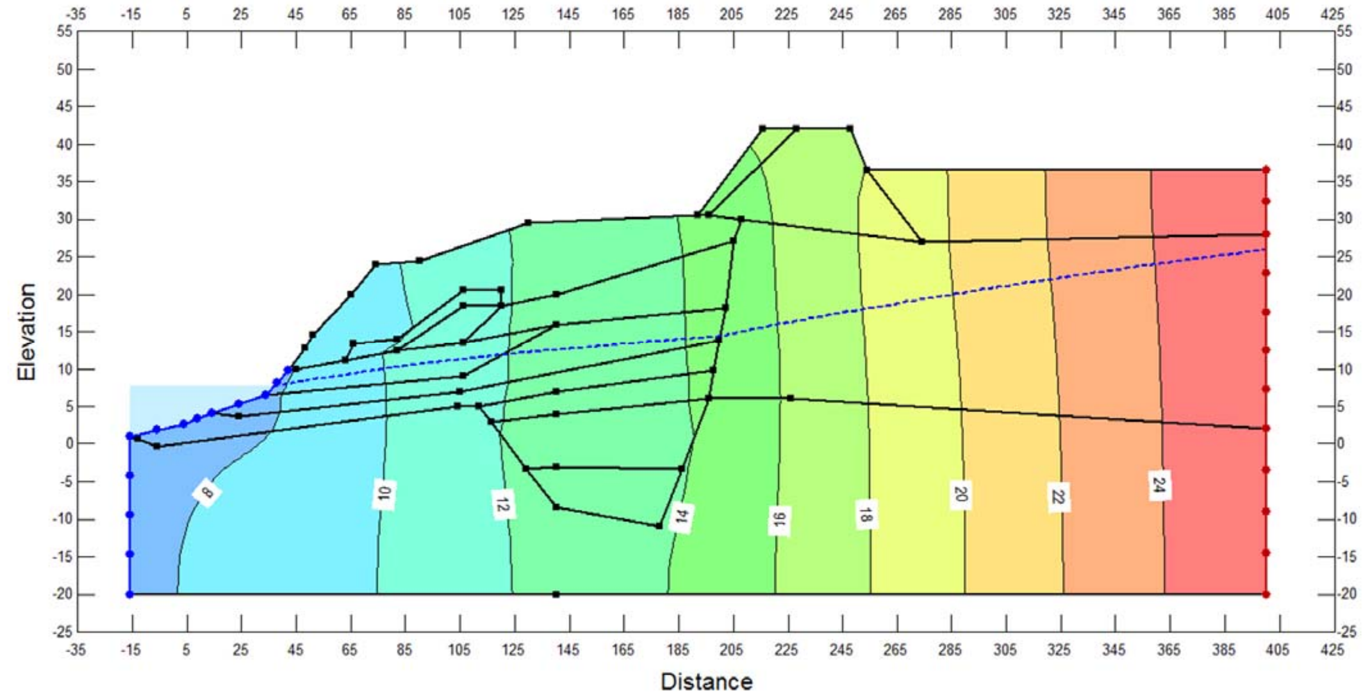
STABILITY MODEL

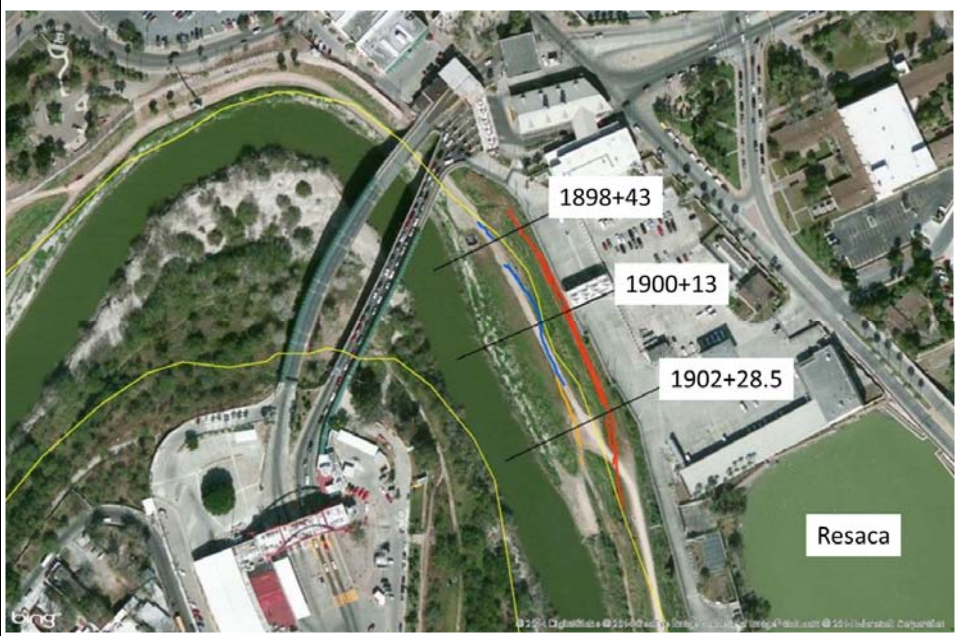
TRANSIENT FOS (HYDROGRAPH)

STATION 1898+43

FEB-2015 **PLATE - 15**







material	K_{sat} (ft/s)	n	m_v (1/psf)	ratio
CH Pleistocene	3.30E-08	0.44	3.60E-06	0.2
CL-Holocene	3.30E-08	0.43	2.50E-06	0.2
CL	1.00E-07	0.45	1.00E-06	1
SM	3.30E-07	0.3	5.00E-06	0.2
ML	1.00E-07	0.43	1.00E-05	0.2
2012 Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Historic Fill	3.30E-08	0.4	3.74E-06	0.2
soft ML	1.00E-07	0.45	1.00E-05	1

Boundary Conditions	type	magnitude (ft)
River	head	7.77
Protected side	head	25.98

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

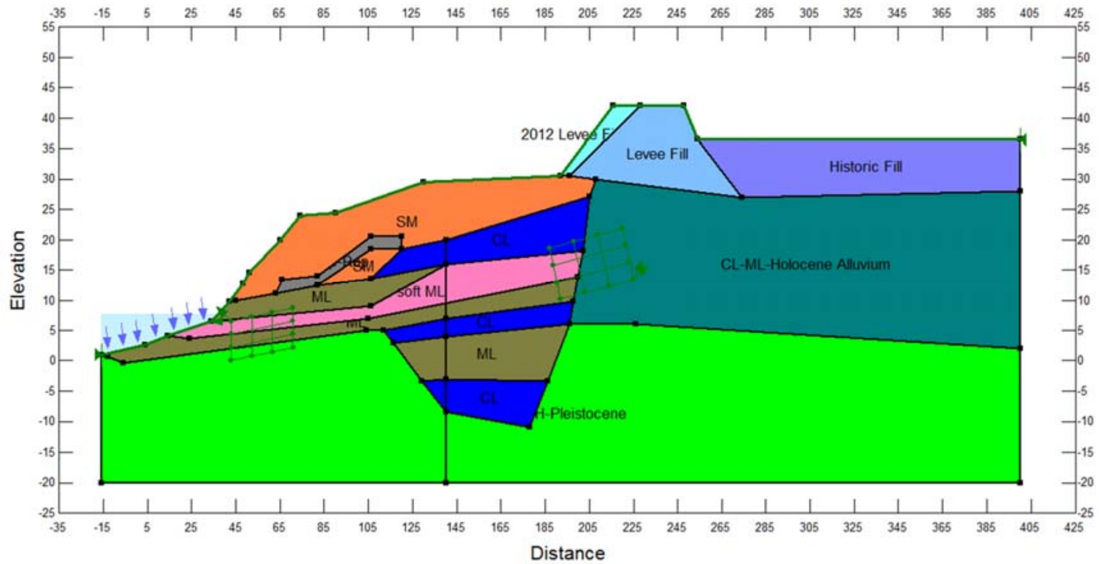
IBWC-BROWNSVILLE LEVEE

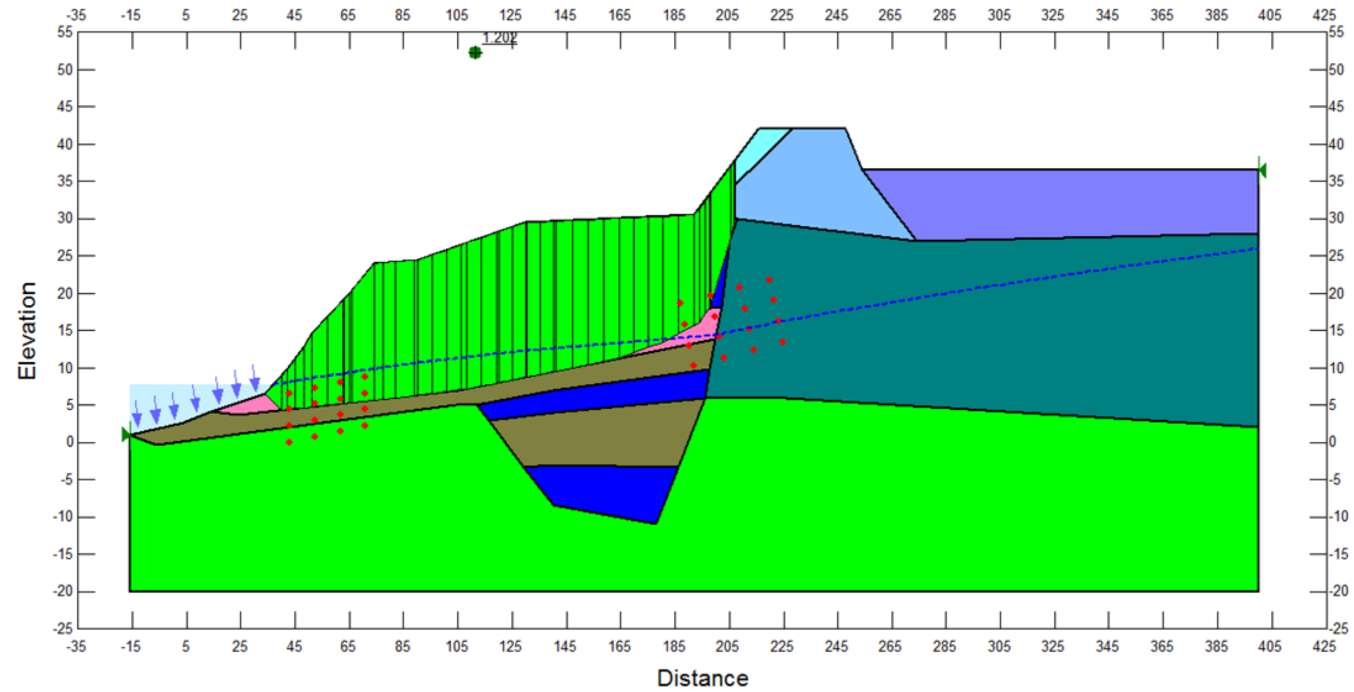
STEADY STATE SEEPAGE, SATURATED MODEL

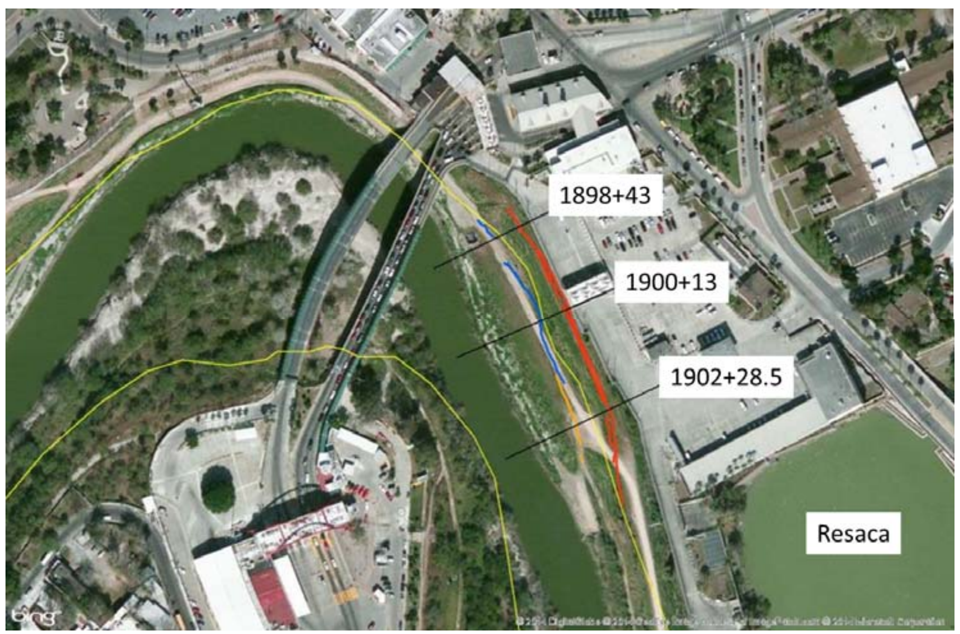
STEADY STATE SEEPAGE (WSE 7.77 FT)

STATION 1902+28.5

FEB-2015 **PLATE - 16**







material	unit weight (pcf)	c (psf)	phi (deg)
CH Pleistocene	121.98	200.00	24.00
CL-Holocene	123.37	800.00	17.30
CL	120.00	300.00	0.00
SM	117.00	0.00	32.00
ML	119.38	300.00	32.60
2012 Levee Fill	127.34	620.00	29.20
Levee Fill	127.34	620.00	29.20
Historic Fill	127.34	200.00	24.00
soft ML	125.98	260.00	0.00

Minimum factor of safety (FoS): 1.20

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

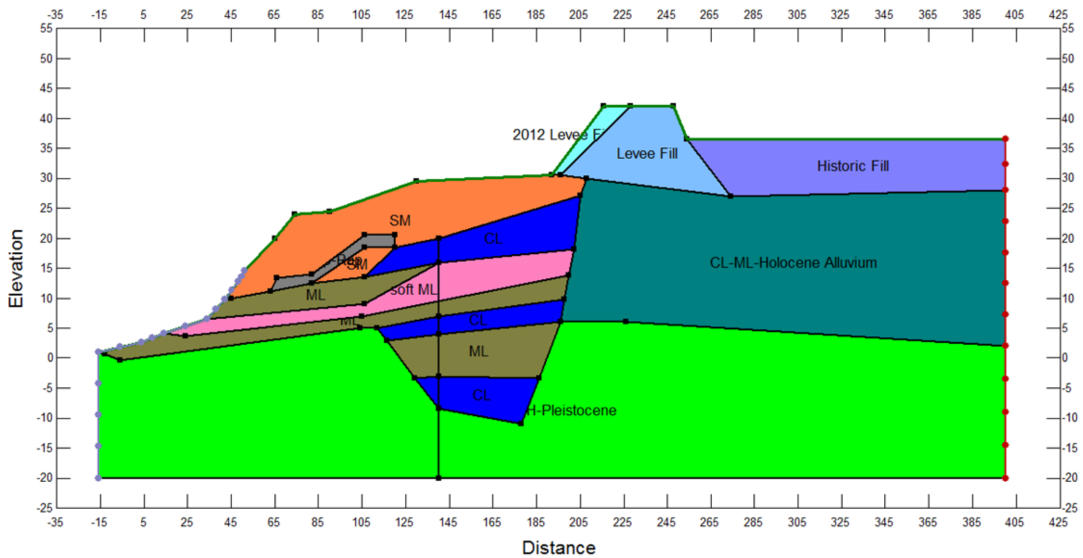
IBWC-BROWNSVILLE LEVEE

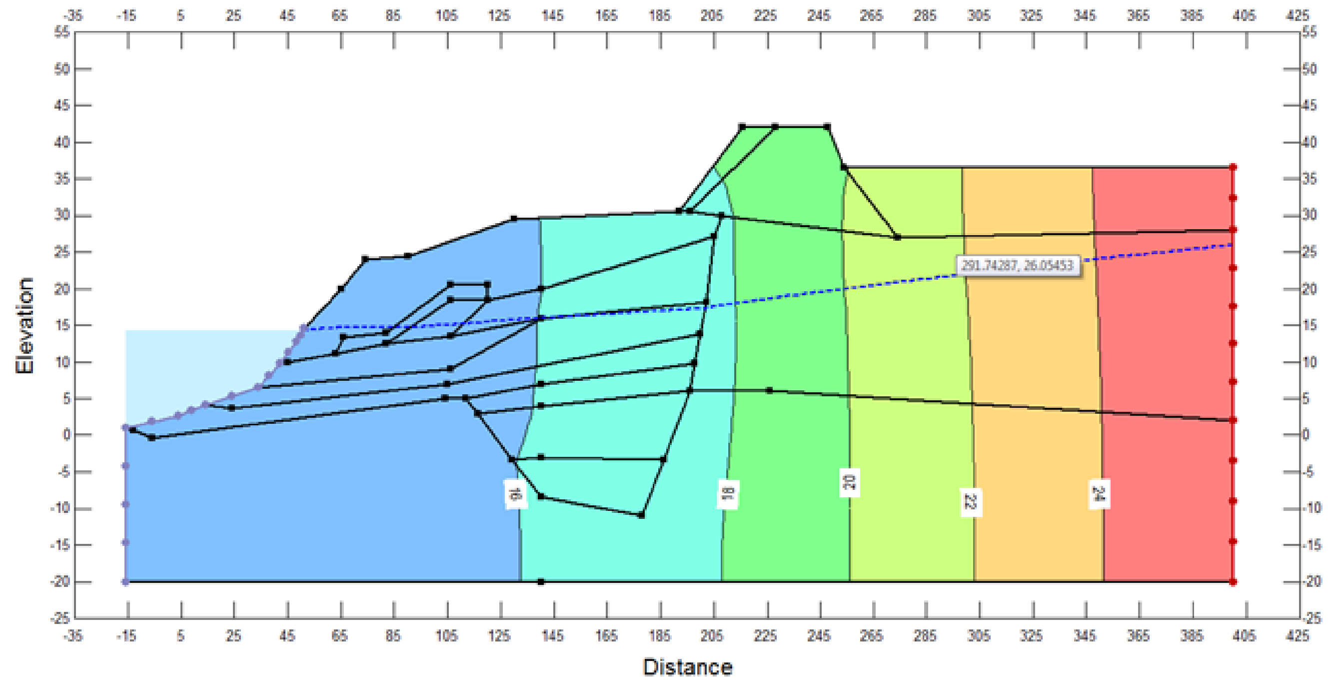
STABILITY MODEL

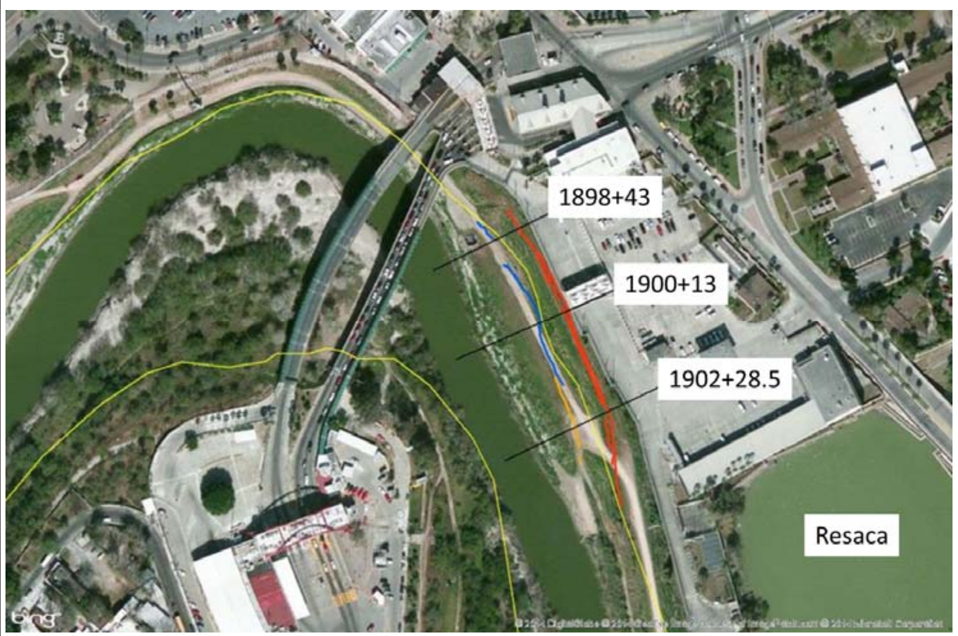
STEADY STATE FOS(WSE 7.77 FT)

STATION 1902+28.5

FEB-2015 **PLATE - 17**







material	K_{sat} (ft/s)	n	m_v (1/psf)	ratio
CH Pleistocene	3.30E-08	0.44	3.60E-06	0.2
CL-Holocene	3.30E-08	0.43	2.50E-06	0.2
CL	1.00E-07	0.45	1.00E-06	1
SM	3.30E-07	0.3	5.00E-06	0.2
ML	1.00E-07	0.43	1.00E-05	0.2
2012 Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Historic Fill	3.30E-08	0.4	3.74E-06	0.2
soft ML	1.00E-07	0.45	1.00E-05	1

Boundary Conditions	type	magnitude (ft)
River	head	14.31
Protected side	head	25.98

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

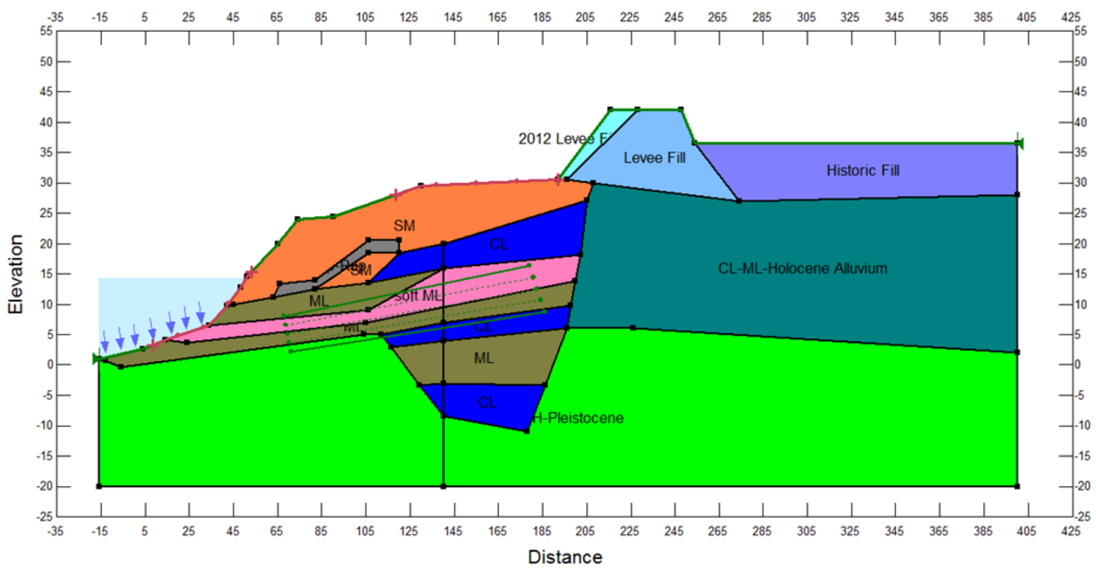
IBWC-BROWNSVILLE LEVEE

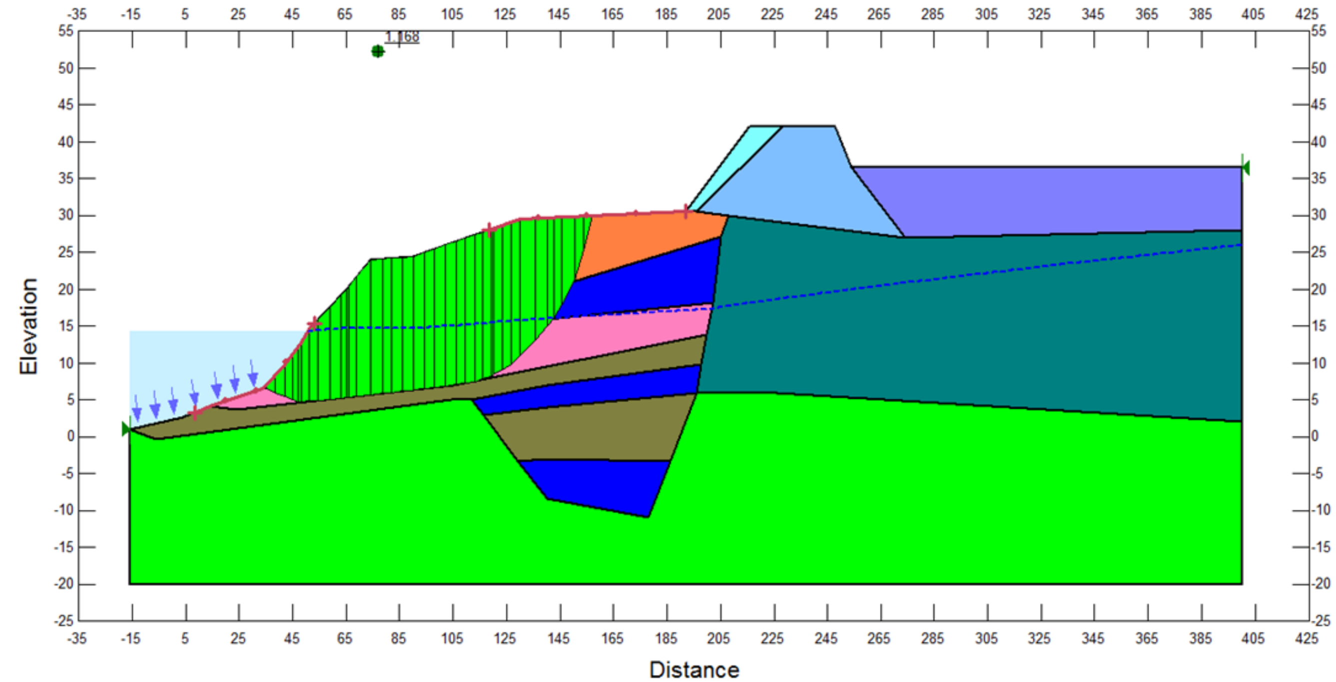
STEADY STATE SEEPAGE, SATURATED MODEL

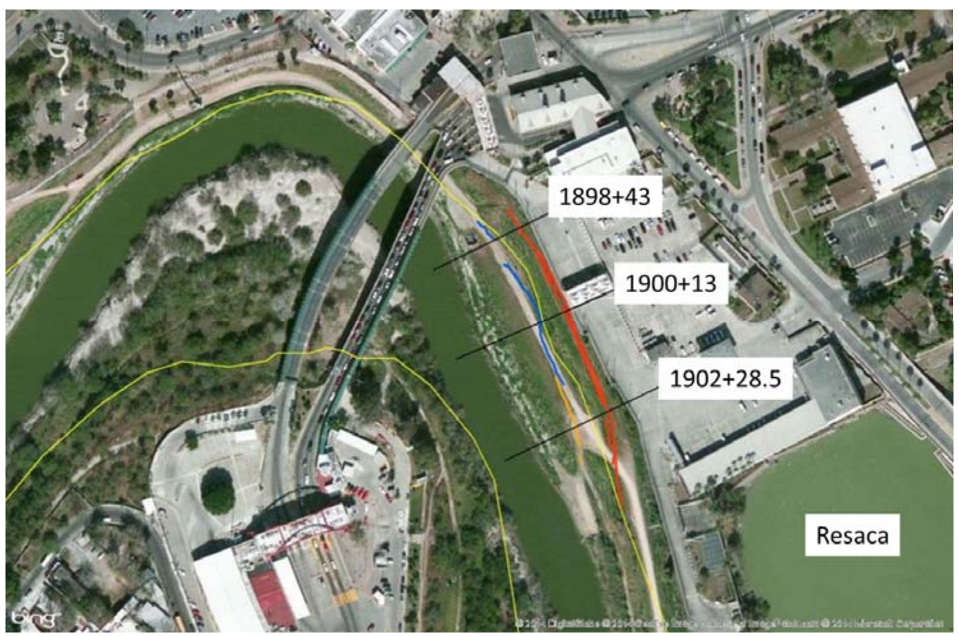
STEADY STATE SEEPAGE (WSE 14.31 FT)

STATION 1902+28.5

FEB-2015 **PLATE - 18**







Minimum factor of safety (FoS): 1.17

material	unit weight (pcf)	c (psf)	phi (deg)
CH Pleistocene	121.98	200.00	24.00
CL-Holocene	123.37	800.00	17.30
CL	120.00	300.00	0.00
SM	117.00	0.00	32.00
ML	119.38	300.00	32.60
2012 Levee Fill	127.34	620.00	29.20
Levee Fill	127.34	620.00	29.20
Historic Fill	127.34	200.00	24.00
soft ML	125.98	260.00	0.00

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

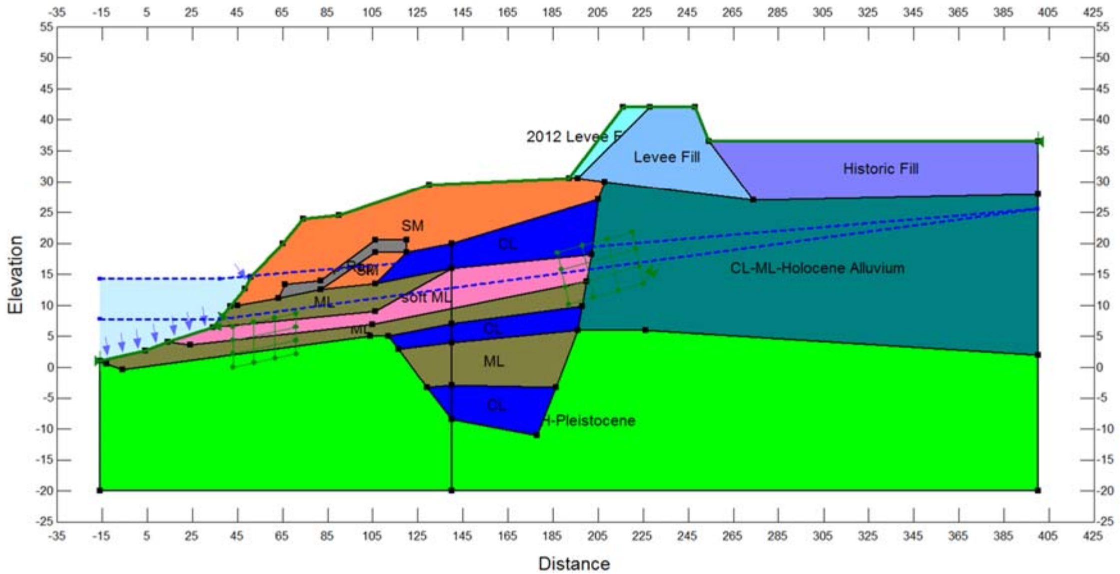
IBWC-BROWNSVILLE LEVEE

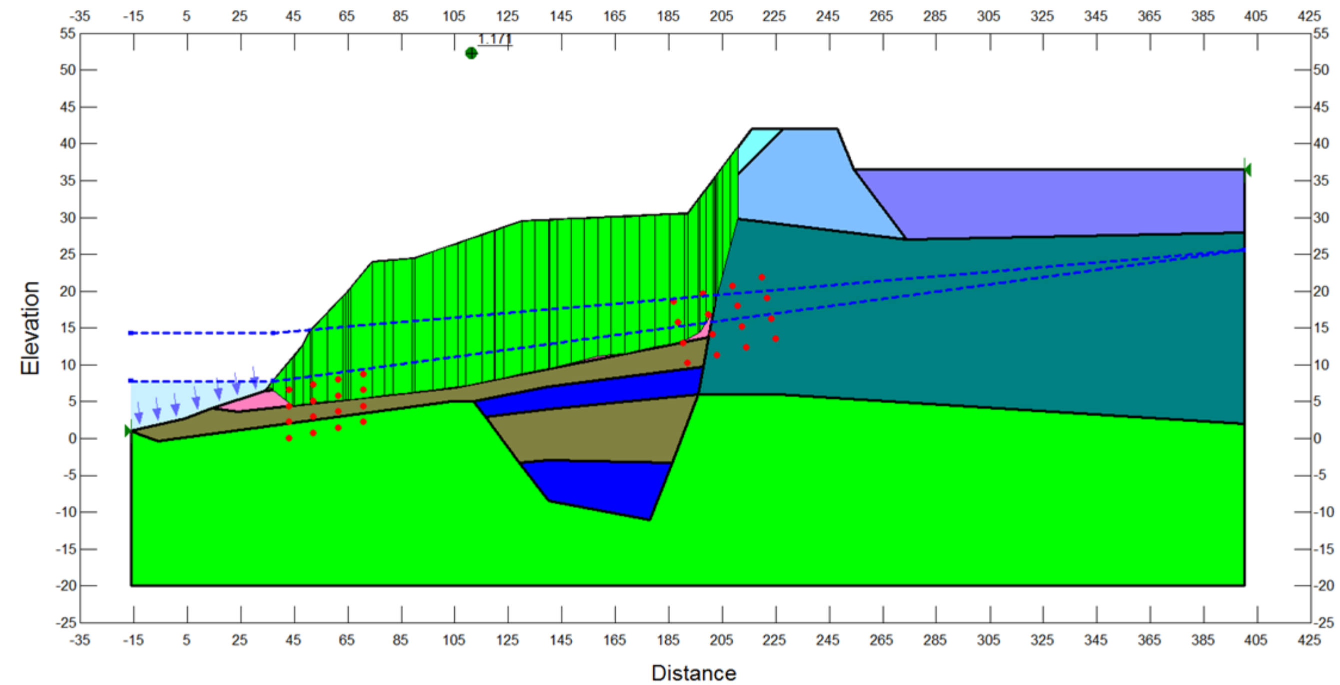
STABILITY MODEL

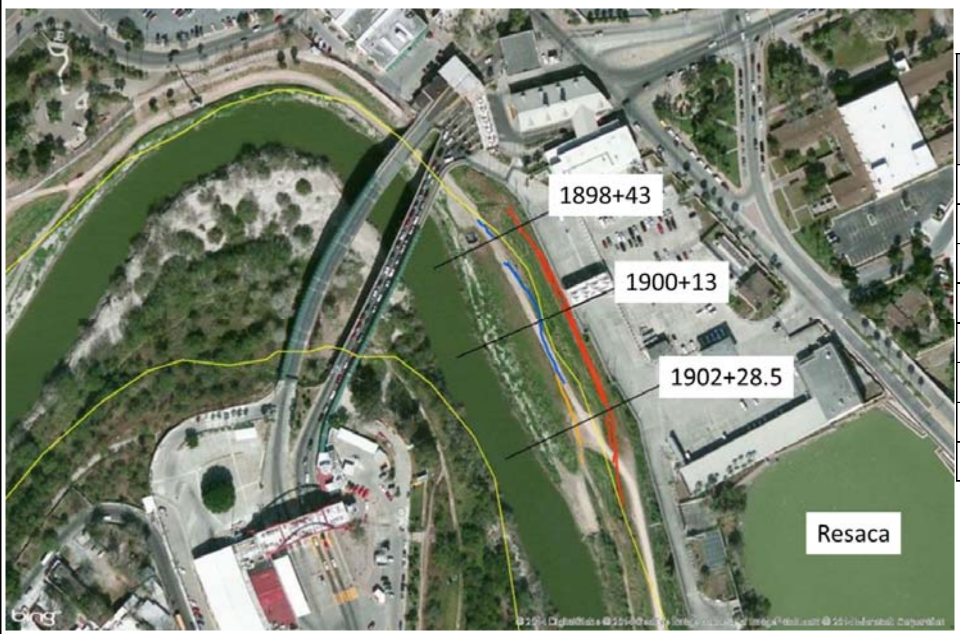
STEADY STATE FOS(WSE 14.31 FT)

STATION 1902+28.5

FEB-2015 **PLATE - 19**







material	unit weight (pcf)	c' (psf)	phi' (deg)	total stress c (psf)	total stress phi (deg)
CH Pleistocene	121.98	200.00	24.00	2320.00	0.00
CL-Holocene	123.37	800.00	17.30	400.00	0.00
SM	117.00	0.00	32.00	0.00	32.00
ML	119.38	300.00	32.60	0.00	29.00
2012 Levee Fill	127.34	620.00	29.20	5000.00	0.00
Levee Fill	127.34	620.00	29.20	5000.00	0.00
Historic Fill	127.34	200.00	24.00	400.00	15.00
soft ML	125.98	200.00	0.00	200.00	0.00

Minimum factor of safety (FoS): 1.17

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

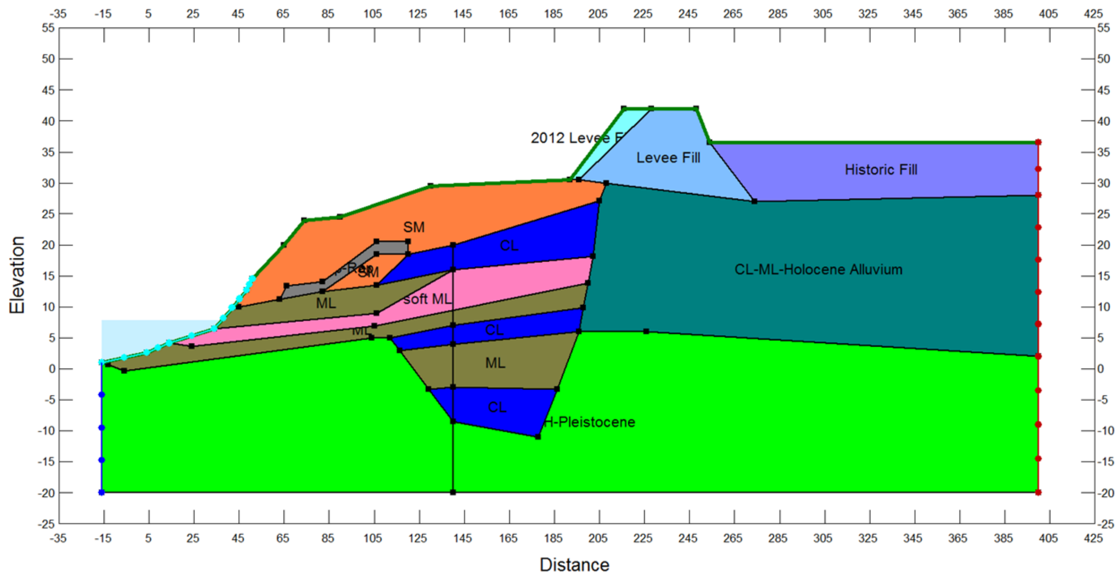
IBWC-BROWNSVILLE LEVEE

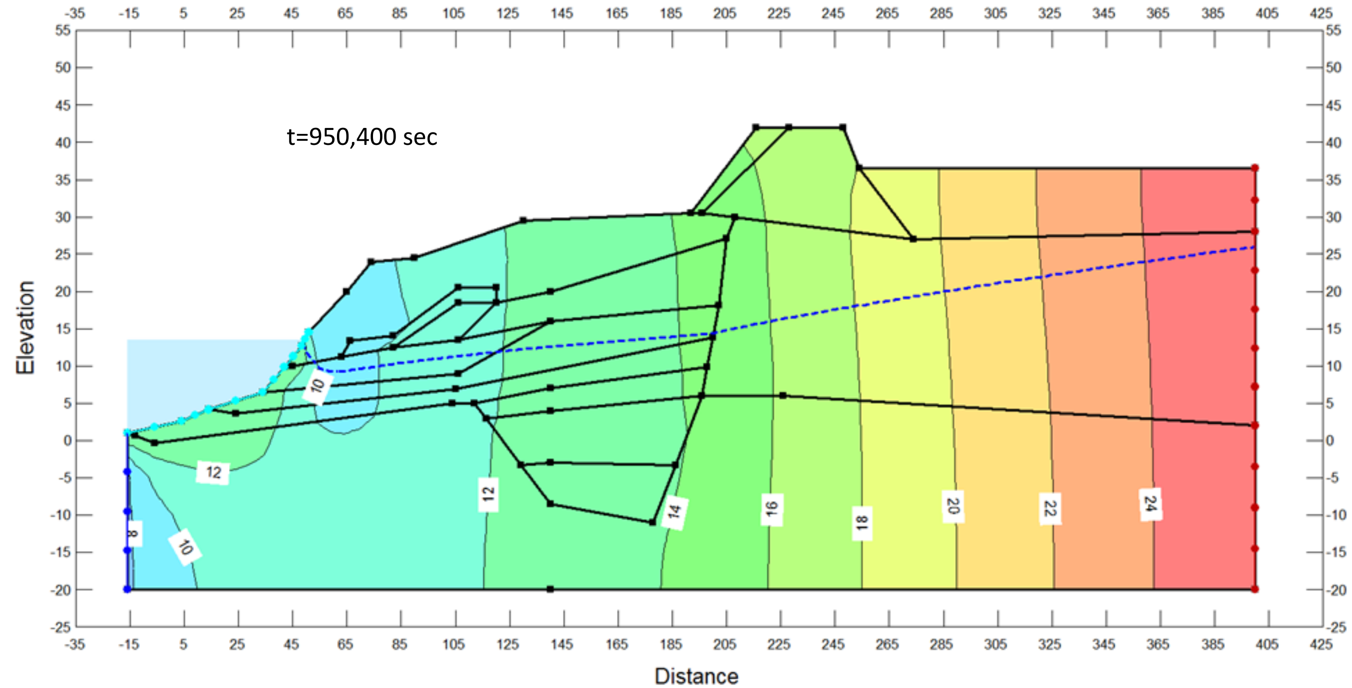
RAPID DRAWDOWN STABILITY

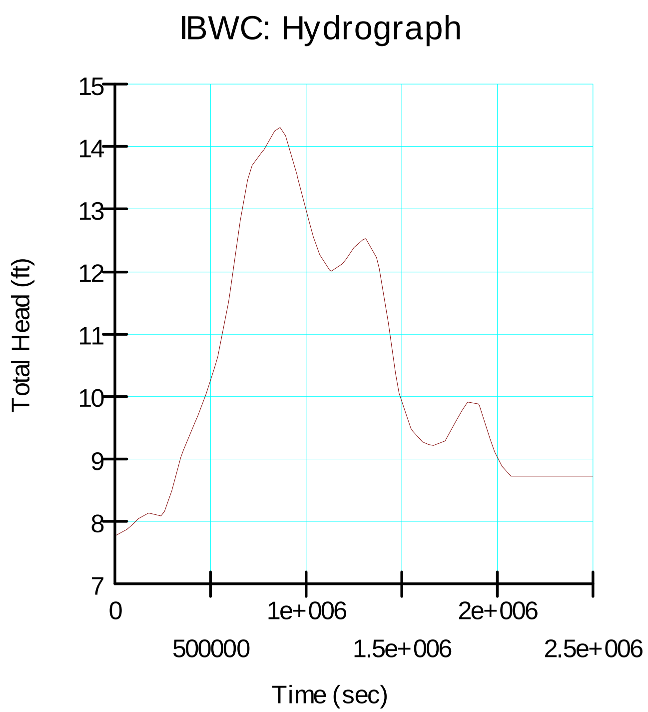
RAPID DRAWDOWN (WSE 7.77 & 14.31 FT)

STATION 1902+28.5

FEB-2015 **PLATE - 20**







material	K_{sat} (ft/s)	n	m_v (1/psf)	ratio
CH Pleistocene	3.30E-08	0.44	3.60E-06	0.2
CL-Holocene	3.30E-08	0.43	2.50E-06	0.2
CL	1.00E-07	0.45	1.00E-06	1
SM	3.30E-07	0.3	5.00E-06	0.2
ML	1.00E-07	0.43	1.00E-05	0.2
2012 Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Levee Fill	3.30E-08	0.4	3.74E-06	0.2
Historic Fill	3.30E-08	0.4	3.74E-06	0.2
soft ML	1.00E-07	0.45	1.00E-05	1

Boundary Conditions	type	magnitude (ft)
River*	head	7.77
Protected side	head	25.59

*function above channel surface, see plot lower left corner (light blue)

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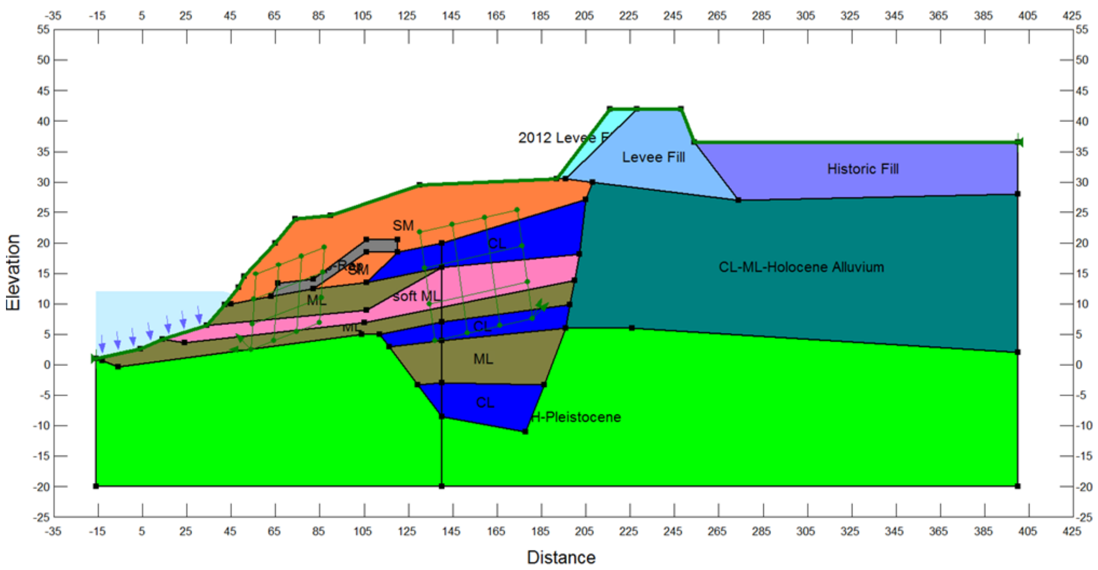
IBWC-BROWNSVILLE LEVEE

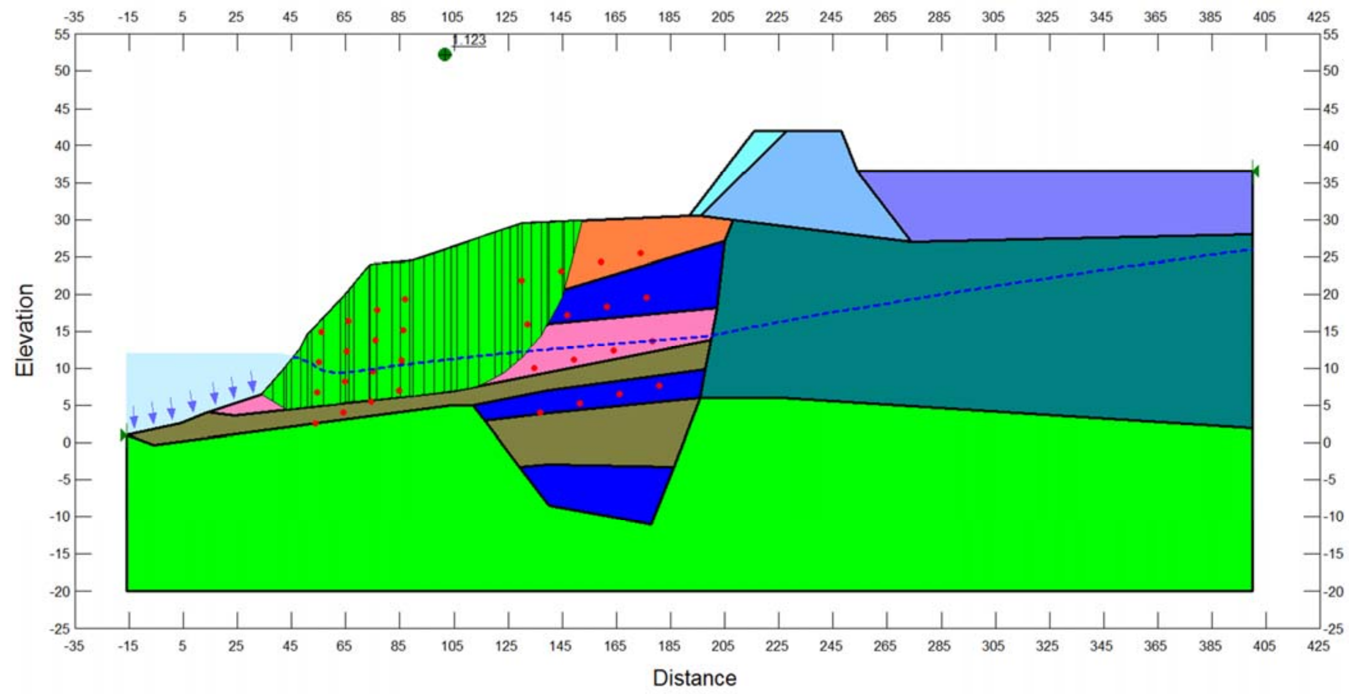
HYDROGRAPH, SATURATED MODEL

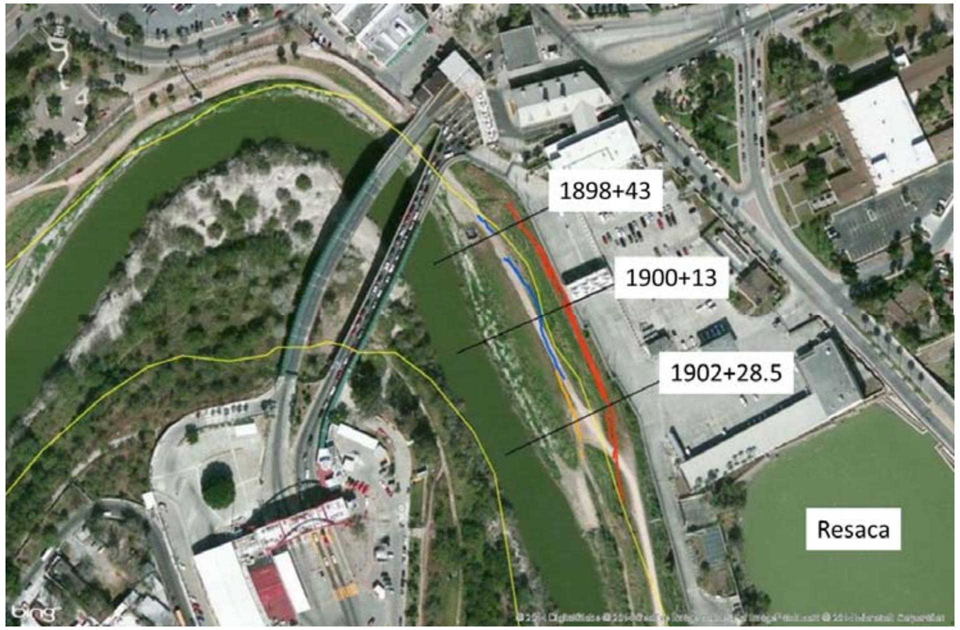
STEADY STATE SEEPAGE (WSE 14.31 FT)

STATION 1902+28.5

FEB-2015 **PLATE - 21**







material	unit weight (pcf)	c (psf)	phi (deg)
CH Pleistocene	121.98	200.00	24.00
CL-Holocene	123.37	800.00	17.30
CL	120.00	300.00	0.00
SM	117.00	0.00	32.00
ML	119.38	300.00	32.60
2012 Levee Fill	127.34	620.00	29.20
Levee Fill	127.34	620.00	29.20
Historic Fill	127.34	200.00	24.00
soft ML	125.98	260.00	0.00

Minimum factor of safety (FoS): 1.12

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IBWC-BROWNSVILLE LEVEE

STABILITY MODEL

TRANSIENT FOS (HYDROGRAPH)

STATION 1902+28.5

FEB-2015 **PLATE - 22**

Appendix M: TT 2011 Recommendations Memo

TETRA TECH

MEMORANDUM

TO: Frank Duran (IBWC)

FROM: Andy Gong, P.E. (Tetra Tech)

SUBJECT: **IBWC U.S. Levee Embankment Protection – Gateway International Bridge**

Cc: Ike Pace, P.E. (Tetra Tech)

DATE: March 30, 2011

PROJECT LOCATION

The Gateway International Bridge connects Brownsville, Texas to Matamoros, Tamaulipas, Mexico. The bridge currently includes a southbound span and a northbound span (Figure 1). The southbound (upstream) span crosses the Rio Grande at River Mile 54.475; the northbound span crosses the Rio Grande at River Mile 54.435.

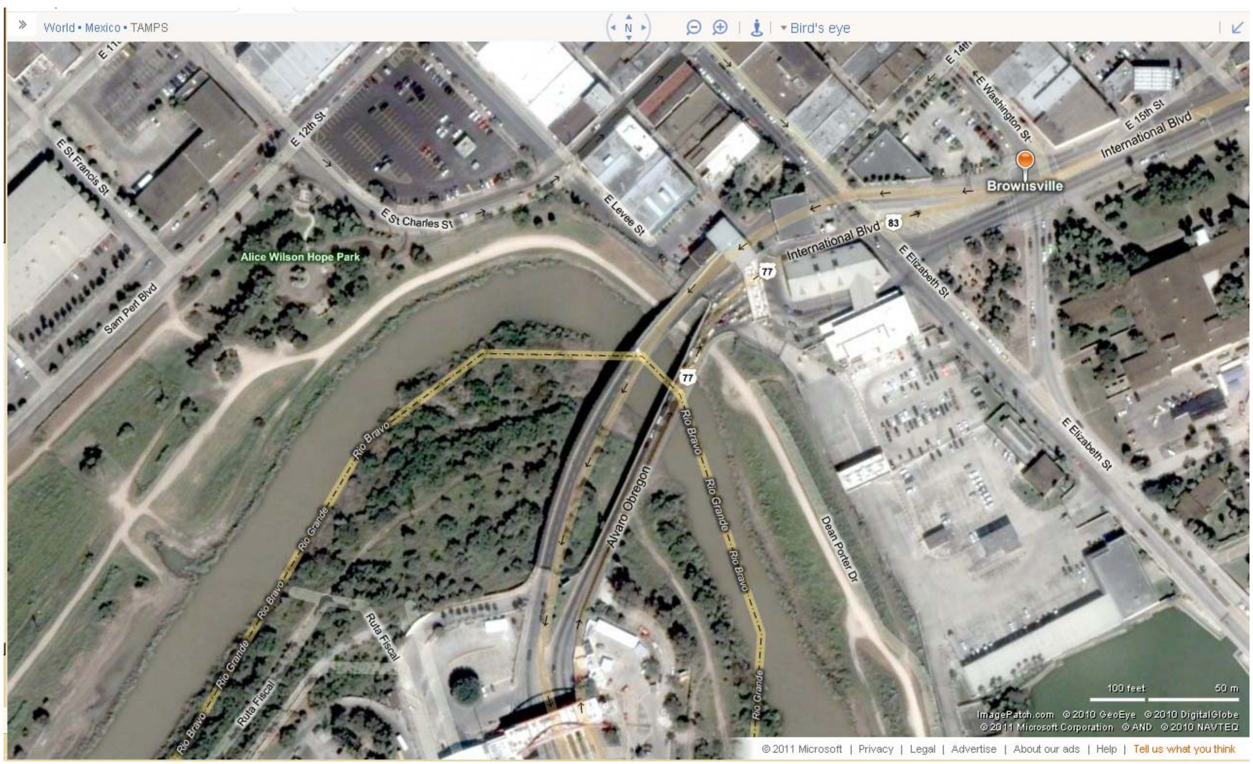

Figure 1. Gateway International Bridge Crossing of the Rio Grande (flow from left to right)

The IBWC is responsible for operation and maintenance of the U.S. levee along the left bank of the Rio Grande. Since the Rio Grande serves as the U.S. – Mexico border, the U.S. Department of Homeland Security (DHS) constructed a border security fence that is located in the access road along the crown of the levee (Figure 2). The fence obstructs access to the top of levee embankment, so access by the IBWC for flood fighting may be limited. The location of the levee embankment along the outside of the bend makes the embankment particularly subject to scour and erosion. To reduce the need for access to the levee during flood events, the IBWC is considering construction of an erosion protection along the riverward slope of the levee embankment. This technical memorandum summarizes existing hydraulic conditions and the risk of the embankment to erosion. Additionally, the results of analyses of revetment alternatives are presented.

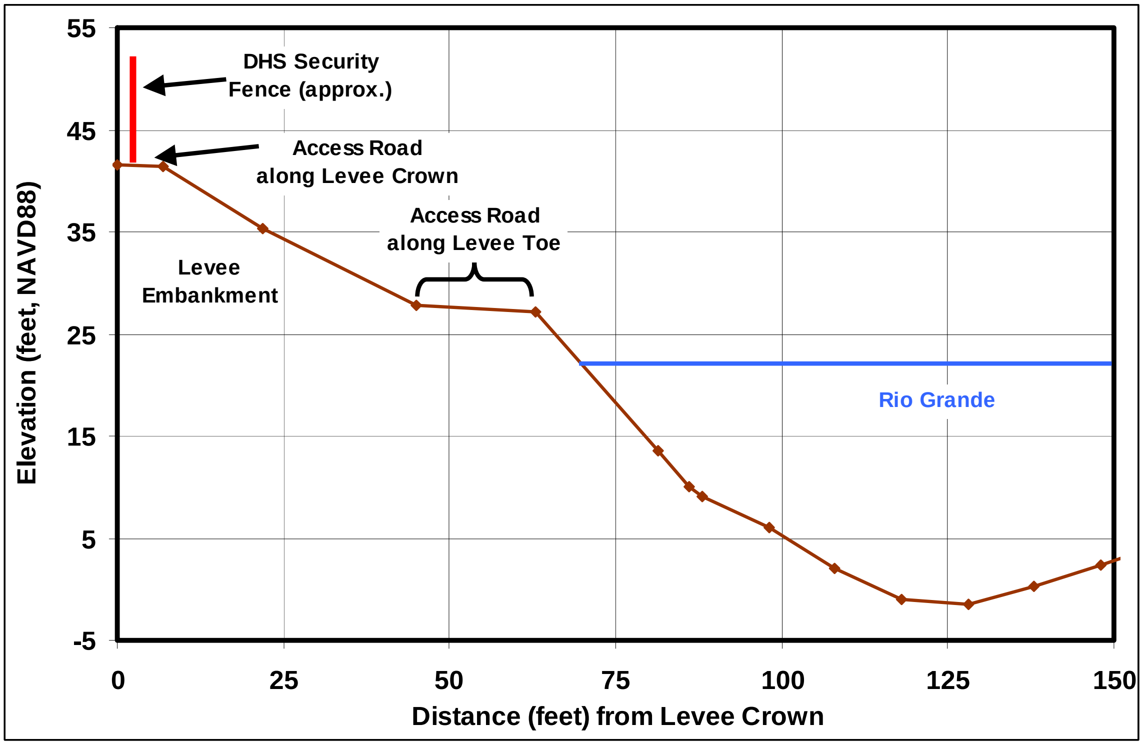

Figure 2. U.S. Levee Embankment, Access Roads, and DHS Security Fence

EXISTING CONDITIONS

The IBWC provided a hydraulic model of the Rio Grande that was used to quantify existing hydraulic conditions. The model includes 11 cross sections in the vicinity of the bridge (Table 1). The model includes a flow profile associated with the design flood, which for the reach adjacent to the Gateway International Bridge is 20,000 cfs. For this design flood, the HEC-RAS model was used to calculate the water surface elevation, channel velocity, and the top width of the water surface in the channel. These hydraulic parameters were used with an estimate of the radius of curvature of the bend to estimate the increased velocity along the outside of the bend – the area where embankment protection is under consideration. The resulting velocity was compared to erosion thresholds to identify whether there is need for embankment protection.

For the design flow of 20,000 cfs, Table 1 summarizes pertinent hydraulic parameters calculated using the HEC-RAS model. The radius of curvature of the bend was estimated using aerial photography to be between 550 and 575 feet.

Table 1. Hydraulic Parameters Calculated Using IBWC HEC-RAS Model of the Rio Grande

Section ID	Description	Minimum Channel Elevation[1] (feet)	Water Surface Elevation[1] (feet)	Hydraulic Depth of Main Channel (feet)	Channel Top Width (feet)	Depth-Averaged Channel Velocity (feet/sec.)
55.2		-0.16	36.88	22.9	241.0	3.5
54.5		-1.46	36.61	26.0	164.0	3.2
54.49		4.94	36.51	24.6	180.0	3.9
54.475	U/S side of S/B span	4.94	36.47	24.4	165.4	4.2
54.475	D/S side of S/B span	4.94	36.46	24.3	165.4	4.2
54.47		4.94	36.47	24.6	180.0	3.9
54.46		4.94	36.47	24.6	180.0	3.9
54.45		0.64	36.46	25.2	184.5	3.9
54.435	U/S side of N/B span	0.64	36.39	25.0	165.7	4.4
54.435	D/S side of S/B span	0.64	36.39	25.0	165.7	4.4
54.43		0.64	36.41	25.2	241.0	3.9

[1] Elevations are referenced to the North American Vertical Datum of 1988 (NAVD88)

The depth-averaged channel velocities in Table 1 are averaged across the entire channel section (defined by the bank stations in the HEC-RAS model). Since the concern is the velocities acting along the riverward embankment of the levee, evenly spaced “slices” were cut through the cross section of the channel and the HEC-RAS model calculated the depth averaged velocity within each slice. The minimum and maximum velocities along the left bank are presented in Table 2. The maximum velocities are taken from the toe of the left bank (i.e., the greatest depth); the minimums are taken from the top of the bank as defined by the bank station in the HEC-RAS model.

While the maximum and minimum velocities shown in Table 2 illustrate the variability associated with flow depth; this variability does not account for the greater flow velocity along the outside of a bend compared to the center of the channel. The U.S. Army Corps of Engineers (USACE) Engineer Manual EM 1110-2-1601 *Hydraulic Design of Flood Channels* (1994) provides the following equation to calculate flow velocity along the outside of a bend to facilitate the design of riprap:

$$\frac{V_{ss}}{V_{AVG}} = 1.74 - 0.52 * LOG(R_c / W) \qquad \textbf{(Equation 1)}$$

Where:

V_{SS} = characteristic velocity for side-slopes, depth-averaged velocity at 20% of the slope length up from the toe
V_{AVG} = main channel average velocity at the upstream end of the bend
R_c = centerline radius of the bend
W = main channel water surface width

Table 2. Maximum and Minimum Velocities Calculated Using the HEC-RAS Model along the Left Bank of the Rio Grande

Section ID	Description	Depth-Averaged Channel Velocity (feet/sec.)	Maximum Velocity (feet/sec.)	Minimum Velocity (feet/sec.)
55.2		3.5	4.5	1.3
54.5		3.2	4.4	1.2
54.49		3.9	5.3	1.9
54.475	U/S side of S/B span	4.2	6.2	2.2
54.475	D/S side of S/B span	4.2	6.2	2.2
54.47		3.9	5.3	1.9
54.46		3.9	5.3	1.9
54.45		3.9	5.2	1.7
54.435	U/S side of N/B span	4.4	6.4	2.1
54.435	D/S side of S/B span	4.4	6.4	2.1
54.43		3.9	5.3	1.7

Applying Equation 1 with main channel average velocity at the upstream end of the bend (Section ID 54.5), a radius of curvature between 550 and 575 feet, a main channel average velocity at the upstream end of the bend of 3.2 feet per second, and a main channel water surface width of 165 to 180 feet, the characteristic velocity for side-slopes is between 4.7 and 4.8 feet per second.

The resulting characteristic velocity for side-slopes as well as the maximum velocities computed using the HEC-RAS model show that the riverward slope of the embankment is close to the maximum permissible velocity to prevent erosion of 5 feet per second for various grass covers (USACE 1994; USDA 1954). Additionally, the duration of major flood flows in the Rio Grande can be several weeks, providing sufficient time to fully saturate surface soils and decrease resistance to erosive forces. Therefore, under the existing conditions in which access during a flood is limited, the addition of erosion protection to the riverward slope of the levee embankment is prudent.

As shown in Figure 1, it is noteworthy that there is a zone of vegetation that has established along the left edge of water. This vegetation does not extend up the bank, and characteristics of the vegetation that would affect flow velocity (i.e., height, flexibility, density, root structure) are unknown. While this vegetation may inhibit erosion, given the risk of erosion and the limited access, an erosion protection revetment would be more reliable than assuming the vegetation would prevent erosion.

Given the channel alignment near the Gateway International Bridge (i.e. a bend in the channel with small radius of curvature), scour along the bank is a concern and a likely cause of failure along the bank. The maximum potential bend scour was calculated using data developed by Thorne and Abt (1992). The safe design curve through the data (Equation 2) is intended to be conservative – it represents an upper limit for scour. It is important to note that this equation addresses local scour; if general bed degradation is expected, it would need to be quantified and added to the local scour. No general bed degradation beyond the bend scour is expected in the vicinity of the Gateway International Bridge.

$$\frac{d_{SC}}{d_{BAR}} = 1.07 - 0.44 * \log[(R_c / W_{BAR}) - 2] \qquad \textbf{(Equation 2)}$$

Where:

d_{SC} = maximum depth of scour in the bend
d_{BAR} = mean water depth at upstream crossing
R_c = centerline radius of the bend
W_{BAR} = main channel water surface width at upstream crossing

Applying Equation 2 with the hydraulic characteristics of the upstream crossing (i.e., Section ID 54.5) and a radius of curvature of 550 to 575 feet, the maximum scour depth in the bend is 26 to 27 feet. The R_c / W_{BAR} ratios of 3.3 and 3.4 are between 2 and 22, so the use of this equation is appropriate.

Maynord (1996) developed an alternate equation to estimate potential bend scour:

$$\frac{d_{MAX}}{d_{BAR}} = 1.8 - 0.051\left(\frac{R_c}{W_{BAR}}\right) + 0.0084\left(\frac{W_{BAR}}{d_{BAR}}\right) \qquad \textbf{(Equation 3)}$$

All variables are as defined for Equation 1 and Equation 2. Application of Equation 3 yields maximum water depths of 43 to 44 feet. Existing flow depths in the bend during the design flood are between 31 and 36 feet, indicating that the toe depth of a riprap revetment should be 7 to 13 feet. Using a factor of safety of 1.19 as recommended by Maynore (1996) to more closely resemble the safe design curve, the maximum bend scour depths are 16 to 21 feet. These results indicate the conservatism of the Thorne and Abt (1992) safe design curve.

Based on engineering judgment and the results of both equations, the ultimate bend scour assumed for this location is 21 feet. An analysis of the thalweg profile between approximately RM 52 to RM 67 indicates that at least 5 feet of bend scour exists at the bend at the Gateway International Bridge. Thus, future potential for bend scour is estimated to be 16 feet.

ALTERNATIVE ANALYSIS

An analysis was performed to determine alternatives that would mitigate erosion as a result of the flow velocity as well as to provide a depth of protection based on the expected scour depth. Loose rock revetment was assumed as the erosion protection for several of the alternatives. Future design phases should consider other options for sloped revetment such as concrete slope paving, armorflex, and soil cement.

Using the flow velocities in Table 2 and the USACE sizing methodology (USACE 1994), the recommended rock gradation includes a D_{100} of 9.0 inches, a D_{50} of 6.0 inches, and thickness of 9 inches. For constructability, a thickeness of 12 inches is recommended. These rock dimensions apply to all 4 alternatives presented below. For each alternative the extent of the revetment should extend from downstream of the Gateway Independence Bridge upstream to the point where the security fence no longer impacts maintenance and operation of the levee. The top of the revetment should extend to the top of levee.

ALTERNATIVE 1 – RIPRAP REVETMENT OF UPPER BANK ONLY

One alternative means of embankment protection is the construction of a riprap revetment along the upper bank (i.e., between the access road along the toe and the access road along the crown). This is illustrated in Figure 3.

This alternative would provide embankment protection along the upper bank and reduce the potential for vegetation growth along the bank. This addresses the short-term condition but does not address the long-term condition in which the existing bank below the lower access road could begin to scour. Toe scour is probably the most frequent cause of failure of riprap revetments (USACE 1994). As the lower bank is eroded, the progressive erosion of the embankment will undermine the lower access road along the levee toe and the upper bank riprap revetment. This upper bank revetment would then fail and not provide any protection to the embankment.

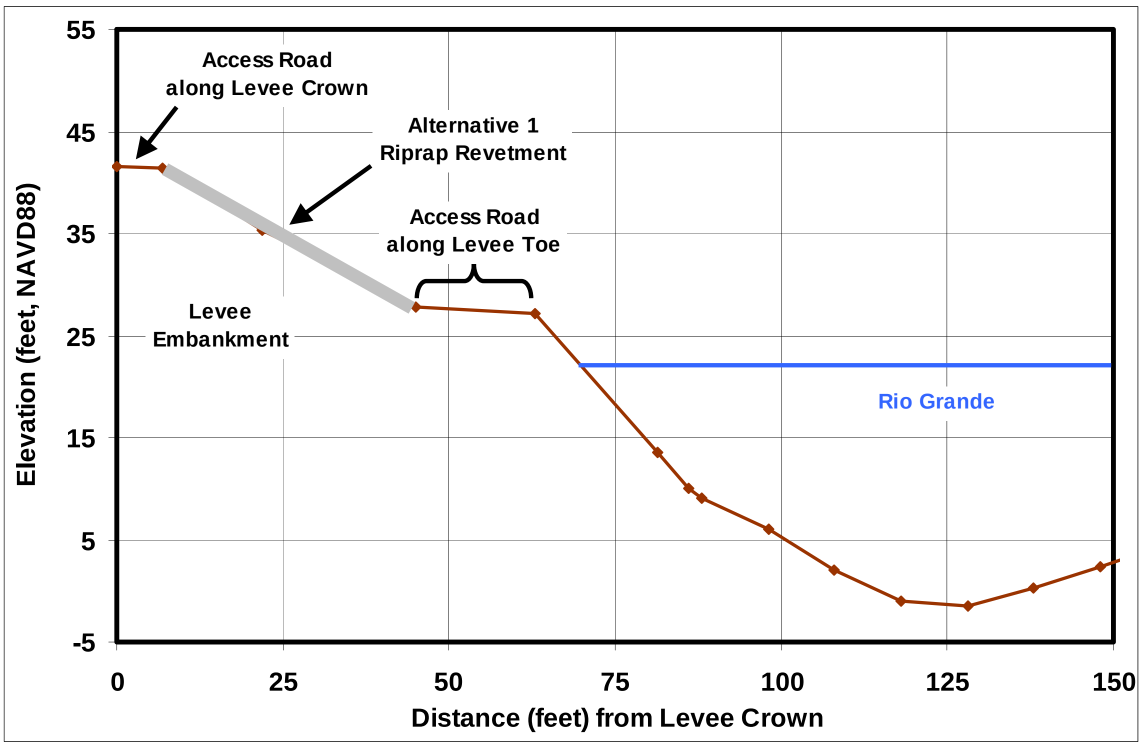

Figure 3. Alternative 1 Embankment Protection – Riprap Revetment on Upper Bank Only

ALTERNATIVE 2 – RIPRAP REVETMENT OF ENTIRE BANK

A second alternative is to construct a riprap revetment along the entire height of the bank from the upper access road down to a depth that will not be impacted by potential maximum scour. This alternative is illustrated in Figure 4. The advantage of this method is that it will fully cover the maximum potential scour depth with a uniform thickness of riprap revetment. The disadvantage of this alternative is that construction would require dewatering and substantial excavation, which will increase the cost of construction and potentially require environmental mitigation.

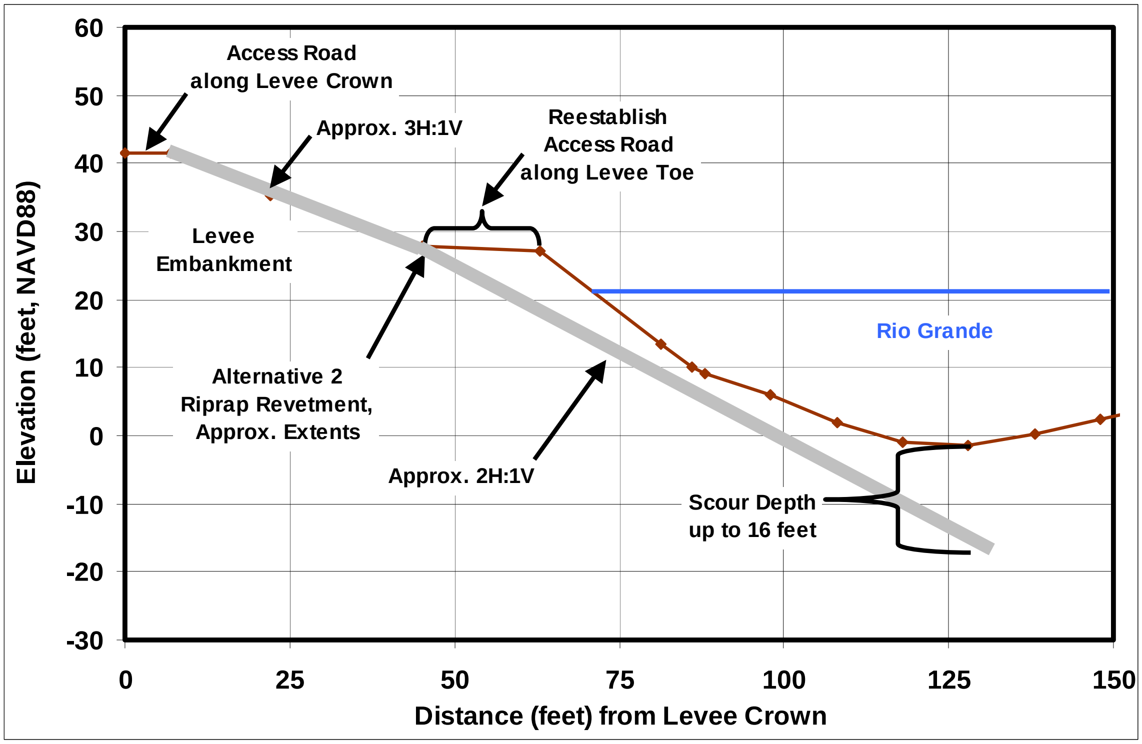

Figure 4. Embankment Protection – Riprap Revetment of Entire Bank (16 feet)

ALTERNATIVE 3 – LAUNCHABLE ROCK

A third alternative is for toe protection to be provided using launchable stone. As scour occurs underneath placed launchable stone, the stone is undermined and rolls/slides down the slope, stopping further scour at the toe of the bank. A trench is excavated, filled with stone, and buried such that toe scour is used as a substitute for mechanical excavation and placement. It is important to note that this alternative provides toe protection only, not the more robust full bank protection recommended in Alternative 2, as well as protection for the upper bank as described in Alternative 1.

Design guidance for trench-fill revetments is available in the *Hydraulic Design of Flood Control Channels* (USACE 1994). Providing an adequate volume of stone is critical because some material is lost downstream in the launching process – the greater the expected scour depth, the greater the percentage of stone lost. The height of the stone section in the trench-fill controls the rate at which rock is released during the launching process. In cases where impinging flow is expected to induce rapid scouring, the height of the stone section should be 2.5 to 3.0 times the desired thickness of the revetment. Widely graded riprap is recommended to reduce rock void and prevent leaching of bank material.

The required volume of stone was calculated using the USACE (1994) methodology as presented in Equation 4:

$$Vol = F_s * T * L_L \qquad \textbf{(Equation 4)}$$

Where:

Vol = until volume of stone required cover an area one foot in width and spanning the launch length to the desired thickness

F_s = safety factor (for vertical launch distances greater than 15 feet, safety factor is 1.5 for dry placement and 1.75 for placement underwater)

T = thickness of stone layer after launching

L_L = launch length, distance over which launched stone is to cover (for the recommended slope of 2H:1V, this distance equals √5 times the scour depth)

The available space to construct the trench is limited due to the depth of the channel; therefore, this alternative can provide only sufficient revetment for the toe (i.e. the expected bend scour depth of 16 feet). Applying Equation 4 with a F_S of 1.5, a T of 1 foot, and L_L of 36 feet (16 feet * √5), the required volume of stone is 54 cubic feet per foot of revetment. Using the recommended 2.5 to 3.0 times the desired thickness of stone layer, the height of the trench-fill should be approximately 2.5 to 3.0 feet. To achieve this required volume of stone, the distance the trench-fill needs to penetrate into the bank is approximately 18 to 22 feet (Figure 5).

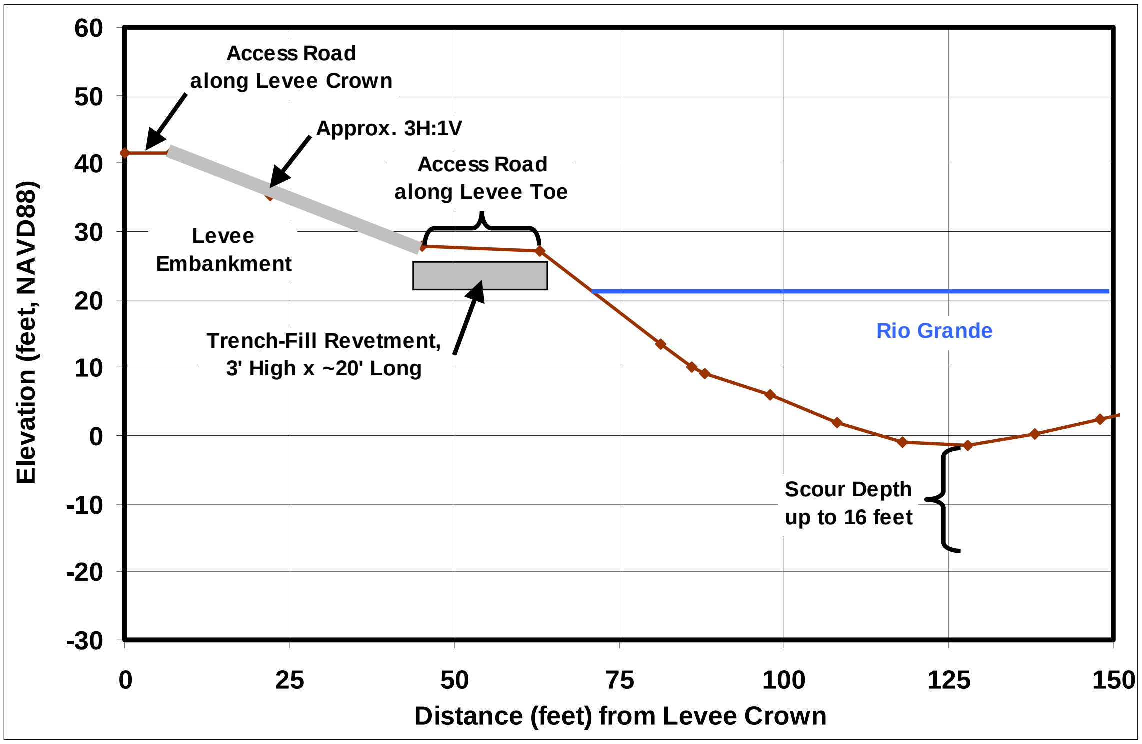

Figure 5. Embankment Protection – Trench-Fill Placement of Launching Stone

ALTERNATIVE 4 – SHEETPILE

The fourth alternative uses sheetpile rather than rock to provide protection for future scour. Riprap revetment is provided along the existing channel slope, under the lower access road, and along the upper bank. As shown in Figure 6, the sheetpile would be left at an additional height during construction to facilitate the placement of rock along the existing channel bank (the additional height will be cut to ground elevation at the end of construction). The depth of sheetpile required to protect against bend scour is 16 feet so approximately an additional 32 feet of embedment is required below the scour depth for stability

(Figure 6). Future design phases would need to determine if sheetpile or king piles are required for stability.

To protect the bank between the top of the sheetpile and the existing lower access road along the levee toe, a new riprap revetment would be constructed. This revetment cannot reduce existing conveyance and can be no steeper than 2H:1V so a new 16-foot wide access road would need to be overbuilt on the levee toe and protected in place.

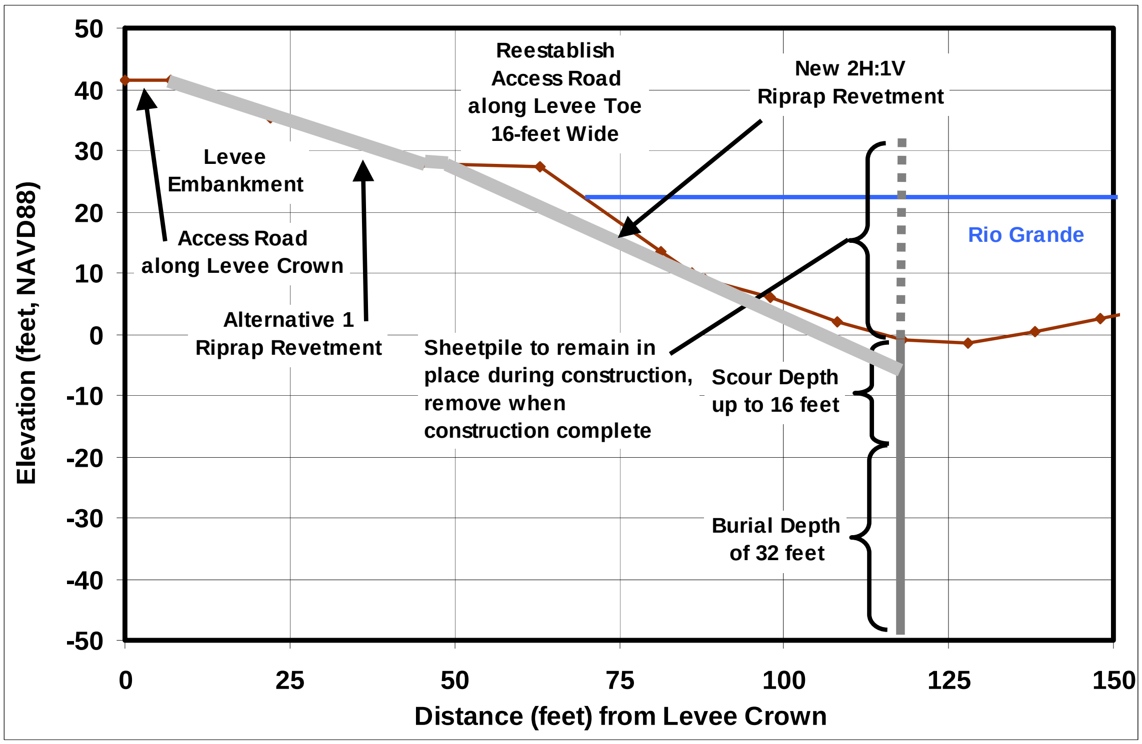

Figure 6. Embankment Protection – Riprap Upper Bank with Sheetpile for Future Scour

DESIGN RECOMMENDATIONS

Each of these alternatives would require an environmental assessment and further investigation of construction feasibility to determine the design constraints. The alternatives presented have the following major advantages and disadvantages that should be considered as part of the design selection.

Alternative 1: Riprap Revetment Upper Bank Only

+ Least environmental impacts
+ Addresses short-term maintenance concerns on the upper bank
+ No dewatering operations needed
- Does not provide protection due to scour

Alternative 2: Riprap Revetment of Entire Bank

+ Provides protection for future scour and addresses maintenance concerns along the entire bank
- Most environmental impacts
- Diversion of river and dewatering must be considered

Alternative 3: Launchable Rock Protection

+ Addresses short-term maintenance concerns on the upper bank
+ Likely no dewatering operations needed
+ Provides for scour protection at the toe
- Full bank protection is not provided

Alternative 4: Sheet Pile Protection

+ Provides protection for future scour and addresses maintenance concerns along the entire bank
- Dewatering must be considered (likely no river diversion required)
+ Less environmental impacts as compared to Alternative 2

REFERENCES

Maynord, S.T. 1996. *Toe-Scour Estimation in Stabilized Bendways.* Journal of Hydraulic Engineering. Vol. 122(8). p. 460 – 464.

Thorne, C.R., and S.R. Abt. 1992. *Analytical and Empirical Prediction of Scour Pool Depth and Location in Meander Bends.* Prepared for U.S. Army Engineer Waterways Experiment Station. Vicksburg, Mississippi. 66 p.

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