International Boundary and Water Commission
United States Section

Remediation Design of Levee and Floodplain Failure Within the Upper Brownsville Levee Reach Lower Rio Grande Flood Control Project

Contract No. IBM15D0001
Task Order No. IBM15T0015

Design Documentation Report
Final Submittal

Prepared for:
United States International Boundary and Water Commission (USIBWC)
United States Section
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Design & Consultancy for natural and built assets

STATEMENT OF LIMITATIONS

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ATTACHMENTS

ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ACBM	Articulated Concrete Block Mattress
Arcadis	Arcadis U.S., Inc.
ASTM	American Society for Testing and Materials
BMP	Best Management Practice
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CMP	Corrugated Metal Pipe
DDR	Design Documentation Report
DSM	Deep Soil Mixing
EA	Environmental Assessment
FOS	Factor of Safety
FEMA	Federal Emergency Management Agency
fps	Feet per Second
HEC-RAS	Hydrologic Engineering Center River Analysis System
HDPE	High Density Polyethylene
LRGFCP	Lower Rio Grande Flood Control Project
NAVD 88	North American Vertical Datum of 1988
PSI	Professional Services Industries, Inc.
PVC	Polyvinyl Chloride
Raba-Kistner	Raba-Kistner Consultants, Inc.
RCP	Reinforced Concrete Pipe
RFI	Request for Information
Team	Arcadis U.S., Inc., and Munoz and Dannenbaum JV

Tetra Tech	Tetra Tech, Inc.
Trinity	Trinity Testing Laboratories, Inc
USACE	U. S. Army Corps of Engineers
USIBWC	United States International Boundary and Water Commission

EXECUTIVE SUMMARY

Arcadis U.S., Inc., and Munoz and Dannenbaum JV (the Team) received a Task Order for Services from United States International Boundary and Water Commission to provide design services for remediation of levee and floodplain failure within the Upper Brownsville Levee Reach of the Lower Rio Grande Flood Control Project.

The project is located in the Lower Rio Grande Flood Control Project in Brownsville, Texas. The specific project area extends from Station 1892+00 to Station 1904+85.

The proposed remediation consists of the following.

- Remediating the floodplain area and stabilizing the river bank soil slope using soil mix columns, stone columns, an Articulated Concrete Block Mattress (ACBM) revetment erosion protection system, and rock riprap.
- Restoring full functionality of Gatewell Structure 205.
- Verifying the levee at this location meets freeboard criteria established by the Federal Emergency Management Agency in CFR Title 44, Section 65.10.

This design documentation report documents decisions made in the design process and development of contract documents for construction of the remediation of the levee and floodplain failure.

1.0 INTRODUCTION

1.1 AUTHORIZATION

Arcadis U.S., Inc. (Arcadis) and Munoz and Dannenbaum JV (the Team) received a Task Order for Services from the United States International Boundary and Water Commission (USIBWC). The Team agreed to provide design services for the remediation of the Upper Brownsville Reach 4 Levee and adjacent floodplain failure. The USIBWC issued Task Order No. IBM15T0015 under Contract No. IBM15D0001 dated September 28, 2015. Three modifications have been issued from the USIBWC since the original Task Order. The sequence of the original scope of work and associated modifications are described below.

- The original Task Order was issued on September 28, 2015. Arcadis was tasked to evaluate the United States Army Corps of Engineers (USACE) Investigation to assess the Alternative I remediation, which consists of excavating and re-grading the channel banks and levee embankment at a slope of five horizontal to one vertical (5H:1V). Additionally, Arcadis was tasked with taking readings of the three inclinometers installed by the USACE. Arcadis submitted a "Geotechnical Assessment Memorandum, 60 Percent Submittal," dated February 19, 2016, to USIBWC as part of the 60 percent design submittal (Arcadis 2016). The memorandum presented information and recommendations that varied slightly from the original intent of the project, which was to construct the Alternative I remediation. The intent of the memorandum was to explain that the additional slope stability analyses for the Alternative I remediation design did not provide an adequate Factor of Safety (FOS)[1] against slope failure.
- Modification 001 (M001) was issued on March 15, 2016, to extend the time for the USIBWC to complete a technical review of the "Geotechnical Assessment Memorandum 60 Percent Submittal" (Arcadis 2016).
- Modification 002 (M002) was issued on May 26, 2016. Arcadis was tasked with performing an additional geotechnical investigation and evaluation of other alternatives to mitigate the slope movement. The geotechnical investigation included drilling and sampling four borings, geotechnical laboratory testing on select samples, installation and monitoring of four inclinometers installed in the borings and performing a Pile Integrity Test (PIT) on the existing buried bulkhead. Four separate alternatives were initially analyzed to remediate the slope. Arcadis submitted the "Final Geotechnical Assessment Report," dated July 31, 2017 (Arcadis 2017) to USIBWC to present the results of this additional geotechnical investigation and evaluation.
- Modification 003 (M003) was issued on October 27, 2017. Arcadis was tasked with providing design and construction recommendations for implementation of Alternative No. 3, "Combination of the deep soil mix columns and stone columns," based on the findings of the "Final Geotechnical Assessment Report" (Arcadis 2017). Arcadis submitted the "Draft

[1] FOS is the ratio of the forces tending to resist failure divided by the forces tending to cause failure. Generally, an FOS greater than 1.0 indicates stable conditions. An FOS in the range of 1.3 to 1.5 is generally considered to be the minimum acceptable FOS in geotechnical engineering design. An FOS about equal to 1.0 indicates marginally stable conditions on the verge of failure, and an FOS less than 1.0 generally represents failure.

Geotechnical Report," dated January 4, 2018 (Arcadis 2018) to USIBWC to present the results of this modification study. The final version of this geotechnical report is included as Attachment B of this Design Documentation Report (DDR).

This DDR documents decisions made in the design process and development of contract documents for construction of the remediation of the levee and floodplain failure.

1.2 PROJECT LOCATION

The project is located in the Lower Rio Grande Flood Control Project (LRGFCP) in Brownsville, Texas. The specific project area extends from Station 1892+00 to Station 1904+85 on the Rio Grande levee. The project consists of improvements to the floodplain area below the existing levee and improvements to the existing Gatewell Structure 205.

1.3 PURPOSES OF THE PROJECT

The purpose of the project is to remediate the floodplain area by improving the stability of the slope. The USIBWC has made improvements to the LRGFCP to safely contain the design flood event on the Rio Grande. The existing levee system from Station 1892+00 to Station 1904+85 was refurbished in October 2013.

In March 2014, the USIBWC discovered cracks and a partial slope failure on the newly refurbished levee section and adjacent floodplain between Stations 1892+00 and 1904+85. The USIBWC engaged the USACE Engineer Research and Development Center to determine the causes for the partial slope failure and to provide remediation alternatives.

The USACE concluded that the local geologic conditions combined with a series of river water level events led to a partial slope failure. The USACE provided four remediation alternatives for addressing the partial slope failure. The USIBWC determined that USACE Remediation Alternative I would be the most effective at addressing the partial slope failure.

Upon further studies by the Team, consisting of obtaining additional data at the site and performing additional slope stability analyses, it was determined that USACE Remediation Alternative I would not completely remediate the slope failure. The Team determined that additional slope stability remediation was required to stabilize the existing levee and floodplain slope.

An additional purpose of the project is to restore full functionality of Gatewell Structure 205. The USIBWC determined that Gatewell Structure 205 has failed.

The final purpose of the project is to verify that the levee at this location meets Federal Emergency Management Agency (FEMA) criteria as described in Code of Federal Regulations (CFR) Title 44, Section 65.10, Paragraph (b) (2).

1.4 SCOPE OF THE PROJECT

The objective of the project is to remediate the right bank of the Rio Grande to stabilize the slope and to enable the levee to meet the requirements of the USACE Engineer Manual 1110-201913, *Design and Construction of Levees*, and the Federal Emergency Management Administration's levee certification requirements in 44 CFR Section 65.10. The scope of the project is to provide a complete design for remediating the levee and floodplain slope. The proposed improvements consist of (1) installing soil mix columns and stone columns to stabilize the slope, and (2) protecting the existing river bank from erosion. The proposed improvements include slope stabilization measures and slope erosion protection measures.

An additional scope of the project is to conduct a site assessment of Gate Well Structure 205 and to provide a design that will restore full functionality. The proposed improvements consist of repairing the existing Gatewell Structure 205, including construction of a new outfall pipe and a new outfall structure. The completed project documents include drawings, specifications, the DDR, and opinion of construction cost.

2.0 REVIEW OF PREVIOUS REPORTS AND STUDIES

2.1 TETRA TECH DESIGN REPORT

The USIBWC contracted with Tetra Tech, Inc. (Tetra Tech) to prepare levee rehabilitation improvement design and construction documents for the Upper Brownsville (Reach Four) Levee. The Tetra Tech rehabilitation addressed the levee and the Gatewell structures within Reach 4. Levee rehabilitation was completed in October 2013. The USIBWC provided these documents to the Team.

The levee rehabilitation raised the existing levee to a minimum elevation to comply with FEMA freeboard criteria and provided a minimum top width of 16 feet. Rock riprap was installed along riverside levee slopes from Station 1895+34 to Station 1896+94.

Tetra Tech's review of structures included Gatewell Structure 205 located near Station 1894+51. However, Tetra Tech did not inspect Gatewell Structure 205.

The Tetra Tech Design Report references a study prepared by USIBWC in 2003. The study determined river flows for the LRGFCP. According to USIBWC (2007), the design flood in the Team's study area, approximately River Station 54, is 20,000 cubic feet per second (cfs).

2.2 GATEWELL STRUCTURE 205

The USIBWC provided an electronic copy of a drawing of the Gatewell Structure 205. The drawing copy quality is poor; however, it is adequate for the Team's use. It appears that the drawing is dated 1948.

Based on observations by the Team on August 21, 2015, the river side of the structure has been modified. The open apron has been replaced by pipe and the catch basin on the land side has been modified to include two storm water pipes that enter the catch basin.

2.3 USIBWC HYDRAULIC ANALYSIS

The Tetra Tech report references a Hydraulic Study prepared by USIBWC in 2003. The study was created using Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 3.0. The HEC-RAS Model contains river cross sections in the Team's study area, approximately River Station 54. The modeling determined river water surfaces of approximately elevation 36.5 feet based on a discharge of 20,000 cfs. The modeling determined an average river flow velocity of 3.89 feet per second (fps) and a maximum velocity of 6.4 fps in the Team's study area. The USIBWC provided this documentation to the Team.

2.4 USIBWC ENVIRONMENTAL ASSESSMENT

The USIBWC prepared an Environmental Assessment (EA) dated September 2007 for the proposed rehabilitation of the Brownsville Levee System in the Lower Rio Grande Flood Control

Project (USIBWC 2007). The proposed rehabilitation consisted of raising the existing levee height up to about 2 feet, thus increasing the levee footprint up to about 12 feet. The EA reported a Finding of No Significant Impact. The USIBWC provided these documents to the Team.

2.5 GEOTECHNICAL REPORTS

2.5.1 Raba-Kistner Geotechnical Addendum - Subreach 4

Tetra Tech contracted with Raba-Kistner Consultants, Inc. (Raba-Kistner) to provide geotechnical engineering services for Subreach 4 of the Lower Rio Grande Flood Control Project Levee System. The original Technical Memorandum for Subreach 4 (Station 1270+72 to 1904+85) is identified as Raba-Kistner Report No. AMA08-115-00 dated July 24, 2009. The Team has a copy of this Report.

The Raba-Kistner report contained in Appendix H of the Tetra Tech Levee Rehabilitation Design Report is titled Geotechnical Addendum – Subreach 4. This report, dated June 1, 2009, contains additional geotechnical analyses for the levee between Stations 1717+00 and 1746+00. Seepage was analyzed at Station 1342+00. The Team has a copy of this report.

2.5.2 Raba-Kistner Evaluation of Placed Fill

The Raba-Kistner report included in Appendix G of the Tetra Tech Levee Rehabilitation Design Report is titled Evaluation of Placed Fill, Summary Report. This report, dated April 14, 2011, contains an evaluation of existing fill placed by the USIBWC in-house field crews on portions of the levee. The report concludes that "recent fill placement repairs to the existing levee embankments did not appear to be compacted to the standard densities required for USIBWC levee construction." The Raba-Kistner report in Appendix G of the Tetra Tech Design Report did not include any appendices.

2.5.3 Professional Services Industries, Inc., Report of Subsurface Exploration and Evaluation, Cameron County International Bridge (called Gateway International Bridge in this Report)

Cameron County contracted with Professional Services Industries, Inc. (PSI) to conduct a geotechnical exploration and foundation analysis for the Cameron County Gateway International Bridge in Brownsville, Texas. The report, dated March 7, 1984, presents observations of Pier No. 5 settlement and recommendations for providing additional pier support by installing a new foundation system for Pier No. 5. The USIBWC provided this report to the Team. The Team understands the recommendation was implemented.

2.5.4 Trinity Testing Laboratories, Inc., Truck Lane Improvements, Gateway International Bridge

Cameron County Engineering contracted with Trinity Testing Laboratories, Inc. (Trinity) to provide a report of subsurface exploration for the proposed truck lane improvements at the Gateway International Bridge in Brownsville, Texas. The report is dated June 7, 1992. The proposed improvements consisted of extending the bridge abutment by about 40 feet, thus widening the bridge span between Pier #5 and the abutment on the U.S. bound lanes. This improvement was intended to provide a larger turning radius. Pier #5 was reinforced in 1984 and was considered adequate to support the new loads.

The current Pier #5 is supported on 14-inch-diameter, augered, pressure-grouted piles (Augercast) at a tip elevation of -35 feet. The report stated that similar Augercast piles are suitable for the proposed abutment extension.

Boring 1, with a ground surface elevation of +30 feet, was drilled to elevation -45 feet. Boring 2, with a ground surface elevation of +41 feet, was drilled to elevation -34 feet. Water was encountered at elevation -1 foot in Boring 1 and at elevation 0 foot in Boring 2.

The Trinity report (Trinity 1992) stated that the soil conditions encountered in their two borings are similar to soil conditions found in the 1984 borings conducted by PSI (discussed above).

2.6 USACE GEOTECHNICAL REPORT

In March 2014, the USIBWC discovered cracks and a partial slope failure on the newly refurbished levee section and adjacent floodplain between Stations 1892+00 and 1904+85. The USIBWC engaged the USACE Engineer Research and Development Center to determine the causes for the partial slope failure and to provide remediation alternatives.

The USACE prepared a report titled "Geotechnical Evaluation of the Brownsville Levee Cracking and Partial Slope Failure," dated July 2015 (USACE 2015b), which was provided to the Team. The USACE evaluation included reviewing previous geotechnical and hydraulic analyses reports, providing a summary of the known historical records of the Rio Grande in this study area, performing field investigations and laboratory testing, presenting a discussion of the geologic and groundwater conditions, and conducting seepage and stability analyses.

The USACE concluded that the local geologic conditions combined with a series of river water level events led to a partial slope failure, and the USACE provided four remediation alternatives for addressing it. The USIBWC determined that USACE Remediation Alternative I would be the most effective at addressing the partial slope failure.

3.0 SURVEY

All elevations in this report are in U.S. survey feet and referenced to the North American Vertical Datum of 1988 (NAVD 88). The Team has prepared a Surveyor's Report describing the topographic survey that was conducted in support of this study. The report is included in Attachment A.

4.0 GEOTECHNICAL

The Team has prepared a geotechnical report describing the geotechnical conditions for this Study, which is included in Attachment B.

5.0 REMEDIATION DESIGN

5.1 BACKGROUND

The following is a brief presentation of events leading up to the current remediation design.

- The existing levee system from Station 1892+00 to Station 1904+85 was refurbished in October 2013.
- River level drawdown occurred between April and June 2014. River level drawdowns typically occur as part of upstream dam operations.
- The USIBWC discovered cracks and a partial slope failure on the newly refurbished levee section and adjacent floodplain between Stations 1892+00 and 1904+85 in March 2014.
- The USACE prepared a report titled "Geotechnical Evaluation of the Brownsville Levee Cracking and Partial Slope Failure," dated July 2015 (USACE 2015b). The USACE did not describe what is meant by the term "partial slope failure" in its report. The USACE concluded that the local geologic conditions combined with a series of river water level events led to a slope failure. The USIBWC determined that USACE Remediation Alternative I would be the most effective at addressing the partial slope failure.
- During various site visits, the Team observed cracks resulting from apparent initial soil movement along a specific levee length as opposed to observing a complete rotational failure.
- The Team conducted additional site investigations and performed additional slope stability analyses. The Team determined that USACE Remediation Alternative I would not completely remediate the levee slope failure. The Team determined that slope stability remediation including methods such as soil mix columns and stone columns were required to stabilize the levee slope. Slope stability remediation consists of soil mix columns, stone columns, an Articulated Concrete Block Mattress (ACBM) revetment erosion protection system (revetment system), and rock riprap erosion protection.

5.1.1 Upstream from the Two Gateway International Bridges

In the Team's opinion, a portion of the proposed remediation design (soil mix columns and stone columns) is not required upstream of the west Gateway International Bridge or from Station 1892+00 to approximately Station 1895+55. However, the ACBM revetment system and rock riprap are required in this area. The Team's opinion is based on the following information.

- USIBWC and USACE have not observed any cracks or slope failures upstream of the Gateway International Bridge (from Station 1892+00 to approximately Station 1895+55).
- Soils encountered in Boring DP 201 (Raba Kistner 2009) located upstream of the Bridges differ from the upper soils encountered in the various borings available downstream of the east Gateway International Bridge.
- The USACE Subsurface Profile A-A' supports the Team's opinion that the soils encountered in the borings upstream of the west Gateway International Bridge generally differ from the

upper soils encountered in the various borings available downstream of the east Gateway International Bridge.

The Team's proposed slope erosion protection system will include geotextile, rock riprap, and an ACBM revetment system placed on the existing river slope.

5.1.2 At the Two Gateway International Bridges

In the Team's opinion, the proposed rock riprap erosion protection is required to be installed upstream and downstream of the two Gateway International Bridge piers from approximately Station 1895+34 to Station 1896+94. Our opinion is based on the following information.

- USIBWC and USACE have not observed any cracks or slope failures in the Gateway International Bridge area (approximately from Station 1895+55 to Station 1896+60).
- Soils encountered in Borings B-1 and B-2 (PSI 1984) generally differ from the upper soils encountered in the various borings available downstream of the east Gateway International Bridge.
- Soils encountered in Borings B-1 and B-2 (Trinity 1992) generally differ from the upper soils encountered in the various borings available downstream of the east Gateway International Bridge. The two Trinity borings encountered soils similar to those encountered in the two PSI borings.
- The USACE Subsurface Profile A-A' supports the Team's opinion that the soils encountered in the two PSI borings and the two Trinity borings at the two Gateway International Bridges generally differ from the upper soils encountered in the various borings available downstream of the east Gateway International Bridge.

The Team's proposed slope protection system will include geotextile and properly sized rock riprap placed on the existing river slope.

5.1.3 Downstream of the Two Gateway International Bridges

The Team's proposed remediation design should be installed downstream of the east Gateway International Bridge from approximately Station 1896+00 to approximately Station 1904+85. Our opinion is based on the following information.

- USIBWC and USACE have observed cracks or slope failures downstream of the east Gateway International Bridge (from approximately Station 1899+00 to Station 1904+85). Cracks were observed beginning about Station 1899+00 and extending downstream to about Station 1904+85.
- Soils encountered in the various borings are susceptible to the seepage conditions described in the USACE report. The USACE Subsurface Profile A-A' clearly illustrates the soil conditions.

The Team's proposed slope protection system will include soil mix columns, stone columns, geotextile, properly designed soil filter, rock riprap, and an ACBM revetment system placed on the existing river slope.

5.2 DESIGN CRITERIA AND CALCULATIONS

5.2.1 Rio Grande

USIBWC provided information concerning the river flows and historical water levels to the Team, which we incorporated in the design calculations.

5.2.2 Factor of Safety Criteria

In accordance with the USACE Engineering Manual 1110-2-1913, Design and Construction of Levees (2000), the USACE recommends a long-term FOS of 1.3 and the end-of-construction FOS of 1.4 for standard levee embankments.

5.2.3 River Bank Slope Stability

The river bank slope area subject to slope instability is downstream of the east Gateway International Bridge (from approximately Station 1896+60 to Station 1904+85). The area to be stabilized will begin at elevation +10 feet and extend up the river bank slope to the top of the levee.

The Team performed slope stability analyses using GeoStudio 2007 version 7.23 SLOPE/W software by Geo-Slope International. Spencer's method was used to compute the theoretical factor of safety. The geotechnical report in Attachment B discusses the slope stability analysis.

The Team proposes to use soil mix columns and stone columns as remediation methods to stabilize the levee slope. The recommended procurement process for selecting a construction contractor is described in Section 8.0 of this DDR.

5.2.4 Soil Mix Columns

Soil mixing involves blending a binder with soil in situ to produce a soil binder mix that has increased strength and reduced compressibility properties compared to the untreated soil. Binder materials can consist of Portland cement, lime, fly ash, or other materials as well as binder material blends.

Soil mix columns are constructed using vertical-axis mixing equipment with multiple mixing blades mounted on one or more mixing shafts to form single or multiple overlapping columns. Wet soil mixing involves introducing water to create a soil-binder-water slurry. Dry soil mixing involves introducing the binder into the soil without adding water.

The soil mix columns are laid out in a linear fashion perpendicular to the soil slope (perpendicular to the river in this project). The soil mix columns in each line are overlapped to create a soil mix column shear wall.

The Team determined how much the existing slope soil mass must be strengthened to achieve an acceptable slope stability FOS. Incorporating soil mix columns into the soil mass will increase the soil strength to improve the slope stability FOS. An acceptable soil mix design is presented in the geotechnical report (Attachment B).

5.2.5 Stone Columns

Construction of stone columns involves the partial replacement of unsuitable soils with a compacted vertical column of aggregate (stone). Stone column construction is typically accomplished by down-hole compaction methods. Creating a stone column involves introducing backfill material (typically aggregate) into the soil so that dense aggregate columns are formed. The compacted aggregate or stone is tightly interlocked with the surrounding soil.

Stone column construction techniques include vibro-replacement, vibro-displacement, or other augering and compaction techniques. Vibro-replacement is a wet, top-feed process using jetting water to aid the ground penetration by the vibrator. Vibro-displacement is a dry, top- or bottom-feed process. Installation techniques also include augering and introducing the aggregate into the open hole. Casing may be required to keep the angered hole open depending on soil and groundwater conditions.

The Team determined that the slope stability FOS can be increased by installing stone columns within the slope soil mass. Incorporating stone columns into the soil mass will increase the soil slope stability FOS by increasing the resistance on the potential shear plane. Stone column design is presented in the geotechnical report (Attachment B).

5.2.6 Geotextile

Geotextile is proposed for this project for three uses: 1) geotextile will separate the existing soil and the overlying rock riprap; 2) geotextile placed under the proposed soil filter will protect the in situ soil from erosion caused by groundwater seepage or river flows; and 3) geotextile is typically placed on the exposed soil surface, in this case an aggregate soil filter, when placing the ACBM revetment system.

The geotextile is designed according to current design criteria (Koerner 2012) and Active Standard ASTM E 11-17 (http://www.astm.org/cgi-bin/resolver.cgi?E11). The Team referred to Specification Section 31.05.19, Geotextile for Earthwork. Geotextile incorporated into the ACBM revetment system will be designed by the manufacturer of the ACBM revetment system. The Team prepared calculations for the geotextile (Attachment C.1).

Geotextile will be incorporated into and attached under the ACBM revetment system in the following locations:

- Station 1892+12 to Station 1895+34
- Station 1896+94 to Station 1904+50

Geotextile placed under the ACBM revetment system will be designed by the ACBM manufacturer.

Geotextile placed under the rock riprap and under the soil filter will consist of the geotextile identified in Specification Section 31.05.19. Geotextile will be placed under the rock riprap in the following locations:

- 18-inch-thick riprap
  - Station 1895+46 to Station 1896+82
  - Station 1904+62 to Station 1904+85
- 48-inch-thick riprap
  - Station 1892+00 to Station 1892+12
  - Station 1895+34 to Station 1895+46
  - Station 1896+82 to Station 1896+94
  - Station 1904+50 to Station 1904+62

5.2.7 Aggregate Soil Filter

The objective of a filter used in seepage control is to efficiently control the movement of water seeping from the natural river bank slope. Filters must retain the protected materials (stability), allow relatively free movement of water (permeability), and have sufficient discharge capacity. Suitable aggregate soil filter material is typically concrete sand combined with fine to coarse sand or gravel. The aggregate soil filter will be placed and compacted to the required relative density.

Underseepage in pervious soils found on the river bank soil slope may result in excess hydrostatic pressures beneath the levee. The USACE determined that underseepage in the pervious soils contributed to the partial slope failure observed downstream of the east Gateway International Bridge. The seepage results from the difference in water levels in the Rio Grande and Lake Brown (Resaca). The difference in water levels was exacerbated when a river level drawdown occurred with a high-water level in Lake Brown between April and June 2014.

A suitable aggregate soil filter was designed using industry-accepted design procedures from the USACE (Design and Construction of Levees, CECW-EC, Manual No. 1110-2-1913; USACE 2000) and from FEMA (Filters for Embankment Dams, Design and Construction; FEMA 2011). Soil properties were taken from the USACE report for use in the design.

The Team determined that an aggregate soil filter should be located under the ACBM revetment system downstream of the Gateway International Bridge in the partial slope failure area. The aggregate soil filter is not required under the ACBM revetment system upstream of the Gateway International Bridge.

The aggregate soil filter will be placed on the exposed river bank on 2-horizontal to 1-vertical (2H:1V) and flatter slopes. River bank slopes steeper than 2H:1V are typically found near the water's edge. These steeper slopes will be regraded to 2H:1V slopes before placement of the aggregate soil filter. The Team prepared calculations for the proposed aggregate soil filter (Attachment C.2).

Table 1 presents the recommended gradation of the aggregate soil filter. The minimum thickness of the soil filter is 24 inches. Suitable aggregate soil filter material is typically concrete sand combined with fine to coarse sand or aggregate.

TABLE 1 - SOIL FILTER DESIGN

Design Target			
Sieve Size	Percent Passing	Percent Retained ea Sieve	Percent Retained
3"	100%	0%	0%
2"	97%	3%	3%
3/4"	93%	4%	7%
3/8"	91%	2%	9%
No. 4	83%	8%	17%
No. 8	75%	8%	25%
No. 16	65%	10%	35%
No. 30	40%	25%	60%
No. 40	18%	22%	82%
No. 50	10%	8%	90%
No. 100	7%	3%	93%
No. 200	0%	7%	100%

Design Range		
Sieve Size	Percent Passing	
3"		100%
2"	95%	100%
3/4"	90%	100%
3/8"	85%	95%
No. 4	75%	90%
No. 8	60%	80%
No. 16	55%	75%
No. 30	30%	55%
No. 40	15%	30%
No. 50	5%	20%
No. 100	5%	10%
No. 200		0%

5.2.8 Articulated Concrete Block Mattress Revetment System

An ACBM revetment system is a matrix of individual concrete blocks placed together to form an erosion resistant overlay with specific hydraulic performance characteristics. The system includes interlocking concrete blocks connected by corrosion resistant steel cables or special synthetic cables in both directions. The cables are connected to adjacent mattress cables and to soil anchors along the outside edge of the individual mattresses. The term *articulated* implies the ability of the blocks to conform to changes in grade while maintaining interlocking and connecting features. The revetment system includes a geotextile underlay that allows water infiltration and exfiltration to occur while providing underlying soil particle retention.

The ACBM revetment system provides the following characteristics beneficial to the proposed use on this project.

- Economically comparable with conventional rock riprap
- Effective erosion control and armoring system for river level changes
- Resistant to hydraulic shear and overtopping

- Protects the proposed soil filter installed on the river bank slope
- Can be installed above and below water
- Long-term durability and maintainable with expansion and contraction capability
- Environmentally sound and aesthetically pleasing allowing native vegetation growth
- Easy to install with typical construction equipment
- Traversable with light- to moderate-weight maintenance trucks and mowers.

The Team designed a suitable ACBM revetment system using industry-accepted design procedures from the National Concrete Masonry Association (2010) and ACBM vendors. The ACBM revetment system will be placed on areas of the river bank slope upstream of the two Gateway International Bridges, and downstream of the two Gateway International Bridges as an alternative to rock riprap.

The selected ACBM revetment system will be placed on areas of the river bank slope as shown on the contract drawings. The ACBM revetment system design is based on the river flow of 20,000 cfs at river water level elevation +37 feet. The average river water velocity is 3.89 fps and the maximum river water velocity is 6.4 fps. The ACBM revetment system was designed based on 6.4 fps and was checked against the design flood event using a river water velocity of 10 fps. The river bank slope varies from about 1.25H:1V upstream of the Gateway International Bridges to a constructed 2H:1V downstream of the Gateway International Bridges. The ACBM revetment system was designed based on the 1.25H:1V river bank slope.

The designed ACBM revetment system will use 4-inch-thick concrete open cell blocks. In addition, the bottom two rows of blocks on the ACBM revetment system placed in the river will be 9-inch-thick concrete closed cell blocks. The blocks are connected by cables attached in both directions, called “bi-directional.” This design is suitable for placing upstream and downstream of the Gateway International Bridges.

The Team prepared calculations for the proposed ACBM revetment system (Attachment C.3) using a concrete block from a specific vendor in the calculations. However, this same type of concrete block is typically available from other vendors.

5.2.9 Rock Riprap

Rock riprap is typically used as an erosion protection system on slopes exposed to run-off or erosion by flowing water. The Team designed the rock riprap using industry-accepted design procedures from the USACE (2000). The Team applied Specification Section 35.42.37, Riprap. Rock riprap will be used at several locations along the river bank soil slope as described below. The Team prepared calculations for the proposed rock riprap (Attachment C.4).

Rock Riprap Combined with the ACBM Revetment System. The ACBM revetment system will be placed at the upstream beginning of the project, Station 1892+12, continuing downstream to Station 1895+34, and at the downstream end of the project, Station 1896+94, continuing downstream to Station 1904+50. We have designed suitable rock riprap to be placed adjacent to the ACBM revetment system at its upstream and downstream ends and between Stations 1895+34 and 1896+94.

Rock riprap will be placed at the upstream and downstream ends of the ACBM revetment system in a minimum layer thickness of 48 inches. Rock riprap at these locations will consist of 14-inch maximum rock size identified in Specification Section 35.42.37, Riprap.

- 48-inch-thick rip rap
  - Station 1892+00 to Station 1892+12
  - Station 1895+34 to Station 1895+46
  - Station 1896+82 to Station 1896+94
  - Station 1904+50 to Station 1904+62

Rock Riprap at the Two Gateway International Bridges. The Team recommends using rock riprap in the vicinity of the two Gateway International Bridge foundation elements. We have designed suitable rock riprap to be placed in and around the bridge foundation elements. Rock riprap will be placed at the upstream and downstream ends of the ACBM revetment system at the Gateway International Bridges.

Rock riprap will be placed in and around the two Gateway International Bridge foundations in a layer at least 18 inches thick. Rock riprap will be placed at the downstream end of the project in a layer at least 18 inches thick. Rock riprap at these locations will consist of 200 pounds and 500 pounds <u>maximum</u> rock weight identified in Specification Section 35.42.37, Riprap.

- 18-inch-thick riprap
  - Station 1895+46 to Station 1896+82 [500 pounds rock size]
  - Station 1904+62 to Station 1904+85 [200 pounds rock size]

5.2.10 Launchable Stone

River bank erosion protection must be able to withstand movement or scour at the river bank toe of slope. Rock riprap is proposed on this project as the erosion protection system on slopes exposed to run-off or erosion by flowing water. Placing a large amount of riprap at the toe of a river bank exposed scour at the toe, termed launchable stone, is a common method of reducing damage by erosion as the riprap migrates, or launches, downslope as a river bank is undercut by river flows.

We have designed the required launchable stone to be placed at the toe of the upstream and downstream ends of the ACBM revetment system and at the toe of the rock riprap slope at the two Gateway International Bridge foundations. The Team designed the launchable stone (rock riprap) using industry-accepted design procedures from the USACE (Toe Scour and Bank Protection Using Launchable Stone, Technical Report HL-95-11).

Rock riprap will be placed as launchable stone at the upstream and downstream ends of the ACBM revetment system. Launchable rock riprap will consist of 200 pounds <u>maximum</u> rock weight identified in Specification Section 35.42.37, Riprap.

Rock riprap will be placed as launchable stone at the toe of the rock riprap slope at the two Gateway International Bridges. Launchable rock riprap will consist of 500 pounds maximum rock weight identified in Specification Section 35.42.37, Riprap.

6.0 GATEWELL STRUCTURE 205

6.1 EXISTING CONDITIONS

The USIBWC provided an electronic copy of a drawing of Gatewell Structure 205 (Reference: RFI No. 5). The drawing appears to be dated April 1948. Upslope of Gatewell Structure 205 is a storm water catch basin located at street level (Levee Street). The catch basin is connected to the gatewell structure with a 15-inch-diameter concrete pipe. No other storm water pipes are connected to the catch basin according to the 1948 drawing. Downslope of the gatewell structure there is an approximately 24-foot-long concrete pipe that transitions into an approximately 15-foot-long concrete-lined channel. The channel outfalls into the river. Rock riprap is placed at the outfall channel.

The Tetra Tech Design Report (January 2011) presented a brief discussion of Gatewell Structure 205 in an interior drainage assessment report. However, Tetra Tech did not inspect Gatewell Structure 205. The assessment report, dated January 2011, discusses deficiencies obtained from a prior USIBWC inspection report. In this inspection report, the encroachments and the sluice gate rated an “A.” The gate operational item was labeled “Y.” The grill cover, steps, and paint were labeled “OK.” The other inspection items either had “N/A” or “no assessment was noted.” The pipe conditions were not inspected. The Tetra Tech report stated that the USIBWC needed to inspect Gatewell Structure 205. The report stated that no anticipated relocation or improvements to these existing structures are required unless the structures are damaged.

Based on observations by the Team on August 21, 2015, the downslope or riverside of Gatewell Structure 205 has been modified from its original construction. The open apron has been replaced by pipe. The catch basin on the landside has been modified to include two storm water pipes entering the catch basin. USIBWC personnel reported that the end of the downstream pipe recently collapsed. USIBWC constructed a temporary fix using polyvinyl chloride (PVC) pipe.

The Team conducted a camera investigation of the current downslope outfall pipe. The camera was run from the gatewell structure to the end of the downslope pipe. Beginning at the gatewell structure, the existing pipe consists of 15-inch-diameter reinforced concrete pipe (RCP) extending about 20 feet toward the river. This RCP appears to be in good condition. There is a concrete closure pour collar at the end of the RCP. The collar appears to have been formed around a corrugated metal pipe (CMP) that is no longer present. A black high-density polyethylene (HDPE) pipe (corrugated exterior and smooth interior) begins at the concrete collar and extends about 19 feet toward the river. At the end of the HDPE pipe is about 10 feet of PVC pipe that daylights on the river bank.

USIBWC provided structure data sheets for Gatewell Structure 205 as requested by the Team in RFI No. 14, which are summarized as follows:

Date	Summary	Comments
6/1/12	Functional	Gate travels smoothly and seals correctly, gate ring not mortared
3/11/13	Functional	Needs mortar inside behind ring
4/14/13	Functional	Operates smoothly, needs cement patch work
1/22/14	Functional	Gate travels smoothly and seals correctly, paint and numbers, missing wedge blocks around box
2/3/14	Functional	Move ladder, it is in front of hole, restricting water flow, missing wedge blocks around box
3/10/14	Functional	Gate travels smoothly and seals correctly, missing wedge blocks around box
4/15/15	Functional	Gate travels smoothly and seals correctly, missing wedge blocks around box
4/22/15	Functional	Gate travels smoothly and seals correctly, paint, missing wedge blocks around box

CFR Title 44, Section 65.10, Paragraph (b) (2) states "All openings must be provided with closure devices that are structural parts of the system during operation and design(ed) according to sound engineering practice." Gatewell Structure 205 contains a sluice gate that is a structural part of the Gatewell and that can be opened and closed. Based on the structure data sheets, the Team concludes that the structure and gate are functionally operational.

CFR Title 44, Section 65.10, Paragraph (c) (1) requires operation plans for closures that include documentation of the flood warning system, a formal plan of operation, and provisions for periodic operation. Compliance with Paragraph (c) (1) is the responsibility of the USIBWC.

6.2 PROPOSED REPAIR

Based on the results of the camera investigation, the Team recommends replacing the existing HDPE and PVC pipe with 15-inch-diameter RCP.

6.2.1 Design Criteria

The RCP is designed according to the Concrete Pipe Design Manual (American Concrete Pipe Association (2011), and Specification Section 33.05.39.41, Reinforced Concrete Pipe. The load applied to the RCP is AASHTO HS 20. The trench will be excavated and backfilled according to Specification Section 31.23.00, Excavation and Backfill for Structures. Our design is based on Section 3.3, Shaping and Bedding, and Class A bedding illustrated on Figure 10 in Specification Section 31.23.00. The RCP inside diameter is 15 inches. The compacted backfill will consist of cement stabilized sand. The proposed cast-in-place concrete pipe collar extension will follow Specification Section 03.30.00, Cast-in-Place Concrete.

The RCP will exit the trench through a planned opening in the proposed ACBM revetment system. The opening area in the ACBM revetment system surrounding the RCP will be strengthened with concrete grout.

6.2.3 Design Calculations

The RCP design calculations are presented in Attachment C.5.

6.2.4 Design Results

Class IV RCP, 15 inches in diameter, will be placed in an excavated trench at the downstream end of Gatewell Structure 205. The existing concrete collar will be extended to enclose the beginning of the first joint of the RCP. The RCP will penetrate the proposed ACBM revetment system placed on the river bank slope.

7.0 ADDITIONAL DESIGN CRITERIA

7.1 FREEBOARD

CFR Title 44, Section 65.10, Paragraph (b) (1) states "Riverine levees must provide a minimum freeboard of three feet above the water-surface level of the base flood. An additional one foot above the minimum is required within 100 feet on either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted."

Tetra Tech Design Report (2011) indicates that, at this project location, the freeboard criteria are met based on the USIBWC HEC-RAS model and the proposed top of levee elevations shown on the Tetra Tech Record Drawings accompanying their Design Report.

The levee and floodplain remediation will remove earth materials from the floodplain. Therefore, the base flood elevation will not increase because of the anticipated construction.

7.2 REVEGETATION

Specification Section 32.92.00, Vegetation for Erosion Control (Texas), states that all areas disturbed by construction activities will be revegetated. In the Team's opinion, the areas covered by the ACBM revetment system will not be revegetated and the openings will be filled with aggregate as requested by the USIBWC. Areas around the ACBM revetment system and riprap that are disturbed by construction activities will be revegetated as specified in Specification Section 32.92.00.

7.3 ACCESS ROAD

USIBWC currently has an access road located at the toe of the existing levee. This access road will be used by the construction contractor to perform the work. The specifications require the construction contractor to repair the access road on completion of the construction activities. Reconstruction of the access road will follow Specification Section 32.15.00, Aggregate Road Surfacing.

7.4 LEVEE REGRADING

The slope failure downstream of the two Gateway International Bridges created a scarp on top of the levee and at several locations along the levee toe. Placing the soil filter on the slope will require the slope to be regraded to a slope no steeper that 2H:1V. The specifications require the Contractor to regrade these areas to reduce the slope of the scarp to 2H:1V.

8.0 BEST-VALUE PROCUREMENT PROCEDURE

As described in this DDR, the objective of the project is to remediate the right bank of the Rio Grande to stabilize the slope and to enable the levee to meet the requirements of the USACE Engineer Manual 1110-201913, *Design and Construction of Levees*, and the Federal Emergency Management Administration's levee certification requirements in 44 CFR Section 65.10. The major elements of this project include stabilizing the river bank using deep soil mixing (DSM) and stone columns; installing an articulated concrete block mattress (ACBM) revetment system; placing rock riprap and launchable stone; restoring functionality of Gatewell Structure 205; and providing related slope improvements and revegetation of disturbed areas.

In many typical construction projects, the installation of DSM or stone columns is an important but incidental part of the work that is provided by specialty contractors who have the requisite skills, experience, and equipment to complete the work. The DSM or stone column specialty contractors typically work as subcontractors to a prime contractor who is responsible for completing the project. Because specialty contractors have dedicated, often-unique equipment for installation of DSM and stone columns, and because specialty contractors have proprietary means and methods, they are also often responsible for developing their own detailed designs, which the specialty contractors prepare to meet specific project objectives.

Similarly, the manufacturers of ACBM often have dedicated equipment and proprietary means and methods to produce specialized ACBM products that are specially designed and manufactured to meet project objectives.

For the Brownsville slope remediation project, the installation of DSM, stone columns, and ACBM comprises the bulk of project work; the remaining work is minor and incidental. For this reason, we anticipate that specialty contractors will likely bid this work as the prime contractor.
In this project, the goal is to achieve improved soil volumes that are defined in the contract documents by the footprints and depths of the DSM and stone columns. The USIBWC can seek construction bids using the current designs for the DSM, stone columns, and ACBM as described in this DDR and in the contract documents, which is termed the base design. Because specialty contractors have their own equipment and their own means and methods, offerors may be capable of meeting this project goal using alternative designs for these elements of the work. For example, a DSM specialty contractor may be able use its available equipment and its own means and methods that result in soil-mix columns that differ in diameter and spacing from the base design, but that achieve the improved soil volume defined in the contract documents. The same is true for stone columns.

To encourage participation from the maximum number of qualified offerors, we understand that the USIBWC intends to evaluate proposals submitted for this project and select a source for contract award in accordance with USIBWC's source evaluation and selection procedures that are based on procedures contained in FAR Part 15. The USIBWC will negotiate a contract for construction services based on qualifications, demonstrated technical approach, and reasonable price. The goal is to obtain best value (or, the greatest overall benefit) using the requirements and evaluation factors set forth in the solicitation. The source selection will be based on a comparative assessment of proposals against all source selection criteria in the solicitation.

Using this procedure, the contract selection will be made to the offeror that provides the greatest overall benefit in response to the project requirements. The award will be made to the responsible offeror submitting a proposal that conforms to the solicitation requirements and is most advantageous to USIBWC considering the factors provided in the solicitation.

The source selection will be based on a two-step comparison of proposals against source selection criteria in the solicitation. The first step is evaluation of technical factors, which may include, but are not limited to, technical approach and plan for construction (which may include alternative designs), past performance, experience, safety, construction schedule, and submittal compliance. The second step is evaluation of price.

If a proposal is determined by USIBWC to be technically unacceptable based on evaluation factors other than price, the proposal may be rejected from further consideration. Using this procedure, the USIBWC reserves the right to make award to either a lower-scored/lower-price offeror or to a higher-scored/higher/price offeror based on a rational tradeoff between evaluation factors other than price. In addition, the USIBWC reserves the right to use information outside of the proposal to evaluate the capability of the offerors and the value of offers.

9.0 CONSTRUCTION CONSIDERATIONS

9.1 RIVER BANK IMPROVEMENT AND REGRADING

The river bank area to be improved extends from Station 1892+00 to Station 1904+85.07. The river bank area to be regraded is downstream of the east Gateway International Bridge (from approximately Station 1896+60 to Station 1904+85). The area to be improved begins at elevation +10 feet and extends up the embankment to the top of the levee. Regrading of the slope will be conducted in the areas where the existing slope is steeper than 2H:1V. Specification Section 31.11.00, Preparing Right-Of-Way, provides site clearing directions to the construction contractor.

9.2 SOIL MIX COLUMNS

The construction contractor will mobilize equipment on site when soil column installation commences. Typically, the equipment will include a crane-mounted drilling rig and binder mixing tanks. The construction contractor will provide the equipment operators and the binder mixing personnel and will provide contractor quality control testing equipment and personnel. The construction contractor will be required to clean and regrade the site to the desired grades after soil mix column installation is completed.

9.3 STONE COLUMNS

The construction contractor will mobilize equipment on site when stone column installation commences. Typically, the equipment will include a crane-mounted drilling rig, vibratory, or hammer equipment depending on the stone column installation technique, and the desired stone. The construction contractor will provide the equipment operators and will provide the construction contractor quality control testing equipment and personnel. The construction contractor will be required to clean and regrade the site to the desired grades after stone column installation is completed.

9.4 GEOTEXTILE PLACEMENT

On completion of the stone column installation, a 3-foot-thick layer of compacted soil will be placed and graded over the stone columns. Specification Section 35.41.00, Construction of Levee, provides required material and compaction criteria for the 3-foot-thick soil layer. A geotextile will be placed on the compacted soil surface in the areas that will receive the soil filter.

9.5 AGGREGATE SOIL FILTER CONSTRUCTION

The specified aggregate soil filter will be placed on the geotextile and compacted to the required relative density.

9.6 ACBM REVETMENT SYSTEM CONSTRUCTION

The ACBM revetment system will be placed on the river bank and soil slopes upstream and downstream of the two Gateway International Bridges. The ACBM revetment system includes a geotextile that is placed directly under the ACBM and on top of the aggregate soil filter.

9.7 RIPRAP CONSTRUCTION

9.7.1 Rock Riprap Adjacent to the ACBM Revetment System

Rock riprap will be placed adjacent to the ACBM revetment system at the upstream beginning of the project (Station 1892+00) and at the downstream end of the project (Station 1904+85).

9.7.2 Rock Riprap at the Two Gateway International Bridges

Rock riprap will be placed in and around the bridge foundations of the two Gateway International Bridges. Rock riprap will be placed adjacent to the ACBM revetment system at the upstream and downstream approaches.

9.7.3 Launchable Stone

Launchable stone will be placed at the toe of the upstream and downstream ends of the ACBM revetment system and at the toe of the rock riprap slope at the two Gateway International Bridge foundations.

9.8 GATEWELL STRUCTURE 205

The existing HDPE and PVC pipes will be removed and replaced with 15-inch-diameter RCP. The RCP will be placed in an excavated trench extending toward the river bank beginning at the existing concrete collar will be replaced. The RCP trench will be backfilled according to Specification Section 31.23.00. The RCP will exit the trench through a planned opening in the proposed ACBM revetment system. The opening area in the ACBM revetment system surrounding the RCP will be strengthened with concrete grout.

9.9 ACCESS ROAD

Portions of the access road will be excavated to serve as the anchor trench for the ACBM revetment as shown on the contract drawings. After placing the ACBM anchoring system, the trench will be backfilled with cement stabilized sand. The access road will be regraded above the anchor trench excavation. Aggregate specified by Specification Section 32.15.00, Aggregate Road Surfacing, will be placed on the access road.

The access road in the vicinity of the RCP replacement for Gatewell Structure 205 will be regraded above the excavated and backfilled trench. Aggregate specified by Specification Section 32.15.00, Aggregate Road Surfacing, will be placed on the access road.

9.10 ENVIRONMENTAL ASPECTS

The construction contractor will be provided a copy of the USIBWC Final Environmental Assessment (USIBWC 2007). In addition, the construction contractor will be required to comply with the USIBWC contract documents concerning the environmental aspects of the project.

Specification Section 00.31.24, Environmental Information, requires the construction contractor to preserve the cultural and natural resources and to preserve the environment during the construction phase.

Specification Section 01.57.00, Temporary Controls for Work Areas, provides direction to the construction contractor concerning the work area, staging areas, borrow sites, and field office.

Specification Section 01.57.13, Temporary Environmental Controls, directs the construction contractor to provide and maintain the necessary Best Management Practices (BMP) to protect the environment. USIBWC will obtain the services of a Field Environmental Monitor to monitor compliance with BMPs incorporated into the project. Section 1.18, Migratory Birds, of Specification Section 01.57.13, identifies certain times of the year (March 1 through August 31) when construction activities should not be scheduled. However, construction activities can occur during those times if certain safeguards are followed.

Specification Section 02.21.13, Cultural Resources Surveys, identifies Cultural Resources Surveys that must be conducted prior to the work. Unless a previous Cultural Resource Survey already exists, the construction contractor will be required to conduct Cultural Resource Surveys for all off-site geological material sources prior to using these materials on the project. The USIBWC Final Environmental Assessment (USIBWC 2007) contains a Cultural Resources Survey for the construction site. An independent archeologist retained by USIBWC inspect the project or off-site source during the work.

10.0 CONCLUSIONS

10.1 DESIGN

The Team concluded that the local geologic conditions combined with a series of river water level events led to a partial slope failure. The Team has reached the following conclusions to stabilize the river bank soil slope based on our study of the project. These improvements will not change the original levee footprint. The design options selected for the levee improvement will not affect the FEMA criteria. The proposed improvements will improve the FOS and the rapid drawdown conditions. The recommended procurement process is described in Section 8.0.

10.1.1 Soil Mix Columns

Installation of soil mix columns will increase the strength of the river bank soil slope. This increase in strength along with the stone columns is expected to result in an acceptable slope stability FOS.

10.1.2 Stone Columns

Installation of the stone columns will increase the resistance on the potential shear plane. This increase in resistance along with the soil mix columns is expected to result in an acceptable slope stability FOS.

10.1.3 Geotextile

Geotextile will act as a separator between the existing soil and the rock riprap and will act to protect soil from erosion by groundwater seepage and river flows. Geotextile is typically placed on the exposed soil surface when placing the ACBM revetment system.

10.1.4 Aggregate Soil Filter

The aggregate soil filter controls the movement of seepage under the levee and exiting onto the river bank soil slope. The aggregate soil filter will retain the protected materials (stability), allow relatively free movement of water (permeability), and have sufficient discharge capacity.

10.1.5 ACBM Revetment System

The Team recommends an ACBM revetment system on the river bank slope upstream of the two Gateway International Bridges. The Team recommends an ACBM revetment system as an alternative to rock riprap on the soil slope downstream of the two Gateway International Bridges.

10.1.6 Rock Riprap

10.1.6.1 Upstream of the West International Bridge

Rock riprap will be placed on the existing river slope immediately upstream and downstream of the ACBM revetment system as an erosion protection measure.

10.1.6.2 Vicinity of the Two Gateway International Bridges

Rock riprap will be placed on the existing river slope in and around the Gateway International Bridge columns.

10.1.6.3 Downstream of the East Gateway International Bridge

The proposed slope protection system will include soil mix columns, stone columns, geotextile, soil filter, and the ACBM revetment system placed on the existing river bank soil slope.

10.2 GATEWELL STRUCTURE 205

The Team has designed a 15-inch diameter RCP to replace the existing HDPE and PVC pipes.

10.3 FREEBOARD

The remediation of the levee and floodplain failure will remove earth materials from the floodplain. Therefore, the base flood elevation will not increase because of the anticipated construction.

10.4 CONSTRUCTION QUANTITIES

The Team has prepared a list of quantities based on the design assumptions, criteria, drawings, specifications and results.

11.0 REFERENCES

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USIBWC, Lower Brownsville Levee Rehabilitation, Cameron County, Texas, Station 1904+85.12 to 2566+63.04

ATTACHMENT A

SURVEYOR'S REPORT

Munoz & Dannenbaum JV
SBA Approved 8(a) Joint Venture
Mercedes | McAllen | Laredo | El Paso | Houston | Fort Worth | Dallas | Austin

2020 E. Expressway 83
Mercedes, TX 78570 | 956.565.4637

Date: March 27, 2019

Project: IBM15D001 Brownsville Levee Failure Remediation
Location: Upper Brownsville Levee Reach 4, in the vicinity of Gateway International Bridge, approximately from Station 1892+00 to 1904+85, Brownsville, Cameron County, Texas.

Relative Dates:
- ***Initial Start: November 9, 2015***
- ***Completion Survey Work December 11, 2015***

Personnel:
- ***Survey Field Crew: Rolando Arellano and Ruben Garcia***
- ***Office: Danny Gaitan (Survey Crew Supervisor) and Felix Rodriguez (RPLS)***

SURVEYOR'S REPORT

Topographic survey
of
Upper Brownsville Levee Reach 4, in the vicinity of Gateway International Bridge, approximately from Station 1892+00 to 1904+85, Brownsville, Cameron County, Texas.

Location Map in Brownsville, Texas

Purpose:

This Surveyor's Report describes the topographic survey that was conducted in support of the design work associated with IBM15D001 – Brownsville Levee Failure Remediation. The actual survey work was conducted between IBWC Station 1892+00 to 1904+85, which is approximately 1,285 feet of river levee from the border wall to the water's edge on the U.S. side of the Rio Grande within the vicinity of the Brownsville Gateway international Bridge. It also included IBWC Gatewell Structure No. 205 and the collapsed storm drainage pipe that daylights on the riverside of the levee at IBWC Station 1894+51.4.

1. The Survey Crew ran a 3-wire level loop from Published NGS survey markers located near the vicinity of the survey site. Survey Team referenced NGS datum as noted on Construction plans for the I.B.W.C Upper Brownsville Levee Rehabilitation United States Section Cameron County - from Sta. 1270+72.00 to Sta. 1904+85.12 Cameron County Texas; however, these plans did not show any NGS survey markers within the vicinity of our survey site. The Survey Crew references the nearest NGS vertical survey marker "K 678" located in the Brownsville downtown area and ran a 3-wire level loop to the site to the top IBWC BM Disk No "205-RC" found on top of Gatewell Structure 205. The 3-wire level loop was utilized to check closure to Published NGS survey marker "RM 2", located inside The Texas Southmost College campus. Both published NGS Survey markers are in NAVD 88 datum. It is hereby noted that the found IBWC BM Disk No "205-RC" stamped elevation appears to be in a different datum. The stamped elevation on said disk is 42.11; however, our level run record reading is 41.56 (NADV 88). The vertical work was continued in NAVD 88 datum as referenced from the two current NGS Survey Markers. However, on December 10, 2015, it was determined that the appropriate course of action was to adjust all elevations to the stamped elevation of IBWC BM Disk No. "205-RC". The vertical adjustment will be +0.55' to all elevations.

2. Horizontal control is based on established Western Data system VRS network under NAD 83 Texas coordinate system, South Zone, with no calibration. No NAD 83 Horizontal NGS Survey Markers were available within the vicinity of the survey site. It was decided to keep the VRS alignment due to finding a nail on top of the levee at End Station 1904+85.12 that was approximately 0.18' off the Meldon and Hunt Consulting Engineers noted coordinates for Station 1904+85.12. The Field Crew established traverse points throughout the site by GPS observations; performed a level run thru most, if not all, of those traverse points; then utilized those horizontal control points with the Robotics Total Station to perform an accurate fieldwork. Additionally, the Field Crew used their vertical readings as obtained on their bench level run.

3. Field notes have been completed by the Field Crew.

4. Some Levee Right-of-way maps were obtained from Cameron County Engineering Department, including some recorded information, and some mapping and field note information was obtained from the U.S. IBWC.

5. Some U.S. IBWC monumentation was found and tied in by the Field Crew, including control found along or around Block 59 of the Original Townsite of Brownsville.

6. Areas of concern were tied in by the Field Crew, including Gatewell Structure 205 and its outfall storm drainage pipe that daylights on the riverside of the U.S. IBWC levee.

7. Cross sections were taken at 50 Ft. intervals from the DHS Border fence/wall down to the water's edge of the Rio Grande. On December 10, 2015, the Field Crew obtained vertical data along the U.S. water's edge of the Rio Grande at 100 Ft. intervals where possible.

8. The Field Crew found and tied in some monumentation that included U.S. IBWC monuments, iron rods, spindles, and punch marks, that may pertain to some of the Levee Easements;

9. Raw data reductions of both horizontal and vertical control were done in the office.

10. The CAD Team completed approximately 95% of their work utilizing NAD 83 and NAVD 88 datum. The NAVD 88 datum elevations will be revised to reference the stamped elevation of BM No. “205-RC”.

11. CAD drawings were developed using a scale of 1 inch = 50 feet.

12. Initial delivery of the CAD file is in Civil 3d format and includes a TIN file for Design Team to utilize.

13. Temporary bench marks (TBMs) were set near the DHS border fence/wall at approximate 400 Ft. intervals.

14. Underground utility spotting was completed and determined to fall outside the limits of this project. The Survey Team found evidence of lines under the bridge which appear to be communication related. From the northeast side of the bridge, the lines route along the bridge in a southwesterly direction on the bottom concrete beam; from the northeast side of the bridge, the communication lines go underground and appear to route in a northeasterly direction towards Elizabeth Street.

15. Underground utility mapping information was obtained from the City of Brownsville and from the Brownsville Public Utilities Board; however, most may be irrelevant to the Survey Site except for the area of Gatewell Structure No 205 and the area near the end of the project that shows a Concrete Box with an 8” PVC pipe outlet towards the river and a 24” RCP towards the landside.

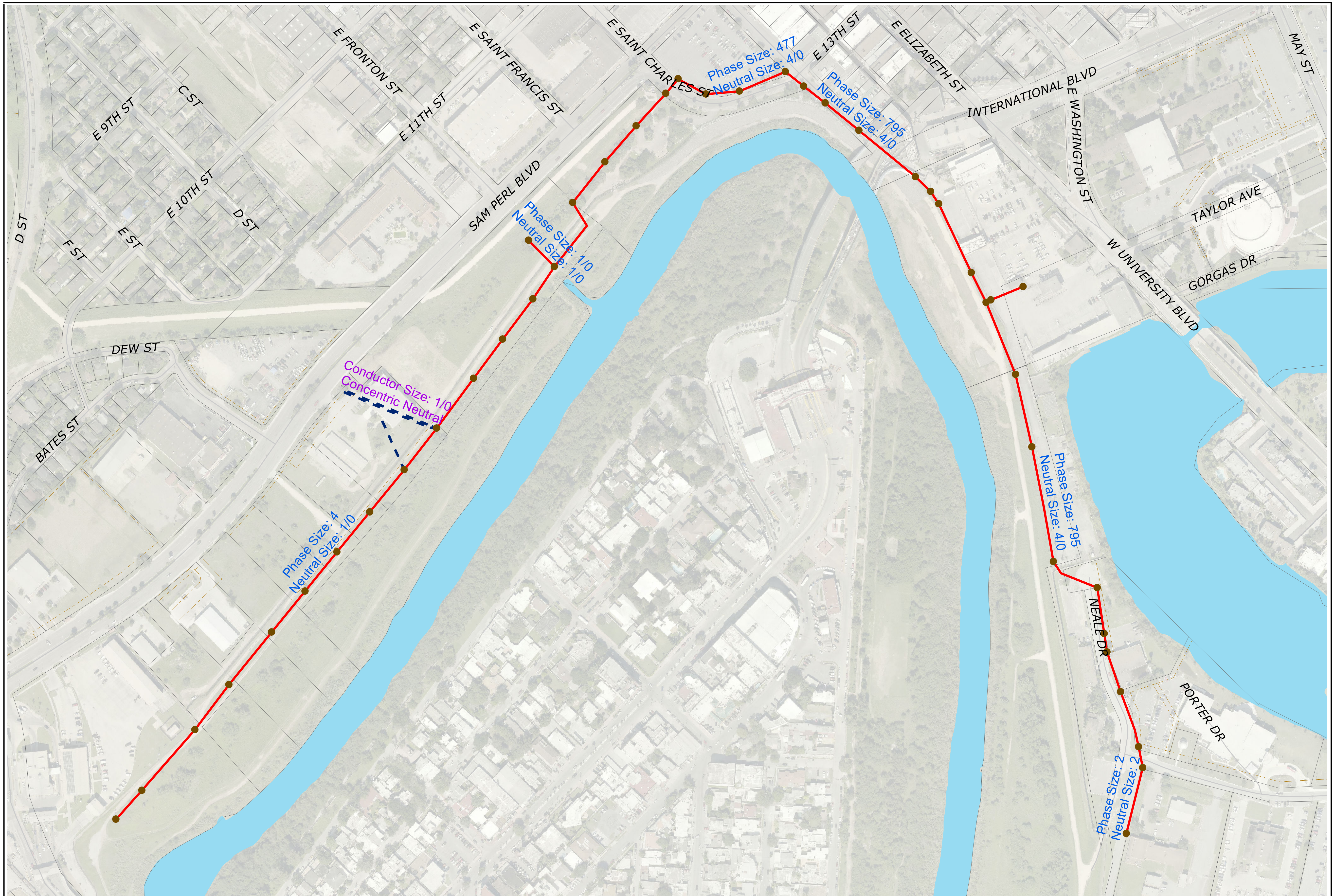
Prepared by:

Carlos Aguilar, R.P.L.S. No 4997
Munoz & Dannenbaum JV

Checked by:

R.T. Brown, R.P.L.S. No. 3881
Munoz & Dannenbaum JV

BROWNSVILLE UTILITY ELECTRICAL IDENTIFICATION



BROWNSVILLE PUBLIC UTILITIES BOARD

N
W E
S

1 inch = 300 feet

Legend

- EXISTING Pole
- UG Primary
- OH Primary
- PARCELS
- EASEMENTS

BROWNSVILLE
PUBLIC UTILITIES BOARD

March 26, 2019 VIA EMAIL JOSEM@gmes.biz

Attn: Jose Luis Muniz
Guzman & Muñoz Engineering
2020 E. Expressway 83
Mercedes, Texas 78570

RE: Response to Request for Information GPFY18-19:50

Dear Mr. Muniz:

This letter is in response to your request for information received March 21, 2019. Enclosed please find documents responsive to your request.

Please be advised that the Brownsville Public Utilities Board (BPUB) Water & Wastewater Engineering Department has indicated that BPUB records do not show any water line utility south of the levee as per the enclosed GIS aerial exhibit.

In regards to your request for the electrical R.O.W. information, you may consider contacting the county.

If you have any questions or need additional information, you may contact me directly at (956) 983-6262.

Respectfully,

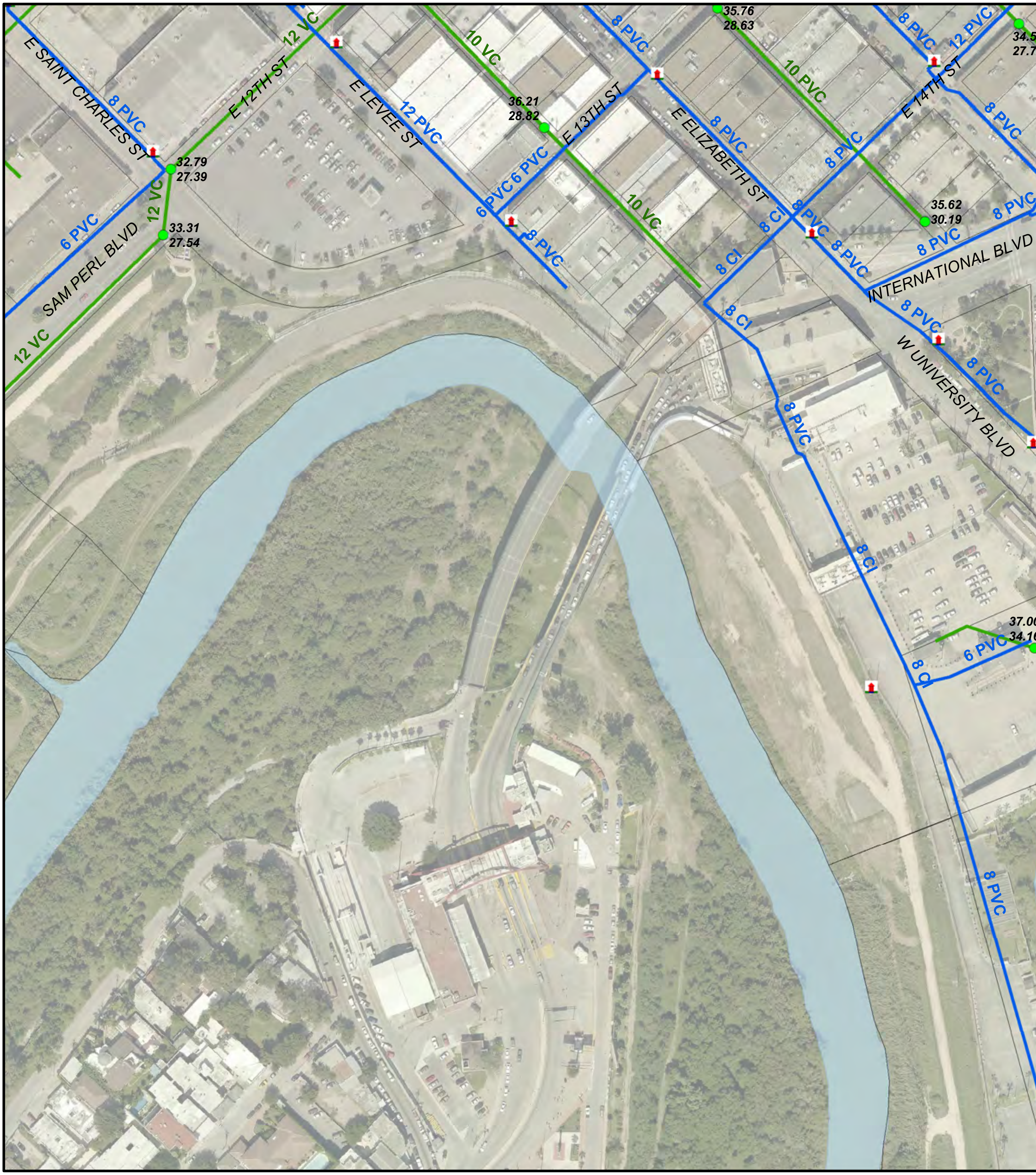
Nancy Tello

Nancy Tello
Enterprise Content, Records, and Policies Manager
for Brownsville PUB

Enclosures

c: File

Hope Park - Rio Grande River, Brownsville TX



Legend
- Water Hydrant
- Water Mainline
- Sewer Manhole
- Sewer Lift Station
- Sewer Force Main
- Sewer Gravity Main

N S W E

1 inch = 200 feet

BROWNSVILLE PUBLIC UTILITIES BOARD

Date: 3/18/2019 Drawn by: LGM

DISCLAIMER - PUBLIC UTILITIES OF BROWNSVILLE

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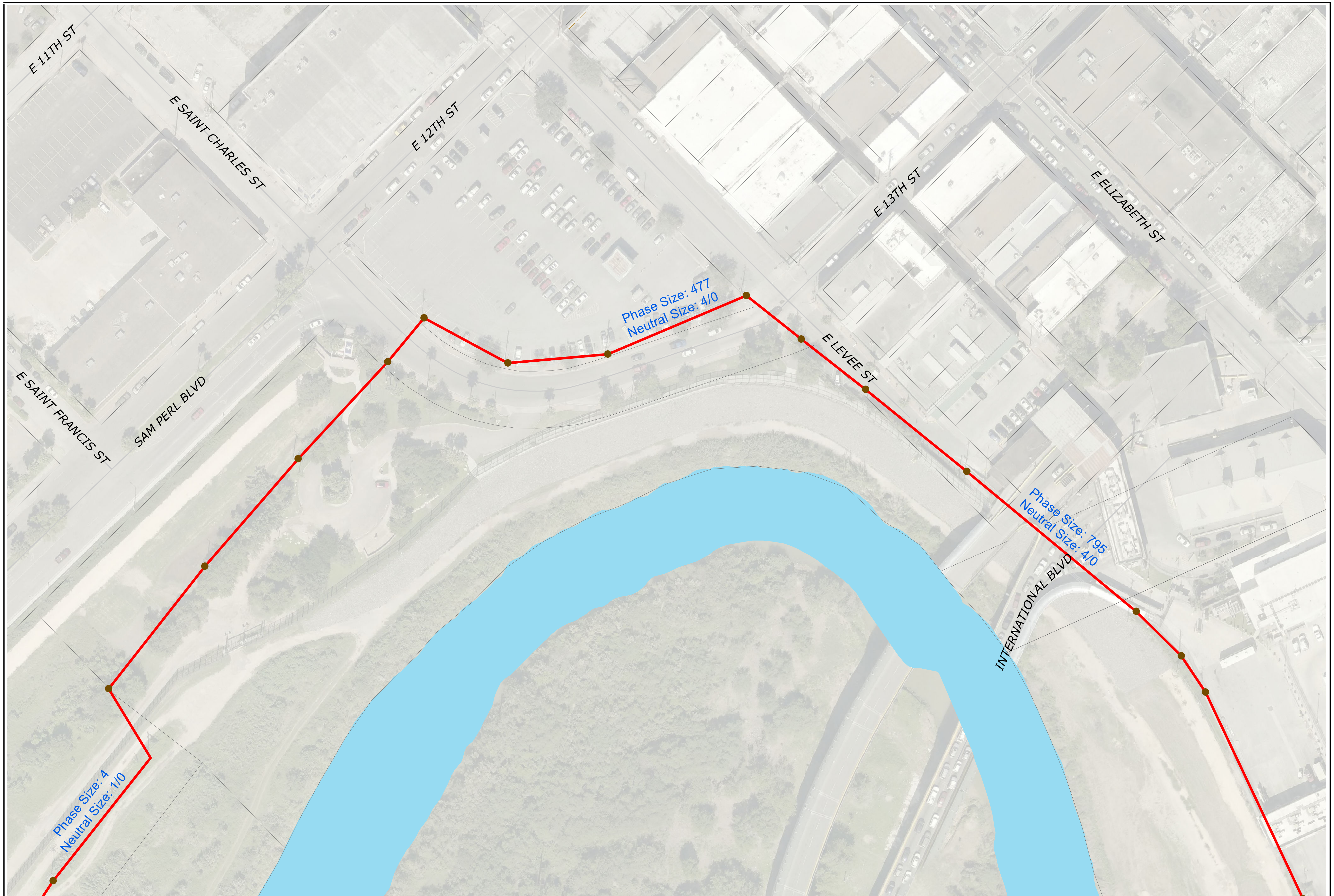
THIS MAP IS FOR REFERENCE ONLY

BROWNSVILLE LEVEE UTILITY ELECTRICAL IDENTIFICATION



BROWNSVILLE
PUBLIC UTILITIES BOARD

N
W E
S

1 inch = 100 feet

Legend

- EXISTING Pole
- OH Primary
- PARCELS
- EASEMENTS
- RESACAS/RIVER
- STREET NAMES

ATTACHMENT B

GEOTECHNICAL REPORT

REMEDIATION OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GEOTECHNICAL REPORT
CONTRACT NO. IBM15D0001 – TASK ORDER IBM15T0015

August 20, 2021

Remediation of Levee Floodplain Failure within the Upper Brownsville Levee Reach Lower Rio Grande Flood Control Project

Geotechnical Report

Contract No. IBM15D0001

Task Order IBM15T0015

Prepared for:

International Boundary and Water Commission, U.S. Section

4171 North Mesa Street, Suite 100C

El Paso, Texas 79902

Prepared by:

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Our Ref.:

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Date:

August 13, 2021

John Wildman
Geotechnical Specialist

Armando Flores
Geotechnical Specialist

Kirk Lowery, P.E.
Principal Geotechnical Engineer

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APPENDICES

1 INTRODUCTION

Arcadis U.S., Inc. (Arcadis) was initially contracted by the United States International Boundary and Water Commission (IBWC) to evaluate and investigate proposed remediation of levee slope failure in the Upper Brownsville Levee Reach of the Rio Grande near the Gateway International Bridge in Brownsville, Texas. The slope remediation, identified as Alternative I, was proposed by the United States Army Corps of Engineers (USACE) in its July 2015 *Geotechnical Evaluation of the Brownsville Levee Cracking and Partial Slope Failure (USACE Report)*. Upon reviewing the *USACE Report* and recommendations, and monitoring inclinometer recordings, Arcadis recognized that movement of soils beneath the levee was occurring deeper than previously analyzed by the USACE. Arcadis' slope stability analyses indicated that the Alternative I remediation would not increase the slope stability safety factor to an acceptable level. Arcadis presented these conclusions and supporting documentation in the *Geotechnical Assessment Memorandum 60 Percent Submittal* dated February 19, 2016 (*60% Memorandum*), see Appendix A.

After presenting these results in a meeting on March 17, 2016, Arcadis was tasked with performing an additional geotechnical investigation and evaluation of other alternatives to mitigate the slope movement. Arcadis's geotechnical investigation included the sampling of four borings, geotechnical laboratory testing on select samples, installation and monitoring of four additional inclinometers in the borings, and the performance of a Pile Integrity Test on the existing buried bulkhead. Four separate alternatives were initially analyzed to identify remediation techniques to mitigate the slope movement:

- Move the levee farther away from the Rio Grande into the Customs Protection and Border Patrol (CPBP) parking lot;
- Install deep soil/cement mix columns through the failure plane;
- Install stone columns through the failure plane; and
- Remove the existing levee and replace with a concrete wall.

Arcadis presented the results of this additional geotechnical investigation and evaluation to IBWC as the *Final Geotechnical Assessment Report* (*Assessment Report*), see Appendix B, submitted on July 31, 2017, and IBWC has since selected a preferred remediation alternative that will reinforce the levee between Stations (STA) 1898+00 to 1904+25 with deep-mixed shear panels to arrest displacement of the levee slopes and stone columns to stabilize near river slopes. This report presents design and construction recommendations for installation of slope failure mitigation mechanisms to IBWC. **The key result of the analyses newly presented in this report is a reduction of the lateral extents of deep-mixed shear panel installation**. Arcadis understands the design and layout presented in this report could be installed, and they are the basis to estimate construction costs for this project. However, Arcadis expects the information provided in this report will be the basis to allow specialty ground improvement contractors to provide a less costly, proprietary remediation design and cost estimate. A separate report will be submitted to IBWC with detailed cost estimates of the design recommendations provided therein.

1.1 Project Location

The project site, shown on Figure 1, is on the left bank of the Rio Grande downstream of the Gateway International Bridge, adjacent to the CPBP facility that is currently not in use.

1.2 Project Background

The project reach lies on a historical cut bank of the Rio Grande, a reach that has experienced significant changes to channel alignment due to river flow and floodplain deposition over time (USACE 2015). The levees were originally built by the local government in the early 1900s in response to historical flooding of the Rio Grande Valley. Flooding continued into the 1930s with more than 20 floods pushing the river out of banks, including floods from three hurricanes. The International Boundary Commission assumed responsibility of the levee system in 1932 and implemented a major rehabilitation program that lasted into the 1940s to update them to be consistent with the standard USACE levee section of less than 25 feet in height with at least 2H:1V side slopes. To provide 100-year flood protection for Federal Emergency Management Agency (FEMA) certification between the Donna Pump and Brownsville, an approximate 3-foot raise was designed for the levee system. In the project reach, the existing levee was at the approximate required grade but was widened in accordance with the design. The levee fill was completed in October 2013, and the reach was sent to FEMA for accreditation on January 12, 2016.

In 2014, surficial cracking was observed on the levee crest and toe, approximately between levee STA 1898+00 to 1904+00, see Figure 2. Subsequently, three inclinometers, I-32, I-33, and I-34 as shown on Figure 1, were installed at STA 1900+13, near the center of the failing soil mass. The USACE identified and evaluated a failure surface with base elevations (EL) between 5 and 15 feet (NAVD88)[1] based on soil characteristics exhibited in cone penetrometer tests (CPTs). Alternatives to mitigate the slope movement were provided for this shallow failure surface. As documented in Arcadis' *60% Memorandum*:

- Arcadis determined from measurements in inclinometers I-32, I-33, and I-34 that the failure surface occurred at the alluvium and Pleistocene interface, deeper than the previously analyzed shallow failure considered by the USACE. The deep failure surface was detected at an approximate elevation of -10 feet.
- A tension or separation crack was noted in front of the buried bulkhead, at approximate STA range 1899+50 to 1900+85.
- Just north of the bulkhead, cracking of the retaining wall in the CPBP parking area was noted.

2 PREVIOUS FIELD INVESTIGATIONS AND LABORATORY EVALUATIONS

While it is not the intention of this report to detail specifics of the field explorations and laboratory evaluations completed by the USACE, Arcadis, and others, because previously submitted reports present these details, results of the field and laboratory programs will be of particular interest to ground improvement contractors who may want to bid on the remediation project. Accordingly, boring logs have been included as Appendix C and are organized such that Arcadis boring logs are followed by USACE inclinometer installation logs and 2008 Raba Kistner logs. Similarly, Appendix D contains USACE CPT soundings, profiles, and predicted strength profiles. Tables 1 and 2 present summaries of pertinent completed borings and CPTs within the project reach, respectively.

[1]All elevations specified in reference to the North American Vertical Datum of 1988 (NAVD88)

Table 1. Summary of Pertinent Borings Completed Within Project Reach

Boring ID	Completed By	Easting (ft)	Northing (ft)	Estimated Elevation (ft)	Total Depth (ft)	Depth to Groundwater (ft)
B-1	Arcadis	1314115.4	16489693.5	40.8	100	24.5
B-2	Arcadis	1314074.6	16489682.1	30.0	80	12.5
B-3	Arcadis	1314010.8	16489662.8	28.0	80	17.2
B-4	Arcadis	1313942.7	16489838.7	25.9	80	23
BRN-P3-31	USACE	1314012.2	16489717.8	27.7	61.5	Not Measured
BRN-P3-32	USACE	1314160.8	16489594.1	40.2	80	Not Measured
BRN-P3-33	USACE	1314120.5	16489576.2	30.3	70	Not Measured
BRN-P3-34	USACE	1314034.7	16489543.4	24.2	60	Not Measured
BRN-P3-35	USACE	1314160.4	16489405.8	27.4	70	Not Measured
BRN-P3-36	USACE	1314159.7	16489314.1	24.1	60	Not Measured
DP-202	Raba Kistner	1314225.6	16489339.9	30.6	50	22

Table 2. Summary of USACE Cone Penetrometer Tests Completed Within Project Reach

CPT ID	Easting (ft)	Northing (ft)	Elevation (ft)	Total Depth (ft)
BRN-P1-01C	1314445.6	16489194.5	34.5	50.7
BRN-P1-02C	1314329.0	16489186.8	41.3	50.7
BRN-P1-03C	1314263.5	16489177.3	29.9	10.5
BRN-P1-04C	1314322.0	16489396.7	35.6	61.8
BRN-P1-05C	1314214.4	16489384.4	31.5	70.4
BRN-P1-06C	1314130.4	16489345.2	24.0	9.2
BRN-P1-07C	1314212.7	16489583.2	36.5	70.5
BRN-P1-08C	1314117.9	16489557.7	30.4	70.4
BRN-P1-09C	1314048.6	16489529.7	24.0	6.4
BRN-P1-10C	1314112.1	16489801.0	37.9	63.8
BRN-P1-11C	1314016.0	16489758.2	27.9	60.2
BRN-P1-12C	1313977.4	16489721.4	26.2	9.7

CPT ID	Easting (ft)	Northing (ft)	Elevation (ft)	Total Depth (ft)
BRN-P1-13C	1313668.8	16490042.6	28.5	12.6
BRN-P1-31C	1314005.1	16489622.4	23.9	6.1
BRN-P2-14C	1313959.2	16489836.7	26.1	8.5
BRN-P2-15C	1314068.6	16489644.9	29.9	51.2
BRN-P2-16C	1314085.1	16489595.3	30.3	50.4
BRN-P2-17C	1314100.7	16489554.7	30.4	50.5
BRN-P2-18C	1314139.8	16489514.8	30.5	50.9
BRN-P2-19C	1314166.9	16489470.3	30.6	70.4
BRN-P2-20C	1314186.3	16489365.3	30.7	66.9
BRN-P2-21C	1314209.4	16489288.7	28.8	6.9
BRN-P2-21C-a	1314209.4	16489288.7	28.8	5.9
BRN-P2-22C	1314233.1	16489237.5	28.5	9.2
BRN-P2-23C	1314344.0	16488789.9	36.3	53.8
BRN-P2-24C	1314388.9	16488798.1	41.8	63.7
BRN-P2-25C	1314252.4	16489409.2	40.7	62.3
BRN-P2-26C	1314179.4	16489591.3	40.4	69.4
BRN-P2-27C	1314165.2	16489584.4	40.6	73.5
BRN-P2-28C	1314138.7	16489571.1	31.5	50.9
BRN-P2-29C	1314088.2	16489548.2	30.3	50.7

In general, the strata encountered during field explorations were fine-grained, medium stiff to stiff lean and fat clay levee soils and historical fill underlain by soft to medium stiff alluvial silt and clay, underlain by stiff Pleistocene clay. Select soil samples taken from the boreholes summarized in Table 1 were analyzed in the laboratory for index and strength properties. Arcadis assigned laboratory tests for percent passing the No. 200 sieve (ASTM D1140), moisture content (ASTM D2216), and Atterberg limits (ASTM D4318). Additionally, soil strength testing of selected individual samples of the predominantly fine-grained soils included unconfined compression (ASTM D2166) and unconsolidated, undrained triaxial compression (ASTM D2850) tests to evaluate short-term shear strength. Consolidated, undrained triaxial compression tests (ASTM D4757) were performed on consecutive samples to determine the long-term shear strength characteristics.

USACE laboratory tests included Atterberg limits (ASTM D4318), grain size analysis (ASTM D422), Unified Soil Classification System (USCS) classification of soils (ASTM D2487), moisture content

(ASTM D2216), controlled expansion consolidation (USACE EM 1110-2-1906 Appendix VIII), direct shear (USACE EM 1110-2-1906, Appendix IX), and unconsolidated, undrained triaxial compression (ASTM D2850). USACE laboratory summaries and data follow Arcadis laboratory data sheets in Appendix E. Table 3 summarizes laboratory tests completed for both the Arcadis and USACE investigations.

Table 3. Summary of Laboratory Testing Completed for Arcadis and USACE Geotechnical Investigations

Test Type	Test Standard	Arcadis	USACE
Moisture content	ASTM D2216	✓	✓
Percent passing No. 200 sieve	ASTM D1140	✓	
Atterberg limits	ASTM D4318	✓	✓
USCS classification of soils	ASTM D2487		✓
Grain size analysis	ASTM D422		✓
Unconfined compressive strength	ASTM D2166	✓	
Unconsolidated, undrained triaxial compression	ASTM D2850	✓	✓
Consolidated, undrained triaxial compression	ASTM D4757	✓	
Controlled expansion consolidation	USACE EM 1110-2-1906		✓
Direct shear	USACE EM 1110-2-1906		✓

3 SUMMARIES OF PREVIOUS SEEPAGE AND STABILITY ANALYSES

This portion of the report summarizes the previous seepage and slope stability analyses completed by the USACE and Arcadis. Accordingly, it presents a synopsis of the methodology, cross-sections considered, material properties, and results of analyses. This section of the report is not intended to be a stand-alone representation of site-specific conditions influencing slope failure such as geologic history, site construction history, river stage, and hydrogeologic conditions. These factors influence the stability of the failing soil mass and have been detailed in previous reports. Similarly, both the USACE and Arcadis provide detailed discussions of causes for slope instability that are not detailed herein. For a complete discussion of the causes of instability and influencing factors, refer to the previous reports, *USACE Report, 60% Memorandum, and Assessment Report*.

Both the USACE and Arcadis used the GeoStudio suite modeling software by Geo-Slope International to evaluate seepage and slope stability. Both entities analyzed groundwater conditions using SEEP/W©, a two-dimensional finite element program commonly used to model unconfined and confined seepage issues, including groundwater movement and pore water distribution within porous materials such as soil and rock. Similarly, both the USACE and Arcadis used Spencer's method of force and moment equilibrium within SLOPE/W©, a two-dimensional limit equilibrium program used to evaluate slope stability, to compute the theoretical factor of safety (FOS) against slope failure. FOS is the ratio of forces contributing to slope failure to the forces resisting slope failure. USACE Engineering Manual (EM) 1110-2-1902 *Slope Stability* provides FOS acceptance criteria for specific embankment conditions.

3.1 USACE Analyses – July 2015 – STA 1898+43, STA 1900+13, and STA 1902+28.5

3.1.1 Seepage Analyses

The USACE evaluated geologic sections at STA 1898+43, STA 1900+13, and STA 1902+28.5 while considering steady-state conditions for two scenarios – high river stage with water surface elevation (WSE) at EL 14.31 feet and low river stage with WSE at EL 7.77 feet, based on available river gauge data. The USACE also performed a transient and a sudden drawdown analysis for each section. Hydraulic conductivity values for saturated soil and conductivity anisotropy ratios were based on values reported in a 2011 Tetra Tech report, with minor adjustments, as reported in Section 6.4 of the *USACE Report*. Hydraulic conductivity properties are summarized in Table 4, while the soil-water characteristic curves used in the transient are detailed in the *USACE Report*. Seepage analyses established groundwater conditions used in subsequent slope stability analyses.

Table 4. Summary of Hydraulic Conductivity Values for Saturated Soil Used in USACE Seepage Modeling[2]

Material	Hydraulic Conductivity, K_{sat} (ft/s)	Porosity, n	Volume Compressibility, m_v (1/psf)	Anisotropy Ratio
CH Pleistocene	3.30×10^{-8}	0.44	3.60×10^{-6}	0.2
CH Holocene	3.30×10^{-8}	0.43	2.50×10^{-6}	0.2
SM	3.30×10^{-7}	0.3	5.00×10^{-6}	0.2
ML	1.00×10^{-7}	0.43	1.00×10^{-5}	0.2
2012 Levee Fill	3.30×10^{-8}	0.4	3.74×10^{-6}	0.2
Levee Fill	3.30×10^{-8}	0.4	3.74×10^{-6}	0.2
Historic Fill	3.30×10^{-8}	0.4	3.74×10^{-6}	0.2
Soft ML	1.00×10^{-7}	0.45	1.00×10^{-5}	1

3.1.2 Slope Stability Analyses

Four scenarios of groundwater conditions were considered by the USACE for the levee slope stability analyses including steady state with WSE at EL 14.31 feet, steady state with WSE at EL 7.77 feet, transient analysis based on river hydrograph, and rapid/sudden drawdown from WSE at 14.33 feet to 7.77 feet.

[2]Table 4 replicated from Table 6.2 of *USACE Report*.

Shear strengths used in the slope stability models, summarized in Table 5, were generally taken from the 2011 Raba Kistner analyses, which were prepared for the 2011 Tetra Tech design. Shear strengths for soft silt and historical fill, however, were estimated by the USACE based on CPT, standard penetration tests (SPT), and other lab testing results. Table 6 summarizes slope failure FOS for each section and groundwater condition analyzed by the USACE. Plates from the *USACE Report* showing seepage and slope stability models are included as Appendix F. The approximate failure surfaces analyzed by the USACE are represented by the green line presented on Figure 3.

Table 5. Summary of Shear Strength Properties Used in USACE Slope Stability Modeling[3]

Material	Unit Weight (pcf)	Cohesion, c (psf)	Effective Cohesion, c' (psf)	Internal Angle of Friction, ϕ (°)	Effective Internal Angle of Friction, ϕ' (°)
CH Pleistocene	121.98	2320	200	0	24
CH Holocene	123.37	400	800	0	17.3
SM	117	0	0	32	32
ML	119.38	0	300	29	32.6
2012 Levee Fill	127.34	5000	620	0	29.2
Levee Fill	127.34	5000	620	0	29.2
Historic Fill	127.34	400	200	15	24
Soft ML	125.98	200	200	0	0

Table 6. Summary of USACE's Lowest Slope Stability Factors of Safety[4]

Section	Steady-State, WSE EL 7.77 feet	Steady-State, WSE EL 14.31 feet	Rapid Drawdown	Transient Using Hydrograph
1898+43	1.11	1.10	1.06	1.10
1900+13	1.26	1.10	1.00	1.02
1902+28.5	1.20	1.17	1.17	1.12

3.1.3 Conclusions

As evident in Table 6, the USACE recognized that rapid drawdown conditions at STA 1903+13 represented the most critical section analysed, exhibiting an FOS of 1.00, and, as shown in the slope

[3]Table 5 replicated from Table 6.1 of *USACE Report.*

[4]Table 6 replicated from Table 6.3 of *USACE Report.*

stability plates included as Appendix F, the USACE concluded that the bottom extent of the critical failure block ranges from approximate EL +5 feet to +10 feet. In Section 7.5, Inclinometer Data, of the *USACE Report*, however, the USACE recognizes deflection of approximately 0.5 inch near the upper extent of the Pleistocene clay in the existing inclinometers near STA 1900+13; the USACE evaluated this deflection as a simple pinch point resultant of deformation in the overlying soils while qualifying the inclinometer data as preliminary. This deflection near the Pleistocene interface is approximately 15 to 20 feet deeper than the USACE-estimated failure surface near EL +5 to +10 feet, as shown on Figure 3. Ultimately the USACE recommended a series of remediation alternatives, including Alternative I, re-grading the slope, and installing riprap protection at the toe.

3.2 Arcadis Analyses – February 2016 – STA 1900+13

Analyses completed in the *60% Memorandum* evaluated the USACE Alternative I mitigation alternative, which proposed to re-grade the riverbank slope to 5H:1V to arrest the failing soil mass. For a complete discussion of the analyses, see the full *60% Memorandum*.

3.2.1 Seepage Analyses

To consider best-case and worst-case groundwater conditions, Arcadis evaluated steady-state seepage considering a WSE of 7.77 feet and evaluated rapid drawdown from WSE at 14.31 feet to 7.77 feet. The hydraulic conductivity properties used in USACE seepage analyses were used in Arcadis seepage analyses, and the seepage models developed provided groundwater conditions considered in subsequent slope stability models.

3.2.2 Slope Stability Analyses

Arcadis began its evaluation of the USACE Alternative I remediation by initiating a parametric sensitivity analysis of shear strength properties used in the USACE slope stability analyses. However, during a site visit in December 2015, Arcadis measured deflection in the three USACE-installed inclinometers near STA 1900+13, and each inclinometer exhibited riverward deflection greater than 1.5 inches as compared to measurements recorded approximately a year earlier by the USACE. Inclinometer I-32 exhibited a deflection of 1.5 to 2 inches from approximately 32 feet to 36 feet beneath the levee crest, at approximate EL +4 feet. Inclinometer I-33 exhibited a deflection greater than 2.5 inches at a depth of approximately 40 feet below the levee toe. The inclinometer probe could not be advanced past a depth of 40.5 feet, at approximate EL -9 feet, indicating the inclinometer casing had buckled or collapsed at that depth. Inclinometer I-34, near the river slope crest, exhibited a deflection of approximately 1.5 to 2 inches between 32 and 34 feet below grade, at approximate EL -8 feet.

Deflection of inclinometers at depths substantially deeper than the failure surface estimated by the USACE caused Arcadis to analyze a deep failure surface near the alluvium/Pleistocene interface, represented by the red line or deep failure surface on Figure 3. However, considering the deep failure surface and USACE recommended shear strength properties, slope stability models indicated a stable, non-failing slope. Accordingly, Arcadis completed a back-calculated, parametric sensitivity analysis of shear strength properties at STA 1900+13 to find shear strength properties that yield an FOS of approximately 1.0 for the deep failure surface.

Three combinations of shear strength properties were developed and evaluated for steady-state seepage conditions. Combination 1 considered effective internal friction angle of all silt present in the section to be equal to zero while shear strength properties of other materials were varied to yield an FOS along the deep failure plane equal to approximately 1.0. Combination 2 considered cohesion of all materials to equal zero and effective internal friction angle of each material was adjusted to yield an FOS of approximately 1.0. Combination 3 considered effective cohesion of material modeled as silt to equal zero while other properties were varied to yield an FOS of approximately 1.0. As a note, material modeled as "soft silt" was differentiated from silt for this analysis.

A fourth combination of shear strength properties was developed and evaluated for rapid drawdown conditions. Table 7 summarizes the back-calculated shear strength properties for steady-state combinations while Table 8 summarizes back-calculated rapid drawdown conditions. Table 9 summarizes slope FOS considering back-calculated shear strength properties for the deep failure surface evident in deflected inclinometers.

Table 7. Summary of Back-Calculated Shear Strength Property Combinations That Yield Factors of Safety Equal to Approximately 1.0 for the Deep Failure Surface Near STA 1900+13 (Combinations 1 Through 3)

Material	Unit Weight (pcf)	Combination 1		Combination 2		Combination 3	
		c' (psf)	ϕ' (°)	c' (psf)	ϕ' (°)	c' (psf)	ϕ' (°)
CH Pleistocene	121.98	200	12	0	10	90	12
CH Holocene	123.37	460	13	0	10	225	12
SM	117	0	32	0	32	0	32
ML	119.38	230	0	0	11	0	8
2012 Levee Fill	127.34	300	12	0	11	105	12
Levee Fill	127.34	300	12	0	11	105	12
Historic Fill	127.34	200	24	0	11	95	12
Soft ML	125.98	150	0	0	8	150	0

Table 8. Summary of Back-Calculated Shear Strength Properties Used in Arcadis Rapid Drawdown Slope Stability Modeling (Combination 4)

Material	Unit Weight (pcf)	Cohesion, c (psf)	Effective Cohesion, c' (psf)	Internal Angle of Friction, ϕ (°)	Effective Internal Angle of Friction, ϕ' (°)
CH Pleistocene	121.98	2320	150	0	24
CH Holocene	123.37	400	200	0	17.3
SM	117	0	0	32	32

Material	Unit Weight (pcf)	Cohesion, c (psf)	Effective Cohesion, c' (psf)	Internal Angle of Friction, ϕ (°)	Effective Internal Angle of Friction, ϕ' (°)
ML	119.38	0	190	29	32.6
2012 Levee Fill	127.34	5000	400	0	29.2
Levee Fill	127.34	5000	400	0	29.2
Historic Fill	127.34	400	200	15	24
Soft ML	125.98	200	150	0	0

Table 9. Summary of Slope Factors of Safety of Unmitigated Slope Considering Back-Calculated Shear Strength Properties and Deep Failure Surface at STA 1900+13

	Shear Strength Combination 1	Shear Strength Combination 2	Shear Strength Combination 3	Shear Strength Combination 4
Steady-State, WSE EL 7.77 feet	1.10	1.10	1.10	
Rapid Drawdown				1.04

After defining appropriate, back-calculated shear strength properties of earthen material within the section near STA 1900+13, Arcadis evaluated the USACE Alternative I mitigation alternative which recommends re-grading the riverbank slope to a 5H:1V slope ratio. Table 10 presents a summary of the results of the Alternative I evaluation. Appendix G presents both unmitigated and re-graded slope stability models referenced in this section of the report.

Table 10. Summary of Slope Factors of Safety of Re-Graded 5H:1V Slope (USACE Alternative I) Considering Back-Calculated Shear Strength Properties and Deep Failure Surface at STA 1900+13

	Shear Strength Combination 1	Shear Strength Combination 2	Shear Strength Combination 3	Shear Strength Combination 4
Steady-State, WSE EL 7.77 feet	1.06	0.92	0.96	
Rapid Drawdown				1.01

3.2.3 Conclusions

Based on the depth of deflection of inclinometers installed near STA 1900+13, Arcadis recognized the depth of slope failure was substantially deeper than considered in the *USACE Report*. By identifying the deep failure surface, Arcadis was able to fully define the failure surface within the slope stability models presented in the *60% Memorandum* and back-calculate representative shear strength properties for materials present in the section at STA 1900+13. Then, considering the re-graded geometry of the USACE Alternative I mitigation alternative, Arcadis recognized that re-grading the levee slope to 5H:1V does not only fail to adequately mitigate the deep failure surface but it also increases the propensity for failure by reducing resisting forces in the neutral block and subsequently reducing FOS for all shear strength combinations and groundwater conditions evaluated.

3.3 Arcadis Analyses – July 2017 – STA 1899+15 and STA 1900+13

In the Arcadis *Assessment Report*, the evaluation of slope remediation alternatives to mitigate slope failure along the deep failure plane was described. These analyses included additional field investigation, installation of additional inclinometers, and the analyses summarized below. For a complete discussion of the analyses, see the full *Assessment Report* in Appendix B.

3.3.1 Seepage Analyses

Like the seepage analyses completed for the *60% Memorandum*, Arcadis evaluated steady-state seepage considering a WSE of 7.77 feet and evaluated rapid drawdown from WSE at 14.31 feet to 7.77 feet. The hydraulic conductivity properties used in USACE seepage analyses were used in Arcadis seepage analyses, and the seepage models developed provided groundwater conditions considered in subsequent slope stability models.

3.3.2 Slope Stability Analyses

As part of the geotechnical investigation, the shear strength testing presented in Appendix E was reviewed to verify the shear strength values interpreted in the *60% Memorandum*. The laboratory tests resulted in shear strengths that are significantly higher than the shear strengths that were back-calculated for the *60% Memorandum*. Because these laboratory values were not considered truly representative of the actual field conditions, the series of analyses described below used the back-calculated shear strength properties determined in the *60% Memorandum* and summarized in Tables 9 and 10.

Arcadis submitted a technical memorandum on June 13, 2016, showing that moving the levee into the CPBP parking lot would not meet acceptable slope stability FOS. As part of the analysis presented in the *60% Memorandum*, the levee was cut down to EL 31 feet and the lowest slope FOS for the steady-state seepage conditions was 1.3. As specified in Table 6-1b in USACE EM 1110-2-1913, *Design and Construction of Levees*, the required FOS for existing levees considering steady-state seepage is 1.4. Because the calculated FOS for steady-state seepage considering a cut levee did not meet design criteria, the installation of a T-wall was not considered a viable alternative due to insufficient global stability. Installation of deep-mixed shear panels or stone columns through the failure surface is a viable alternative and was analyzed. Similarly, a combination of stone columns near the river and deep-mixed shear panels near the levee could arrest the slope movement.

3.3.3 STA 1899+15 Levee Failure Plane

Prior to initiating slope mitigation design, Arcadis evaluated an unimproved levee section at STA 1899+15 to determine if this section, considering the back-calculated shear strength properties, could be more critical than the previously analyzed section at STA 1900+13. The section at STA 1899+15 (Figure 4), was developed using Arcadis' borings B-1, B-2, and B-3. Table 11 compares the slope stability FOS promulgating in the levee, represented by the red line or the deep failure surface on Figure 3, using existing conditions for the sections at STA 1899+15 and STA 1900+13. Appendices G and H present the slope analysis models for STA 1900+13 and STA 1899+15, respectively.

Table 11. Comparison of Slope Stability Factors of Safety for Existing Soil Conditions at STA 1899+15 and STA 1900+13

Analyzed Condition	STA 1899+15	STA 1903+13
Steady State – Shear Combination 1	1.11	1.10
Steady State – Shear Combination 2	1.10	1.10
Steady State – Shear Combination 3	1.12	1.10
Rapid Drawdown	0.90(1)	1.04

(1)The FOS for rapid drawdown conditions at STA 1899+15 was misreported in the *Assessment Report* as 1.08.

Due to the misreporting of FOS for rapid drawdown conditions at STA 1899+15, slope mitigation using stone columns and deep soil mix (DSM) shear panels was designed for the section at STA 1900+13.

3.3.4 Slope Failure Plane

Refer to previous reports for descriptions of the failure plane associated with slope failure and surficial cracking in the levee toe, levee crest, and landward of the levee. Because of the deep failure plane, the entire slope on the river-side section of the levee requires stabilization to achieve a suitable FOS. The cracks shown on the east side of Figure 2 is where the slope failure plane daylights in the levee crest, presented in the previous section. This failure plane is represented by the red line or deep failure surface on Figure 3. A riverbank failure is defined as the slope movement that is shown as promulgating from the toe of the levee (see the cracks on the west side of Figure 2), as shown by the purple line or toe failure surface on Figure 3.

3.3.5 Riverbank Failure Plane

Arcadis analyzed a deep failure surface that daylights between the levee toe and riverbank slope. The intent of this analysis was to determine with the back-calculated shear strength values if a failure surface near the riverbank slope would occur. Considering best-case conditions, Steady-State Seepage – Shear Strength Combination 3 soil and water conditions, this specified surface yielded an FOS just below 1 as shown in Appendix I. These models indicate the slopes between the river and the levee toe will need to be mitigated for a deep-seated failure surface.

3.3.6 Slope Failure Mitigation Design

Recognizing the potential for levee and river slope failures, Arcadis initially considered these systems to function independently of one another and designed mitigation alternatives to remediate both the riverbank slope and the failure plane daylighting within the levee. A series of alternatives were presented to IBWC in the *Assessment Report*, but only the selected alternative – DSM shear panels installed beneath the levee and stone columns installed immediately landward of the riverbank slope – will be discussed herein.

3.3.7 DSM Shear Panel Design for the Levee Failure Plane

Deep-mixed shear panels were assessed in general accordance with guidelines outlined by Filz and Templeton's (2011) Design Guide for Levee and Floodwall Stability Using Deep-Mixed Shear Walls (*Design Guide*). This *Design Guide* presents specific, step-by-step methods to assess the stability of deep-mixed shear panels that support levees and floodwalls. However, this *Design Guide* is specific to deep-mixed shear panels that underlie the crest and protected side of levees and exist in a subsurface characterized by perfectly horizontal layering. This differs from the shear panels necessary for the river-side improvement of the IBWC levee in the complexly layered subsurface within the project limits. Accordingly, variations from the methods described in the *Design Guide* were employed to properly assess potential sliding, overturning, crushing, shear, shear on vertical planes, and extrusion failures. The analyses presented in the *Assessment Report* have been updated, and refinements to design are presented in Section 4.3.

3.3.8 Stone Column Design for the Riverbank Failure Plane

Stone columns were assessed in accordance with the recommendations in Appendix B of Stability Analyses of Embankments Founded on Stone Columns by Filz and Navin (2006). These guidelines present three methods for slope stability analysis of slopes supported by stone columns – the circular sliding surface method, the assigned strength property method, and the profile method. It also references Barksdale and Bachus (1983), which presents the same three methods. Both Filz and Navin (2006) and Barksdale and Bachus (1983) recommend the profile method as the preferred method of analysis for slopes supported by stone columns. Accordingly, the profile method was selected for this analysis. Ultimately, the design of stone columns for the predominance of the project reach, STA 1899+00 to 1904+25, remains unchanged from the recommendations detailed in the *Assessment Report*. For this portion of the project, nine rows of 36-inch-diameter stone columns installed 6 feet center-to-center in an equilateral triangle arrangement are required to adequately arrest the riverbank failure plane.

4 UPDATED ANALYSES AND REFINED STONE COLUMN AND DSM SHEAR PANEL CONFIGURATION

Previous analyses examined groundwater conditions and slope stability of specific existing slope sections to identify the most critical section within the project reach, and slope mitigation alternatives were developed to arrest slope failure at the most critical section. These slope failure mitigation alternatives developed for the most critical section were recommended in the *Assessment Report* to be installed throughout the entirety of the project reach. However, because subsurface conditions within the project limits are not uniform, the designed slope improvement mechanisms selected by IBWC – deep-mixed shear panels underlying the levee slope and stone columns installed through the failure plane in the riverbank slope – may not be necessary where existing subsurface conditions are less prone to slope failure than the most critical section.

Accordingly, new analyses have been completed to verify the lateral and vertical extents of ground improvement necessary to effectively and efficiently arrest the slope failure while avoiding installation of ground improvement mechanisms where they are not needed. Specifically, a new section was evaluated near STA 1903+96 to assess conditions toward the southern limit of the project reach. Additionally, isopach maps of the top of Pleistocene clay were developed to validate the depth of recommended stone column and DSM panel installation. Ultimately, the previously designed combination of deep-mixed shear panels beneath the levee and stone columns installed near the river slope is necessary throughout the length of observed surficial cracks, approximate STA 1898+00 to 1904+25, while south of STA 1904+25, where no surface cracks have been observed, only stone columns are necessary to mitigate slope failure.

4.1 Arcadis Analyses – January 2018 – STA 1903+96

4.1.1 Seepage Analyses

Similar to previous analyses, Arcadis evaluated steady-state seepage considering a WSE of 7.77 feet and evaluated rapid drawdown from WSE at 14.31 feet to 7.77 feet. The hydraulic conductivity properties defined in USACE seepage analyses were used in Arcadis seepage analyses, and the seepage models developed provided groundwater conditions considered in subsequent slope stability models. Seepage models at STA 1903+96 are included as Appendix J.

4.1.2 Unimproved Slope Stability Analyses

Similar to analyses presented in Arcadis' *Assessment Report*, back-calculated shear strength properties were used in the slope stability assessments for the section near STA 1903+96. The subsurface profile for this section, see Figure 5, was developed considering USACE CPT soundings BRN-P1-01C and BRN-P1-02C as well as borings BRN-P3-36, BRN-P3-35, and DP-202. Like previous analyses, two potential failure planes were considered – one that promulgates at the levee toe (toe failure surface on Figure 3) and another that daylights within the levee (deep failure surface on Figure 3). As presented in the *Assessment Report*, slope movement has generally appeared after a rapid drawdown event. As such, the Rapid Drawdown Seepage – Shear Strength Combination 4 soil and water conditions were used in the analyses. Appendix K presents slope stability analyses of the unimproved section at STA 1903+96 showing a FOS of 1 along the deep failure surface and less than 1 along the toe failure surface.

4.1.3 Riverbank Failure Surface - Stone Column Analyses

To improve the FOS along the riverbank failure plane, stone columns were modeled in the section at STA 1903+96 as described in Section 3.3.2.2, considering 36-inch stone columns spaced 6 feet apart in an equilateral triangle arrangement. For this section, it was determined that six rows of stone columns will adequately arrest the slope movement along the riverbank failure plane. This is a one-third reduction in stone columns as compared to the nine rows necessary for the section at STA 1900+13. Appendix L presents the stone column analyses at STA 1903+96, and Table 12 presents a summary of the analyses.

4.1.4 Levee Failure Surface

Before evaluating deep-mixed shear panels at STA 1903+96, Arcadis considered the improving effect of stone columns at the toe on the levee failure plane because the stone columns discussed in Section 4.1.3 will intersect the levee failure plane and offer additional resistance against shear failure. Because the failure surface at STA 1903+96 has not been defined by inclinometer deflection, a fully specified slip surface is not considered appropriate for this section. Accordingly, Arcadis used an optimized Entry and Exit analysis within Slope/W® to allow the program to iteratively search for a more critical failure surface. This reduced the FOS along the levee failure surface at STA 1903+96, considering six rows of stone columns near the riverbank slope, to 1.26. USACE EM 1110-2-1902 recommends FOS during rapid drawdown conditions to be 1.1 to 1.3. Because of the geological constraints discussed in Section 4.2, Arcadis considers the six rows of stone columns installed near the riverbank slope to be sufficient to adequately arrest the slope failure and recommends that no deep-mixed shear panels be installed south of STA 1904+25. The stone column analyses considering the levee failure surface at STA 1903+96 are included as Appendix L, and Table 12 presents a summary of the analyses.

Table 12. Summary of Stone Column-Improved Section at STA 1093+96

Failure Surface	Unimproved Conditions	Stone Columns
Toe (Levee Toe)	0.82 (Appendix K)	1.45 (Appendix L)
Deep (Levee Crest)	1.00 (Appendix K)	1.25 (Appendix L)

4.2 Depth of DSM Shear Panel and Stone Column Installation

After completing the analyses at STA 1903+96, Arcadis recognized a correlation between the elevation of the top of Pleistocene clay and unimproved slope FOS at the three sections analysed. In the southernmost section at STA 1903+96, the top of Pleistocene clay (TOP) riverward of the levee ranged from approximate EL -8 to +5 feet, and the unimproved slope FOS, considering rapid drawdown conditions, was 1.00. In the northernmost section at STA 1899+15, the TOP riverward of the levee ranged from approximately, -10 to -12 feet, and unimproved FOS, considering rapid drawdown conditions, was 0.90. Near the middle of the failing soil mass at STA 1900+13, TOP riverward of the levee ranged from approximately -8 to -6 feet, and unimproved slope FOS, considering rapid drawdown conditions, was 1.04. At first glance, it seemed that unimproved slope FOS decreased with decreasing elevation of TOP and that the Pleistocene surface dipped to the north. Considering the implications of these observations, especially when considered in tandem, Arcadis recognized the need to develop an isopach map that estimates elevation of TOP. This map would serve two purposes. First, it would allow Arcadis to observe trends in the spatial distribution of the elevation of the top of Pleistocene to identify a potentially more critical failure surface than previously recognized. Secondly, a TOP isopach map would validate the recommendation from the *Assessment Report* that stone columns and DSM shear panels be installed to EL -15 feet.

4.2.1 Development of Top of Pleistocene Clay Isopach Map

If stone columns and DSM shear panels do not adequately intercept the failure surface, the TOP, the slope failure will not be arrested. Accordingly, the first attempt in generating TOP isopachs was to define lowest-likely elevation of TOP. This included an independent assessment of TOP based on CPT soundings, boring logs, and USACE cross sections. TOP in CPT soundings was identified as the depth below which CPT tip resistance consistently exceeded 160 tons per square foot. This was often associated with a relatively sharp spike in tip resistance. For boring logs, depth to TOP was identified considering lithology, SPT blow counts and/or pocket penetrometer values, and sampling frequency. Following the first assessment of depth to TOP in the boring logs, CPTs, and cross sections, a back check was completed, considering worst-case, deepest-likely depth to TOP in CPTs and logs. Then, the maximum depth to TOP considering the independent review, the depth reported by the USACE, and the deepest-likely back check assessment was subtracted from the existing ground elevation to estimate lowest-likely elevation of the TOP throughout the project limits. The data points were then plotted using ArcMap GIS software, and the TOP was contoured using the natural neighbor subroutine within the 3D Analyst suite of data analyses. From this first TOP isopach map, discrepancies in estimation of TOP were recognized and rectified, and subsequent iterations of the TOP isopach maps demonstrated increasing agreement between data points. Additionally, subsequent iterations of the TOP isopach map considered kriging and inverse density weighted (IDW) data interpolation techniques to contour the TOP isopachs, as well as the natural neighbor. Then, when a map of the observed surficial cracking was overlain on the natural neighbor, kriging, and IDW isopach contour maps, a series of project-wide trends became increasingly evident. Specifically:

- The highest elevations of TOP are in the northern and southern extents of the project reach. These high points of the TOP coincide with northern and southern extents of surficial cracking mapped by the USACE. This is best demonstrated with the natural neighbor and kriging isopach maps presented in Appendix M.
- Surficial cracking between these limiting high points occurs predominantly parallel to the contours of the TOP isopach. This is best demonstrated with the natural neighbor and kriging isopach maps presented in Appendix M. Also of note, the surficial cracks occur parallel to the existing ground surface contours.
- The lowest-most observed TOP is approximate EL -11.5 feet. This is best demonstrated with the natural neighbor and IDW isopach maps presented in Appendix M.

4.2.2 Discussion of Top of Pleistocene Clay Isopach Maps

Development of the top of Pleistocene isopach maps generated increased clarity into the root cause of the slope failure observed within the project limits. While the natural neighbor and IDW maps demonstrate and validate key trends, the map created using kriging interpolation is most useful in recognizing the root cause of slope failure. Kriging is a method that considers the spatial distribution of *all* data points within the data set to create isopachs of the feature surface. This differs from natural neighbor interpolation which is predominantly a linear interpolation between singular data points and from IDW that considers groups of data points but only within a specified radius from the data point of inquiry. Accordingly, the kriging method can often depict large-scale trends that may be obscured by the natural neighbor or IDW methods. In the map produced using kriging interpolation, not only can the trends described in the first and second bullet points described in the previous section be seen, but a pattern in the isopachs that resembles the cut bank of a former meander of the historical Rio Grande is visible. The high points of the TOP contours in the northern and southern portions of the project reach that coincide with the northern and southern extents of surficial cracking define the limits of the historical meander and imply geological constraints limiting the extent of slope failure. Where the cut bank of the historical meander occurs parallel to the existing ground surface, a couple of phenomena are occurring.

First, this is where overburden soils overlying the TOP are underlain by the most steeply dipping TOP surface. In geological terms, this can be conceptualized as the difference between true dip and apparent dip of adjacent strata. Where the strike line (line of constant elevation, i.e., contour line) of adjacent strata is parallel, the angle of inclination (often called the “dip” of the strata) is greatest at 90 degrees from the strike line. Accordingly, the “true dip” of a geologic unit is measured normal to the strike line. If the strike lines of adjacent strata are askew from parallel, the angle of inclination separating the strata is termed as the “apparent dip,” and apparent dip is by definition, and by geometric relation, always less than the true dip. Thus, where TOP isopachs are parallel to existing ground surface contours, overburden soils are acting along the true dip (steepest) plane of the TOP surface.

Second, the cut bank of the historical meander parallel to the existing ground surface is where the greatest frequency of surficial cracks is located.

Conclusions from the development and assessment of the series of TOP isopach maps include:

- The existing levee was constructed on the cut bank of a historical meander of the Rio Grande. The limits of this meander are controlling the limits of the failing soil mass.
- The deepest observed elevation to TOP is approximate EL -11.5 feet. This confirms Arcadis’ recommendation from the *Assessment Report* to install stone columns and DSM shear panels to EL -15 feet.
- The section at STA 1900+13 is considered appropriate for ground improvement design because it represents the most steeply dipping TOP surface and is normal to the observed direction of slope movement. Although the section at STA 1899+15 exhibits an unimproved slope FOS slightly less than the section at STA 1900+13, the geometric configuration of the TOP surface suggests the section at STA 1900+13 demonstrates a more critical failure surface.

4.3 Refinement of DSM Panel Spacing

As part of the verification and validation of the DSM shear panel design, Arcadis used the design calculation spreadsheet presented in the *Assessment Report* to work the examples provided in the *Design Guide*. It was noted that our previous calculations were overly conservative. Specifically, when calculating the weights of the soils in the analyses to check sliding and overturn, the uplift pressure of the groundwater was subtracted from the total unit weight of the soils to produce an effective unit weight. This uplift pressure was also added, basically a second time, within the force and moment calculations. This was incorrect and to rectify the calculations the total unit weight of the soils was used to calculate the weights of the soils instead of the effective unit weight. The results of this correction allowed the center-to-center spacing of the shear panels to increase from 17 feet to 20 feet and still maintain acceptable FOS. The refined DSM shear panel analyses are presented in Appendix N. Table 13 summarizes geometry and strength characteristics for the DSM shear panels while Table 14 summarizes calculated FOS against sliding, overturning, crushing, vertical shearing, and extrusion between panels. To maintain acceptable global factors of safety, the specified unconfined compressive strength of the deep mix zone (DMZ) was increased from 100 pounds per square inch (psi) as presented in the *Assessment Report* to 110 psi.

Table 13. Summary of DSM Shear Panel Geometric and Strength Characteristics

Description	Variable	Value	Units
Average thickness of Deep Mix Zone (DMZ)	H	47.8	foot
Width of DMZ normal to centerline of levee	B	45	foot
Diameter of single deep-mixed column	d	72	Inch
Center-to-center panel spacing	s	20	foot
Column overlap distance	e	1	ft
Specified unconfined compressive strength of deep-mixed soil (DMS)	q_{dm}	110	pounds per square inch
Time between construction and loading	-	28 days	-
Design FOS of DMS strength	f_d	1.5	-
Probability DMZ strength > specified strength	p_{dm}	80%	-
DMS shear strength	s_{dm}	5512	pounds per square foot
DMZ shear strength	s_{dmz}	1435	pounds per square foot

To ensure global stability of the levee and river slope following installation of stabilization mechanisms, Arcadis completed a final check of the proposed system. In this analysis, Arcadis considered:

- The section at STA 1900+13;
- Rapid drawdown conditions;
- Back-calculated shear strength properties of soil units;
- Nine rows of 36-inch-diameter stone columns, spaced 6 feet center-to-center in an equilateral triangle arrangement;
- DSM shear panels installed in accordance with the specifications in Table 13 and that begin 10 feet riverward of the levee crest and extend 45 feet toward the river; and
- Optimized exit and entry analysis within Slope/W® to iteratively search for a more critical failure surface than the fully specified failure surface considered in the *Assessment Report*.

As shown in Table 14, the resulting FOS was 1.48, which exceeds USACE recommendations and confirms that the stone columns and DSM shear panels recommended to be installed from STA 1800+00 to STA 1904+25 will adequately arrest the failing slope. This slope stability model is included as the last page of Appendix N.

Table 14. Summary of DSM Shear Panel Factors of Safety

Mode of Failure	Abbreviation	Required FOS	Calculated FOS
Sliding	F_s	1.3	1.56
Overturning	F_o	1.4	1.40
Crushing	F_c	1.4	1.71
Vertical Shearing	F_v	1.4	1.77
Extrusion	F_e	1.3	>10
Global Stability - Spencer's Method	F_g	1.3	1.48

4.4 DSM Shear Panels and Stone Columns Layout

To summarize the design analyses described in this report, the layout of the DSM shear panels and stone columns are presented on Figure 6. Figures 7 and 8 present sections through the areas where DSM shear panels and stone columns would be installed and where only stone columns would be installed, respectively. Arcadis reiterates the design and layout presented in this report could be installed, and they are the basis to estimate construction costs for this project. However, Arcadis expects the information provided in this report will be the basis to allow specialty ground improvement contractors to provide a less costly, proprietary remediation design and cost estimate.

5 CONSTRUCTION RECOMMENDATIONS

Some earthwork activities will be required to provide level working areas for the crane equipment and spoil areas to dispose/spread out any cuttings. The quantity of cut and fill for the DSM shear panels and the stone columns will require removal and replacement of the levee as well as some incidental cut and fill operations to level the equipment. Additionally, quality assurance (QA) activities are important for ground improvement techniques such as DSM and stone column installation. The following sections provide specific recommendations for the various construction activities.

5.1 Leveling Pad at the Levee

The DSM shear panels design presented in this report shows a portion will be located on the river-side section of the levee. To create a level platform or leveling pad for the DSM crane, either fill will need to be placed or a portion of the levee will need to be excavated. Arcadis recommends no more than a 4-foot vertical cut be made in the levee. To create leveling pads in the levee will probably require one to two cuts or stair steps in the levee.

5.2 Levee Compaction

After completing installation of the DSM columns, as presented in this report, the levee will have to be reshaped to fill in equipment leveling pads. The excavated levee material should be replaced at a moisture content 4 percent below to 3 percent above the optimum moisture content as determined by the Standard Proctor Compaction Test, ASTM D698, in 6- to 9-inch loose lifts. Each lift should be compacted to a minimum dry density of 95 percent of the maximum dry density as determined by ASTM D698.

5.3 DSM and Stone Column QA

Because this project will most likely have a specialty contractor submit proprietary designs, contractor submittals will be a very important part of the QA process. Additionally, documentation of the installation and testing is critical.

5.3.1 DSM Column QA

The following are key contractor's submittal requirements for the DSM columns:

1. Perform an independent bench-scale mixing program and prepare a report. The contractor will typically focus on the mix designs most suitable for the contractor's equipment and procedures.
2. Construct deep mixed test columns prior to production mixing to demonstrate that the proposed mix design, mixing equipment, and mixing procedures can satisfy the specification requirements. This program will allow the contractor an opportunity to try different mix designs in the field, guided by the results of bench-scale laboratory mixing and testing. Typically, the demonstration elements are cored from top to bottom so that the thoroughness of mixing and strength of the soil-cement can be determined.
3. A deep mixing work plan, including proposed materials, equipment, mixing procedures, and column layout and identification.
4. A quality control plan, including the procedures, measurements, and documentation that will be generated to control column geometry, binder properties, mixing procedures, coring procedures, testing procedures, and column protection.

As part of the QA process, the owner's representative should be involved in the observation of materials handling, slurry preparation, slurry testing, mixing equipment, mixing procedures, coring and sampling, specimen storage, and specimen testing. Specific observations should at a minimum include:

1. Geometric layout, including column plan view dimensions, verticality, top and bottom elevations, and overlap;
2. Thoroughness of mixing; and
3. Strength testing meets minimum specified requirements.

5.3.2 Stone Column QA

The Contractor's submittals for stone column installation is usually the work plan and the quality control plan, similar to the DSM columns QA submittal. The aggregate used for the columns should consist of angular rock with a predominant grain size range of ⅛- to 1½-inch-diameter with no less than 5 percent passing the No. 40 sieve. Testing and observation of the stone column installation should include:

1. Gradation, specific gravity, loose density, and compacted density tests should be run on the stone to be installed.
2. During construction, stone consumption, in terms of buckets of a known weight or volume, should be monitored as a function of depth. Based on the loose and in-place compacted density of the stone, it is possible to estimate the column diameter. Decreased rate of stone consumption may

indicate caving of the hole or failure to attain adequate displacement and replacement of the surrounding ground.

3. No stone column should have a diameter less than 90 percent of the minimum diameter specified.

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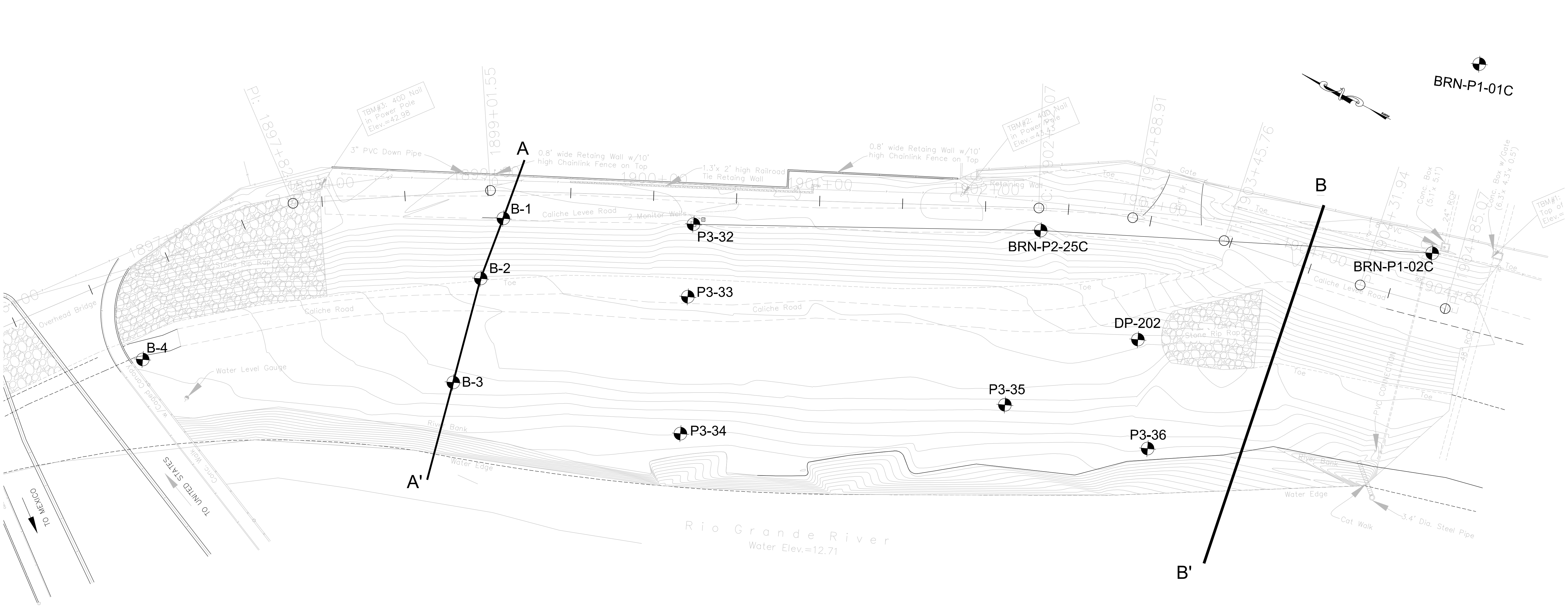
FIGURES



Note:

Inclinometers ARC-1 through ARC-4 correspond to Arcadis' borings B-1 through B-4 and Inclinometers I-32 through I-34 correspond to USACE's borings P3-32 through P3-34

0 60' 120'

USIBWC
GEOTECHNICAL REPORT

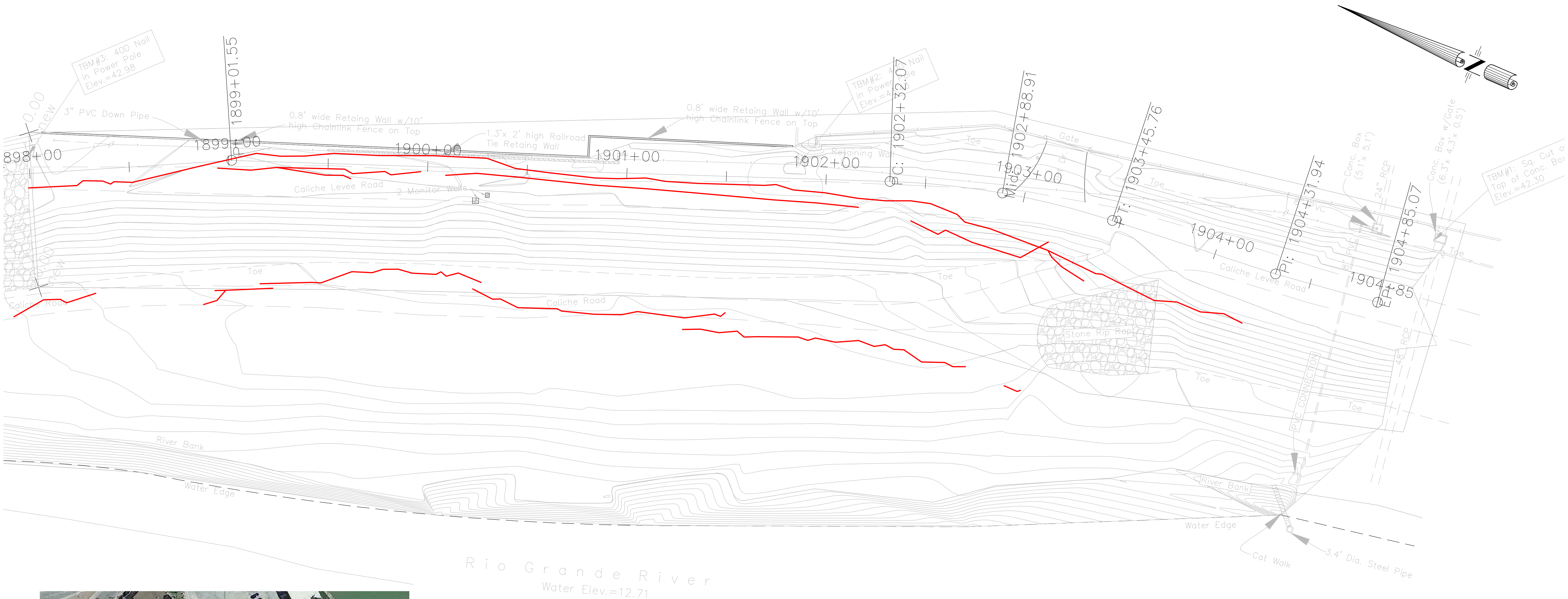
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

BORING LOCATION MAP

ARCADIS

FIGURE
1

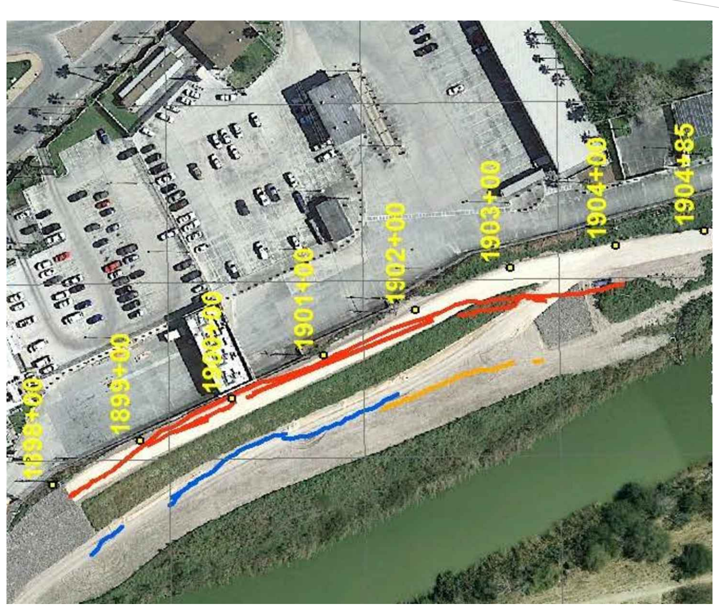
CITY:(Reqd) DIV/GROUP:(Reqd) DB:(Reqd) LD:(Opt) PIC:(Opt) PM:(Reqd) TM:(Opt) LYR:(Opt)ON=*;OFF=*REF*
G:\APROJECT\USIBWC\LA003315.0000\Figures\IBWC DWGs_11-14-17\Figure 2\FIGURE 2.dwg LAYOUT: LAYOUT1 SAVED: 11/13/2017 9:21 AM ACADVER: 19.1S (LMS TECH) PAGESETUP: ---- PLOTSTYLETABLE: ACAD.CTB PLOTTED: 11/16/2017 10:37 AM BY: FLORES, ARMANDO
XREFS: PROJECTNAME: ----
B-SIZE-H-BORDER-FEET
EX-TOPO-CROSS SECTION



LEGEND:
SURFICIAL CRACKS MAPPED BY USACE, 2014

SOURCE:

THIS LOCATION MAP WAS RECREATED FROM FIGURE 4.1 OF THE USACE GEOTECHNICAL EVALUATION OF THE BROWNSVILLE LEVEE CRACKING AND PARTIAL SLOPE FAILURE REPORT DATED JULY 2015.



LOCATION MAP

0 50' 100'

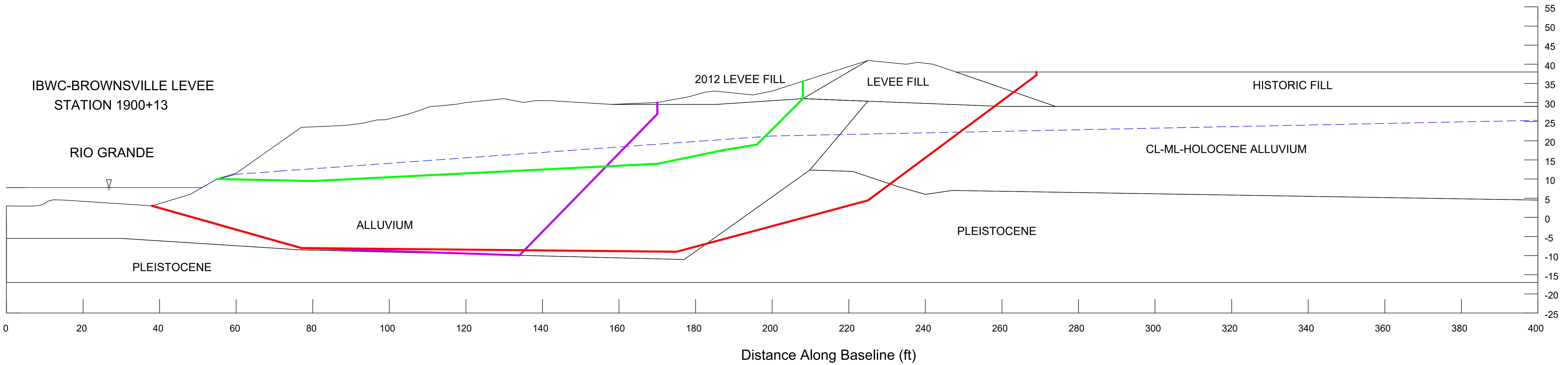
USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

LOCATION OF LEVEE WITH CRACKING

ARCADIS

FIGURE
2



LEGEND:

TOE FAILURE SURFACE

DEEP FAILURE SURFACE

USACE FAILURE SURFACE

WATER SURFACE

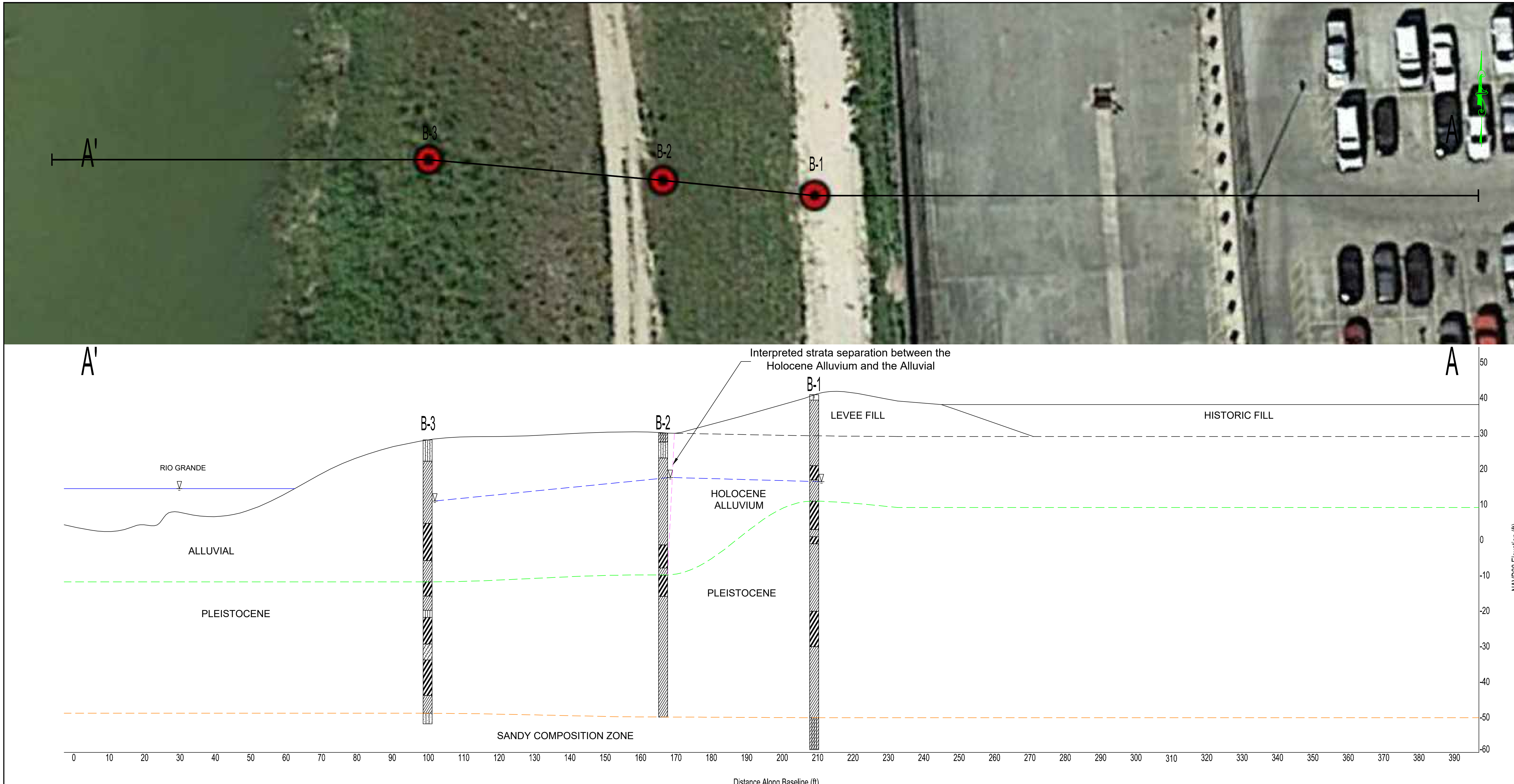
USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

POTENTIAL FAILURE SURFACES

ARCADIS

FIGURE
3



LEGEND:

- SANDY SILT (ML)
- LEAN CLAY (CL)
- SILTY SAND (SM)
- SILTY CLAY (CL-ML)
- FAT CLAY (CH)
- CALICHE BASE MATERIAL
- SILT (ML)

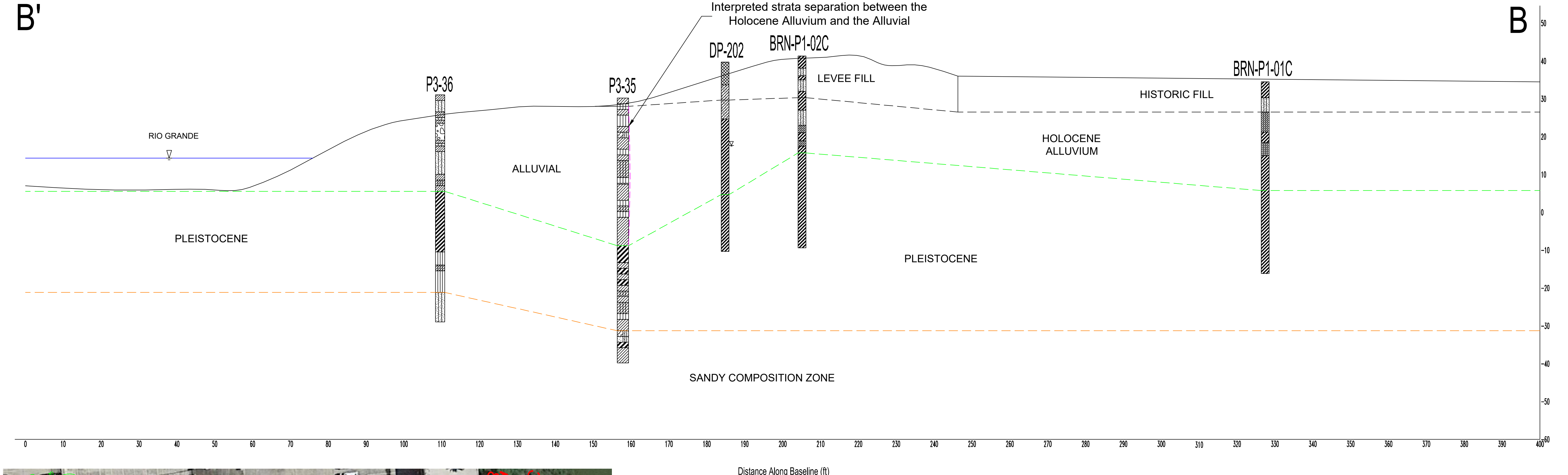
USIBWC
GEOTECHNICAL REPORT

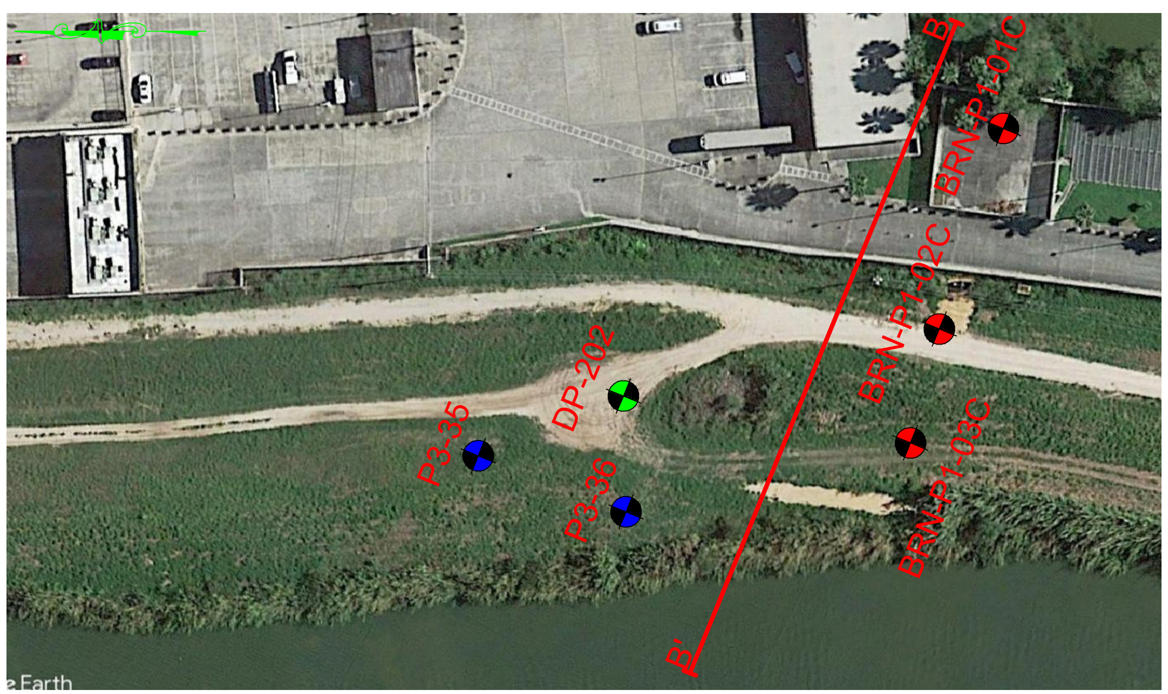
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GEOLOGICAL CROSS SECTION A-A'

ARCADIS

FIGURE
4





LEGEND:

SANDY SILT (ML)	LEAN CLAY (CL)	SILTY SAND (SM)	ERDC SOIL BORING
SILTY CLAY (CL-ML)	FAT CLAY (CH)	POORLY GRADED GRAVEL (GP)	RABA-KISTNER SOIL BORING
CALICHE BASE MATERIAL	SILT (ML)	CLAYEY SAND (SC)	ERDC CPT

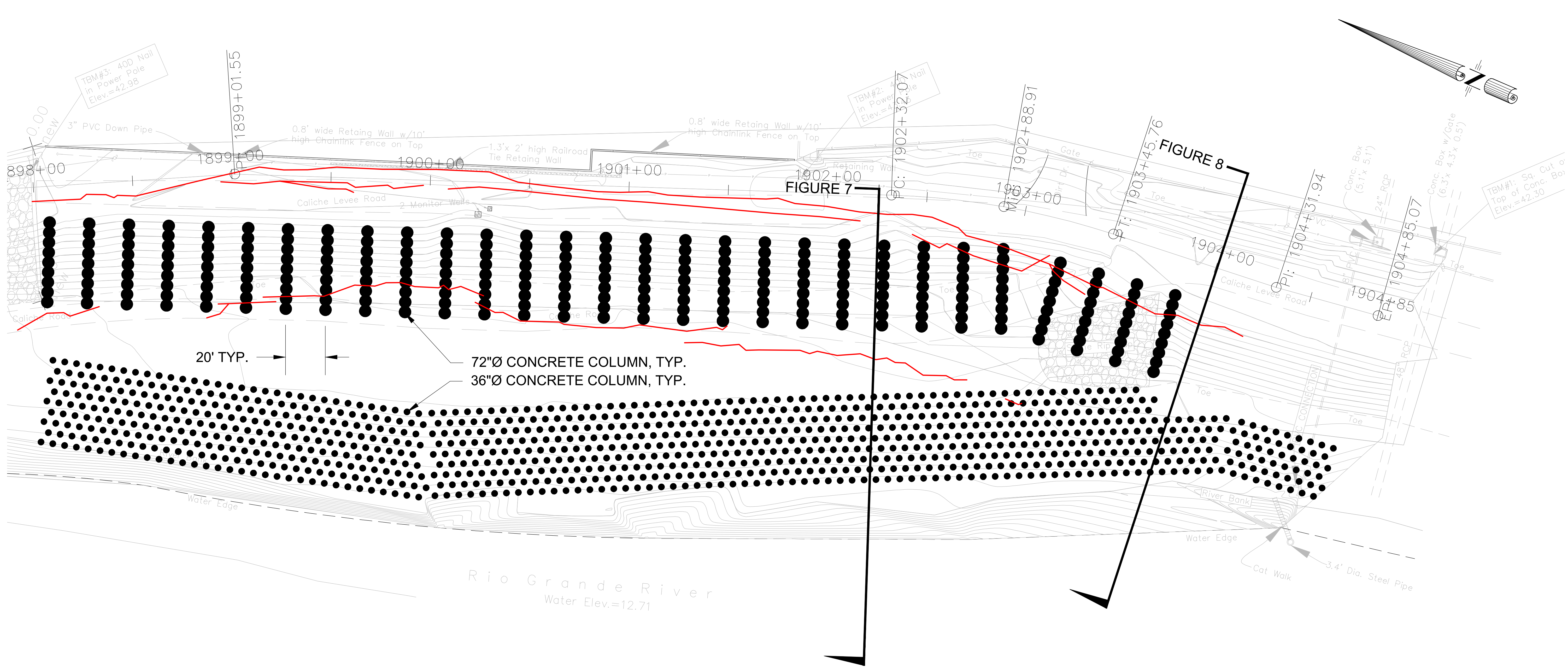
USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GEOLOGICAL CROSS SECTION B-B'

ARCADIS

FIGURE
5



LEGEND:

SURFICIAL CRACKS MAPPED BY USACE, 2014

NOTES:

1. SHEAR PANEL AND STONE COLUMN (9 to 8 ROWS) INSTALLATION: STA. 1898+00 TO STA. 1903+84
2. STONE COLUMN (6 ROWS) INSTALLATION: STA. 1903+84 TO STA. 1904+85
3. INDIVIDUAL SOILS/CEMENT COLUMN DIAMETER: 72"Ø SOIL/CEMENT MIXED COLUMN, OVERLAPPING 12"
4. WIDTH OF SOIL/CEMENT TREATMENT PERPENDICULAR TO C/L: 46' (APPROX.)
5. CENTER TO CENTER PANEL DISTANCE: 20' O.C.
6. INDIVIDUAL STONE COLUMN DIAMETER: 36"Ø STONE COLUMN WITH EQUILATERAL CONFIGURATION
7. WIDTH OF STONE COLUMNS PERPENDICULAR TO C/L: 45' (APPROX.)
8. CENTER TO CENTER STONE COLUMN DISTANCE: 6' O.C.
9. BASE ELEVATION: -15.0'

0 50' 100'

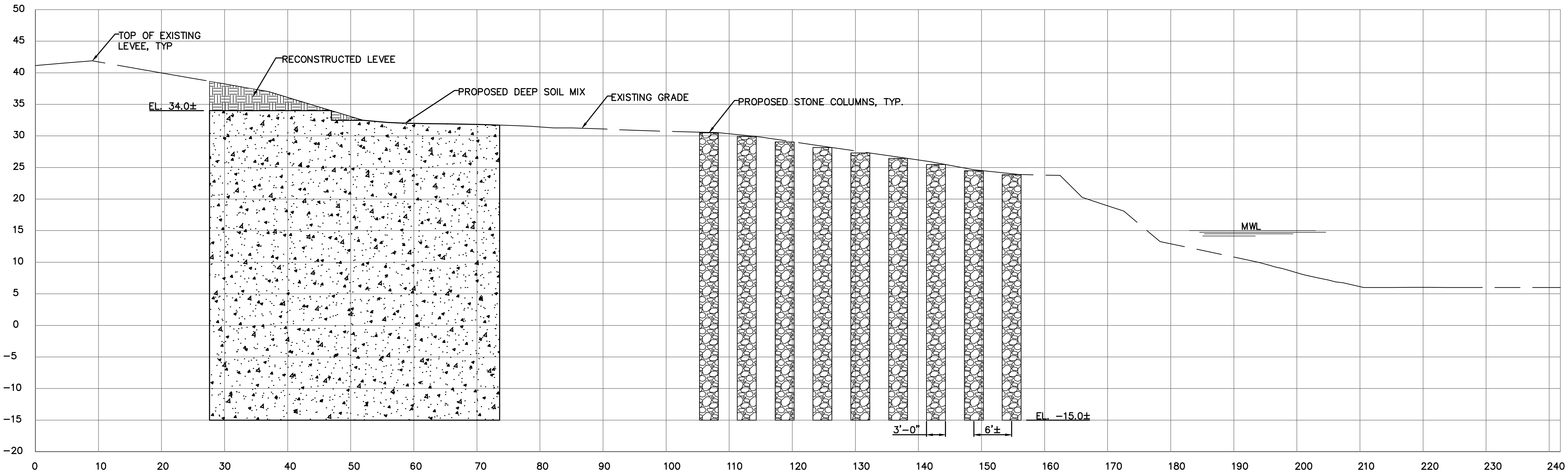
USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

LOCATION OF LEVEE WITH CRACKING
DEEP SOIL MIX AND STONE COLUMN CONFIGURATION

ARCADIS

FIGURE
6



NOTES:

1. SHEAR PANEL AND STONE COLUMN (9 ROWS) INSTALLATION: STA. 1898+00 TO STA. 1903+84
2. INDIVIDUAL SOILS/CEMENT COLUMN DIAMETER: 72"Ø SOIL/CEMENT MIXED COLUMN, OVERLAPPING 12"
3. WIDTH OF SOIL/CEMENT TREATMENT PERPENDICULAR TO C/L" 46' (APPROX.)
4. CENTER TO CENTER PANEL DISTANCE: 20' O.C.
5. INDIVIDUAL COLUMN DIAMETER: 36"Ø CONCRETE COLUMN WITH EQUILATERAL CONFIGURATION
6. CENTER TO CENTER COLUMN DISTANCE: 6' O.C.
7. DEPTH OF TREATMENT: EL. 34.0' TO EL. -15.0'
8. DSM LEVELING PAD ASSUMED TO BE HORIZONTAL AT ELEVATION 34 FT AND ELEVATION 32.47 FT. LEVEE WILL BE RECONSTRUCTED AT COMPLETION OF SHEAR PANEL INSTALLATION

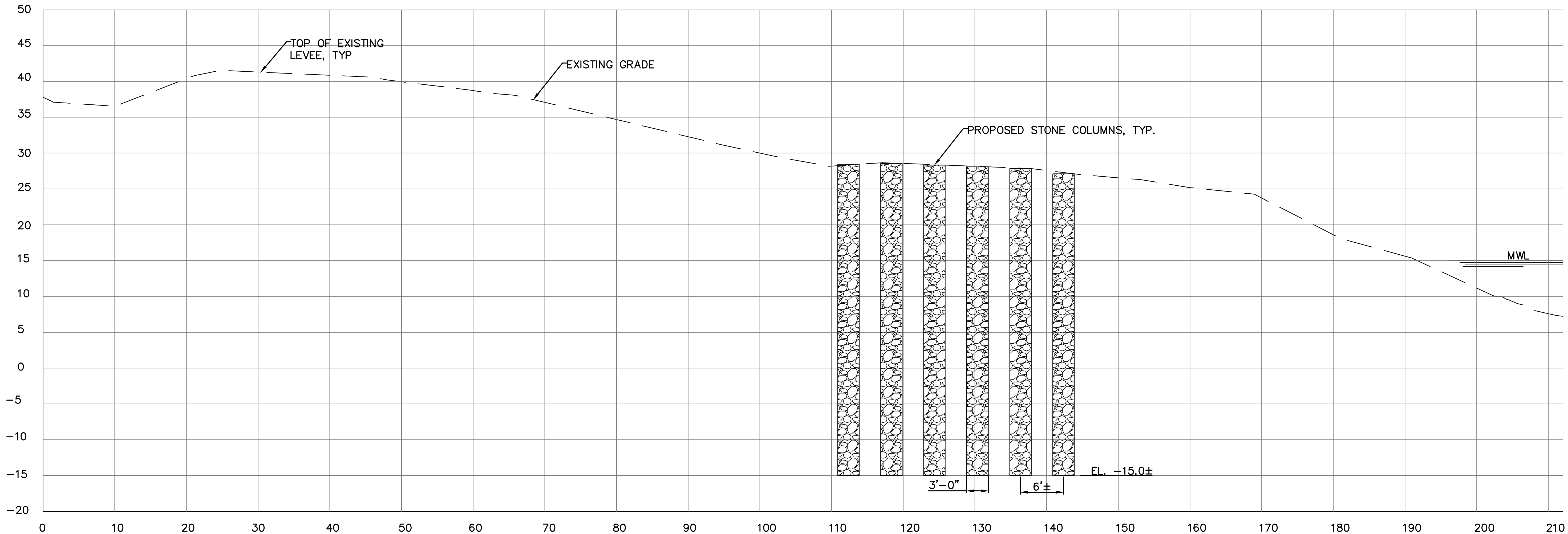
USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

DEEP SOIL MIX AND STONE COLUMN SECTION

ARCADIS

FIGURE
7



NOTES:

1. STONE COLUMN (6 ROWS) INSTALLATION: STA. 1903+84 TO STA. 1904+85
2. INDIVIDUAL COLUMN DIAMETER: 36"Ø STONE COLUMN WITH EQUILATERAL CONFIGURATION
3. CENTER TO CENTER COLUMN DISTANCE: 6' O.C.

USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

STONE COLUMN SECTION

ARCADIS

FIGURE
8

CITY:(Reqd) DIV/GROUP:(Reqd) DB:(Reqd) LD:(Opt) PIC:(Opt) PM:(Reqd) TM:(Opt) LYR:(Opt)ON=*;OFF=*REF*
G:\APROJECT\USIBWC\LA003315.0000\Figures\IBWC DWGs_11-14-17\Figure 9_12-18-17\FIGURE 9.dwg LAYOUT: LAYOUT1 SAVED: 12/18/2017 4:03 PM ACADVER: 19.1S (LMS TECH) PAGESETUP: ---- PLOTSTYLETABLE: ACAD.CTB PLOTTED: 12/18/2017 4:12 PM BY: FLORES, ARMANDO
XREFS: PROJECTNAME: ----
B-SIZE-H-BORDER-FEET
EX-TOPO-CROSS SECTION

APPENDIX A

Arcadis’ February 2016 60% Memorandum

Arcadis Project No. LA003315.0000

Geotechnical Assessment Memorandum
60 Percent Submittal

Remediation Design of Levee Floodplain Failure within the Upper Brownsville Levee Reach Lower Rio Grande Flood Control Project IBM15D0001 – IBM15T0015

Task 2 – Geotechnical Investigation and Analysis

February 19, 2016

This memorandum is part of the final design package and as part of the final report under the certification of the professional engineer.

TABLE OF CONTENTS

FIGURES AND TABLES

ATTACHMENTS

1 INTRODUCTION

This memorandum presents the results of the review and assessment by Arcadis U.S., Inc. (Arcadis) of the geotechnical investigation and analysis for the Remediation Design of Levee and Floodplain Failure within the Upper Brownsville Levee Reach, around levee station 1900+00. Arcadis was tasked by the United Stated International Boundary and Water Commission (IBWC) with evaluating the United States Army Corps of Engineers (USACE) Investigation[1] in order to assess the Alternative I remediation, which consists of excavating and regrading the channel banks and levee embankment at five horizontal to one vertical (5H:1V). The limits of the proposed remediation are from levee station 1892+00 to 1904+85, which are referred to as the project reach throughout this memorandum. Additionally, Arcadis was tasked with taking readings of the three inclinometers installed by the USACE.

This geotechnical memorandum is part of the 60 percent design submittal. The information and recommendations presented herein differ slightly from the original intent of the project, which was to construct the Alternative I remediation. The intent of this memorandum is to justify suspending the current project because the slope stability analyses shows the Alternative I remediation design will not provide an adequate Factor of Safety (FOS).

Information from the USACE Investigation, as well as current information for the inclinometers installed to monitor for additional movement were used to characterize site conditions in order to model the slope stability FOS for current loading conditions. The USACE Investigation included site surveillance; Cone Penetrometer Test (CPT) soundings; mechanical borings and lab testing; instrumentation of the project reach with piezometers, inclinometers, and ground surveys; and bathymetric and terrestrial Light Detection And Ranging (LiDAR) surveys of the project reach. The chronology of the site investigation is summarized as follows.

- July 2014; Site visit with crack mapping.
- August 2014; Phase I & II Assessment
    - CPT program with 32 soundings
    - Ground survey for three reach cross sections
- September 2014; Phase II Assessment
    - Bathymetric survey of river channel
    - LiDAR survey of project reach
    - 6 mechanical soil borings with instrumentation
- October 2014; Site surveillance and monitoring of piezometers, inclinometers and ground survey
- December 2014; Surveillance and monitoring of piezometers, inclinometers and ground survey
- February 2015; Field activities complete, used for seepage and stability analyses
- July 2015; USACE Investigation Report complete

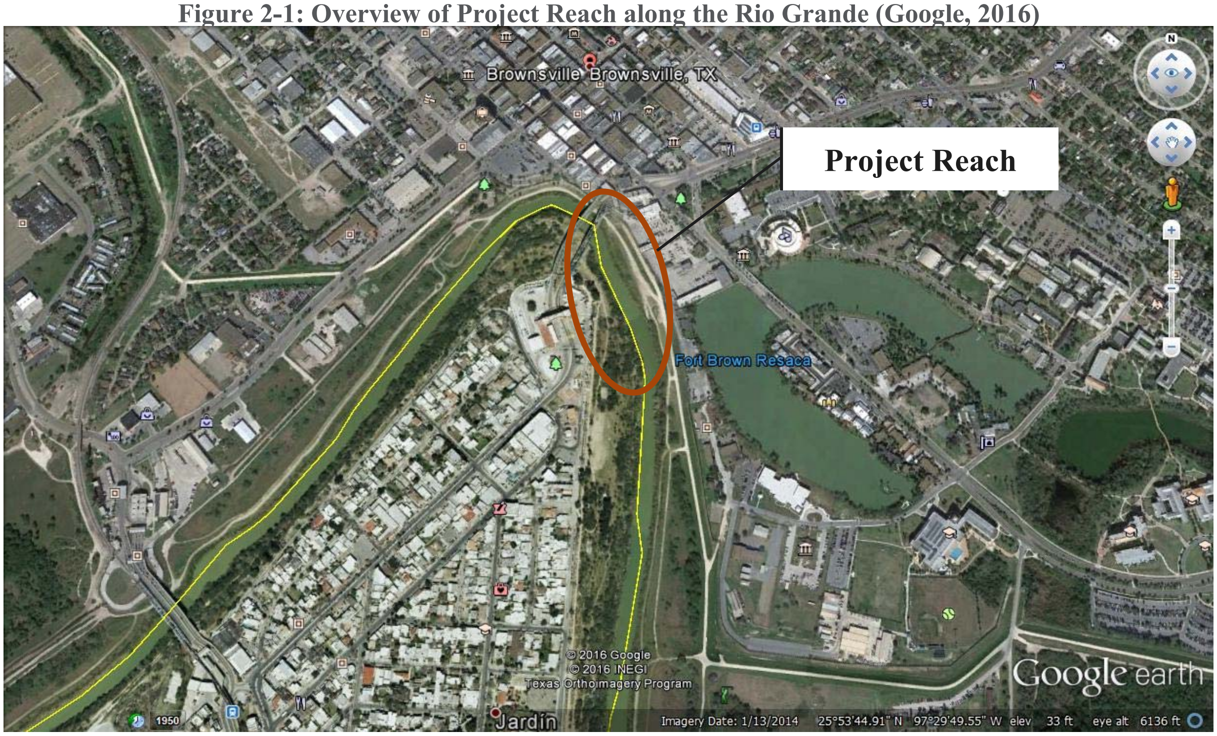
[1] Geotechnical Evaluation of the Brownsville Levee Cracking and Partial Slope Failure, dated July 2015.

As part of the current evaluation, Mr. Kirk Lowery of Arcadis took inclinometer and water level readings on December 17, 2015 and he was accompanied on a site visit by Mr. Jason Vazquez of Arcadis on January 29, 2016. During the January visit, another set of inclinometer readings were taken and a site reconnaissance was completed. Additionally, Arcadis analyzed the slopes at Station 1900+13 where the inclinometers were installed. The site conditions were well defined from the USACE Investigation, therefore no additional investigations were conducted for the current assessment. Models were developed using USACE section data and loading conditions, and the strength properties and potential failure planes within the section were assessed to verify that the proposed remediation will be able to improve the global stability of the reach section.

2 SITE CONDITIONS

The project site, shown in Figure 2-1, is on the left bank of the Rio Grande River downstream of the Gateway International Bridge, adjacent to the U.S. Customs and Border Patrol facility that is currently not in use. In 2014, surficial cracking was detected on the levee crest and toe approximately between levee stations 1898+00 to 1904+00. The project reach lies on a historical oxbow of the Rio Grande, with significant changes to the channel alignment due to river flow and floodplain deposition over time (USACE, 2015).

Figure 2-1: Overview of Project Reach along the Rio Grande (Google, 2016)



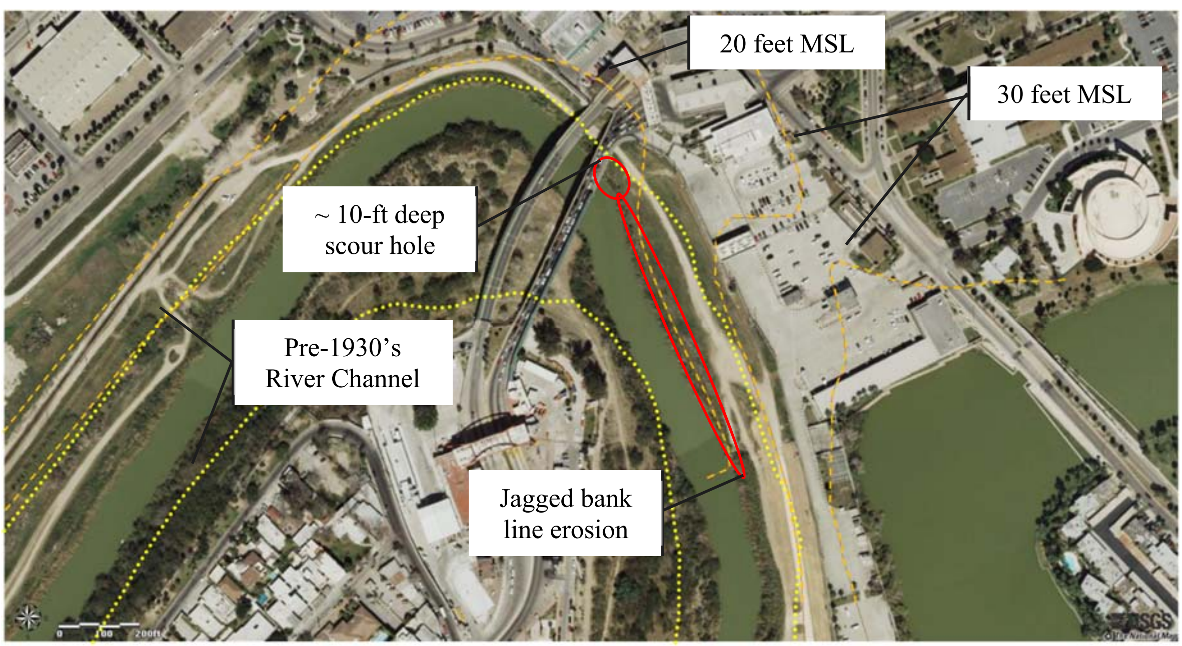
The Levees were originally built by the local government around the 1900's in response to historical flooding of the Rio Grande Valley. Flooding continued into the 1930's with more than 20 floods pushing the river out of banks, including floods from three hurricanes. The USIBWC assumed responsibility of the levee system in 1932 and implemented a major rehabilitation program that lasted into the 1940's in order to update them to be consistent with the standard USACE levee section of less than 25-foot in height with at least 2H:1V side slopes. A 3-foot levee raise was completed in 2012-2013 to provide 100-year flood protection for FEMA certification. The levee raise was complete in October 2013 (USACE, 2015), and the reach was sent to FEMA for certification on January 12, 2016 (IBWC, 2016).

2.1 Topography and Bathymetry

The topography of the project reach is typical of the Lower Rio Grande Valley floodplain, with relatively steep banks from river migration, sometimes including terraces of alluvial material from drastic changes of river elevation over time. The USACE Investigation concluded that the area on the landside of the levee may have up to 15 feet of historical fill since 1911, as borrow was used to bring developed land adjacent to the levee on the protected side, out of the floodplain.

The historical fill material has a relatively moderate slope towards the river. High ground is located near the Fort Brown Resaca (Lake Brown) at around 30 feet Mean Sea Level (MSL)[2], which then drops to around an elevation of 20.0 feet near the bridge as shown on the aerial photo in Figure 2-2. The levee crest is around elevation 40 feet for most of the project reach, indicating that the fill was placed right up to the levee embankment. No information on the material properties or placement methods of the historical fill material has been obtained.

Figure 2-2: USGS Topography and Key Findings of Bathymetry (National Map, 2016)



The bathymetry of the project reach is a result of the river flow through the major bend that occurs where the bridge crosses the river. The riverine infill deposit that makes up the project reach downstream of the bridge has been naturally formed since the 1930's, as a result of significantly reduced flow from the construction of four dams upstream as well as from increased

[2] All elevations provided in MSL, with horizontal control of North American Datum of 1983 (NAD83)

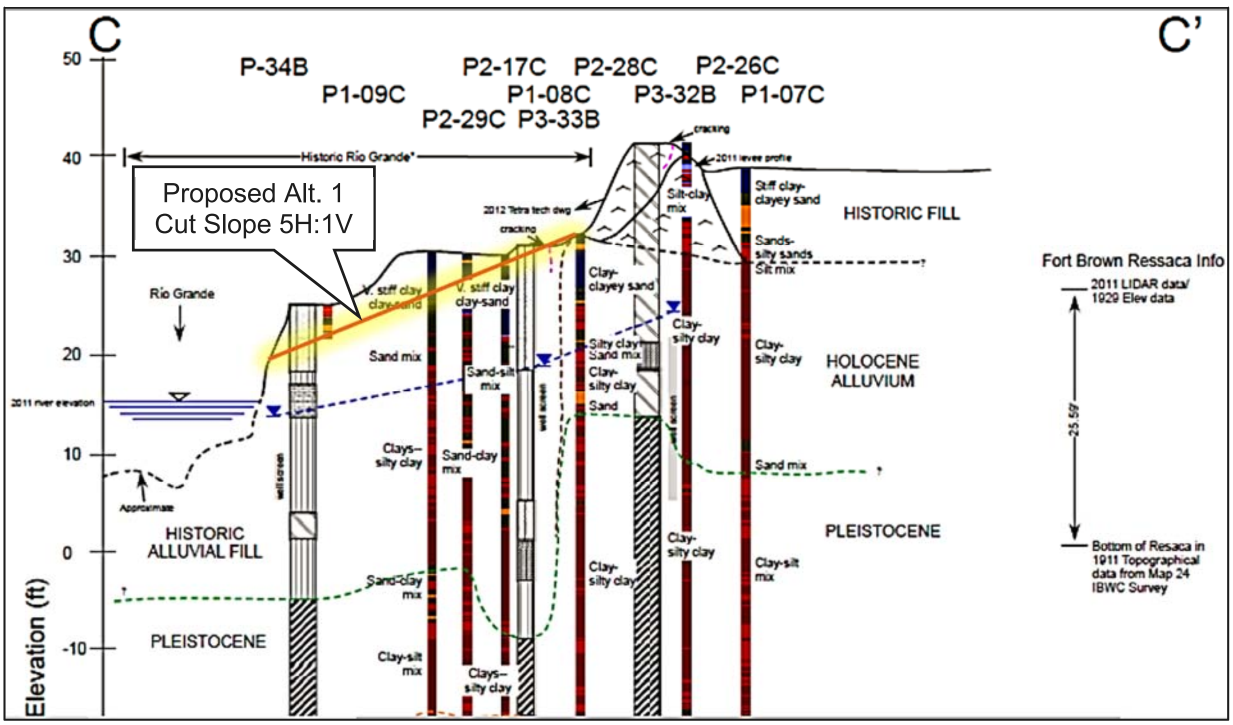
diversions for water supply and irrigation needs. Figure 2-2 also depicts the river channel that extended all the way to levee toe prior to 1930, as concluded by the USACE Investigation.

The bathymetric and side-scan sonar data indicates that there is a major scour hole just beyond the bridge foundation, which is about 10 feet deep. The north river bank through the project reach is extremely jagged with steep bank slopes and what appears to be bank sloughing or caving below the waterline through most of reach downstream of the bridge. USACE concluded that the erosion and caving of this bank material impacts the stability through this reach; therefore Alternative I includes installation of launchable rip rap stone or other revetment that may help control this erosion.

2.2 Levee Section Geometry and Profile

The geometry and profile for levee section for this assessment at station 1900+13 were developed from the USACE Investigation as defined for Cross Section C-C', which is shown in Figure 2-3. This is a cross section from the 2012 Tetra Tech design of the Lower Rio Grande Valley levee improvements. The USACE incorporated information from its investigation to better define the subsurface profile for its analyses.

Figure 2-3: Section Geometry and Profile at Station 1900+13 (USACE, 2015)



Arcadis used this section for our slope stability analysis, based on topographic information provided by USIBWC which does not have the clean geometry as depicted in the design plans. The Alternative I proposed levee remediation consists of cutting the alluvial bank fill material back to a 5H:1V slope and reinforcing with launch-able rip rap or other suitable revetment, as discussed above. Thus an additional set of slope stability analyses were conducted for the proposed Alternative I configuration as discussed in Section 4 of this memorandum.

2.3 Hydraulic Profiles and Gradients

The Rio Grande River and Lake Brown are the two water sources that influence the levee embankment and foundation section along the project reach. The river flow is variable, with typically low river levels due to high water supply diversions, but significant storm and flooding events raise the river level for an extended period, and then river levels draw down as the storm flow recedes. This type of fluctuation can impact the hydraulic gradient for the levee embankment and foundation materials. USACE concluded that the sudden draw down of the river level in April 2014, potentially had a direct impact on the levee cracking and slope failure that is being assessed.

The river stage for the project reach was interpolated by USACE from the downstream river gage using 2011 LIDAR to approximate the river elevation at the bridge and determined it is about 7-foot higher than the Brownsville Gage, which is maintained by USIBWC in coordination with NOAA and other agencies.

The hydraulic loading conditions for the USACE analyses were developed using information from this gage. Additional review of the river gage data for the time period since the completion of the USACE field activities in February 2015 will be evaluated as part the assessment discussed in Section 5.

The Lake Brown pool elevation remains relatively constant, but previous reports indicate that there is some connection of the sandy layers back to the river channel. The topography also indicates that the many shallow water bodies situated near the Rio Grande have a natural grade back towards the channel, which indicates they slowly drain back towards the river. The water year 2015 was one of the wettest of record, with severe storms in May through June as well as in October through November. Therefore the groundwater level is currently high, as was exhibited in the water level readings taken in December 2015. Additional review of the available groundwater data for the time period since the completion of the USACE field activities in February 2015 will be evaluated as part the assessment discussed in Section 5.

For the current assessment, the seepage profiles developed by USACE were used to model the river elevation and groundwater regime. The results from this analyses seems reasonable and appropriate of the most likely loading conditions for the project reach. Additional evaluation of the influence the hydraulic profile has on the slope stability will be further assessed by Arcadis in the next phase of this project.

2.4 Site Reconnaissance

The site was visited by the Arcadis team on December 17, 2015 and on January 29, 2016 in order to take instrument measurements and conduct site reconnaissance. During the December trip, no unusual cracking or settlement was detected on the levee embankment or river bank soils. The instruments were located and measurements were recorded, although dense vegetation made inspection difficult.

The site was visited again on January 29, 2016 for additional surveillance and monitoring. A closer inspection of the levee crest and landside shoulder detected minor areas of soil movement, which appeared to be associated with the bulkhead wall that runs for a short distance along the

Figure 2-4: Levee crest at Station 1899+50, looking northeast at wooden bulk head wall and Border Patrol facility parking lot

levee. There is a wooden bulkhead wall that runs adjacent to the levee from approximately station 1899+50 to 1901+50, see Figure 2-4, its depth and purpose are unknown. Along the levee toe, there were indications of a very minor scarp at the grade break which appeared to have been there for some time. This area was not mapped during the January visit, but should be further investigated once the levee has been mowed. Photo Plate 1 in Appendix A include typical site conditions for the levee crest and toe from this visit.

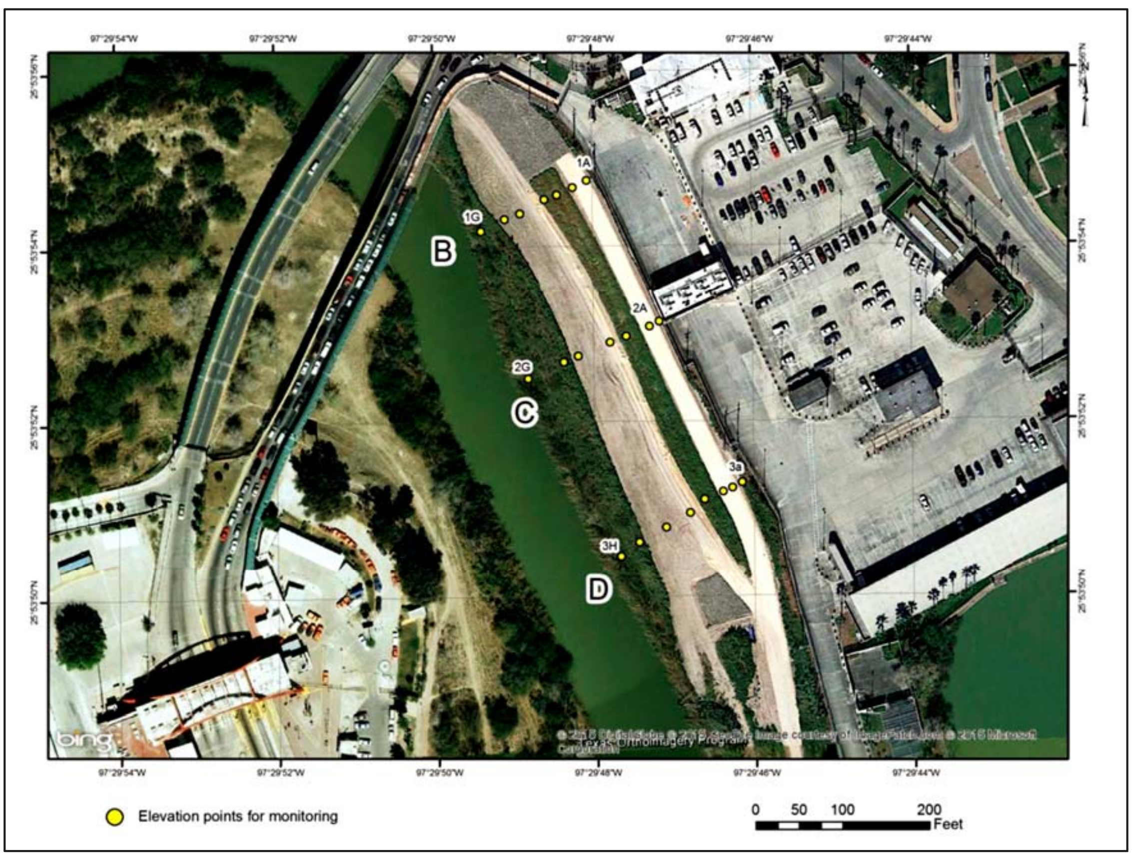
During the January site visit, USIBWC arranged access to the adjacent Border Patrol compound to inspect the parking lot and retaining wall situated in the downstream levee slope. Minor distress was noted by joint separation between monoliths at one wall location showing exposed rebar, and approximately 65 feet north another joint between monoliths appears to be separating. Between these two monoliths, light cracking in the concrete driveway between the wall and about 12 feet east of the wall is more pronounced as compared to the concrete in the surrounding area, as shown in Photo Plate 2.

Additionally, a utility crew appeared to pumping water from a nearby manhole in the parking lot, an activity that is not common for the USIBWC operations staff to see. Further assessment of these conditions is discussed in the conclusions and recommendations.

2.5 Survey Assessment

Figure 2-5 shows the details of the ground survey used to detect surface movement, which consisted of establishing 21 survey monuments, or pins, in a series of 3 sets of 7 pins transecting the limits of the levee. These pins were set in August 2014, several months after the cracks were detected in April. Measurements were taken with a total station using a fixed base point, since traditional GPS methods did not provide the level of accuracy needed to detect surface movement through the project reach.

Figure 2-5: Survey Monument (Pin) Locations for Monitoring Surface Movement (USACE, 2015)



The survey pins were measured four times by USACE between August and October 2014. The results indicated nominal movement, with the max cumulative displacement of 0.18 foot for the survey period occurring on pin 1A at the north most corner of the section. This pin is in the same area where the distress was noted in the parking lot and retaining wall of the customs facility.

IBWC has been taking measurements twice a month beginning November 2015. Arcadis received this information on February 17, 2016 and is currently assessing what has been measured since the USACE field activities were complete.

3 SUBSURFACE CONDITIONS

The subsurface conditions are complex based on the depositions that make up the Rio Grande floodplain, with alluvial deposits overlying Pleistocene-age soils. Historical fill and Holocene Age Alluvium composed mostly of medium stiff to stiff lean and fat clays were deposited above the Pleistocene-age soils behind the levee. Toward the riverside, the alluvial soils are predominantly silty clay and silt that are soft to medium stiff in consistency with localized layers of predominantly sandy soil that overly the Pleistocene-age soils. The Pleistocene-age soils are generally very stiff, jointed and slickensided fat clays with variable silt and sand content.

3.1 Previous Investigations

The USACE Investigation describes a thorough evaluation of the available geotechnical investigations as conducted for various improvements along the project reach. The historical information specific to the project reach is summarized below.

- Two borings that were located upstream/downstream of the project reach, Raba-Kistner (2009);
- Nine borings from areas adjacent to the bridge and customs facility; Lockwood, Andrews, and Newnam, Inc. (1962c); Professional Service Industries (1984), and TrinityTesting Laboratories, Inc. (1992).

After the existing site conditions were evaluated, USACE completed a detailed site investigation to better define the subsurface conditions specific to the project site, which included;

- 32 CPT soundings
- 6 mechanical soil borings

Soil sampling and laboratory testing were conducted to determine the soil classification and engineering properties. In addition, the boring logs and testing results for the previous borings were used to interpret the complex geologic profile of the project reach.

3.2 Geologic Profile

The geology of the project reach consists of Pleistocene Alluvial soils, overlain with Holocene Alluvial soils, and Historical Alluvial that has been deposited since 1846. Information from the USACE Investigation suggests that the Pleistocene sediment was likely exposed to weathering for several hundred years before the Holocene Alluvial soils were deposited, which would account for the variability in the subsurface soils near the Pleistocene interface.

The depositional sequence of materials is important for the current analyses in order to identify the potentially problematic soil conditions, as well as the potential failure plane for slope instability. Four typical sections were developed for the USACE Investigation, as shown in Figure 3-1, one longitudinal and three transverse based on the levee improvement plans prepared by Tetra Tech. The information collected by USACE was overlain on the 2012 Tetra Tech design sections, to develop the profiles used for the USACE seepage and stability analyses.

Figure 3-1: Project Reach with location USACE Borings and Sections (USACE, 2015)

The USACE Investigation resulted in development of the following basic geologic profile.

- The riverbank is composed primarily of a 30- to 35-foot-thick layer of gray to dark gray, fine-grained historical alluvium;
- 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), which is;

- Underlain by the stiffer, and darker Pleistocene, jointed and slickensided fat clays with variable silt and sand contents.

Our review of the geologic profile indicated that Pleistocene interface may be deeper than indicated by USACE, specifically for the critical section at station 1900+13. Arcadis has interpreted the depth of the alluvium directly from the USACE boring logs and from the USACE's R-1 CPT Sections. This information was plotted on Section C-C', Figure 2-3, as shown on Section I in Attachment B to show the interpreted alluvium/Pleistocene interface. Section II shows by a greenish color where siltier materials are located at each CPT. The base of these siltier soils especially in comparison with the boring logs would tend to indicate the interface of the alluvium and Pleistocene. Section III in Attachment B shows the normalized shear strength with overburden CPT results. This was also used to help determine the location of the alluvium/Pleistocene interface and to show where relatively soft layers are present. Generally, the red colors shown on this section tend to indicate the soils are soft while the dark blue indicate the soils are stiff.

3.3 Material Properties

The general geologic profile was used to interpret zones of similar materials, these strata are made up layers of soils as classified using the Unified Soil Classification System (USCS). Laboratory tests were used to confirm the USCS classifications for the borings, as well as to determine the specific engineering properties for each soil material. The tests used for the USACE Investigation include; CPT results, grain-size analysis, Atterberg limits, classification of soils, consolidation tests, direct shear test, unconsolidated-undrained (UU) triaxial tests, and moisture content determination.

Table 3-1: USACE Material Properties for Slope Stability Analyses (USACE, 2015)

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

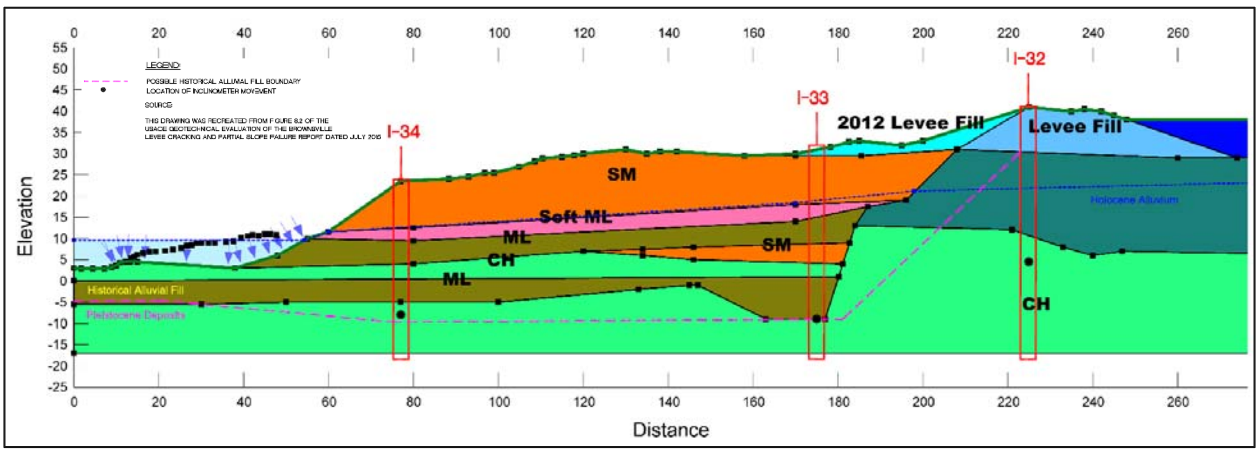
For the current assessment, Arcadis used the properties from the USACE Investigation to calibrate the stability model, and then assessed the influence the key parameters have on the stability of the section. Shear strength parameters were varied to assess the impacts of strength loss due to loading and/or weathering conditions of the subsurface materials. The results of this assessment are presented in Section 4 of this memorandum.

3.4 Inclinometer Assessment

In September 2014, three inclinometers I-32, I-33 and I-34 were set in the P-32B, P-33B and P-34B boreholes at the levee crest, toe, and at the edge of the river bank, respectively. The USACE established baseline measurements in October 2014; and took additional measurements in December 2014 and in January 2015. Measurements were taken during the Arcadis site visits in December 2015 and then again in January 2016.

Figure 3-2 has been prepared to show the initial USACE slope stability model and inclinometer location within the section, along with approximate locations of potential subsurface movement based on inclinometer measurement results. Comparing the Figure 3-2 and the Section I in Attachment B, the alluvium/Pleistocene interface is at higher elevations in the USACE slope stability model than the interface interpreted by Arcadis. The approximate location of the movement, as shown by the largest black circle within the boring location outline on the figure, also tends to provide a good indication of the softer alluvium and stiffer Pleistocene interface.

Figure 3-2: Model of Levee Section with Inclinometers and Geologic Interpretation (Arcadis, 2016)



Arcadis has plotted the USACE initial inclinometer readings and one other USACE readings along with Arcadis December 2015 readings, see Attachment C. The January 2016 readings were not plotted due to conflicts between the processed data and the previous data that could not be resolved. These plots include the soil boundaries as presented in the USACE Investigation. The inclinometer plates depict the results of the inclinometer movement, which is summarized as follows.

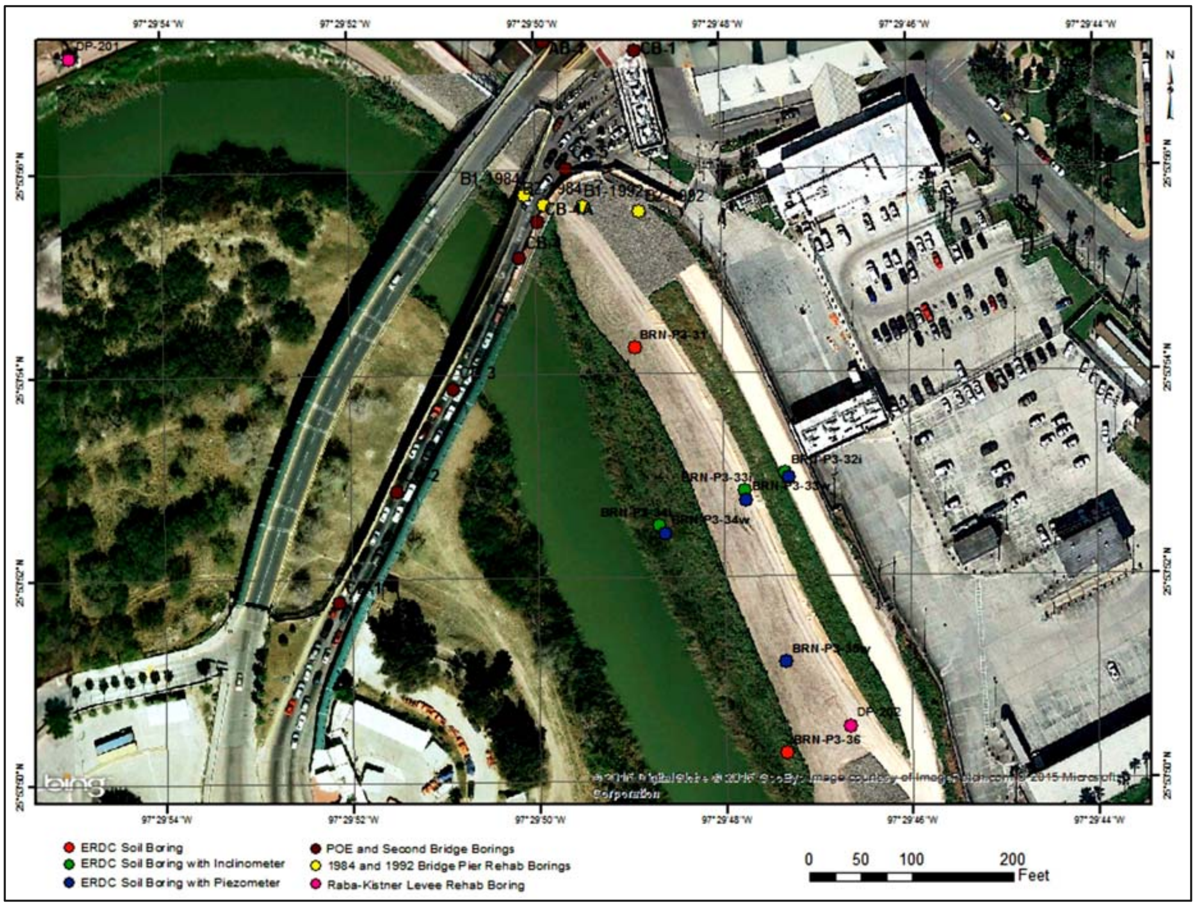
- Since December of 2014, Inclinometer I-32 appears to have has moved toward the river an additional 1.5 to 2 inches between depths of 32 feet and 36 feet beneath the levee crest.
- Since January of 2015, Inclinometer I-33 appears to have moved towards the river over 2.5 inches between depths of 40 feet and 42 feet beneath the toe of the levee. The inclinometer probe could not be advanced beyond a depth of 40.5 feet during the January 2016 readings indicating that the inclinometer casing has buckled or collapsed at that depth.

- Since January of 2015, the Inclinometer I-34 appears to have moved towards the river an additional 1.5 to 2 inches between depths of 32 and 34 beneath the edge of the river bank.

3.5 Piezometer Assessment

Four piezometers, or monitoring wells, were installed in September 2014 to measure the groundwater across the critical section near station 1900+13 (with coupled inclinometers) and assess the flow regime, as shown in Figure .

Figure 3-3: USACE Boring and Instrument Locations (USACE, 2015)



The piezometers were measured continuously using data loggers for the USACE Investigation between August and October 2014. The results indicated groundwater appears to be influenced by the levels of the Rio Grande and of Lake Brown. Arcadis measured the water levels in the piezometers on December 17, 2015 and the measurements show a relatively high groundwater water level as compared to those measured by the USACE. Table 3-2 includes the maximum and minimum readings as taken by the USACE, along with the December 2015 measurements. Also included in the table are the top of casing elevation, the screen elevations and the geological classification where the well screen was set. The ground water levels were not measured during the January 29, 2016 site visit.

Table 3-2: Water Level Data

Piezometer	Elevation (NAVD88)						Screen Interval
	Top of Casing	Top of Screen	Btm of Screen	Max Water	Min Water	Dec 2015	
BRN-P3-32W shallow	39.93	19.93	4.93	23.80	22.55	24.49	Holocene
BRN-P3-32W deep	39.95	-25.05	-35.05	21.35	20.17	21.72	Pleistocene
BRN-P3-33W	30.61	16.61	5.61	18.39	17.58	19.12	Alluvium
BRN-P3-34W	23.08	8.08	3.08	12.94	11.22	12.11	Alluvium
BRN-P3-35W*	31.67	-14.83	-31.13	23.42	22.31	23.32	Pleistocene

4 SLOPE STABILITY ANALYSIS

Slope stability was analyzed using GeoStudio 2007 version 7.23 SLOPE/W© software by Geo-Slope International. Spencer's method of force and moment equilibrium was used to compute the theoretical FOS. The seepage conditions were modeled from the phreatic surface developed from the USACE Investigation, for both steady state seepage and rapid drawdown.

For this analysis, the slip surface for a block failure was specified to pass through the location where movement was measured in each inclinometer. The analyzed cross section is from the 2012 Tetra Tech design of the Lower Rio Grande Valley levee improvements. USACE incorporated the information from its investigation to better define the subsurface profile for its analyses. Arcadis used this section for our slope stability analysis, and modified the profile based on the interpretation of the alluvium/Pleistocene interface.

4.1 Factor of Safety Criteria

Factor of Safety with respect to slope stability is defined as the ratio of resisting forces to that of the applied forces, therefore as the FOS approaches unity (1.00) the slope becomes unstable and failure is considered imminent. The USACE recommends a long-term FOS of 1.3 and the end-of-construction FOS to be 1.4, for standard levee embankments, in accordance with the USACE Engineering Manual 1110-2-1913, *Design and Construction of Levees (2000)*.

4.2 Previous Analyses

Four loading cases were considered by the USACE for the levee slope stability, based on typical standards from the USACE Engineering Manual 1110-2-1902, *Slope Stability (2003)*. These load combinations and the seepage analysis for each loading case are provided in the following subsections of this memorandum. The shear strengths used were from the 2011 Raba Kistner analyses as prepared for the 2012 Tetra Tech design (USACE, 2015), except the soft silt and the historical fill were estimated by the USACE based on CPT, standard penetration tests (SPT) and other lab testing results.

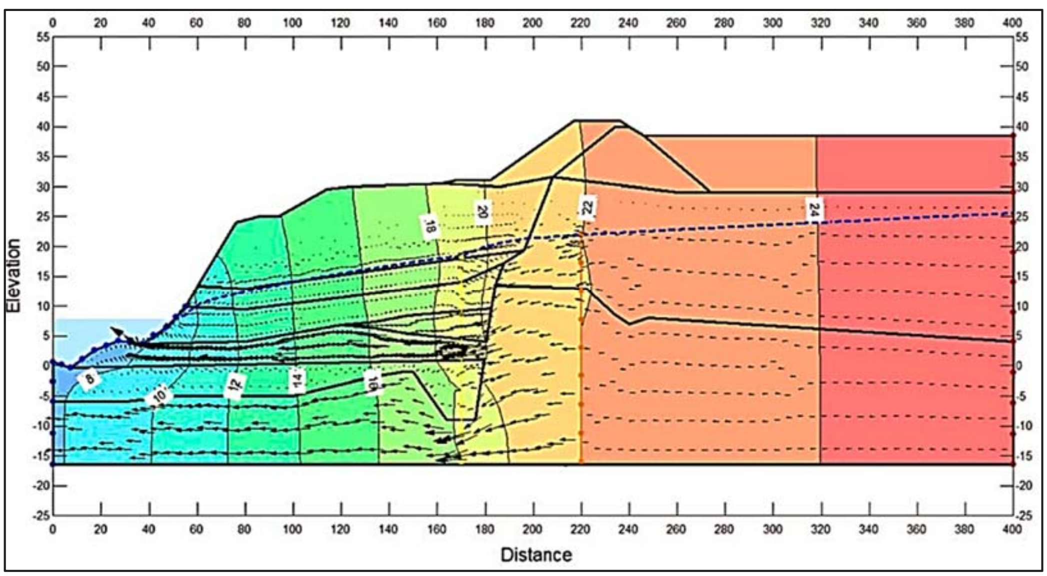
The USACE Investigation concluded that water levels in Lake Brown are typically at least 12 feet higher than the river elevation, even during high flow events. This impact on the pore water pressure for the soil materials was captured by assigning total head values for each surface across the profile. For the levee station 1900+13, piezometers provided the means of assigning additional boundary conditions based on the water level measurements for similar river loading. The boundary conditions applied for the levee section at station 1900+13 were as follows;

- Landside Total Head = 25.6 feet
- Levee Crest (P3-32) Total Head = 22.0 feet
- Riverside Toe (P3-33) Total Head = 19.0 feet
- Low River Head = 7.7 feet and High River Head = 14.5 feet

4.2.1 Low River – Steady State Seepage

Figure 4-1 provides the USACE seepage analysis results for the low river condition using steady state seepage. The colored contours indicate the pore water pressure across the levee section profile. The phreatic surface remains high until it reaches the soft silt layer, and the pressures quickly dissipate to equalize with river level.

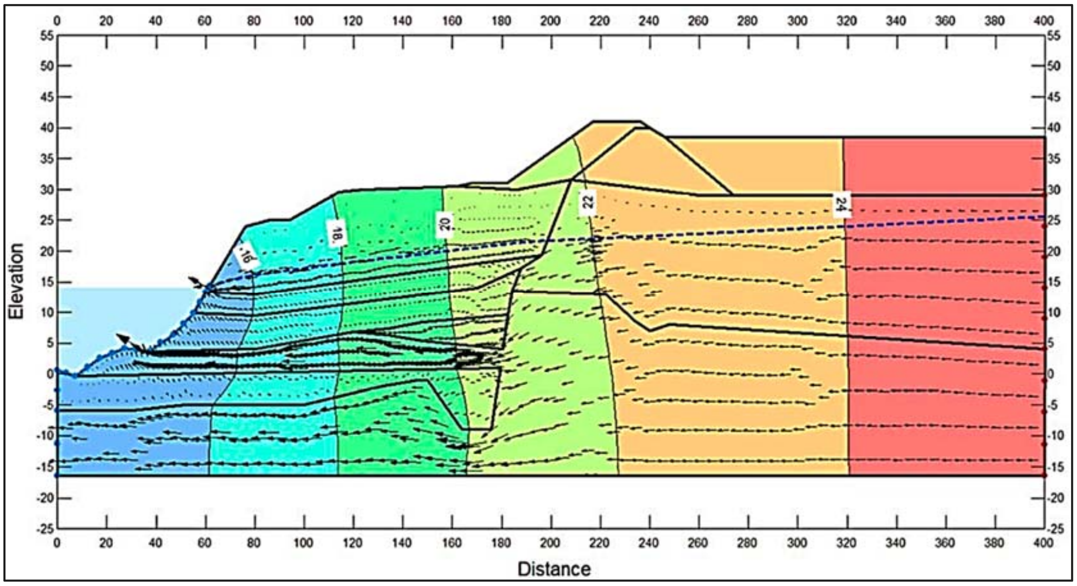
Figure 4-1: USACE Seepage Results for Low River Level (USACE, 2015)



4.2.2 High River – Steady State Seepage

Figure 4-2 provides the USACE seepage analysis results for the low river condition using steady state seepage. This time the phreatic surface remains high until it reaches the high river level.

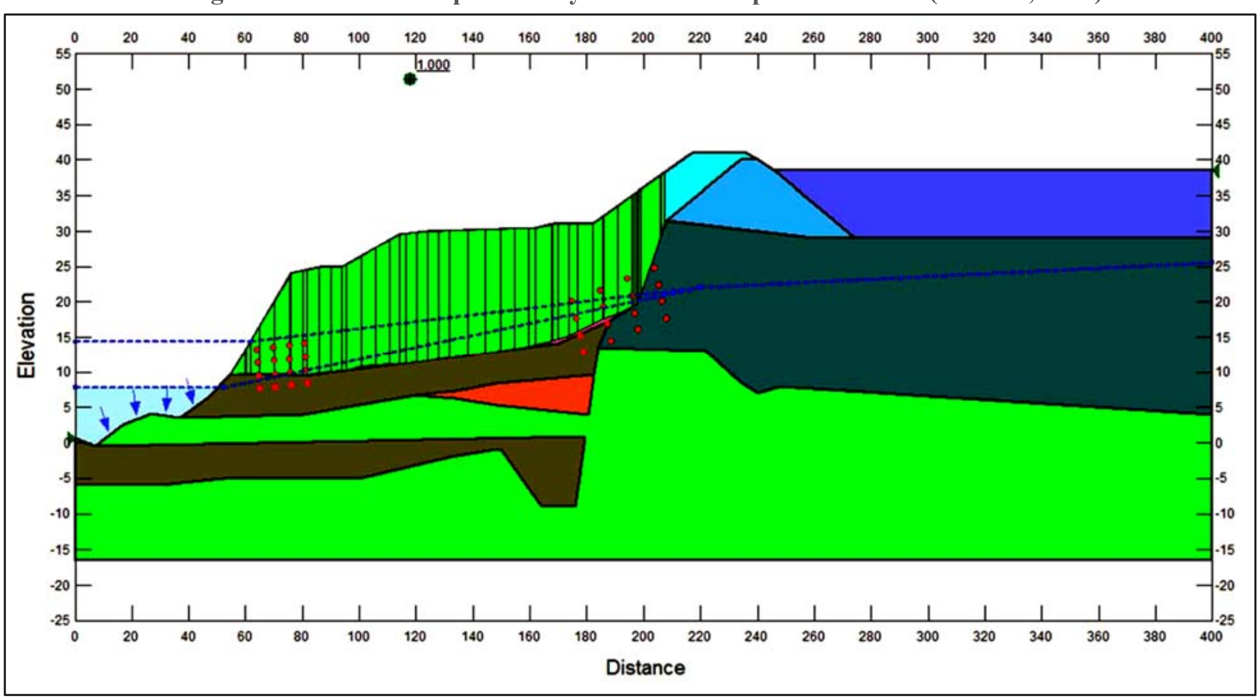
Figure 4-2: USACE Seepage Results for High River Level (USACE, 2015)



4.2.3 Rapid Drawdown Conditions

Figure 4-3 provides the USACE seepage analysis results for the rapid drawdown condition where the water rapidly drops from the high river to low river level. The USACE speculated that the height of the drawdown is about equal to the height of soft silt zone for this section, and the USACE concluded that this contributed to the slope failure experienced in 2014.

Figure 4-3: USACE Slope Stability Results for Rapid Drawdown (USACE, 2015)

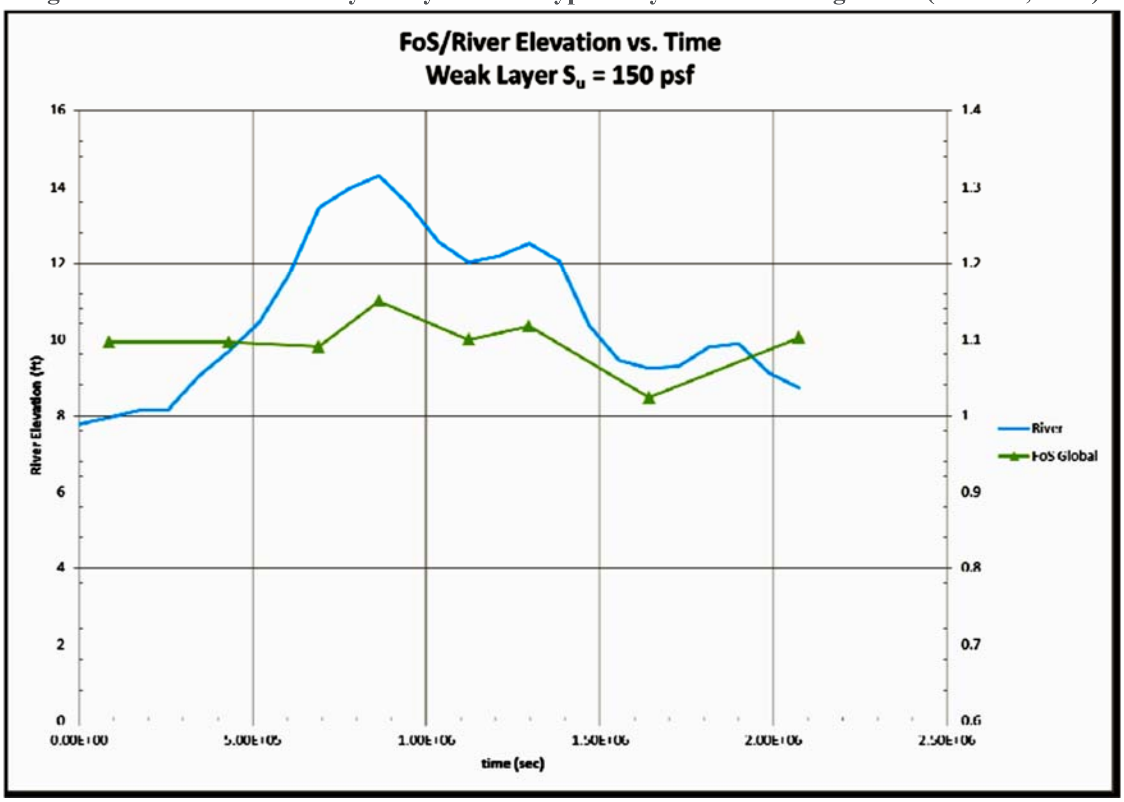


4.2.4 Transient Seepage Conditions

To simulate dynamic hydraulic loading conditions, as is typical for the Rio Grande system, a transient stability analysis was conducted as part of the USACE Investigation. The transient analysis uses unsaturated soil mechanics to adjust the material properties based on the volumetric water content of the soils and the associated pore water pressure, based on the hydraulic loading hydrograph, which is time dependent. This analysis confirms that the FOS for the levee section approaches unity for relatively normal hydraulic loading events.

The transient analysis also permits a time dependent assessment of the FOS based on the loading hydrograph, as shown in Figure 4-4 reproduced from the USACE Investigation.

Figure 4-4 Transient Stability Analyses for a Typical Hydraulic Loading Event (USACE, 2015)



4.2.5 USACE Slope Stability Results Summary

The USACE evaluated the weakest soils and most likely slippage plane by reviewing the CPT section, see Section III in Attachment C. On this section the pink color represents underconsolidated or significantly weak soils that may exhibit movement or failure. This layer of weak soil appears in the CPT closest to the river, CPT BRN-29C, at elevations of about 10 to 15 feet. The slope stability for the different conditions were analyzed passing the failure plane through these weak soils and the results are presented in Table 4-1.

Table 4-1: USACE Slope Stability Analyses Results for Levee Station 1900+13 (USACE, 2015)

Load Case	Rio Grande Water Level (feet)	Calculated Factor of Safety
Steady State Seepage – High River	14.3	1.26
Steady State Seepage – Low River	7.7	1.10
Rapid Drawdown	14.3 to 7.7	1.00
Transient Seepage Conditions	Hydrograph	1.02

4.3 Current Analyses

The USACE Investigation indicated that a blocky progressive failure mode was the most likely cause for the slope movement that resulted in the cracks detected in April 2014. This type of instability can be attributed to a number of factors that include: relatively weak alluvial soil materials, construction of the levee raise, significant river fluctuations, as well as seepage from Lake Brown. Initially, we began our analysis by changing soil strengths (parametric analysis) in the USACE section at Station 1900+13 along the relatively shallow failure surface presented in the USACE report. However, after observing movement in the inclinometers at much greater depths than those analyzed by the USACE, the failure plane in our slope stability analyses was shifted downward. A fully specified failure surface was analyzed because the recorded deflection in the inclinometers provided elevations where the soil movement is occurring.

The sensitivity of the model for the deeper failure plane at station 1900+13 was analyzed by varying the shear strength. In order to quantify the possible effects of implementing the Alternative I levee remediation design, we prepared an analysis of the proposed condition with 5H:1V slopes from the levee toe to the channel bank, as shown on Figure 2 3.

4.3.1 Sensitivity Analyses

For the sensitivity analysis, the shear strengths of the soil layers were varied to obtain a FOS of approximately one to represent the existing conditions of imminent slope failure. Initially, steady-state low river water conditions similar to those modeled by the USACE were used in the analyses. Three different combinations of the shear strengths were used with the steady state water conditions and a low water level to model imminent failure. The soil conditions are shown on SS-1, SS-3 and SS-5 in Attachment C. Additionally, the soil strengths were varied for one rapid drawdown case similar to those modeled by the USACE. This case is shown as RD-1 in Attachment C.

4.3.2 Alternative I Analyses

Using the analysis of the current conditions, as described in the previous section of this memorandum, the Alternative I remediation where the slope is cut to 5H:1V was analyzed using these various shear strength combinations to determine a new FOS. In all cases, the FOS did not improve after cutting the slope to 5H:1V. The results of the analyses are presented on SS-2, SS-4, SS-6 and RD-2 in Attachment C.

5 CONCLUSIONS AND RECOMMENDATIONS

The USACE report concluded that the partial slope failure that occurred in 2014 was the result of several independent factors that include; site specific subsurface conditions, hydraulic loading events, along with changed conditions from the recent levee raise construction. The USACE recommended continuing dedicated maintenance and monitoring of the project reach, as well as for remediating the conditions that resulted in the levee instability. Alternative I was selected as the appropriate solution, and Arcadis was contracted to more thoroughly analyze the remediation design and to monitor the inclinometers.

In our initial assessment, Arcadis concludes that the proposed remediation of cutting the slope back to 5H:1V from the levee toe to the river bank will not improve the overall stability of the section for the project reach. Our analyses indicate that the FOS does not improve for the reconfigured slopes, because the deeper failure surface causes the centroid or central mass point to shift closer to the levee. For the USACE analysis, the centroid was closer to the Rio Grande and thus the cutting of the slope was reducing the upslope driving force. With a centroid further upslope, cutting of the slope does not significantly reduce the driving force.

In addition, the failure mechanism appears progressive. Shallow movement through the alluvium probably occurred as documented by the USACE and this movement relieved some of the confining pressure of the soils further upslope. This initial movement relieved the confining pressure of the jointed/ slickensided clays enabling development of a slippage plane through the jointed material, and created a natural slippage plane between the soft alluvium and stiff Pleistocene soils. Furthermore, the slippage plane that has developed may be located deeper beneath the levee embankment than previously evaluated, which would increase the possibility of a more global slope failure that encompasses the entire levee section (rather than just the riverside slope).

Arcadis recommends that additional investigation and evaluation be conducted for the project reach to confirm the site and subsurface conditions. In conjunction with the additional evaluation of reach conditions, we recommend that other alternatives be developed and further evaluated as potential remediation solutions. This approach is needed so that remediation activities can effectively improve the stability of the levee embankment in accordance with the full intent of design criteria for flood risk management infrastructure.

5.1 Investigation and Evaluation

It is clear that the slope instability for the project reach is influenced by several factors associated with the site and subsurface conditions, therefore it is crucial that these conditions be well defined and monitored to confirm and maintain an understanding of the levee performance. The USACE investigation was comprehensive and detailed enough to establish good baseline conditions with respect to historical performance of the levee system, and also provided instruments that can be used to continue to assess current conditions. Arcadis recommends that the following activities be completed to further investigate and evaluate the levee site, subsurface, and loading conditions for the project reach.

- **Surveillance and Monitoring:** Develop and implement an updated surveillance and monitoring plan for the project reach. This plan should include details for monthly monitoring of the survey pins, inclinometers, and piezometers currently installed. Evaluation of the data should also be included, with threshold levels defined to detect adverse or worsening conditions. This plan should include careful tracking of information as it is collected and evaluated, and should be updated regularly as conditions change.
- **Subsurface Investigation:** Additional borings are needed to confirm material properties, to better determine the depth to the Pleistocene interface, and to install and/or replace instrumentation.
- **Instrumentation:** Additional instrumentation should be established for the project reach. Water levels from the river and from Lake Brown should be measured and recorded as part of the surveillance and monitoring program.

5.2 Evaluation of Alternatives

The geotechnical assessment indicates that Alternative I will not be the appropriate remediation strategy for the project reach, therefore Arcadis recommends that other alternatives be evaluated to remediate the levee embankment and reduce the possibility of future instability. The evaluation consists of conducting seepage/stability analyses for each of the recommended alternatives to determine which alternative improves the FOS to acceptable levels for all expected loading conditions. In addition, conceptual designs should be developed sufficiently to determine a rough order of magnitude cost assessment in order to compare the benefits and constructability of each alternative with respect to improving site conditions and levee performance.

Alternative II from the USACE report consisted of reinforcing the channel bank with a sheet pile wall. However, this solution does not appear to be viable because the sheet pile would cut-off seepage from Lake Brown and create destabilizing hydrostatic pressures in the slope. An additional alternative that also does not appear viable would be to buttress the slope with fill, rock gabion or mechanically stabilized earth (geogrids). These options would require filling in a portion of the Rio Grande River and move the centerline of the river away from the Brownsville side. Significant permitting, hydraulic and geotechnical analysis including additional borings would need to be undertaken to fully evaluate the alternative. Also, a buttress would need to be significantly armored so that it would not erode due to the velocities in the bend in the river. The recommended remediation alternatives include the following;

I. Alternative III from the USACE Investigation consists of increasing the resistance within the foundation section using soil mixing methods and techniques.

II. Similar to Alternative III from the USACE Investigation, increasing the resistance within the foundation section using stone columns is another alternative.

III. Another alternative would include removing the levee and replacing it with a concrete floodwall founded on a deep pile-supported, or similar foundations. However, this alternative may be problematic depending on where the slope movement terminates upslope. If the movement extends behind the existing levee, then this alternative may not be viable.

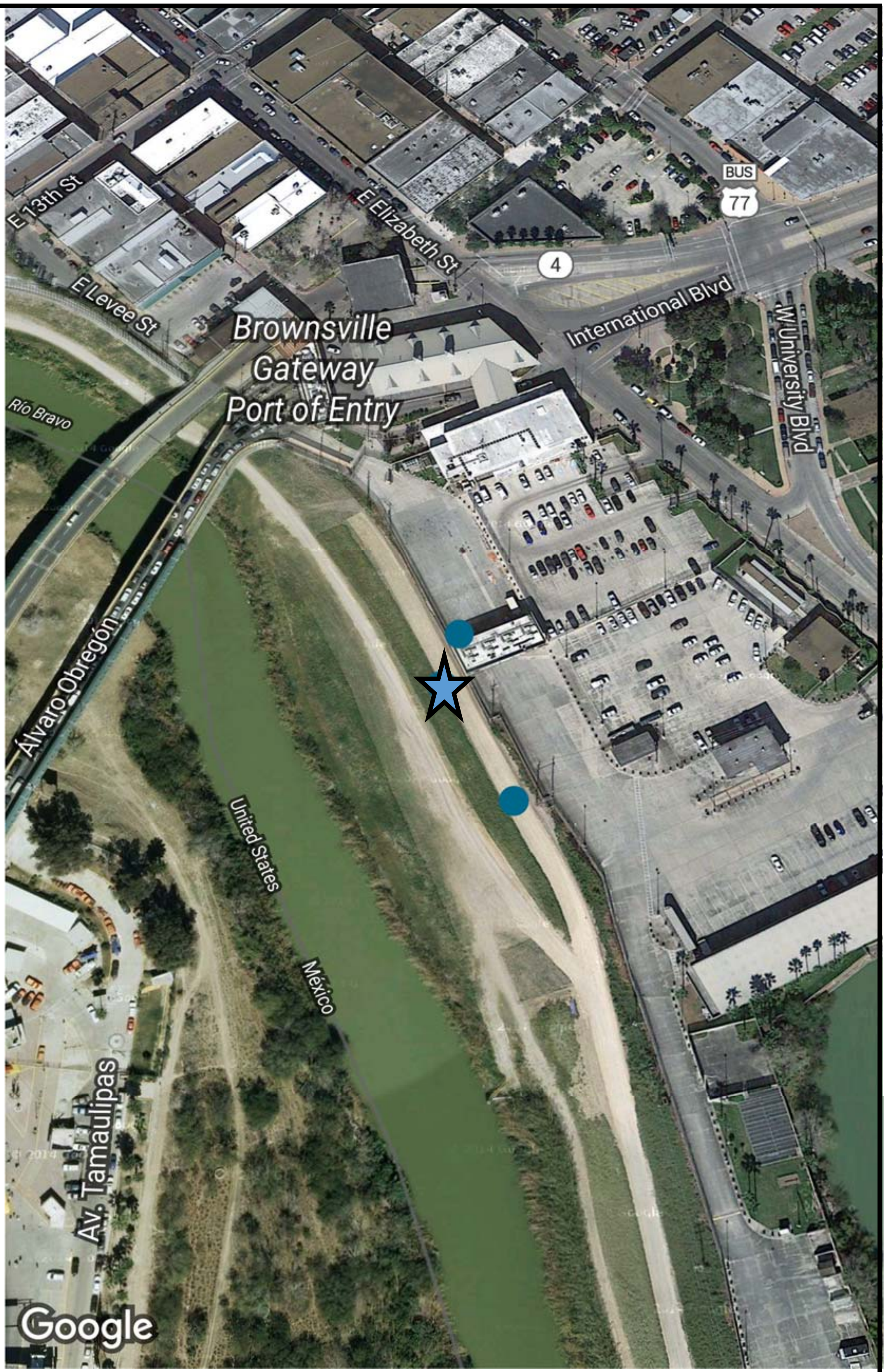
In addition, other possible remediation alternatives may be identified during the next phase of this project. If additional remediation alternatives, they will be similarly evaluated for rough order-of-magnitude costs, benefits, and constructability.

ATTACHMENT A

Site Visit Photos

Levee crest at around STA 1900+13

Levee toe at around STA 1900+13



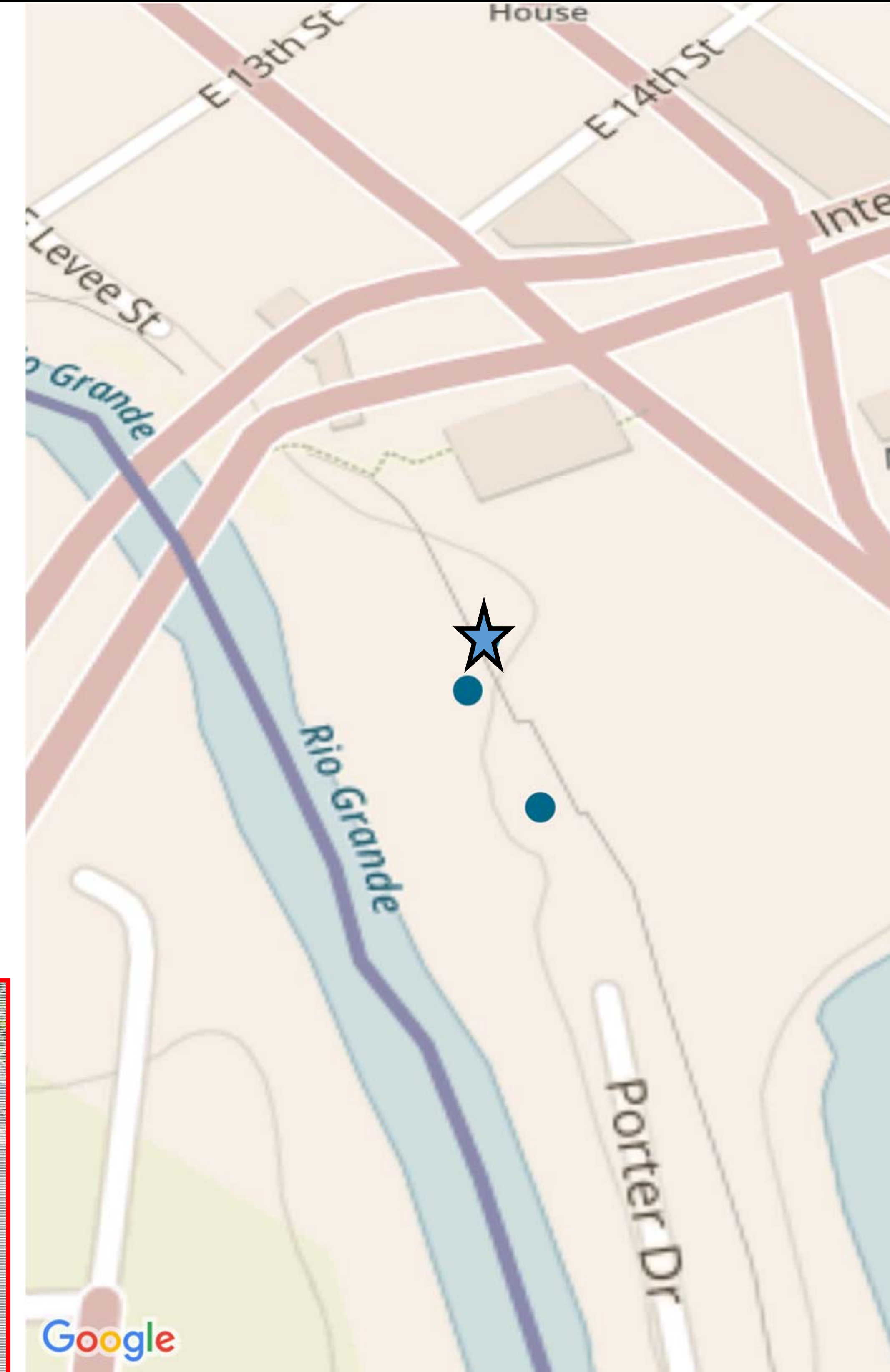
IBWC

60% GEOTECHNICAL ANALYSYS

Remediation Design of Levee Floodplain Failure
within the Upper Brownsville Levee Reach
Lower Rio Grande Flood Control Project
IBM15D0001 – IBM15T0015

JANUARY 29, 2015—SITE VISIT PHOTOS

ARCADIS Design & Consultancy for natural and built assets



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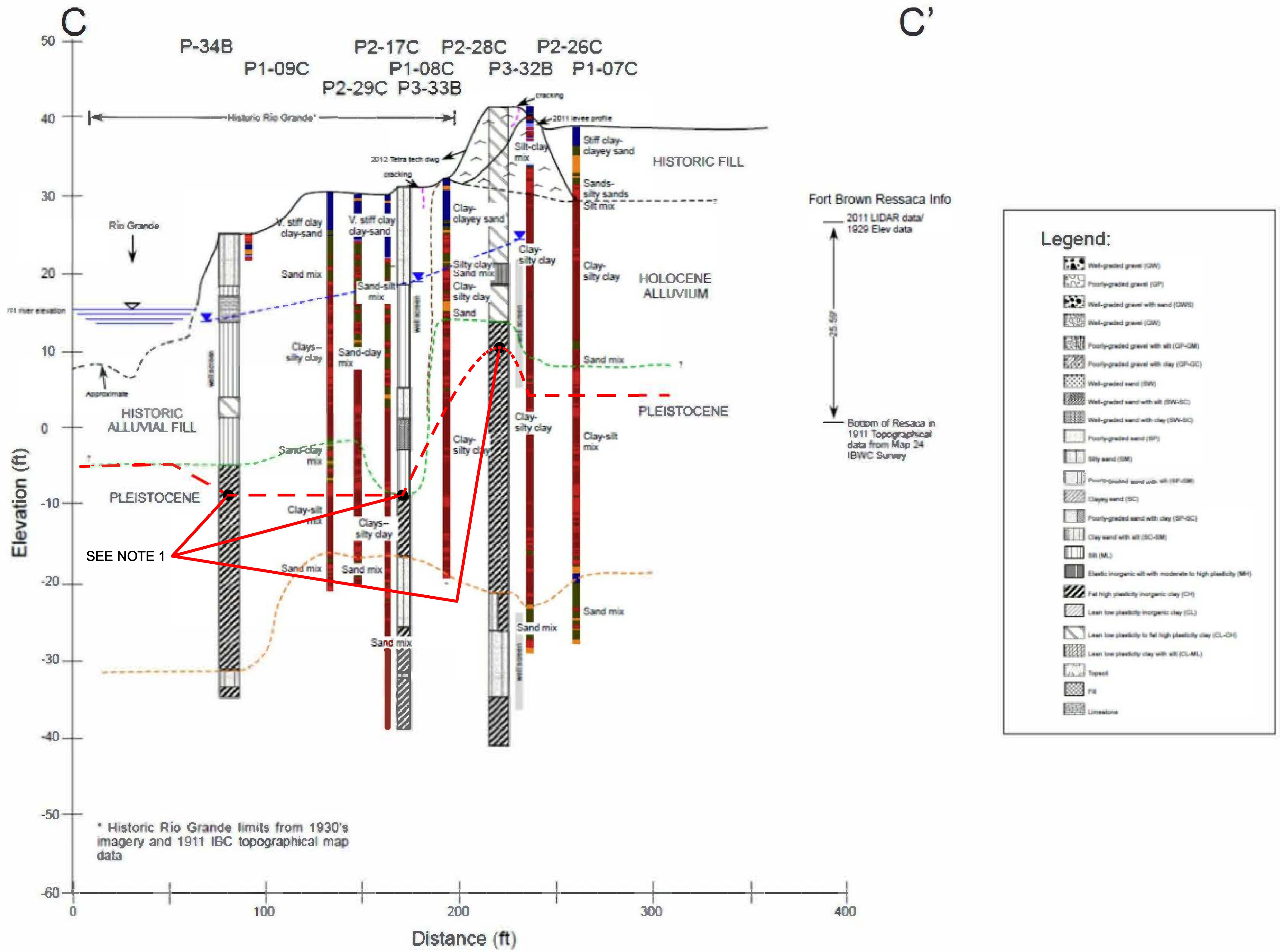
JANUARY 29, 2015—SITE VISIT PHOTOS

ARCADIS Design & Consultancy for natural and built assets

ATTACHMENT B

Geologic Cross Sections



LEGEND:

POSSIBLE PLEISTOCENE INTERFACE

SOURCE:

THIS DRAWING WAS RECREATED FROM FIGURE 5.7 OF THE USACE GEOTECHNICAL EVALUATION OF THE BROWNSVILLE LEVEE CRACKING AND PARTIAL SLOPE FAILURE REPORT DATED JULY 2015.

NOTES:

1. INTERPRETED INTERFACE FOR BORINGS P-34B, P-33B AND P-32B COME DIRECTLY FROM THE BORING LOGS.
2. INTERPRETED INTERFACE FOR THE CPTs COMES FROM CROSS SECTION R1 (CPT PREDICTED SOIL CLASSIFICATION AND CPT PREDICTED STRENGTH) FROM THE USACE REPORT.

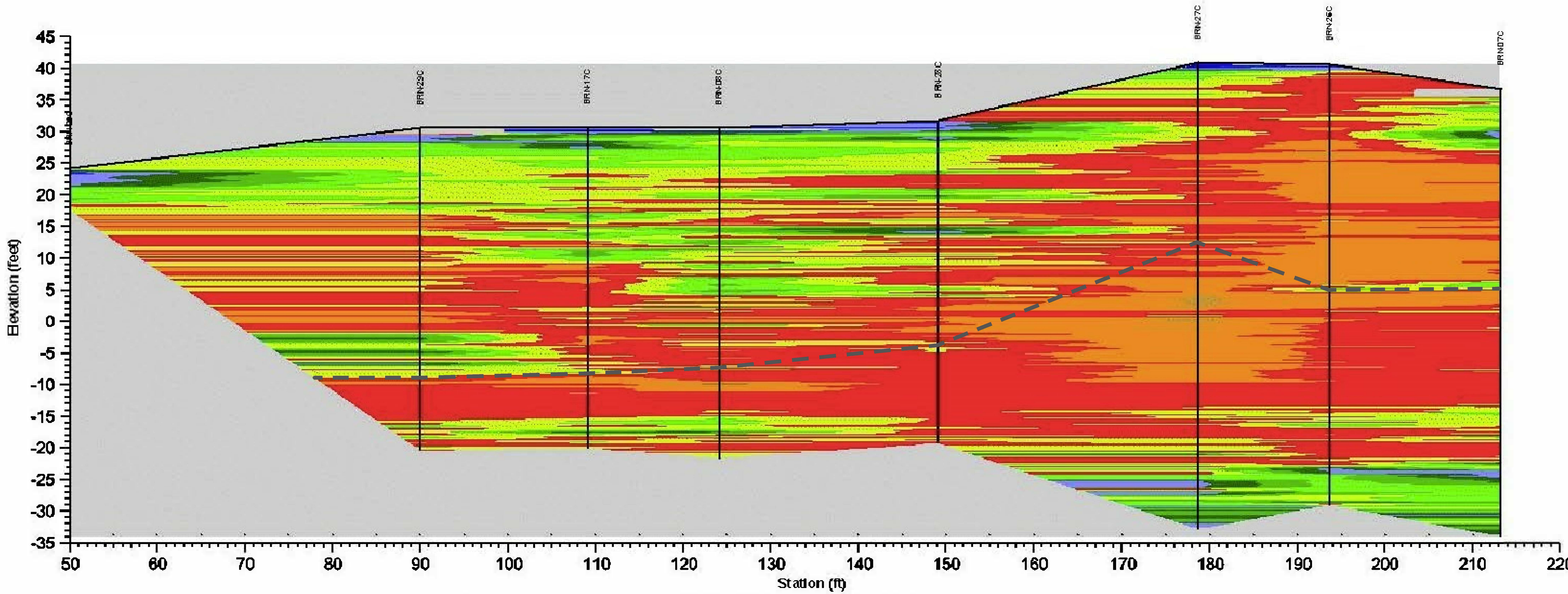
NOT TO SCALE

IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GEOLOGICAL CROSS SECTION C-C' STATION 1900+13

ARCADIS | I

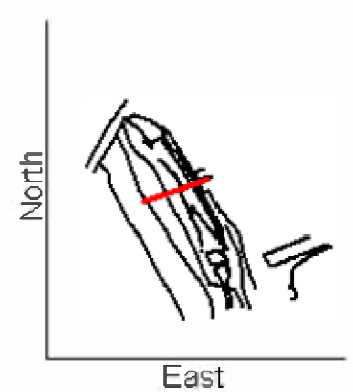


LEGEND:

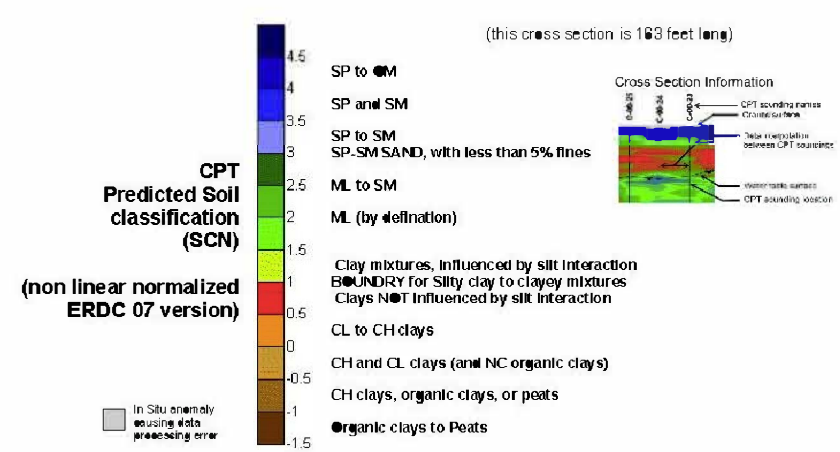
POSSIBLE PLEISTOCENE INTERFACE

SOURCE:

THIS DRAWING WAS RECREATED FROM THE CROSS SECTION R1 (CPT PREDICTED SOIL CLASSIFICATION) OF THE USACE GEOTECHNICAL EVALUATION OF THE BROWNSVILLELEVEE CRACKING AND PARTIAL SLOPE FAILURE REPORT DATED JULY 2015.



Cross Section:
R1



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PRELIMINARY GEOTECHNICAL MEMORANDUM

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

CROSS SECTION:R1 STATION 1900+13

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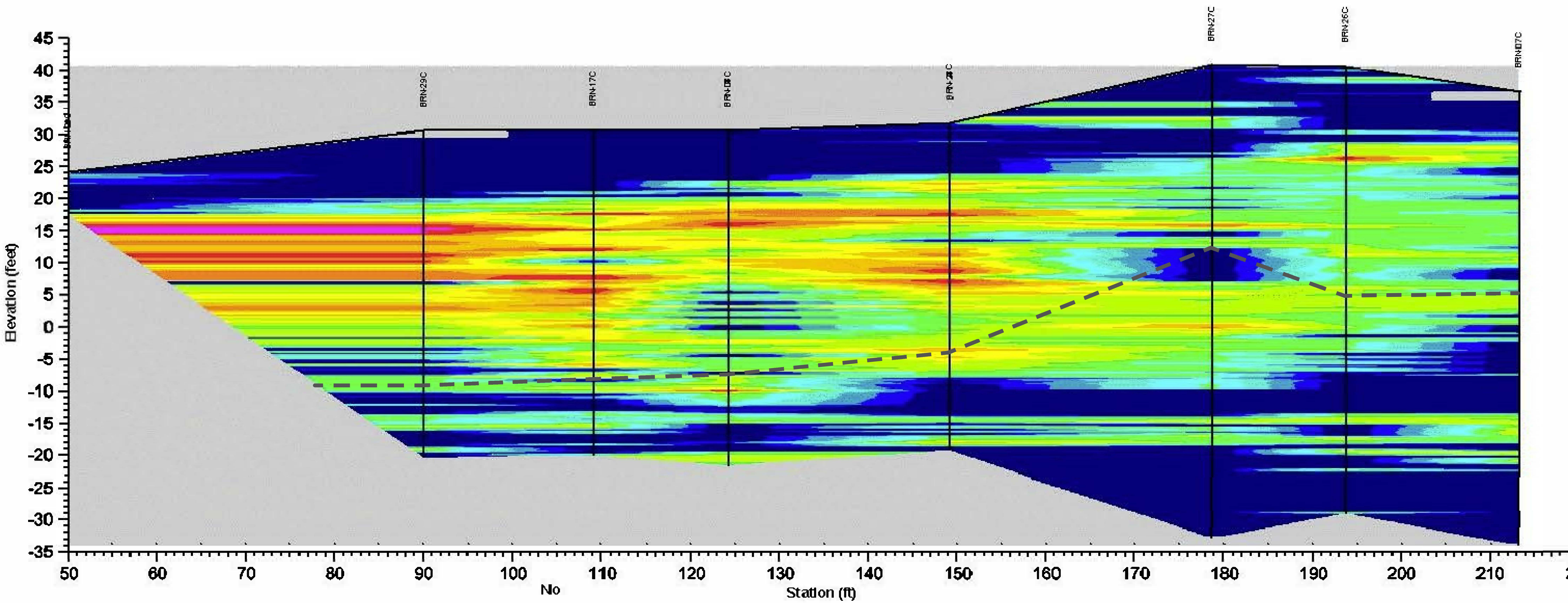
II

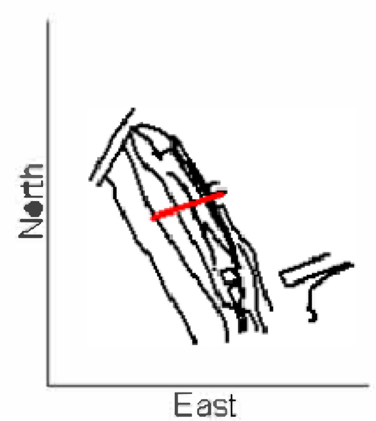


LEGEND:

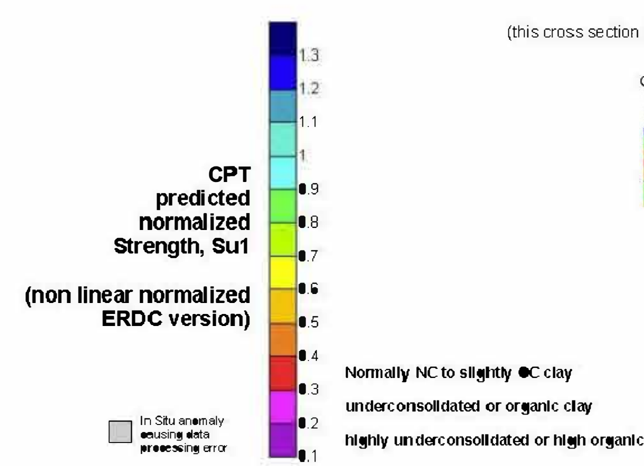
POSSIBLE PLEISTOCENE INTERFACE

SOURCE:

THIS DRAWING WAS RECREATED FROM THE CROSS SECTION R1 (CPT PREDICTED STRENGTH) OF THE USACE GEOTECHNICAL EVALUATION OF THE BROWNSVILLELEVEE CRACKING AND PARTIAL SLOPE FAILURE REPORT DATED JULY 2015.

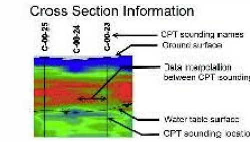


Cross Section:
R1



(this cross section is 163 feet long)

Cross Section Information



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PRELIMINARY GEOTECHNICAL MEMORANDUM

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

CROSS SECTION: R1 STATION 1900+13

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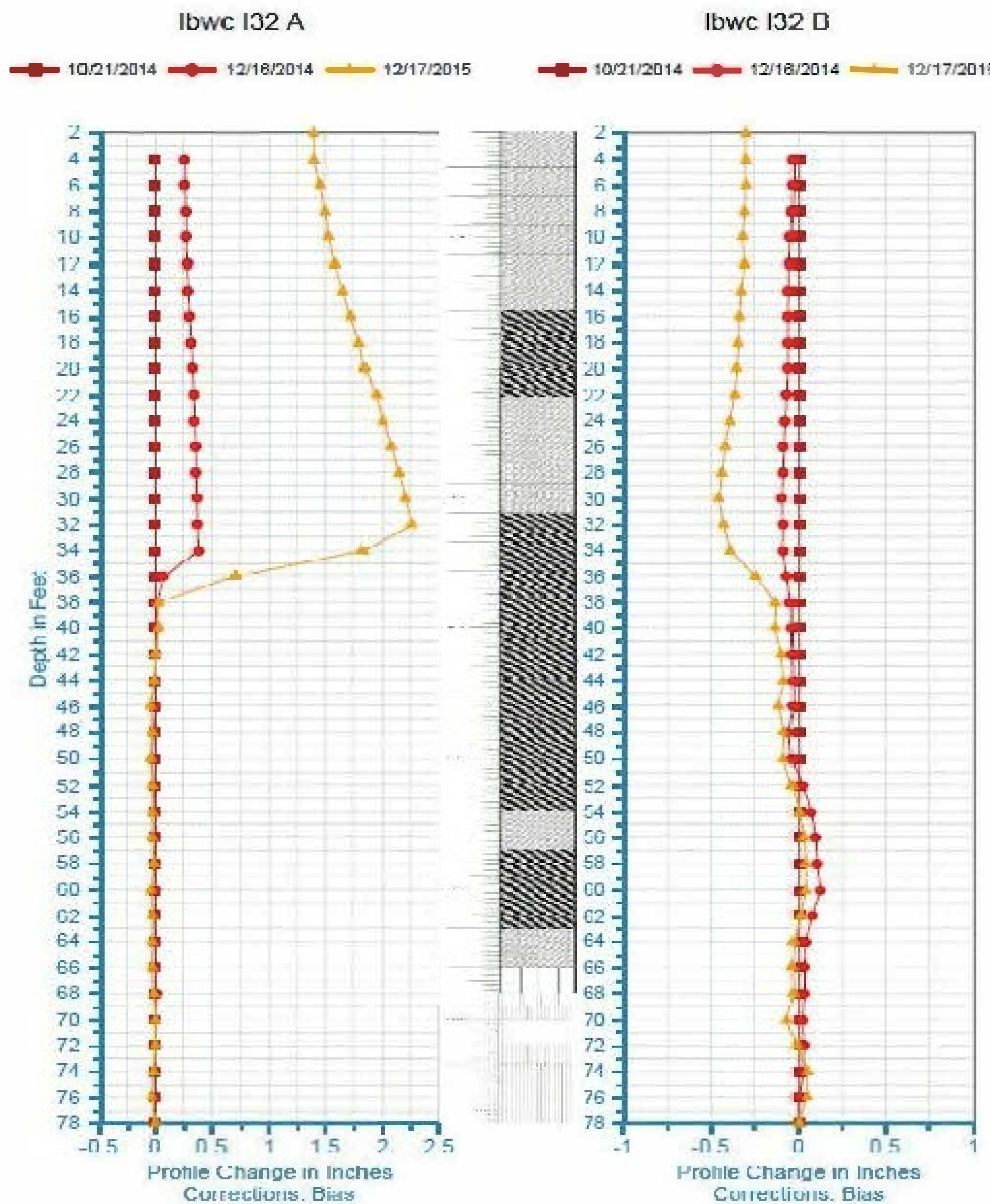
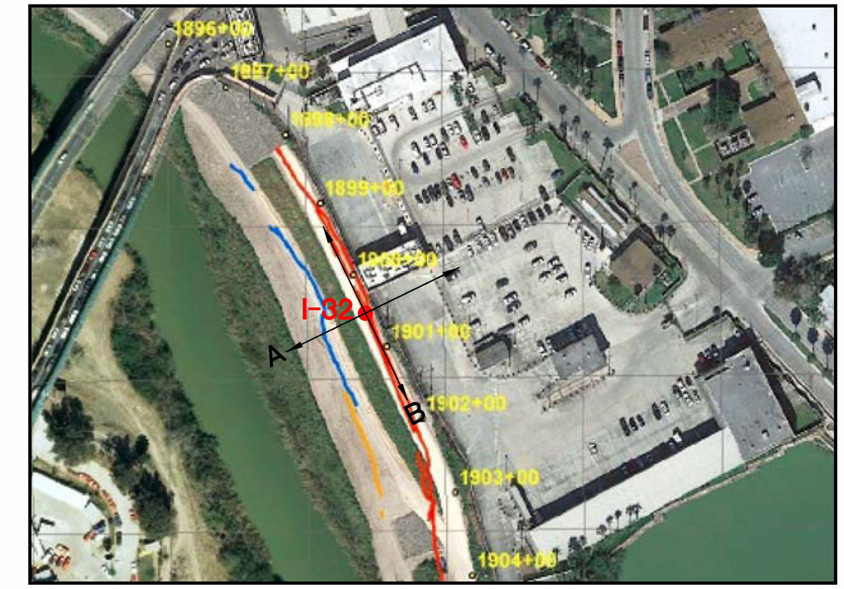
III

ATTACHMENT C

Inclinometer Plates





LOCATION MAP

SOURCE:

THIS LOCATION MAP WAS RECREATED FROM FIGURE 4.1 OF THE USACE GEOTECHNICAL EVALUATION OF THE BROWNSVILLE LEVEE CRACKING AND PARTIAL SLOPE FAILURE REPORT DATED JULY 2015.

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PRELIMINARY GEOTECHNICAL MEMORANDUM

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

INCLINOMETER I-32 STATION 1900+13

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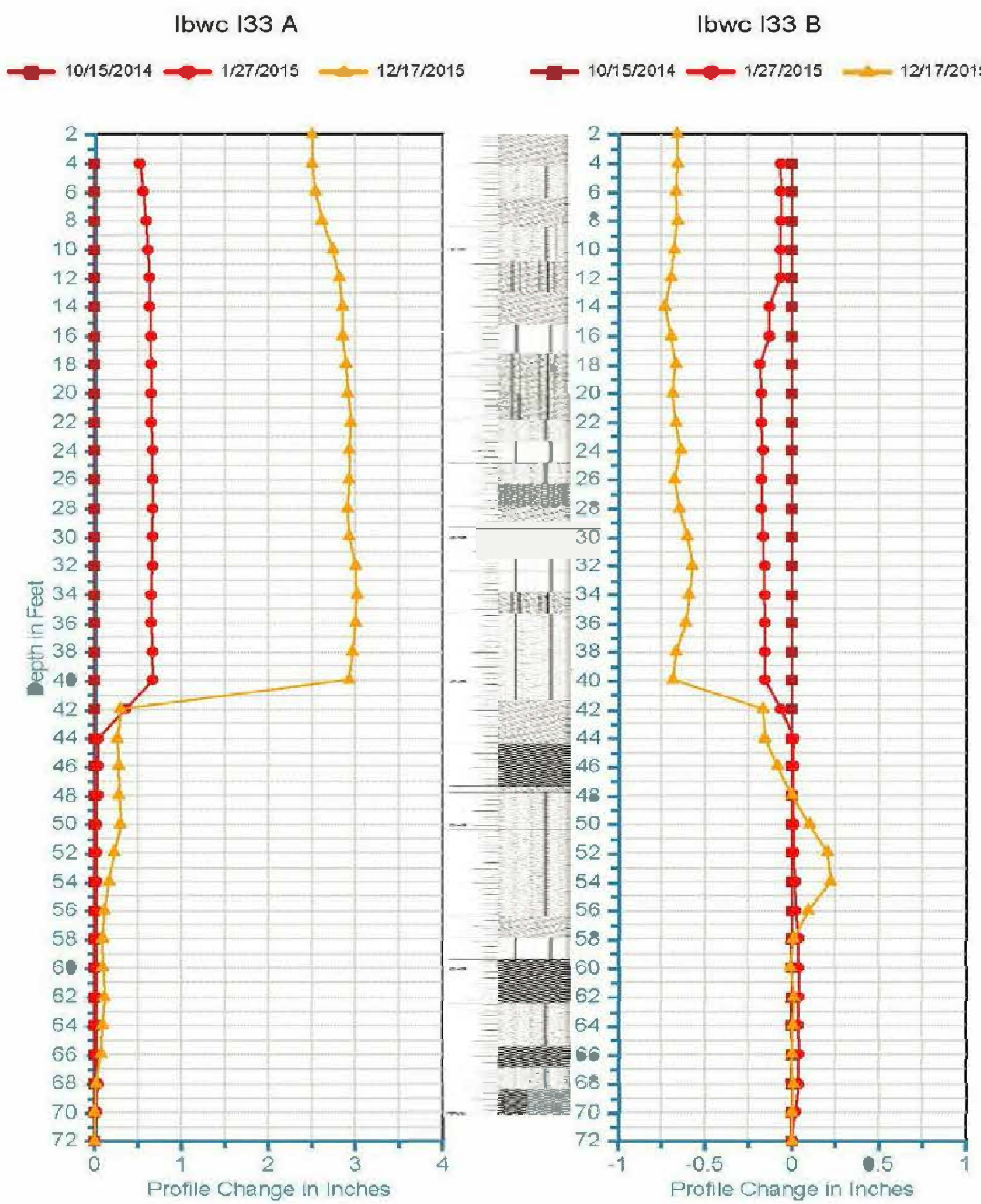
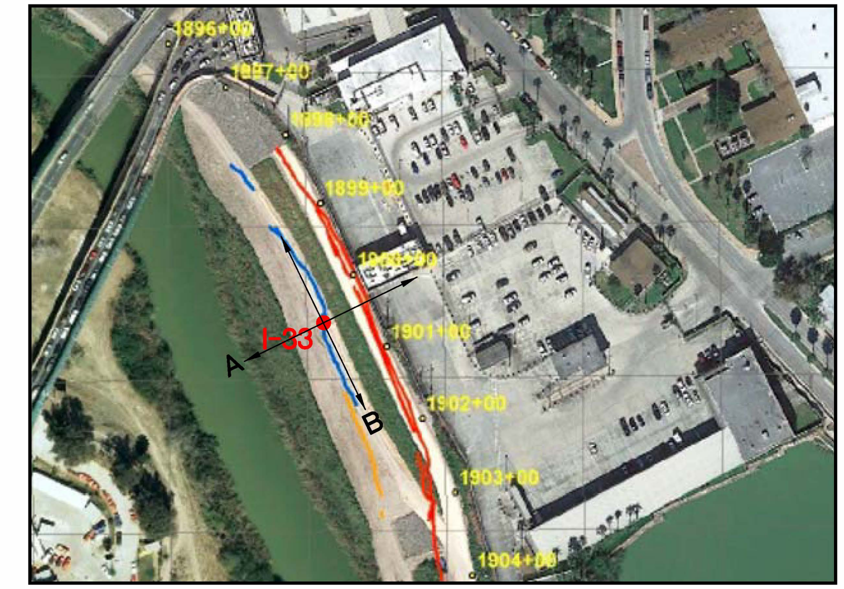
PLATE
1





LOCATION MAP

SOURCE:

THIS LOCATION MAP WAS RECREATED FROM FIGURE 4.1 OF THE USACE GEOTECHNICAL EVALUATION OF THE BROWNSVILLE LEVEE CRACKING AND PARTIAL SLOPE FAILURE REPORT DATED JULY 2015.

NOT TO SCALE

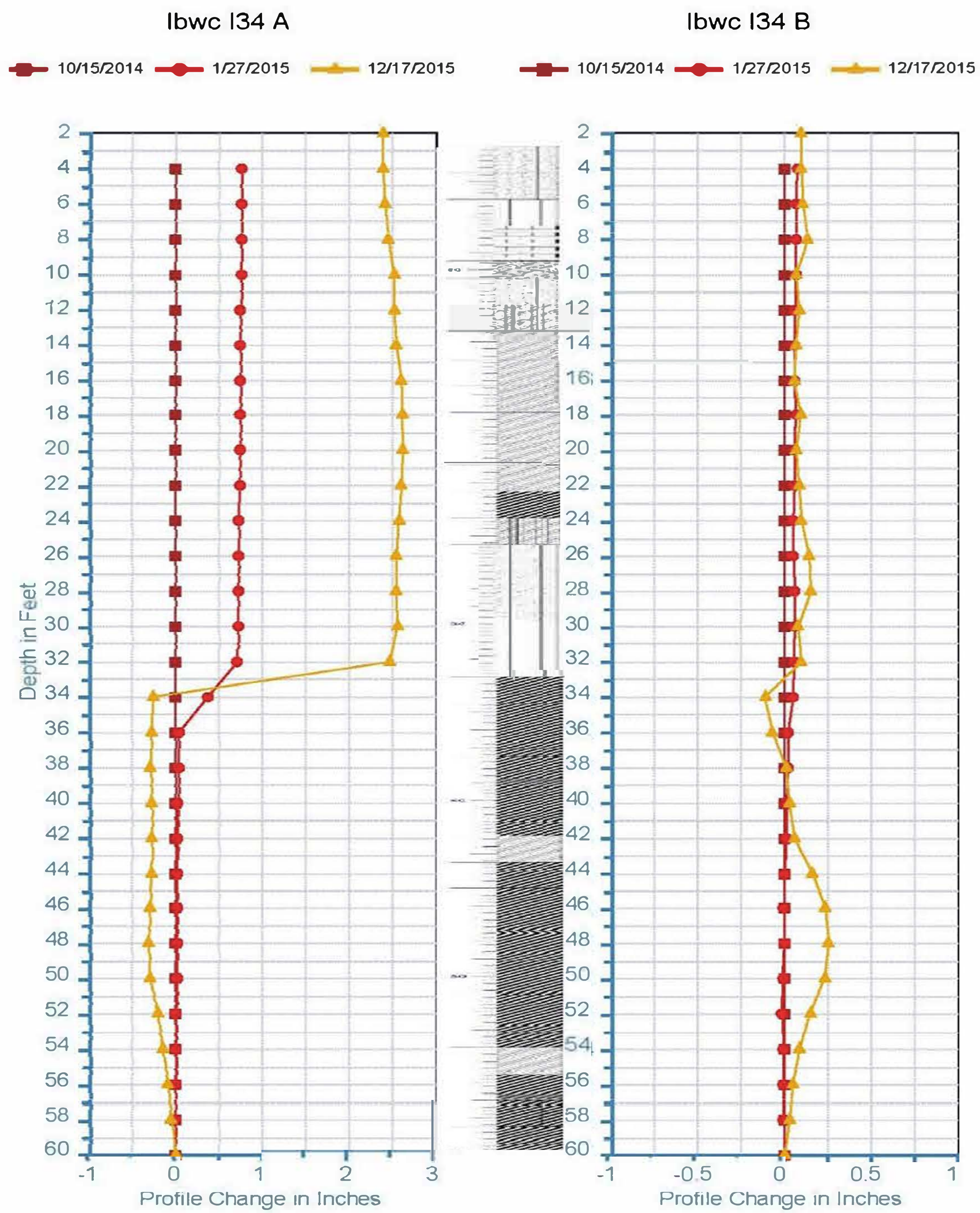
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

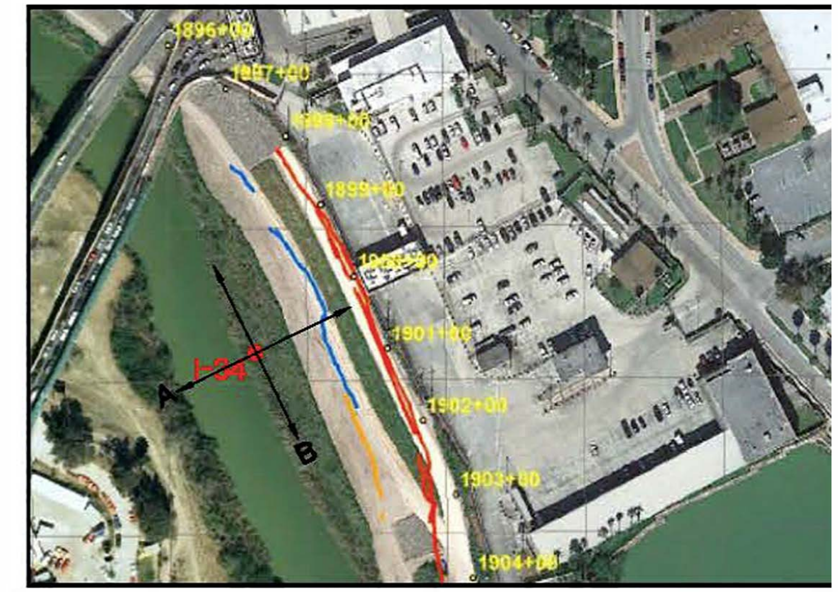
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

INCLINOMETER I-33 STATION 1900+13

ARCADIS

PLATE 2





LOCATION MAP

SOURCE:

THIS LOCATION MAP WAS RECREATED FROM FIGURE 4.1 OF THE USACE GEOTECHNICAL EVALUATION OF THE BROWNSVILLE LEVEE CRACKING AND PARTIAL SLOPE FAILURE REPORT DATED JULY 2015.

NOT TO SCALE

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PRELIMINARY GEOTECHNICAL MEMORANDUM

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

INCLINOMETER I-34 STATION 1900+13

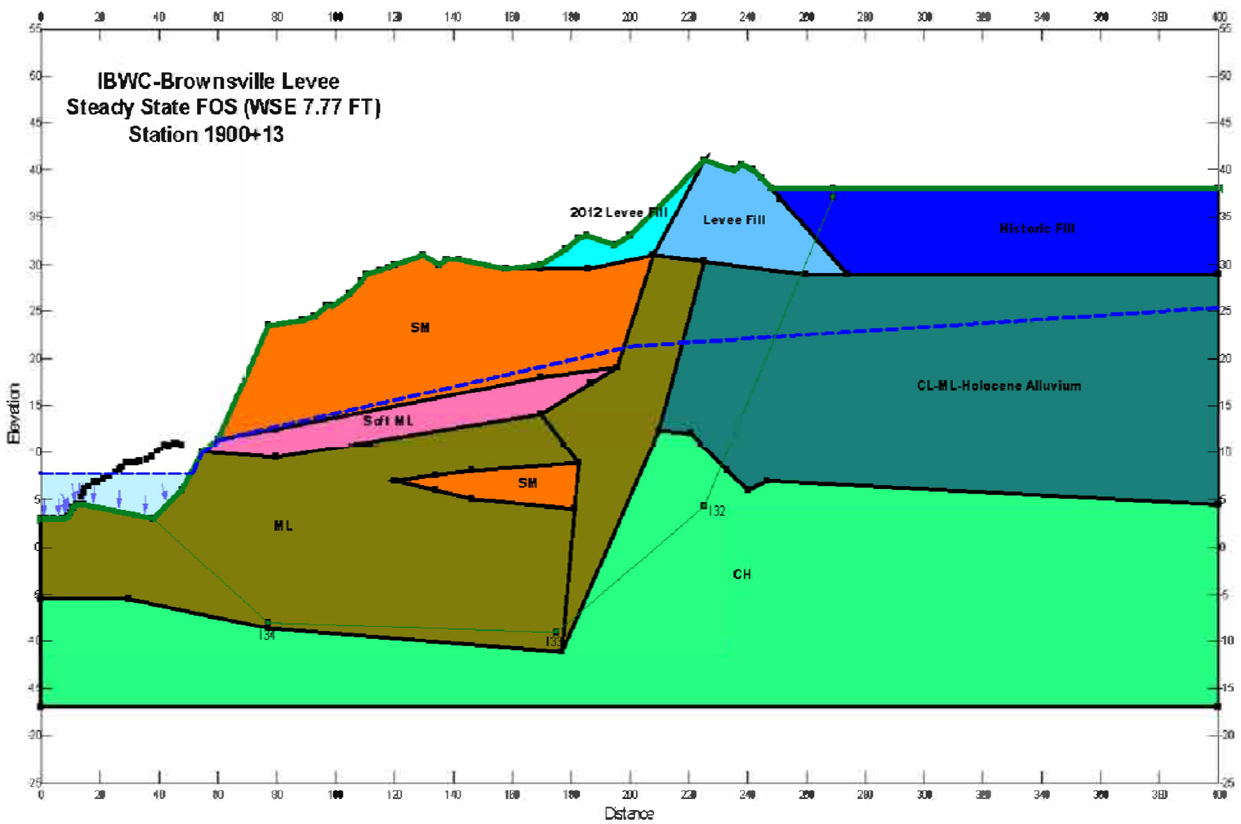
ARCADIS

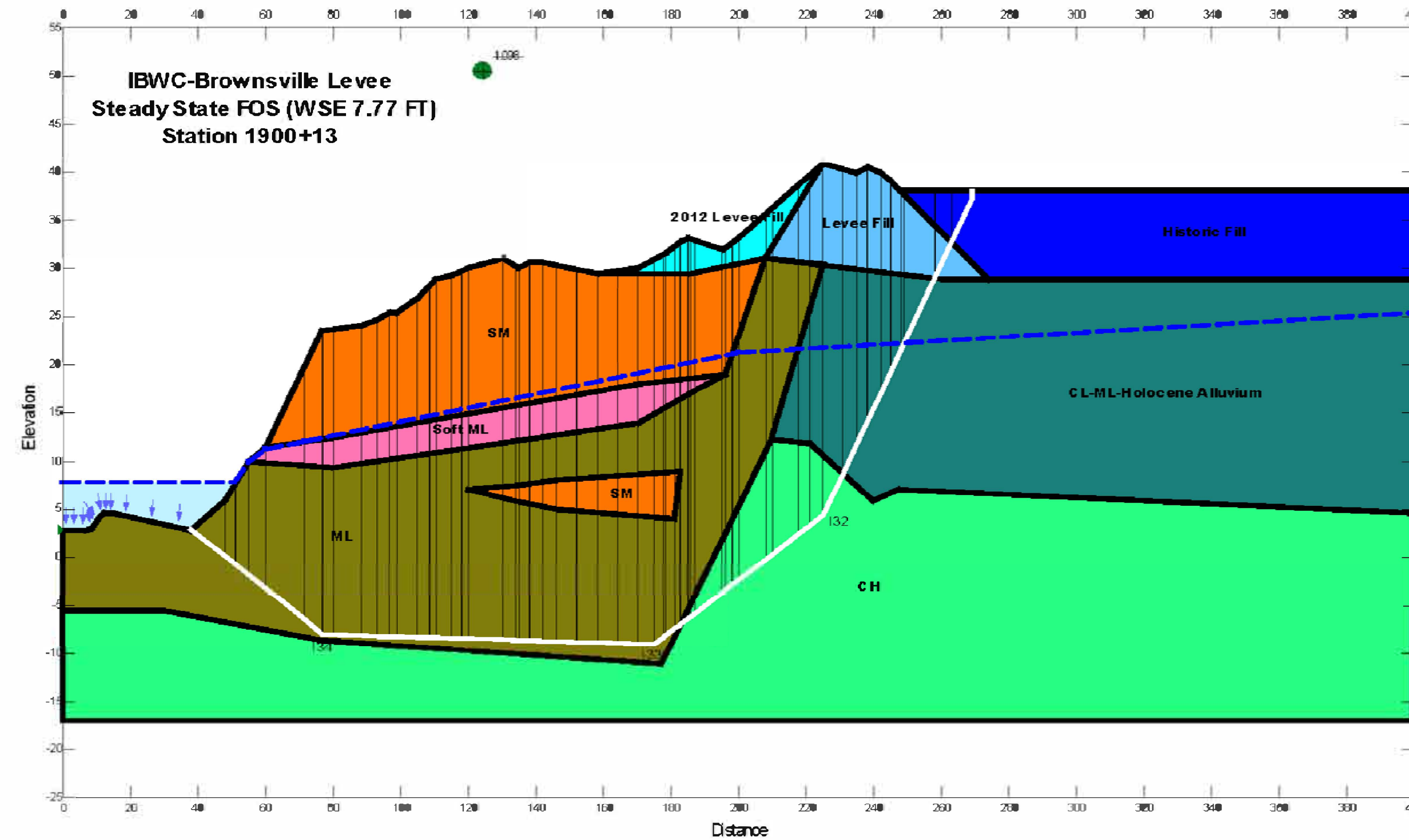
PLATE
3

IB: R. PETRIE S. BELL LD: SOTHONMEN PIC:(Opt) PM:(Reqd) TM:(Opt) LYR:(Opt)ON=*;OFF=*REF*
SAVED: 2/18/2016 5:29 PM ACADVER: 18.1S (LMS TECH) PAGESETUP: — PLOTSTYLETABLE: — PLOTTED: 2/19/2016 11:37 AM BY: FLORES, ARMANDO
LAYOUT: I-34

ATTACHMENT D

Slope Stability Analyses





Minimum Factor of Safety (FOS): 1.098

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1			

NOT TO SCALE

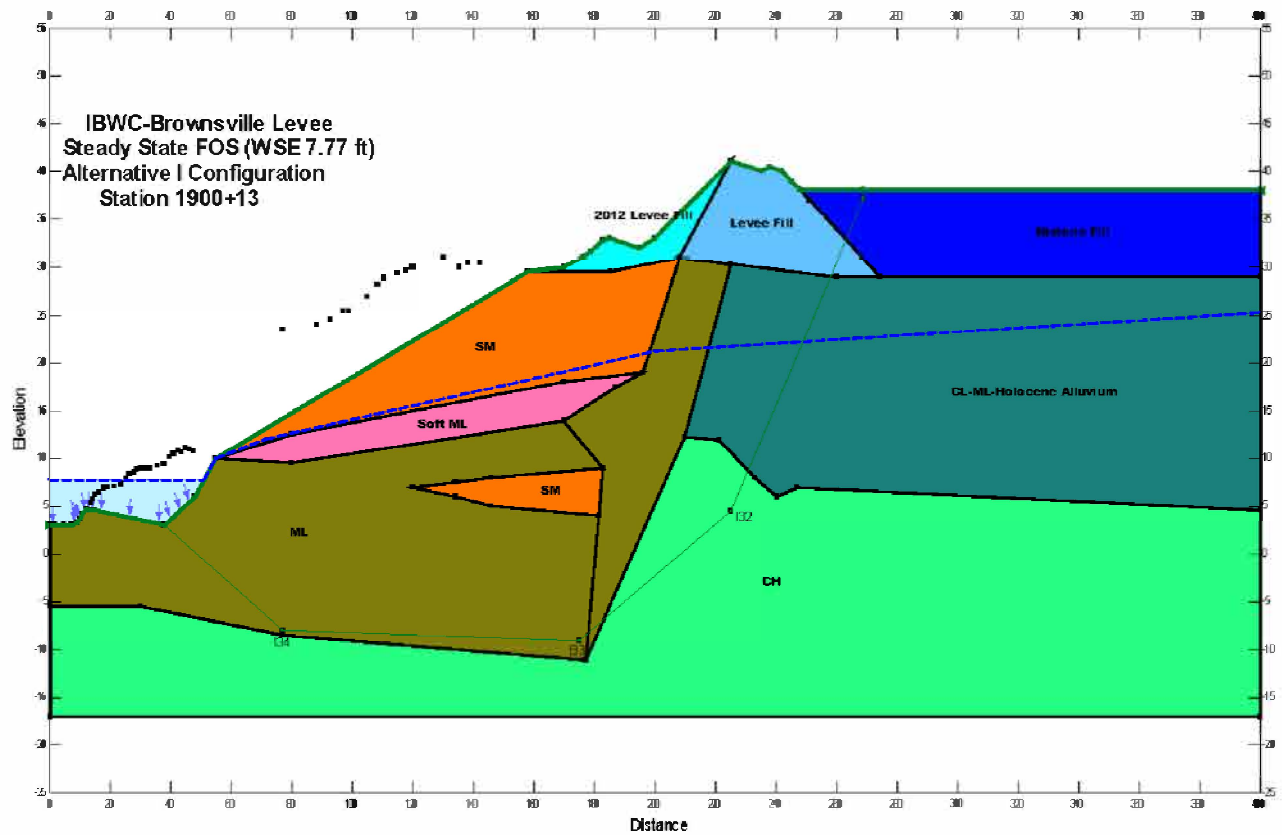
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

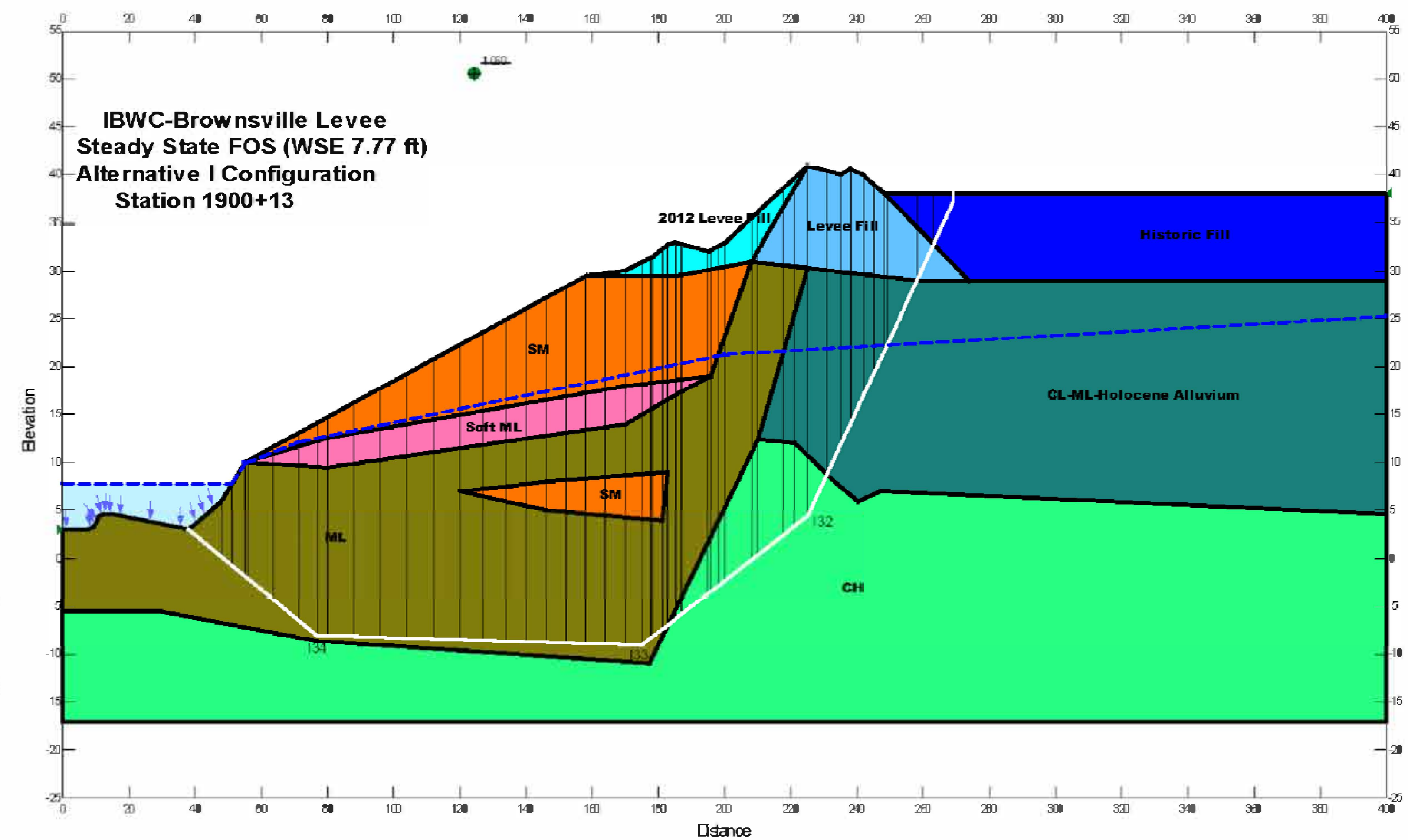
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - EXISTING SLOPE
STEADY STATE SEEPAGE

ARCADIS

SS-1





Minimum Factor of Safety (FOS): 1.060

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
Combination 1			

NOT TO SCALE

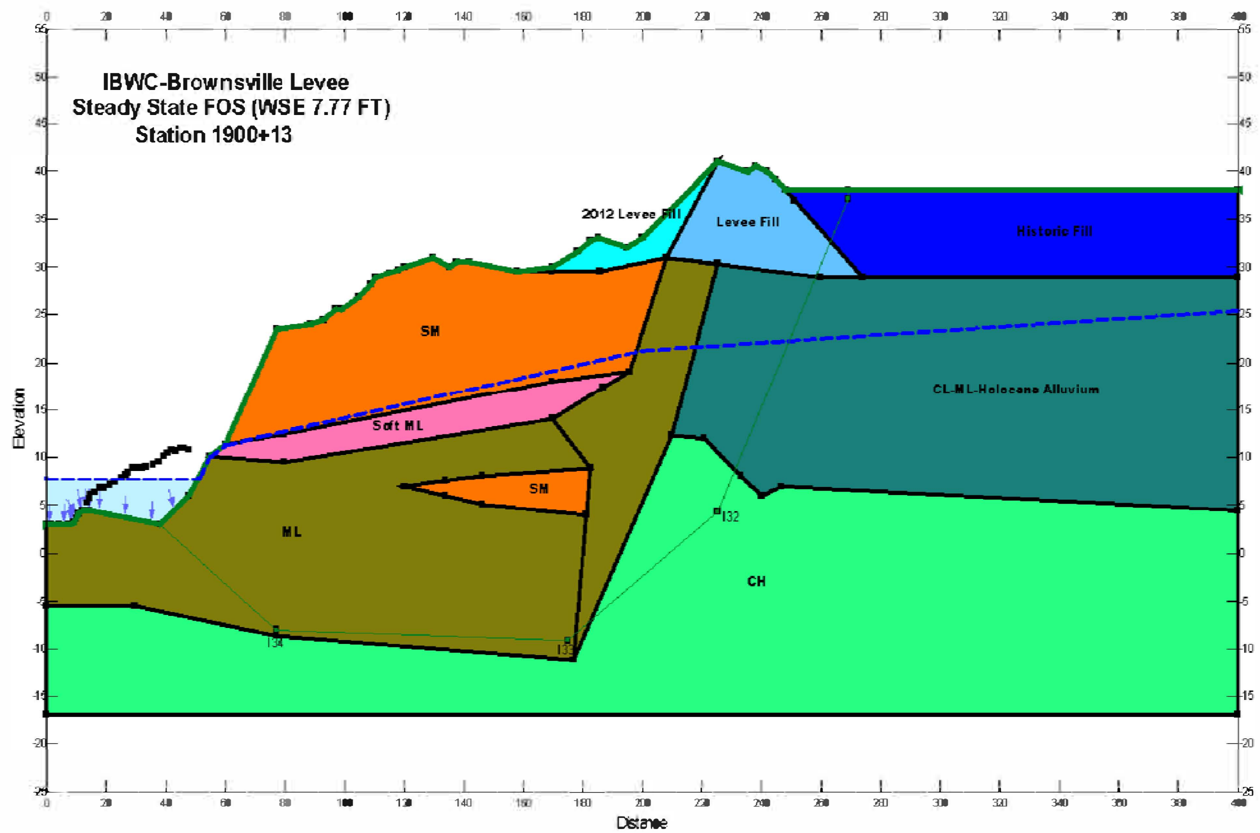
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

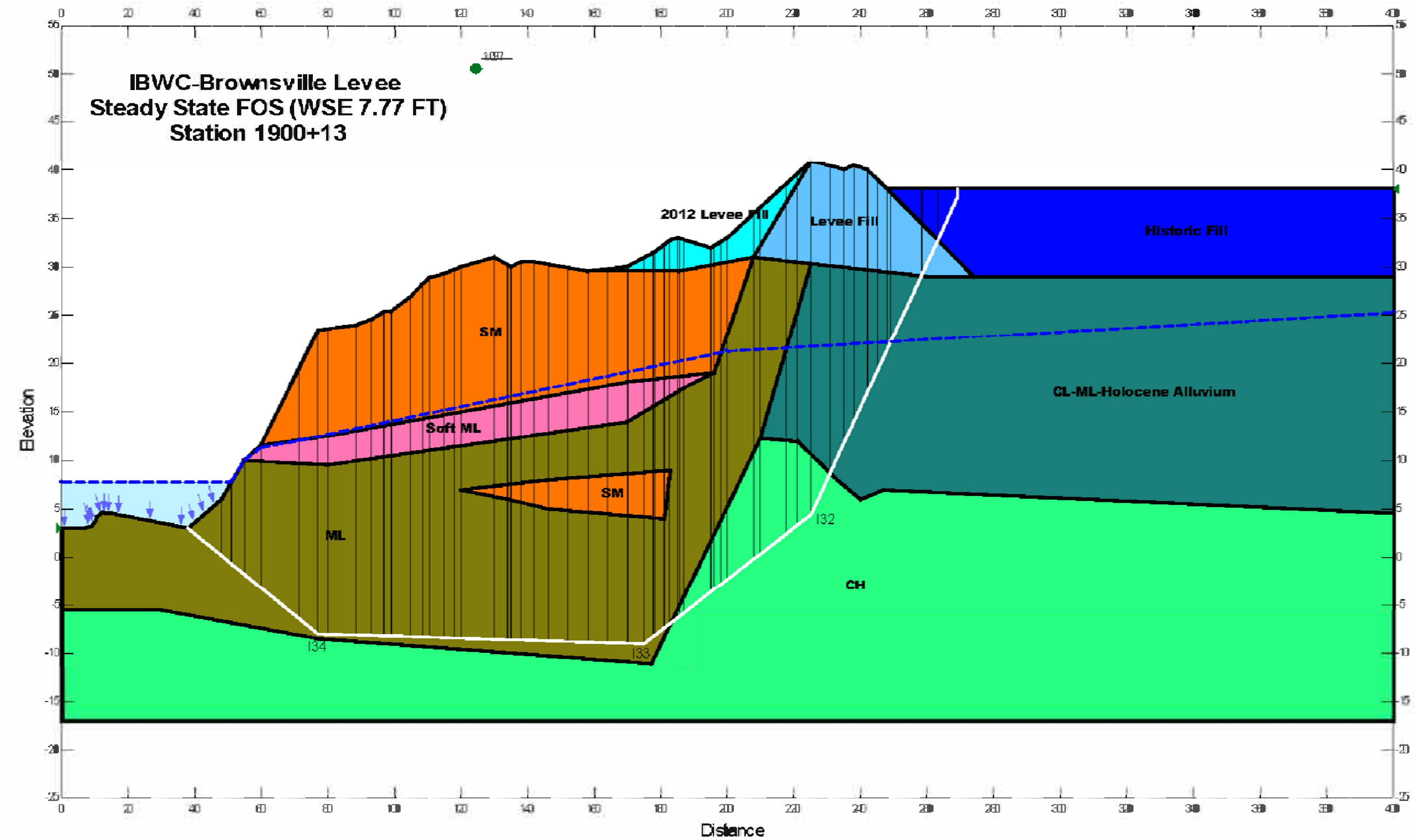
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - ALTERNATIVE 1
STEADY STATE SEEPAGE

ARCADIS

SS-2





Minimum Factor of Safety (FOS): 1.097

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8

Combination 2: The shear strenght of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1

NOT TO SCALE

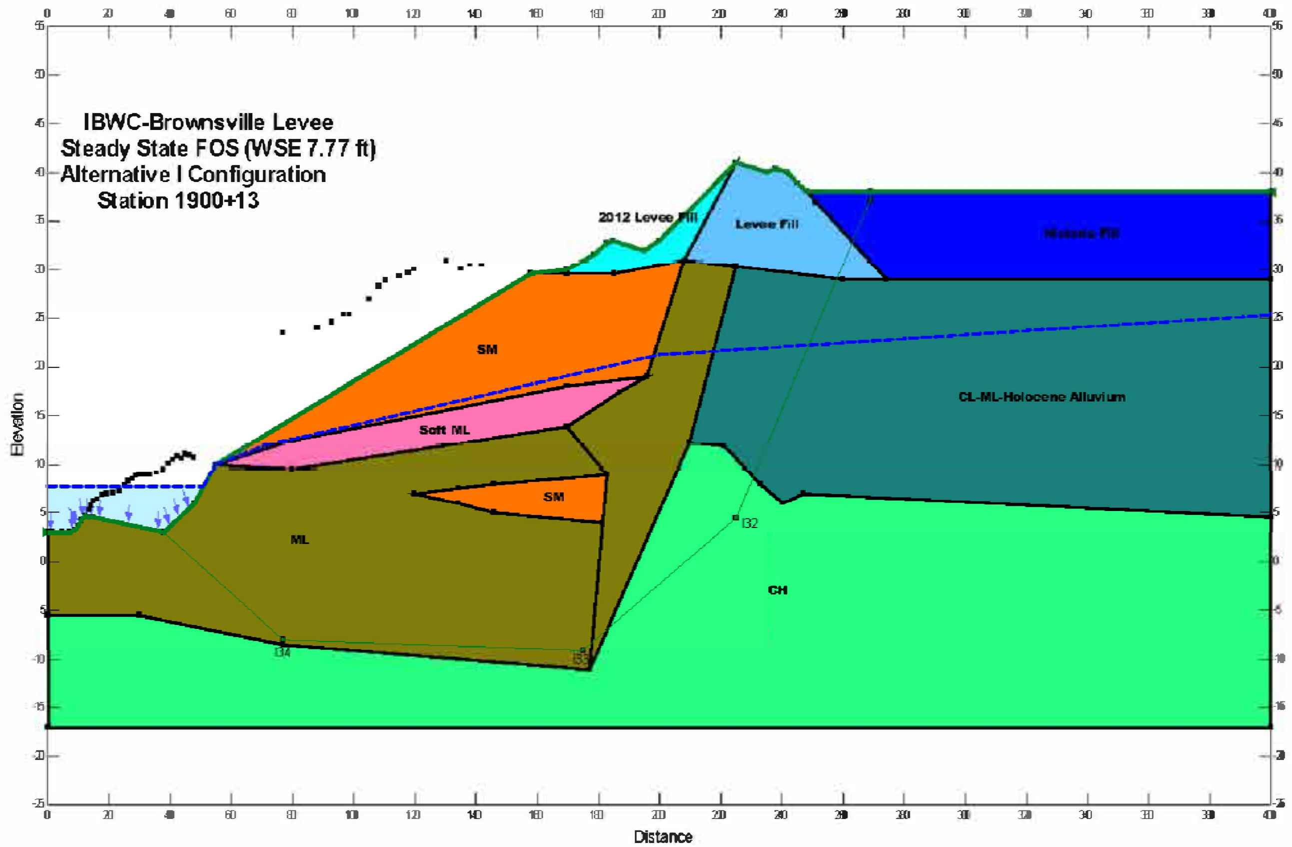
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

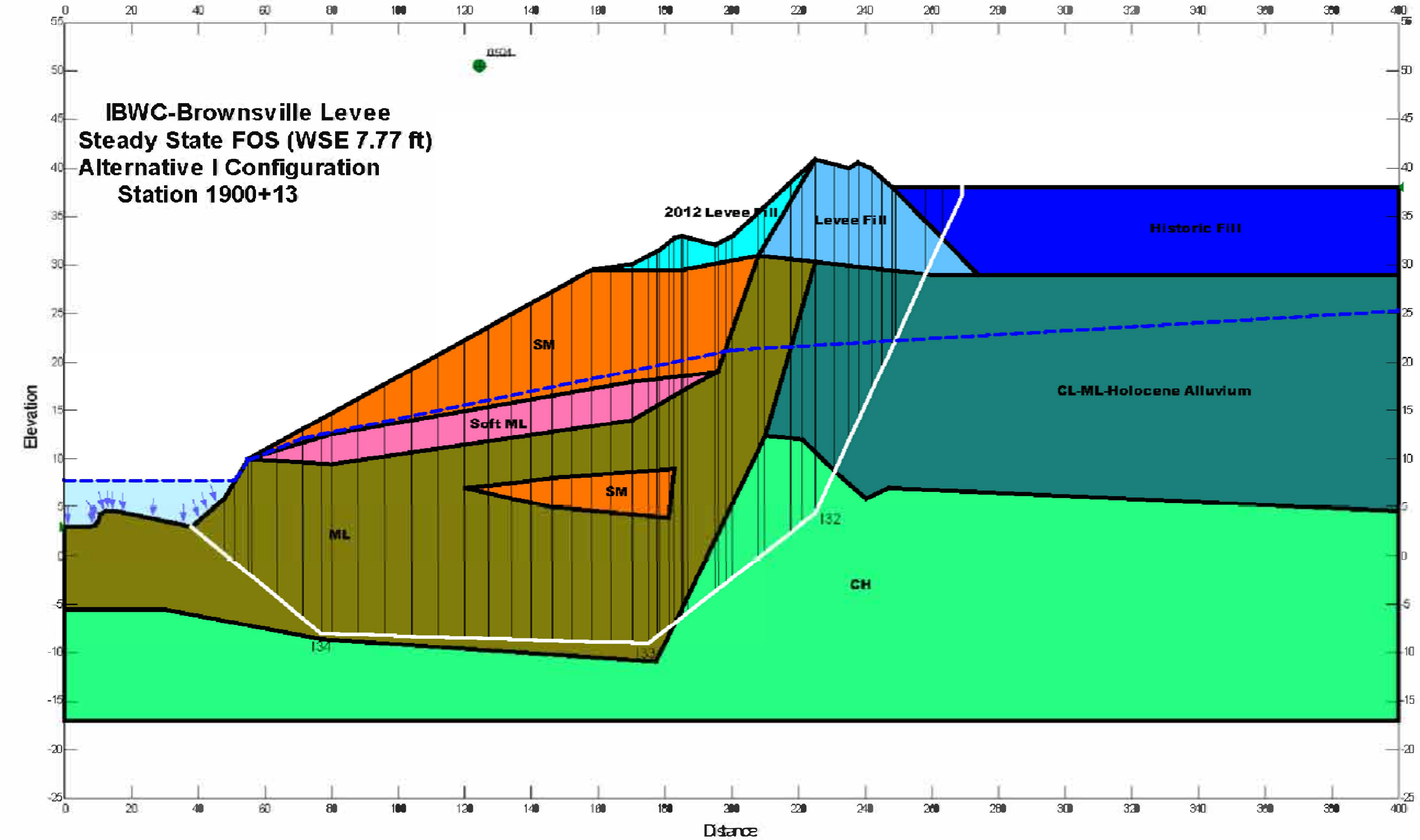
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - EXISTING SLOPE
STEADY STATE SEEPAGE

ARCADIS

SS-3





Minimum Factor of Safety (FOS): 0.924

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
Combination 2			

NOT TO SCALE

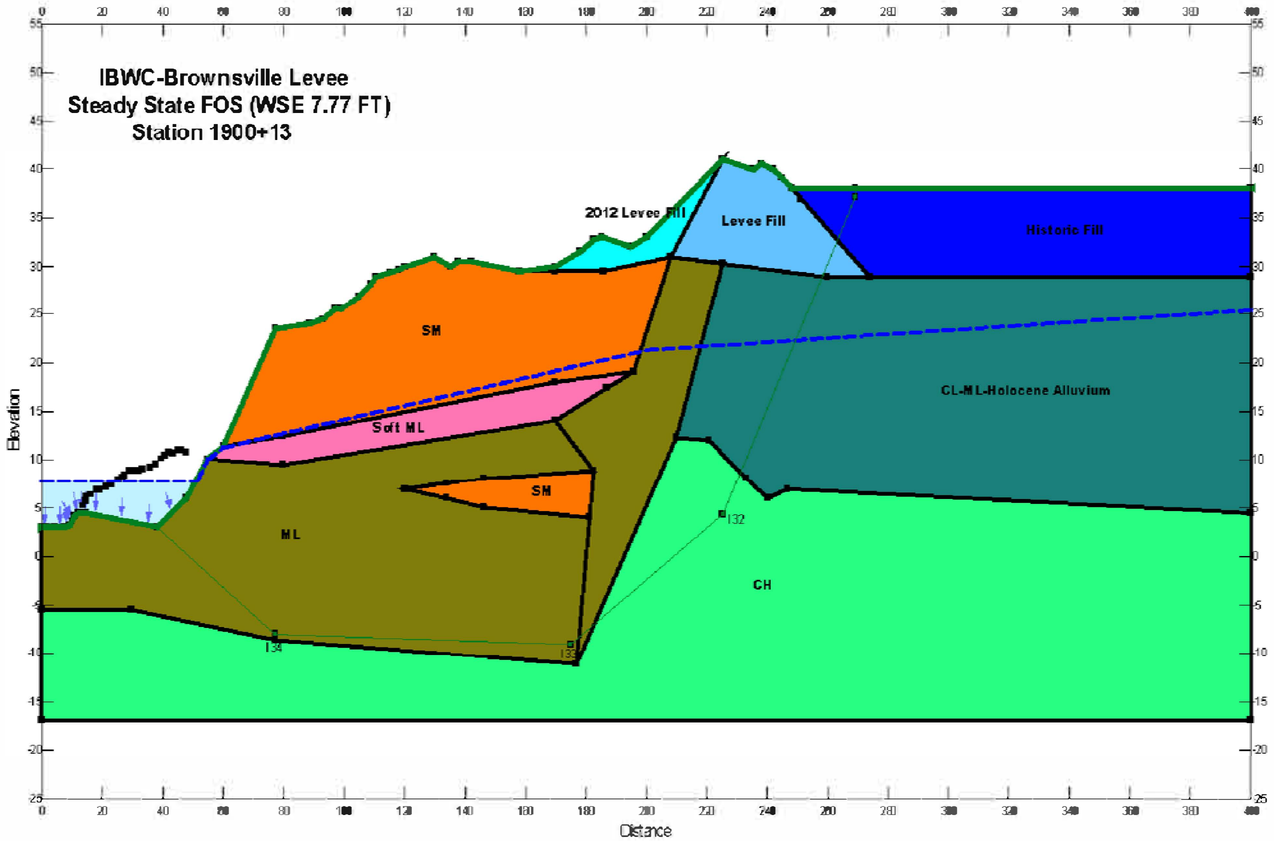
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

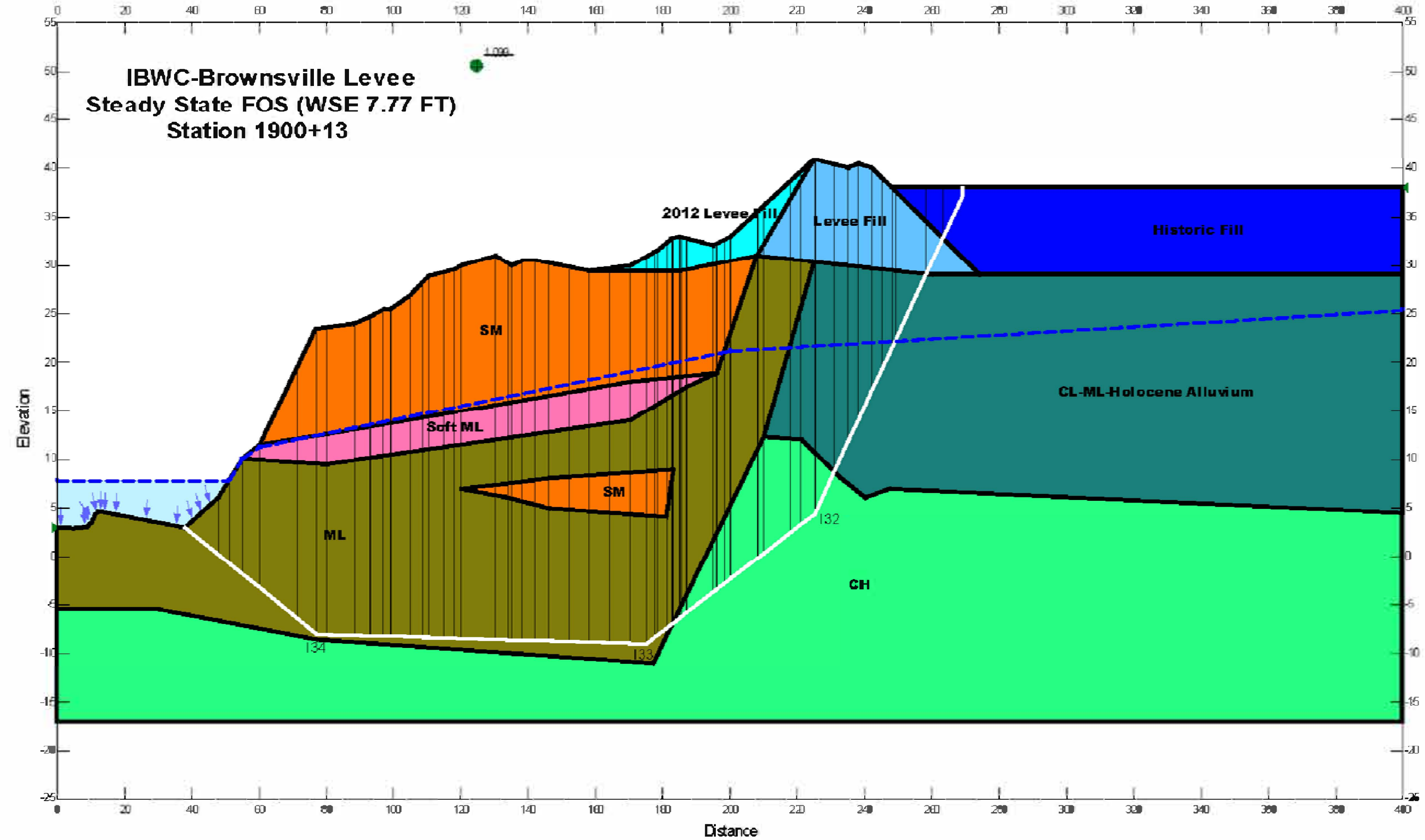
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - ALTERNATIVE 1
STEADY STATE SEEPAGE

ARCADIS

SS-4





Minimum Factor of Safety (FOS): 1.099

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

NOT TO SCALE

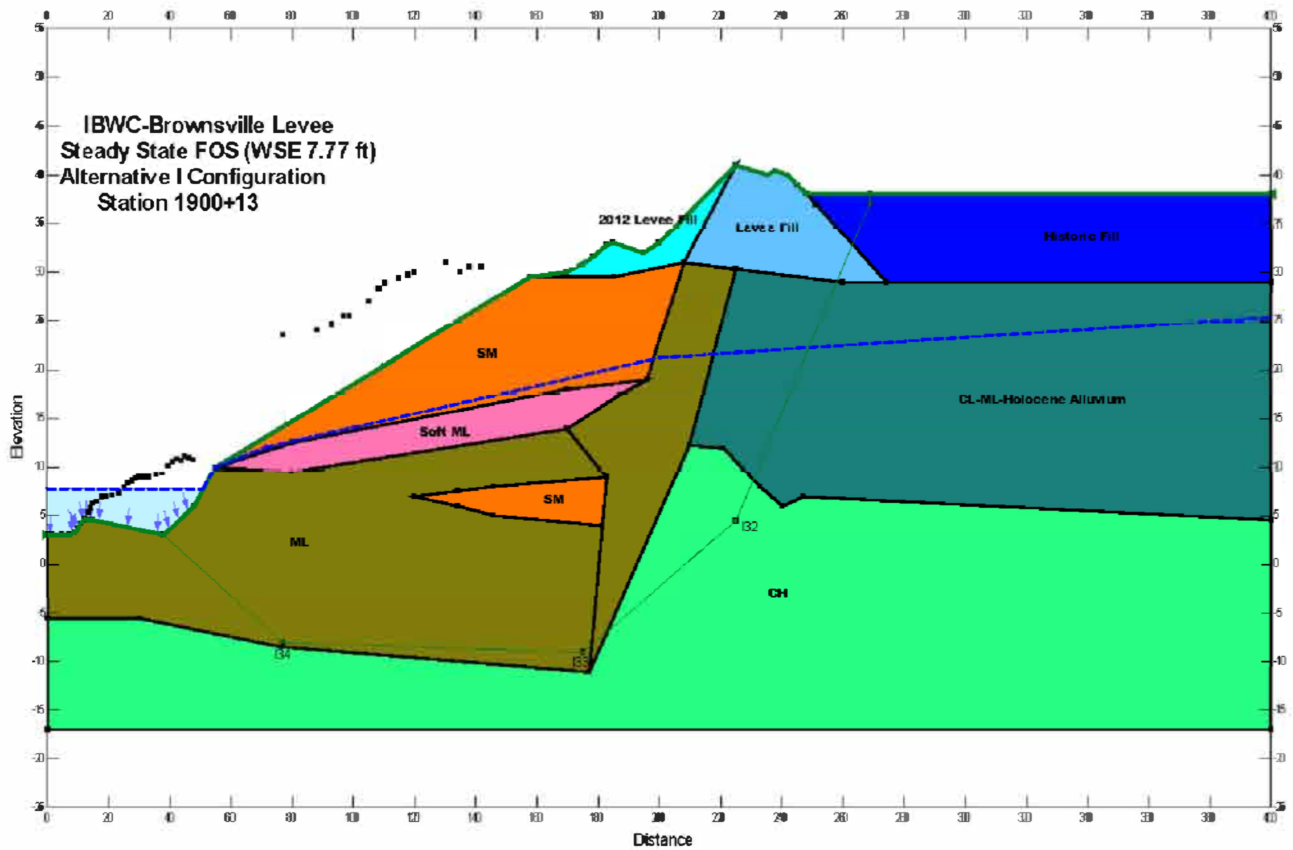
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

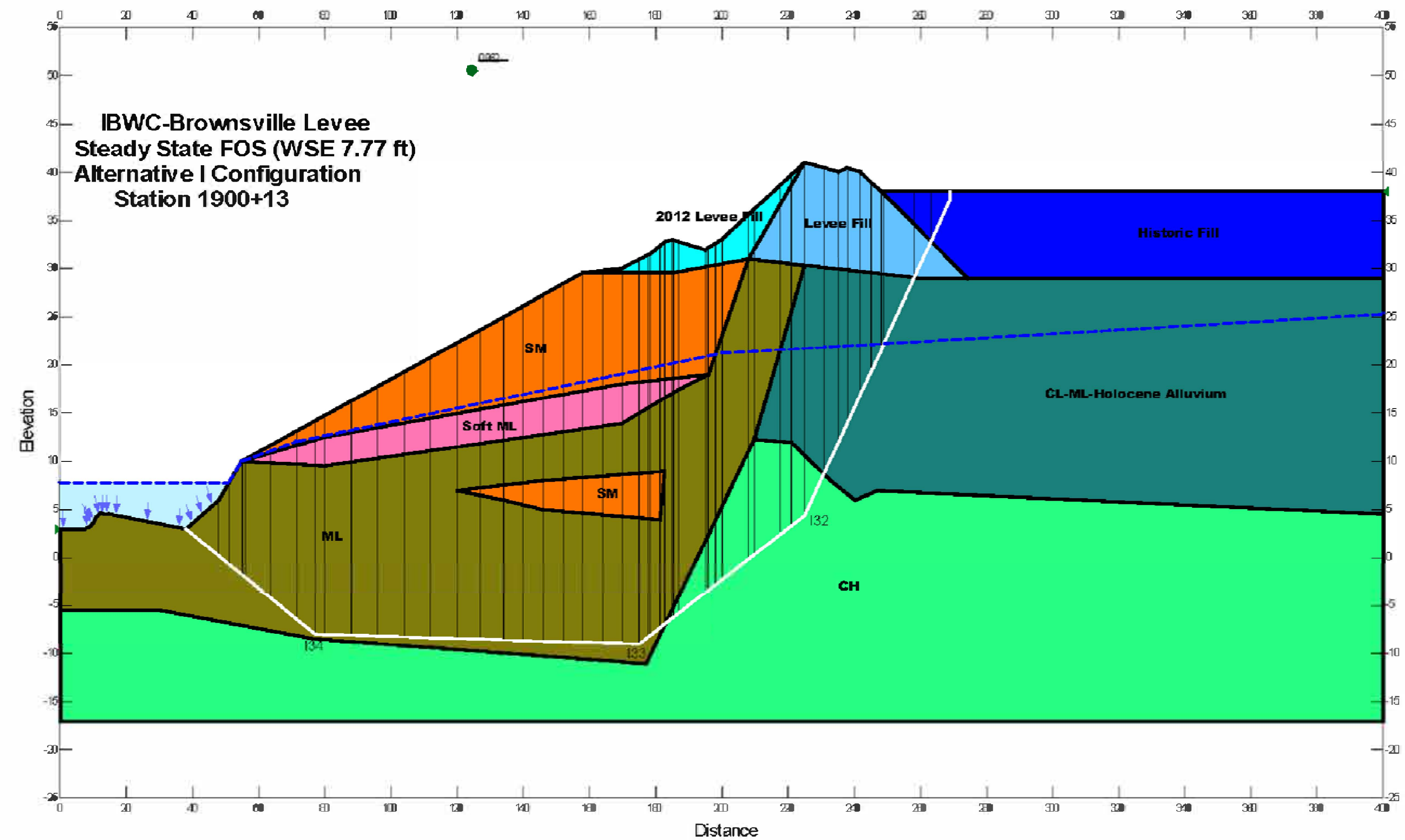
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - EXISTING SLOPE
STEADY STATE SEEPAGE

ARCADIS

SS-5





Minimum Factor of Safety (FOS): 0.962

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
Combination 3			

NOT TO SCALE

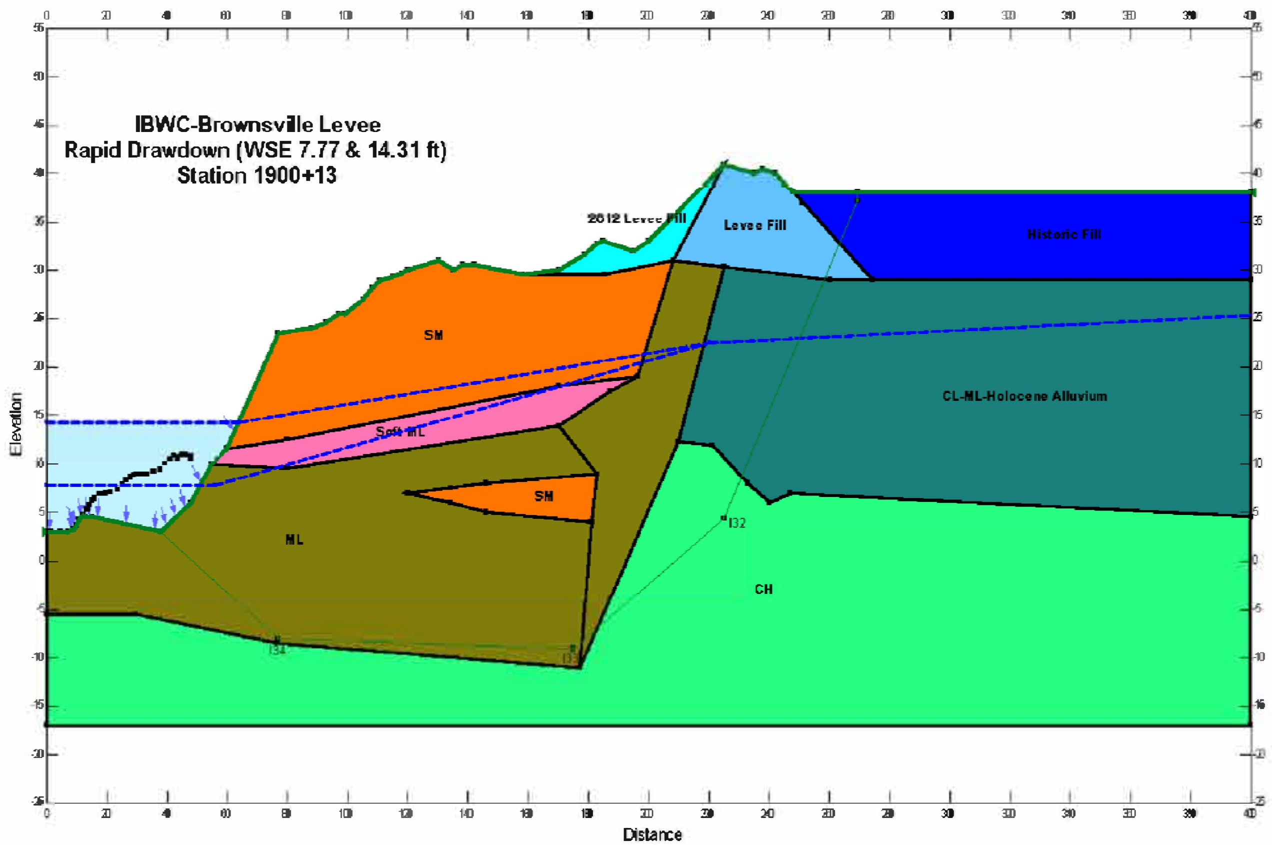
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

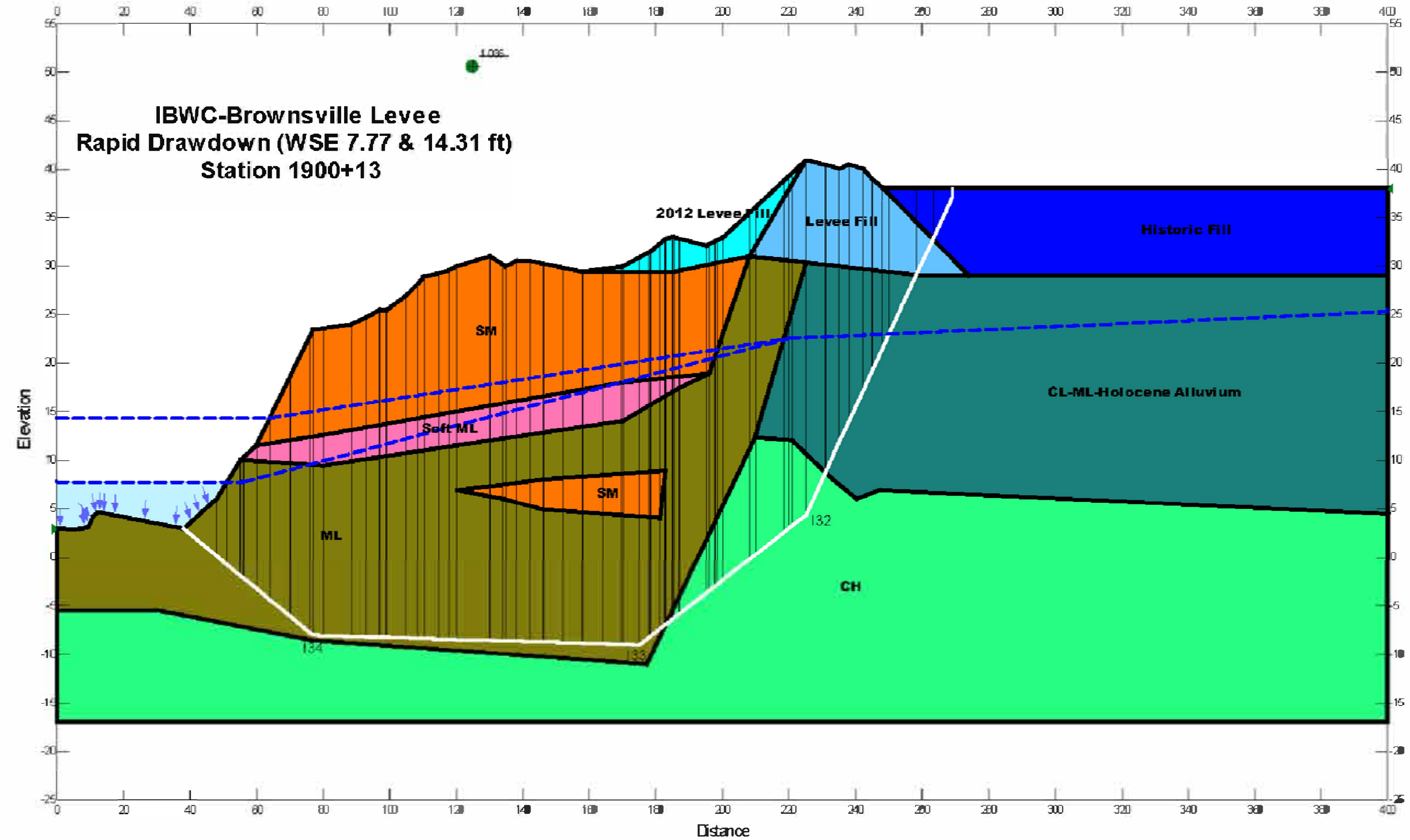
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - ALTERNATIVE 1
STEADY STATE SEEPAGE

ARCADIS

SS-6





Minimum Factor of Safety (FOS): 1.036

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1					

NOT TO SCALE

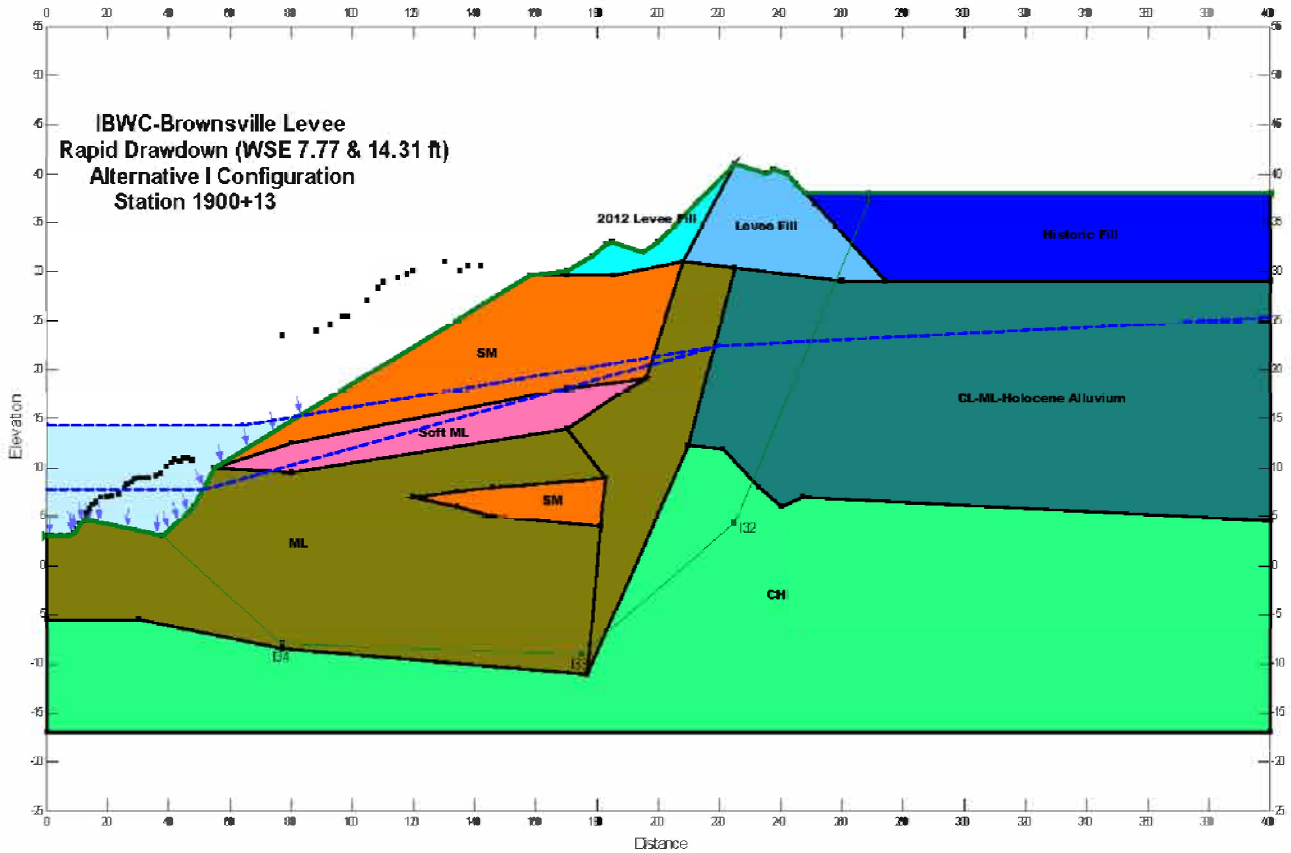
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

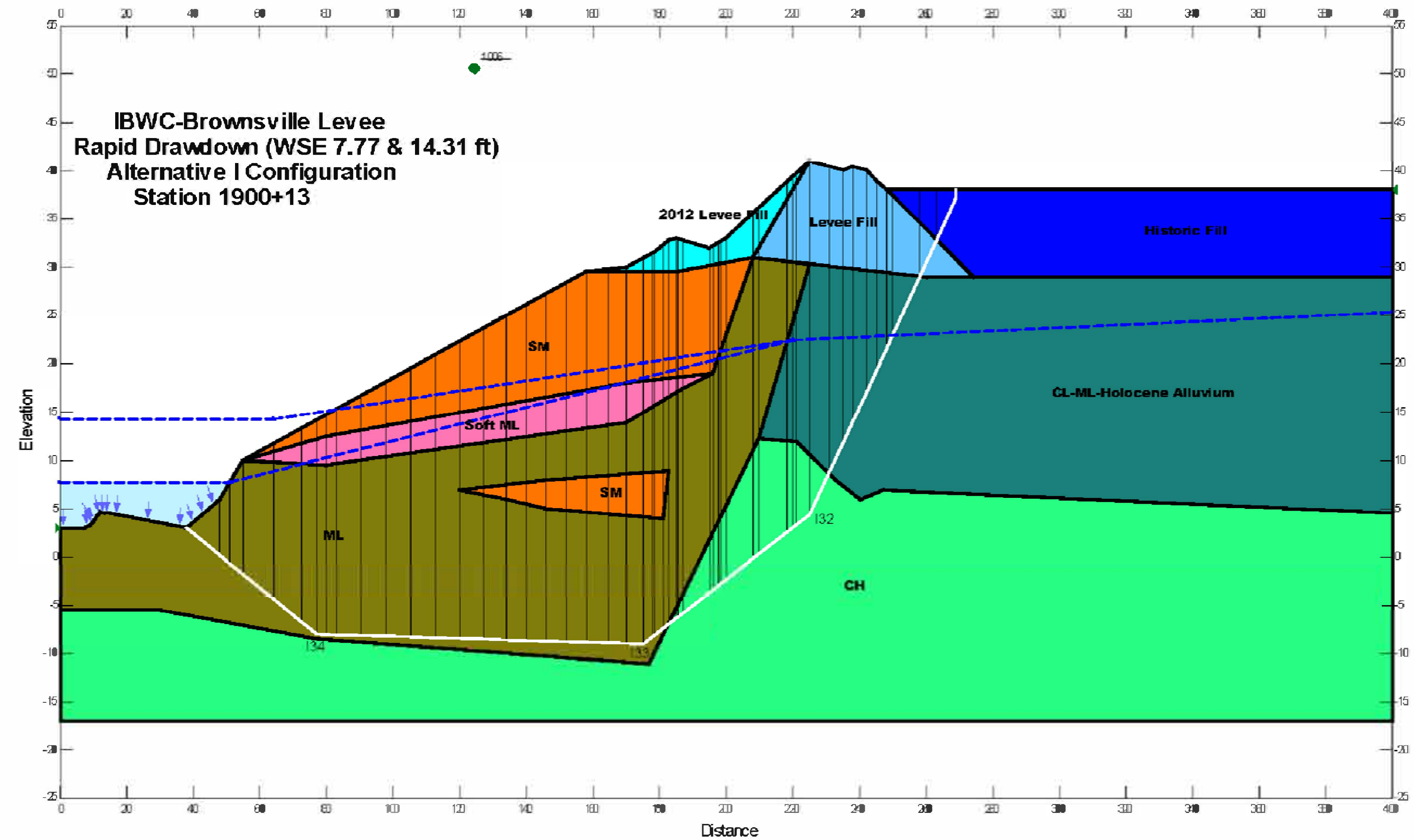
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - EXISTING SLOPE
RAPID DRAWDOWN

ARCADIS

RD-1





Minimum Factor of Safety (FOS): 1.006

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
Combination 4					

NOT TO SCALE

IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - ALTERNATIVE 1 RAPID DRAWDOWN

ARCADIS

RD-2

APPENDIX B

Arcadis' July 2017 Geotechnical Assessment Report

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

FINAL

GEOTECHNICAL ASSESSMENT REPORT CONTRACT NO. IBM15D0001 – TASK ORDER IBM15T0015

July 31, 2017

Remediation Design of Levee Floodplain Failure within the Upper Brownsville Levee Reach Lower Rio Grande Flood Control Project

FINAL

Geotechnical Assessment Report

Contract No. IBM15D0001

Task Order IBM15T0015

Prepared for:

International Boundary and Water Commission, U.S. Section

4171 North Mesa Street, Suite 100C

El Paso, Texas 79902

Prepared by:

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10352 Plaza Americana Drive

Baton Rouge, Louisiana 70816

Tel 225 292 1004

Fax 225 218 9677

Our Ref.:

LA003315.0001

Date:

July 31, 2017

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John Wildman
Geotechnical Specialist

Armando Flores
Geotechnical Specialist

Kirk Lowery, P.E.
Principal Geotechnical Engineer

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1 INTRODUCTION

Arcadis U.S., Inc. (Arcadis) was contracted by the United States International Boundary and Water Commission (IBWC) to evaluate and investigate the Alternative I remediation proposed by the United States Army Corps of Engineers (USACE). This alternative as presented in the USACE's July 2015 *Geotechnical Evaluation of the Brownsville Levee Cracking and Partial Slope Failure* (USACE Investigation) consists of excavating and regrading the channel banks and levee embankment at five horizontal to one vertical (5H:1V) slope. The limits of the proposed remediation extend from Levee Station 1898+00 to 1904+85, henceforth referred to as the project reach throughout this report. Additionally, Arcadis was tasked with recording measurements of the three inclinometers (I-32, I-33 and I-34) installed by the USACE. Inclinometers readings indicated the movement of the soils beneath the levee occur deeper than previously analyzed by the USACE. Arcadis slope stability analyses showed, as presented in the *Geotechnical Assessment Memorandum 60 Percent Submittal* dated February 19, 2016 (60% Memorandum), that the Alternative I remediation would not increase the slope stability safety factor to an acceptable level.

After presenting these results in a meeting on March 17, 2016, Arcadis was tasked with performing an additional geotechnical investigation and evaluation of other alternatives to mitigate the slope movement. The geotechnical investigation included the sampling of four borings, geotechnical laboratory testing on select samples, installation and monitoring of four inclinometers installed in the borings and the performance of a Pile Integrity Test (PIT) on the existing buried bulkhead. Four separate alternatives were initially analyzed in an attempt to mitigate the slope:

- Move the levee further away from the Rio Grande River into the Customs Protection and Border Patrol (CPBP) parking lot;
- Install deep soil/cement mix columns through the failure plane;
- Install stone columns through the failure plane; and,
- Remove the existing levee and replace with a concrete wall.

The results of the field investigation and the analyses are provided in the report. Additionally, estimated costs for each viable alternative and an evaluation of the alternatives to choose a preferred alternative are also provided.

1.1 Project Location

The project site, shown in Figure 1, is on the left bank of the Rio Grande River downstream of the Gateway International Bridge, adjacent to the CPBP facility that is currently not in use.

1.2 Project Background

The project reach lies on a historical oxbow of the Rio Grande, with significant changes to the channel alignment due to river flow and floodplain deposition over time (USACE, 2015). The levees were originally built by the local government around the 1900's in response to historical flooding of the Rio Grande Valley. Flooding continued into the 1930's with more than 20 floods pushing the river out of banks, including floods from three hurricanes. The International Boundary Commission (IBC) assumed responsibility of the levee system in 1932 and implemented a major rehabilitation program that lasted into the 1940's in order to update them to be consistent with the standard USACE levee section of less than 25-foot in height with at least 2H:1V side slopes. To provide 100-year flood protection for Federal Emergency Management Agency (FEMA) certification between the Donna Pump and Brownsville, the levee system was designed for an approximate 3-foot raise. In the project reach, the existing levee was at the approximate required grade but was widened in accordance with the design. The levee fill was completed in October 2013 (USACE, 2015), and the reach was sent to FEMA for certification on January 12, 2016.

In 2014, surficial cracking was observed on the levee crest and toe, approximately between levee stations 1898+00 to 1904+00. Three inclinometers, I-32, I-33 and I-34 as shown on Figure 1 were installed at Station 1900+13, near the center of the failing soil mass. The USACE had evaluated a failure surface based on soil characteristics exhibited in the CPTs with base elevations (EL) between 5 feet and 15 feet (NAVD88)[1]. Alternatives to mitigate the slope movement were provided for this failure surface (USACE, 2015). As documented in the 60% Memorandum:

- Arcadis determined from measurements in inclinometers I-32, I-33 and I-34 that a failure surface occurred deeper than the previously analyzed by the USACE at the alluvium and Pleistocene interface. This movement was detected at an approximate elevation of -10 feet.
- A tension or separation crack was noted in front of the buried bulkhead, at approximate Station range 1899+50 to 1900+85.
- Just north of the bulkhead, cracking of the retaining wall in the CPBP parking area was noted.

2 FIELD INVESTIGATION

The field investigation was performed as per the Draft Work Plan as provided on March 22, 2016, and incorporated into the contract documents. The main purpose of the field investigation was to install new inclinometers because the monitoring equipment or tilt meter cannot pass the depths where the soil has moved in the inclinometers installed by the USACE. Monitoring the inclinometers is very important for two reasons. The first reason is to help verify that the elevation where movement was recorded in the existing set of inclinometers is geologically consistent throughout the project reach. Currently, movement appears to occur in the soft to medium stiff alluvial soils over the stiff Pleistocene age soils. The other reason is to monitor the existing slope conditions so that a possible action plan could be implemented if the movement starts to accelerate.

Additionally, a boring was sampled and an inclinometer was installed near the Gateway International Bridge so that the slope can be monitored in the long term. As a part of the investigation, the baseline and twelve months of inclinometer readings were completed. The final component of the field investigation was the performance of a PIT to determine the base elevation of the buried bulkhead. Appendix A presents the field report submitted on June 23, 2016 to the IBWC that documents the field activities between June 13 and June 22, 2016.

[1] NAVD88 – All elevations are referenced to North American Vertical Datum 1988

2.1 Drilling and Sampling

Borings B-1, B-2, B-3 and B-4 were sampled and inclinometers ARC-1, ARC-2, ARC-3 and ARC-4 were installed in the borings, respectively. Figure 1 presents the locations of these borings as well as select borings and Cone Penetrometer Tests (CPT) previously installed by the USACE. These selected borings and CPTs were chosen to help create soil cross sections. Three of the boring/inclinometer locations, B-1 through B-3, were aligned where the CPBP's retaining wall shows cracks. These borings were advanced at the top of levee, toe of levee and near the bank of the river, respectively. Boring B-4, the final boring/inclinometer, was located near the Gateway International Bridge.

Boring B-1 was drilled and sampled to a depth of 100 feet and borings B-2, B-3 and B-4 were drilled and sampled to a depth of 80 feet. The borings were generally sampled on 3- to 5-foot centers to a depth of 20 feet, continuously from a depth of 20 feet to a depth of 50 feet, and on 3- to 5-foot centers to the final boring depth. The subcontract driller, Professional Services, Inc. (PSI), and Arcadis field personnel attempted to determine the type of samplers to use for obtaining the soil. Predominantly coarse-grained or friable soils are typically sampled with split-spoons advanced approximately 18 inches and fine-grained soils are sampled with Shelby tubes advanced approximately 24 inches. Standard Penetration Tests (SPT) were performed by driving the split-spoon sampler with a 140-pound hammer dropped from a height of 30 inches. The number of blows for each 6 inches of sampler advancement was recorded and the number of blows for the final 12 inches is the SPT value. This value is used to help classify the relative density of non-cohesive soils. The depth intervals between which Shelby tube and split-spoon samples were obtained are illustrated as ST and SS, respectively, under the "Sample" column of the boring logs in Appendix B. Also included in Appendix B are the USACE boring logs and CPT logs referenced in this report.

After completing the sampling of the borings, inclinometers were set in the boreholes. Shelby tube samples were extruded in the field, wrapped in wax paper and aluminum foil, sealed in plastic bags and placed in sample/core carton boxes prior to transport to the laboratory. The split-spoon samples were retained in sealed plastic bags prior to transport to the laboratory. Boring B-1 at depths of 32 to 36 feet, boring B-2 at depths of 38 to 42 feet, and at boring B-3 at depths of 30 to 34 feet, the Shelby tube samples were transported to the geotechnical laboratory and extruded at the laboratory.

2.2 Inclinometer Installation

Upon completion of the borings B-1, B-2, B-3 and B-4, 2.75-inch diameter inclinometers designated as ARC-1, ARC-2, ARC-3 and ARC-4 were installed full depth within the borehole. The three USACE inclinometers identified as I-32, I-33 and I-34, corresponding to borings P3-32, P3-33 and P3-34 as shown on Figure 1, have a diameter of 3.34 inches. The annular space was tremie-grouted to the surface with a cement/bentonite mix. After allowing the grout to setup for at least a week, a flush mounted surface cover for the inclinometers was set in concrete to prevent damage to the inclinometer from mowing equipment and other vehicular activities. The location and the elevation of the inclinometers were surveyed by Munoz & Dannenbaum Joint Venture. The installation of the four inclinometers are explained in detail in the field report shown in Appendix A while the elevations and geographic locations are presented on the boring logs in Appendix B.

2.3 Inclinometer Readings

At least one week after the inclinometers were installed, allowing for the grout to setup, the initial or baseline reading of the inclinometers were performed. Readings of the inclinometers has continued on a monthly basis, and the July 2017 Inclinometer report is presented in Appendix C. As the July report indicates, the inclinometer probe could not pass through the constricted area of the USACE inclinometers at depths of 32 feet, 38 feet, and 30 to 31 feet in inclinometers I-32, I-33 and I-34, respectively. The cumulative displacement plots after a year of collecting data shows no movement in ARC-4, located near the Gateway Bridge, and only a very slight movement at a depth of 38 feet in ARC-3 located near the bank of the Rio Grande. Inclinometers ARC-1, located on the levee crest, and ARC-2, located at the levee toe, show progressive displacement of about $^1/_8$ inch toward the Rio Grande at depths of 28 feet and 38 feet, respectively. This movement appears to be occurring within two feet of the interpreted Alluvium and Pleistocene interface presented in Figure 2.

While taking the November 2016 inclinometer readings, a surface tension crack was noticed near ARC-1. By February 2016, the surface tension crack had extended from ARC-1 south approximately 14 feet, and six pin flags were installed to monitor the crack's width and length. In March 2017, new surface tension cracks were observed south of the wooden bulkhead in two sections and ten additional pin flags were installed. In May 2017, the surface tension crack near the west outer edge of Inclinometer ARC-1 increased in width by about one inch as compared to previous months readings. Prior to arriving on June 14, 2017, the entire site had been mowed and the mowing removed the 16 pin flags that were previously placed in February and March. In addition, the movement of the mowing equipment had caused some of the surface cracks to close and 26 additional pin flags were installed where the surficial tension cracks were visible. In the last monthly site visit, July 12, 2017, some of the pin flags were removed or disturbed due to the vehicular activities on the levee roadway and the pins that were removed were set back to the original locations. The locations of the pin flags are shown in Attachment B of the Appendix C July 2017 Inclinometer report.

2.4 Pile Integrity Test (PIT)

Between approximate Stations 1899+50 to 1900+85 an existing wooden bulkhead is buried within the levee. The history and depth of this bulkhead is not known. As presented in Arcadis' February 19, 2016 60% Memorandum, a tension crack appeared on the river side of this bulkhead. Arcadis speculated this bulkhead is supporting the soils behind, on the protected side, from failing toward the river and recommended determining the depth of the bulkhead. This speculation is further supported by the observation just north of the bulkhead that the retaining wall on the property boundary with the CPBP has cracked and separated between monoliths. Further discussion of the observations at the bulkhead and retaining wall can be reviewed in the 60% Memorandum.

PSI was contracted to do non-destructive testing, PIT, where the bulkhead is struck and compression waves are sent through the structure. The instrumentation mounted to the bulkhead initially registers the strike and the response wave. Based on the timber properties and the response times, the bulkhead depth is estimated. As shown in PSI's PIT Report in Appendix D, the depth of the bulkhead appears to be 18 to 26 feet deep. This depth range coincides with the assumed failure surface as presented in the 60% Memorandum's slope stability analyses. With the existence of the tension or separation cracks at the front side of the bulkhead, the location of the approximate failure surface as presented in the 60%

Memorandum, and the depth of the bulkhead, it is probable that the existing failure surface terminates at the bulkhead and vertically propagates to the top of the levee on the flood side of the bulkhead.

3 GEOTECHNICAL LABORATORY TESTING

PSI was employed to provide geotechnical laboratory services. After receiving the samples from the field, geotechnical laboratory assignments were made by Arcadis' geotechnical engineers.

Soil classification testing of the individual samples included percent passing the No. 200 sieve (ASTM D1140), moisture content (ASTM D2216), and Atterberg limits (ASTM D4318). Results of these tests are presented in tables in Appendix E. Unconfined compression (ASTM D2166) and unconsolidated, undrained triaxial compression (ASTM D2850) tests were performed on selected individual samples of the predominantly fine-grained soils encountered to evaluate the short-term shear strength. Results of these tests are presented as stress-strain curves presented in Appendix E. Consolidated, undrained triaxial compression tests (ASTM D4757) were performed on consecutive samples to determine the long-term shear strength characteristics. Results of these tests are presented as Mohr Circle and stress-strain diagrams in Appendix E.

4 SOIL DESCRIPTIONS

The subsurface conditions are complex based on the depositions that make up the Rio Grande floodplain, with alluvial deposits overlying Pleistocene-age soils. Historical fill and Holocene Age Alluvium composed mostly of medium stiff to stiff lean and fat clays were deposited above the Pleistocene-age soils beneath and behind the levee. Toward the riverside, the alluvial soils are predominantly silty clay and silt that are soft to medium stiff in consistency with localized layers of predominantly sandy soil that overly the Pleistocene-age soils. The Pleistocene-age soils are generally very stiff, jointed and slickensided fat clays with variable silt and sand content.

The borings for this investigation encountered similar soils to those described in the 60% Memorandum. Sections A and B shown on Figures 2 and 3 present sections through the levee and along the top of the levee, respectively. The Figure 2 geologic cross-section passes through borings B-1, B-2 and B-3 and is geologically very similar to the parallel USACE section as shown in Appendix F. This section is a reproduction of the 60% Memorandum's Attachment B Section I. A full description of the discrepancy between the USACE's and Arcadis' interpretation of the Alluvium and Pleistocene age soils interface is presented in the 60% Memorandum. Basically, the elevations of the material interface by Arcadis were determined directly from the boring logs and from information presented on the CPT results.

4.1 Near Surface Geologic Description

The geology of the project reach consists of Pleistocene Alluvial soils, overlain with Holocene Alluvial soils, and Historical Alluvial that has been deposited since 1846. Information from the USACE Investigation suggests that the Pleistocene sediment was likely exposed to weathering for several hundred years before the Holocene Alluvial soils were deposited, which would account for the variability in the subsurface soils near the Pleistocene interface. The depositional sequence of materials is important, and as presented in the 60% Memorandum, the failure surface appears to occur on the interface of the Alluvium and Pleistocene age soils.

4.2 Material Properties

As part of the geotechnical investigation, short-term and long-term shear strength testing was undertaken to verify the shear strength values interpreted in the 60% Memorandum. Unfortunately, all the laboratory tests appear to have shear strengths significantly higher than those back calculated in the 60% Memorandum. Back calculation of the shear strengths was developed for steady-state seepage and rapid drawdown conditions by varying soil properties for a predetermined slope failure surface, as determined by the movement in the USACE inclinometers, so that the factor of safety (FOS) approached approximately unity or 1. Appendix F presents reproductions of the 60% Memorandum's Attachment D slope stability sections used to develop the shear strengths. Table 4-1 presents these back calculated shear strength properties for the various water conditions.

Table 4-1: Back Calculated Shear Strength Properties

	Steady State Seepage						Rapid Drawdown			
	Case 1		Case 2		Case 3		Effective Stress		Total Stress	
Material	C' (psf)	ϕ' (deg)	C' (psf)	ϕ' (deg)	C' (psf)	ϕ' (deg)	C' (psf)	ϕ' (deg)	C (psf)	ϕ (deg)
CH Pleistocene	200	12	0	10	90	12	150	16	2320	0
CL Holocene	460	13	0	10	225	12	200	14	400	0
SM	0	32	0	32	0	32	0	32	0	32
ML	230	0	0	11	0	8	190	0	0	29
Levee Fill	300	12	0	11	105	12	400	20	5000	0
Historical Fill	200	24	0	11	95	12	200	20	400	15
Soft ML	150	0	0	8	150	0	150	0	168	0

Notes: Case 1 – The ML shear strength was set with no ϕ' and the shear strength of the other strata were varied.
Case 2 – The soils exhibit no cohesive properties.
Case 3 – The ML shear strength was set with no C' and the shear strength of the other strata were varied.
C' – Effective stress cohesion intercept
ϕ' – Effective stress angle of internal friction
C – Total stress cohesion intercept
ϕ – Total stress angle of internal friction .

5 SLOPE STABILITY

Slope stability of the new section was initially analyzed to determine whether this section would be more critical with the same soil properties as the section presented in the 60% Memorandum, at Station 1900+13. After completing this evaluation, the mechanism(s) on the cause of the slope movement was evaluated so that a comprehensive mitigation scheme is provided. Slope stability was analyzed using GeoStudio 2007 version 7.23 SLOPE/W© software by Geo-Slope International. Spencer's method of force and moment equilibrium was used to compute the theoretical FOS.

5.1 Station 1899+15

The slope for Section A, shown on Figure 2 at approximate Station 1899+15, was analyzed. At this location, the geometric configuration at the top of the levee, toe of the levee and at the edge of the riverbank were estimated from the survey points collected by Munoz & Dannenbaum, and the profile of the river bed was developed from the bathymetric data presented in the USACE report. Seepage conditions were modeled from the phreatic surface developed from the USACE Investigation, for both steady state seepage and rapid drawdown. This section, similar to the one presented in the 60% Memorandum at Station 1900+13, was analyzed in order to determine if this section could be more critical than the previously analyzed section. Appendix F presents reproductions of the section analyzed and presented in the 60% Memorandum. Appendix G presents the results of the Figure 2 section, with a more simplified soil stratigraphy. Table 5-1 compares the results of the two sections and as can be seen, the slopes at Station 1900+13 appear slightly more critical to analyze for determining a mitigating solution.

Table 5-1: Comparison of Slope Stability Sections

Piezometer	Factor of Safety	
	Station 1900+13	Station 1899+15
Steady State Seepage – Case 1	1.10	1.11
Steady State Seepage – Case 2	1.10	1.10
Steady State Seepage – Case 3	1.10	1.12
Rapid Drawdown	1.04	1.08

5.2 River Stage

To better understand the cause of the slope instability and the movement within the inclinometers, river gage data was reviewed. The rapid rise and fall of the water creates a rapid drawdown condition where the water levels in the soils are elevated during the rise in the river and slowly dissipate after the water level falls in the river. IBWC Stage/Discharge records of the Rio Grande near Brownsville -Texas Gage, Latitude 25.87567 degrees and Longitude -97.45468 degrees for January 2013 through July of 2017. The river stage for the project reach was interpolated by USACE from this downstream river gage using 2011 LIDAR to approximate the river elevation at the bridge and determined it is about 7-foot higher than the Brownsville Gage. This data was plotted and is provided in Appendix H.

This data shows that the rise and fall of the river generally when water is released in mid-May to early June. Between the USACE's final reading of inclinometers P3-32, P3-33 and P3-34 in January 2015 and Arcadis initial readings in December 2015, the Rio Grande rose significantly to Stage 24.2 feet on June 6th. This stage elevation, approximately 31.2 feet at the project site, would put the water at or near the toe of the levee. By June 9th the river had dropped over 10 feet. When Arcadis read the inclinometers in

December 2016, up to four inches which is considered significant movement was noted in the inclinometers. This movement was most likely triggered by the rapid fall of the river. In 2016, the data shows the river rose from approximate Stage 1.9 feet on May 15th to Stage 10.2 feet on May 18th and fell to Stage 4.9 feet on May 21st. This elevation differential is similar to that used for the rapid drawdown case. Also, this rise and fall occurred between the January and June 2016 readings of the USACE inclinometers. In January, the inclinometer probe was able to go to the bottom of the inclinometers whereas in June it could not pass at certain depths within these inclinometers. The impassable depths are presented in the Appendix C July 2017 inclinometer report. Arcadis speculates that one of the main contributing causes of the slope stability is when the river rises and falls quickly or the rapid drawdown case.

5.3 Lake Brown

It is possible that a steady state seepage failure could be occurring where the water from Lake Brown is seeping into the Rio Grande. However, this mechanism to cause movement could have happened at any time over the life of the levee. It is very possible this seepage contributes to the slope instability, but this contribution is considered slight.

5.4 Bank Erosion

As the Rio Grande drains basically to the west, water is passing under the Gateway International Bridge and exiting an approximate 150-degree turn in the river. Because of the river geometry and the presence of the bridge, the velocities in the river are probably higher in the project area than further downstream. This area was armored in the 1950s but the armoring appears to be covered and not well defined. The soils along the river bank appear in the proximate borings to be relatively silty soft to medium stiff in consistency. These conditions lend themselves to scour which may have helped trigger a shallower failure surface, as analyzed by the USACE. However, movement along the deeper failure surface would require moving a large land mass with a very large scouring event to relieve the resisting soil mass. Review of the recent surveys compared to historical surveys does not show this type of large erosion event.

5.5 Levee Construction

The most likely major cause for the deep-seated failure surface, see Figure 4, was the construction of the 2012-2013 added fill. Not long after construction of this fill was completed and the rise and fall of the river, escarpments near the toe and on top of the levee appeared. There is no doubt that the rapid rise and fall of the Rio Grande from controlled release of upstream dams significantly contributes to the slope instability. However, these releases and the subsequent rise and fall of the Rio Grande has occurred for many years prior to the addition of the fill in 2012 and 2013. This added fill would add driving weight to any failure surface that promulgate along or in the levee, such as the deep failure surface shown on Figure 4, but it would have little to no effect on the failure surfaces that promulgate towards the riverside toe. Basically, the driving weight of the levee is outside the failure plane that appears near the levee toe, see toe failure surface on Figure 4.

5.6 Discussion

Prior to discussing possible causes, Arcadis notes that when a slope starts to move, the movement begins at the downslope and promulgates upslope. This is important to note because once the movement starts, the soils closest to the river or downslope tend to strain and get weaker.

The exact cause of the slope movement appears to be a combination of circumstances that is difficult to directly qualify and/or quantify. The USACE identified weak, underconsolidated, subsurface soils between EL 5 and EL 15 that had possibly moved/slid at some time maybe due to erosion of the bank or because of the rise and fall of the Rio Grande, see USACE failure surface on Figure 4. However, review of the USACE inclinometer I-34 data, near the river, does not show any movement between these elevations. Arcadis speculates that this shallow failure surface may have occurred at some time in the last 10 years and that some of the upper soils were weakened, having moved. When the load of the levee was placed and after the rise and fall of the Rio Grande, resistance along the Alluvium/Pleistocene interface mobilized. The movement or straining of the soils may have propagated toward the toe of the levee through the soils that were previously weakened along the USACE's speculated failure surface, see the upper portion of the toe failure in Figure 4. If this occurred, the resistance along the Alluvium/Pleistocene interface would have become weaker, see Figure 4 where the toe and deep failure surfaces overlap, which could have triggered a longer failure surface into the levee, as shown as the deep failure surface on Figure 4.

Determining the triggering mechanism is very important so that the design to arrest the slope movement encompasses the entire problem. For example, if there was shallow movement near the river, see USACE failure surface on Figure 4, and a deeper movement in the levee only, see deep failure surface on Figure 4, then the USACE design alternative could be implemented to mitigate the shallow movement and a deep-seated levee failure movement could be mitigated separately. However, if the deep-seated movement occurs near the toe of the levee, see toe failure surface on Figure 4, as well as within the levee, see deep failure surface on Figure 4, then the USACE design alternative will need to be re-analyzed. Arcadis has revised the slope stability analyses to account for these deeper failure surface as presented in the following section of this report.

Finally, determining whether there is a deep-seated failure surface promulgating toward the toe of the levee would be difficult and would require a little bit of luck. The best way to prove this movement would be to set an inclinometer along the active face of the failure surface that appears at the levee toe, see toe failure surface on Figure 4. This inclinometer would have to be strategically placed so that the measured movement would be below EL 5 and above EL -10. Geometrically, this could be estimated; however, if movement was shown at some elevation above EL 5, there is no guarantee that a deeper movement is not occurring. For this reason, Arcadis recommends that a deep-seated mitigation needs to be assessed from the toe of the levee as well as from the top of the levee.

5.7 Station 1900+13 Levee Toe

A slope stability failure surface along the same bottom failure plane as the one surfacing in the levee was analyzed to surface near the levee toe. The intent of this analysis was to determine with the back calculated shear strength values if a failure surface near the toe would occur. This specified surface analyzed with the Steady State Seepage – Case 3 soil and water conditions yielded a FOS just below unity or 1 as shown in Appendix I. The 5:1 cut slope or the USACE's Alternative I recommendation was then analyzed as shown in Appendix I. This slope stability analysis shows the FOS goes down by flattening the slopes near the river which indicates the slopes between the river and the levee will need to be mitigated for a deep-seated failure surface by other means than geometric modification.

6 ALTERNATIVES

Initially, Arcadis had analyzed the four alternatives listed in Section 1 of this report assuming the slopes near the river would be stabilized by cutting the slopes as recommended by the USACE. However, analysis shows a failure surface at the Alluvium/Pleistocene interface that surfaces at the toe of the levee does not have an acceptable FOS with this cut slope alternative. Arcadis submitted a technical memorandum on June 13, 2016 showing that moving the levee into the CPBP parking lot would not meet acceptable slope stability FOS. As part of the analysis presented in the memorandum, the levee was cut down to EL 31 and the lowest FOS for the steady state seepage analyses was 1.3. This FOS is not considered acceptable; therefore, the installation of a T-wall is not considered a viable alternative due to insufficient global stability. Installation of soil mix columns or stone columns through the failure surface are viable alternatives and were analyzed. Also, a combination of stone columns near the river and soil mix columns near the levee could arrest the slope movement.

The required slope stability FOS, Table 6-1b in USACE Engineering Manual 1110-2-1913 Design and Construction of Levees (EM 1110-2-1913), for existing levees is 1.4 for Steady State Seepage and 1.0 to 1.2 for Rapid Drawdown. Since the soil properties have been determined by back calculating the slope movement, EM 1110-2-1913 table 6-1b, allows the FOS to be lower. Arcadis' alternatives were designed to allow FOS of 1.35 for Steady State Seepage and 1.1 for Rapid Drawdown.

6.1 Deep Soil Mixing

To arrest the soil movement, deep soil mix columns would be installed to a depth below the Alluvium/Pleistocene interface in panels or continuous columns perpendicular to the levee. These columns would be installed by rotating augers and injecting cement to bind the mixture. The panels are spaced, along the levee alignment, based on the size of the individual columns so that continuous soil modification is not needed.

Deep mixed shear panels were assessed in general accordance to guidelines outlined by Filz and Templeton's (2011) *Design Guide for Levee and Floodwall Stability Using Deep-Mixed Shear Walls* (Design Guide). This design guide presents specific, step-by-step methods to assess the stability of deep mixed shear panels that support levees and floodwalls. However, this design guide is specific to deep mixed shear panels that underlie the crest and protected side of levees and exist in a subsurface characterized by perfectly horizontal layering. This differs intrinsically from the shear panels necessary for the river-side improvement of the IBWC levee in the complexly layered subsurface within the project limits. Accordingly, sound variations from the methods described in the design guide were employed to properly assess potential sliding, overturning, crushing, shear, shear on vertical planes, and extrusion failures. These variations are described herein.

Because the Design Guide addresses a deep mixed zone (DMZ) on the dry side of the levee, it was necessary to mirror on a vertical plane the freebody conceptualization of the DMZ to the flood side to properly define forces acting on the DMZ. Once the proper equations were defined, forces acting on the DMZ were evaluated. The Design Guide employs the USACE computer program Stability with Uplift to use the Method of Planes to define a number of forces acting on the DMZ. Stability with Uplift is most often applied to a perfectly-horizontally layered subsurface profile and was considered impractical for representing the subsurface present at the critical section at Station 1900+13. However, the root equations for the Method of Planes were modified to account for the recognized slip surface geometry at the critical section to ultimately assess forces in the same manner as the Design Guide. The perfectly horizontal layering represented in the Design Guide allows for direct calculation of moment arm height, measured from the base of the DMZ, of lateral earth pressure within each layer. For this analysis, the moment arm height of lateral earth pressures was assessed as the center of gravity height from the base of DMZ of the vertical stress distribution at the leveeward and riverward edges of the DMZ. Finally, the Design Guide calculations are based on a factor of safety that reduces shear stress parameters. It is suggested that multiple FOS are selected and used to compute total forces contributing to potential sliding, overturning, crushing, shear, shear on vertical planes, and extrusion failures. The summation of forces are then plotted against the selected factor of safety to graphically determine the actual factor of safety or the factor of safety that corresponds to a summation of forces equal to zero. Arcadis recognized that the GoalSeek function within Microsoft Excel could more accurately and more efficiently calculate the actual factor of safety for each potential failure mode. Accordingly, this analysis used Excel to numerically calculate factors of safety, as opposed to graphically determining the safety factors, as suggested in the Design Guide.

For the geometry at this site, sliding and overturning of the curtain controlled the design. The resistance to the active driving forces are based on the shear at the base and the passive forces near the river. If the curtain is placed too close to the river, then there is very little passive resistance. A number of soil panel lengths were tried without installing soil mix columns beneath the levee. The driving force against the soil mix column from the top of the levee at approximate EL 41 to the failure surface at approximate EL -10 is large enough so that the allowable FOS for sliding or overturn could not be met with this layout. To create enough passive resistance to arrest sliding and overturn of the deep failure surface as shown on Figure 4, the soil mix columns would need to terminate near the levee toe as shown in Appendix J. However, this soil mix column configuration would not mitigate the toe failure surface as shown in Figure 4. The only way to arrest the movement for the entire slope would be to eliminate the driving forces and stabilize a majority of the existing failure surface by removing the levee, installing soil mix columns and rebuild the levee on the soil mix columns. Figure 5 presents a plan view of the layout and Figure 6 presents a section view of the soil mix columns. As shown in the figures, the soil mix columns were extended to near the river slope because the Figure 4 toe failure surface's FOS were not acceptable between the edge of the river slope and toe of the levee. Slope stability of the Figure 4 USACE, toe and deep failure surfaces were not analyzed because the entire failure surface will be stabilized with the DMZ. The top of the near river slopes are 30 feet above the Alluvium/Pleistocene Interface with less than 30 feet of distance to the base of the river slope. Geometrically, the top of the river slope would develop small driving forces to the base of the deep failure plane so that slope movement would commence far into the river which is not realistic.

Spacing of the panels is usually controlled by extrusion between the panels or basically the "bridging" effect between panels. In this case since the driving forces have been removed, a 25 percent replacement ratio was used to determine 20-foot center-to-center spacing of the columns. Generally, 25 to 30 percent replacement ratios are specified for stabilizing slopes with DMZ.

6.2 Stone Columns

To arrest the soil movement, stone columns would be installed to a depth below the Alluvium/Pleistocene interface in columns spaced between the levee and the Rio Grande in the identified area that is moving. These columns would be installed by rotating augers, removing soil and replacing with stone. In some instances, casing may be necessary to keep the augered excavation from collapsing. The spacing of the columns are based on the required soil replacement needed to provide an adequate FOS for slope stability.

Stone columns were assessed in accordance with the recommendations in Appendix B of *Stability Analyses of Embankments Founded on Stone Columns* of Filz and Navin (2006). These guidelines present three methods for slope stability analysis of slopes founded on stone columns – the circular sliding surface method, the assigned strength parameter method, and the profile method. It also references Barksdale and Bachus (1983) which presents the same three methods. Both Filz and Navin (2006) and Barksdale and Bachus (1983) recommend the profile method as the preferred method of analysis for slopes supported by stone columns. Accordingly, the profile method was selected for this analysis.

The profile method allows the use of limit equilibrium computer software to represent the area of stone column-improved ground as a function of the stone column diameter, center-to-center column spacing, and the geometric configuration of the columns relative to one another. The most common geometric configurations of stone column installation are an equilateral triangle configuration and a square configuration, with equilateral triangle being most common. Accordingly, this analysis analyzed an equilateral triangle configuration.

The profile method defines an effective unit cell to represent the circular, plan-view region influenced by a single stone column. For an equilateral triangle configuration, the unit cell diameter, De, is defined as 1.05s, to account for any variance in column offset during construction, where s is the center-to-center spacing of the columns. The area of the unit cell, A, is computed as the area of a circle with diameter De, and is compared to the area of the stone column. New York State Department of Transportation's *Geotechnical Design Manual* (NYSDOT, 2013) specifies that the area of the stone column, As, should be assessed as the area of a circle with the diameter of the initial boring, which typically exceeds the diameter of the completed column by about 0.25 to 0.75 inches. Neither Filz and Navin (2006), Barksdale and Bachus (1983), nor published international design standards specify that the considered area of the stone column should exceed the area of the completed column. Accordingly, this analysis considered the area of individual stone columns to be that of the completed column. Considering the area of the initially bored hole would also increase the assessed area of improvement, thus considering as the area of the completed column yields a more conservative evaluation. The profile method then employs geometric relations and the ratio of As/A, called the area replacement ratio, αs, to scale the column diameter and column spacing such that the improved ground can accurately and appropriately be considered as equivalent vertical strips in two-dimensional limit equilibrium slope stability modeling.

For this assessment, stone columns evaluated as 36-inch diameter, were spaced 6 feet apart. This spacing yielded an industry-common area replacement ratio of 25 percent. Shear strength parameters for the stone columns were defined with an angle of internal friction equal to 40 degrees. Initially assessing rapid drawdown conditions along the failure plane defined in the 60% Memorandum, it was determined that 12 rows of columns are necessary at the levee to arrest the deep failure surface as shown on Figure 4, and 9 rows are necessary to maintain stability for the toe failure surface as shown on Figure 4. Accordingly, both levee (deep failure surface in Figure 4) and toe (toe failure surface in Figure 4) configurations of stone columns geometry were evaluated for global stability beneath and through each set of columns using the four shear strength combinations. Appendix K presents the stone column spacing calculations for 25 percent replacement and the various slope stability trial surfaces in an attempt to fail through or around the stone columns. Although the models in Appendix K show the stone columns would go through the levee, the levee would have to be removed to provide a level working surface; the stone columns installed; and, the levee would have to be replaced.

Table 6-1 presents the most critical and acceptable slope stability FOS depending on where the failure surface terminates. Figures 7 and 8 present plan and section views of the stone column configuration, respectively.

Table 6-1: Summary of Stone Column-Improved Slope Stability Safety Factors

Slope Stability Factors of Safety				
	Shear Strength Combination			
Failure Surface Location	Steady State Seepage Case 1	Steady State Seepage Case 2	Steady State Seepage Case 3	Rapid Drawdown
Through the Levee (deep failure surface)	1.40	1.38	1.41	1.31
Toe of Levee (toe failure surface)	1.41	1.36	1.37	1.36

6.3 Deep Soil Mixing and Stone Columns

In an attempt not to remove the levee to stabilize the slope, Arcadis analyzed the combination of deep soil mixing panels to arrest the levee slopes and stone columns to stabilize the near river slopes. Basically, a combination of analyses was separately undertaken. Results of the near river stone column analyses which will arrest movement along the toe failure surface as presented on Figure 4 are presented in Section 6.2 of this report. Whereas the deep soil mix solution was separately analyzed to mitigate the deep failure surface as presented on Figure 4.

Arcadis has determined that a 45-foot wide DMZ, offset ten feet riverward from the levee crest, extending to EL -15 would adequately arrest the deep failure surface. Because the diameter of the individual columns constructed by the contractor are currently unknown, two alternative diameters and subsequent center-to-center panel spacing were initially assessed. The first would entail 31.5-inch diameter columns spaced on 7.5-foot centers and the second would be 72-inch diameter columns spaced on 17-foot centers. As the analyses proceeded and after discussions with specialty contractors, the 72-inch diameter columns were chosen and the results are presented herein. Geometric and strength attributes of the proposed columns are presented in Table 6-2, and Table 6-3 presents factors of safety for potential sliding, overturning, bearing, shear on vertical planes, and extrusion failures. The calculated FOS are presented in Appendix K.

Table 6-2: Geometric and Strength Attributes of DMZ

Description	Variable	72-Inch Diameter	Units
Thickness of DMZ	H	44.8	Ft
Width of DMZ normal to CL of levee	B	45	Ft
Diameter of single deep-mixed column	d	72	Ft
Center-to-Center panel spacing	s	17	Ft
Column overlap distance	e	1	Ft
Specified UCS of deep-mixed ground	q_{dm}	100	Psi
Time between construction and loading	-	28 days	-
Design FOS of deep-mixed ground strength	f_d	1.5	-
Probability DMZ strength > specified strength	p_{dm}	80%	-
Deep mixed material shear strength	s_{dm}	5,011	Psf
Deep mixed zone shear strength	s_{dmz}	1,534	Psf

Table 6-3: Summary of DMZ Factors of Safety

Summary of Design Factors of Safety			
Mode of Failure	Abbreviation	Required FOS	72-Inch Diameter Actual FOS
Sliding	F_s	1.3	1.506
Overturning	F_o	1.4	1.455
Crushing	F_c	1.4	1.69
Vertical Shearing	F_v	1.4	1.96
Extrusion	F_e	1.3	>10

Global stability without the river side stone columns was initially analyzed for various failure surfaces through and around the soil mix columns using the four shear strength combinations. The results of the analyses are presented in Appendix J. This configuration was considered the most critical because it could occur during construction. Global stability around the soil mix columns and stone columns would have failure surfaces similar to what is shown on pages J-14 through J-21 of Appendix J. Although the models in Appendix J show the deep soil mix columns would go through a portion of the levee, a small portion of the levee would have to be removed to provide a level working surface; the deep soil mix columns installed; and, the levee would have to be replaced.

Table 6-4 presents the most critical and acceptable slope stability FOS. Figures 9 and 10 present plan and section views of the combined deep soil mix and stone column configuration, respectively.

Table 6-4: Summary of Slope Stability Factors of Safety for Proposed DMZ

Slope Stability Factors of Safety				
	Shear Strength Combination			
Failure Surface Location	Steady State Seepage Case 1	Steady State Seepage Case 2	Steady State Seepage Case 3	Rapid Drawdown
Through the Levee (deep failure surface)	1.55	1.35	1.46	1.41

7 ENGINEER'S OPINION OF PROBABLE CONSTRUCTION COST

REDACTED BY USIBWC

REDACTED BY USIBWC

8 CONCLUSIONS AND RECOMMENDATIONS

Escarpments appeared near the toe and within the levee between Levee Station 1898+00 to 1904+85 shortly after the installation of the 2012-2013 levee lift. The monitoring of inclinometers installed by the USACE perpendicular to the levee show movement at the Alluvium/Pleistocene interface at approximate EL -10. This movement is occurring over 10 feet below the bottom of the Rio Grande and approximately 35 to 40 feet below natural grade. This relatively deep failure surface is causing a large ground mass movement that cannot be mitigated by removing the existing levee or flattening existing slopes. Theoretically, filling in the Rio Grande could mitigate the slope movement but this option is not considered viable. Three alternatives have been analyzed and costs estimated to mitigate the slopes:

1. Deep soil mix columns at a cost of approximately $7.5 million;
2. Stone columns at a cost of approximately $8.8 million; and,
3. A combination of the deep soil mix columns and stone columns at a cost of approximately $6.6 million.

If the deep soil mix columns or soil mix columns alone are installed then construction beneath the levee would need to occur outside of hurricane season or temporary flood protection measures would need to be installed. These measures could include removing the levee in sections and placing HESCO baskets or leaving the back portions of the levee near the CPBP in place. These costs were not included in

Tables 7-1 and 7-2 because the exact timing of installation, possibly between November and June, is not known.

Arcadis has contacted multiple specialty contractors to solicit unit costs and discuss the options presented in this report. During these discussions, the following information was documented:

- Staging and installation of deep soil mixing to the 45- to 50-foot depths should be accomplished using the dry mix method.
- One of the specialty contractors reviewed the boring logs and was not very confident that his company could install the stone columns to depths of 45 to 50 feet but he was more confident with installing the near river 30- to 40-foot deep stone columns. Another contractor thought installing the stone columns possible, somewhat risky, and he provided costs.

After processing these discussions, Arcadis believes there would be some risk with the installation of deeper stone columns that would be placed beneath the levee. Arcadis recommends installing the combination of the deep soil mix columns/panels to arrest the deep failure surface movement and the stone columns to arrest the toe failure surface movement. The basis for this recommendation are:

1. Lower cost;
2. Minimize the potential for having to put in temporary flood protection measures; and,
3. Less risk for installing stone columns more than approximately 40 feet deep.

Although not provided in Tables 7-1, 7-2 and 7-3 or on the figures, Arcadis also recommends armoring the bank of the Rio Grande to help protect against erosion similar to what was recommended by the USACE and provided in Arcadis' *Design Report 60 Percent Submittal* (60% Submittal) dated February 2016. An estimated cost to excavate and place armoring including installing rip-rap around the Gateway International Bridge piers was provided in the 60% Submittal. These estimated costs, $1,627,000 with no contingency and $1,871,000 with the 60% Submittal contingency of 15 percent, are provided for planning purposes as presented in Appendix L. It should be noted that these armoring costs exclude the unnecessary excavation costs to slope the bank at five horizontal to one vertical.

As this project proceeds into final plans and specifications, Arcadis recommends the new inclinometers continue to be monitored to see if any movement occurs so that this movement can be compared with the movement that occurred in the USACE inclinometers. The ground modification recommendations presented in this report were geometrically optimized based on the worst case back calculated shear strength values with the most critical levee section. Arcadis recommends during the 60 percent plans and specifications resubmittal the preferred alternative design be refined to more efficiently layout and possibly reduce the construction cost. This refinement would include analyzing sections outside the bulkhead toward the north, Figure 2 of this report, and toward the south, Figure 5-8 of the USACE Investigation.

9 REFERENCES

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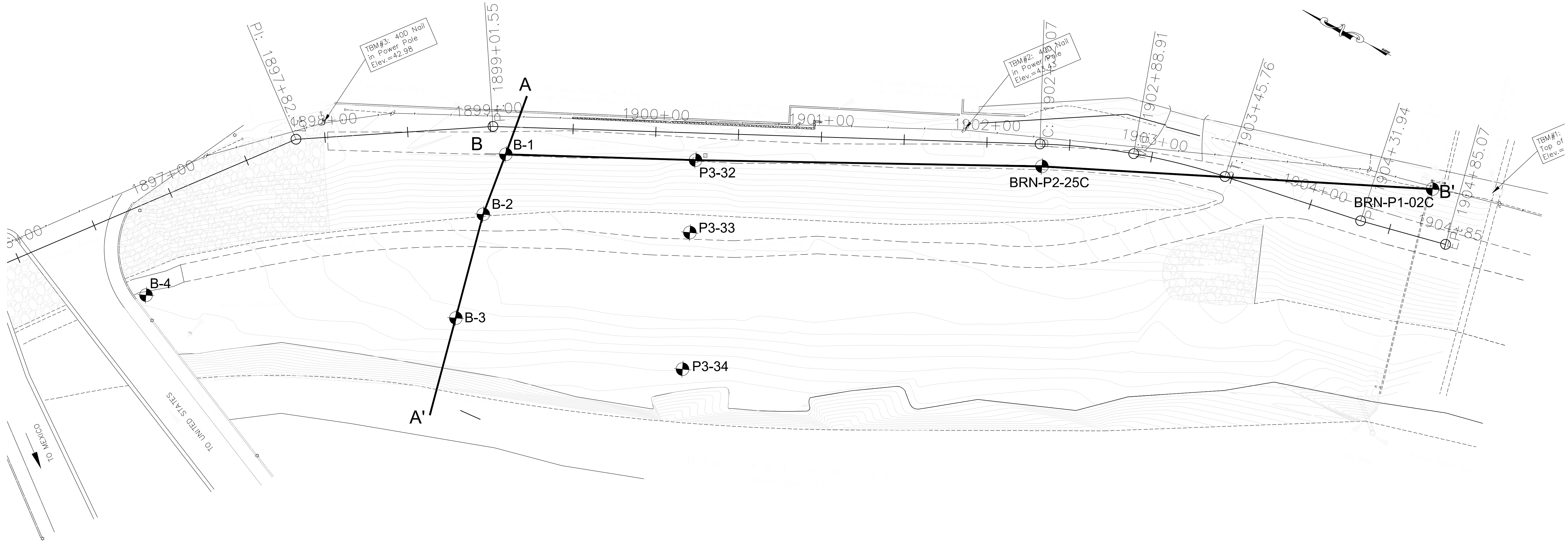
FIGURES



Note:

Inclinometers ARC-1 through ARC-4 correspond to Arcadis' borings B-1 through B-4 and Inclinometers I-32 through I-34 correspond to USACE's borings P3-32 through P3-34

0 60' 120'

USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

BORING LOCATION MAP

ARCADIS

FIGURE
1

CITY:(Reqd) DIV/GROUP:(Reqd) DB:(Reqd) LD:(Opt) PIC:(Opt) PM:(Reqd) TM:(Opt) LYR:(Opt)ON=*;OFF=*REF*
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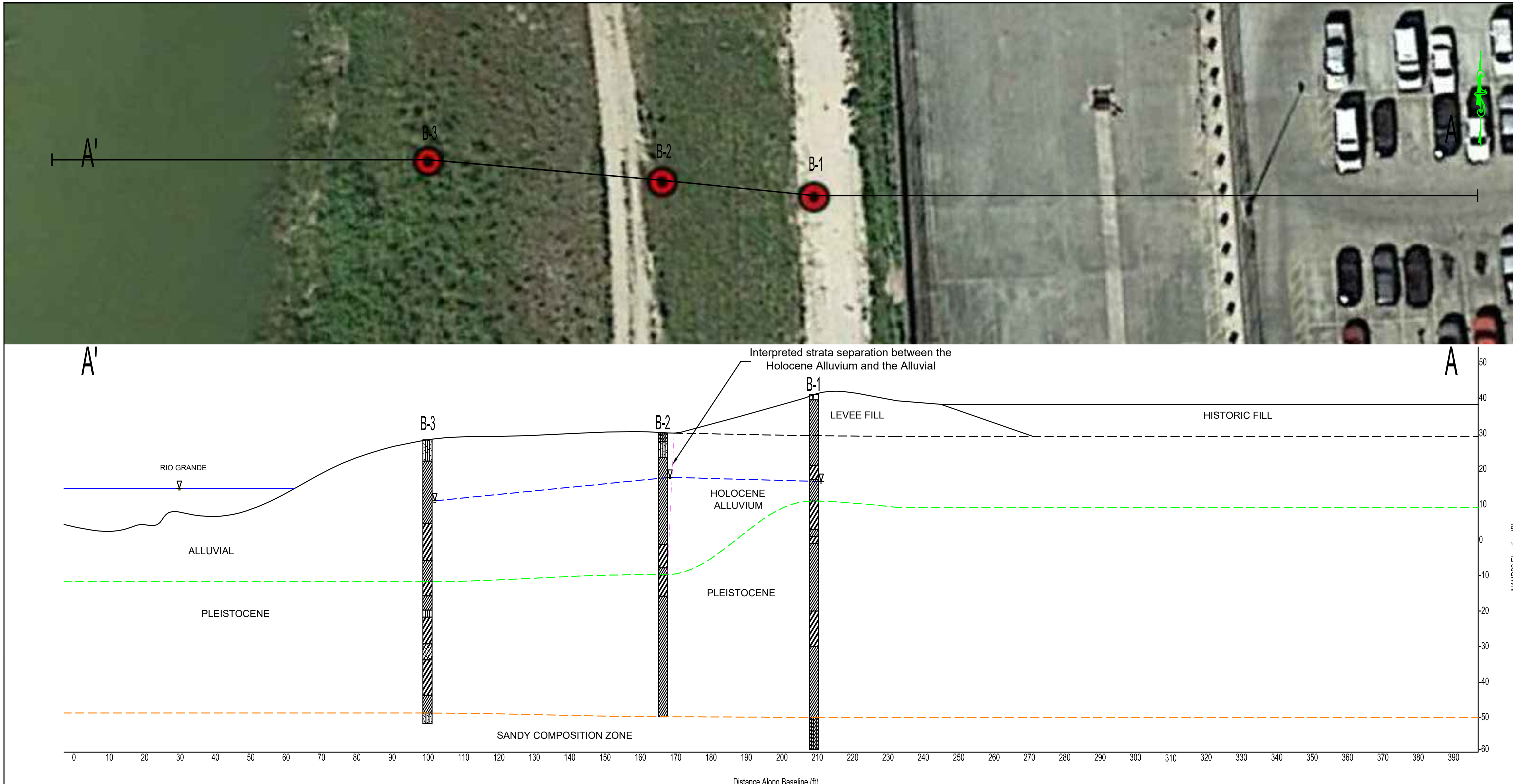
XREFS: PROJECTNAME: ----
B-SIZE-H-BORDER-FEET
EX-TOPO-CROSS SECTION2



LEGEND:

- SANDY SILT (ML)
- LEAN CLAY (CL)
- SILTY SAND (SM)
- SILTY CLAY (CL-ML)
- FAT CLAY (CH)
- CALICHE BASE MATERIAL
- SILT (ML)

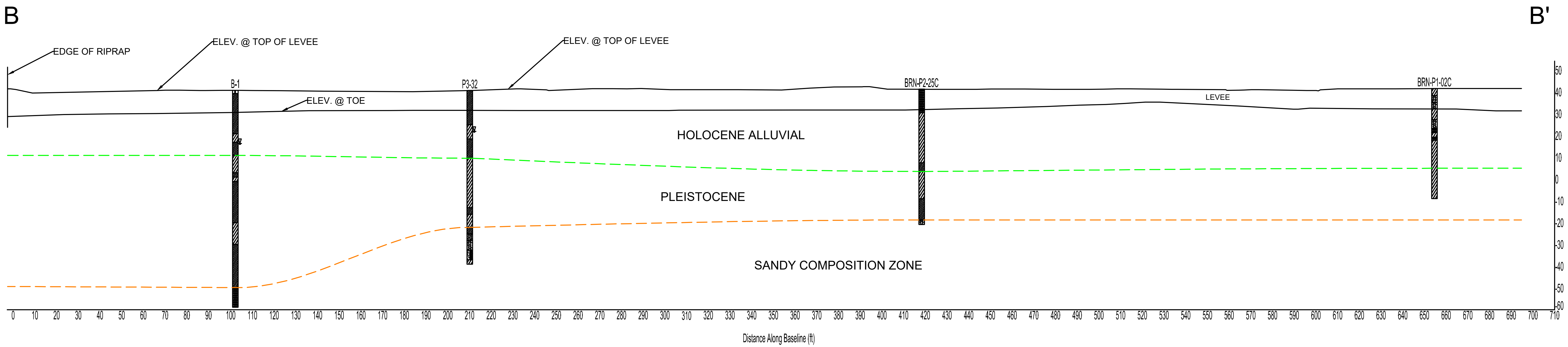
USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GEOLOGICAL CROSS SECTION A-A'

ARCADIS

FIGURE
2



LEGEND:

SANDY SILT (ML)
SILTY CLAY (CL-ML)
CALICHE BASE MATERIAL
LEAN CLAY (CL)
FAT CLAY (CH)
SILT (ML)
SILTY SAND (SM)
POORLY GRADED SAND WITH SILT (SP-SM)

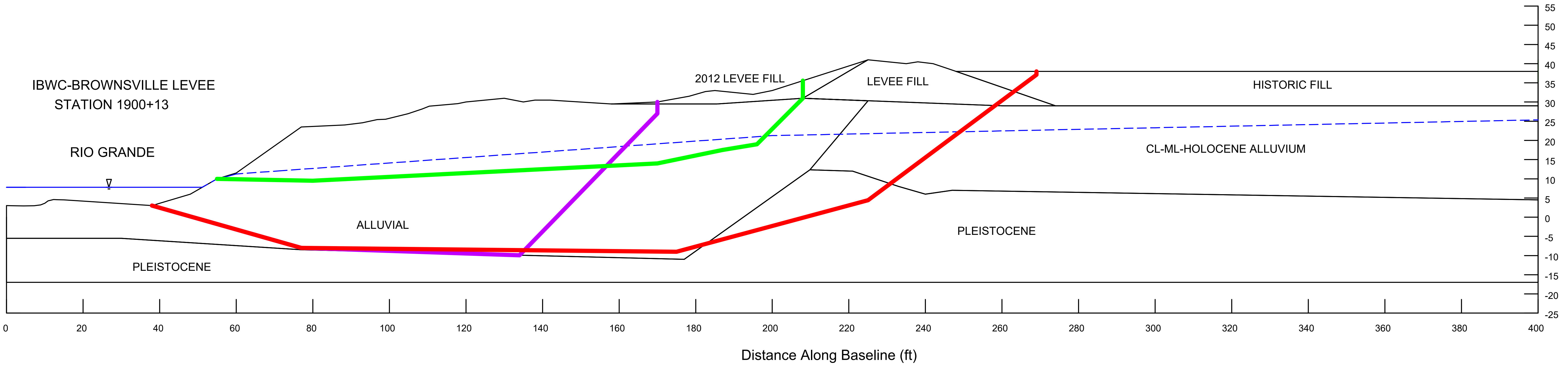
USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GEOLOGICAL CROSS SECTION B-B'

ARCADIS

FIGURE
3



LEGEND:

TOE FAILURE SURFACE

DEEP FAILURE SURFACE

USACE FAILURE SURFACE

WATER SURFACE

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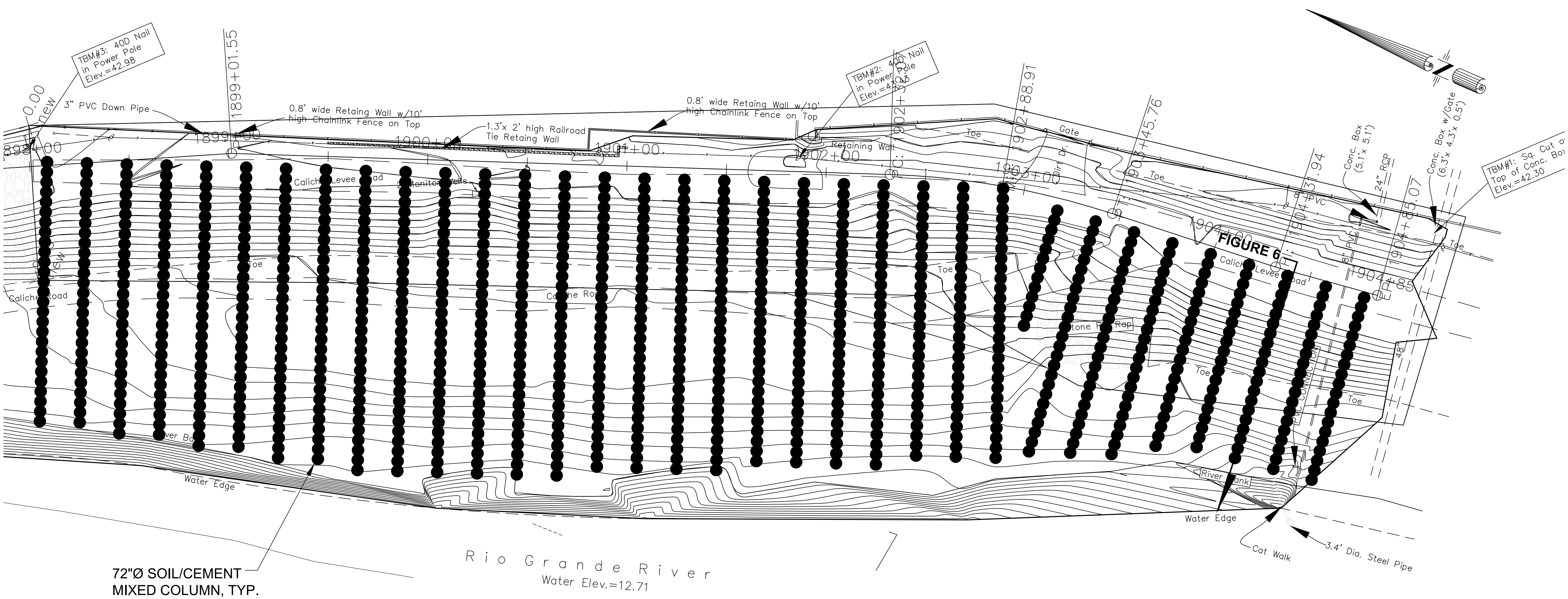
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

POTENTIAL FAILURE SURFACES

ARCADIS

FIGURE
4



NOTES:

1. SHEAR PANEL INSTALLATION: STA. 1898+00 TO STA. 1904+85
2. INDIVIDUAL COLUMN DIAMETER: 72"Ø SOIL/CEMENT MIXED COLUMN, OVERLAPPING 12"
3. WIDTH OF TREATMENT PERPENDICULAR TO C\L: 135' (AVERAGE)
4. CENTER TO CENTER COLUMN DISTANCE: 20' O.C.
5. BASE OF TREATMENT: EL. -15.0'

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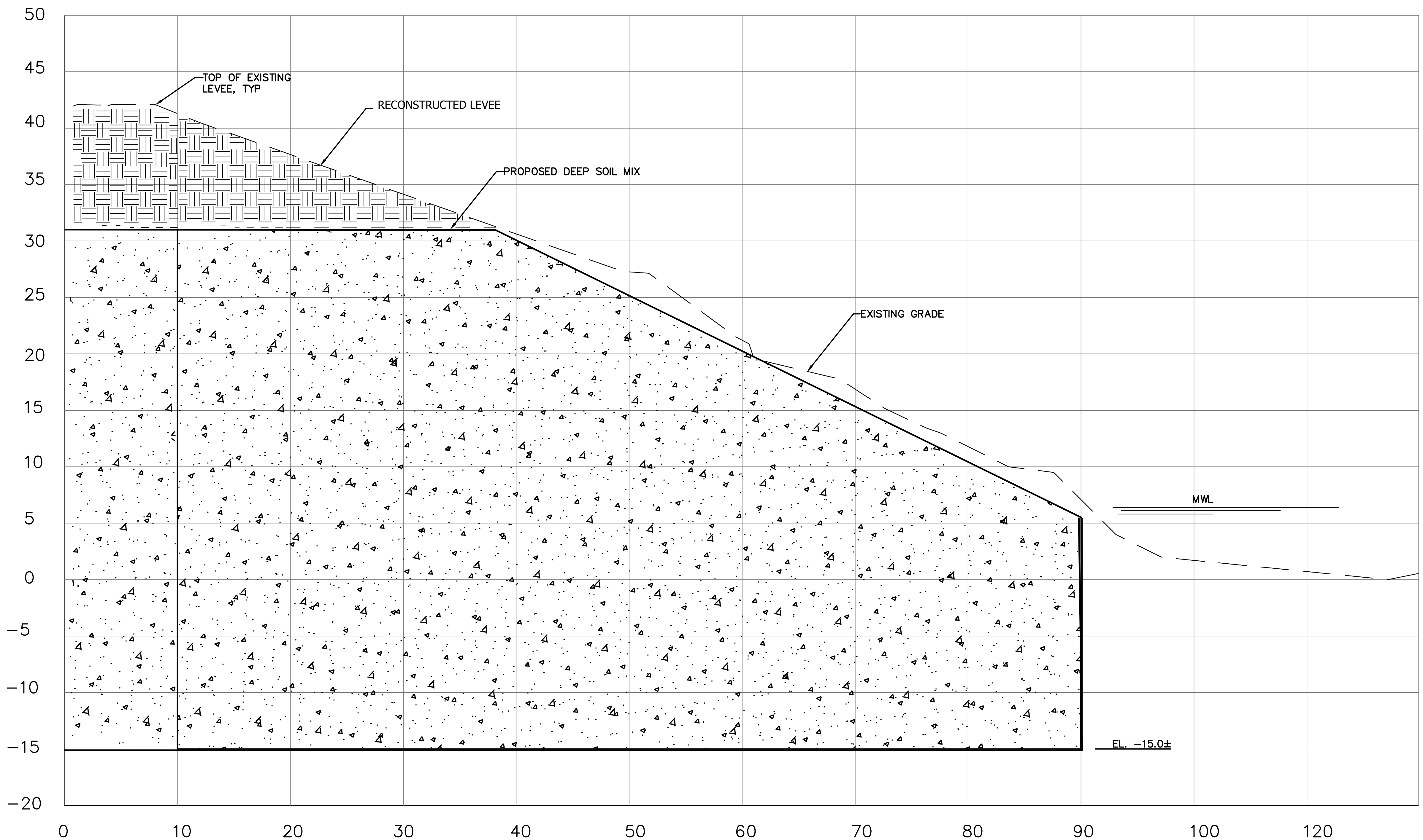
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

DEEP SOIL MIX CONFIGURATION

ARCADIS

FIGURE 5



NOTES:

1. LEVEE WILL BE LEVEL GRADED TO EL. 31.0'
2. SHEAR PANEL INSTALLATION: STA. 1898+00 TO STA. 1904.85
3. CENTER TO CENTER PANEL DISTANCE: 20' O.C.
4. BASE OF TREATMENT: EL. -15.0'
5. LEVEE WILL BE RECONSTRUCTED AT COMPLETION OF SHEAR PANEL INSTALLATION

CITY:(Reqd) DIV/GROUP:(Reqd) DB:(Reqd) LD:(Opt) PIC:(Opt) PM:(Reqd) TM:(Opt) LYR:(Opt)ON=*;OFF=*REF*
C:\Users\Alove.ARCADIS-US\Desktop\IBWC\FIGURE 5.dwg LAYOUT: LAYOUT1 SAVED: 12/2/2016 4:02 PM ACADVER: 19.1S (LMS TECH) PAGESETUP: ---- PLOTSTYLETABLE: ACAD.CTB PLOTTED: 12/2/2016 4:06 PM BY: LOVE, AVERY
XREFS: PROJECTNAME: ----
B-SIZE-H-BORDER-FEET
EX-TOPO-CROSS SECTION

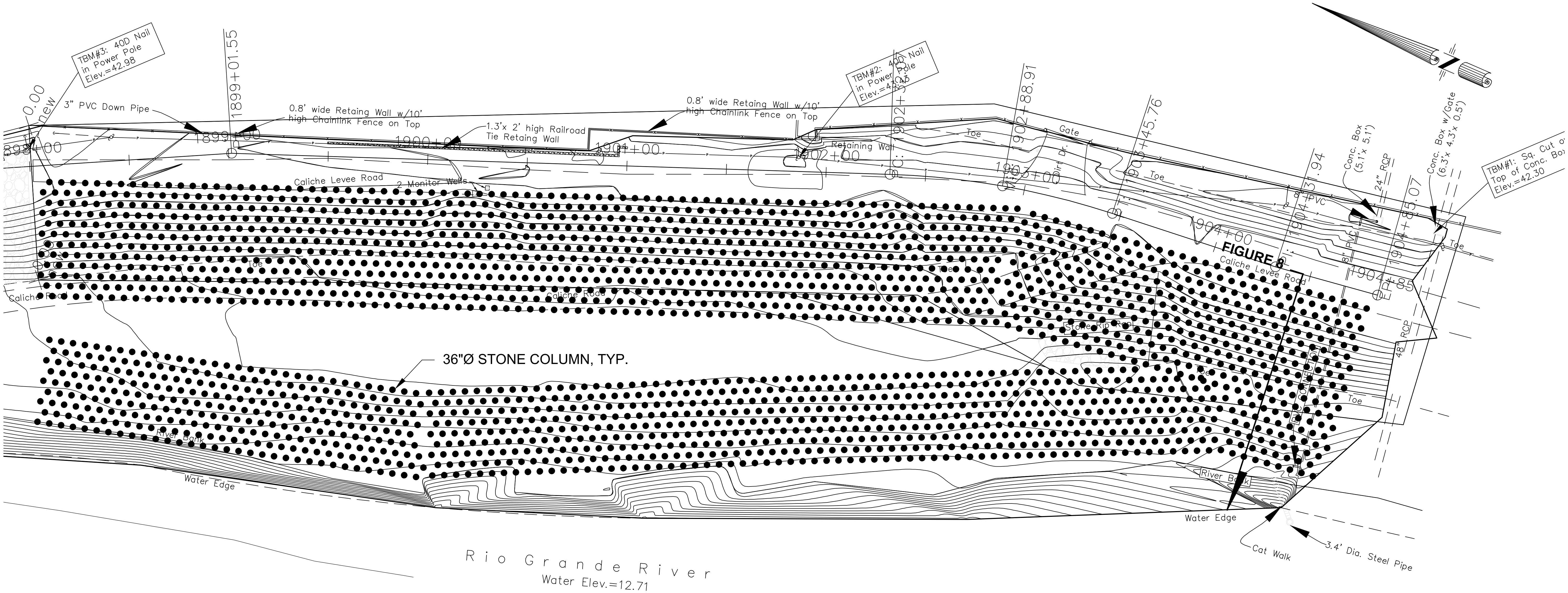
USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

DEEP SOIL MIX SECTION

ARCADIS

FIGURE
6



NOTES:

1. STONE COLUMN INSTALLATION: STA. 1898+00 TO STA. 1904+85
2. INDIVIDUAL COLUMN DIAMETER: 36"Ø STONE COLUMN WITH EQUILATERAL CONFIGURATION
3. WIDTH OF TREATMENT PERPENDICULAR TO C\L: 45' (APPROX.) AT TOE, 60' (APPROX.) AT LEVEE
4. CENTER TO CENTER COLUMN DISTANCE: 6' O.C.
5. BASE ELEVATION: -15.0'

0 50' 100'

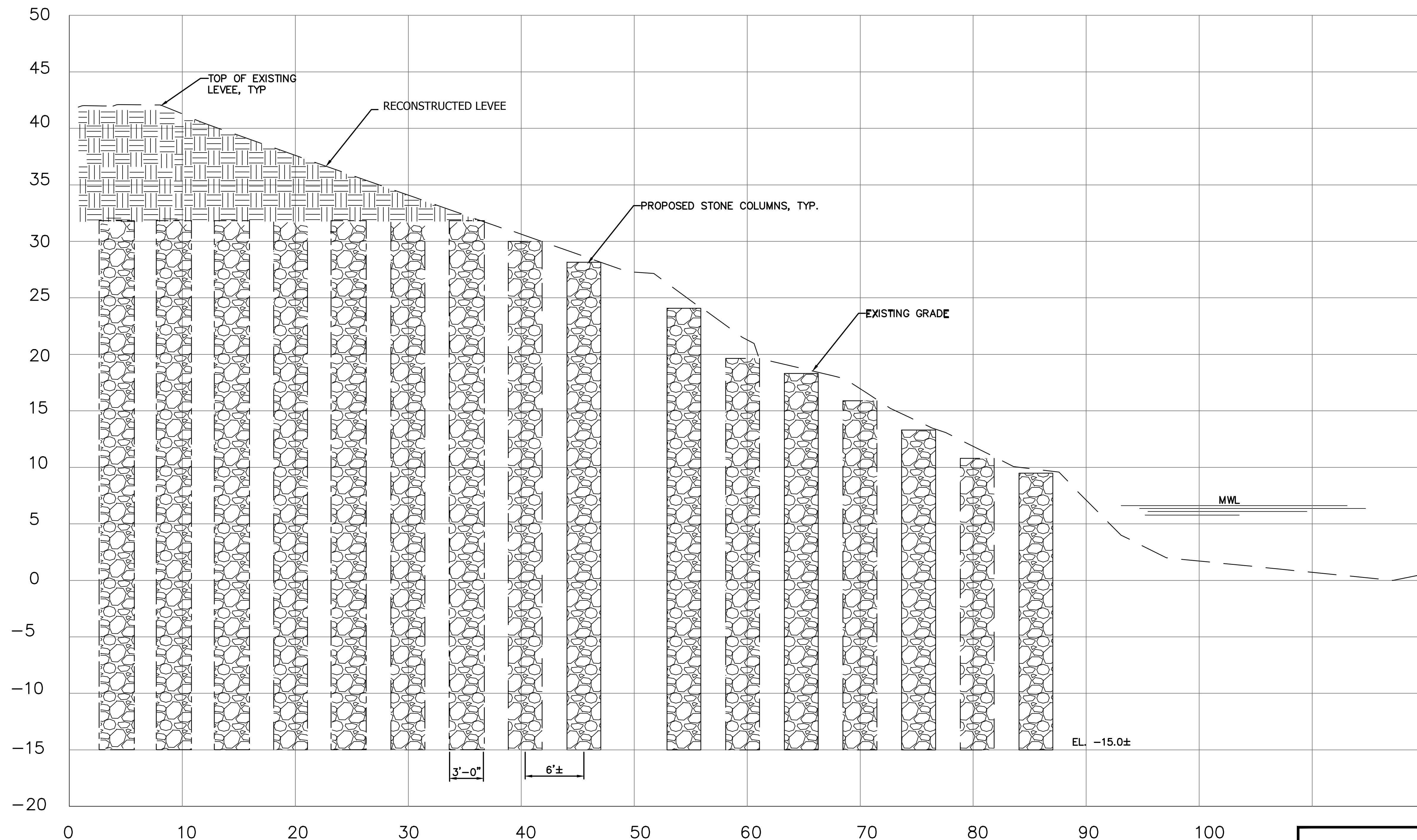
USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

STONE COLUMN CONFIGURATION

ARCADIS

FIGURE 7



NOTES:

1. STONE COLUMN INSTALLATION: STA. 1898+00 TO STA. 1904+85
2. INDIVIDUAL COLUMN DIAMETER: 36"Ø CONCRETE COLUMN WITH EQUILATERAL CONFIGURATION
3. WIDTH OF TREATMENT PERPENDICULAR TO C\L: 45' (APPROX.)
4. CENTER TO CENTER COLUMN DISTANCE: 6' O.C.
5. BASE ELEVATION: -15.0'
6. LEVEE WILL BE RECONSTRUCTED AT COMPLETION OF STONE COLUMN INSTALLATION

USIBWC
GEOTECHNICAL REPORT

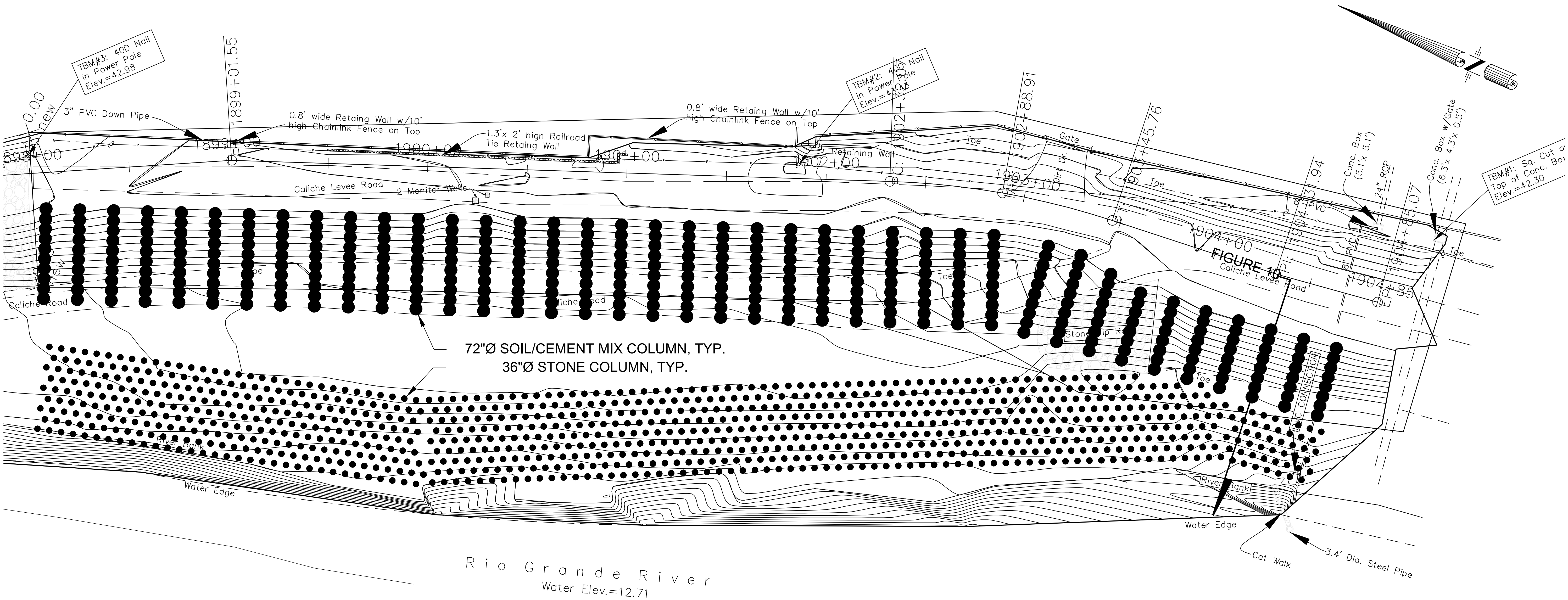
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

STONE COLUMN SECTION

ARCADIS

FIGURE
8

CITY:(Reqd) DIV/GROUP:(Reqd) DB:(Reqd) LD:(Opt) PIC:(Opt) PM:(Reqd) TM:(Opt) LYR:(Opt)ON=*;OFF=*REF*
C:\Users\Alove.ARCADIS-US\Desktop\IBWC\FIGURE 7.dwg LAYOUT: LAYOUT1 SAVED: 12/2/2016 4:02 PM ACADVER: 19.1S (LMS TECH) PAGESETUP: ---- PLOTSTYLETABLE: ACAD.CTB PLOTTED: 12/2/2016 4:13 PM BY: LOVE, AVERY
XREFS: PROJECTNAME: ----
B-SIZE-H-BORDER-FEET
EX-TOPO-CROSS SECTION



NOTES:

1. SHEAR PANEL AND STONE COLUMN INSTALLATION: STA. 1898+00 TO STA. 1904+85
2. INDIVIDUAL SOILS/CEMENT COLUMN DIAMETER: 72"Ø SOIL/CEMENT MIXED COLUMN, OVERLAPPING 12"
3. WIDTH OF SOIL/CEMENT TREATMENT PERPENDICULAR TO C\L: 45' (APPROX.)
4. CENTER TO CENTER PANEL DISTANCE: 17' O.C.
5. INDIVIDUAL STONE COLUMN DIAMETER: 36"Ø STONE COLUMN WITH EQUILATERAL CONFIGURATION
6. WIDTH OF STONE COLUMNS PERPENDICULAR TO C\L:45' (APPROX.)
7. CENTER TO CENTER STONE COLUMN DISTANCE: 6' O.C.
8. BASE ELEVATION: -15.0'

USIBWC
GEOTECHNICAL REPORT

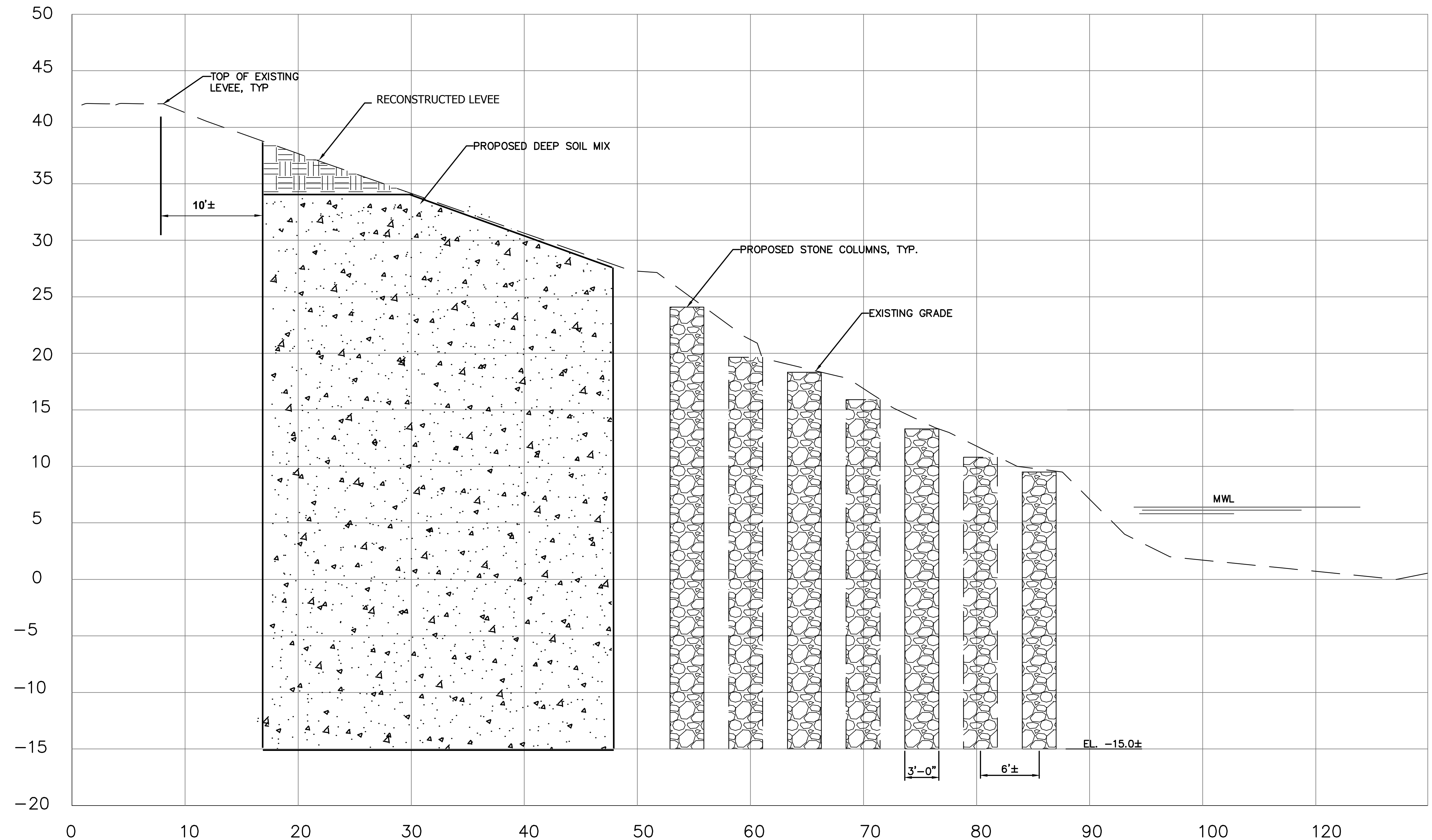
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

DEEP SOIL MIX AND STONE COLUMN CONFIGURATION

ARCADIS

FIGURE 9

CITY:(Reqd) DIV/GROUP:(Reqd) DB:(Reqd) LD:(Opt) PIC:(Opt) PM:(Reqd) TM:(Opt) LYR:(Opt)ON=*;OFF=*REF* C:\Users\lalove.ARCADIS-US\Desktop\IBWC\FIGURE 8.dwg LAYOUT: LAYOUT1 SAVED: 12/2/2016 4:02 PM ACADVER: 19.1S (LMS TECH) PAGESETUP: ---- PLOTSTYLETABLE: ACAD.CTB PLOTTED: 12/2/2016 4:07 PM BY: LOVE, AVERY
XREFS: B-SIZE-H-BORDER-FEET EX-TOPO-CROSS SECTION PROJECTNAME: ----



NOTES:
1. SHEAR PANEL AND STONE COLUMN INSTALLATION: STA. 1898+00 TO STA. 1904+85
2. INDIVIDUAL SOILS/CEMENT COLUMN DIAMETER: 72"Ø SOIL/CEMENT MIXED COLUMN, OVERLAPPING 12"
3. CENTER TO CENTER PANEL DISTANCE: 17' O.C.
4. INDIVIDUAL STONE COLUMN DIAMETER: 36"Ø STONE COLUMN WITH EQUILATERAL CONFIGURATION
5. CENTER TO CENTER STONE COLUMN DISTANCE: 6' O.C.
6. LEVEE WILL BE RECONSTRUCTED AT COMPLETION OF SHEAR PANEL INSTALLATION

USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

DEEP SOIL MIX AND STONE COLUMN SECTION

ARCADIS

FIGURE
10

APPENDIX A

Field Report

Field Report

Geotechnical Soil Exploration to the Rehabilitation of Levee and Floodplain Failure

within Upper Brownsville Levee Reach

International Boundary and Water Commission

Brownsville, Texas

Cameron County

June 23, 2016

On Monday June 13, 2016, Arcadis U.S., Inc. (Arcadis), Professional Services, Inc. (PSI) and JEDI Drilling (JEDI) arrived at the project site to do a geotechnical investigation of the Brownsville Upper Levee Segment in Cameron County, Texas. As part of the investigation, geotechnical borings were sampled and advanced, and inclinometers were installed to assess the subsurface conditions of the soils and to monitor the movement of the levee.

The drilling and sampling procedures are as follows:

1. Continuous flight augering with sampling at 3- to 5-foot intervals between the depths of 0 feet to 20 feet.
2. Mud rotary wash with continuous sampling, every 2 feet, between the depths of 20 to 50 feet.
3. Mud rotary wash with sampling at 3- to 5-foot intervals from a depth of 50 feet to the boring completion depth.

The task performed on site by Arcadis, PSI and JEDI on Monday June 13, 2016 through Saturday June 18, 2016 are as follows:

<u>Monday June 13, 2016:</u>

- The project team arrived on site and held a safety briefing to review the site Health and Safety Plan with discussion of drilling operations and the installation of inclinometers.
- JEDI drilled and sampled the borehole located at the toe of the levee, Boring B-2, and Arcadis with PSI documented the soil properties and packaged the samples.
- Boring B-2 was completed to a total depth of 80 feet.
- The inclinometer casing for ARC-2 was installed to a depth of 78 feet with two feet of stick-up above ground surface within Boring B-2. The pumping of the grout mixture was postponed to June 14, 2016.

<u>Tuesday June 14, 2016:</u>

- The project team arrived on site and held a safety briefing to review the site Health and Safety Plan.
- JEDI mixed and pumped the grout mixture into the annular space between ARC-2 and the borehole.
- Arcadis and PSI measured the three existing inclinometers, I-32, I-33 and I-34. Inclinometers I-32 and I-34, had a zone approximately 30 feet below the ground surface that the probe could barely

pass. During the measurement process, the probe rotated itself 90 degrees clockwise within the casing from the A-axis direction or perpendicular to the levee and came out on the B-axis direction or parallel to the levee. Therefore, these readings will not be able to provide a useful plot but the depth of the zone movement was confirmed. At Inclinometer I-33, the probe could barely pass about 35 feet deep and no useful readings could be obtained, similar to the January 2016 reading.

- JEDI started drilling and sampling the borehole located at the edge of the riverbank, Boring B-3, and Arcadis with PSI documented the soil properties and packaged the samples.
- During the drilling operation a major gravel layer was encountered that complicated the sampling and recovery. Water in the boring was not able to be maintained because of the gravel layer even using drilling mud. For this reason the hole was abandoned and a new boring closer to the river was advanced.
- A second borehole was drilled and sampled with an offset of approximately 10 feet towards the river in the same alignment from the previous boring. Sampling started at a depth of 25 feet, since the first 25 feet was logged with the initial hole.
- A Shelby tube was lost just after taking the sample at a depth of 38 feet, and the boring could not be advanced due to the tube. Therefore, the second borehole was abandoned and a new boring located towards the levee was advanced.
- A third borehole was located with an offset of 3 feet to 5 feet towards the levee between the first and second boreholes. Drilling did not commence on this borehole until June 15, 2016.

<u>Wednesday June 15, 2016:</u>

- The project team arrived on site and held a safety briefing to review the site Health and Safety Plan.
- JEDI started drilling and sampling at a depth of 38 feet in the third borehole, and Arcadis with PSI documented the soil properties and packaged the samples.
- JEDI installed the protective cap (type: monument case) and constructed the 2-foot by 2-foot by 4-inch concrete pad to protect inclinometer ARC-2.
- PSI performed the Pile Integrity Test (PIT) on the existing wooden bulkhead in the levee. The test was performed in three segments along the structure: edge looking north, middle span and edge looking south.
- Boring B-3 was completed to a total depth of 80 feet.
- The inclinometer casing for ARC-3 was installed to a depth of 78 feet with two feet of stick-up within Boring B-3.
- The grouting mixture was pumped into the annular space between ARC-3 and the borehole.

<u>Thursday June 16, 2016:</u>

- The project team arrived on site and held a safety briefing to review the site Health and Safety Plan.
- JEDI drilled and sampled the borehole located at the top of the levee, Boring B-1, and Arcadis with PSI documented the soil properties and packaged the samples.
- JEDI installed the protective cap (type: monument case) and constructed the 2-foot by 2-foot by 4-inch concrete pad to protect inclinometer ARC-3.

- The initial and second borehole drilled on June 14, 2016 were backfilled with grout.
- PSI performed the second PIT test on the existing wooden bulkhead to gather more data. The test was executed in the same manner as the first test.
- Boring B-1 was completed to a total depth of 100 feet.
- The inclinometer casing for ARC-1 was installed to a depth of 98 feet with two feet of stick-up within Boring B-1. The pumping of the grout mixture was postponed to June 17 2016.

Friday June 17, 2016:

- The project team arrived on site and held a safety briefing to review the site Health and Safety Plan.
- JEDI mixed and pumped the grout into the annular space between ARC-1 and the borehole.
- JEDI installed the protective cap (type: monument case) and constructed the 2-foot by 2-foot by 4-inch concrete pad to protect inclinometer ARC-1.
- Arcadis and PSI measured the existing inclinometer I-32 for a second time to verify the data. During the measurement process, the probe had a minor difficulty passing through the casing between the depths of 30 to 35 feet. It was observed that between those depths the inclinometer probe was jumping/transitioning from 90 degrees clockwise from its original position. To confirm the depth in which the probe was jumping the track, the same process was repeated without taking any measurements and it showed that the inclinometer probe was shifting between depths of 30 to 35 feet. For that reason these readings will not be able to provide a useful plot but the depth of the zone movement was confirmed.
- JEDI drilled and sampled the boring located near the North Abutment of the Gateway Bridge, and Arcadis with PSI documented the soil properties and packaged the samples.
- Boring B-4 was completed to a total depth of 80 feet.
- Arcadis and PSI recorded baseline readings for the new inclinometers ARC-2 and ARC-3.
- The inclinometer casing for ARC-4 was installed to a depth of 78 feet with 2 feet of stick-up above the ground surface with Boring B-4. The pumping of the grout mixture was postponed to June 18, 2016.

Saturday June 18, 2016:

- JEDI mixed and pumped the grout into the annular space between ARC-4 and the borehole.
- JEDI installed the protective cap (type: monument case) and constructed the 2-foot by 2-foot by 4-inch concrete pad to protect inclinometer ARC-4.

Tuesday June 21, 2016:

- Munoz & Dannenbaum Joint Venture surveyed the inclinometers

Wednesday June 22, 2016:

- PSI completed the baseline inclinometer readings for ARC-1 and ARC-4.

APPENDIX B

Boring Logs

Removed to Eliminate
Redundancy

APPENDIX C

July 2017 Inclinometer Report

MEMO

Arcadis U.S., Inc.
10352 Plaza Americana Drive
Baton Rouge
Louisiana 70816
Tel 225 292 1004
Fax 225 218 9677

To:
Edgar Iniguez
Contracting Officer Representative
IBWC

Copies:
Padinare Unnikrishna (USIBWC)
Armando Flores (Arcadis)

From:

Kirk Lowery, P.E.

Date:

July 18, 2017

Arcadis Project No.:

LA003315.0000

Subject:
July 2017 Final Summary Report of Inclinometer Readings
Remediation Design of Levee Floodplain Failure within the Upper Brownsville Levee Reach Lower Rio Grande Flood Control Project – IBM15D0001 – IBM15T0015

1. Introduction

Arcadis U.S., Inc. (Arcadis), is pleased to submit this summary technical memorandum including data charts of the slope inclinometer readings at the IBWC site. The baseline readings for the new inclinometers, ARC-1, ARC-2, ARC-3 and ARC-4, were taken in June 2016 and the eleventh set of readings were measured on July 12, 2017. Under the current scope of work, this month will be the last measurement to be taken for these inclinometers.

Exact readings for the existing inclinometers, I-32, I-33 and I-34, were not made when Arcadis visited the site July 12, 2017. However, the probe was placed in the inclinometer casings and the depth at which it would not pass was recorded. Arcadis measured the depth range in which the inclinometer probe could not pass through the constricted area of the pipe and are as follows:

I-32 (Top of the Levee): Depth Range: 32 feet

I-33 (Toe of the Levee): Depth Range: 38 feet

I-34 (Below Toe of Levee): Depth Range: 30 feet to 31 feet

In this month's visit to the IBWC site, several pins that were installed on June 14, 2017 on top of the levee near inclinometer ARC-1 and south of the existing retaining wall have been removed or disturbed due to the traffic on the levee roadway. In addition, recent rainfall has started to erode the surface tension cracks and they are becoming less apparent. All the pins that were removed, were set back to their original locations to take this month's measurements. Over the last month, the tension cracks appear to have not changed except for the cracks that are located between the pins 18B and 19B. For those pins, there was a slight increase in width of 0.5 inch. The locations of the cracks and the pins are shown in Attachment B while Attachment C presents photos of the tension cracks.

Table 1 summarizes the pin flags location along the cracks.

Table 1. Pin Flag Locations

Pin Flag No.	Pin Location	Remarks
1B and 2B	3.33' south of inclinometer ARC-1. 8.5" between the two pin flags.	No Change
3B and 4B	11.25" south of inclinometer ARC-1. 9.5" between the two pin flags.	No Change
5B	GPS coordinates N25.898212°, W-97.496628°	-
6B and 7B	7' south of 5B. 9.0" between the two pin flags.	Pin 6B was removed
8B and 9B	16' south of 5B. 9.75" between the two pin flags.	Pins were removed
10B	24' south of 5B.	-
11B	GPS coordinates N25.897973°, W-97.496493°	-
12B and 13B	12' south of 11B. 11.0" between the two pin flags.	Pins were removed
14B and 15B	42' south of 11B. 10.5" between the two pin flags.	Pins were removed
16B	57' south of 11B.	-
17B	6' west of 14B and 15B.	-
18B and 19B	10.5' south of 17B. 12.25" between the two pin flags.	0.5 inch increase
20B	21' south of 17B.	
21B		-

	GPS coordinates N25.897669º, W-97.496309º	
22B and 23B	18' south of 21B. 9.75" between the two pin flags.	Pins were removed
24B and 25B	54' south of 21B. 10.0" between the two pin flags.	Pin 24B was removed
26B	72' south of 21B.	-

The readings for each inclinometer are reflected in the graphical displays provided in Attachment A. Attachment A includes both incremental and cumulative displacement plots. Attachment B shows the inclinometer locations on a Google Map.

The incremental displacement plot compares the mean deviation data to the baseline survey file. This plot reveals the exact depth where displacements are actually occurring. The cumulative displacement is the sum of the displacements from the base of the borehole. This plot shows the change in the position of the casing from the first set of readings.

The A-axis charts in the displacement plots show displacements in the plane perpendicular to the levee while the B-axis charts show displacements in the plane parallel with the levee. A positive reading in the A-axis chart indicates displacement to the west heading toward the Rio Grande, and a positive reading in the B-axis chart indicates displacement to the north heading toward the Gateway Bridge.

2. Digitilt AT Inclinometer

Digitilt AT system was used to survey the inclinometers. The system components include an inclinometer probe, control cable, a Bluetooth reel and the Digitilt Reader app for certified Android-based tablet computer. The equipment is shown in Figure 1.

Figure 1: Digitilt AT System Components.

3. July 2017 Inclinometers Assessment

The depth of the casing restriction for the USACE installed inclinometers, I-32, I-33 and I-34 appears to be the same depth as the previous readings.

Data collected on July 12, 2017 followed the same trend as the baseline reading measured in June 2016. The monthly displacement plots recorded between July 2016 through July 2017 are presented in Attachment A. Data comparisons for each inclinometer are described below:

<u>Inclinometer ARC-1</u>: The base readings for inclinometer ARC-1 were collected on June 22, 2016. The ARC-1 cumulative plot in the A-Axis direction shows a slight progressive movement starting at depths between 28 and 30 feet. This depth corresponds with the interpreted Alluvium/Pleistocene interface presented in Figure 2 of Arcadis' December 2, 2016 *Draft Geotechnical Assessment Report*. Comparing the measurements taken in July 2016 to July 2017, the displacement is 0.12 inches towards the Rio Grande (A-Axis) at a depth of 28 feet. The displacement parallel to the levee does not show any sign of movement in this month's readings.

<u>Inclinometer Arc-2</u>: The base readings for inclinometer ARC-2 were collected on June 17, 2016. The ARC-2 cumulative displacement plot in the A-Axis direction shows a slight displacement between the depths of 38 feet to 40 feet. This depth corresponds with the interpreted Alluvium/Pleistocene interface presented in Figure 2 of Arcadis' December 2, 2016 *Draft Geotechnical Assessment Report*. Comparing the measurements taken in July 2016 to July 2017, the displacement is 0.13 inches towards the Rio Grande (A-Axis) at a depth of 38 feet. The displacement parallel to the levee does not show any sign of movement in this month's readings.

<u>Inclinometer ARC-3</u>: The base readings for inclinometer ARC-3 were collected on June 17, 2016. The ARC-3 cumulative and incremental displacement does not show any sign of movement on the plane perpendicular to the levee nor on the plane parallel to the levee.

<u>Inclinometer ARC-4</u>: The base readings for inclinometer ARC-4 were collected on June 22, 2016. The ARC-4 cumulative and incremental displacement does not show any sign of movement on the plane perpendicular to the levee nor on the plane parallel to the levee.

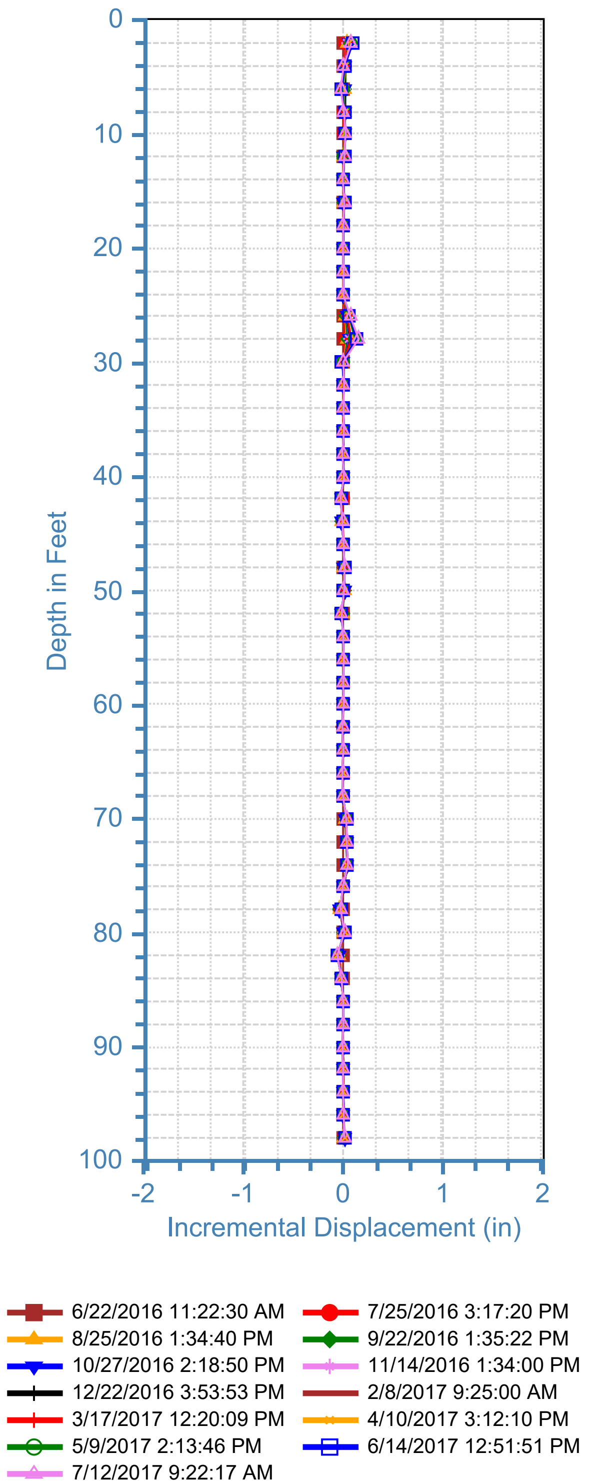
After critically reviewing the cumulative displacement plots, the graphical displays in Attachment A shows that in each month there is an increase in movement towards the Rio Grande (A-Axis direction) for the inclinometers ARC-1 and ARC-2. The displacement for these inclinometers are progressive but moving at a slow rate. This information will be summarized and used to finalize the *Draft Geotechnical Assessment Report*.

ATTACHMENTS:
A – Inclinometer Plots
B – Inclinometer and Levee Cracking Location Map
C – Photos of Surface Tension Cracks

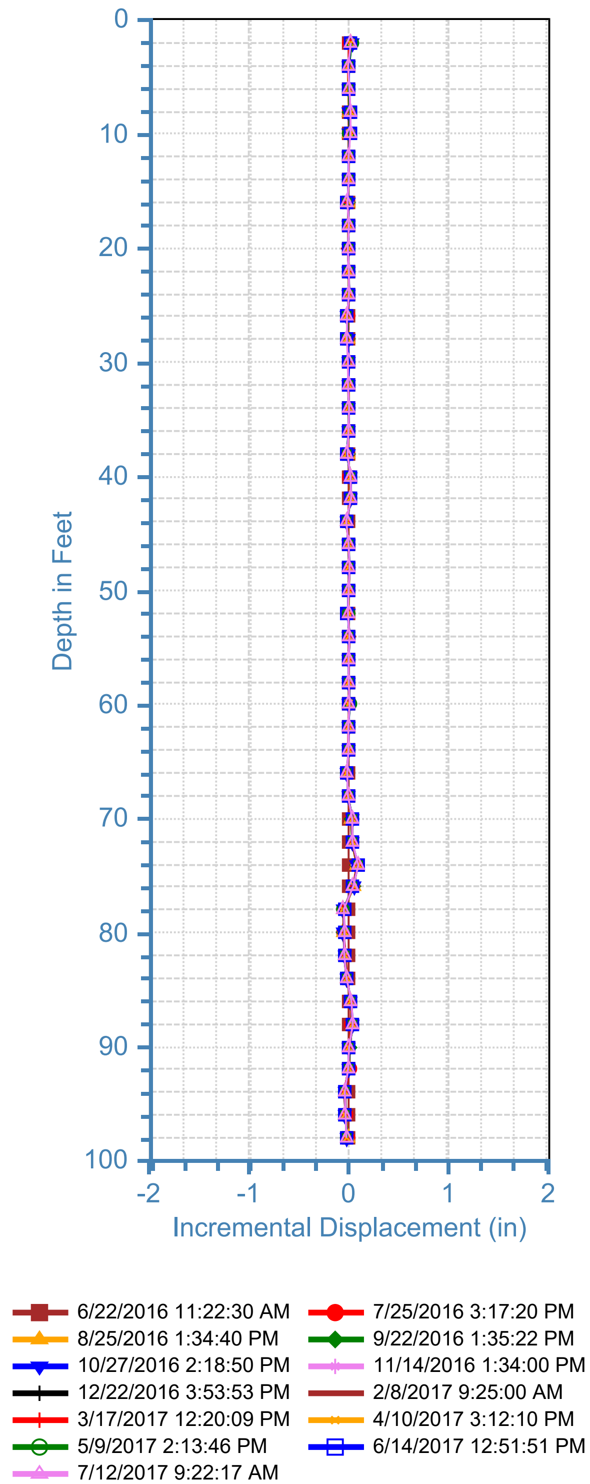
ATTACHMENT A

INCLINOMETER PLOTS

IBWC Arc-1 A - Axis

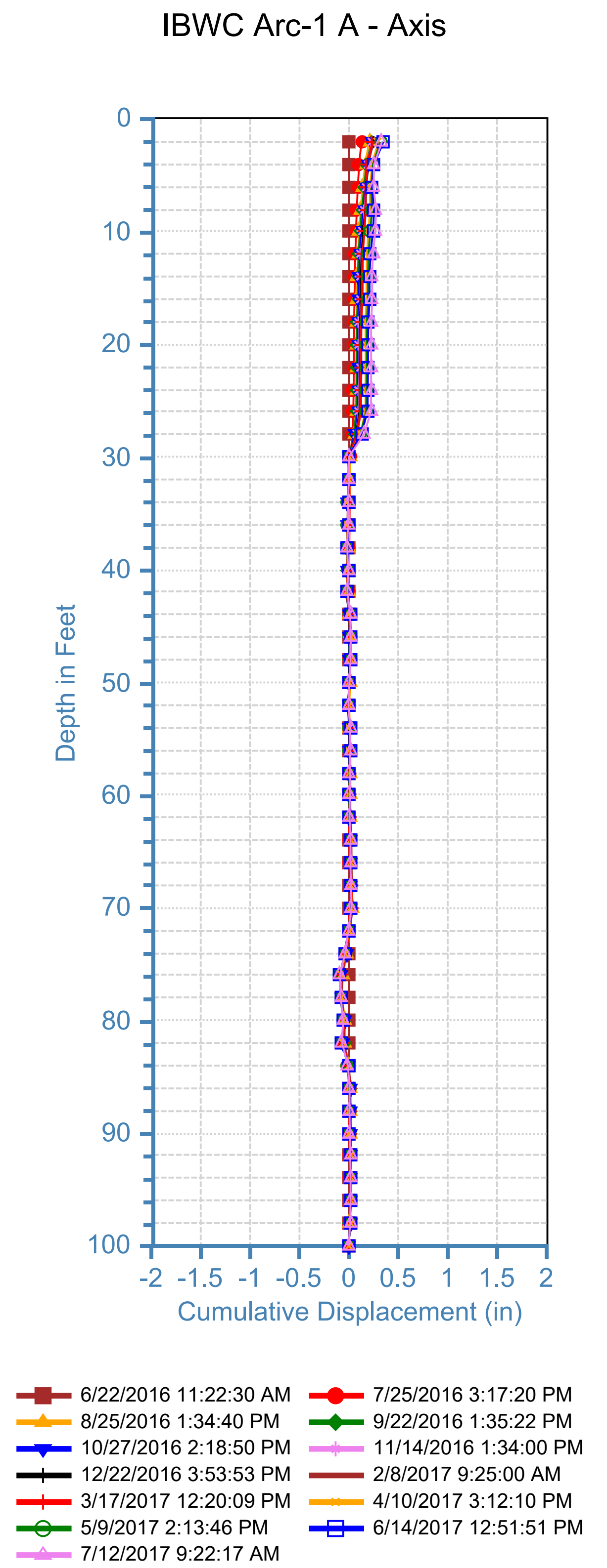


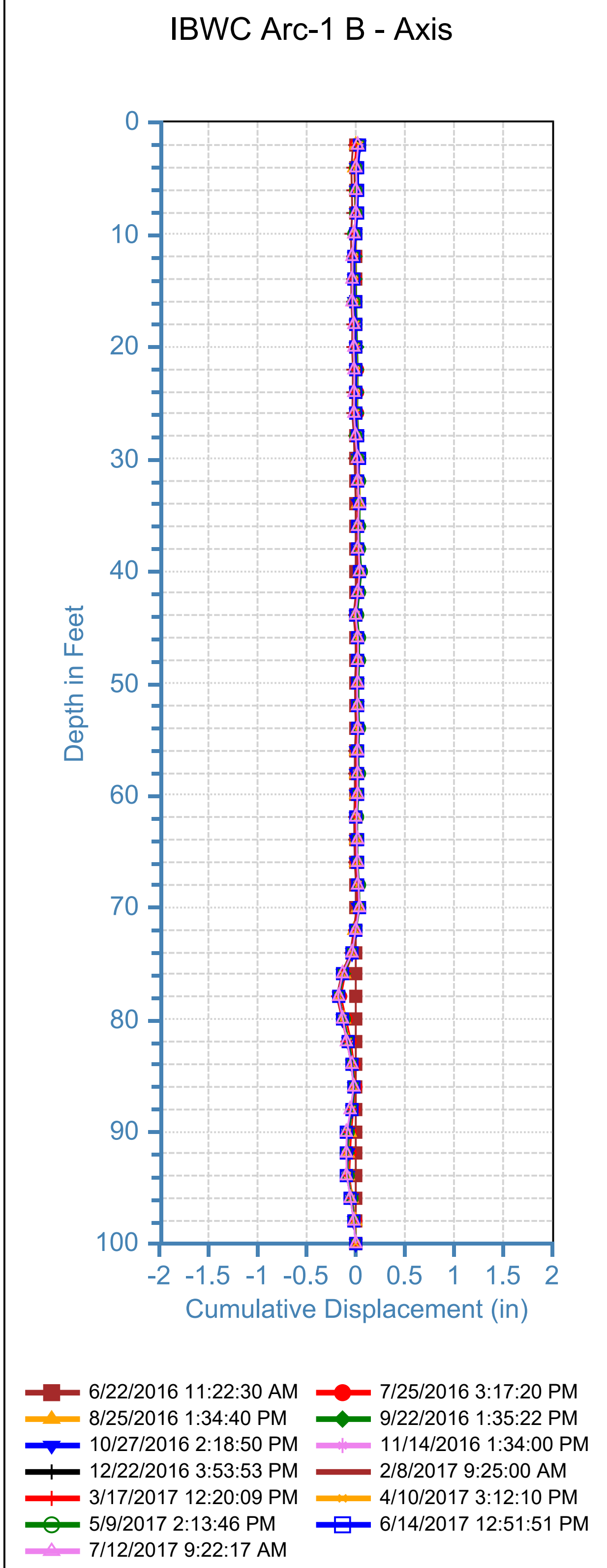
IBWC Arc-1 B - Axis



Base reading on 6/22/2016

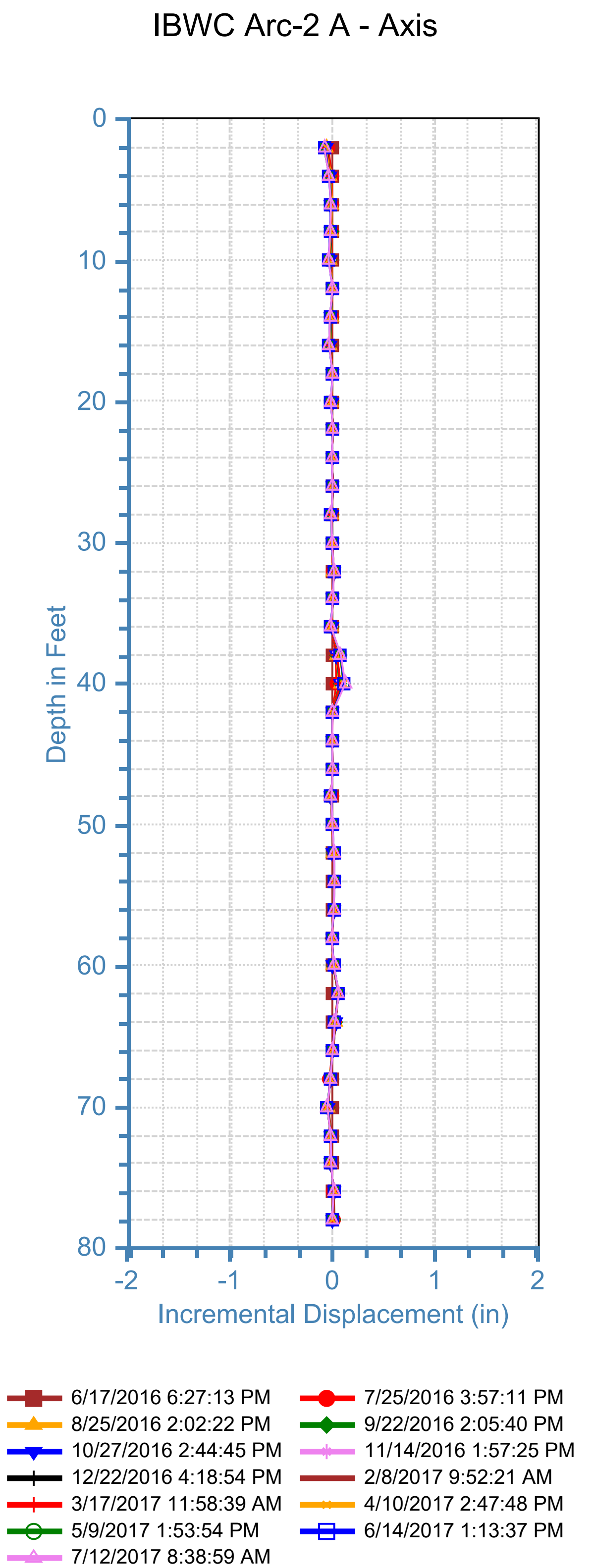
ARCADIS

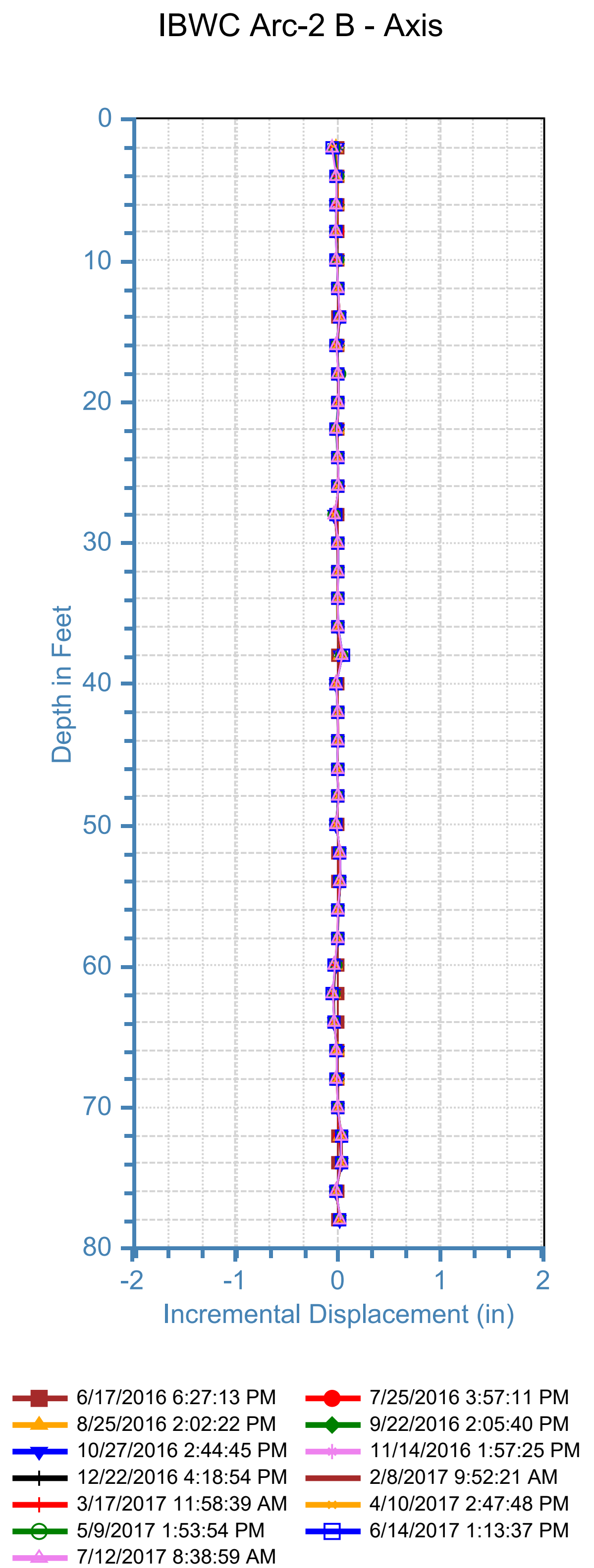




Base reading on 6/22/2016

ARCADIS





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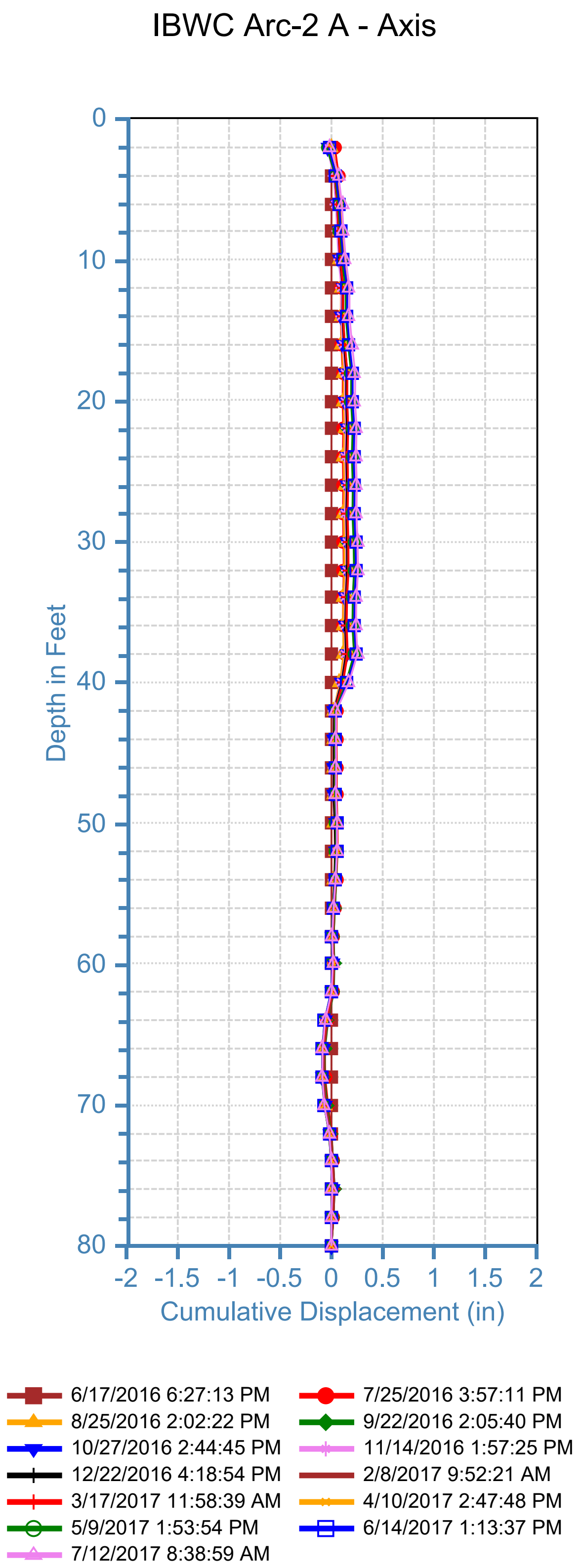

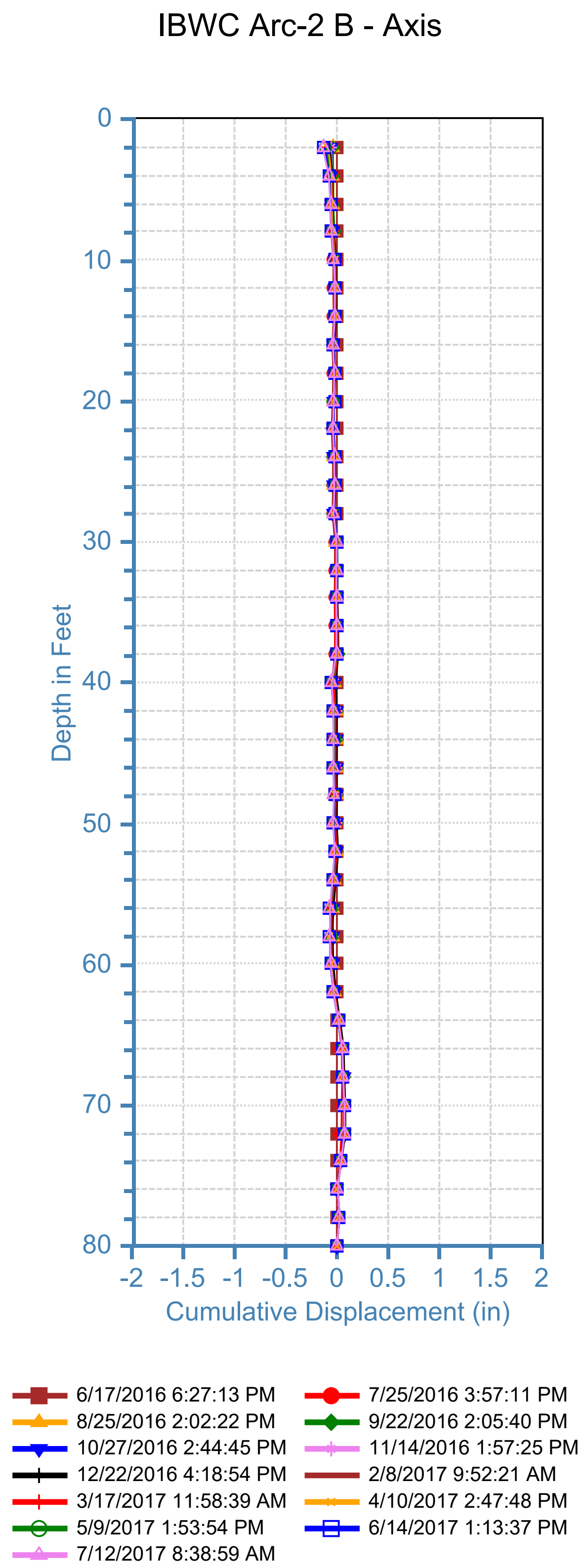

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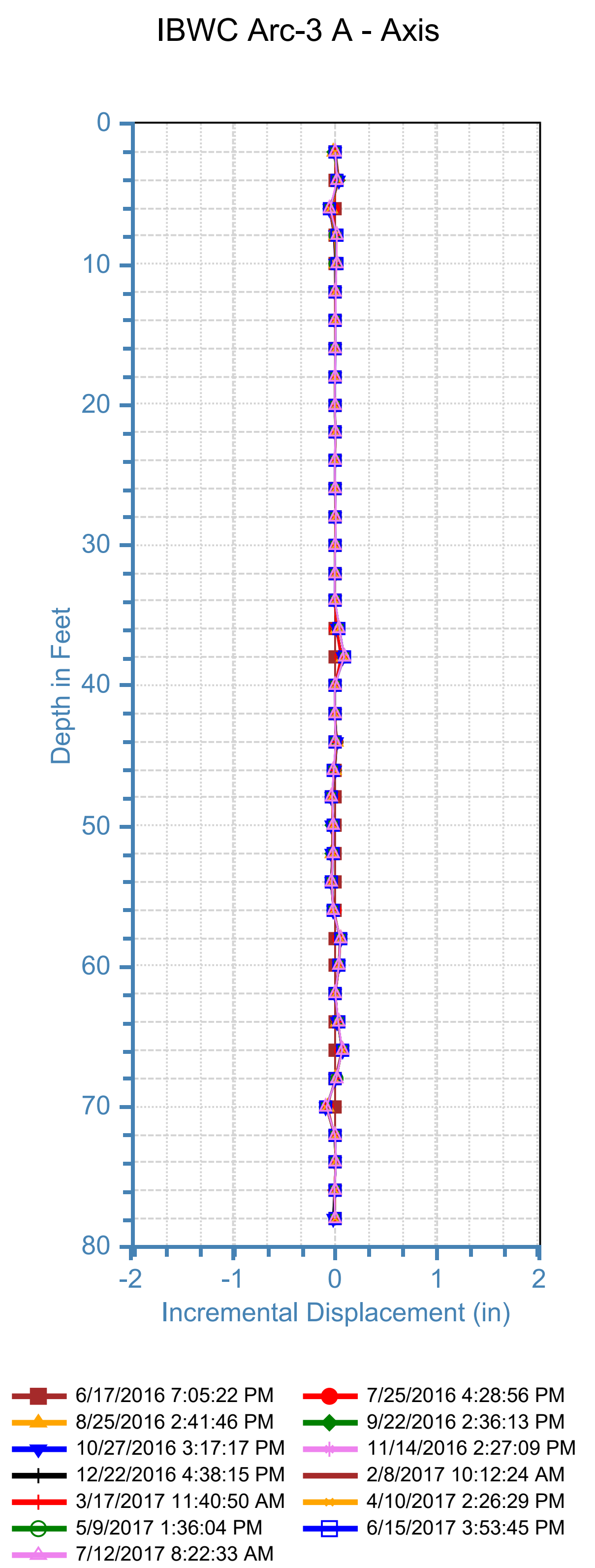

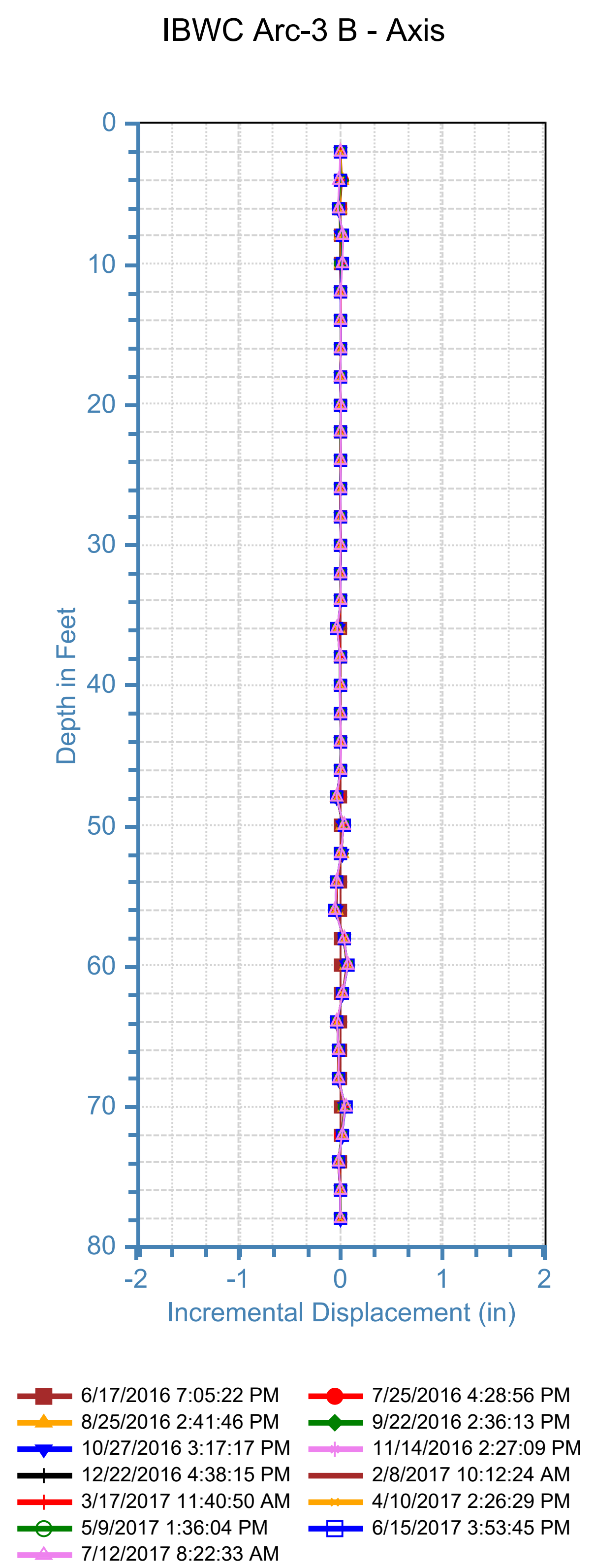

Base reading on 6/17/2016

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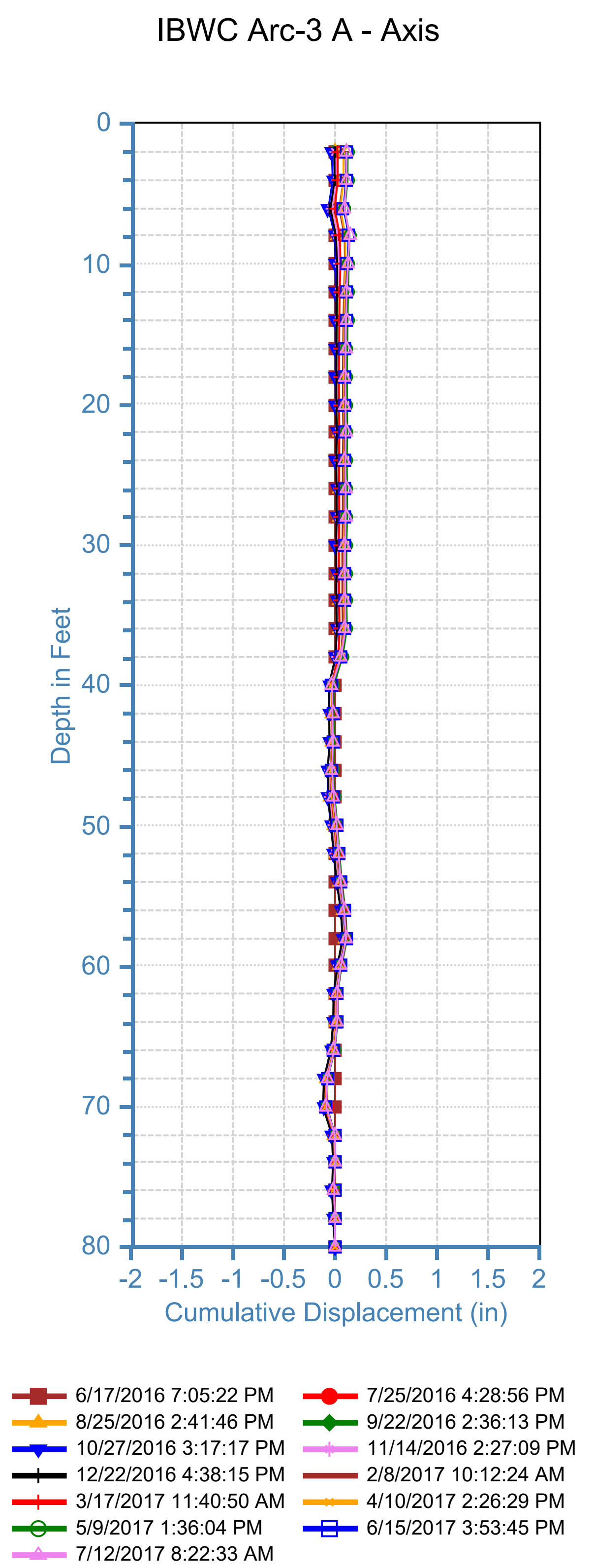

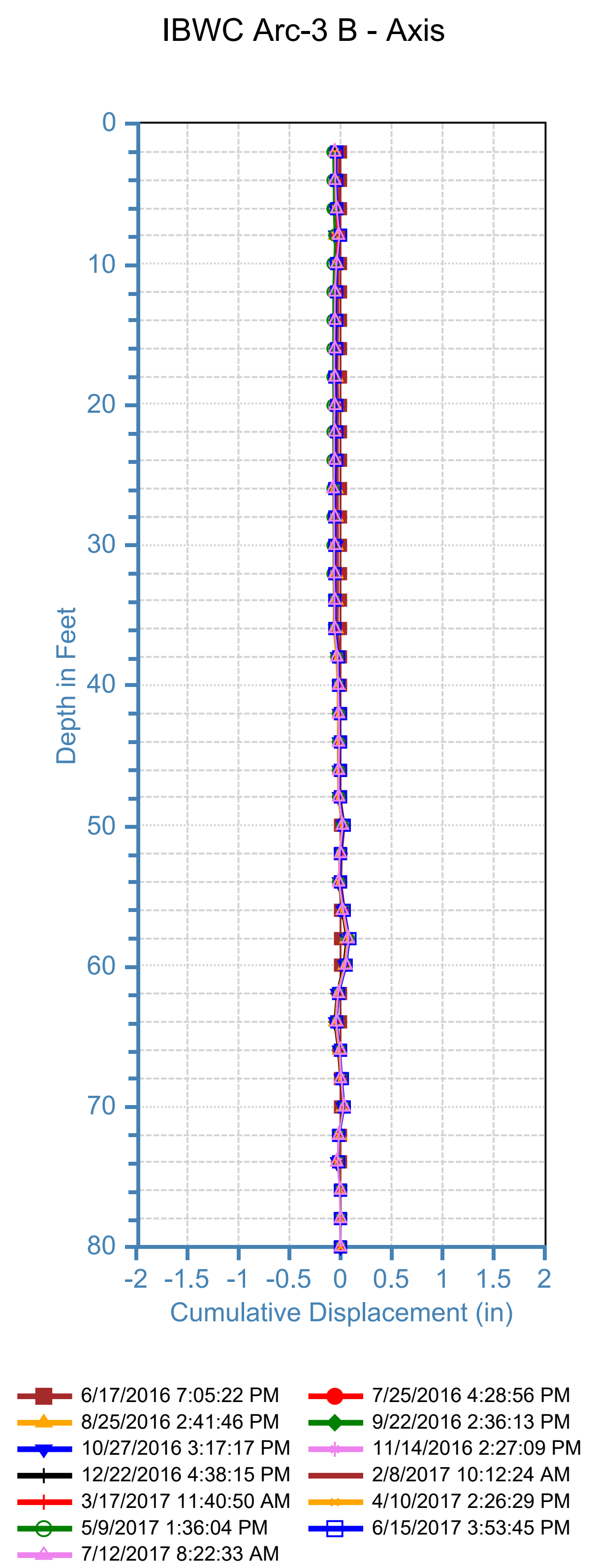

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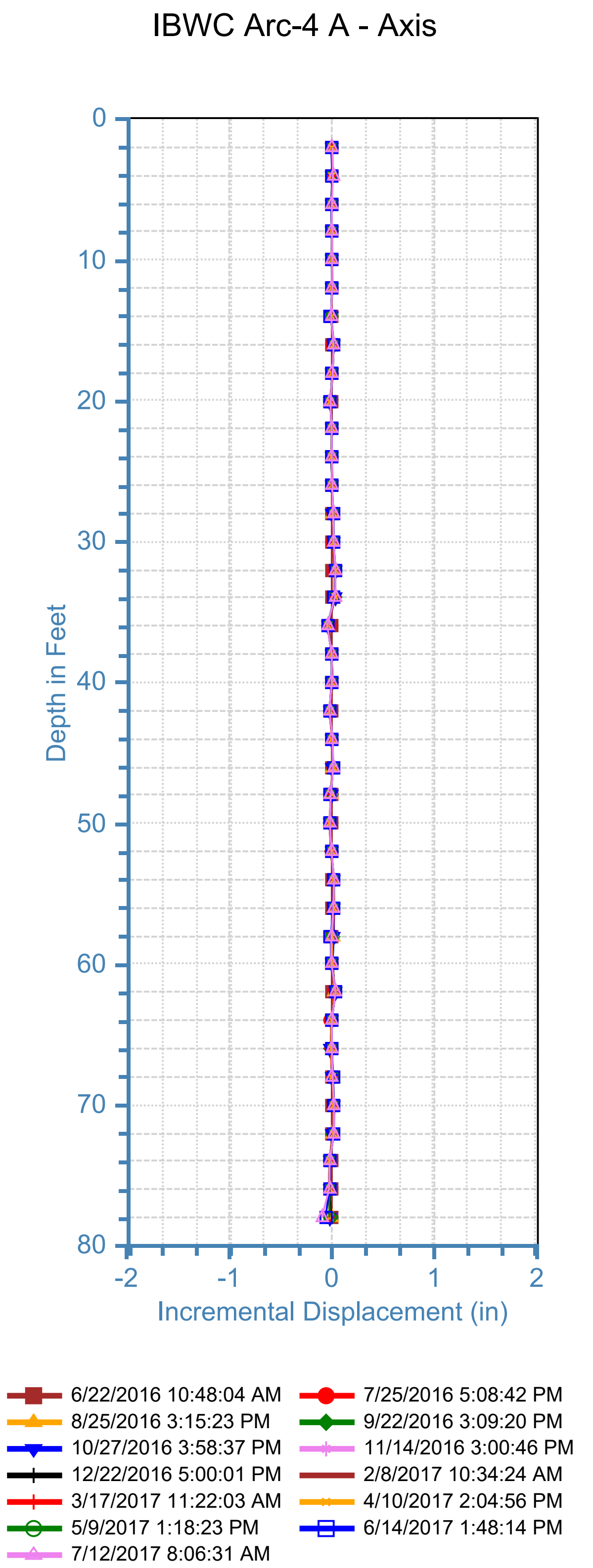
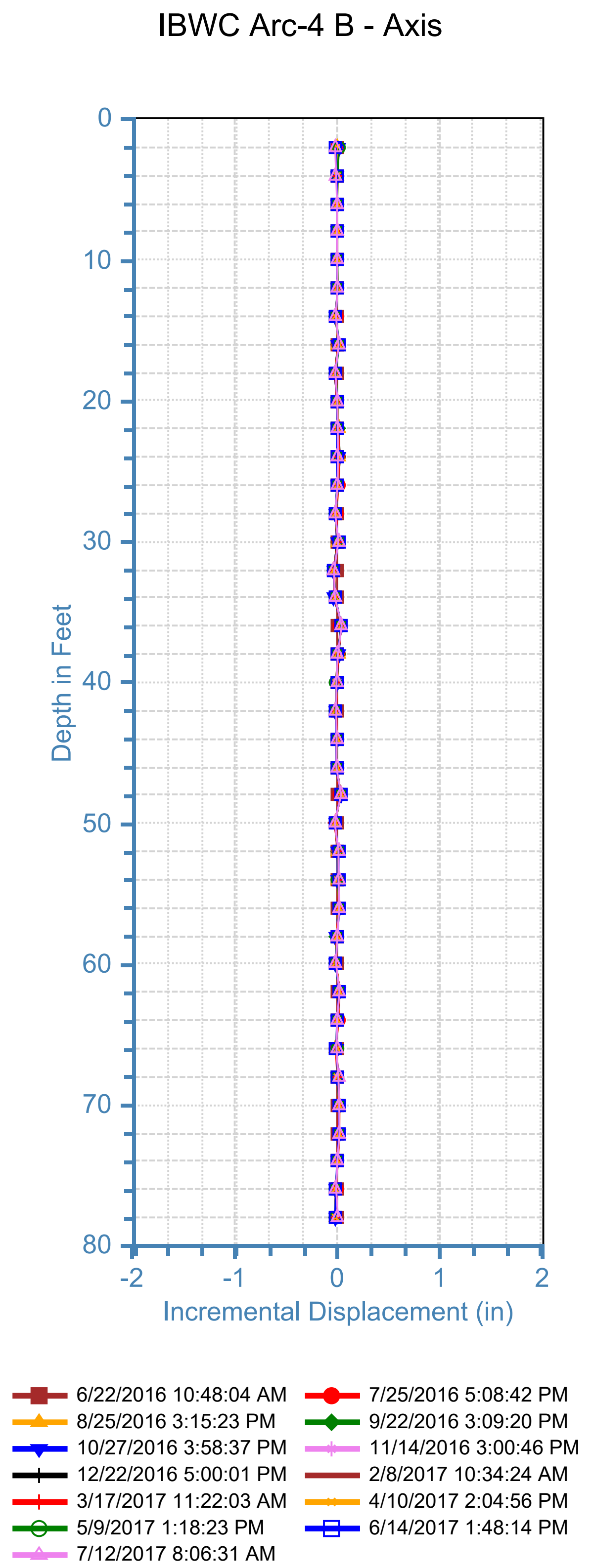
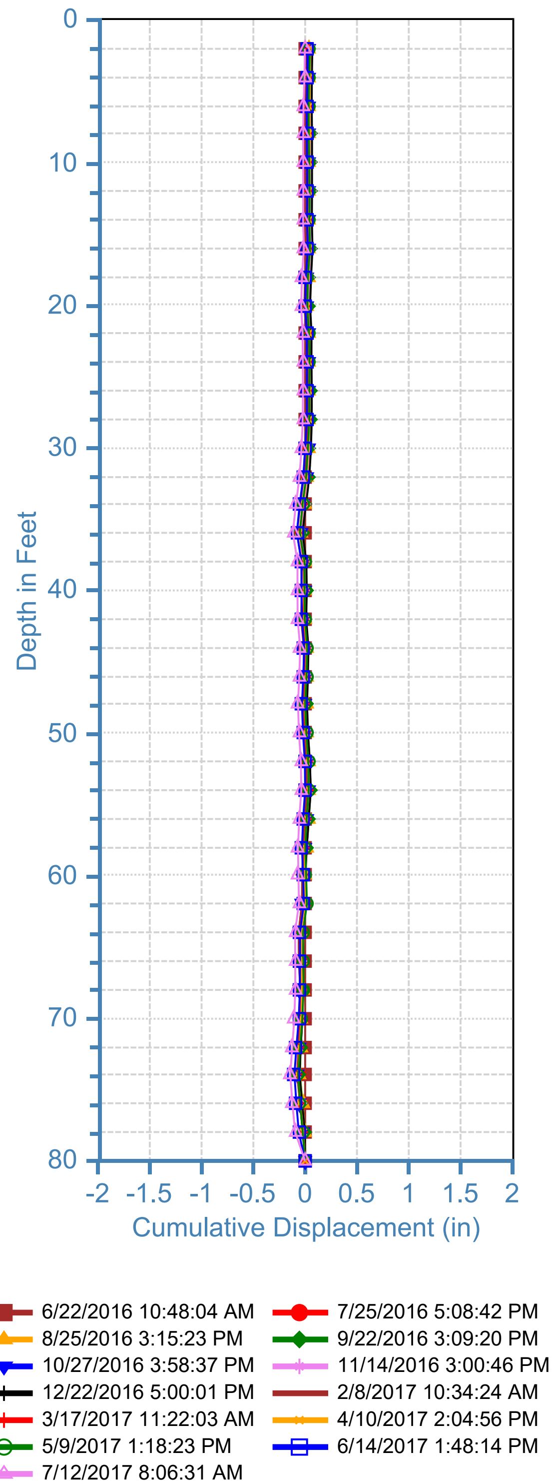




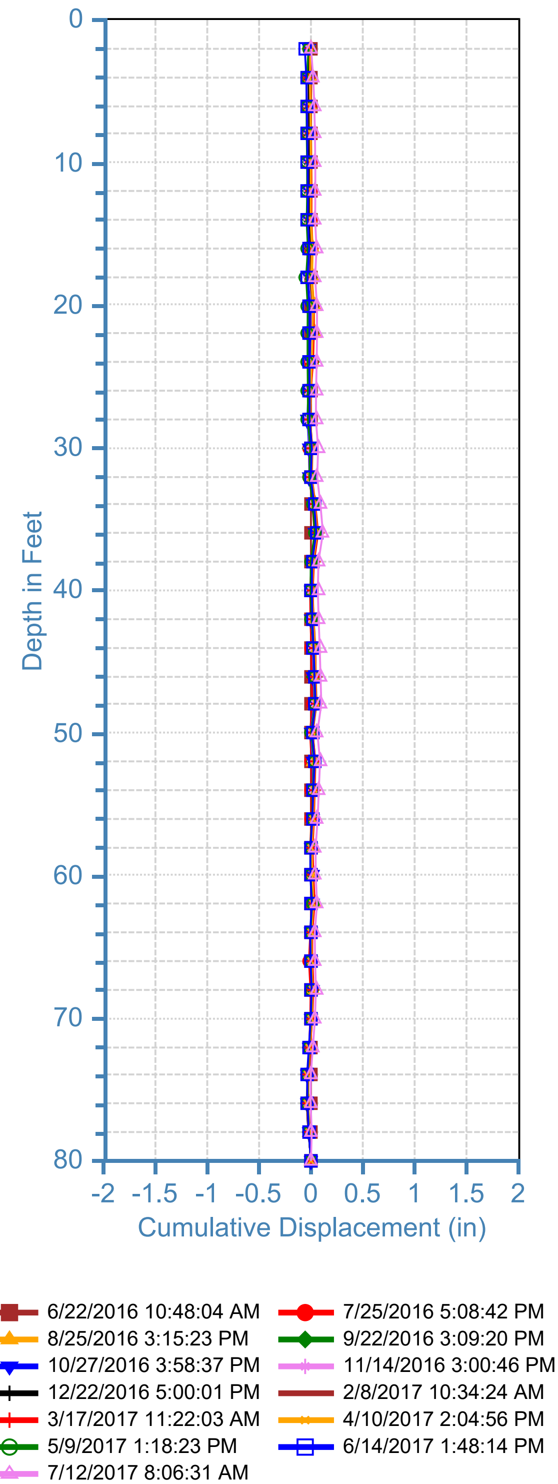
Base reading on 6/22/2016

ARCADIS

IBWC Arc-4 A - Axis



IBWC Arc-4 B - Axis



Base reading on 6/22/2016

ARCADIS

ATTACHMENT B

INCLINOMETER AND LEVEE CRACKING LOCATION MAP

LEGEND:

B-1: 100 FEET BOREHOLE DRILLED AT THE TOP OF THE LEVEE

ARC-1: 98 FEET INCLINOMETER CASING INSTALLED WITHIN BORING B-1

B-2: 80 FEET BOREHOLE DRILLED AT THE TOE OF THE LEVEE

ARC-2: 78 FEET INCLINOMETER CASING INSTALLED WITHIN BORING B-2

B-3: 80 FEET BOREHOLE DRILLED AT THE THE EDGE OF THE RIVERBANK

ARC-3: 78 FEET INCLINOMETER CASING INSTALLED WITHIN BORING B-3

B-4: 80 FEET BOREHOLE DRILLED NEAR THE NORTH ABUTMENT OF THE GATEWAY BRIDGE

ARC-4: 78 FEET INCLINOMETER CASING INSTALLED WITHIN BORING B-4

NOT TO SCALE

IBWC
SUMMARY REPORT OF INCLINOMETER READINGS

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

INCLINOMETER & LEVEE CRACKING LOCATION

ARCADIS

ATTACHMENT
B

ATTACHMENT C

PHOTOS OF SURFACE TENSION CRACKS

Photo 1 - Looking North - Surface Tension Crack of Pin Flags 1B through 4B.

Photo 2 - Looking North - Surface Tension Crack of Pin Flags 5B through 10B.

C-1

Photo 3 - Looking South - Surface Tension Crack of Pin Flags 11B through 20B.

Photo 4 - Looking South - Surface Tension Crack of Pin Flags 21B through 26B.

APPENDIX D

PIT Report

Information To Build On
Engineering • Consulting • Testing

July 13, 2016

Mr. Kirk Lowery P.E., D.GE
Arcadis U.S., Inc.
10352 Plaza Americana Drive
Baton Rouge, Louisiana 70816

Re: Pile Integrity Testing Report
Upper Levee Segment
Brownsville/Ft. Brown Levee
Brownsville, Texas
PSI Project No. 328-1663

Dear Mr. Lowery,

Professional Service Industries, Inc., is pleased to transmit our Pile Integrity Test (PIT) report for the bulkhead timber wall along the east end of the Upper Levee Segment earthen levee in Brownsville, Cameron County, Texas. This report includes the field test results obtained during our site visti on June 15 and 16, 2016 and our evaluation and opinion.

If you have questions pertaining to this report, or if we may be of further service, please contact us at your convenience.

Respectfully submitted,
Professional Service Industries, Inc.

Ayan Mehrotra	Hector J. Lopez, P.E.
Project Manager	Branch Manager
Geotechnical Services	

Reviewed by: Nicholas Roth, Principal Consultant

PILE INTEGRITY TEST SERVICES REPORT

for the

Upper Levee Segment
Brownsville/Ft. Brown Levee
Brownsville, Texas

Prepared for:

Arcadis U.S., Inc.
10352 Plaza Americana Drive
Baton Rouge, Louisiana 70816

Prepared by:

Professional Service Industries, Inc.
724 Central Avenue
Jefferson, Louisiana 70121
Phone (504) 733-9411
Fax (504) 733-9415

PSI Work Order Number: 03281663-1

July 13, 2016

psi Information To Build On
Engineering • Consulting • Testing
724 Central Avenue • Jefferson, LA 70121
Phone 504/733-9411 • Fax 504/733-9415

Ayan Mehrotra

Ayan Mehrotra
Project Manager
Geotechnical Services

Hector J. Lopez, P.E.
Branch Manager

Reviewed by: Nicholas Roth,
Principal Consultant

Information To Build On

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PROJECT UNDERSTANDING

PSI understands that the project consists of monitoring/remedial work along a portion of the alignment of an existing earthen levee. An existing timber bulkhead wall traverses the eastern edge of the existing earthen levee. The timber bulkhead wall generally consists of a row of timber members installed adjacent to each other, and each timber member consists of three (3) individual pieces of timber bolted together in a tongue and groove formation. It is understood that very limited information is available with regard to the penetration depth, and installation procedures utilized for the timber bulkhead wall.

PSI was requested to perform Pile Integrity Testing (PIT) at three (3) locations along the existing timber bulkhead wall. We understand that PIT was requested in an attempt to identify an approximate penetration depth range for the wall. This work was performed in accordance to our Agreement for Subcontract Services and PSI Proposal No. 177516.

SCOPE OF SERVICES

PSI's scope of service included performing Pile Integrity Testing (PIT) at three (3) locations along the wall that were identified in consultation with a Client representative in the field. A description of the three (3) locations is provided below:

- Location 1: Located along the East/West side of the bulkhead approximately 6 feet from the north end of the bulkhead.
- Location 2: Located on the west side of the bulkhead approximately 50 feet from the north end of pile wall.
- Location 3: Located on the west side of the bulkhead approximately 120 feet from the north end of the pile wall (or approximately 15 feet from the south end of the bulkhead).

The scope of service included performing PIT in general accordance with methods described in ASTM D5882-07. PIT was performed in an attempt to identify the approximate penetration depth of the wall. Upon completion of collecting field data, PSI's scope of service included preparation of this report that includes the following items.

- A summary of the project information;
- A summary description of the site and data collected;
- PIT logs compiled by the PIT-W software package; and
- Evaluations of the collected data with general commentary;
- PSI's service did not include acceptance or rejection of the wall and is presenting its findings for informational purposes.

PIT BACKGROUND

PIT is a non-destructive method of testing used to assess the continuity of concrete drilled piers, auger-cast-in-place piles, or pre-cast concrete driven pile foundations. This method is based on a generated compression wave traveling through a continuous column of uniformly constructed material in terms of area, stiffness, and modulus. Changes from the theoretical response provide insight into potential variations resulting from the construction process. This method can be used as a screening procedure to assess the continuous nature of the concrete profile after placement of a pile. This procedure is performed with a series of hand held hammers used to generate a compression wave. Typically, an accelerometer or geophone is placed on top of the test pile to measure the response of a hammer impact.

The accelerometer picks up the initial impact and the resulting return responses are recorded in a data acquisition device or field computer. Following the data acquisition, specific software is used to analyze the waveforms generated during the field test procedures. Testing can be performed using hammers of varying mass to produce a compression wave, which travels the length of the foundation member, reflecting back to the top of the pile when it encounters a change in physical characteristics of the pile member. The wave can be reflected prior to arrival at the bottom of the foundation due to change in cross section of the pile or other physical changes.

METHODOLOGY

On June 15, 2016, PSI representative Mr. Matthew Champagne visited the site to perform the Pile Integrity Testing (ASTM D5882-07). Three (3) locations (as described previously) along the wall were selected for testing in the field in consultation with a Client representative.

The PIT computer processing system requires the following input values to provided interpretation of the data collected in the field:

- An expected compression wave speed of the pile material
- Estimated total pile length below the accelerometer
- Cross-section area of the pile.
- Locations of accelerometers in reference to striking surface
- Hammer weights

PIT testing on timber has some limitations since the material properties of timber are not well-defined and tend to vary significantly based on the type of timber, its age, and amount of degradation. Since the wave speed of the compression wave in timber is not well defined, PSI measured the compression wave speed in the field utilizing two (2) accelerometers, and a “Two Velocity Analysis”. This was performed by attaching one (1) accelerometer to the top of the wall while another accelerometer was “side-mounted” on one side of the wall, about 18 to 24 inches below the pile top. The data was subsequently analyzed to evaluate the difference in the arrival time of the reflected compression wave between the two (2) accelerometers. Since the distance between the two (2) accelerometers is known, and the difference in arrival time can be measured, a wave speed was able to be calculated based on the field measurements. The field measurements indicated a wave speed of about 12,600 feet/per second.

Testing was performed by attaching an accelerometer to a generally flat portion of the top of the wall and striking the timber member with a hammer. The accelerometer was used for data collection to detect and measure the reflecting waves caused by the end of the timber and changes in physical properties of the timber. PSI used a 2-pound hammer to generate the compression waves.

PSI utilized the Pulse Echo Method (PEM) to reduce the field data and evaluate the various pile properties. However, it should be noted that the wave speed of timber can vary greatly due to inherent variation in timber properties as well as due to the degradation of the timber. Variations in the wave spend can affect the apparent length of the wall assessed based on the collected PIT data. If the pulse velocity in the timber is as low as 11,600 ft/sec, the length of the piles could be reduced by up to 10%.

FIELD OBSERVATIONS

During field testing of the piles, PSI noted visible features of the wall that may impact the response of the PIT signal and therefore, the results of the testing. Figures 1 and 2 present pictures taken during PSI’s site visit. Some commentary regarding the pictures is presented subsequently.

Figure 1: Picture of pile top near Location 2

Figure 2: Picture of wall top near Location 1

As seen in Figures 1 and 2, the timber bulkhead wall consisted of sets of three (3) timbers installed in one (1) row to form a continuous wall. The following features could have impacted the response of the PIT signal:

- Generally, the exposed portions of the wall displayed moderate to significant degradation of the timber. The piles selected in the field for testing generally correspond to those sections that displayed the least amount of degradation near the pile top. However, it was noted that the amount of degradation varied significantly across the wall alignment.
- The set of three (3) timber members are bolted together. If the timbers members are in contact below grade, it is possible that the PIT wave signal experienced frictional losses not only due to the soil but also due to adjacent timber member.
- While the accelerometers were placed on one (1) individual timber members (out of the group of 3), it is possible that the two (2) adjacent timber members interfered with the input impulse

and the subsequent wave response. It was not practical to determine what the impact of the interference may have been on the recorded response signals.

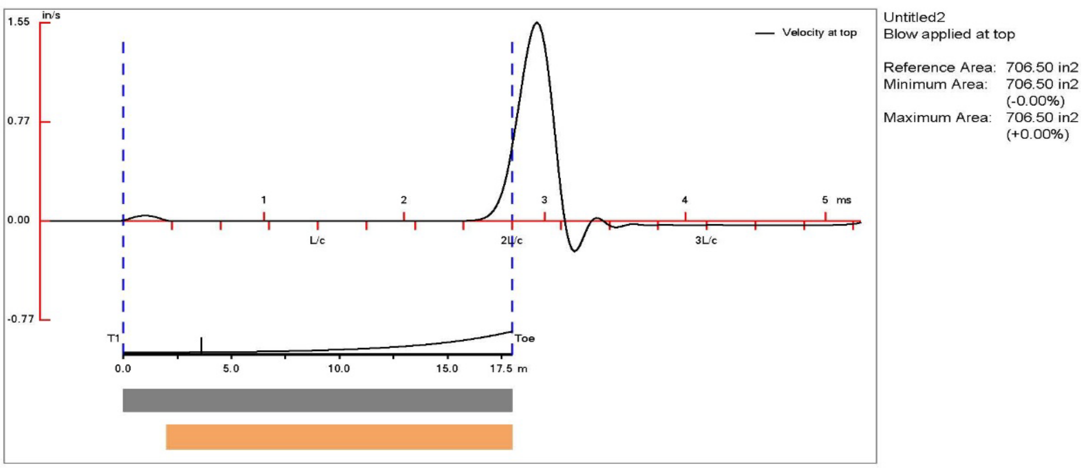
FIELD DATA AND COMMENTARY

For each wall location tested, PSI reviewed the signal traces from each of the hammer strikes using a wave speed of 12,600 ft/s. The wave speed was selected based on the evaluation of the signals from the two (2) accelerometer configuration utilizing the “Two Velocity Analysis” option within the PIT-W software.

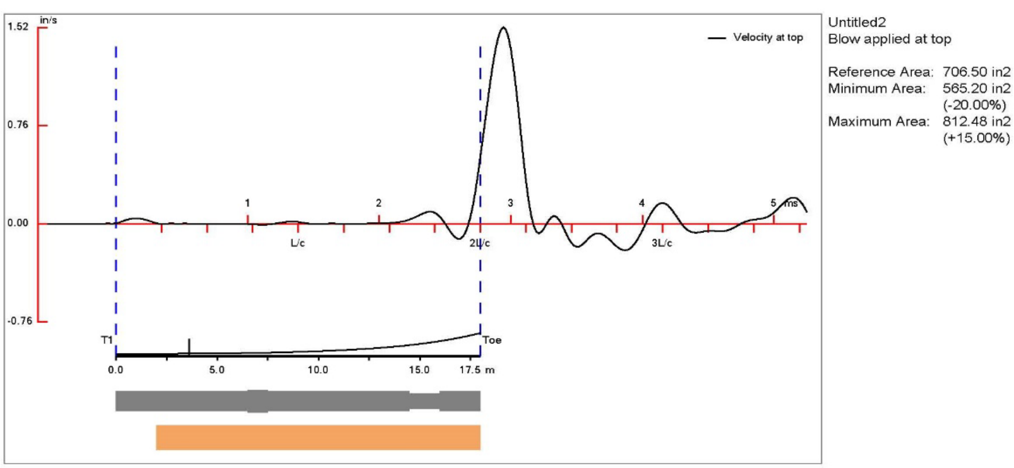
The graphs used to evaluate the piles represent the particle velocity at the striking surface of the piles which are formed by pulse waves reflecting off of various changes in material properties. These pulse waves cause vibrations at the accelerometer anchors and are recorded in the field computer. In addition, PSI has provided commentary and examples regarding the signal changes of each category of piles.

For reference, PSI is providing the following example theoretical images to illustrate typical response curves. The first image shown represents an 18-foot long, 30-inch diameter concrete pier that is consistent with depth. The second image shows the signal response with a 15% increase, or bulge, in the pier section at approximately 7 feet with a 20% reduction, or neck, at approximately 14 feet.

<u>Example Theoretical Pier - Continuous</u>



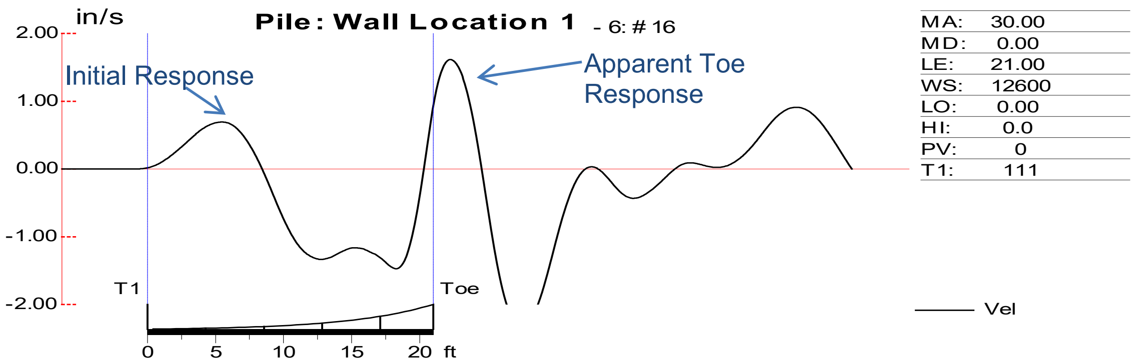
<u>Example Theoretical Pier – Distorted</u>

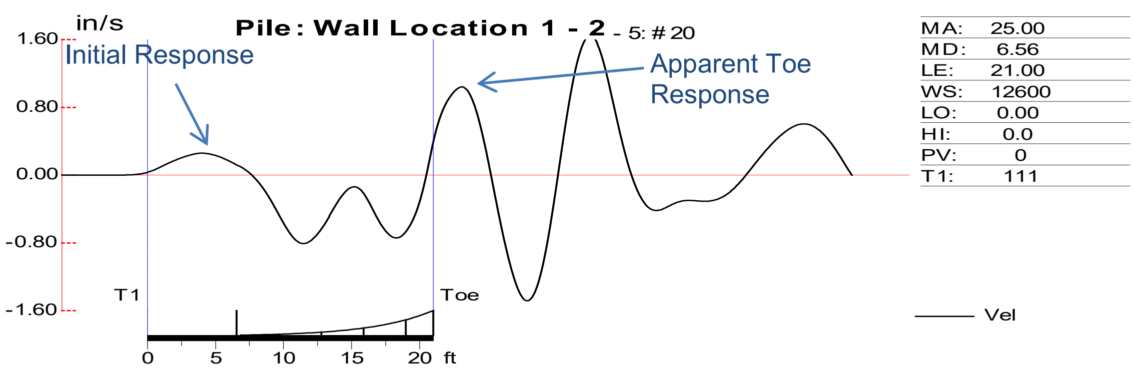


REPRESENTATIVE PIT DATA

After collection of the PIT data in the field, PSI evaluated the resulting graphs using the PIT-W software based on the calculated wave speed. The collected data is divided in terms of the three (3) wall locations tested; Locations 1 thru 3, which were described earlier in this report.

Wall Location 1

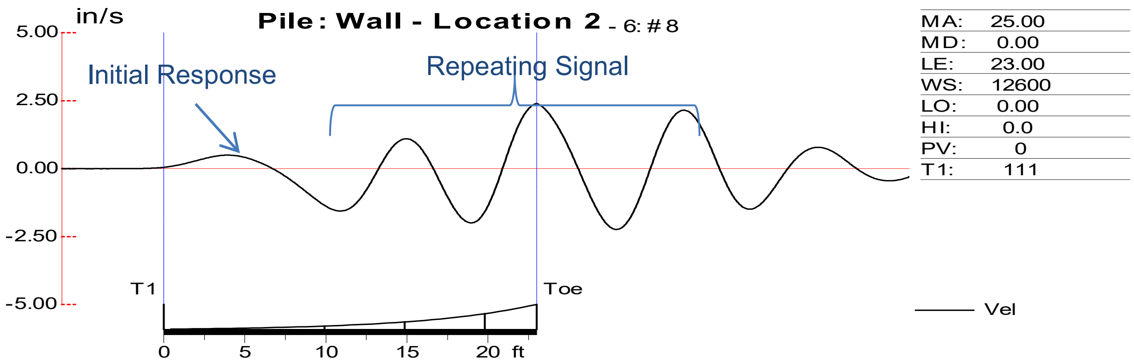




Commentary

The PIT datasets for Location 1 indicate an apparent toe response/wall termination depth at a depth of about 20 to 23 feet. However, the second dataset presented above displays a significant impedance change following the toe response. This could be an indication of one of the following: 1.) One of the timber members, out of the cluster of three (3), is longer than the other two (2); or 2.) There is significant degradation of the timber at a depth of about 20 feet and it is difficult to identify the actual wall termination depth. It is difficult to say with certainty which case is more likely.

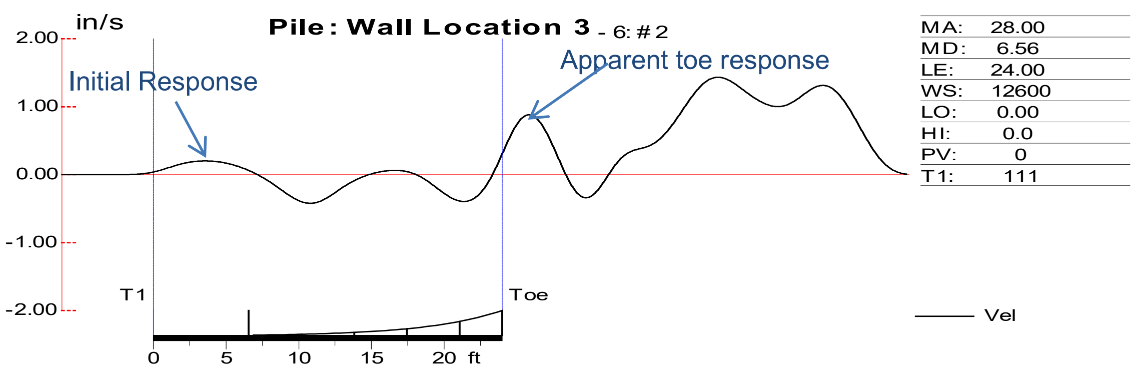
Wall Location 2



Commentary

The PIT datasets for Wall Location 2 generally indicated a repeating signal following the input impulse. This could be due to a defect or severe degradation near the pile top, which reflects the input impulse and causes the repeating signal observed in the dataset. Due to the repeating signal, the data collected at this location is inconclusive.

Wall Location 3



Commentary

The PIT datasets for Wall Location 3 indicate an apparent toe response/wall termination depth of about 24 to 26 feet.

SUMMARY

PSI performed PIT testing at three (3) locations along an existing timber bulkhead wall located at the edge of an existing earthen levee. As requested by the Client, the PIT testing was performed to attempt to identify the wall bottom.

Based on the results of the tests performed in the field, it appears that the wall terminates at a depth of about 18 to 25 feet. However, it should be noted that the apparent toe/wall bottom response noted in the

PIT datasets could also represent the depth at which there is significant degradation of the timber.

It should be noted that PSI is the testing consultant of this project and can only report the information collected in the field. PSI has provided the PIT testing service to help evaluate the approximate wall bottom depth.

LIMITATIONS OF PIT TESTING

PIT testing is a non-destructive testing procedure which is generally performed as a screening criterion to determine if destructive measures are necessary to evaluate the integrity of timber wall members. False positives can be recorded and typically come from changes in cross-sectional area that are not associated with an anomaly, from changes in concrete modulus, (such as the interface between concrete placed from two different trucks), from changes in the stiffness of the soil or rock surrounding the pile, which also dissipate sonic energy, and from testing technique errors such as setting the sensor on weak or powdery concrete.

The information submitted is based on the available information obtained by PSI. The geotechnical engineer warrants that the findings or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed. This report has been prepared for the exclusive use of Arcadis U.S. Inc. for the specific application to the Upper Levee Segment project in Brownville, Texas.

APPENDIX E

Lab Testing Results

Removed to Eliminate Redundancy

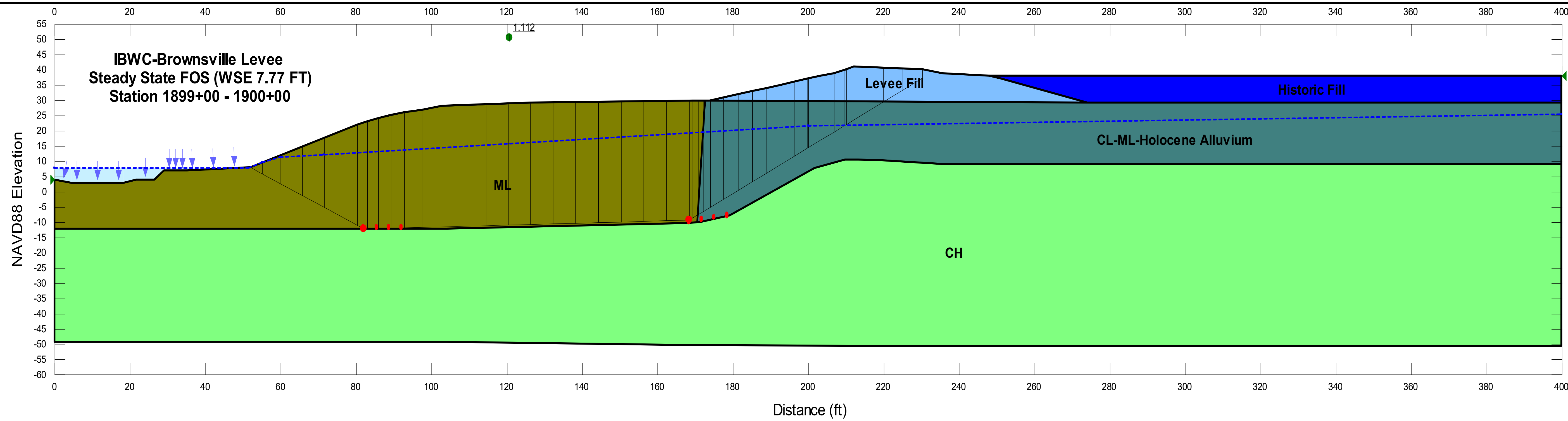
APPENDIX F

Arcadis' 60% Memorandum Figures

Removed to Eliminate
Redundancy

APPENDIX G

Station 1899+15 Slope Stability Analyses



Minimum Factor of Safety (FOS): 1.112

Potential Failure Surface			
Passive Block		Active Block	
Left Block		Right Block	
Projection Angle			
Starting Angle (Degree)	Ending Angle (Degree)	Starting Angle (Degree)	Ending Angle (Degree)
135	146.17	34.26	45

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
ML	119.38	230	0
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1			

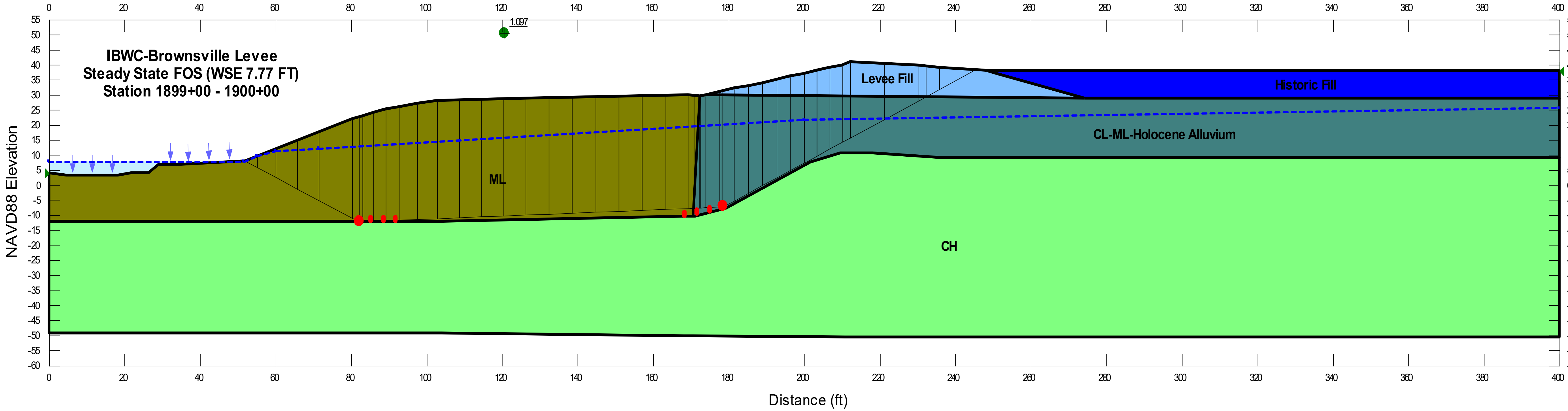
IBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY - NEW MODEL
STEADY STATE SEEPAGE

ARCADIS

APPENDIX



Minimum Factor of Safety (FOS): 1.097

Potential Failure Surface			
Passive Block		Active Block	
Left Block		Right Block	
Projection Angle			
Starting Angle (Degree)	Ending Angle (Degree)	Starting Angle (Degree)	Ending Angle (Degree)
140	146.17	34.26	50

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
ML	119.38	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11

Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1

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GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY - NEW MODEL

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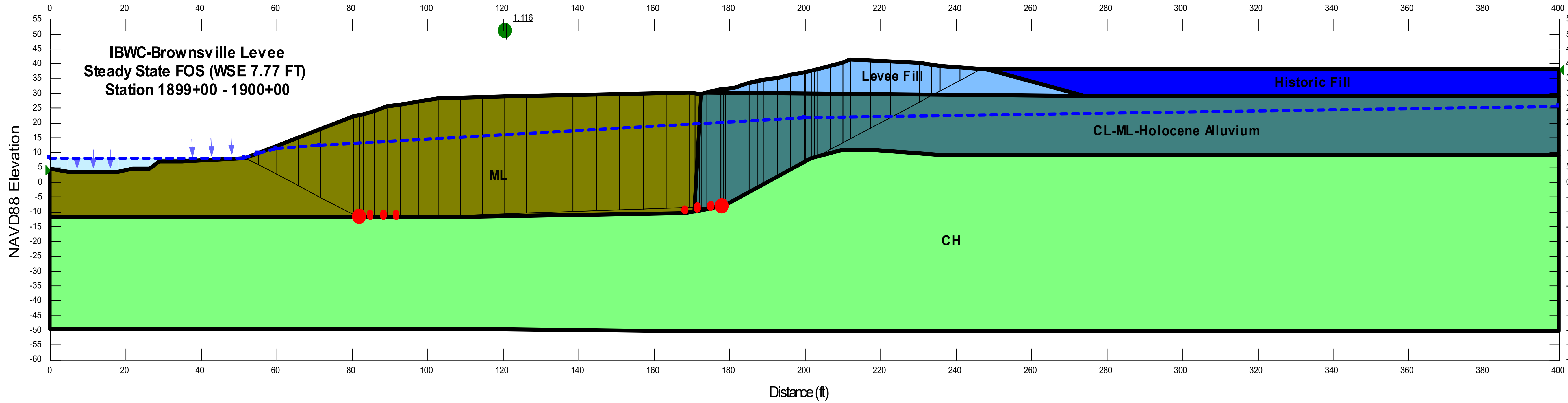


Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
ML	119.38	0	8
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

Minimum Factor of Safety (FOS): 1.116

Potential Failure Surface			
Passive Block		Active Block	
Left Block		Right Block	
Projection Angle			
Starting Angle (Degree)	Ending Angle (Degree)	Starting Angle (Degree)	Ending Angle (Degree)
139	146.17	34.26	49

IBWC
GEOTECHNICAL REPORT

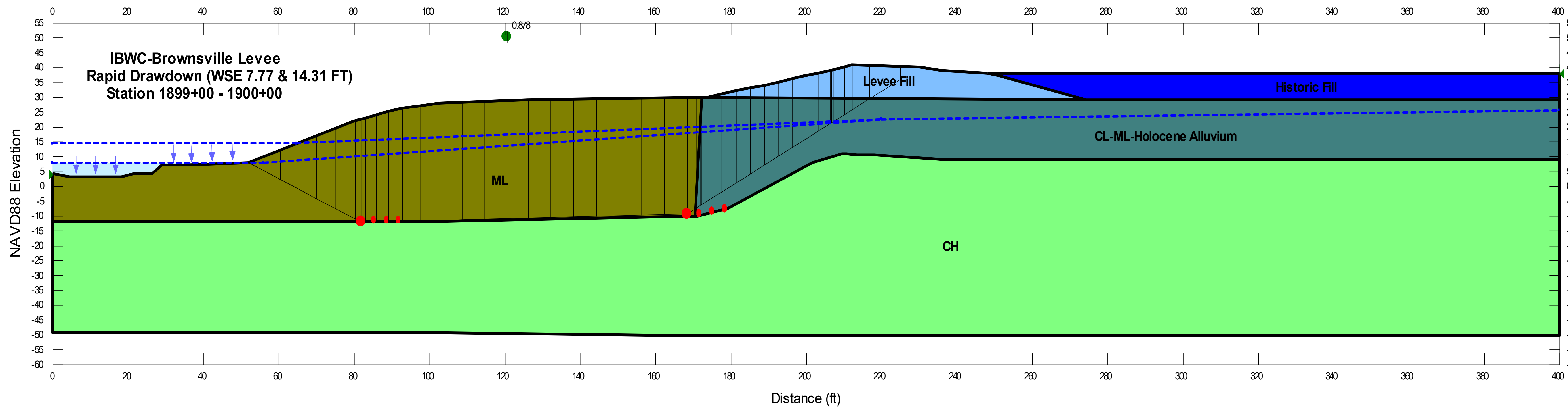
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY - NEW MODEL

STEADY STATE SEEPAGE

ARCADIS

APPENDIX



Minimum Factor of Safety (FOS): 1.078

Potential Failure Surface			
Passive Block		Active Block	
Left Block		Right Block	
Projection Angle			
Starting Angle (Degree)	Ending Angle (Degree)	Starting Angle (Degree)	Ending Angle (Degree)
142	146.17	34.26	52

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
ML	119.38	190	0	0	29
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

IBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY - NEW MODEL

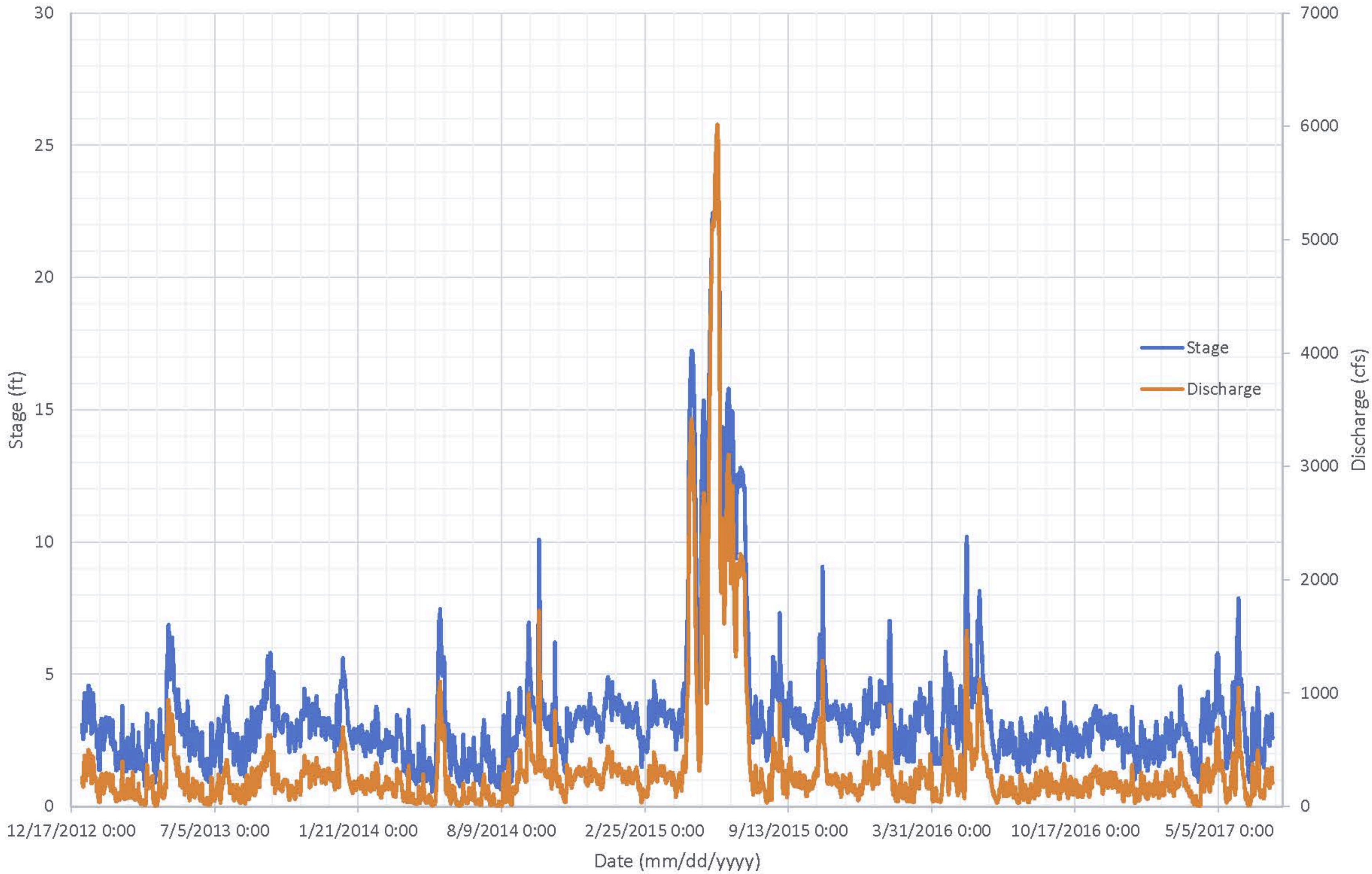
RAPID DRAWDOWN

ARCADIS | **APPENDIX**

APPENDIX H

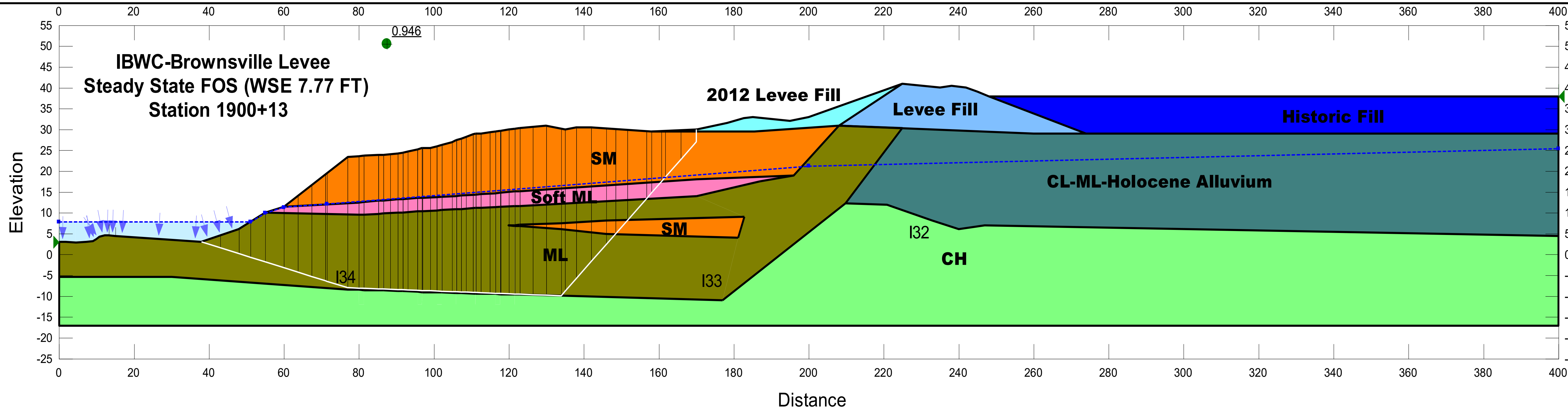
Rio Grande Gauge Data

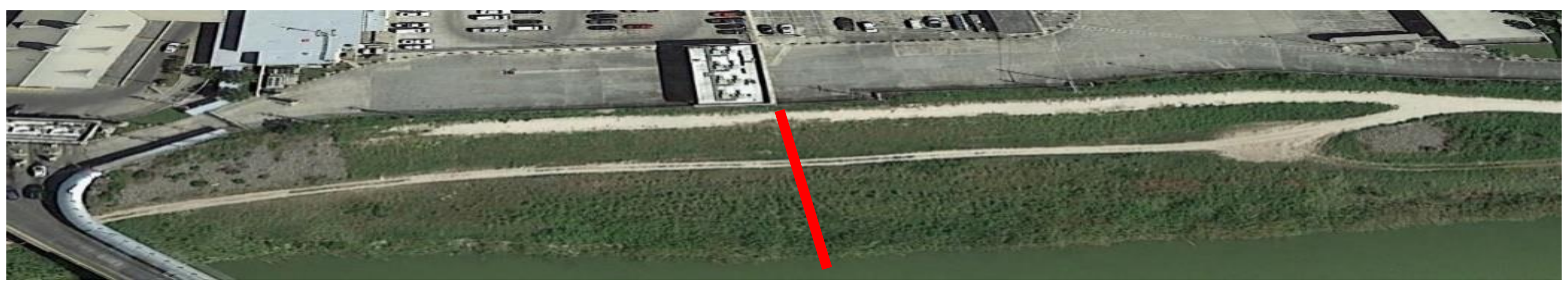
08475000 Water Stage versus Discharge for the Brownsville Gage



APPENDIX I

Slope Stability Analyses at Toe of Levee





Minimum Factor of Safety (FOS): 0.946

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

IBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - EXISTING CONDITIONS RIVERWARD OF LEVEE

STEADY STATE SEEPAGE

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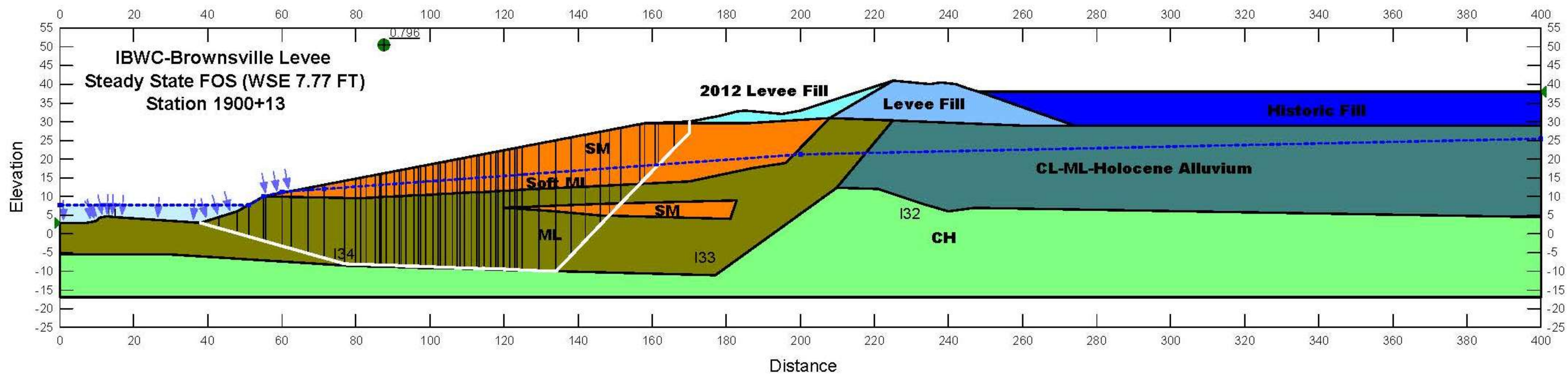
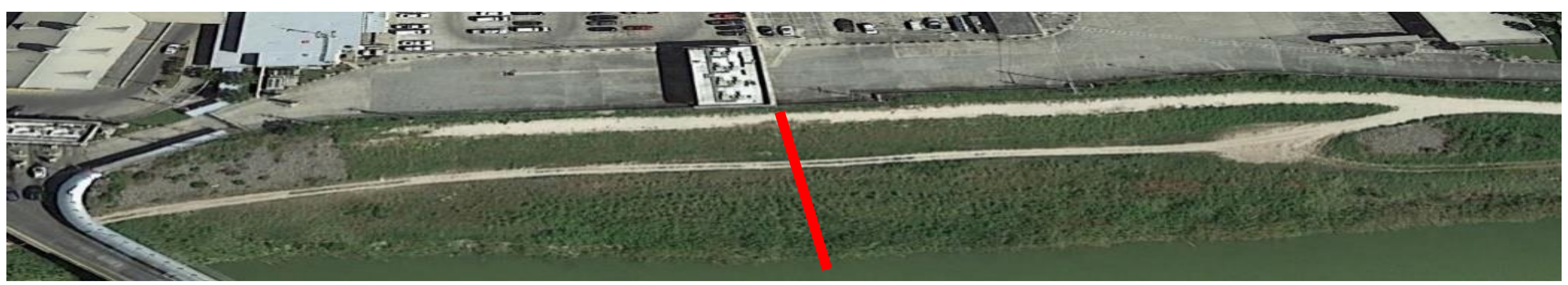
APPENDIX





Minimum Factor of Safety (FOS): 0.796

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

IBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - 5:1 CUT RIVERWARD OF LEVEE

STEADY STATE SEEPAGE

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APPENDIX

APPENDIX J

Deep Soil Mix Analyses

DEEP SOIL MIXING

MIXED SOIL SHEAR WALL DESIGN

For	IBWC - Brownsville Levee Rehab	Job No.	LA003315.0000	Sheet No.	1 of 7
Calc	Mixed Soil Shear Wall Design	Made By	CW	Checked by	KL
		Date	12/1/2016	Date	12/2/2016

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Geometric and Strength Considerations

H = Thickness of Deep Mixed (DM) Zone =	44.8	ft		
B = Width of DM Zone Normal to CL of Levee =	45	ft		
d = diameter of DM column =	72	in. =	6.0	ft
s = center-to-center column spacing =	17	ft		
e = column overlap distance =	12	in. =	1	ft

$A_{columnOL}$ = Area of overlapped column = $(\pi(d/2)^2)*(1-a_e)$ =	26.02	ft^2
A_{PP} = Area of panel in plan = $(Y_{int} - 1)A_{columnOL} + \pi(d/2)^2$ =	236.5	ft^2
A_{PS} = Area of panel in section = HB_{actual} =	2060.8	ft^2
A_{PN} = Area of panel normal to CL of levee = dH =	268.8	ft^2
Y = # of columns to achieve width B = B/(d-e) =	9.00	
Y_{int} = integer # of columns to achieve width B =	9	
$B_{actual} = (Y_{int} - 1)*(d-e) + d$ =	46.00	ft

q_{dm} = specified UCS of deep mixed ground = 100 psi

Column Overlap Calculations

e/d = overlap to diameter ratio =	0.167	(dimensionless)		
α= one-half cord angle = arcos(1-(e/d)) =	0.59	radians		
c = cord length = d(sinα) =	3.32	ft =	39.80	in.
a_e = overlap area ratio = $(2\alpha-\sin 2\alpha)/\pi$ =	0.080	(dimensionless)		
a_s = area replacement ratio = $(\pi d(1-a_e))/(4s\cos\alpha)$ =	0.306	(dimensionless)		
b = effective panel width = $a_s s$ =	5.20	ft		

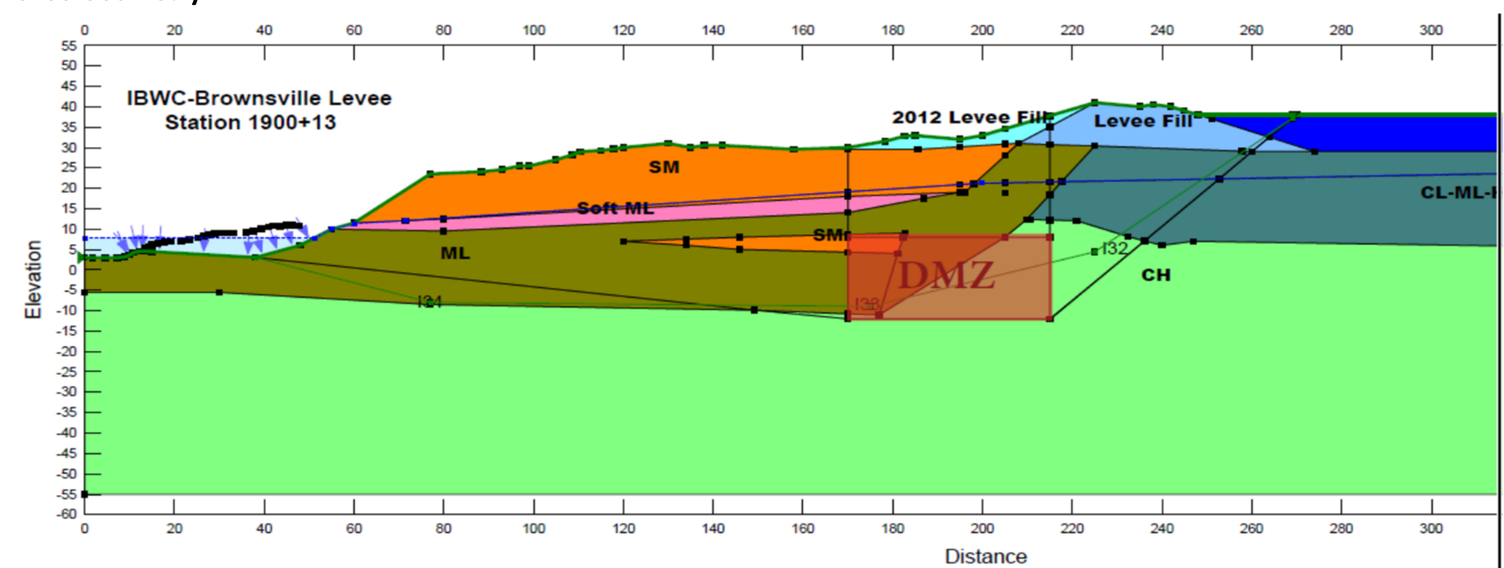
Design Strength of Deep-Mixed Ground

Time between construction and loading:	28 days			
f_c = Ratio of UCS as compared to 28-day UCS =	1	(dimensionless)		
f_d = Design Factor of Safety of Deep Mix Strength =	1.5			
p_{dm} = probability DM strength > specified strength =	80%			
f_v = Values of Variability factor =	0.87	(dimensionless)		
s_{dm} = design DM shear strength = $0.4f_cf_vq_{dm}$ =	34.8	psi =	5011	psf
s_{dmz} = shear strength of DMZ = $a_s s_{dm}$ =	1534.22	psf		

Summary of Design Factors of Safety

Mode of Failure	Abbrev.	Req. FoS	Actual FoS
Sliding	F_s	1.3	1.506
Overturning	F_o	1.4	1.455
Crushing	F_c	1.4	1.69
Vertical Shearing	F_v	1.4	1.96
Extrusion	F_e	1.3	>10
Global Stability - Spencer's Method	F_g	1.4	1.416

Levee Geometry



Design Stratigraphy

Groundwater EL = 20.3 ft (include strata change at groundwater table)

Top EL DMZ = 32.8 ft

Top EL (ft.)	Thickness (ft)	Bottom Depth(ft)	Material	γ_{moist} (pcf)	γ_{Eff} (pcf)	σ'_{Layer} (psf)	σ'_{base} (psf)	Area within Central Block (ft^2)	Weight (lbs/ft)
32.8	2.7	2.7	Levee Fill	127	127	342.9	342.9	137	17399.0
30.1	9.8	12.5	SM Vadose	117	117	1146.6	1489.5	427.8	50052.6
20.3	1	13.5	SM Sat	117	55	54.6	1544.1	97.65	5331.7
19.3	0.3	13.8	Soft ML	126	64	19.08	1563.18	48.42	3079.5
19	17	30.8	ML Sat	119	57	962.2	2525.38	769	43525.4
2	14	44.8	CH	122	60	834.4	3359.78	550.4	32803.8
-12	-							System:	152192.0

Note: Area calculations from GeoStudio model. Area of unsaturated ML (ML Vadose) addedd to SM Vadose for consideration in system weight calculation

For	IBWC - Brownsville Levee Rehab	Job No.	LA003315.0000	Sheet No.	2 of 7
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Sliding and Overturning Calculations

Assumed trial FoS = 1.454763

Active Side Shear Forces

Groundwater EL = 21.5 ft (include strata change at groundwater table)

Material	Top EL (ft)	Thicknes (ft)	Bottom Depth(ft)	OCR (Max=10)	γ_{moist} (pcf)	γ_{Eff} (pcf)	σ'_{Layer} (psf)	σ'_{base} (psf)	c (psf)	c' (psf)	c_m (psf)	c'_m (psf)	ϕ (deg.)	ϕ' (deg.)	ϕ_m (deg.)	ϕ'_m (deg.)	K_o	V_a (lbs/ft)	h_{va} (ft above base)
Levee Fill	37.8	7.1	7.1	7.7	127	127	901.7	901.7	5000	400	3436.98531	274.9588	0	20	0.0	14.0	1.000	24403	45.07
ML Vadose	30.7	9.2	16.3	2.9	119	119	1094.8	1996.5	0	190	0	130.6054	29	0	20.9	0.0	0.941	4781	38.02
ML Sat	21.5	3.02	19.32	2.1	119	56.6	170.932	2167.432	0	190	0	130.6054	29	0	20.9	0.0	1.000	394	31.99
CL Sat	18.48	6.3	25.62	1.7	123	60.6	381.78	2549.212	400	200	274.958825	137.4794	0	14	0.0	9.7	0.909	3182	27.32
CH	12.18	24.18	49.8	1.1	122	59.6	1441.128	3990.34	2320	150	1594.76119	103.1096	0	16	0	11.2	0.824	15333	11.68
	-12	49.8															System:	48092	22.13

Active Side Weight Force

Groundwater EL = 22 ft (include strata change at groundwater table)

Material	Top EL (ft)	Area (ft^2)	Area %	γ_{moist} (pcf)	γ_{Eff} (pcf)	$W'_{stratum}$ (lbs/ft)	c (psf)	c' (psf)	c_m (psf)	c'_m (psf)	ϕ (deg.)	ϕ' (deg.)	ϕ_m (deg.)	ϕ'_m (deg.)	$L_{failure\ plane}$ (ft)	$L_{failure\ plane}$ (%)	Weighted avg ϕ'_m	$H_{active\ face}$ (ft)	Weighted avg c'_m (psf/ft)
Hist. Fill	38	60.46	4.20	127	127	7678.42	400	200	274.9588	137.47941	15	20	10.4	14.0	8.2	11.0	1.2	0	0
Levee Fill	40.5	411.51	28.62	127	127	52261.77	5000	400	3436.985	274.95883	0	20	0.0	14.0	5.2	7.0	0.0	7.1	24402.59572
CL Vadose	30.3	273.92	19.05	123	123	33692.16	400	200	274.9588	137.47941	0	14	0.0	9.7	9.9	13.3	0.0	0	0
CL Sat	22	363.30	25.26	123	60.6	22015.98	400	200	274.9588	137.47941	0	14	0.0	9.7	22.5	30.3	2.9	6.3	866.1202988
ML Vadose	30.7	57.15	3.97	119	119	6800.85	0	190	0	130.60544	29	0	20.9	0.0	0	0.0	0.0	9.2	0
ML Sat	22	3.98	0.28	119	56.6	225.268	0	190	0	130.60544	29	0	20.9	0.0	0	0.0	0.0	3.02	394.4284345
SM Vadose	31	0.00	0.00	117	117	0	0	0	0	0	32	32	23.2	23.2	0	0.0	0.0	0	0
SM Sat	22	0.00	0.00	117	54.6	0	0	0	0	0	32	32	23.2	23.2	0	0.0	0.0	0	0
Soft ML	19	0.00	0.00	126	63.6	0	168	150	115.4827	103.10956	0	0	0.0	0.0	0	0.0	0.0	0	0
CH	12	267.68	18.61	122	59.6	15953.728	2320	150	1594.761	103.10956	0	16	0.0	11.2	28.43	38.3	4.3	24.18	2493.189146
	A_{total} =	1438	ft^2		W_a =	138628.176	lbs/ft							System:	74.23		8.4	49.8	565

Note: Area calculations from GeoStudio model

α_a = measured angle from horizontal of base of active wedge = 42 °

D_a = active wedge driving force, considering no shear resistence = $W_a \tan\alpha_a$ = 124821.4 lbs/ft

U_a = active wedge uplift force = $(U_{Ha}/2)*L_{ua}$ = 39750.75 lbs/ft

where L_{ua} = measured length of active wedge uplift force = 37.75 ft

R_a =active wedge resisting force = $2(W_a - U_a\sin(90-\alpha_a))\tan\phi'_{m\ avg} + 2c'_{m\ avg}H_a\tan(90-\alpha_a)$ = 94647.38 lbs/ft

P_a = active side pressure considering shear resistence = $D_a - R_a$ = 30173.99 lbs/ft

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Sliding and Overturning Calculations (cont.)

Assumed trial FoS = 1.454763

Passive Side Shear Forces

Groundwater EL : 19.1 ft (include strata change at groundwater table)

Material	Top EL (ft)	Thicknes	Bottom	OCR	γ_{moist} (pcf)	γ_{Eff} (pcf)	σ'_{Layer} (psf)	σ'_{base} (psf)	c (psf)	c' (psf)	c_m (psf)	c'_m (psf)	ϕ (deg.)	ϕ' (deg.)	ϕ_m (deg.)	ϕ'_m (deg.)	K_o	V_p (lbs/ft)	h_{vp} (ft above
Levee Fill	30	0.5	0.5	10.0	127	127	63.5	63.5	5000	400	3436.98531	274.9588	0	20	0.0	14.0	1.000	1718	41.67
SM Vadose	29.5	10.4	10.9	5.2	117	117	1216.8	1280.3	0	0	0	0	32	32	23.2	23.2	1.162	3489	35.45
SM Sat	19.1	1.1	12	3.0	117	54.6	60.06	1340.36	0	0	0	0	32	32	23.2	23.2	0.928	575	30.55
Soft ML	18	4	16	2.5	126	63.6	254.4	1594.76	168	150	115.482707	103.1096	0	0	0.0	0.0	1.000	412	27.99
ML Sat	14	5.4	21.4	2.0	119	56.6	305.64	1900.4	0	190	0	130.6054	29	0	20.9	0.0	1.000	705	23.29
SM Sat	8.6	4.3	25.7	1.6	117	54.6	234.78	2135.18	0	0	0	0	32	32	23.24514	23.2	0.736	2742	18.43
ML Sat	4.3	15.1	40.8	1.2	119	56.6	854.66	2989.84	0	190	0	130.6054	29	0	20.85836	0.0	1.000	1972	8.66
CH	-10.8	1.2	42	1.0	122	59.6	71.52	3061.36	2320	150	1594.76119	103.1096	0	16	0	11.2	0.812	705	0.60
	-12	42															System:	12319	18.67

Passive Side Weight Force

Groundwater EL = 11.25 ft (include strata change at groundwater table)

Material	Top EL (ft)	Area (ft^2)	Area %	γ_{moist} (pcf)	γ_{Eff} (pcf)	$W'_{stratum}$ (lbs/ft)	c (psf)	c' (psf)	c_m (psf)	c'_m (psf)	ϕ (deg.)	ϕ' (deg.)	ϕ_m (deg.)	ϕ'_m (deg.)	$L_{failure\ plane}$ (ft)	$L_{failure\ plane}$ (%)	Weighted avg ϕ'_m	$H_{passive\ face}$ (ft)	Weighted avg c'_m (psf/ft)
Levee Fill	30	3.00	0.08	127	127	381	5000	400	3436.985	274.95883	0	20	0.0	14.0	0	0.0	0.0	0.5	1718.492656
SM Vadose	29.5	1253.01	32.27	117	117	146602.17	0	0	0	0	32	32	23.2	23.2	0	0.0	0.0	10.4	0
SM Sat	11.25	59.96	1.54	117	54.6	3273.816	0	0	0	0	32	32	23.2	23.2	0	0.0	0.0	1.1	0
Soft ML	11.25	365.00	9.40	126	63.6	23214	168	150	115.4827	103.10956	0	0	0.0	0.0	0	0.0	0.0	4	412.4382375
ML Sat	10	2048.22	52.75	119	56.6	115929.252	0	190	0	130.60544	29	0	20.9	0.0	113.3	85.3	0.0	5.4	705.2693861
SM Sat	9	125.50	3.23	117	54.6	6852.3	0	0	0	0	32	32	23.2	23.2	0	0.0	0.0	4.3	0
ML Sat	10	0.00	0.00	119	56.6	0	0	190	0	130.60544	29	0	20.9	0.0	0	0.0	0.0	15.1	1972.142172
CH	-10.8	28.50	0.73	122	59.6	1698.6	2320	150	1594.761	103.10956	0	16	0.0	11.2	19.5	14.7	1.6	1.2	123.7314713
A_{total} =		3883.19	ft^2		W_p =	297951.138	lbs/ft							System:	132.8		1.6	42	117

Note: Area calculations from GeoStudio model

α_p = measured angle from horizontal of base of passive wedge = 6.5 °

D_p = passive wedge driving force, considering no shear resistence = $W_p \tan\alpha_p$ = 33947.24 lbs/ft

U_p = passive wedge uplift force = $(U_{Hp}/2)^*L_{up}$ = 111917.5 lbs/ft

where L_{up} = measured length of active wedge uplift force = 132 ft

R_p = passive wedge resisting force = $2(W_p - U_p\cos\alpha_p)\tan\phi'_{m\ avg} + 2c'_{m\ avg}H_p\tan(90-\alpha_p)$ = 21631.64 lbs/ft

P_p = passive side pressure considering shear resistence = $D_p - R_p$ = 12315.6 lbs/ft

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Sliding and Overturning Calculations (cont.)

N = Resultant vertical force = W + V_a - V_p = 187965.3 lbs/ft

U = uplift force = $H_{H20}*\gamma_{H20}*B_{actual}$ = 92713.92 lbs/ft

N' = Effective resultant vertical force = N - U = 95251.35 lbs/ft

x_W = location of DMZ weight force = 23.45 ft

h_p = height of action of passive force = 15.45 ft

h_a = height of action of active force = 18.30 ft

Note: See page 6 for x_w calculation; page 7 for h_a and h_p calculations

x_N = location of resultant N = B_{actual} - ($P_p h_p$ + Wx_w + $V_a B_{act}$ - $P_a h_a$) / N = 17.17 ft

x_U = location of uplift force = x center of hydrostatic gravity in central block = 23.28 ft

$x_{N'}$ = location of effective vertical force = (Nx_N - Ux_U)/N' = 11.22 ft

T = shear force along base = $2x_{N'}c'_m$ + N'tanϕ'_m = 21088.57 lbs/ft

Where:

Base Strata	c (psf)	c' (psf)	c_m (psf)	c'_m (psf)	ϕ (deg.)	ϕ' (deg.)	ϕ_m (deg.)	ϕ'_m (deg.)
CH	2320	150	1594.761	103.1096	0	16	0.0	11.2

ΣP_H = Sum of horizontal Forces = P_p + T - P_a = 3230 lbs/ft

Check: If ΣP_H > 0 then trial FOS < $F_{s\ Actual}$ TRUE? TRUE

Use Goal Seek to determine FOS such that ΣP_H = 0

$FOS_{\Sigma Ph=0}$ = 1.506

From page 1:

a_s = area replacement ratio = (πd(1-a_e))/(4scosα) = 0.306159 (dimensionless)

b = effective panel width = a_ss = 5.20 ft

s = center-to-center column spacing = 17 ft

B_{actual} = (Y_{int} - 1)*(d-e) + d = 46.00 ft

B_{actual}/3 = 15.33 ft

B_{actual}/2 = 23.00 ft

q_{toe} = (N'/b)*[(2s/3$x_{N'}$)-((s-b)/B_{actual})] for $x_{N'} \le B_{actual}/3$

q_{toe} = (N'/B_{actual})*[(3s/b)*(1 - (2$x_{N'}$/B_{actual})) + 1] for $B_{actual}/3 \le x_{N'} \le B_{actual}/2$

q_{toe} = 13793.17 psf

Bearing Capacity Analysis (Hansen Method)

Bottom Elevation of DSM = -12 ft

Average Ground Elevation = 32.8 ft

Trial FoS = 1.4547633

γ_{above} = weighted average of soil unit weight above DMZ = 75.0 pcf

γ_{below} = unit weight of soil below DMZ = 59.6 pcf

D = depth of embedment to base of DMZ = 44.8 ft

e_B = eccentricity normal to CL of levee = M_B/Q = (B_{actual}/2) - $x_{N'}$ = 11.8 ft

L' = Effective panel length = B_{actual} - 2e_B = 22.4 ft

From Tables 4-4 and 4-5 EM 1110-1-1905:

N_c =Hansen cohesion factor = 8.88

N_γ = Hansen soil wedge weight factor = 0.53

N_q =Hansen surcharge factor = 2.76

ζ_{sc} = Hansen shape with eccentricity cohesion factor = 1.07

0.2b/L' for ϕ'_m = 0 ; 1 + (b/L')(N_q/N_c) for ϕ'_m > 0

$\zeta_{s\gamma}$ = Hansen shape with eccentricity soil wedge factor = 0.907

1 for ϕ'_m = 0 ; 1 - (0.4)(b/L') for ϕ'_m > 0

ζ_{sq} = Hansen shape with eccentricity surcharge factor = 1.05

1 for ϕ'_m = 0 ; 1 + (b/L')tanϕ'_m for ϕ'_m > 0

k = depth reference factor = 0.974

D/B_{actual} for D/B_{actual} ≤ 1 ; $\tan^{-1}$(D/B_{actual}) for D/B_{actual} > 1

ζ_{dc} = Hansen depth cohesion factor = 1.39

0.4k for ϕ'_m = 0 ; 1 + 0.4k for ϕ'_m > 0

$\zeta_{d\gamma}$ = Hansen depth soil wedge factor = 1

for all conditions

ζ_{dq} = Hansen depth surcharge factor = 1.25

1 for ϕ'_m = 0 ; 1+2tanϕ*(1-sinϕ)^{2}k for ϕ'_m > 0

q_u = q_{all} = ultimate bearing capacity = $c'_m N_c \zeta_{sc} \zeta_{dc}$ + 0.5L'$\gamma_{below} N_\gamma \zeta_{s\gamma} \zeta_{d\gamma}$ + D$\gamma_{above} N_q \zeta_{sq} \zeta_{dq}$ =

q_u = q_{all} = ultimate bearing capacity = 13793 psf

Check: If q_{toe} > q_{all} then trial FOS > $F_{O\ Actual}$ TRUE? TRUE

Use Goal Seek to determine FOS such that q_{toe} - q_{all} = 0

$FOS_{q\ toe\ -\ q\ all\ =\ 0}$ = 1.455

Note: Bearing capacity calculations do not consider slope, base tilt, or inclined load. Assumes DMZ is below groundwater level and assumes linear relation between bearing capacity correction factors listed in EM-1110-1-1905 Table 4-4

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Crushing of the Deep Mixed Ground

Trial FOS =	1.45476327	
ϕ'_m =	11.2	°
K_o = at-rest earth pressure = $(K_{o\,a} + K_{o\,p})/2$ =	0.818	psf
σ'_v = effective vertical stress at base of DMZ = $(\sigma'_{v\,a} + \sigma'_{v\,p})/2$ =	3526	psf
σ'_h = effective horizontal stress at base of DMZ = $K_o\sigma'_v$ =	2884	psf
s_{dm} = design DM shear strength = $0.4f_cf_vq_{dm}$ =	5011	psf
$q_{toe\ eq2} = (2s_{dm}/F_c) + \sigma_h$ =	9773	psf

Check: if $q_{toe} > q_{toe\ eq2}$ then trial FOS < $F_{c\ Actual}$ TRUE? TRUE

Use Goal Seek to determine FOS such that $q_{toe} - q_{toe\ eq2} = 0$

$FOS_{q\ toe\ -\ q\ toe\ eq2\ =\ 0}$ = 1.69

Shearing on Vertical Planes

$\tau_v = (V_p/H) + (N/H)(1 - (3x_N/2B_{actual}))^2$ for $x_N \le B_{actual}/3$

$(V_p/H) + (3N/4H)(1 - (2x_N/B_{actual}))$ for $B_{actual}/3 \le x_N \le B_{actual}/2$

B_{actual} =	46.00	ft		
$B_{actual}/3$ =	15.33	ft		
$B_{actual}/2$ =	23.00	ft		
x_N =	17.17	ft		
H =	44.8	ft		
V_p =	12319	lbs/ft =	12.3	kips/ft
N =	187965.3	lbs/ft =	188.0	kips/ft
τ_v =	1.072	ksf =	1072	psf

F_v = FoS against vertical shear = $s_{dmz,v}/\tau_v$

where $s_{dmz,v} = (c/s)s_{dm}$ =	977.6629	psf
c = cord length = d(sinα) =	3.32	ft
s = center-to-center column spacing =	17	ft
$s_{dmz,v}/FOS_{trial}$ =	672.0426	psf

Check: if $s_{dmz,v}/FoS_{trial} < \tau_v$, then trial FOS < $F_{v\ Actual}$ TRUE? TRUE

Use Goal Seek to determine FOS such that $\tau_v - (s_{dmz,v}/FOS) = 0$

$FOS_{\tau v\ -\ (sdmz,\ v/FOS)\ =\ 0}$ = 1.96

Extrusion of Soft Ground Between Shear Walls

F_e = Factor of Safety against extrusion = $(2c_e(2+B_{actual}((1/(s-b))+(1/H_e)))/(\sigma_{vf} - \sigma_{vp})$

c_e = average value of total stress cohesion intercept =	CH	2320	psf
H_e = thickness of layer of soft clay to be analyzed for extrusion =		24.18	ft
σ_{vf} = average total vertical stress at base on active side of DMZ =		6080.74	psf
σ_{vp} = average total vertical stress at base on passive side of DMZ =		5002	psf
F_e = Factor of Safety against extrusion =		34	
F_e = Factor of Safety against extrusion for infinite spacing =		17	

Note: Internal formulas for σ_{vf} and σ_{vp} are correct only for the given stratigraphy. If stratigraphy is altered, the internal formulas must be hand-altered, as well.

For	IBWC - Brownsville Levee Rehab	Job No.	LA003315.0000	Sheet No.	6 of 7
Calc	Mixed Soil Shear Wall Design	Made By	CW	Checked by	KL
		Date	12/1/2016	Date	12/2/2016

Key	
Input	
Calculation	
Linked Cell	
Design FOS	
Check Cell	

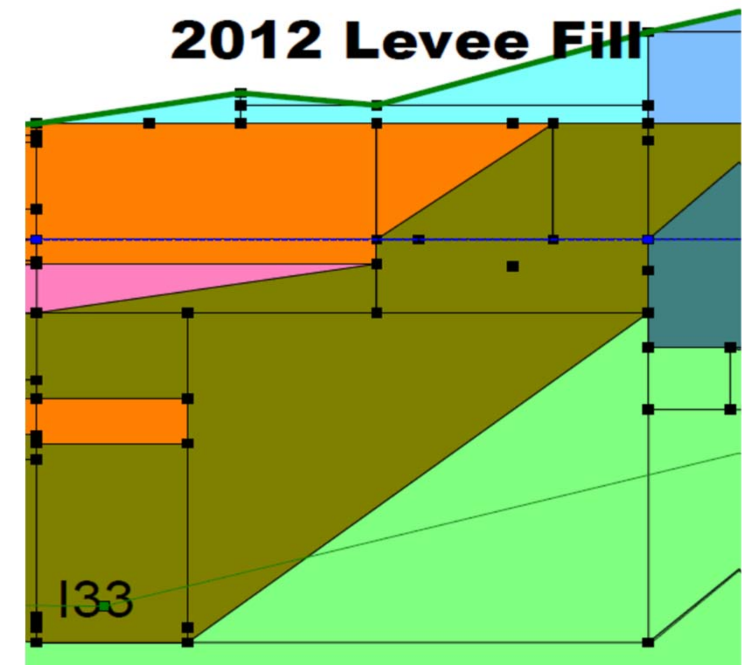
DMZ Center of Gravity Calculations

Groundwater EL = 21 ft (include strata change at groundwater table) $X_{point\ O}$ = 170

i	Material	Top EL (ft)	Area (ft^2)	Area %	γ_{moist} (pcf)	γ_{Eff} (pcf)	W'$_i$ (lbs/ft)	x $_{i1}$	x $_{i2}$	Shape	$x_{COG\ i}$	x $_i$	$W_i x_i$
1	Levee Fill	33	18.75	0.92	127	127	2381.25	170	185	Left Tri	10.005	10.005	23824.406
2	Levee Fill	33	5.00	0.25	127	127	635	185	195	Right Tri	3.33	18.33	11639.55
3	Levee Fill	32	45.00	2.21	127	127	5715	185	215	Rectangle	15	30	171450
4	Levee Fill	37.8	58.00	2.84	127	127	7366	195	215	Left Tri	13.34	38.34	282412.44
5	SM Vadose	30.5	237.45	11.64	117	117	27781.65	170	195	Rectangle	12.5	12.5	347270.63
6	SM Vadose	30.5	61.70	3.03	117	117	7218.9	195	208	Right Tri	4.329	29.329	211723.12
7	SM Sat	21	49.99	2.45	117	54.6	2729.454	170	195	Rectangle	12.5	12.5	34118.175
8	SM Sat	8	41.07	2.01	117	54.6	2242.422	170	181.1	Rectangle	5.55	5.55	12445.442
9	Soft ML	19	50.00	2.45	126	63.6	3180	170	195	Right Tri	8.325	8.325	26473.5
10	ML Vadose	30.5	61.75	3.03	119	119	7348.25	195	208	Left Tri	8.671	33.671	247422.93
11	ML Vadose	30.5	66.50	3.26	119	119	7913.5	208	215	Rectangle	3.5	41.5	328410.25
12	ML Sat	19	50.00	2.45	119	56.6	2830	170	195	Left Tri	16.675	16.675	47190.25
13	ML Sat	21	120.00	5.88	119	56.6	6792	195	215	Rectangle	10	35	237720
14	ML Sat	15	77.70	3.81	119	56.6	4397.82	170	181.1	Rectangle	5.55	5.55	24407.901
15	ML Sat	15	457.65	22.44	119	56.6	25902.99	181.1	215	Right Tri	11.2887	22.3887	579934.27
16	ML Sat	4.3	180.90	8.87	119	56.6	10238.94	170	181.1	Rectangle	5.55	5.55	56826.117
17	CH	15	457.65	22.44	122	59.6	27275.94	181.1	215	Left Tri	22.6113	33.7113	919507.4
		A_{total} =	2039.11	ft^2		W_{system} =	151949.1	lbs/ft				$\Sigma W_i x_i$ =	3562776.4

$x_w = \Sigma w_i x_i / W_{system}$ = 23.44717

Generalized DMZ Geometry for Center of Gravity Calculations



For	IBWC - Brownsville Levee Rehab	Job No.	LA003315.0000	Sheet No.	7 of 7
Calc	Mixed Soil Shear Wall Design	Made By	CW	Checked by	KL
		Date	12/1/2016	Date	12/2/2016

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Key	
Input	
Calculation	
Linked Cell	
Design FOS	
Check Cell	

Active Side Vertical Stress Distribution

Groundwater 21.5 (include strata change at groundwater table)

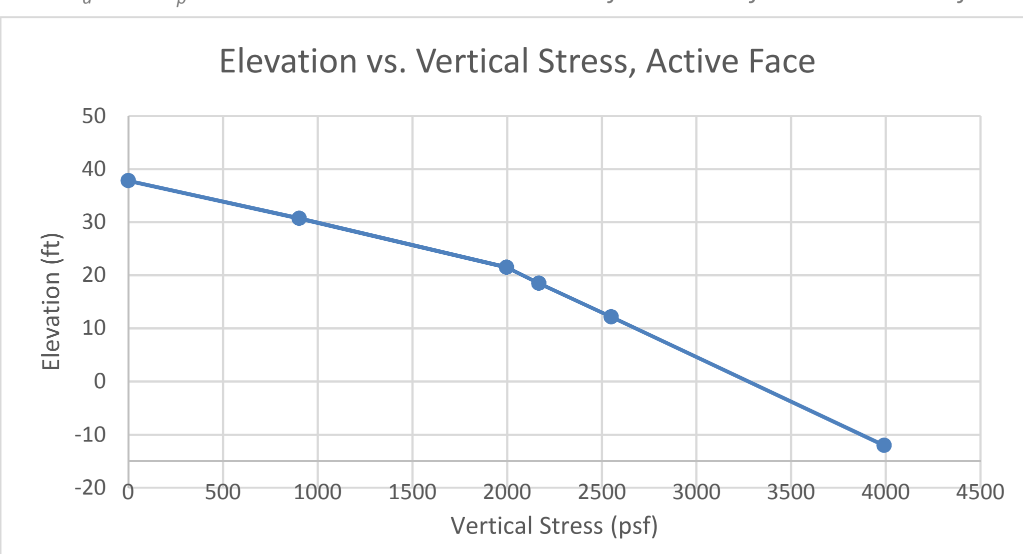
	Top EL (ft)	Thicknes (ft)	Bottom Depth(ft)	γ_{moist} (pcf)	γ_{Eff} (pcf)	σ'_{Layer} (psf)	σ'_{base} (psf)	AUC ob (lbs/ft)	AUC I (lbs/ft)	y COG ob (ft)	y COG i (ft)	AUC t (psf)	% area (%/100)	y i (ft)	Wt avg y i gam (ft)
Levee Fill	37.8	7.1	7.1	127	127	901.7	901.7	0	3201	0.00	45.07	3201	0.03	45.0667	1.235728
ML Vadose	30.7	9.2	16.3	119	119	1094.8	1996.5	8295.64	5036	38.10	36.57	13332	0.11	37.5208	4.284844
ML Sat	21.5	3.02	19.32	119	56.6	170.932	2167.432	6029.43	258	31.99	31.49	6288	0.05	31.9693	1.721834
CL Sat	18.48	6.3	25.62	123	60.6	381.78	2549.212	13654.82	1203	27.33	26.28	14857	0.13	27.2450	3.467429
CH	12.18	24.18	49.8	122	59.6	1441.128	3990.34	61639.95	17423	12.09	8.06	79063	0.68	11.2019	7.586529
	-12	49.8										116741			18.29636

Passive Vertical Stress Distribuiton (include strata change at groundwater table)

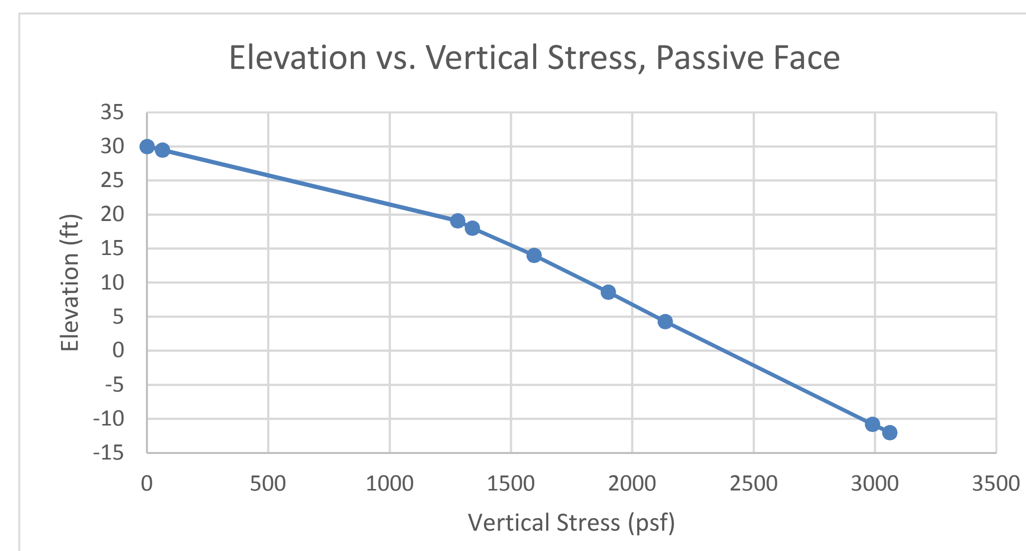
Groundwater EL : 19.1 ft

Material	Top EL (ft)	Thicknes (ft)	Bottom Depth(ft)	γ_{moist} (pcf)	γ_{Eff} (pcf)	σ'_{Layer} (psf)	σ'_{base} (psf)	AUC ob (lbs/ft)	AUC i (lbs/ft)	y COG ob (ft)	y COG i (ft)	AUC t (lbs/ft)	% area (%/100)	y i (ft)	Wt avg y i gam (ft)
Levee Fill	30	0.5	0.5	127	127	63.5	63.5	0	16	0.00	41.67	15.9	0.00	41.6665	0.008849
SM Vadose	29.5	10.4	10.9	117	117	1216.8	1280.3	660.4	6327	36.30	34.56	6987.8	0.09	34.7273	3.24623
SM Sat	19.1	1.1	12	117	54.6	60.06	1340.36	1408.33	33	30.55	30.37	1441.4	0.02	30.5458	0.588972
Soft ML	18	4	16	126	63.6	254.4	1594.76	5361.44	509	28.00	27.33	5870.2	0.08	27.9421	2.194243
ML Sat	14	5.4	21.4	119	56.6	305.64	1900.4	8611.704	825	23.30	22.40	9436.9	0.13	23.2211	2.931461
SM Sat	8.6	4.3	25.7	117	54.6	234.78	2135.18	8171.72	505	18.45	17.73	8676.5	0.12	18.4082	2.136614
ML Sat	4.3	15.1	40.8	119	56.6	854.66	2989.84	32241.22	6453	8.75	6.23	38693.9	0.52	8.3295	4.311515
CH	-10.8	1.2	42	122	59.6	71.52	3061.36	3587.808	43	0.60	0.40	3630.7	0.05	0.5976	0.029027
	-12	42										74753.3			15.44691

Note: h_a and h_p were assessed as vertical distance from base of DMZ to center of vertical stress distribution of active and passive faces, respectively, considering appropriate (effective vs. total) overburden stresses.

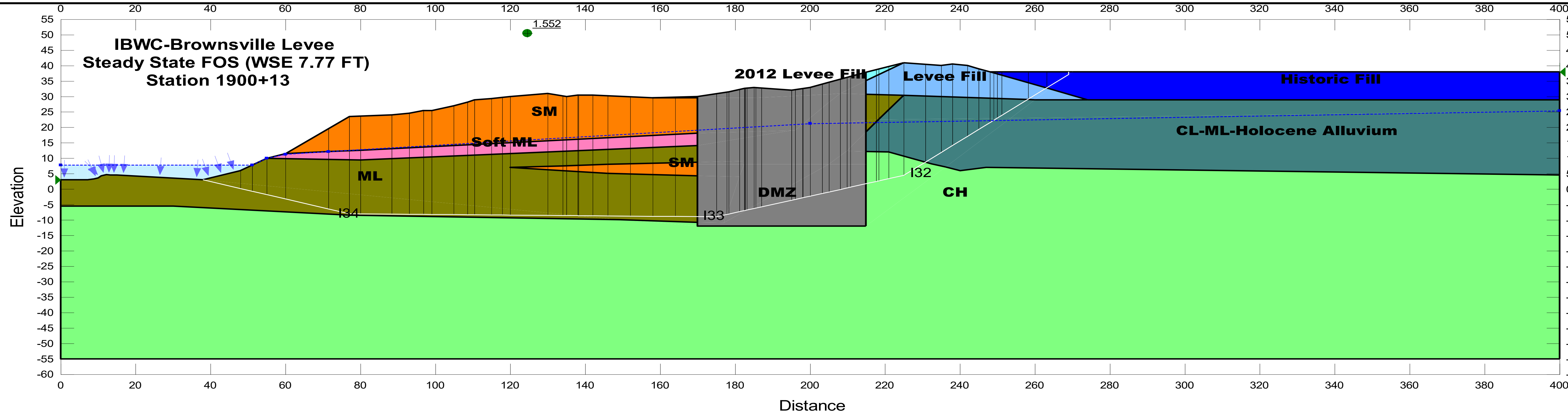


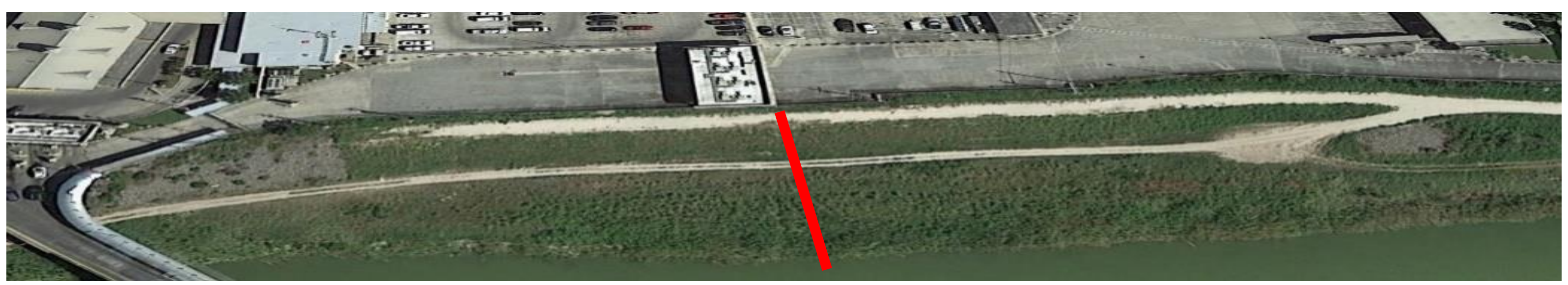
Nomenclature
AUC = Area Under Curve
ob = due to overburden
i = over depth interval
COG = center of gravity
t = total
gam = considering unit weight



SLOPE STABILITY

DEEP SOIL MIXING





Minimum Factor of Safety (FOS): 1.552

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

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REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - DEEP SOIL MIXING ASSESSMENT

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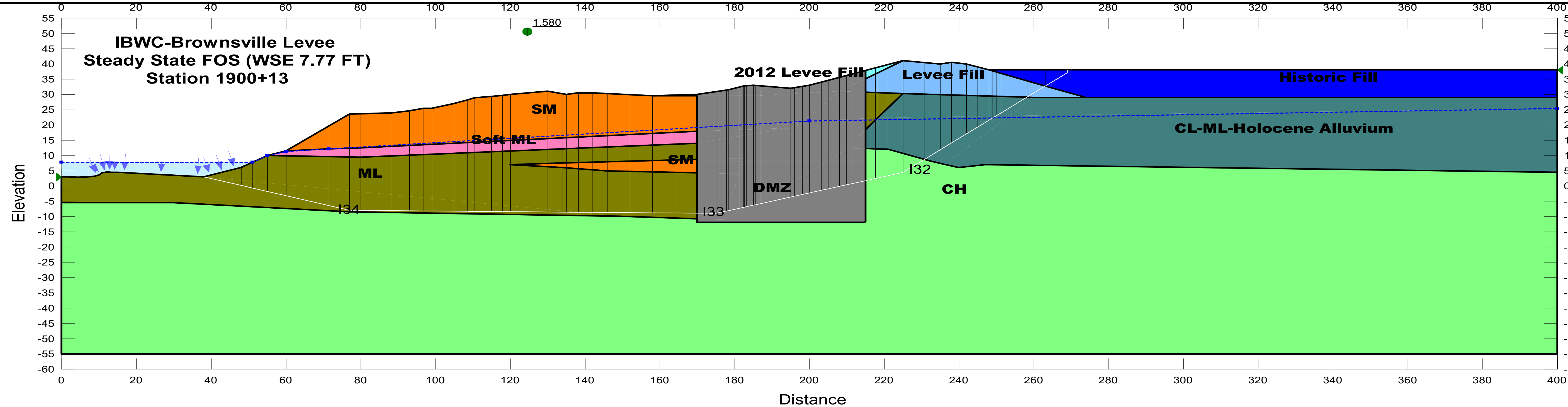

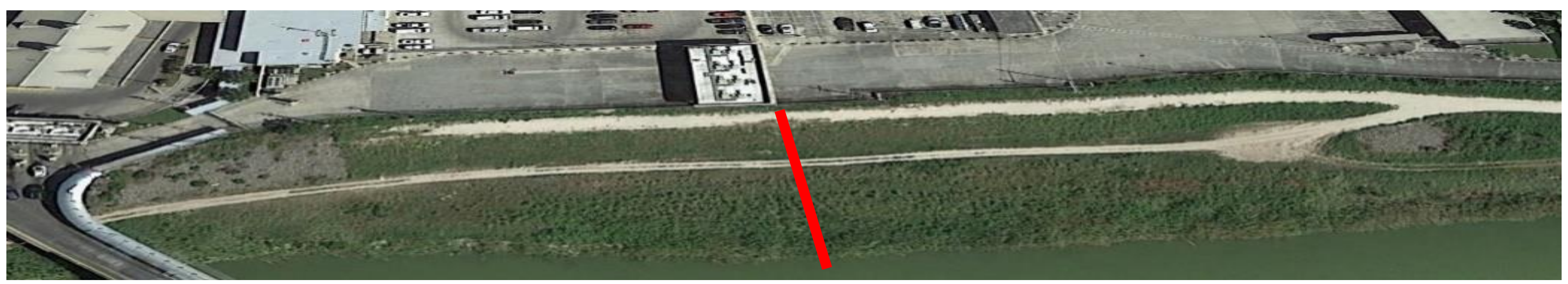

Minimum Factor of Safety (FOS): 1.580

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
DMZ	120	1536.8	0

Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1

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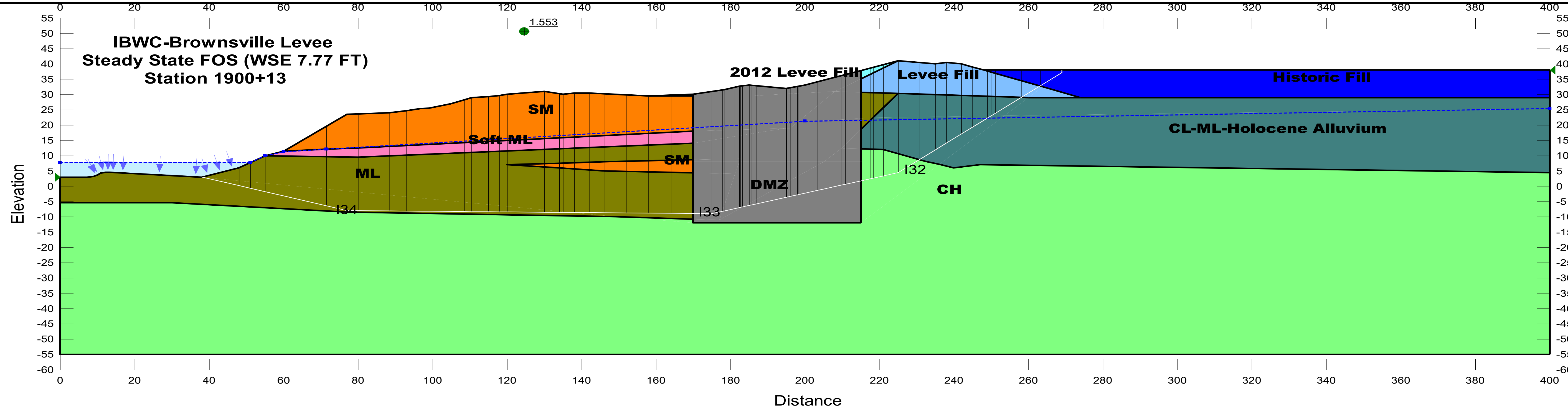

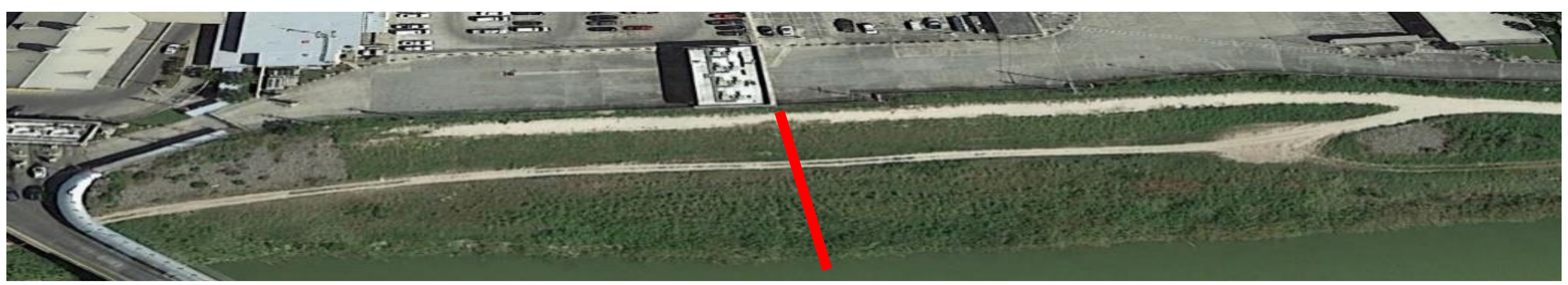

Minimum Factor of Safety (FOS): 1.553

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

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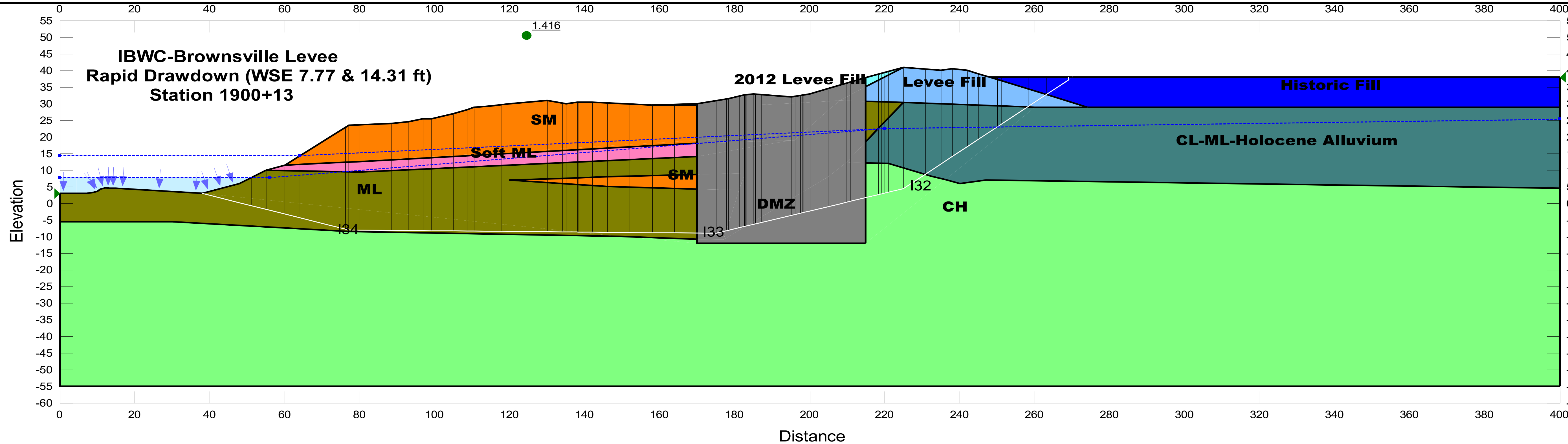

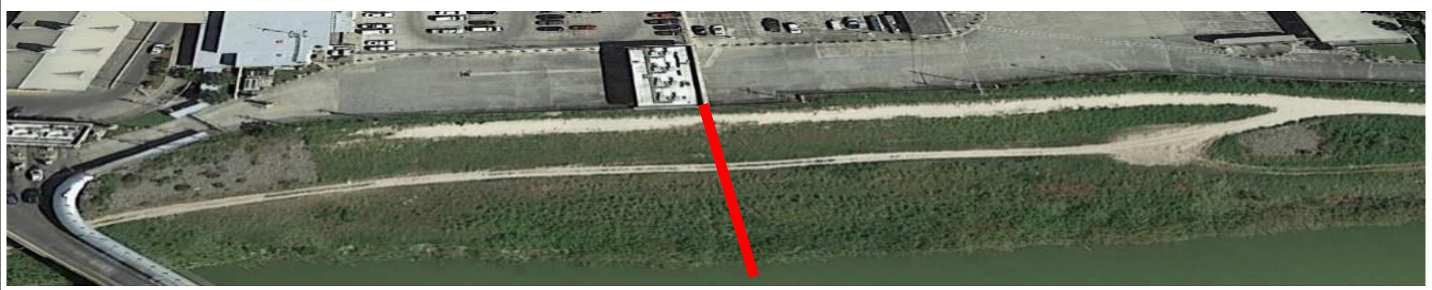

Minimum Factor of Safety (FOS): 1.416

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
DMZ	120	1536.8	0	5011	0

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

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SLOPE STABILITY MODEL - DEEP SOIL MIXING ASSESSMENT

RAPID DRAWDOWN

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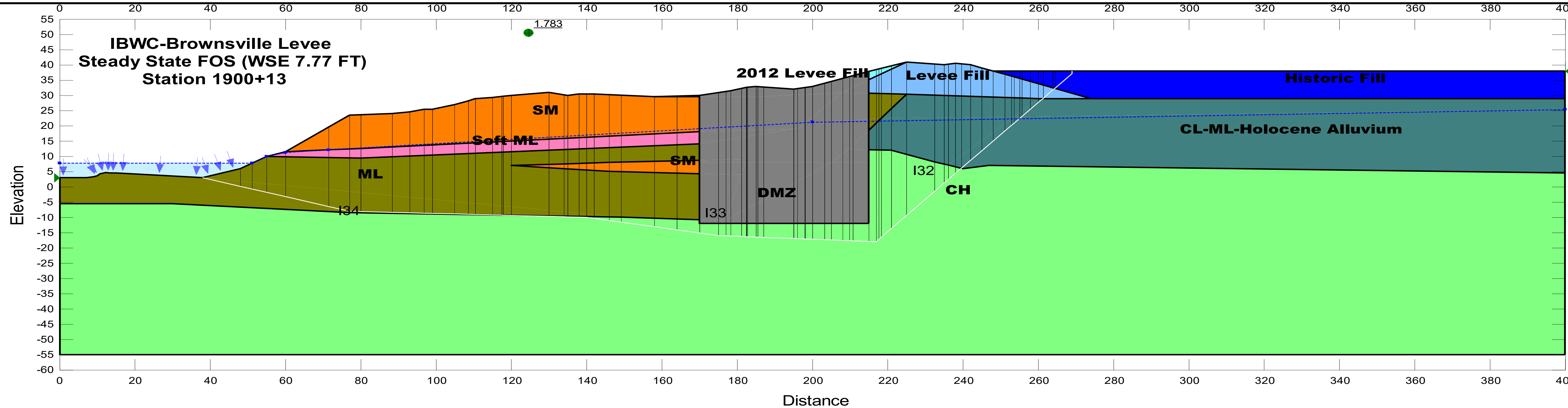

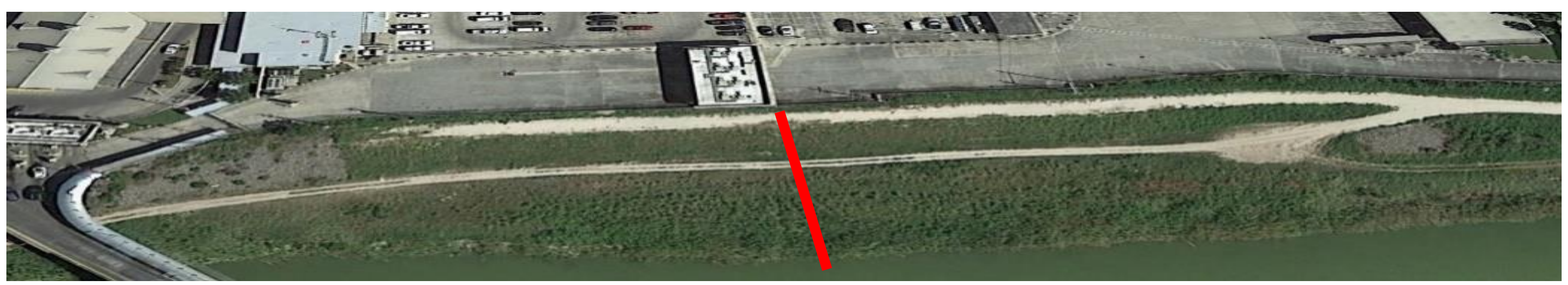

Minimum Factor of Safety (FOS): 1.783

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

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REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

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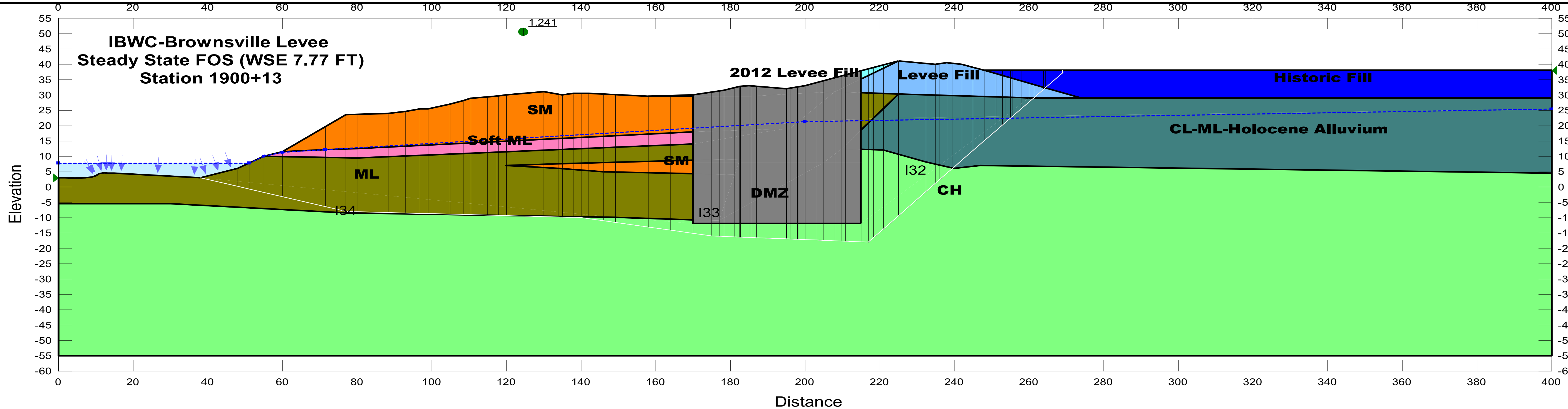

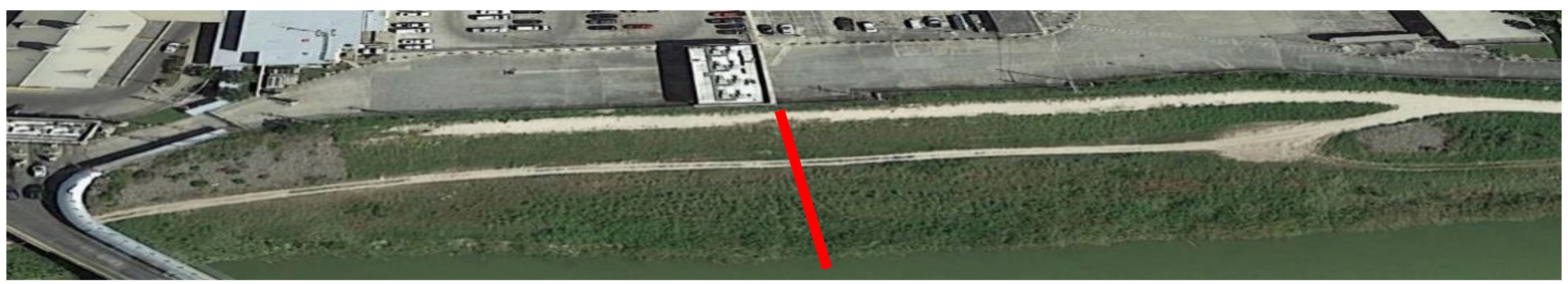

Minimum Factor of Safety (FOS): 1.351

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
DMZ	120	1536.8	0

Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1

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REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

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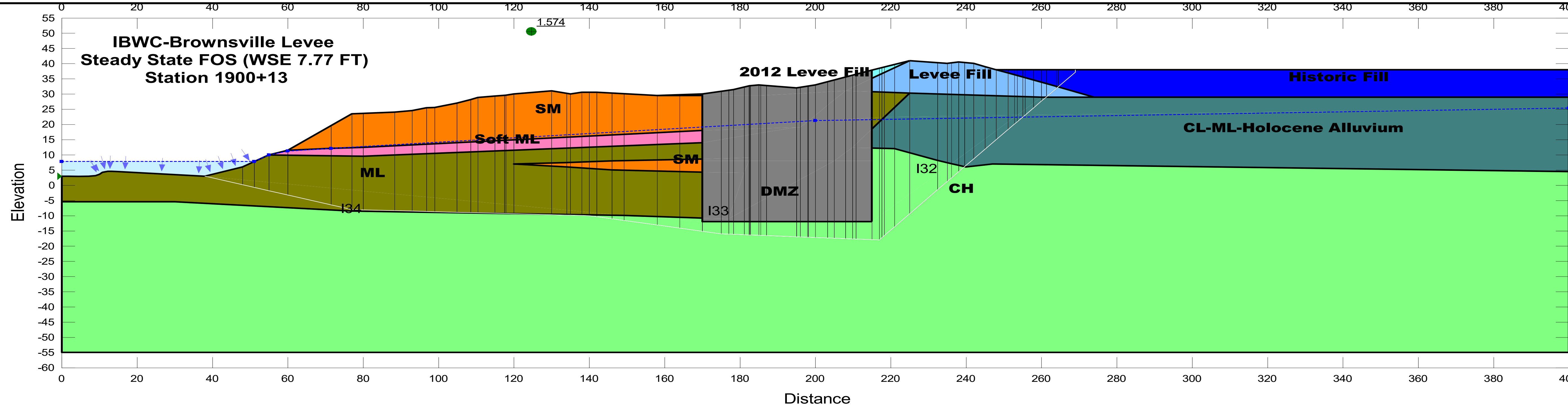

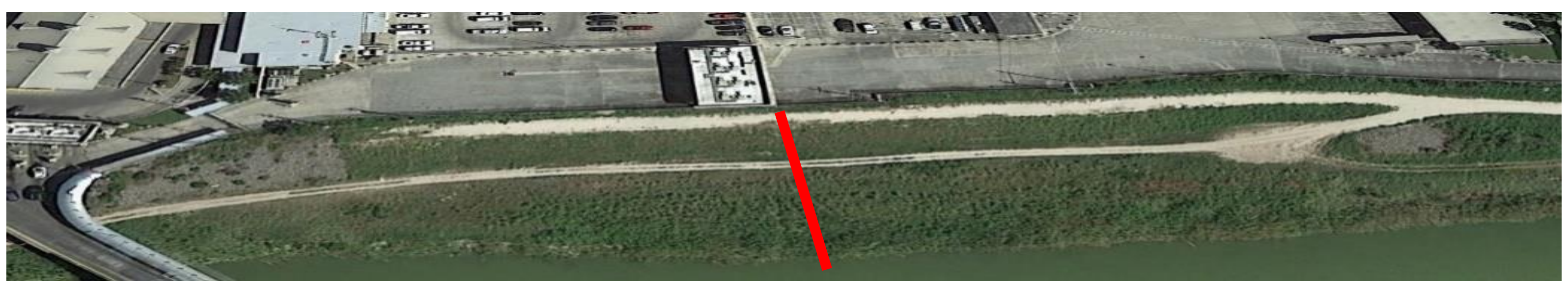

Minimum Factor of Safety (FOS): 1.574

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0
Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1			

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REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
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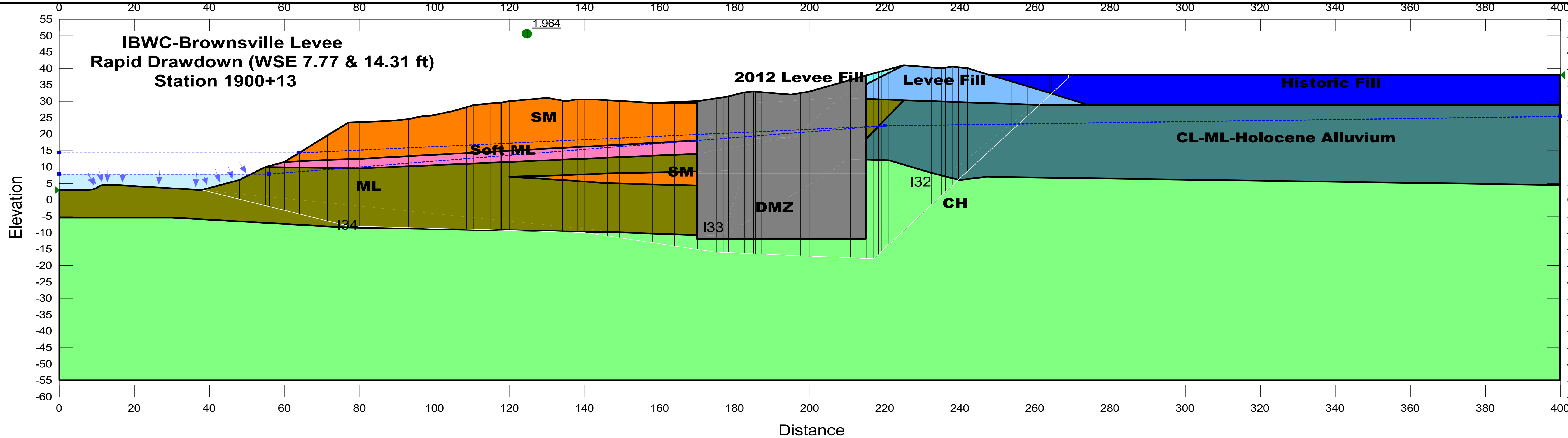

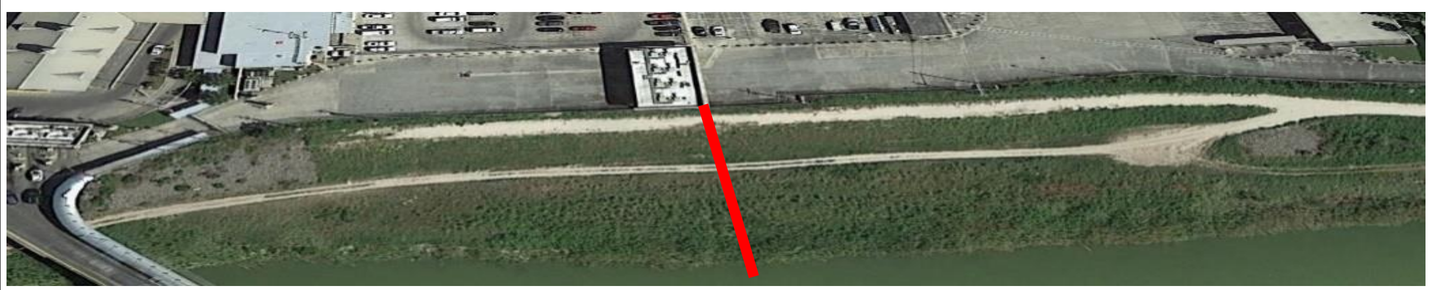

Minimum Factor of Safety (FOS): 1.964

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
DMZ	120	1536.8	0	5011	0

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

IBWC
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REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GLOBAL STABILITY SPEC 1 - DEEP SOIL MIXING ASSESSMENT

RAPID DRAWDOWN

ARCADIS

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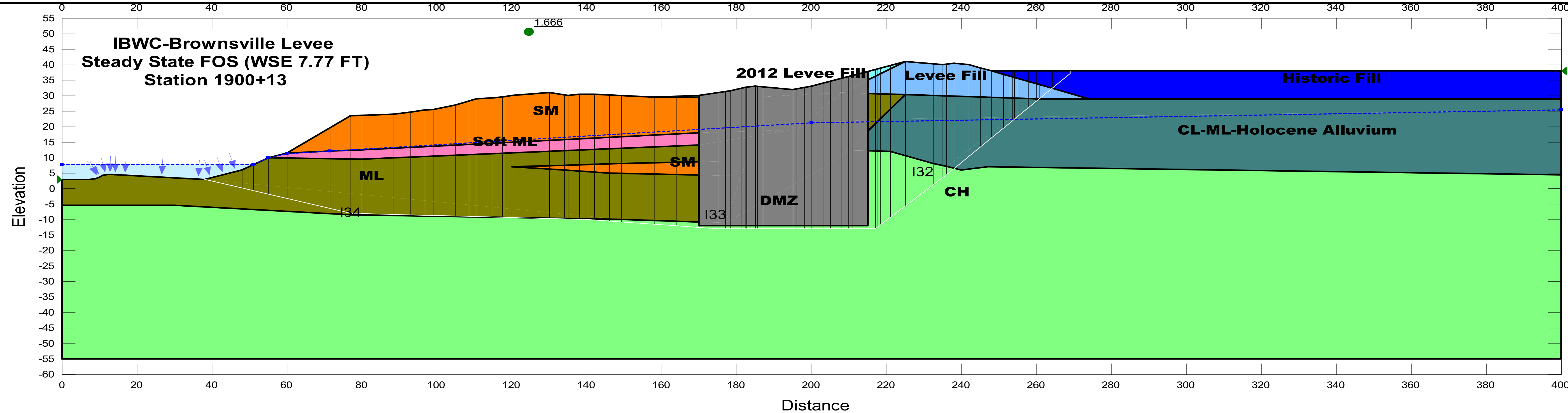
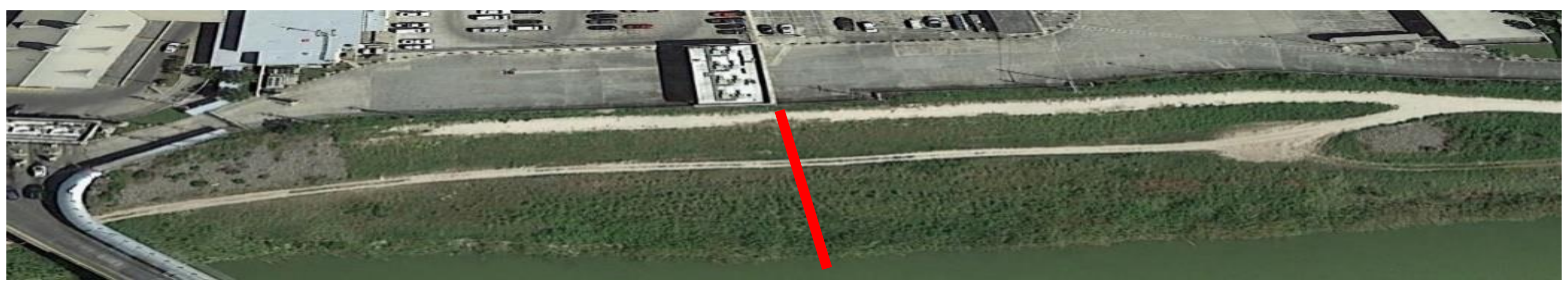


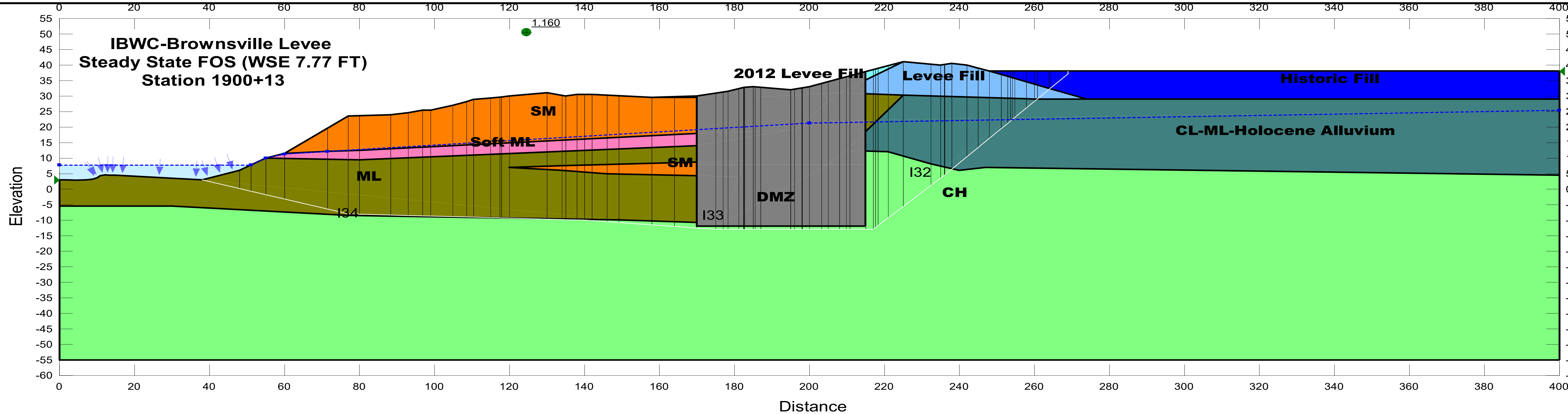


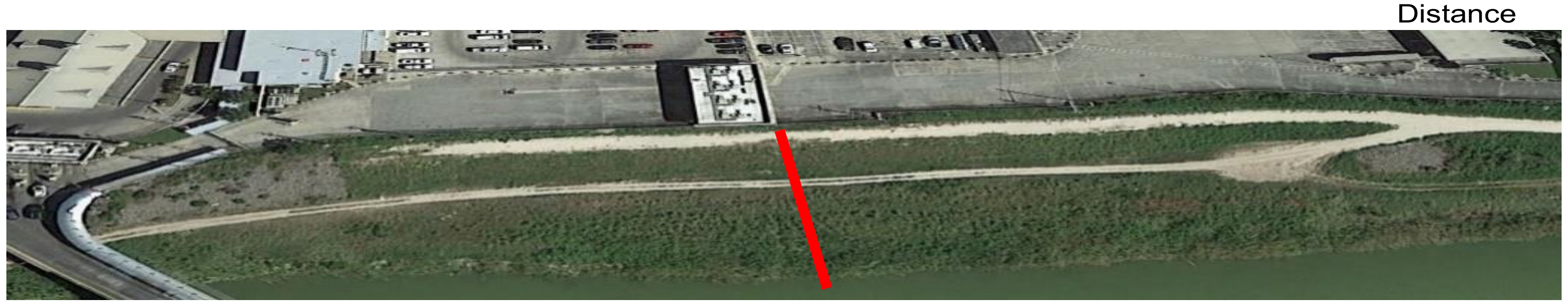
Minimum Factor of Safety (FOS): 1.666

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

IBWC GEOTECHNICAL REPORT	
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT **GLOBAL STABILITY SPEC 4 - DEEP SOIL MIXING ASSESSMENT** **STEADY STATE SEEPAGE**	
ARCADIS	**APPENDIX**





Minimum Factor of Safety (FOS): 1.360

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
DMZ	120	1536.8	0

Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1

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REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

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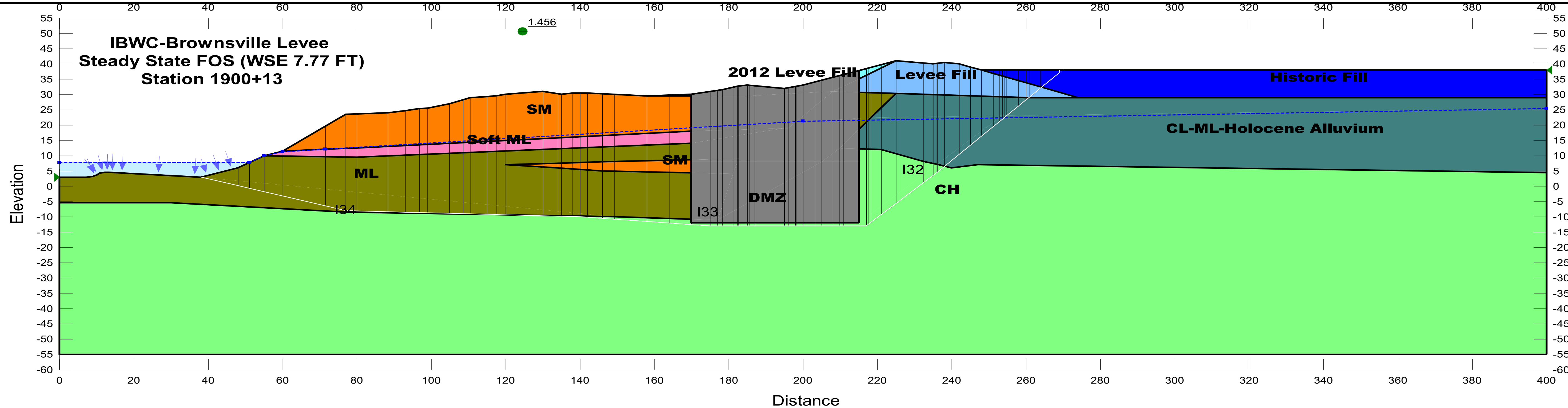

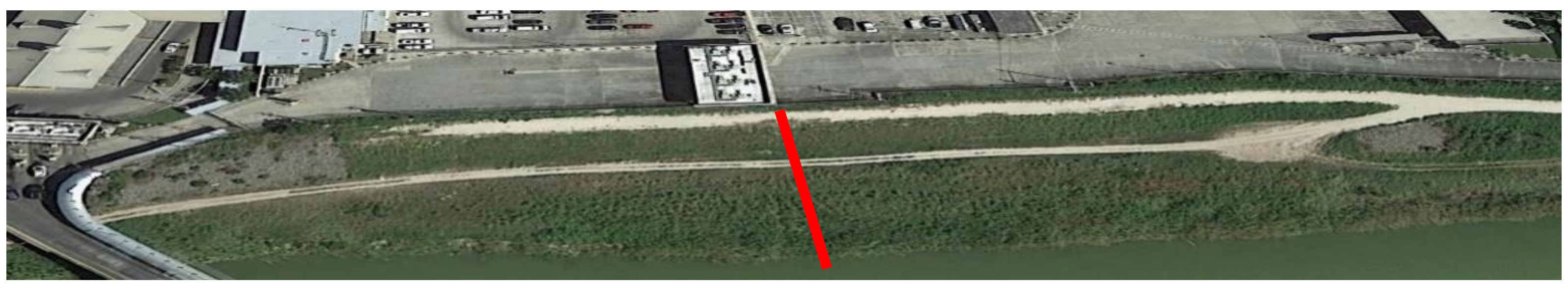

Minimum Factor of Safety (FOS): 1.456

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

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REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GLOBAL STABILITY SPEC 4 - DEEP SOIL MIXING ASSESSMENT

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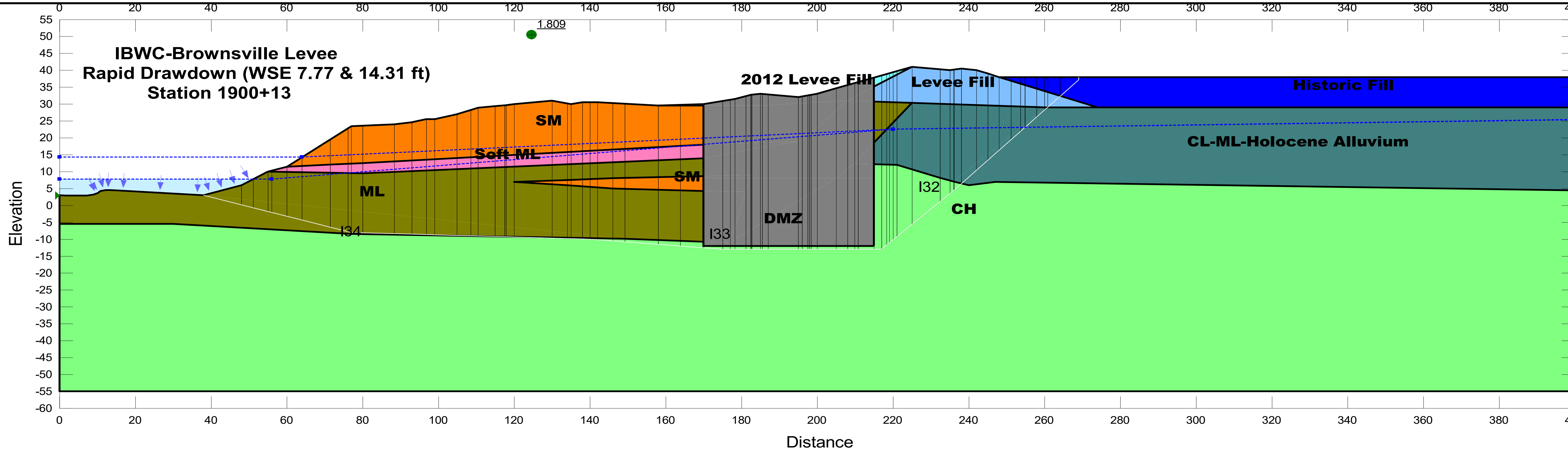

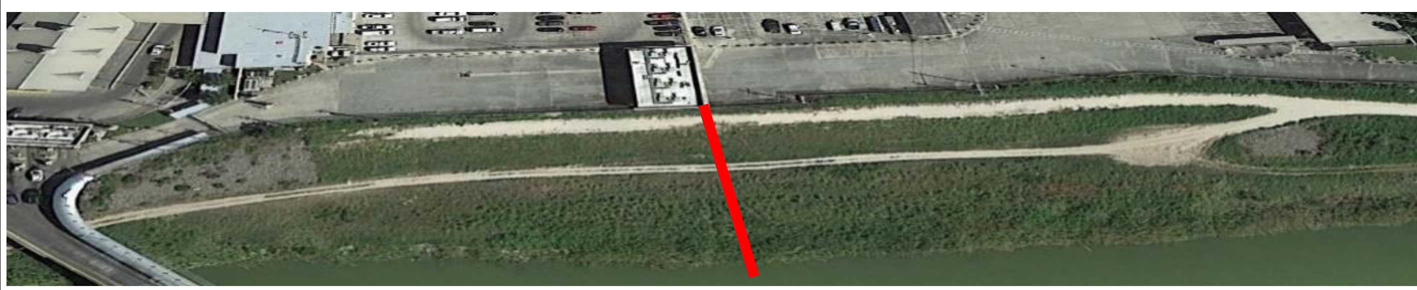

Minimum Factor of Safety (FOS): 1.809

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
DMZ	120	1536.8	0	5011	0

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

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APPENDIX K

Stone Columns Analyses

STONE COLUMNS

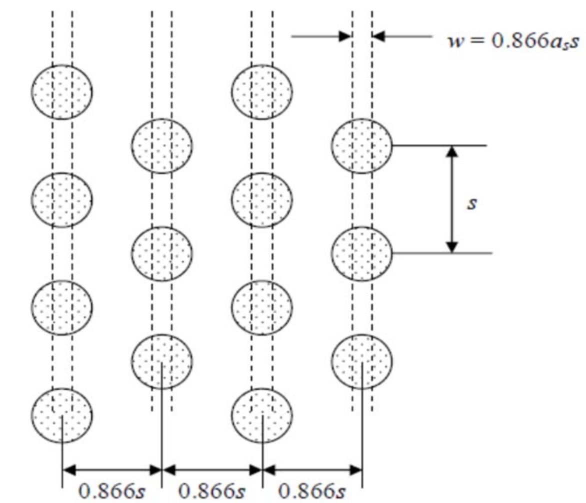
STONE COLUMNS MODELING CALCULATIONS

For	IBWC - Brownsville Levee Rehab	Job No.	LA003315.0000	Sheet No.	1 of 1
Calc	Stone Column Modeling Calculations	Made By	CW	Checked by	KL
		Date	12/1/2016	Date	12/2/2016

ARCADIS

Key	
Input	
Calculation	

d = Column Diameter =	36	in. =	3.0	ft
s = Center-to-Center column spacing =	6	ft		
D_e = Effective unit cell diameter = 1.05s =	6.3	ft		
A = Unit cell area = $\Pi(D_e/2)^2$ =	28.27	sq ft		
A_s = Area of single column = $\Pi(D_e/2)^2$ =	7.07	sq ft		
α_s = Area replacement ratio = A_s/A =	0.250			
a_s = Area improvement ratio = $1/\alpha_s$ =	4.00			
w = Width of modeled SC strip = $0.866a_s s$ =			1.30	
s' = Modeled center-to-center spacing between SC strips = 0.866s =			5.20	
z = Modeled distance between edges of SC strips = s' - w =			3.90	



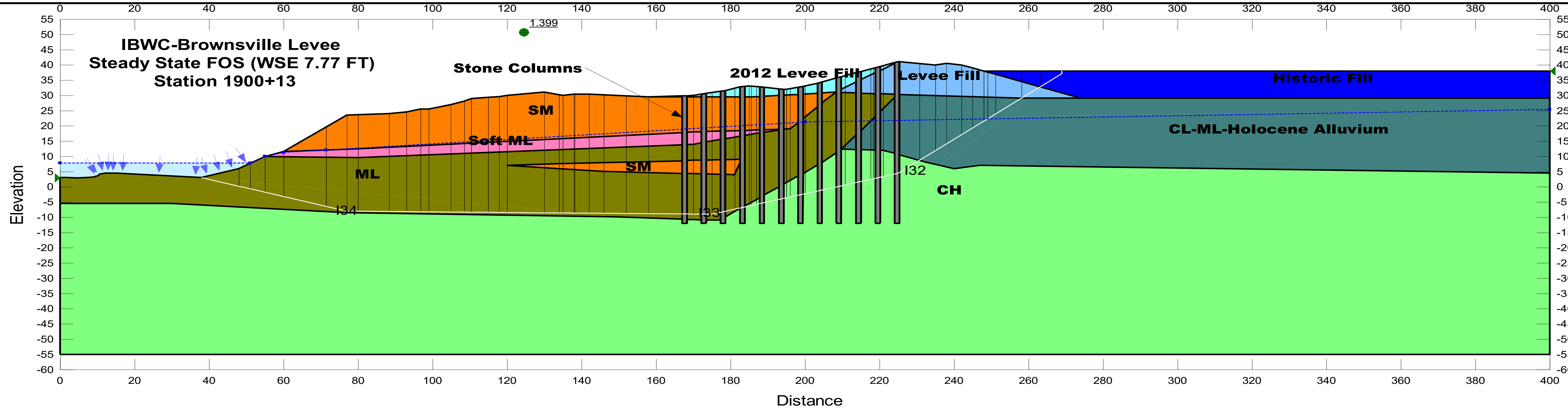
b) Plan View

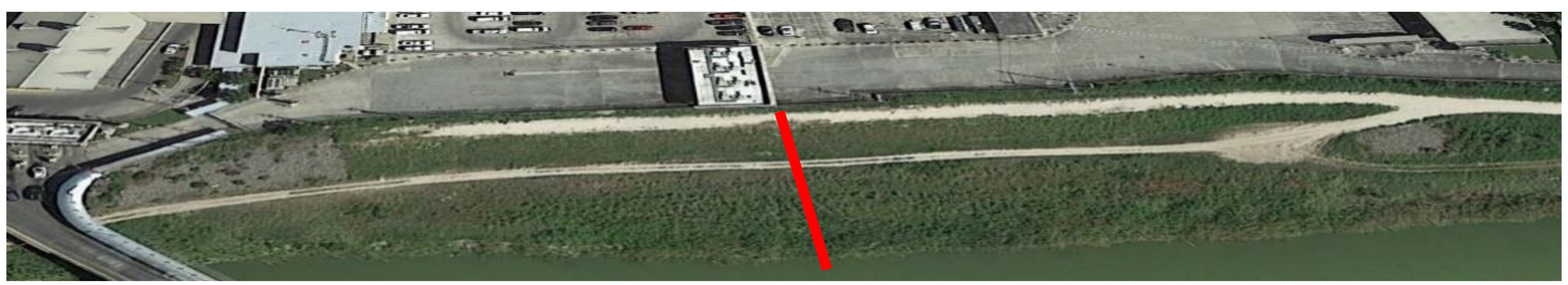
Figure B-2. Stone column strip idealization (after Barksdale and Bachus 1983)

From Filz and Navin, 2006

SLOPE STABILITY

STONE COLUMNS AT THE LEVEE





Minimum Factor of Safety (FOS): 1.399

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

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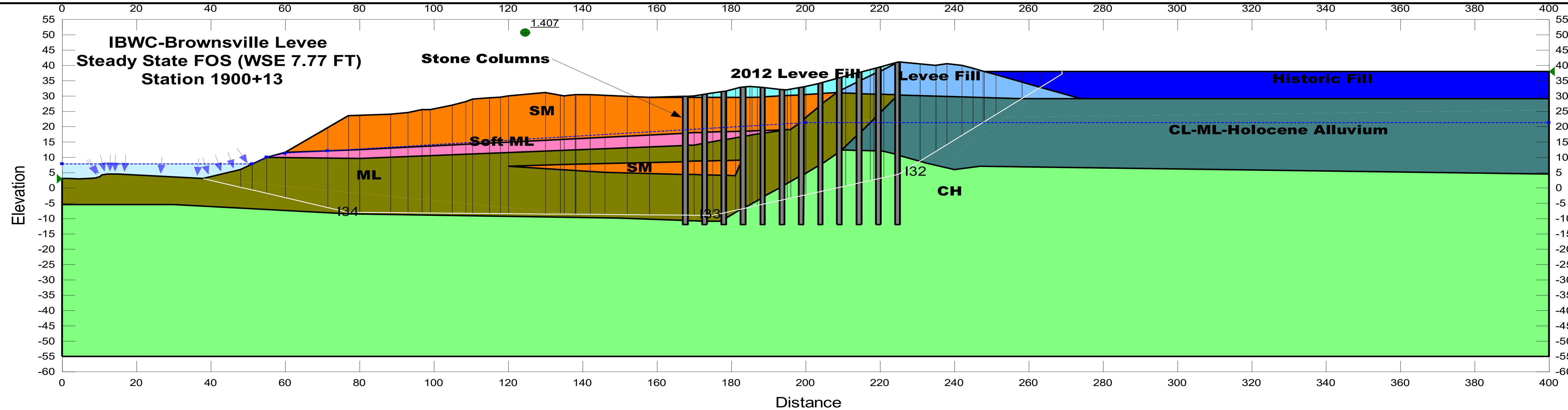

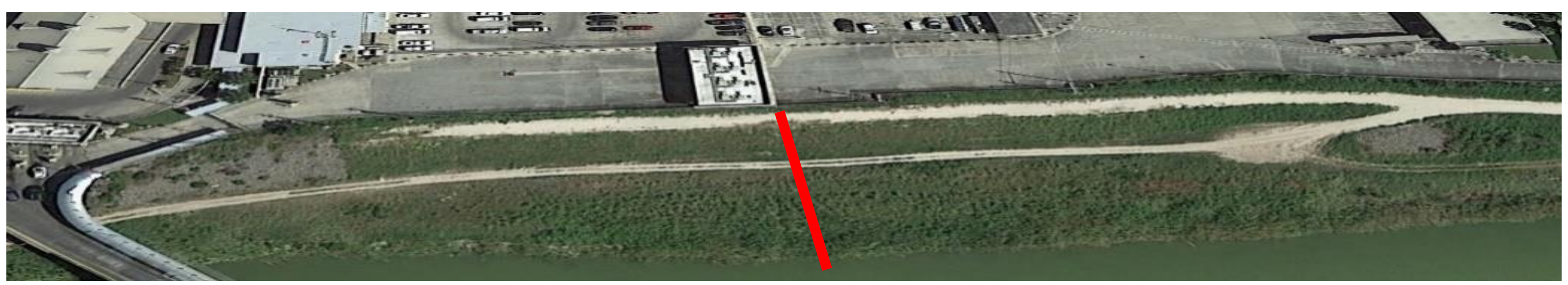

Minimum Factor of Safety (FOS): 1.407

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
DMZ	120	1536.8	0

Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1

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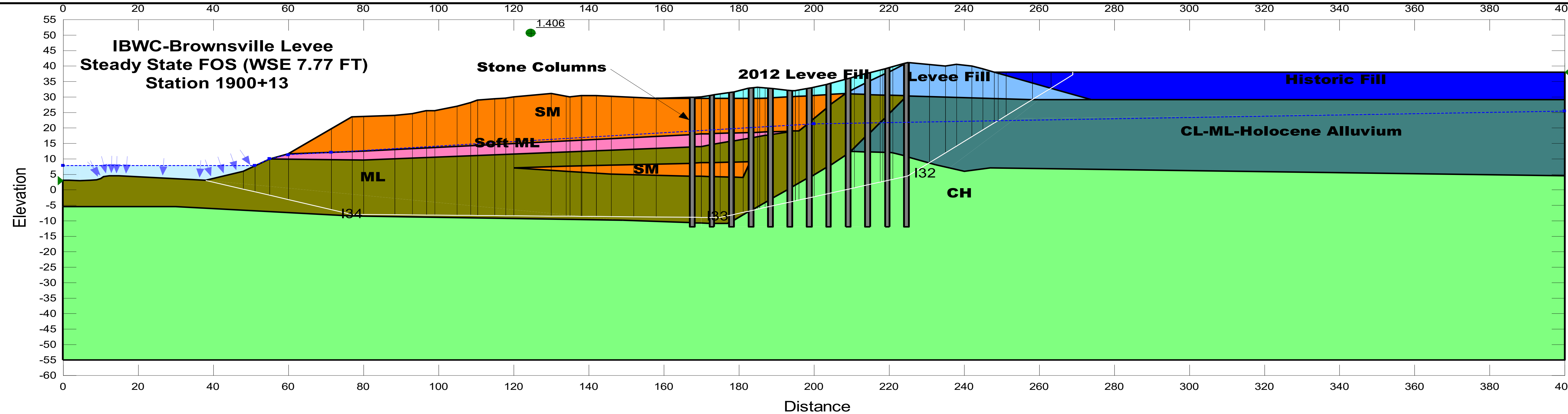

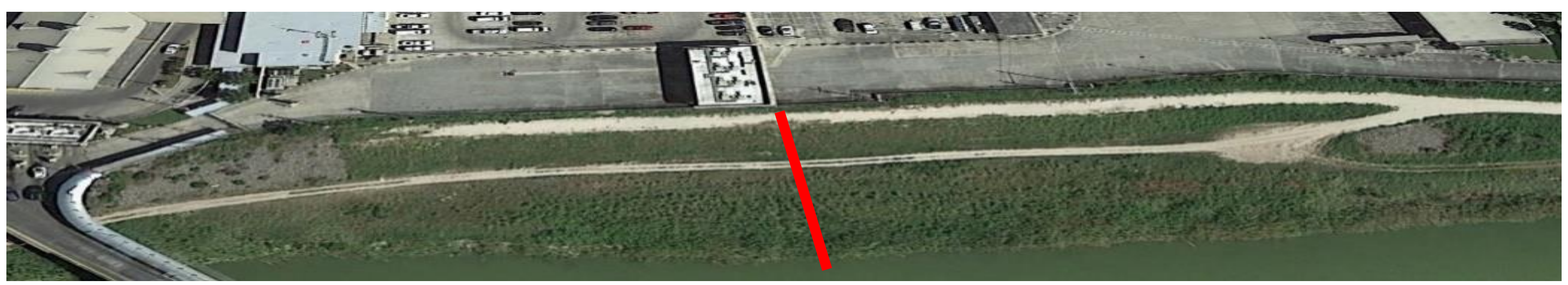

Minimum Factor of Safety (FOS): 1.406

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

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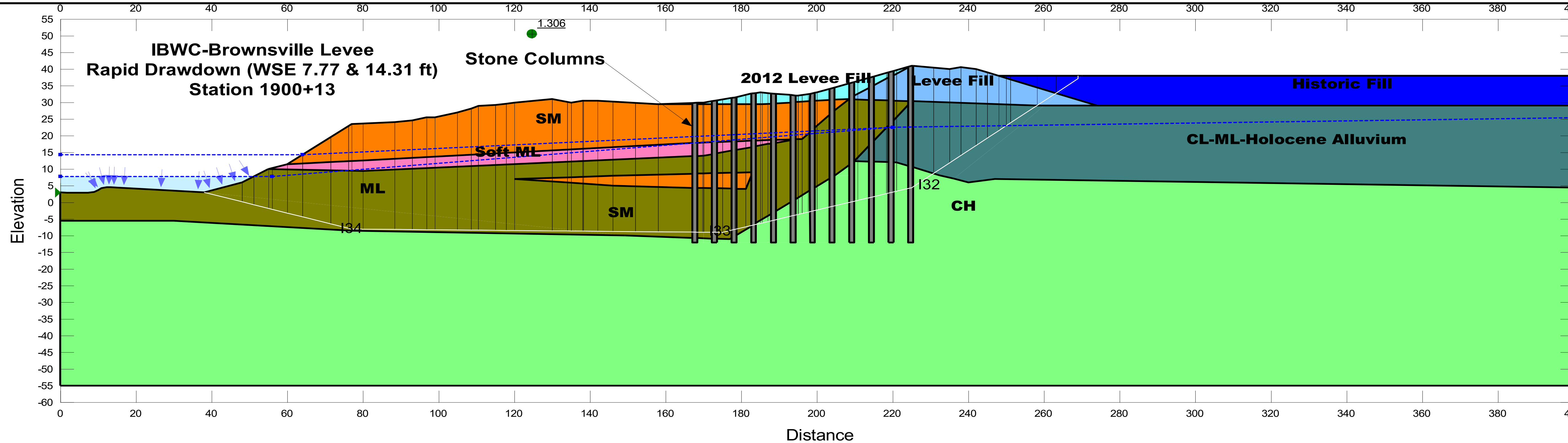

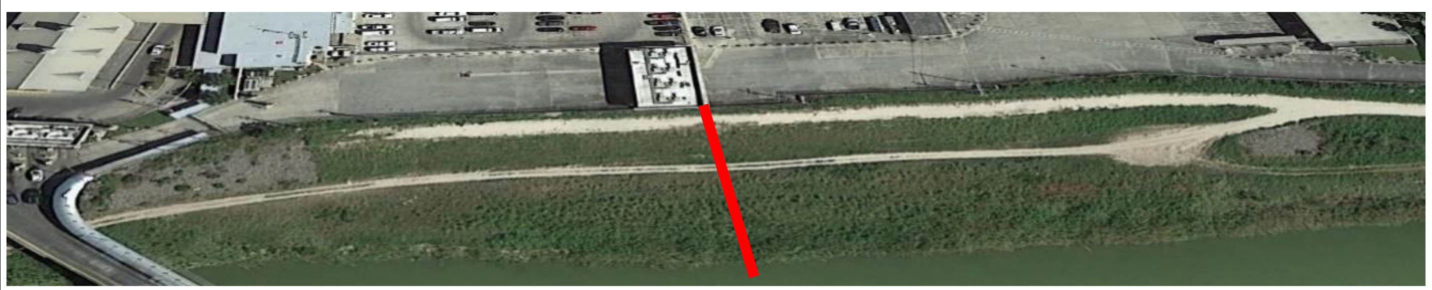

Minimum Factor of Safety (FOS): 1.306

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
DMZ	120	1536.8	0	5011	0

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

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SLOPE STABILITY MODEL - STONE COLUMNS ASSESSMENT

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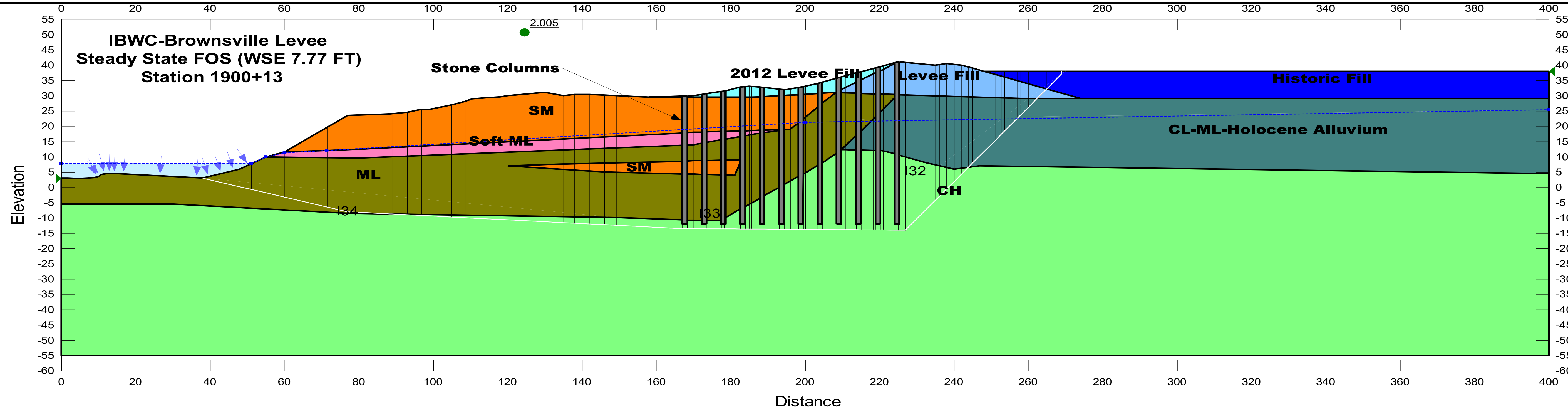

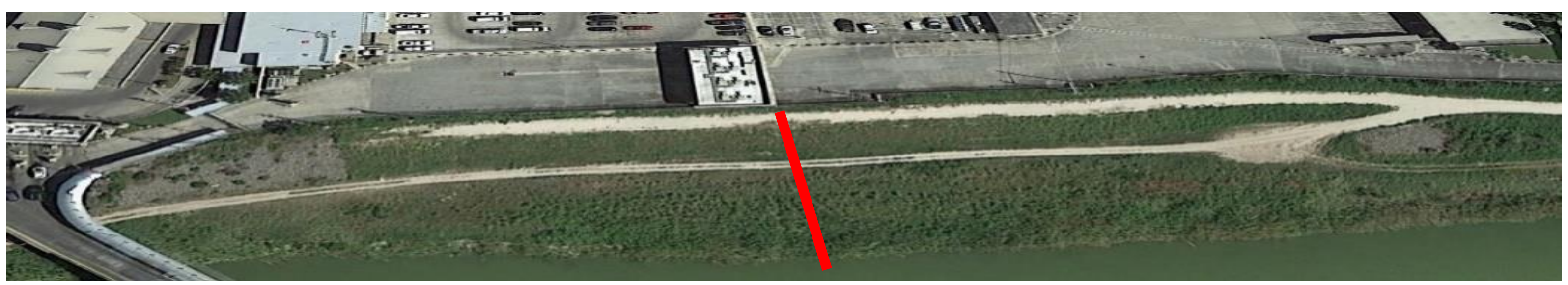

Minimum Factor of Safety (FOS): 2.005

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

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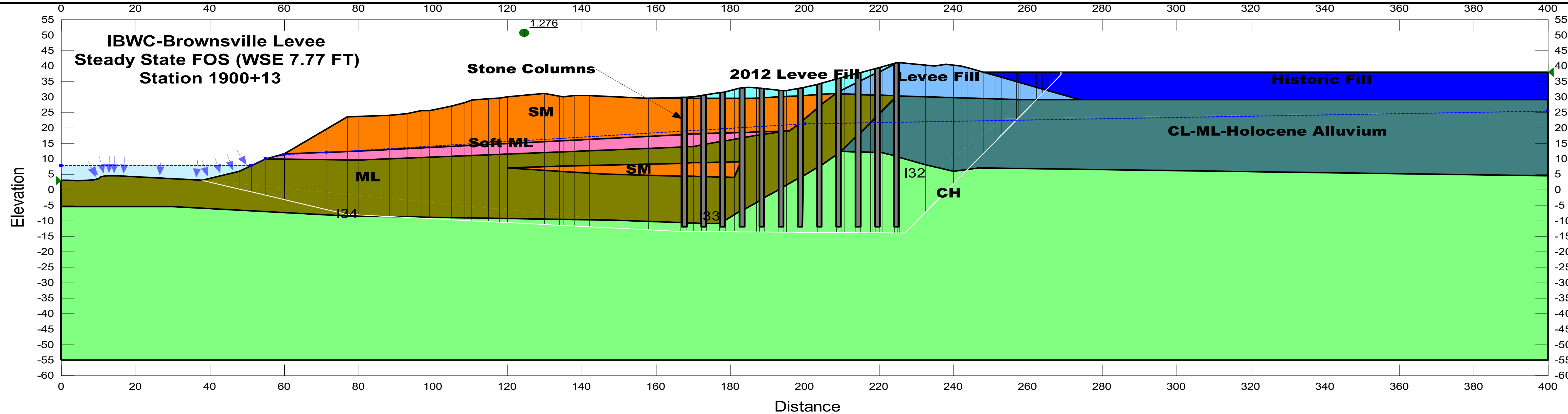

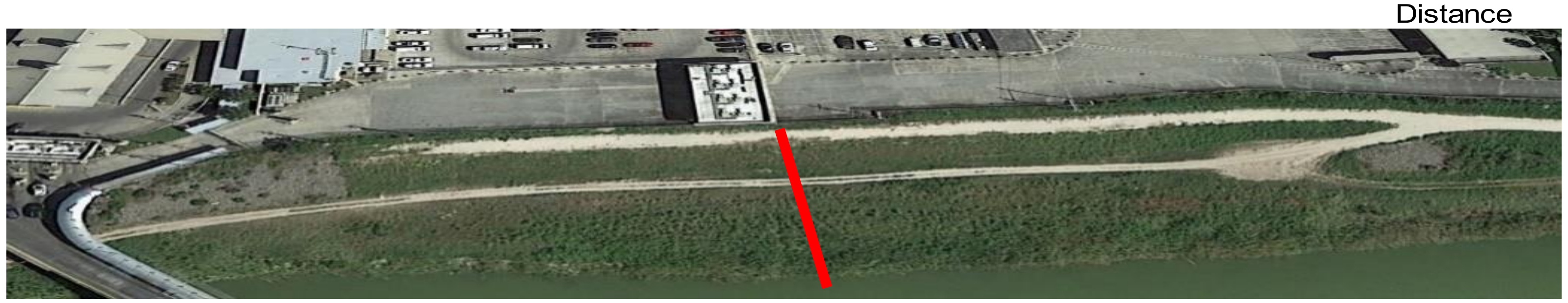

Minimum Factor of Safety (FOS): 1.376

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
DMZ	120	1536.8	0

Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1

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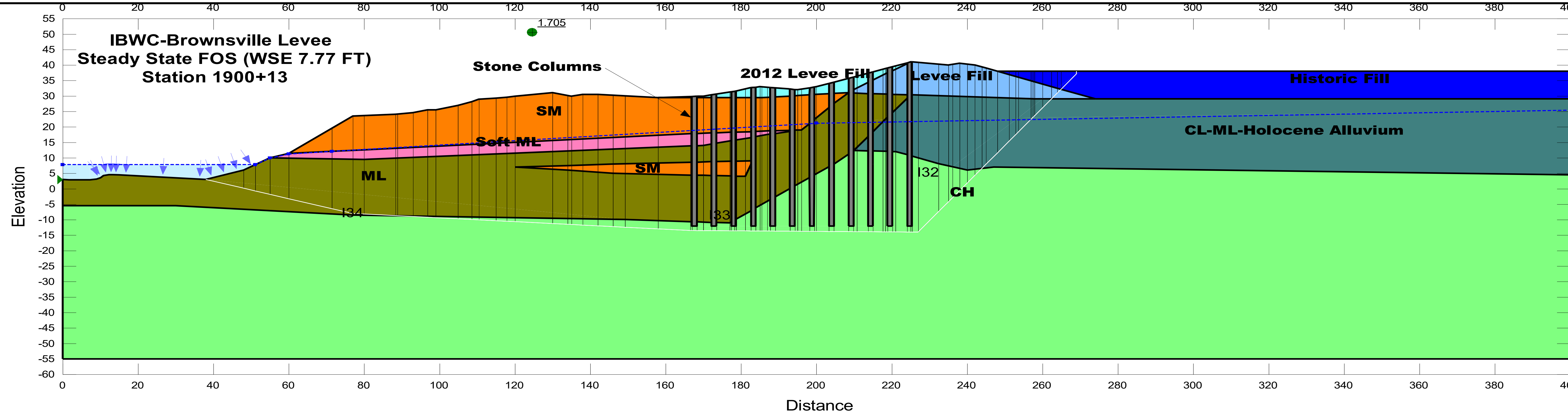

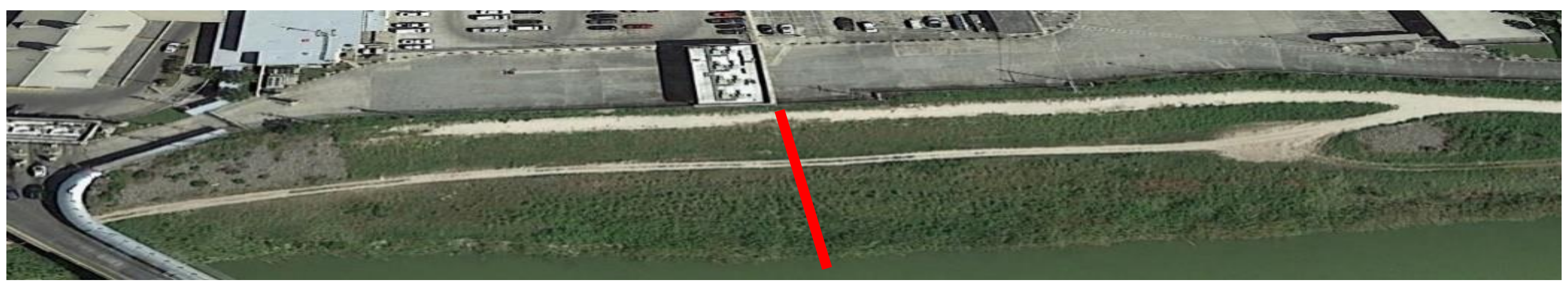

Minimum Factor of Safety (FOS): 1.705

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

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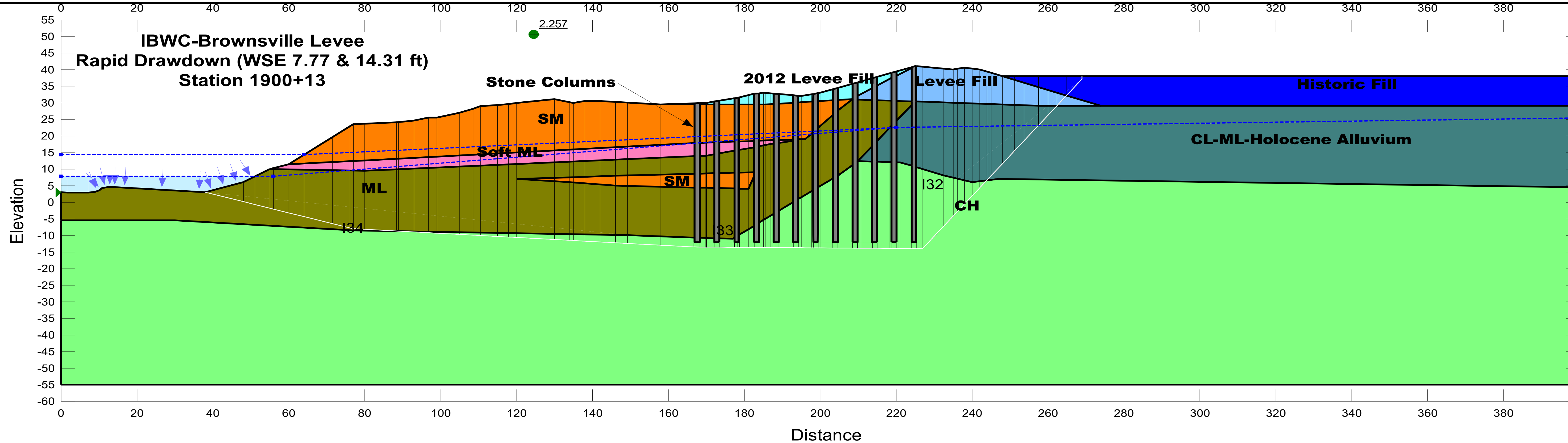

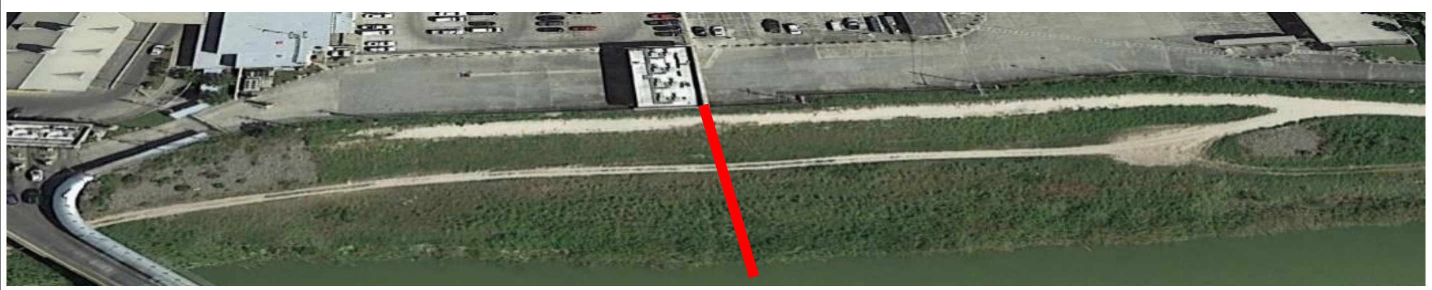

Minimum Factor of Safety (FOS): 2.257

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
DMZ	120	1536.8	0	5011	0

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

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GLOBAL SHALLOW - STONE COLUMNS ASSESSMENT

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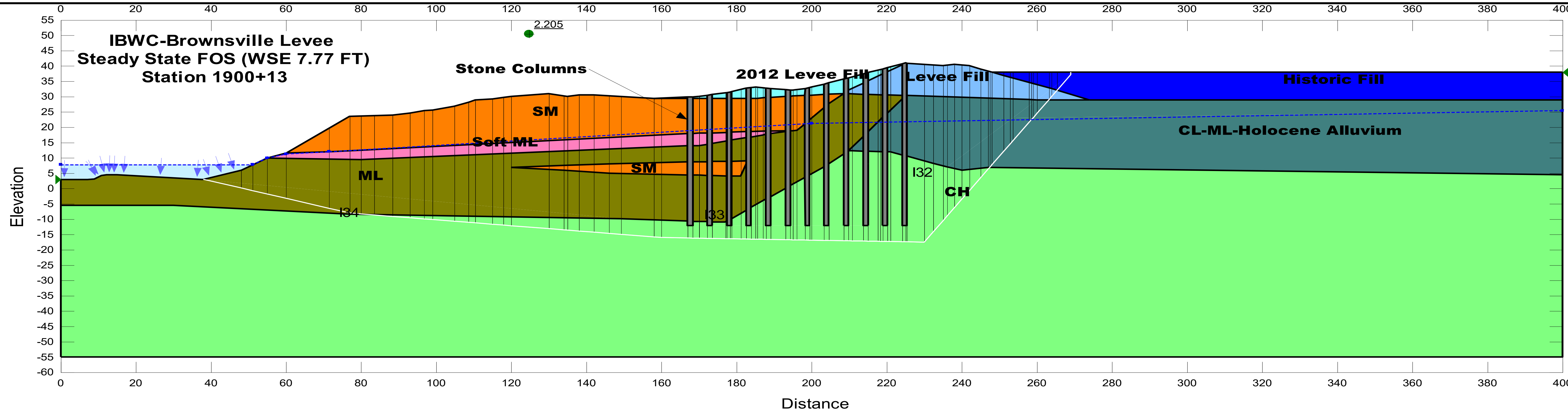

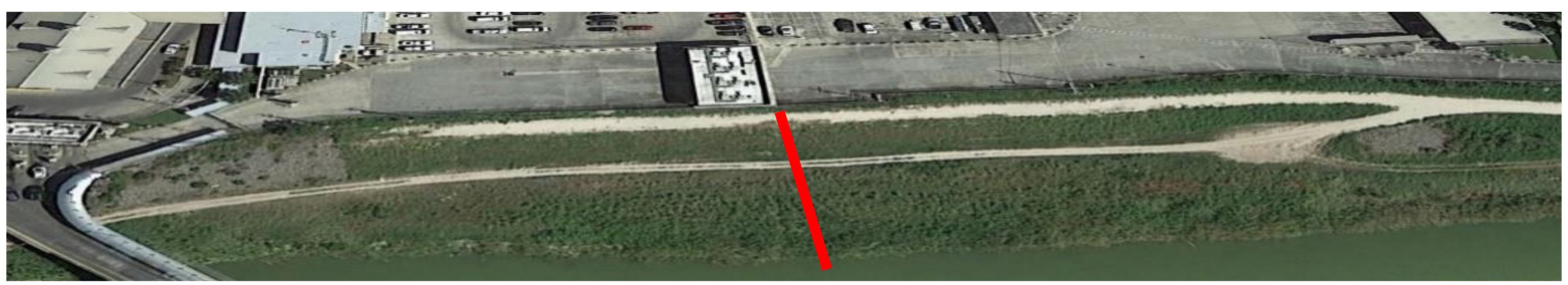

Minimum Factor of Safety (FOS): 2.205

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

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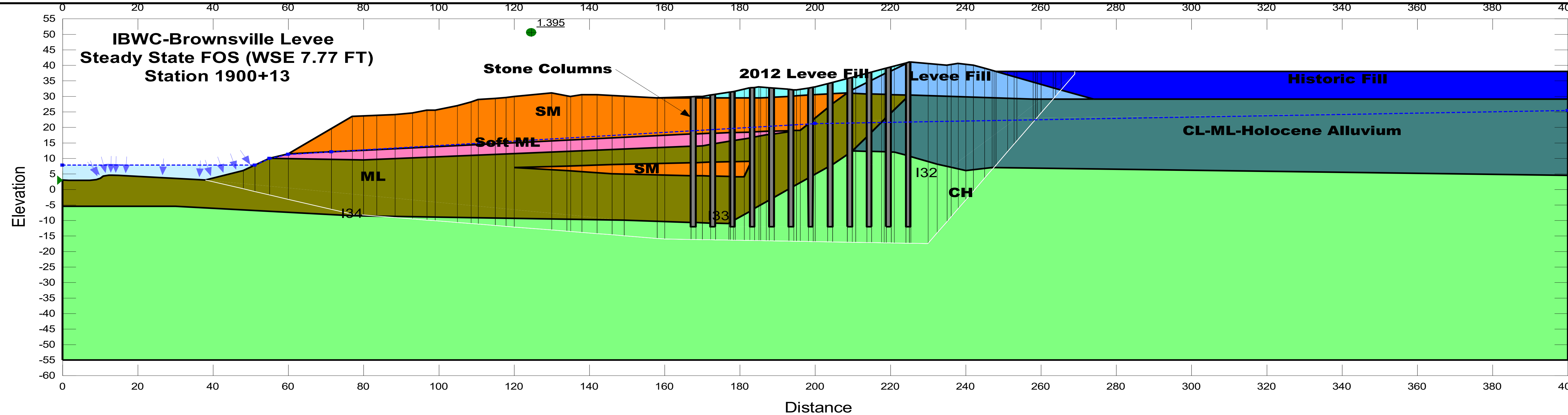

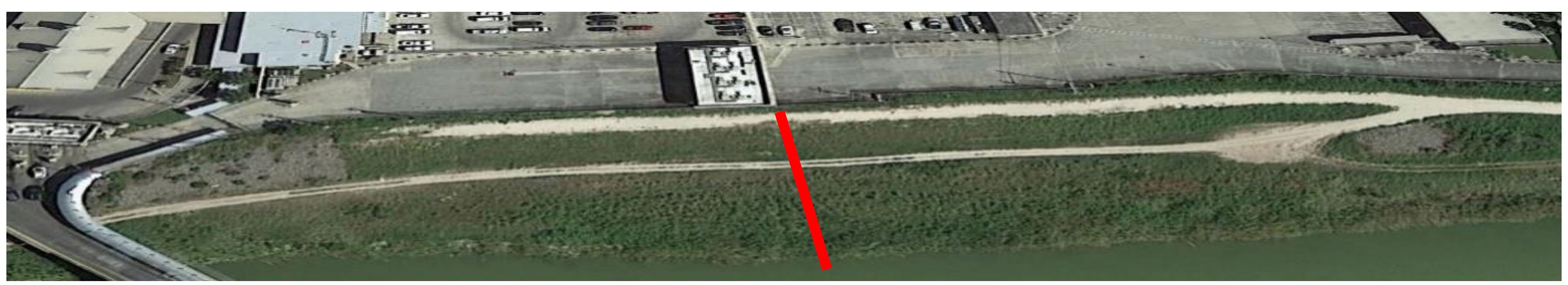

Minimum Factor of Safety (FOS): 1.395

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
DMZ	120	1536.8	0

Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1

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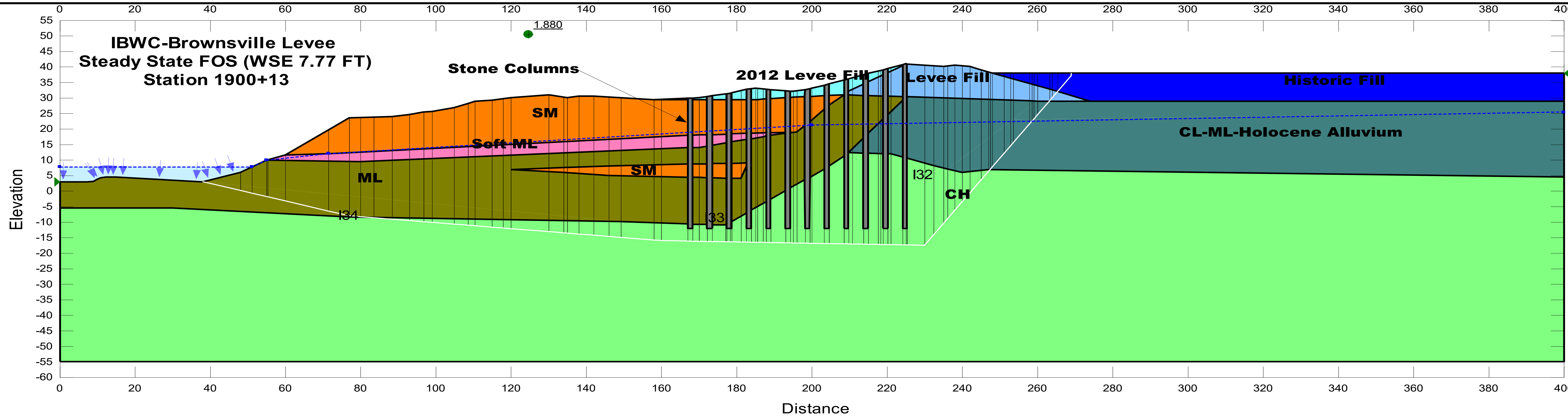

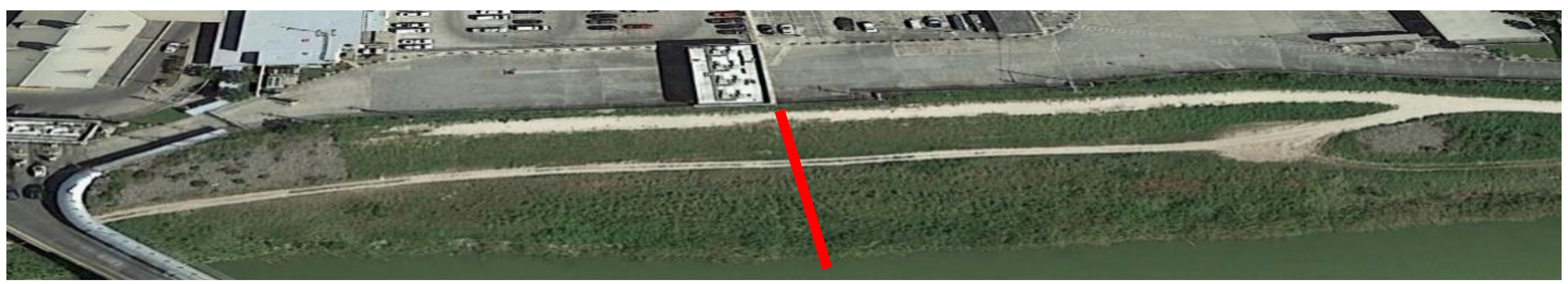

Minimum Factor of Safety (FOS): 1.880

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

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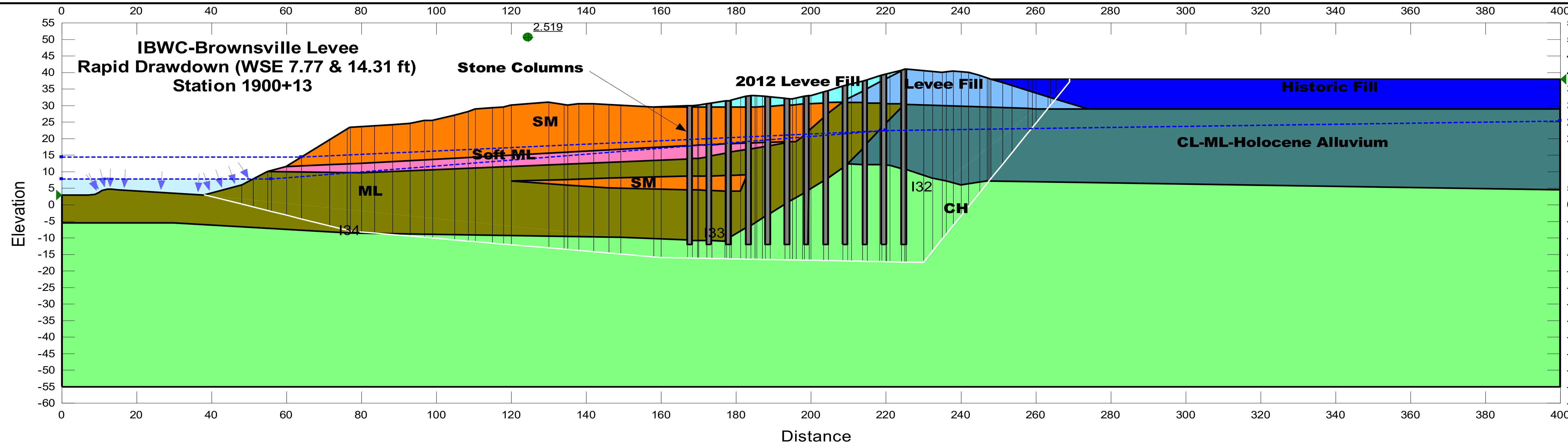

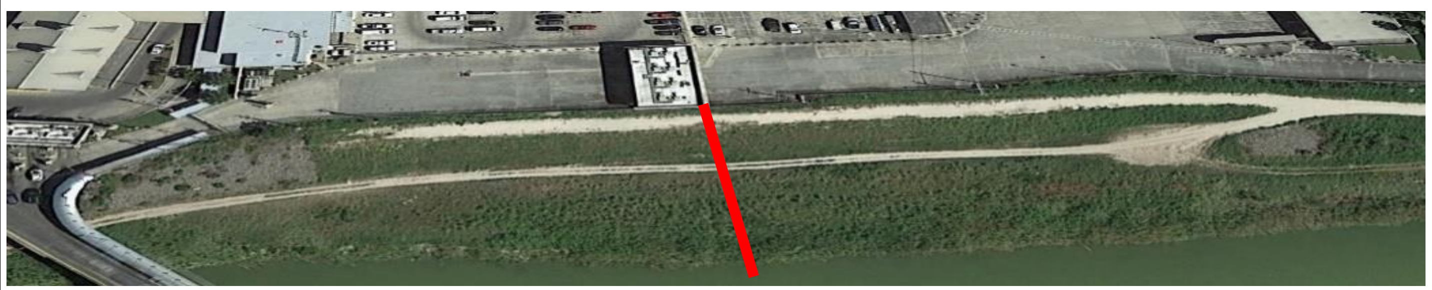

Minimum Factor of Safety (FOS): 2.519

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
DMZ	120	1536.8	0	5011	0

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

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GLOBAL DEEP - STONE COLUMNS ASSESSMENT

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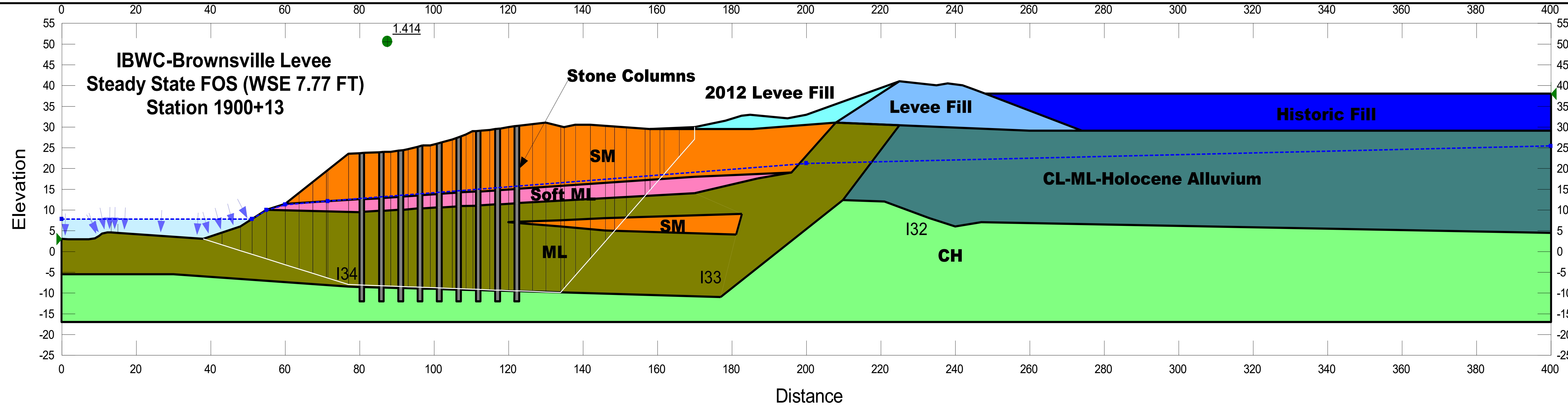
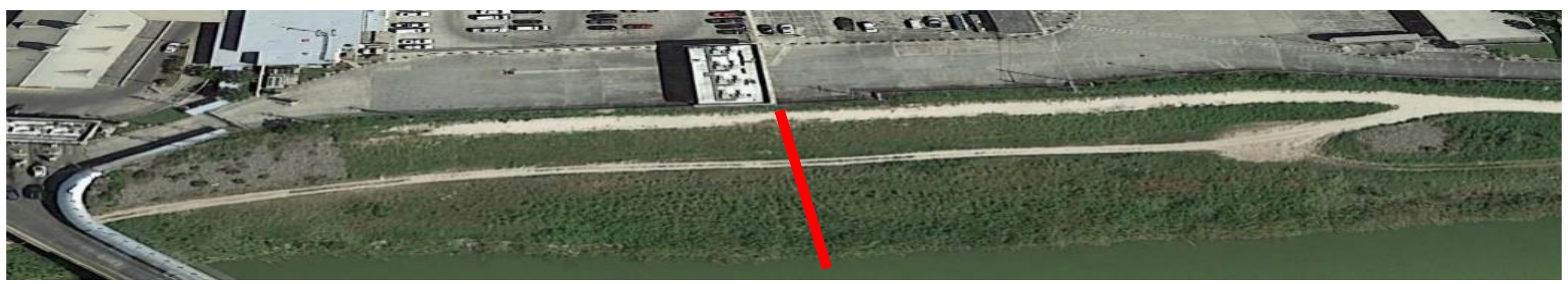
SLOPE STABILITY

STONE COLUMNS NEAR THE EDGE OF THE RIVER BANK





Minimum Factor of Safety (FOS): 1.414

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

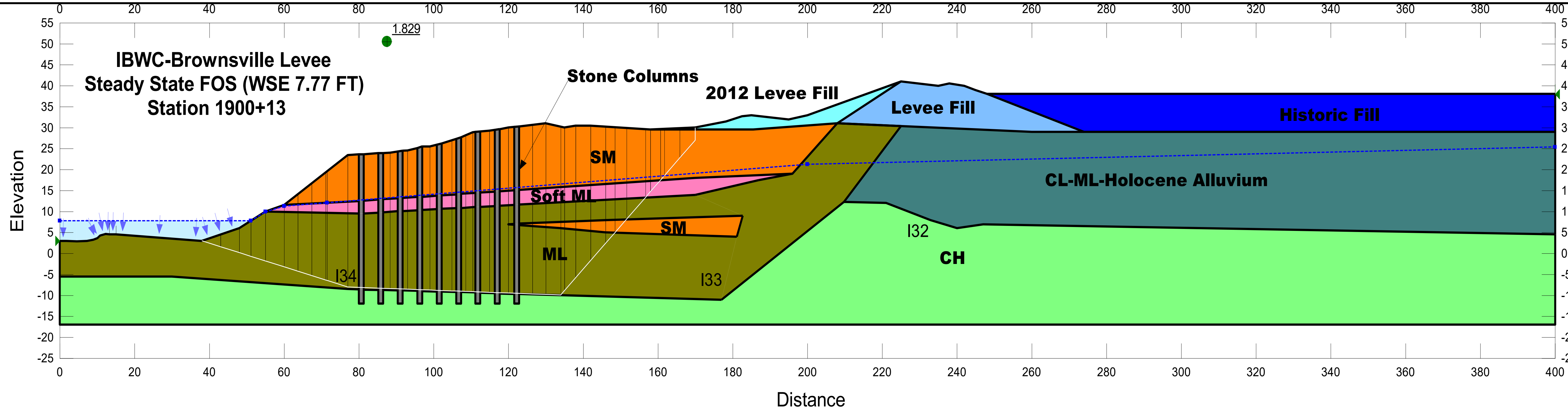
IBWC
GEOTECHNICAL REPORT

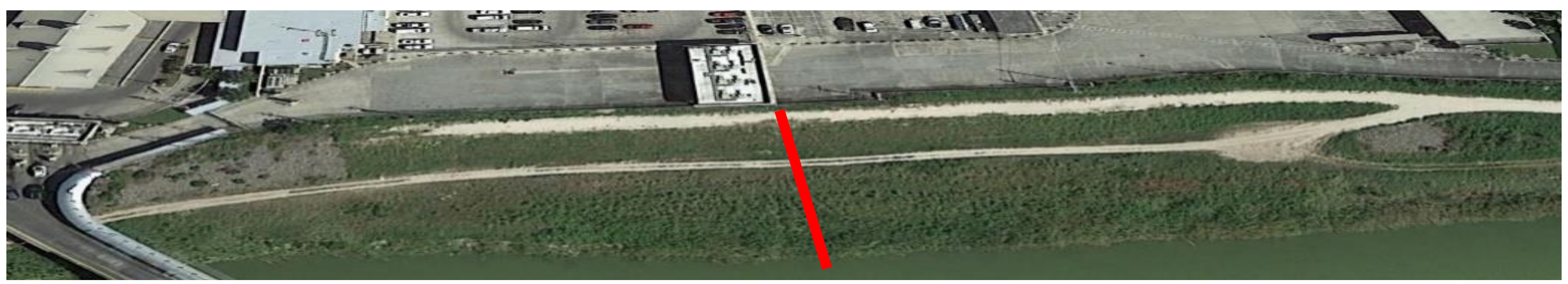
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - STONE COLUMNS ASSESSMENT

STEADY STATE SEEPAGE

ARCADIS | **APPENDIX**





Minimum Factor of Safety (FOS): 1.829

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
DMZ	120	1536.8	0
Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1			

IBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - STONE COLUMNS ASSESSMENT

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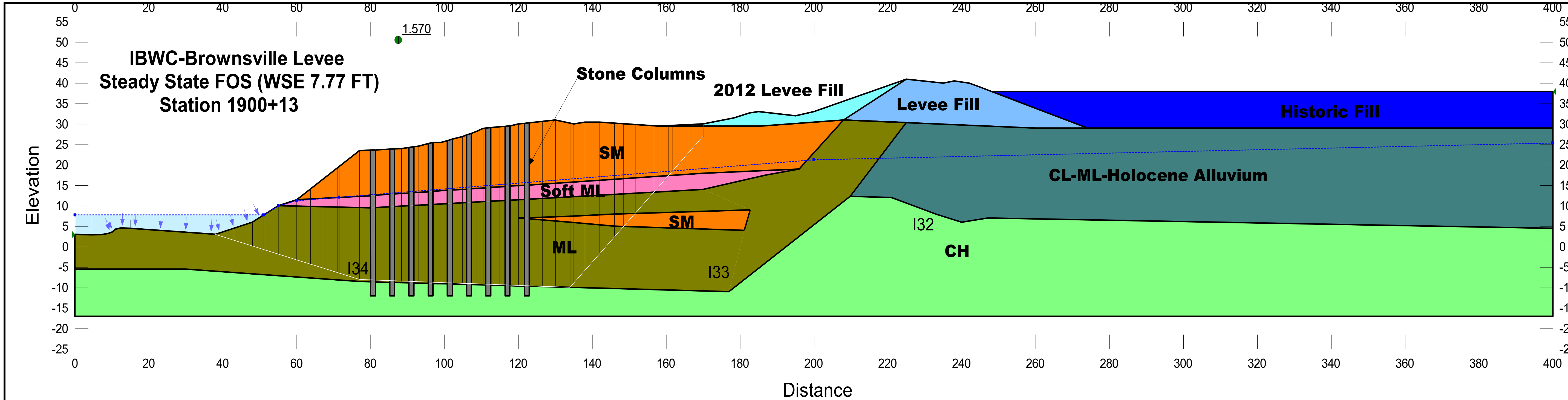
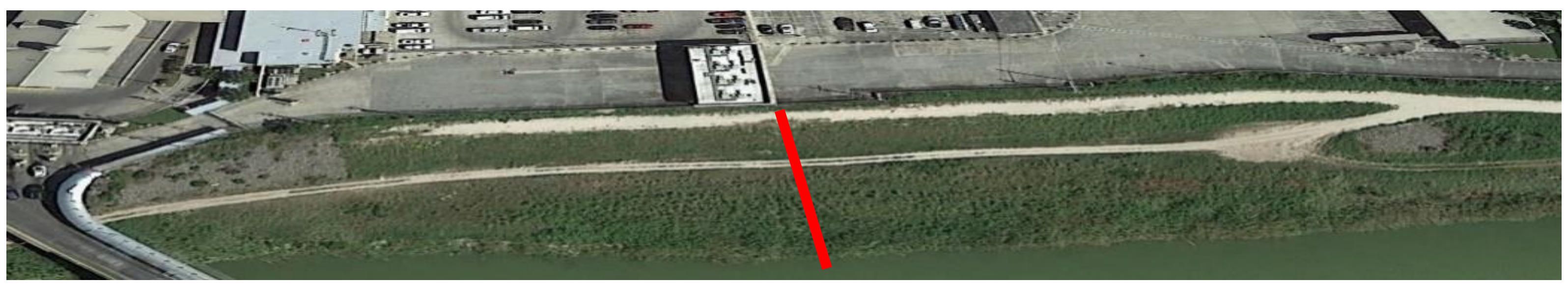
ARCADIS | **APPENDIX**





Minimum Factor of Safety (FOS): 1.570

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

IBWC
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REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - STONE COLUMNS ASSESSMENT

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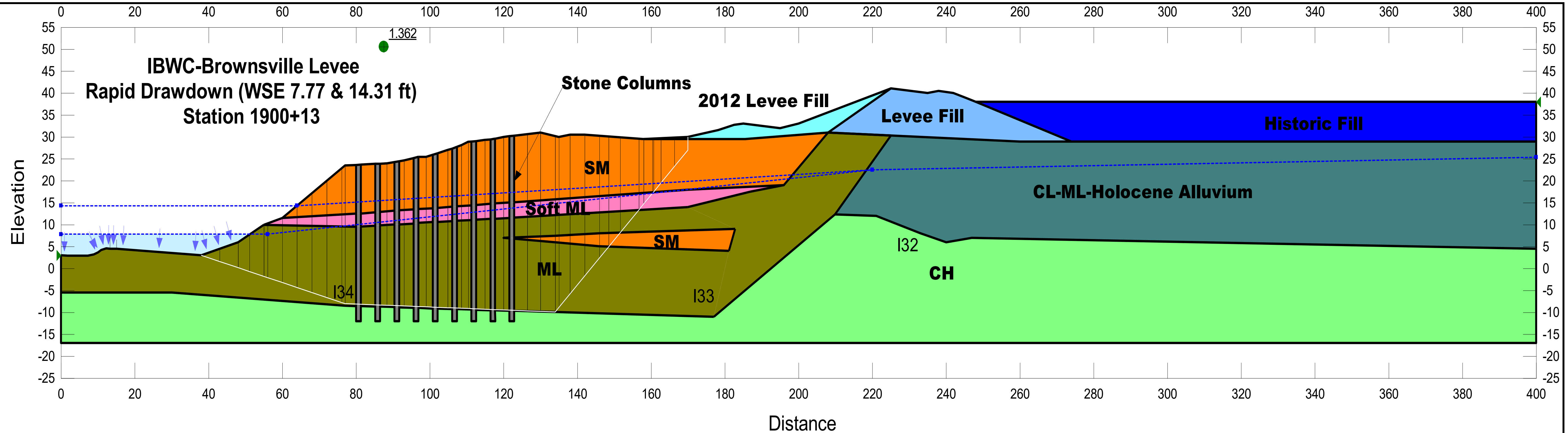
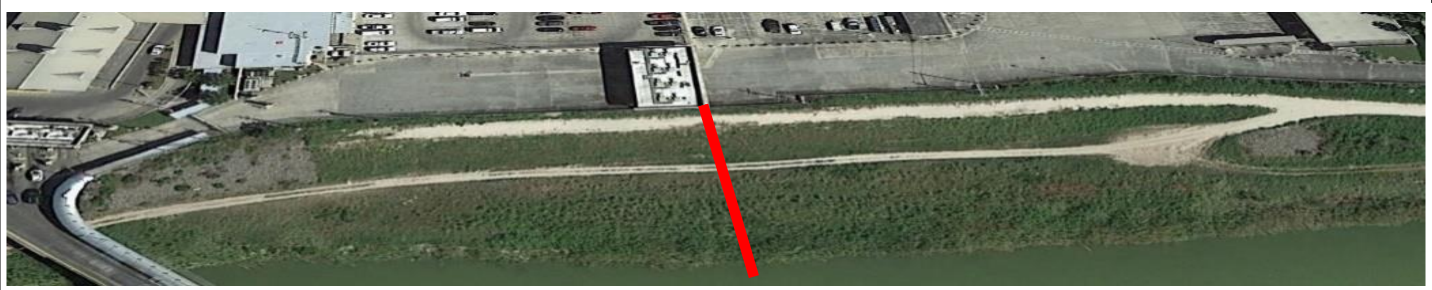




Minimum Factor of Safety (FOS): 1.362

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
DMZ	120	1536.8	0	5011	0

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

IBWC
GEOTECHNICAL REPORT

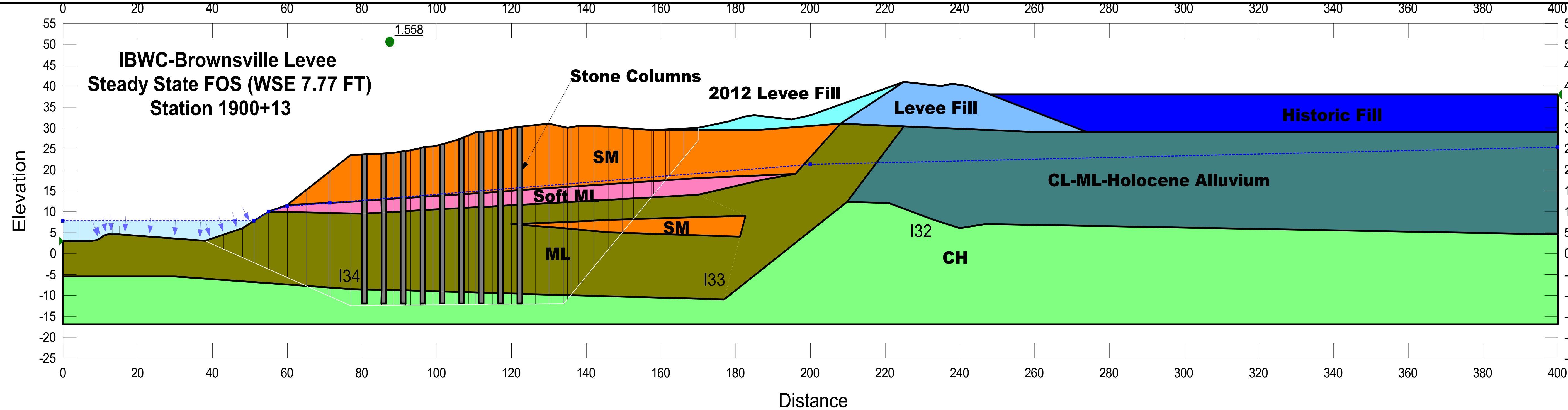
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

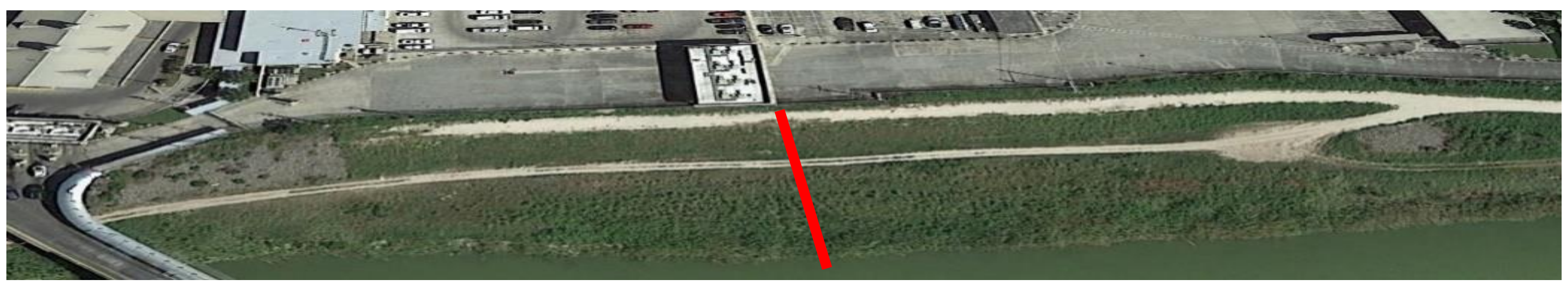
SLOPE STABILITY MODEL - STONE COLUMNS ASSESSMENT

RAPID DRAWDOWN

ARCADIS

APPENDIX





Minimum Factor of Safety (FOS): 1.558

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

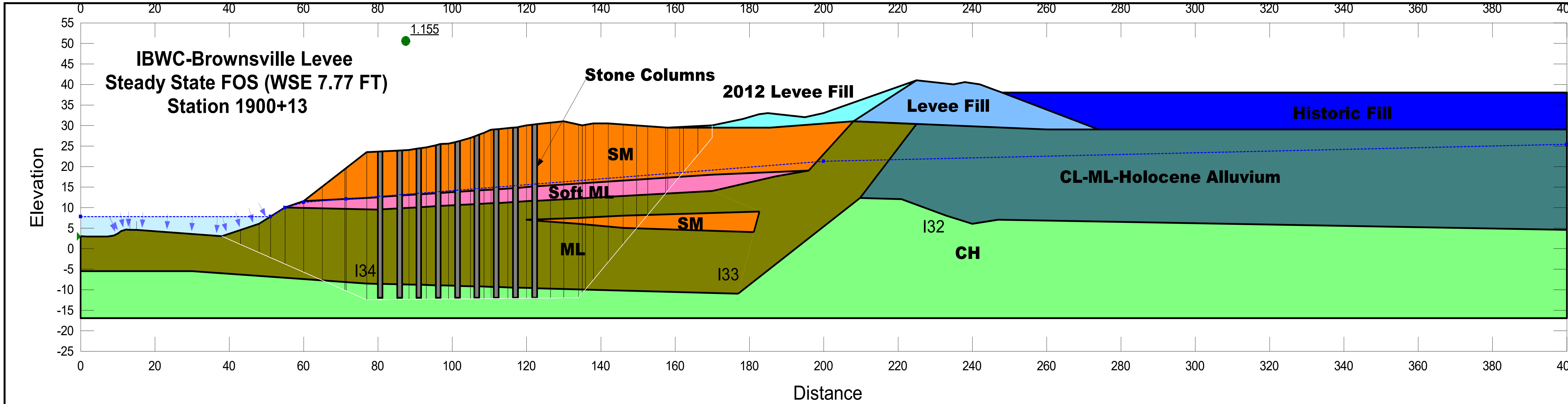
IBWC
GEOTECHNICAL REPORT

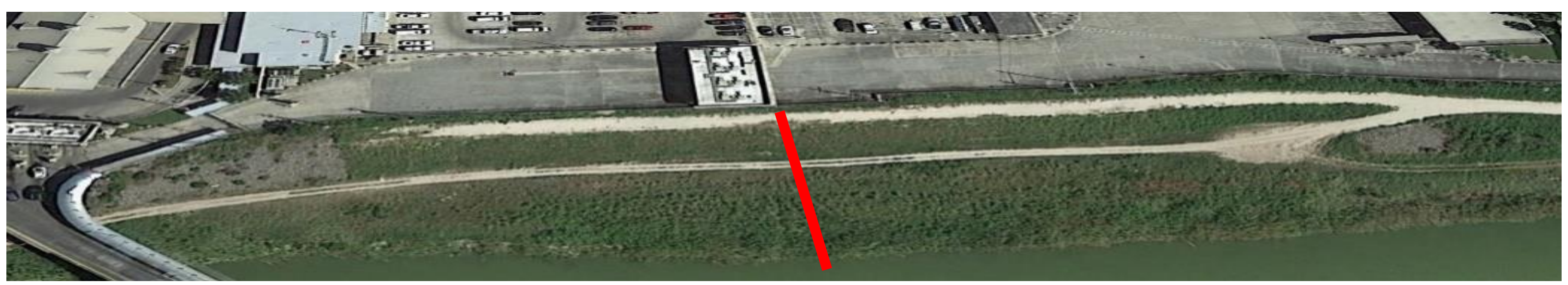
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GLOBAL SHALLOW - STONE COLUMNS ASSESSMENT

STEADY STATE SEEPAGE

ARCADIS | **APPENDIX**





Minimum Factor of Safety (FOS): 1.355

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
DMZ	120	1536.8	0
Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1			

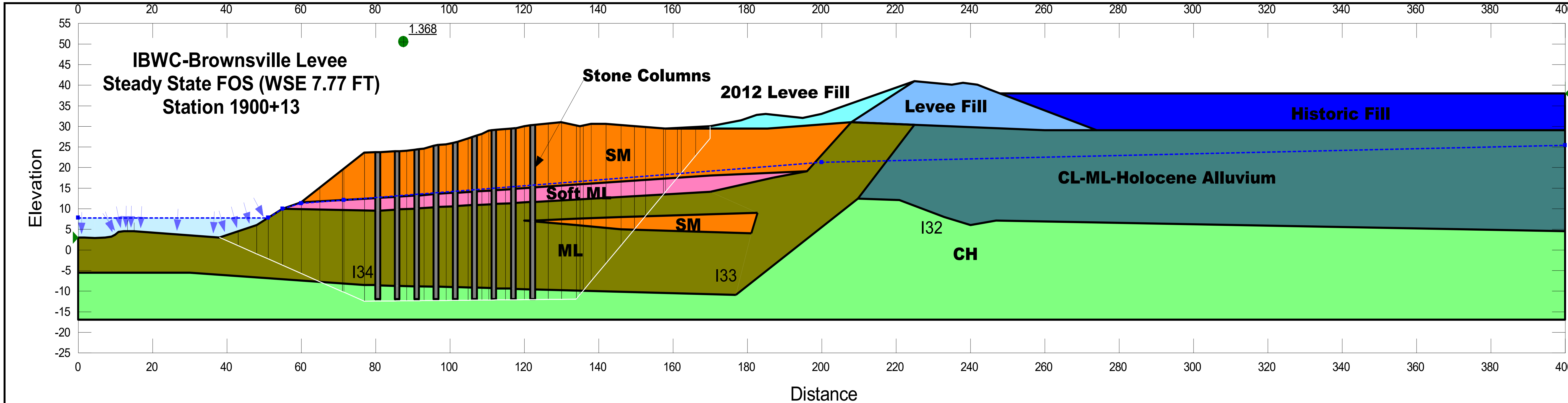
IBWC
GEOTECHNICAL REPORT

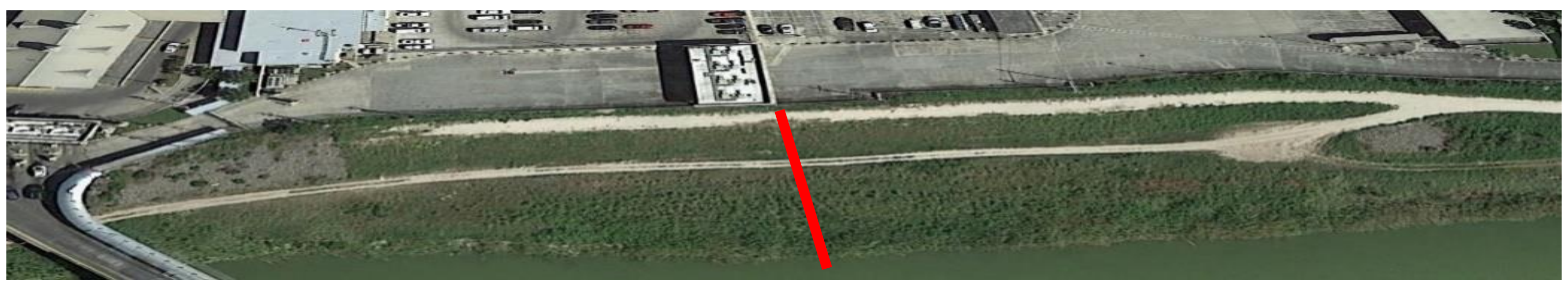
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GLOBAL SHALLOW - STONE COLUMNS ASSESSMENT

STEADY STATE SEEPAGE

ARCADIS | **APPENDIX**





Minimum Factor of Safety (FOS): 1.368

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

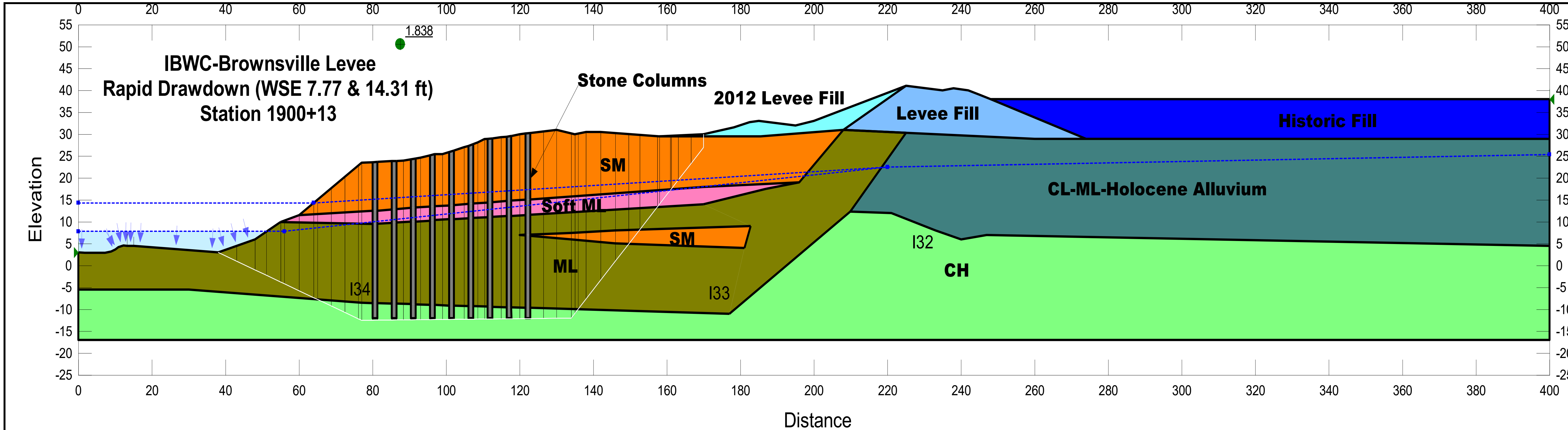
IBWC
GEOTECHNICAL REPORT

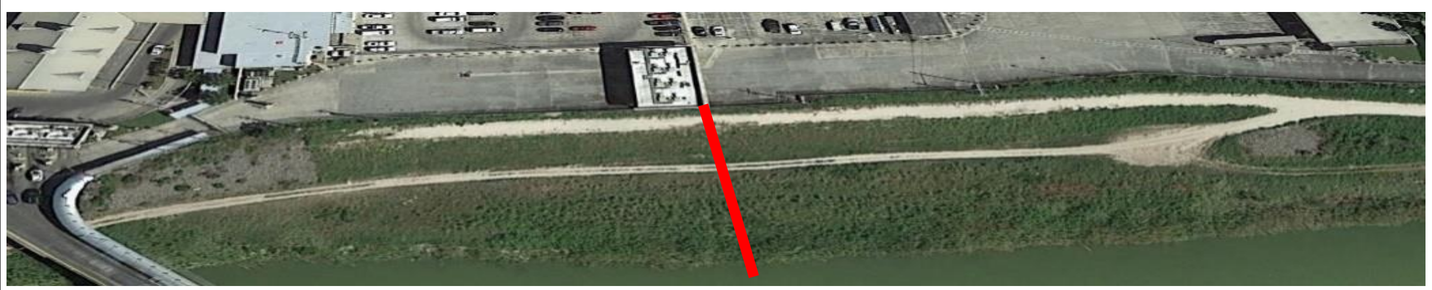
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GLOBAL SHALLOW - STONE COLUMNS ASSESSMENT

STEADY STATE SEEPAGE

ARCADIS | **APPENDIX**





Minimum Factor of Safety (FOS): 1.838

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
DMZ	120	1536.8	0	5011	0

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

IBWC
GEOTECHNICAL REPORT

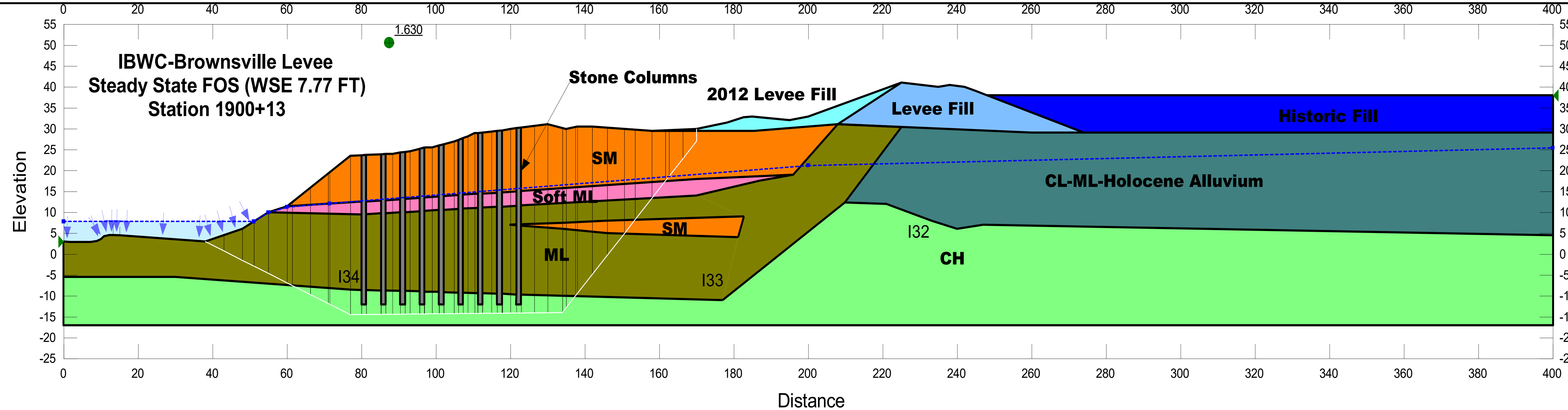
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

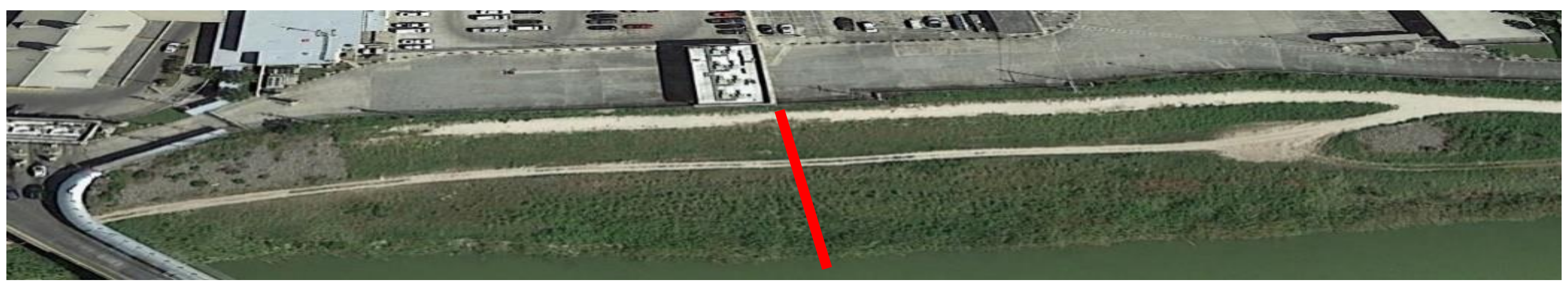
GLOBAL SHALLOW - STONE COLUMNS ASSESSMENT

RAPID DRAWDOWN

ARCADIS

APPENDIX





Minimum Factor of Safety (FOS): 1.630

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
DMZ	120	1536.8	0
Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1			

IBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GLOBAL DEEP - STONE COLUMNS ASSESSMENT

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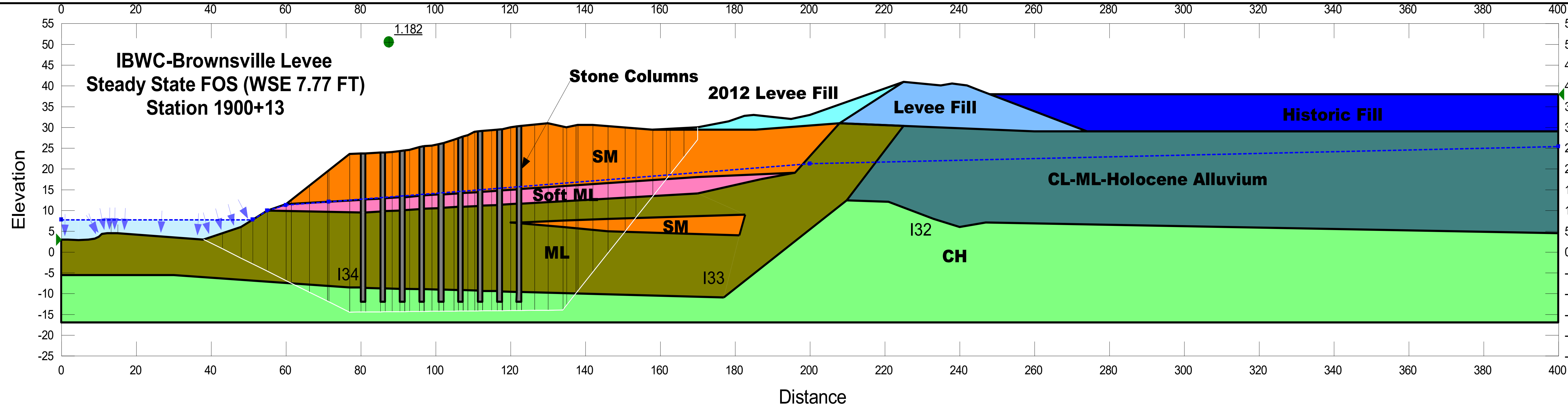

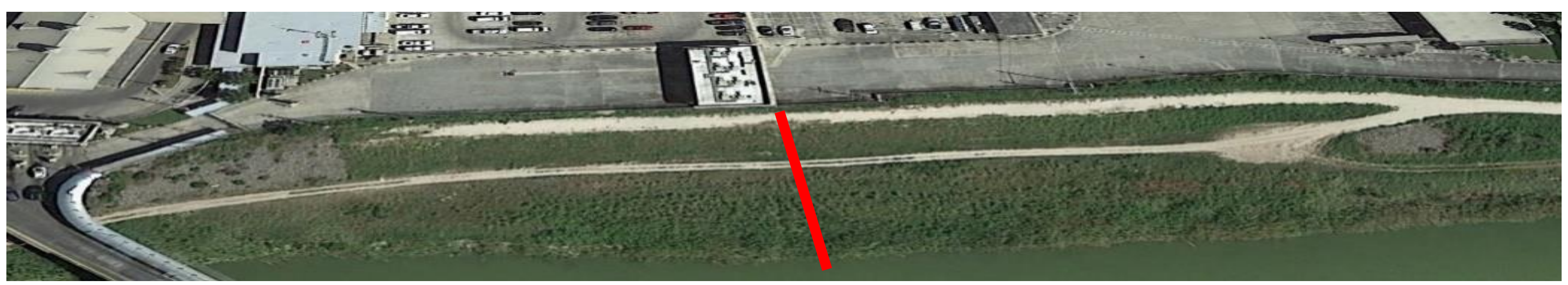

Minimum Factor of Safety (FOS): 1.382

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
DMZ	120	1536.8	0

Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1

IBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GLOBAL DEEP - STONE COLUMNS ASSESSMENT

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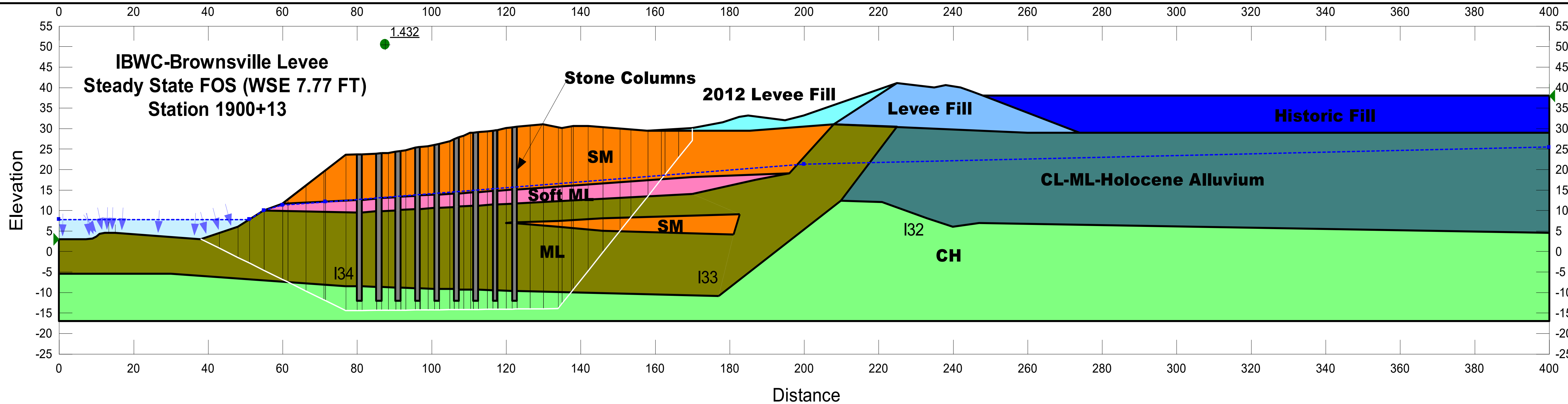

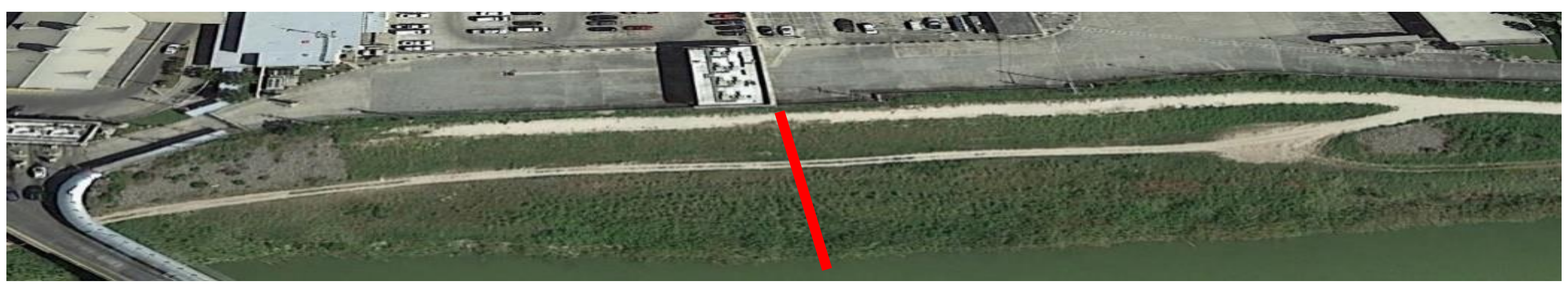

Minimum Factor of Safety (FOS): 1.432

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

IBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GLOBAL DEEP - STONE COLUMNS ASSESSMENT

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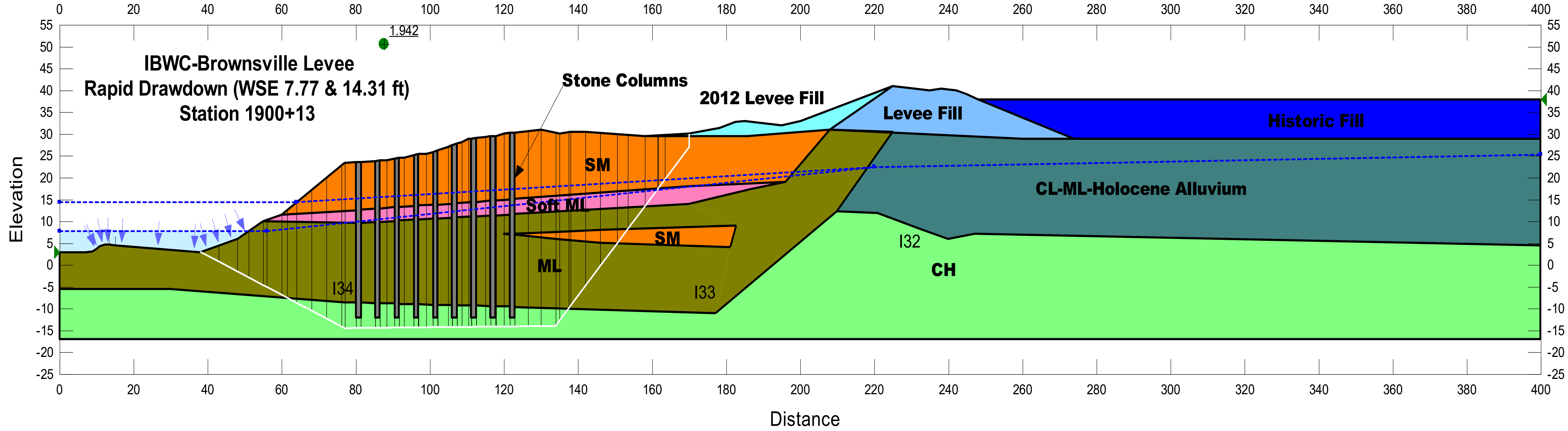

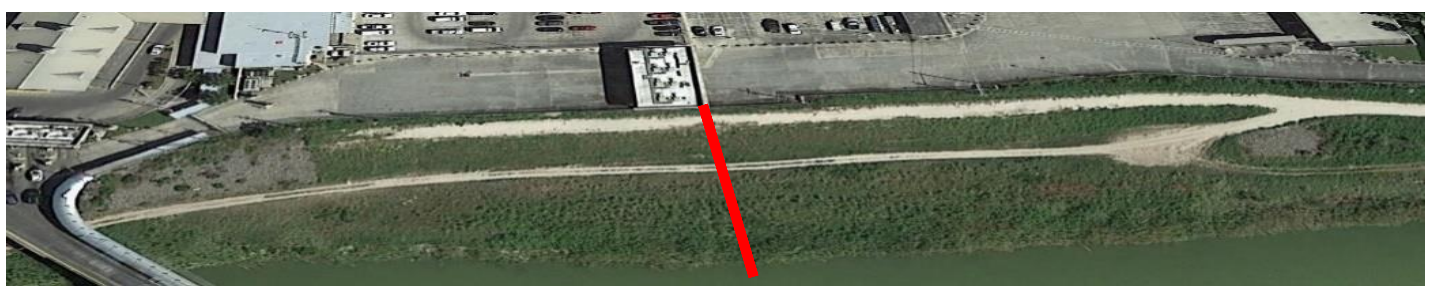

Minimum Factor of Safety (FOS): 1.942

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
DMZ	120	1536.8	0	5011	0

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

IBWC
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REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

GLOBAL DEEP - STONE COLUMNS ASSESSMENT

RAPID DRAWDOWN

ARCADIS

APPENDIX

APPENDIX L

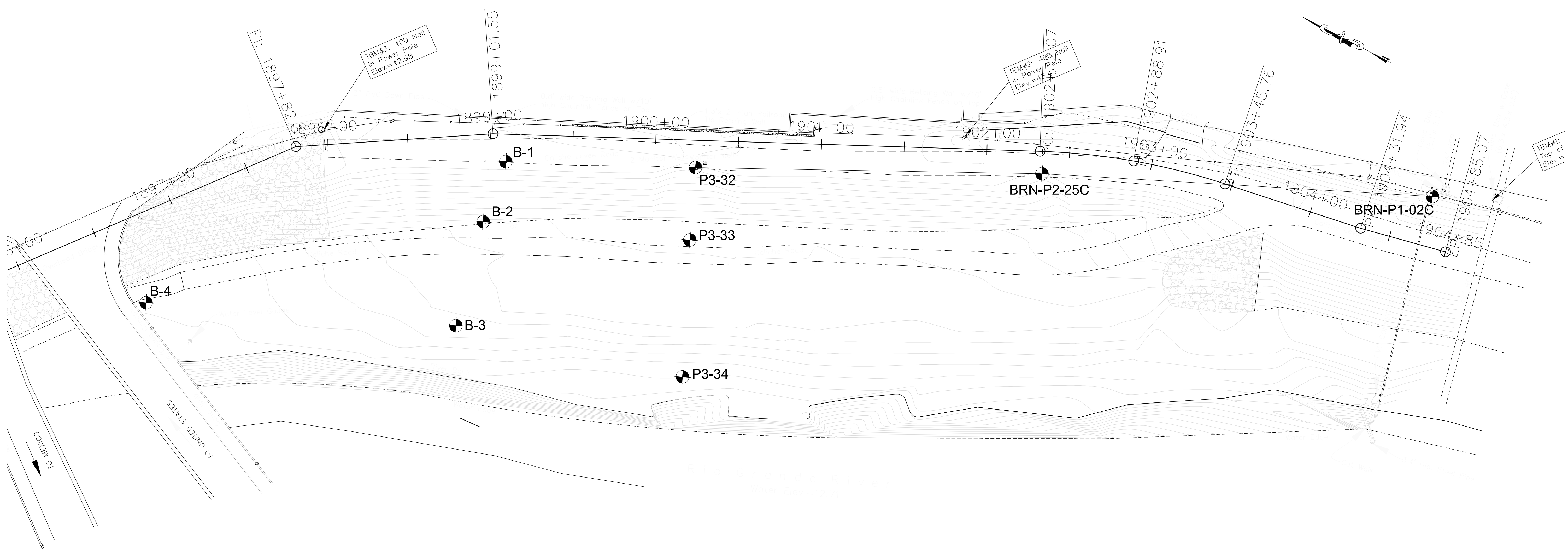
Armoring Detailed Cost

Removed

APPENDIX C

Boring Logs

Arcadis Boring Logs (2016)



Note:

Inclinometers ARC-1 through ARC-4 correspond to Arcadis' borings B-1 through B-4 and
Inclinometers I-32 through I-34 correspond to USACE's borings P3-32 through P3-34

0 60' 120'

USIBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

BORING LOCATION MAP

ARCADIS

FIGURE
1

CITY:(Reqd) DIV/GROUP:(Reqd) DB:(Reqd) LD:(Opt) PIC:(Opt) PM:(Reqd) TM:(Opt) LYR:(Opt)ON=*;OFF=*REF*
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XREFS: PROJECTNAME: ----
B-SIZE-H-BORDER-FEET
EX-TOPO-CROSS SECTION2

ARCADIS

BORING NUMBER B-1

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 40.765 ft NAVD88 **Total Depth** 100 ft.
Driller Freddy Quintaro
Northing 16489693.484 ft **Easting** 1314115.427 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/16/16 **Completed** 6/16/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 24.5 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	%<#200%	PI (%)
40	0	Loose, dark brown, calcareous, caliche base material	SS	1	0-1.5	11	4	5			9		
		Stiff, dark brown, lean CLAY (CL)											
35	5		SS	2	3.5-5	3	4	5			9	93	23
		-with some gravel to 10'	ST	3	8-10				10	2.25			
30	10												
25	15		ST	4	13-15				13	4			20
			ST	5	18-20				19	2			
20	20	Stiff to very stiff, dark brown, fat CLAY, moist, organic odor (CH)	ST	6	20-22				15	2		98	32
		-dark gray below 22'	ST	7	22-24				18	2.25			
15	25	▽ Soft to medium stiff, dark gray, lean CLAY with sand, moist (CL) -stiff below 26'	ST	8	24-26				19	0.5		77	16
			ST	9	26-28				9	2			
	30	-dark brown below 28'	ST	10	28-30				8	2			

Moisture Content (%): 15 30 45

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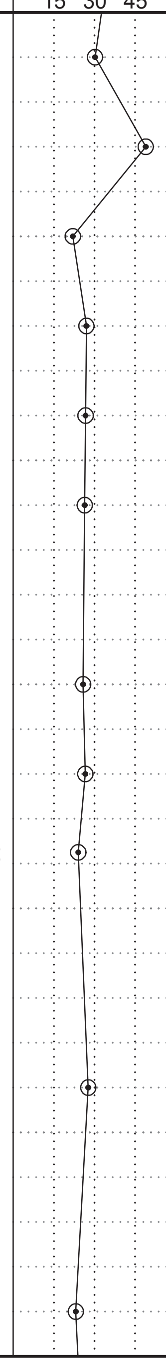
ARCADIS

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 40.765 ft NAVD88 **Total Depth** 100 ft.
Driller Freddy Quintaro
Northing 16489693.484 ft **Easting** 1314115.427 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/16/16 **Completed** 6/16/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 24.5 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Graphic Log	Material Description, Notes	Sample			Soil (Blows/6in.)				Hand Penetrometer (tsf)	N Blows/ft	Moisture Content (%)	%<#200%	PI (%)
				Type	Number	Depth (ft)	0/6	6/12	12/18	Rec. (in.)			15 30 45		
10	30		Stiff, brown, fat CLAY, moist (CH)	ST	11	30-32				8	1.5				33
				ST	12	32-34				22	3			94	50
5	35			ST	13	34-36				23	2			95	31
				ST	14	36-38				22	3.25				
	40		Medium stiff, brown, lean CLAY, moist with fine sand (CL)	ST	15	38-40				22	1.75				
0			Stiff, brown, fat CLAY, moist with fine sand (CH)	ST	16	40-42					4				
			Stiff, brown, lean CLAY, moist (CL)												
-5	45			ST	17	44-46					2			99	29
				ST	18	46-48				23	3.5				
			-very stiff, 48' - 57'	SS	19	48-49.5	10	16	16			32			
-10	50														
				ST	20	53-55				23	4				
-15	55														
	60		-stiff below 57'	ST	21	58-60				21	2			99	15



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ARCADIS

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 40.765 ft NAVD88 **Total Depth** 100 ft.
Driller Freddy Quintaro
Northing 16489693.484 ft **Easting** 1314115.427 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/16/16 **Completed** 6/16/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 24.5 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Graphic Log	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	Moisture Content (%) 15 30 45	%<#200%	PI (%)
-20	60		Stiff, brown, lean CLAY, moist (CL)												
			Very stiff, brown, fat CLAY, moist (CH)												
-25	65														
	70			SS	22	68.5-70	5	10	9			19		96	40
-30			Very stiff, brown, sandy lean CLAY, moist (CL)												
-35	75			SS	23	73.5-75	15	16	11			27		59	11
	80		-stiff, 77' - 82'	SS	24	78.5-80	6	7	8			15			
-40															
-45	85			SS	25	83.5-85	6	7	13			20		84	10
	90		-brownish gray below 87'	SS	26	88.5-90	6	9	11			20			

(Continued Next Page)

ARCADIS

BORING NUMBER B-1

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 40.765 ft NAVD88 **Total Depth** 100 ft.
Driller Freddy Quintaro
Northing 16489693.484 ft **Easting** 1314115.427 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/16/16 **Completed** 6/16/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 24.5 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Graphic Log	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	Moisture Content (%) 15 30 45	%<#200%	PI (%)
-50	90		Very stiff, brownish gray, sandy lean CLAY, moist (CL)												
			Hard, brownish gray, Silty CLAY with sand, moist (CL-ML)												
	95			SS	27	93.5-95	7	18	32			50		73	6
-55															
	100			SS	28	98.5-100	14	21	29			50			

Bottom of borehole at 100.0 feet.

Notes:
SS=Split Spoon
ST=Shelby Tube

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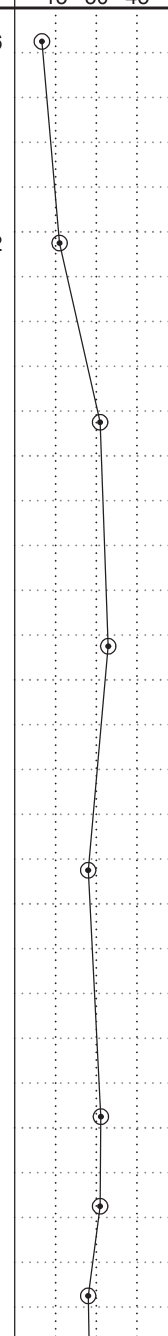
ARCADIS

BORING NUMBER B-2

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 29.954 ft NAVD88 **Total Depth** 80 ft.
Driller Freddy Quintaro
Northing 16489682.064 ft **Easting** 1314074.36 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/13/16 **Completed** 6/13/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 12.5 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Graphic Log	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	Moisture Content (%) 15 30 45	%<#200%	PI (%)
	0		Stiff to very stiff, brown, Silty CLAY, calcareous, moist, with organics (CL-ML)	SS	1	0-1.5	7	6	10			16			
			Medium dense, brown, Sandy SILT, calcareous, with caliche, moist (ML)												
25	5			SS	2	4.5-6	6	6	6			12			
			Soft to medium stiff, brown, lean CLAY, wet, with fine sand (CL)												
20	10			SS	3	8.5-10	1	1	1			2		88	10
			▽												
15	15			SS	4	13.5-15	1	1	2			3			
10	20		-dark gray, sandy and saturated below 18'	SS	5	18.5-20	1	1	1			2		62	
5	25			SS	6	24-25.5	1	2	1			3		54	16
				SS	7	26-27.5	1	1	1	2		2			
				SS	8	28-29.5	1	1	1			2		78	17
0	30														



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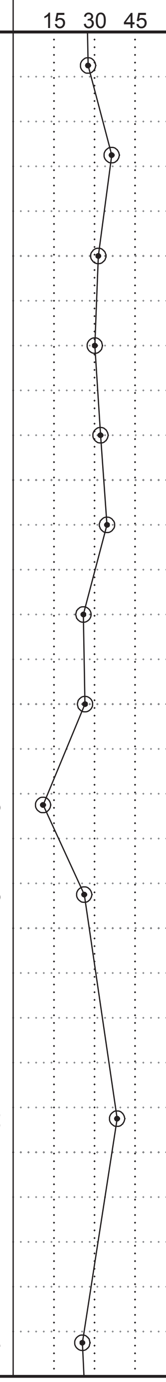
CLIENT	United States International Boundary and Water Commission	**PROJECT NAME**	Brownsville Upper Levee Segment
PROJECT NUMBER	LA0003315.0000	**PROJECT LOCATION**	Brownsville, TX
Contractor	PSI/JEDI Drilling	**Ground Elevation**	29.954 ft NAVD88 **Total Depth** 80 ft.
Driller	Freddy Quintaro	**Northing**	16489682.064 ft **Easting** 1314074.36 ft
Method	Hollow Stem Auger / Mud Rotary	**Date Started**	6/13/16 **Completed** 6/13/16
Rig Type	Truck Mounted Rig CME 75 with Automatic Hammer	**Groundwater level**	12.5 ft bgs
Inspector	Armando Flores (Arcadis)/Rockford Miller (PSI)		

Elevation (ft)	Depth (ft)	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	%<#200%	PI (%)
	30	Soft to medium stiff, dark gray, lean CLAY with sand, saturated (CL)	SS	9	30-31.5	1	2	1			3		
		Medium stiff, dark gray, fat CLAY, moist (CH)	SS	10	32-33.5	1	1	1			2		28
-5	35	-stiff below 34'	ST	11	34-36				19	3		97	41
			ST	12	36-38				18	2.75		99	45
		Stiff, dark gray, lean CLAY with sand (CL)	ST	13	38-40							80	23
-10	40	Very stiff, dark gray, fat CLAY (CH)	ST	14	40-42					3.75		94	39
			ST	15	42-44					4		99	41
-15	45		ST	16	44-46				24	4			
		Stiff, brown, lean CLAY with silt (CL)	SS	17	46.5-48	5	6	9			15		
-20	50		SS	18	48.5-50	3	6	9			15	100	14
-25	55		SS	19	53.5-55	5	6	6			12		
-30	60		SS	20	58.5-60	4	6	8			14		

Moisture Content (%)



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ARCADIS

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 29.954 ft NAVD88 **Total Depth** 80 ft.
Driller Freddy Quintaro
Northing 16489682.064 ft **Easting** 1314074.36 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/13/16 **Completed** 6/13/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 12.5 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Graphic Log	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	Moisture Content (%) 15 30 45	%<#200%	PI (%)
	60		Stiff, brown, lean CLAY with silt (CL)												
-35	65			SS	21	63.5-65	5	5	7			12		99	26
-40	70			SS	22	68.5-70	5	5	7			12			
-45	75			ST	23	73-75				22	3.25			99	23
-50	80		-with sand seam at 79'	ST	24	78-80				26	3.50				

Bottom of borehole at 80.0 feet.

Notes:
SS=Split Spoon
ST=Shelby Tube

The borehole cave in at 13 feet below ground surface. More sand was encountered at that depth.

BORING NUMBER B-3

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 28.024 ft NAVD88 **Total Depth** 80 ft.
Driller Freddy Quintaro
Northing 16489662.802 ft **Easting** 1314010.777 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/15/16 **Completed** 6/15/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 17.2 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Graphic Log	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	Moisture Content (%) 15 30 45	%<#200%	PI (%)
	0		Loose, brown, sandy silt, moist, with organics (ML)	SS	1	0-1.5	3	3	3	10		6			
25	5		-very dense below 3.5' with limestone and caliche	SS	2	3.5-5	25	25	50			75			
20			Medium stiff, brown, sandy lean CLAY, moist (CL)												
	10			SS	3	8.5-10	2	2	3	22		5		76	13
15	15		-brownish gray, 13' - 17'	SS	4	13.5-15	2	2	2			4			
10	20		-gray with organics and some medium to coarse sand below 17'	SS	5	18.5-20	2	2	3			5		75	13
				ST	6	20-22					0.25				
5				SS	7	22-23.5	3	3	2			5		67	
	25		Medium stiff, gray, fat CLAY, moist (CH)	SS	8	24-25.5	2	2	2			4	>>		33
			-with medium to coarse sand to 25'												
0			-with organics, 25' - 28'	SS	9	26.5-28	2	4	3	18		7			
	30		-stiff with fine sand below 28'	SS	10	28.5-30	5	5	8	18		13		85	

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ARCADIS

BORING NUMBER B-3

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 28.024 ft NAVD88 **Total Depth** 80 ft.
Driller Freddy Quintaro
Northing 16489662.802 ft **Easting** 1314010.777 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/15/16 **Completed** 6/15/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 17.2 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Graphic Log	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	Moisture Content (%) 15 30 45	%<#200%	PI (%)
	30		Stiff, dark gray, fat CLAY (CH) -with organics, 30' - 31'	ST	11	30-32								98	32
-5			-with 6" sandy silt layer at 33.5'	ST	12	32-34								99	31
	35		Stiff, brown, lean CLAY, moist (CL)												
				ST	13	36-38								99	27
-10				ST	14	38-40				21	3.5				
	40		Very stiff, brown, fat CLAY, moist (CH)	ST	15	40-42				20	3.5			100	46
-15				ST	16	42-44				12	3.5				
	45		Stiff, brown, lean CLAY (CL)	ST	17	44-46				23	2.25				21
-20			Medium dense, brown, SILT (ML)	ST	18	48-50				22	0.5			100	
	50		Very stiff, brown, fat clay, moist (CH)												
-25				SS	19	53.5-55	12	14	15			29			30
	55														
-30			Dense, brown, Clayey SAND (SC)	ST	20	58-60				22	1.5			33	
	60														

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ARCADIS

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 28.024 ft NAVD88 **Total Depth** 80 ft.
Driller Freddy Quintaro
Northing 16489662.802 ft **Easting** 1314010.777 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/15/16 **Completed** 6/15/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 17.2 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Graphic Log	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	Moisture Content (%) 15 30 45	%<#200%	PI (%)
	60		Dense, brown, Clayey SAND (SC)												
-35			Very stiff, brownish gray, fat CLAY (CH)												
	65			SS	21	63.5-65	10	12	14	18		26			
-40	70			SS	22	68.5-70	7	10	12	18		22		99	39
-45			Stiff, brown, lean CLAY with silty sand (CL)												
	75			ST	23	73-75				21	2.75				
-50			Medium dense, brown, silty SAND (SM)												
	80			ST	24	78-80				22	0.25				

Bottom of borehole at 80.0 feet.

Notes:
SS=Split Spoon
ST=Shelby Tube

Initial water level at 20 feet, then 17.2 ft at 10 minutes delay

Loss of drilling fluid at 25 feet. Boring relocated 10 ft towards the river in the same alignment from the previous boring.

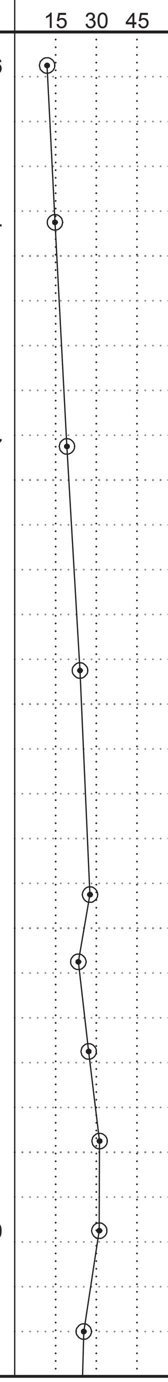
Shelby tube was lost after taking the sample at 38 ft. Second borehole relocated 3 ft to 5 ft towards the levee.

ARCADIS

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 25.92 ft NAVD88 **Total Depth** 80 ft.
Driller Freddy Quintaro
Northing 16489838.668 ft **Easting** 1313942.743 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/17/16 **Completed** 6/17/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 23 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	%<#200%	PI (%)
25	0	Stiff to very stiff, dark brown, silty CLAY, moist, with organics (CL-ML)	SS	1	0-1.5	3	7	9			16		
		Stiff to very stiff, dark brown, lean CLAY, moist (CL)											
	5		SS	2	3.5-5	5	6	8			14		18
20													
	10	-becoming sandy lean clay, 8' - 26'	SS	3	8.5-10	9	8	9			17	75	
15													
	15		SS	4	13.5-15	4	4	3			7		
10													
	20		SS	5	18.5-20	2	3	4			7	64	8
5			SS	6	20-21.5	3	3	4			7		
		∇ -medium stiff, 22' - 24'	SS	7	22-23.5	2	2	2			4		
	25	-soft, 24' - 26'	SS	8	24-25.5	1	1	1			2		
0		-with organics to 26'	SS	9	26-27.5	4	4	6			10	99	23
	30		ST	10	28-30					2			

Moisture Content (%)



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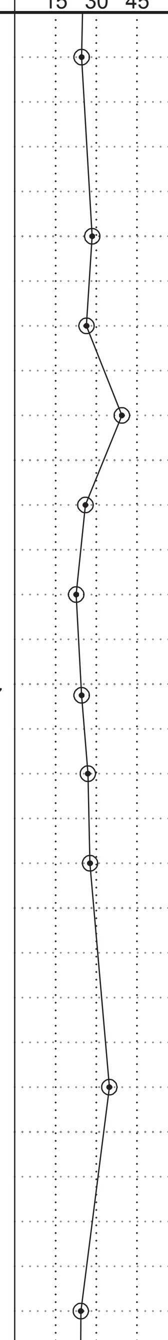
ARCADIS

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 25.92 ft NAVD88 Total Depth 80 ft.
Driller Freddy Quintaro
Northing 16489838.668 ft Easting 1313942.743 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/17/16 Completed 6/17/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 23 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

Elevation (ft)	Depth (ft)	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	%<#200%	PI (%)
-5	30	Stiff, brown, lean CLAY, moist (CL)	ST	11	30-32					2.7			
			ST	12	32-34								
	35	Stiff, medium brown, fat CLAY, moist (CH)	ST	13	34-36				18	3.5		98	50
-10		-medium stiff, 36' - 38'	ST	14	36-38				22	3			
			ST	15	38-40				8	2.25			
-15	40		ST	16	40-42					3			
		Stiff, brown, lean CLAY, moist (CL)	ST	17	42-44				22	1		98	31
-20	45	-very stiff, below 44'	SS	18	44.5-46	9	11	16	22		27		
			ST	19	46-48				14	4.5			
		Medium stiff to stiff, brown, fat CLAY, moist (CH)	ST	20	48-50				17	4.5			32
-25	50												
	55		ST	21	53-55				22	3.5			
-30													
	60		ST	22	58-60				4	2.5		99	36

Moisture Content (%)



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BORING NUMBER B-4

CLIENT United States International Boundary and Water Commission
PROJECT NAME Brownsville Upper Levee Segment
PROJECT NUMBER LA0003315.0000
PROJECT LOCATION Brownsville, TX
Contractor PSI/JEDI Drilling
Ground Elevation 25.92 ft NAVD88 **Total Depth** 80 ft.
Driller Freddy Quintaro
Northing 16489838.668 ft **Easting** 1313942.743 ft
Method Hollow Stem Auger / Mud Rotary
Date Started 6/17/16 **Completed** 6/17/16
Rig Type Truck Mounted Rig CME 75 with Automatic Hammer
Groundwater level 23 ft bgs
Inspector Armando Flores (Arcadis)/Rockford Miller (PSI)

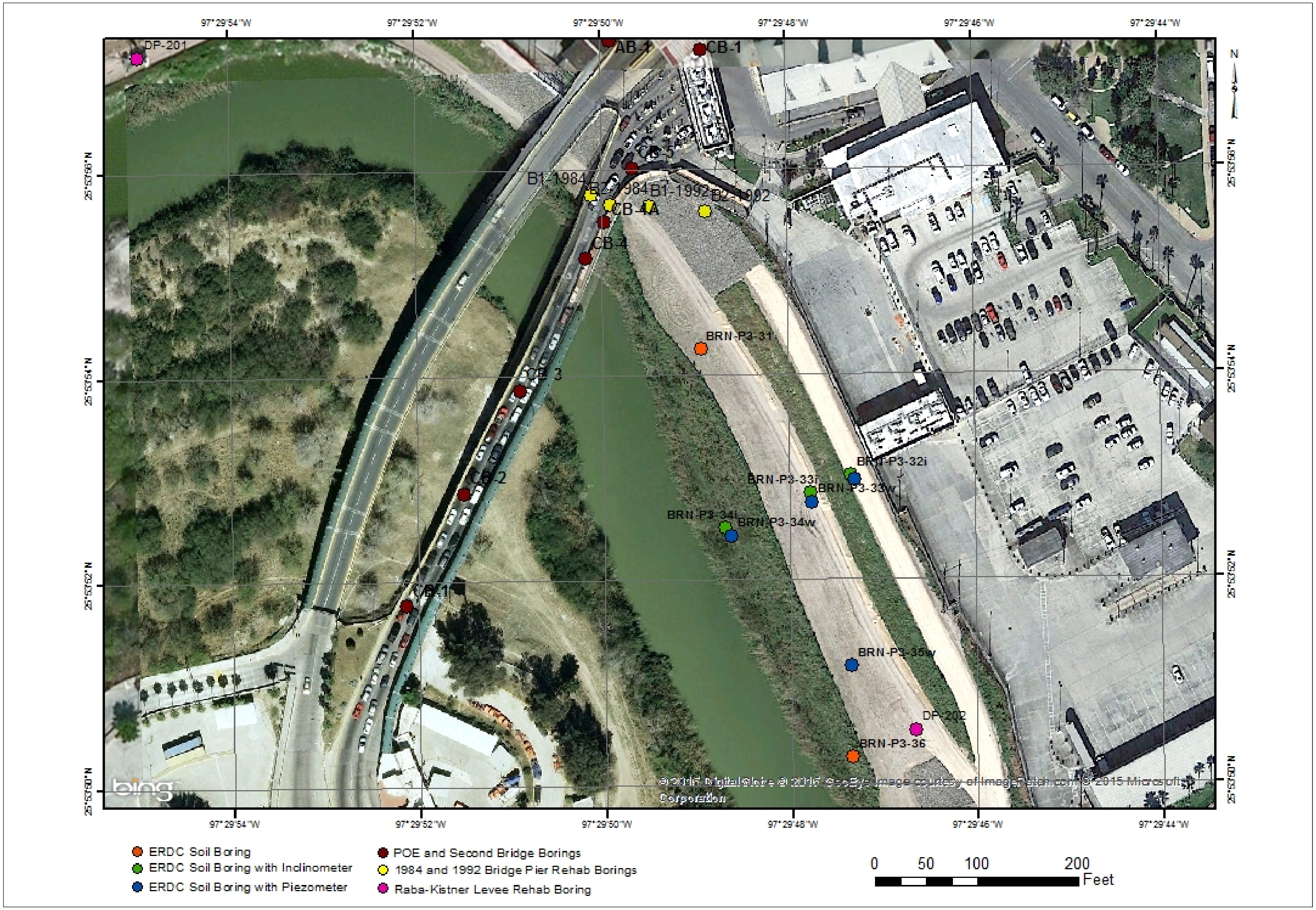
Elevation (ft)	Depth (ft)	Graphic Log	Material Description, Notes	Sample Type	Sample Number	Sample Depth (ft)	Soil (Blows/6in.) 0/6	6/12	12/18	Rec. (in.)	Hand Penetrometer (tsf)	N Blows/ft	Moisture Content (%) 15 30 45	%<#200%	PI (%)
-35	60		Very stiff, brown, fat CLAY, moist (CH)												
				ST	23	63-65				23.5	4.5				
-40	65														
				ST	24	68-70				20	4.5				45
-45	70														
			-stiff below 72'	ST	25	73-75				22	2.75			99	
-50	75														
				ST	26	78-79.5									
	80														

Bottom of borehole at 80.0 feet.

Notes:
SS=Split Spoon
ST=Shelby Tube

USACE Inclinometer Installation Logs (2015)

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.



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 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)*			
			- 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 RECOVERY 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

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) P3-32	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
7. THICKNESS OF OVERBURDEN	16. DATE HOLE: STARTED / COMPLETED
8. DEPTH DRILLED INTO ROCK	17. ELEVATION TOP OF HOLE
9. TOTAL DEPTH OF HOLE 80.0	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 RECOVERY 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 RECOV-ERY 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 RECOVERY 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			- 69.0' to 70.0' light brown silty sand			SPT: 3-7-14
	70					
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
			- 78.0' to 80.0' Clay (CH): tan, dense, stiff			SPT: 8-9-11
	80					
	90					

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
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 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 RECOV-ERY 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 RECOVERY 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			
	50		- 47.7' to 50.3' Sand (SM) at base; visible mica			SPT: 6-6-8
			- 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)*			
	50		- 45.0' to 51.0' Clay (CH): tan, dense, stiff			SPT: 4-4-5 SPT: 3-4-5
			- 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 - Clay (CH)			SPT: 5-9-9
27	60		- 58.5' to 60.0' Clay (CH): tan, dense, stiff			SPT: 7-7-9

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 RECOVERY 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					

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PROJECT	HOLE NO.
IBWC (LAB data included)	P3-35

Hole No. P3-36

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) ELEVATION TOP OF HOLE

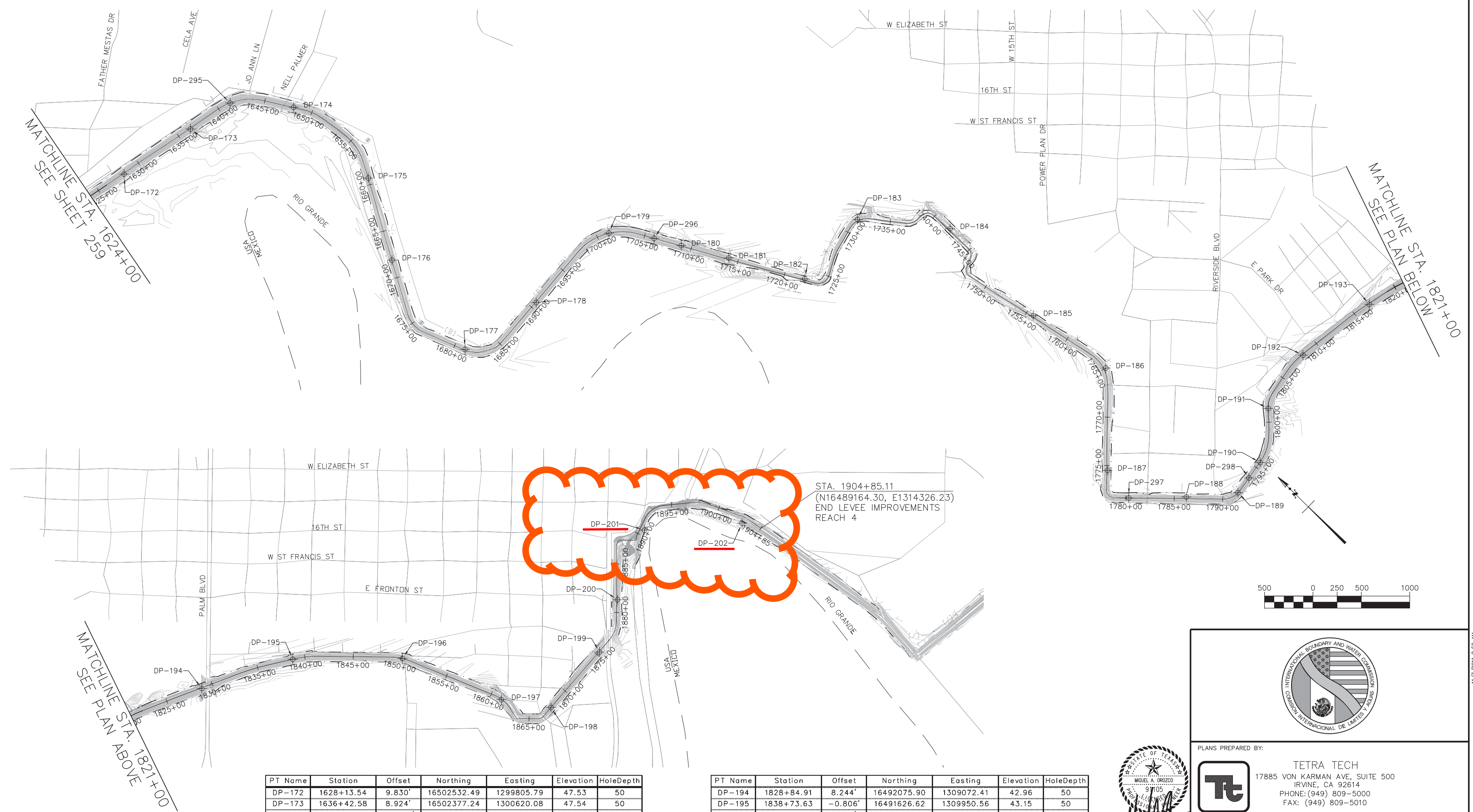
Hole No. P3-36

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

Raba Kistner Boring Logs (2008)



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

PLANS PREPARED BY:

TETRA TECH
17885 VON KARMAN AVE, SUITE 500
IRVINE, CA 92614
PHONE: (949) 809-5000
FAX: (949) 809-5010
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

SHEET 260 OF 299

11/3/2011 8:52 AM

P:\IWATR\127062 DONNA TO BROWNSVILLE\08 DESIGN\SUB-REACH 4\127062C260.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

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
FILL: CLAY, sandy, lean, very stiff, light brown	16		12	
CLAY, fat, stiff to very stiff, brown	14			97
	13			
	22		30	
	12			96
	15			
CLAY, lean, with sand, very stiff, light brown	15			82
- with orange ferrous stains below a depth of about 20 feet	16		12	
CLAY, lean, stiff, grayish-brown	9			96
Driller's Note: Converted to mud rotary drilling method at a depth of about 31-1/2 feet	9		9	
SAND, silty, medium dense, brown	23			15

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 DRILLED: 50.0 ft | DEPTH TO WATER: 30 ft | PROJ. No.: AMA08-115-00
DATE DRILLED: 12/3/2008 | DATE MEASURED: 12/3/2008 | FIGURE: A-202a

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

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

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
SAND, silty, medium dense, brown *(continued)*	25			15
	14			

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.

DEPTH DRILLED: 50.0 ft | DEPTH TO WATER: 30 ft | PROJ. No.: AMA08-115-00
DATE DRILLED: 12/3/2008 | DATE MEASURED: 12/3/2008 | FIGURE: A-202b

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

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

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
FILL: CLAY, fat, with sand, firm to stiff, light brown to brown, with calcareous nodules	6			85
	7		28	
- sand content decreases with depth	9			95
CLAY, sandy, lean, stiff to firm, brown, with calcareous nodules and black ferrous stains	15		13	
	4			
SAND, clayey, medium dense, light brown	13			39
CLAY, fat, stiff to very stiff, brown to light brown, with black ferrous stains	8		32	
	9			89
	10		42	
- 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				
- firm 35 to 37 feet		95		99

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 DRILLED: 50.0 ft | DEPTH TO WATER: 22 ft | PROJ. No.: AMA08-115-00
DATE DRILLED: 12/3/2008 | DATE MEASURED: 12/3/2008 | FIGURE: A-203a

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

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

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
CLAY, fat, stiff to very stiff, brown to light brown, with black ferrous stains *(continued)*				
				97

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.

DEPTH DRILLED: 50.0 ft | DEPTH TO WATER: 22 ft | PROJ. No.: AMA08-115-00
DATE DRILLED: 12/3/2008 | DATE MEASURED: 12/3/2008 | FIGURE: A-203b

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

STATE OF TEXAS
John A. Focht, III
60313
LICENSED
PROFESSIONAL ENGINEER

10/18/2011 9:06 AM

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

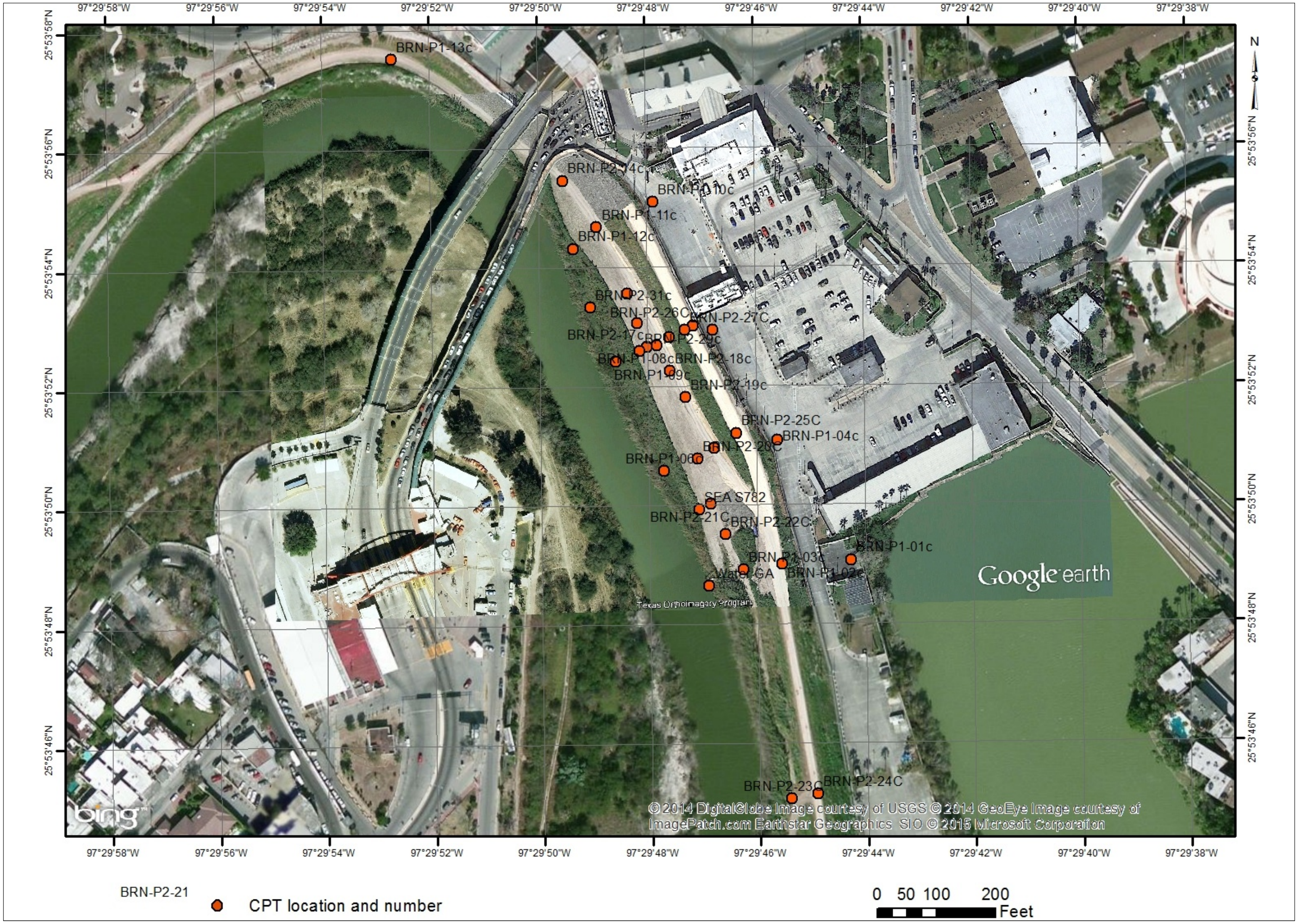
SHEET 283 OF 299

P:\WATR\T27062 DONNA TO BROWNSVILLE\08 DESIGN\SUB-REACH #4\T27062C283.DWG

APPENDIX D

USACE CPT Soundings, Profiles, and Predicted Strength

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

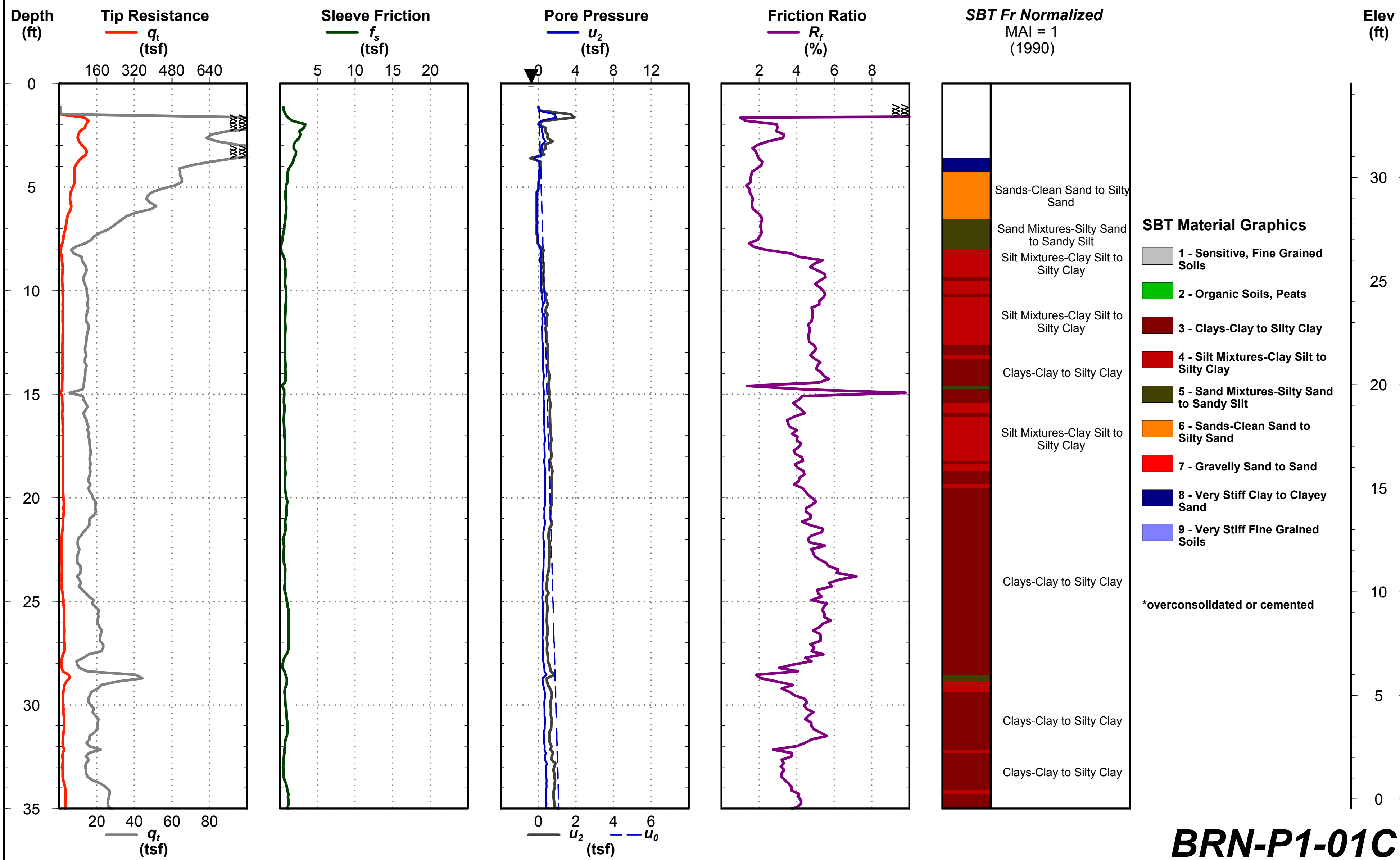


USACE CPT Soundings

Cone Penetration Test BRN-P1-01C

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	**Cone Size:**	



BRN-P1-01C

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

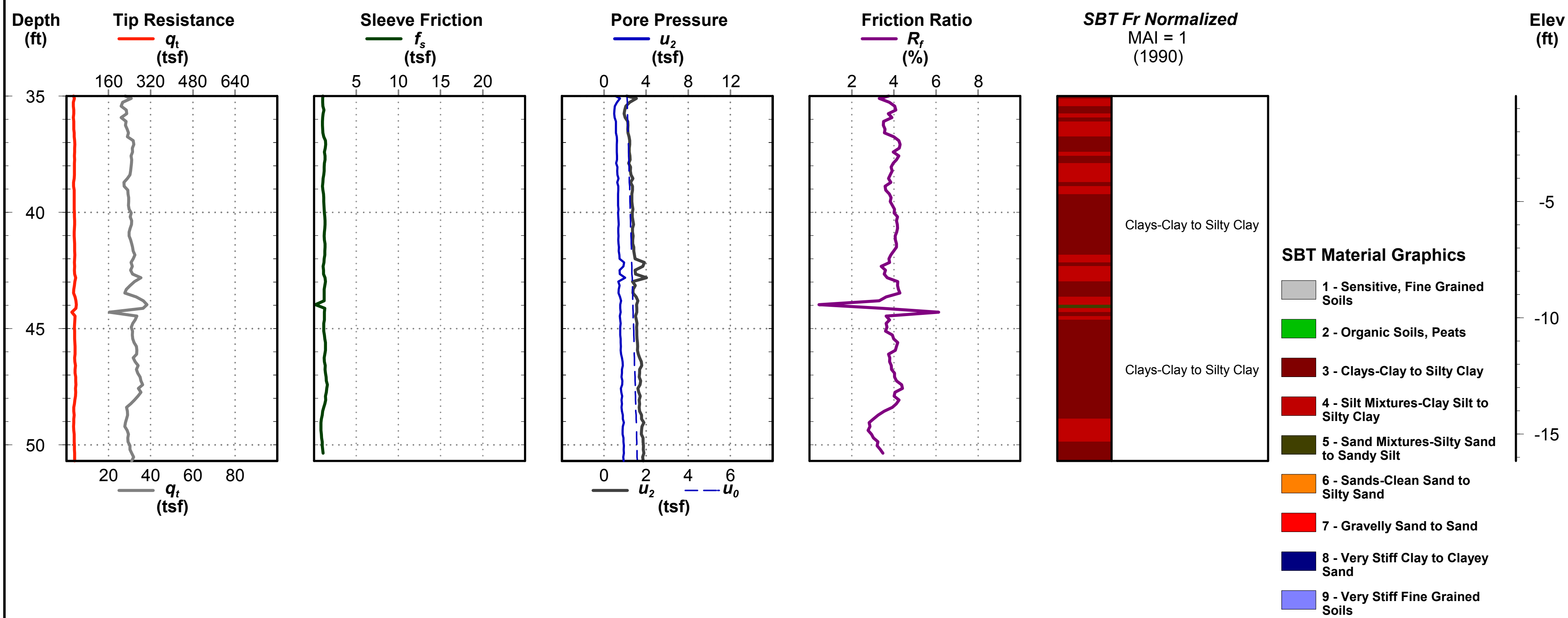
Cone Penetration Test BRN-P1-01C

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:



*overconsolidated or cemented

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

BRN-P1-01C

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

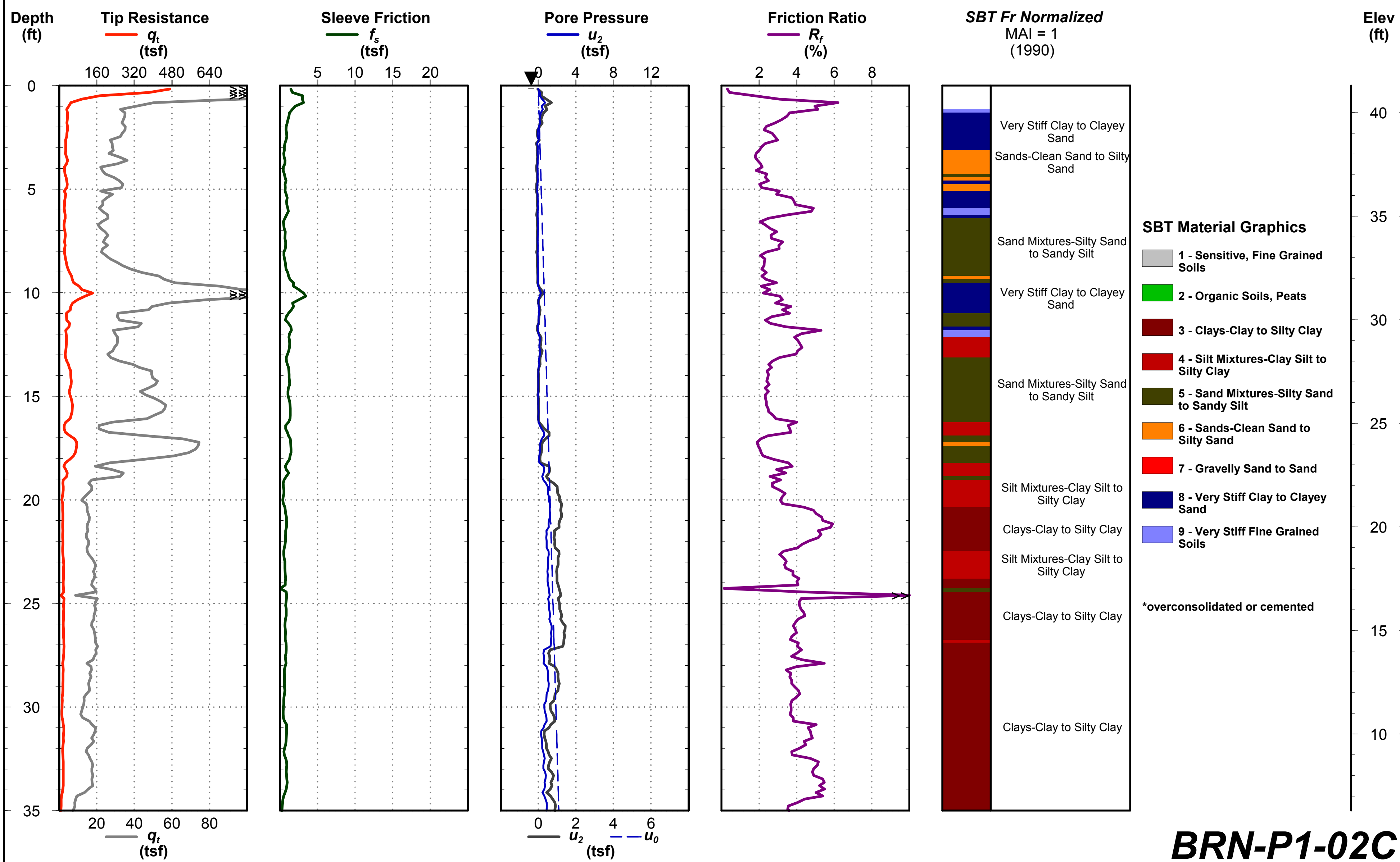
Cone Penetration Test BRN-P1-02C

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:



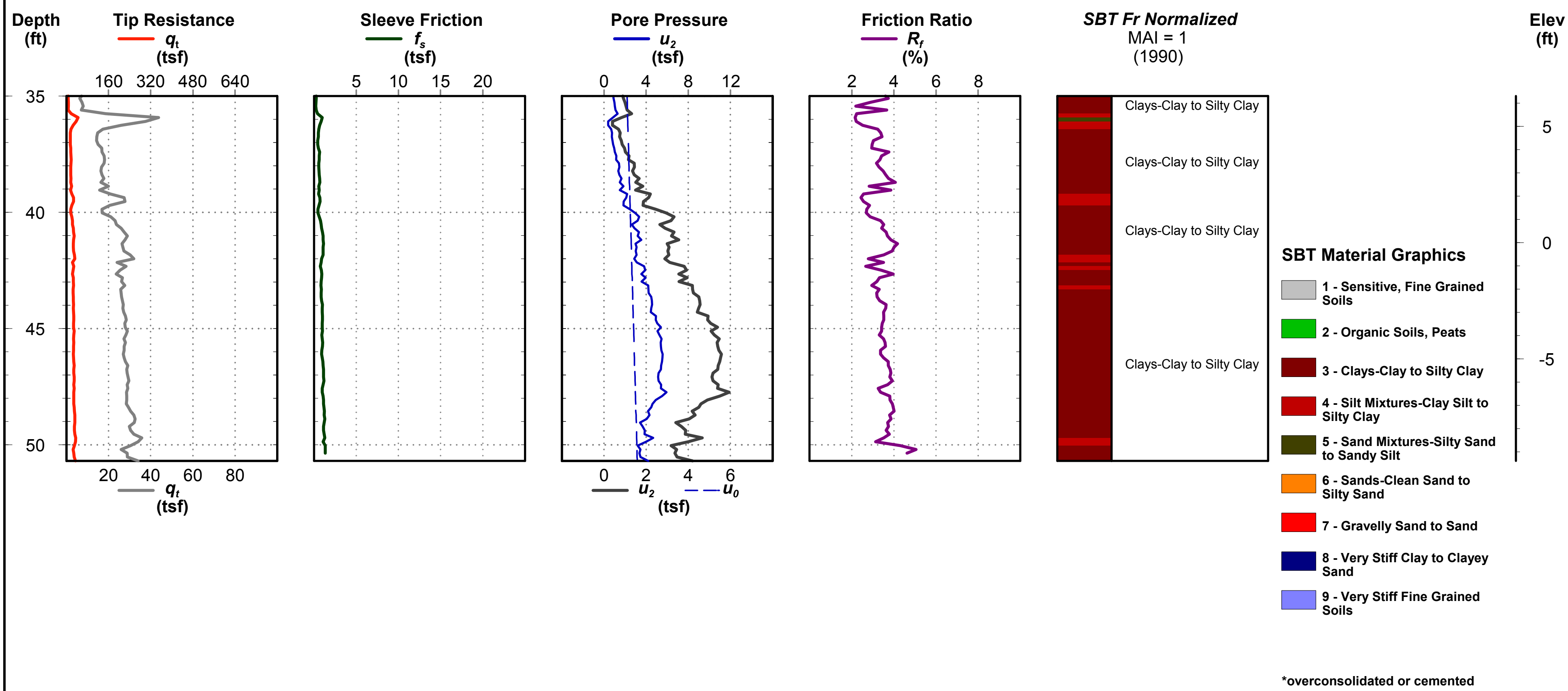
Cone Penetration Test BRN-P1-02C

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:



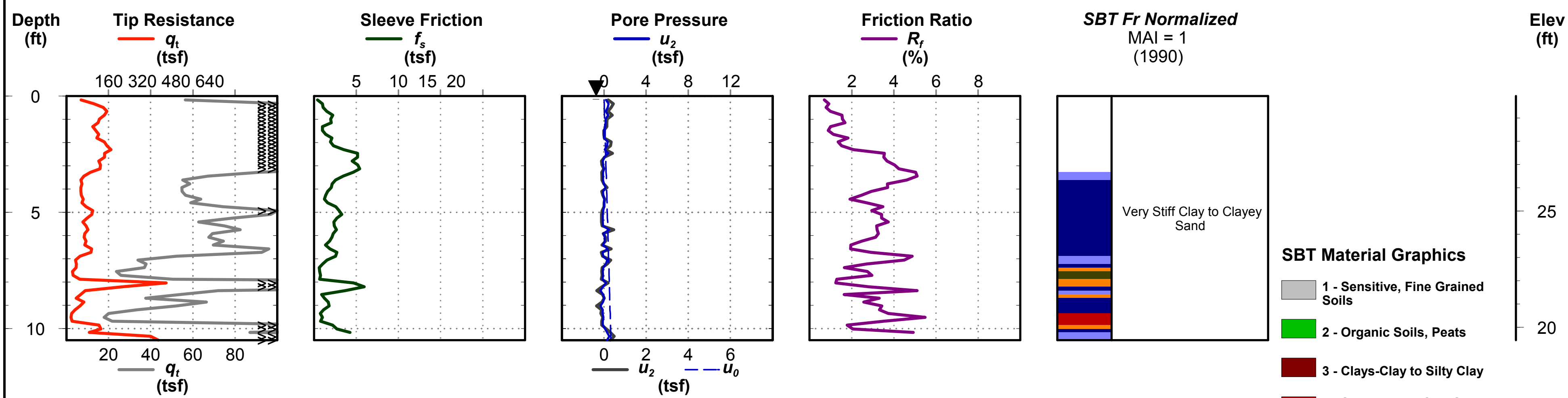
Cone Penetration Test BRN-P1-03C

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

BRN-P1-03C

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

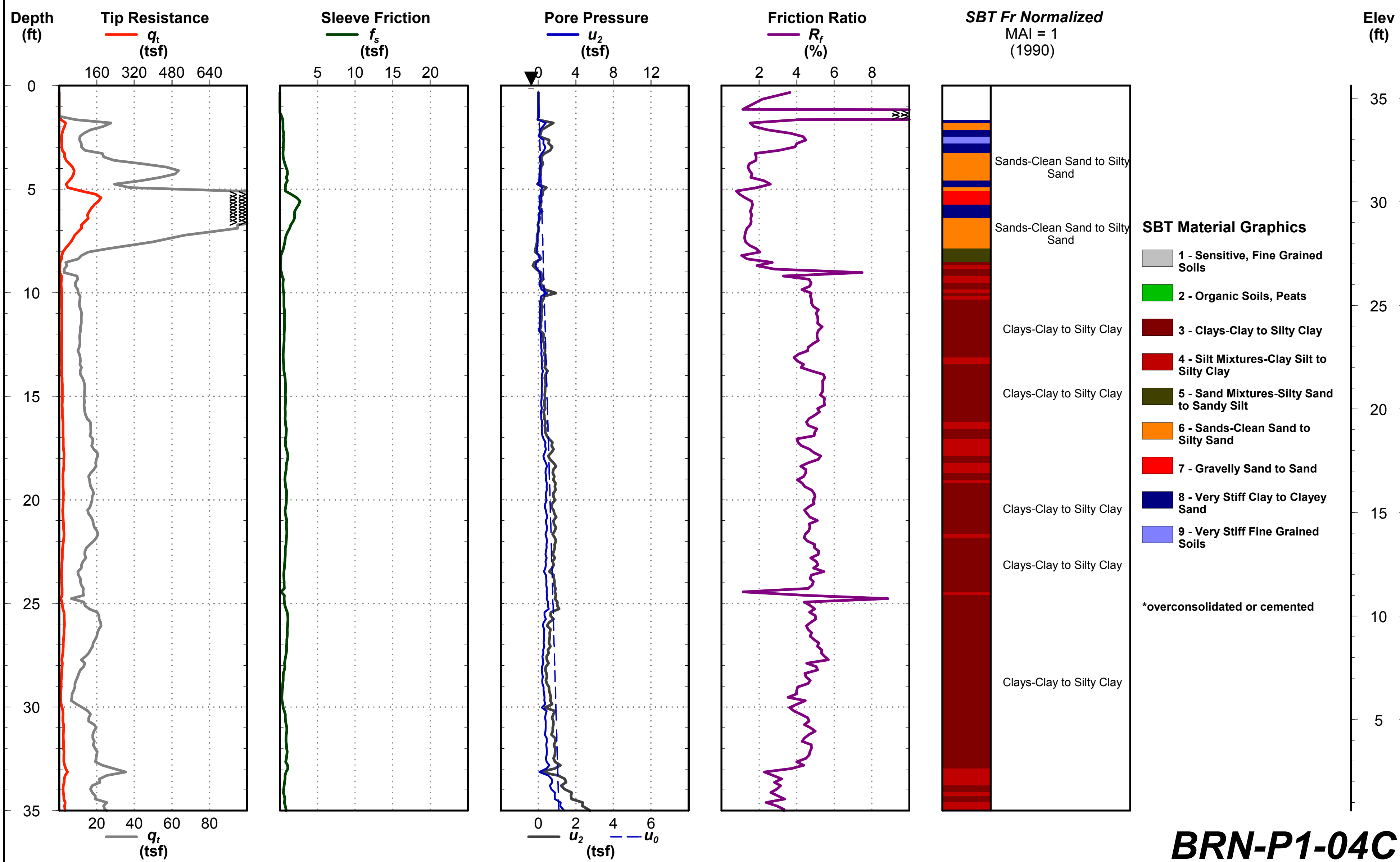
Cone Penetration Test BRN-P1-04C

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:



BRN-P1-04C

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

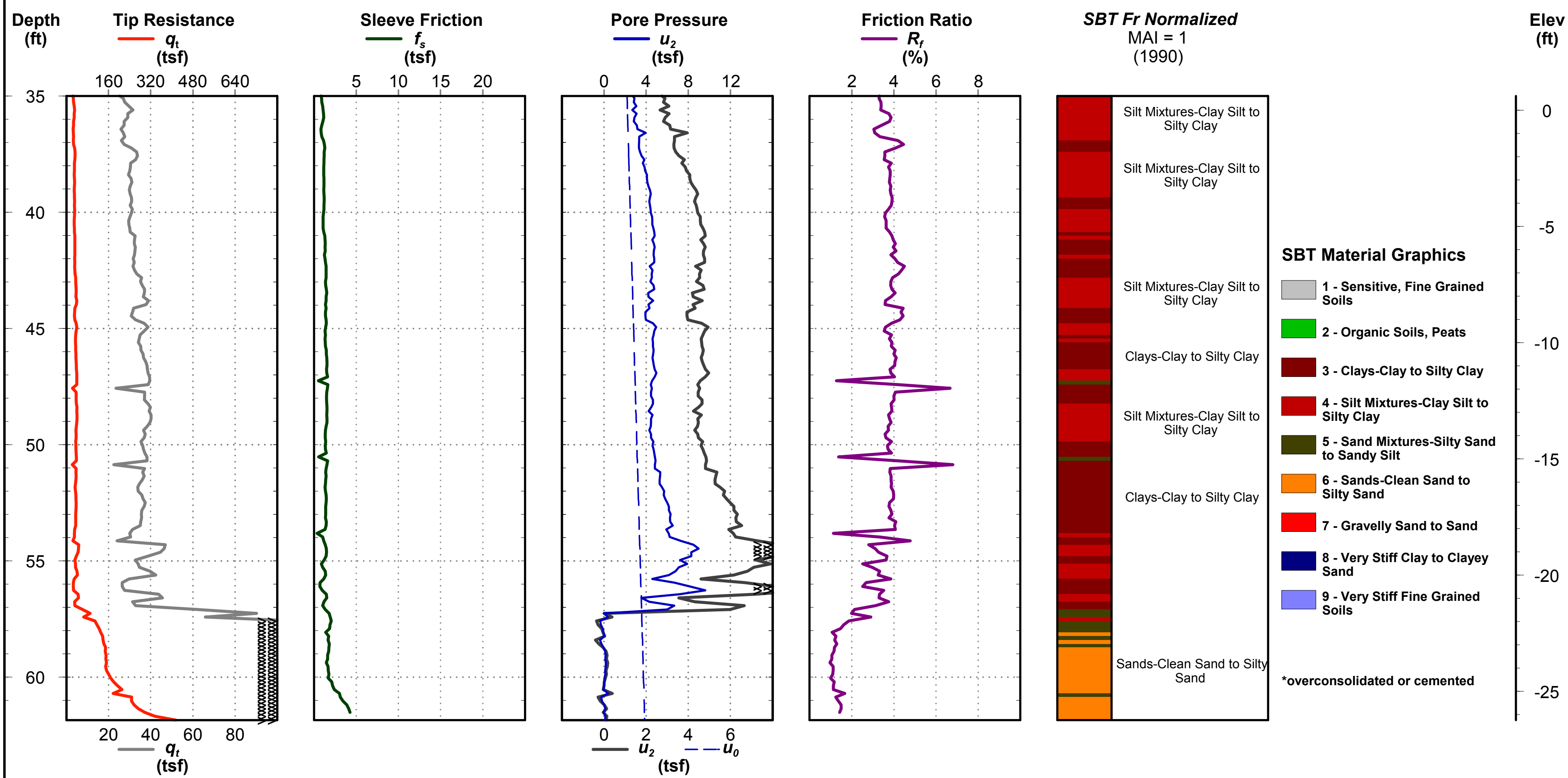
Cone Penetration Test BRN-P1-04C

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:



BRN-P1-04C

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

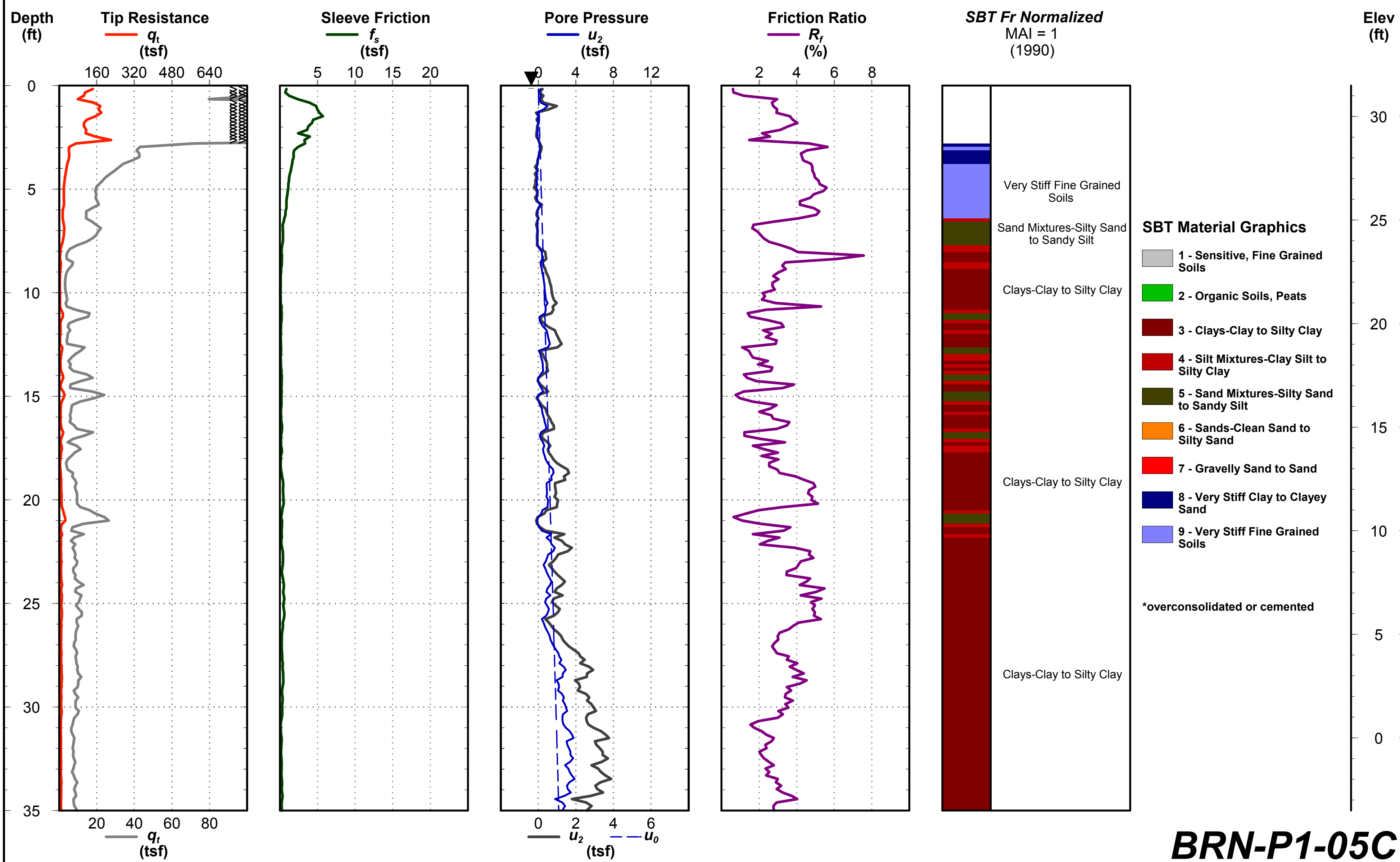
Cone Penetration Test BRN-P1-05C

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:



BRN-P1-05C

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

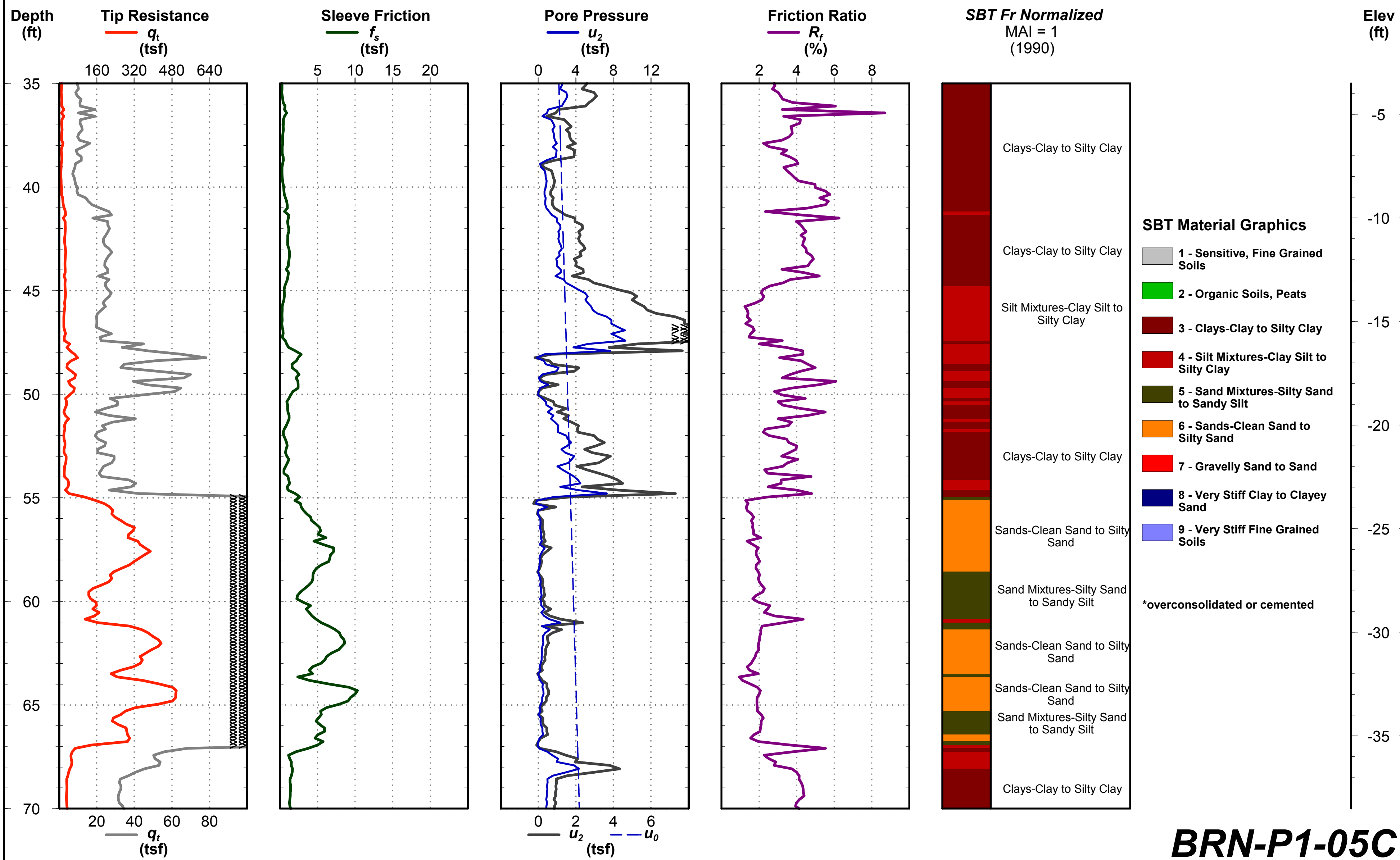
Cone Penetration Test BRN-P1-05C

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

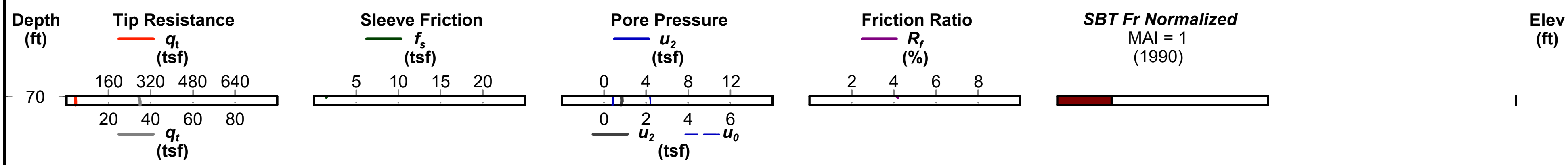
Cone Penetration Test BRN-P1-05C

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

BRN-P1-05C

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

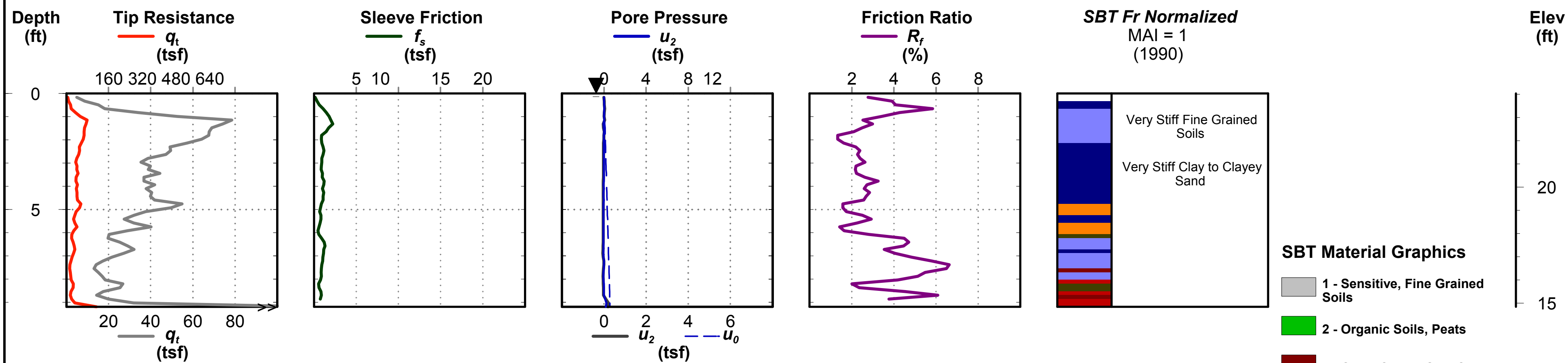
Cone Penetration Test BRN-P1-06C

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

BRN-P1-06C

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

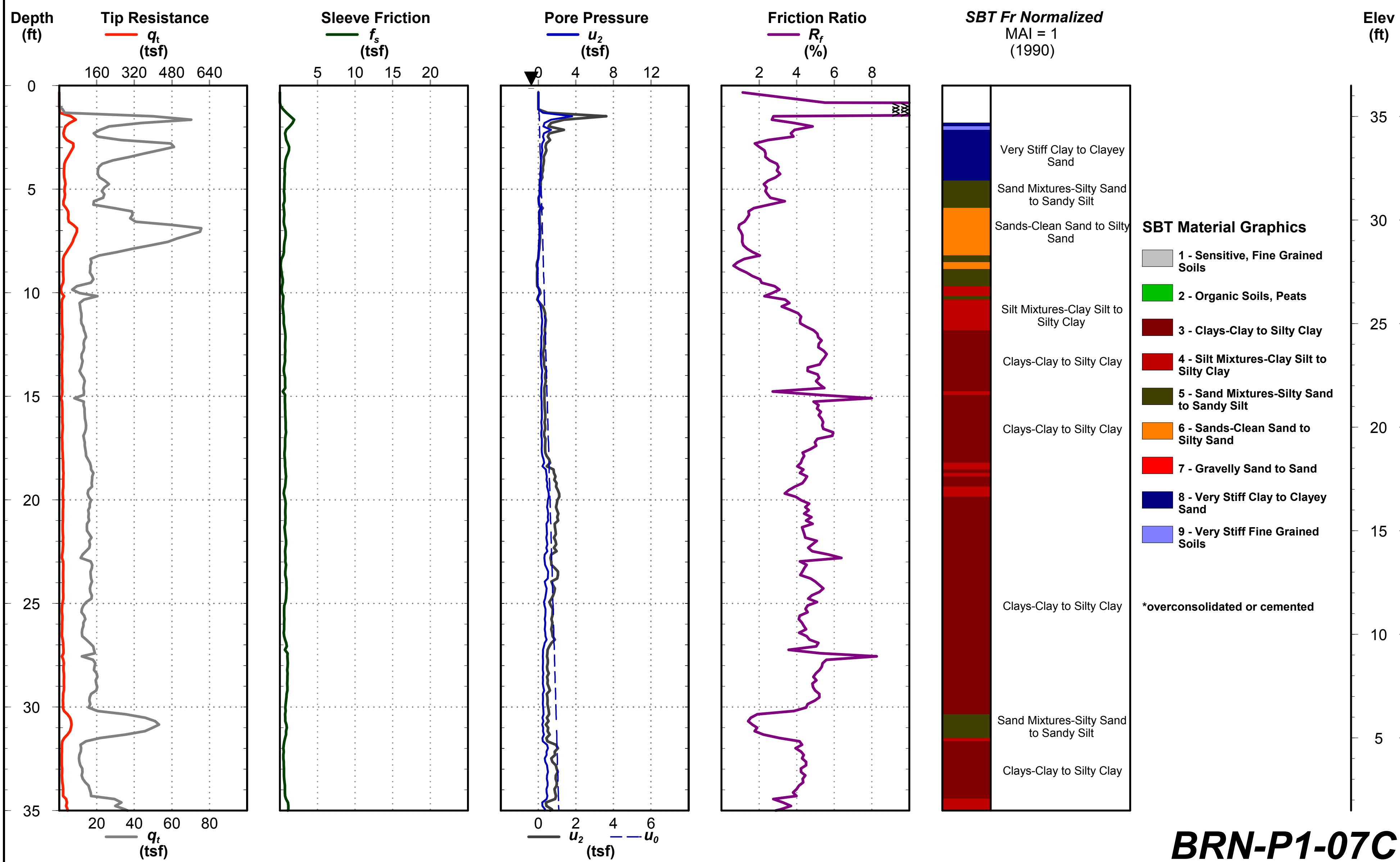
Cone Penetration Test BRN-P1-07C

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

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

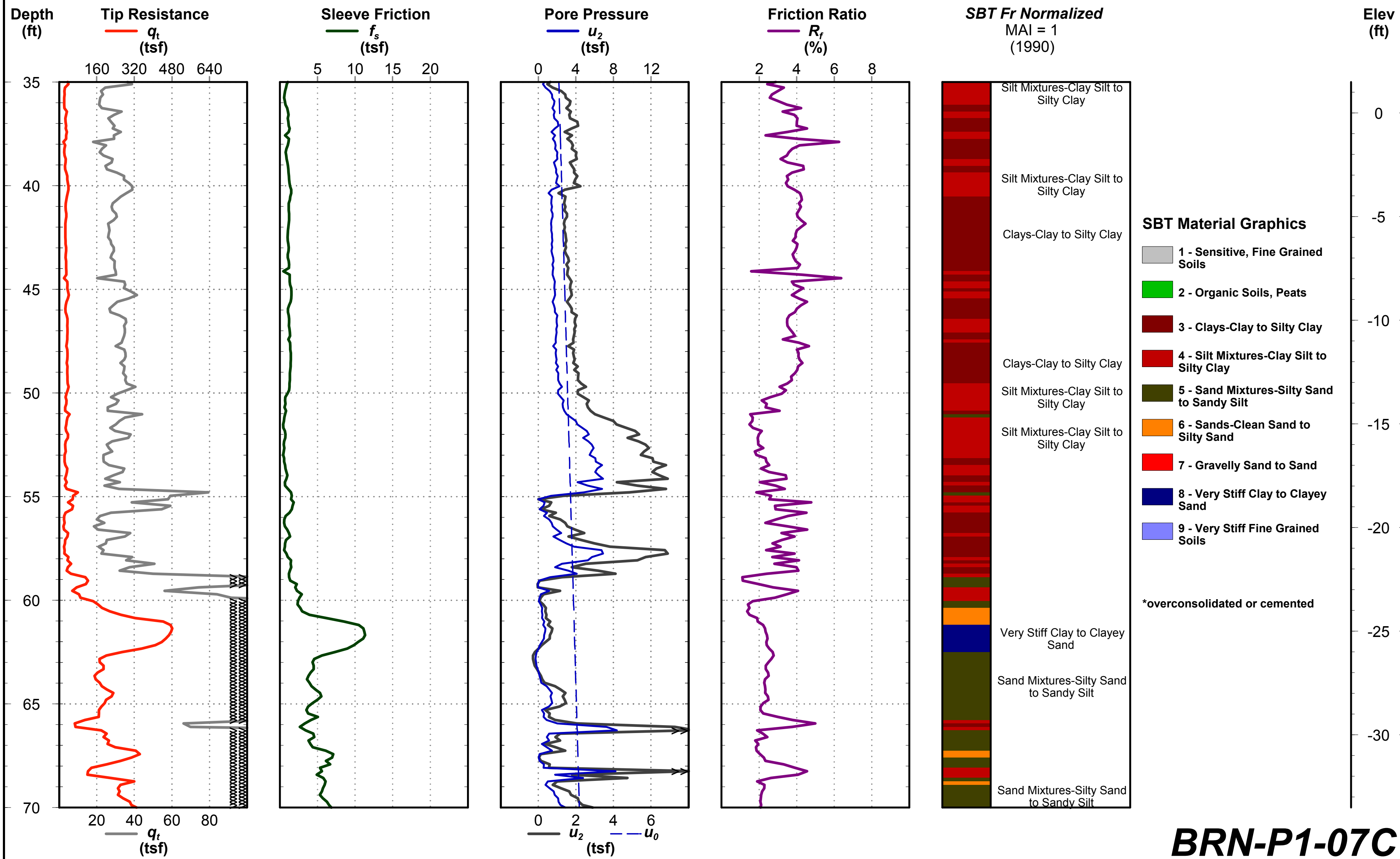
Brownsville, Tx
Project Number :IBWC

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:



BRN-P1-07C

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

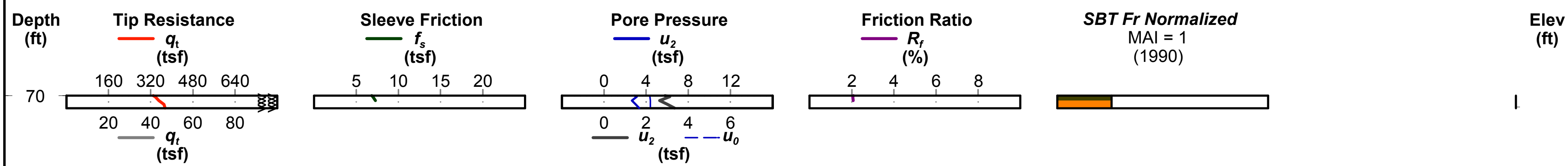
Brownsville, Tx
Project Number :IBWC

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:



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

BRN-P1-07C

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

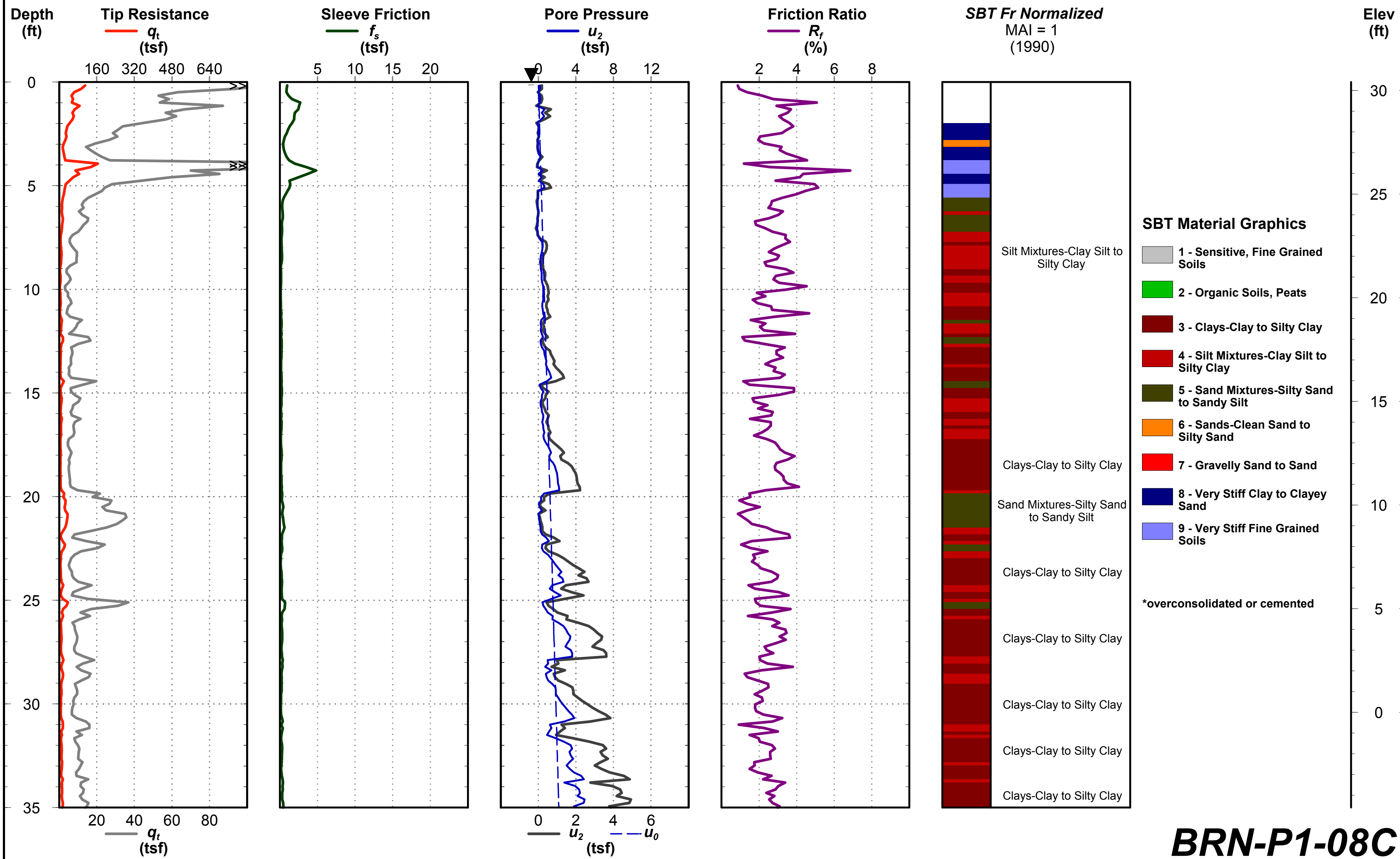
Cone Penetration Test BRN-P1-08C

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:



BRN-P1-08C

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

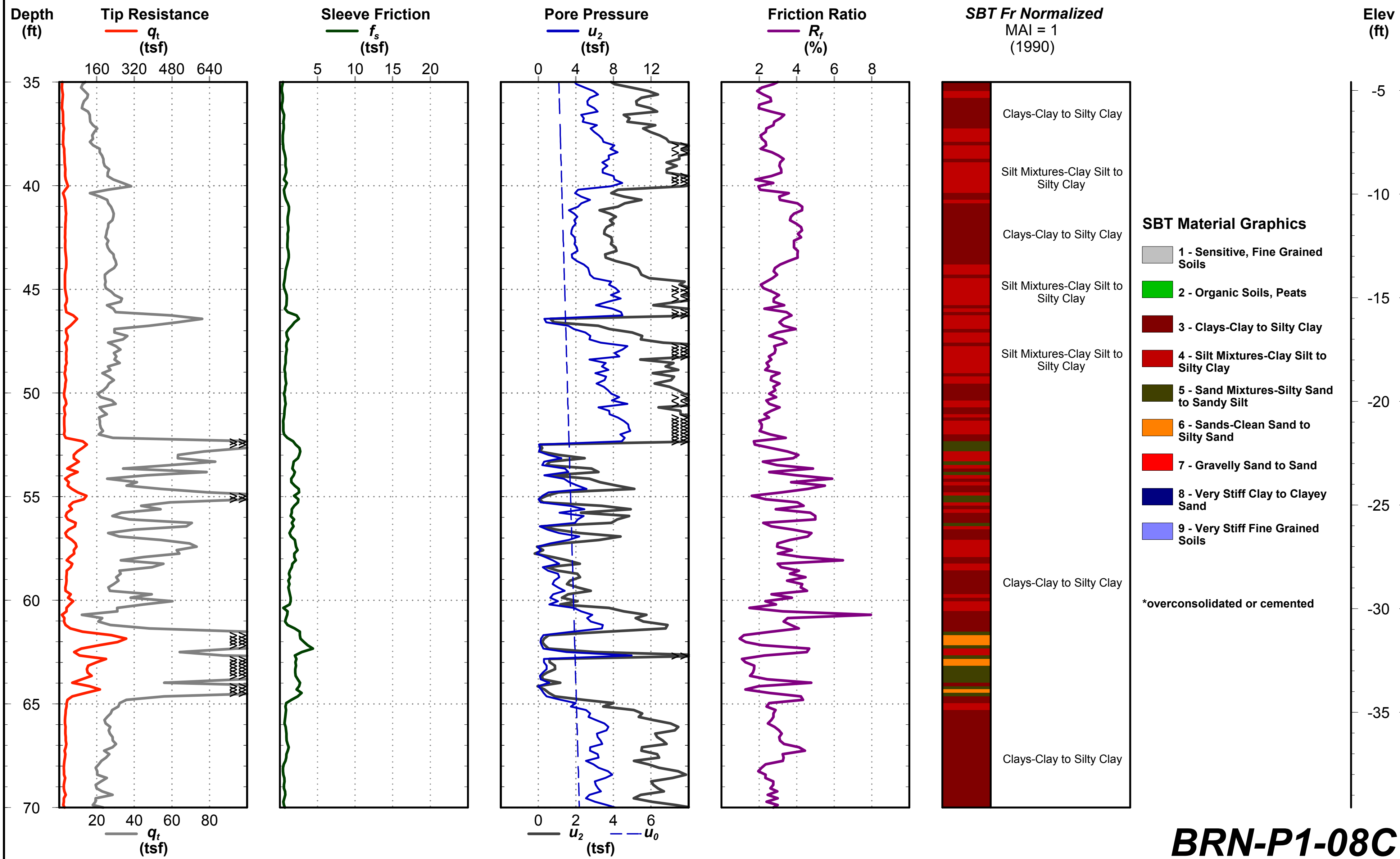
Cone Penetration Test BRN-P1-08C

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:



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:

Depth (ft) | **Tip Resistance** q_t (tsf) 160 320 480 640 | **Sleeve Friction** f_s (tsf) 5 10 15 20 | **Pore Pressure** u_2 (tsf) 0 4 8 12 | **Friction Ratio** R_f (%) 2 4 6 8 | ***SBT Fr Normalized*** MAI = 1 (1990) | **Elev** (ft)

70

q_t (tsf) 20 40 60 80

u_2 — u_0 (tsf) 0 2 4 6

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

BRN-P1-08C

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

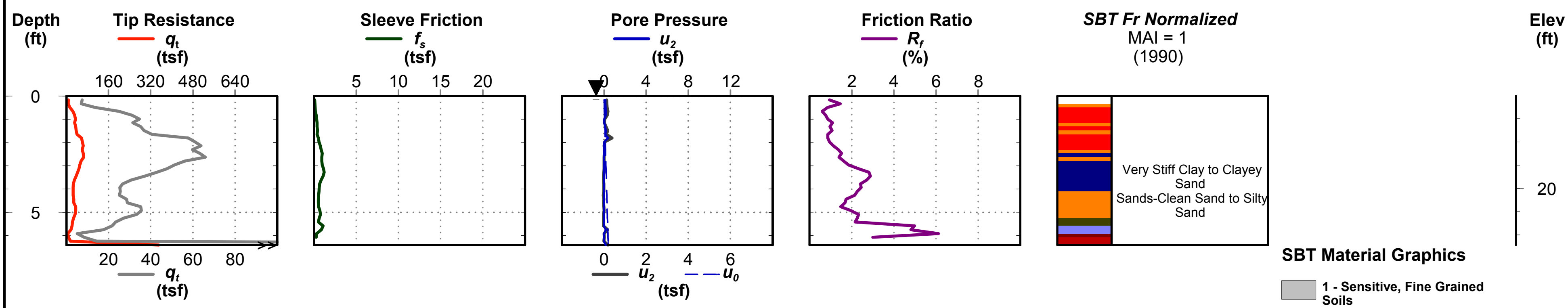
Cone Penetration Test BRN-P1-09C

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:



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

BRN-P1-09C

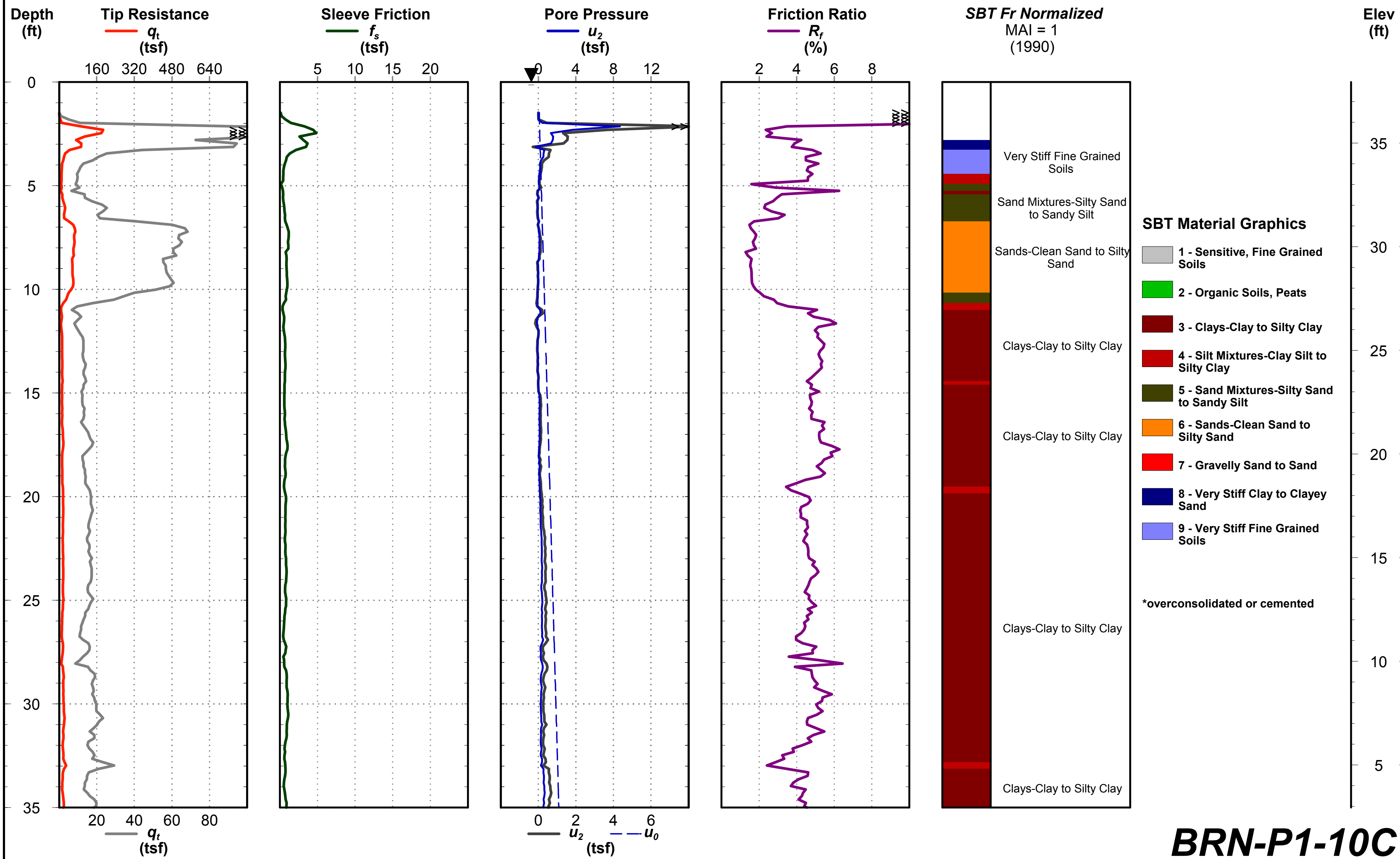
Cone Penetration Test BRN-P1-10C

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:



BRN-P1-10C

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

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

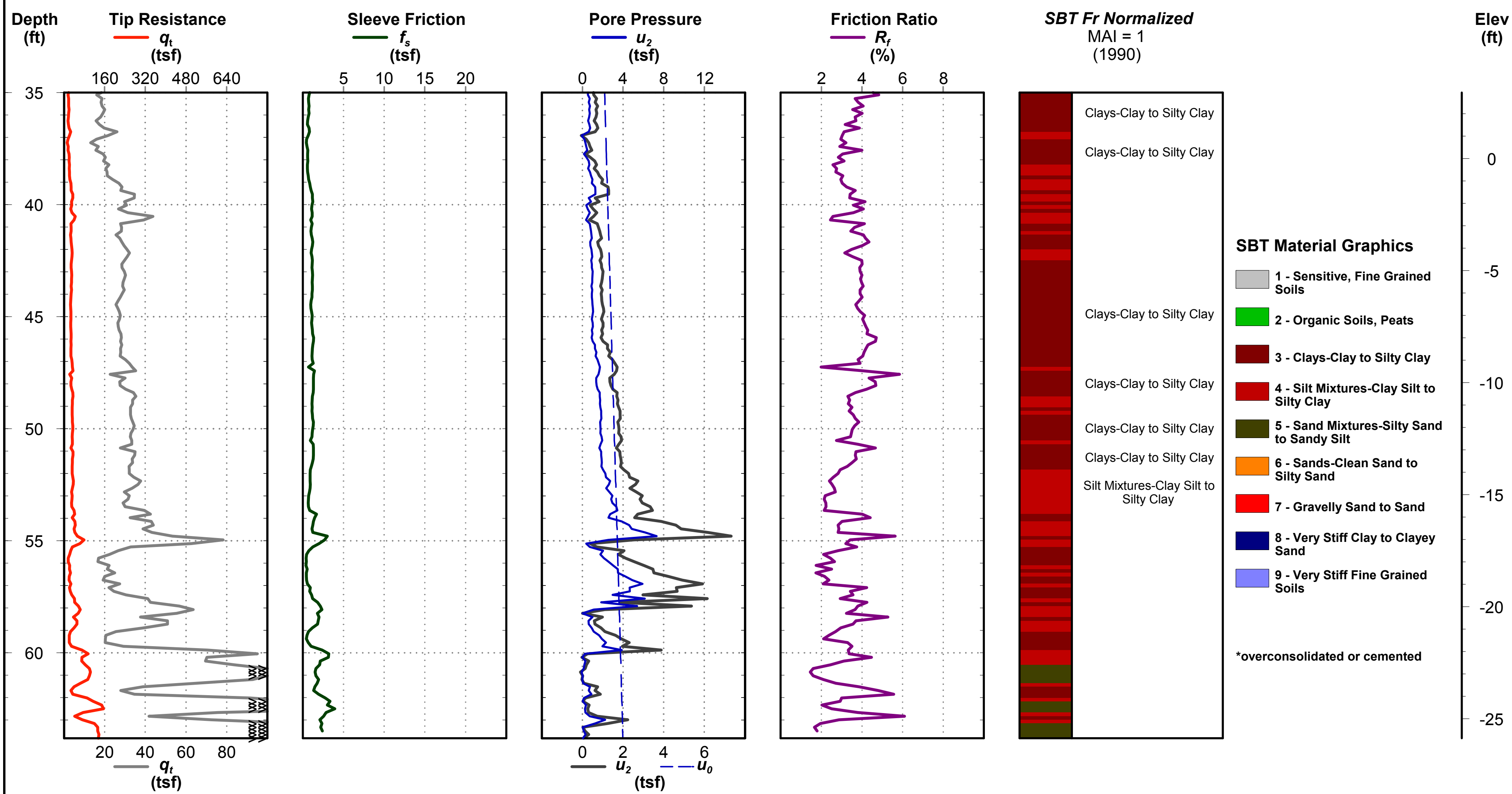
Cone Penetration Test BRN-P1-10C

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:



BRN-P1-10C

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

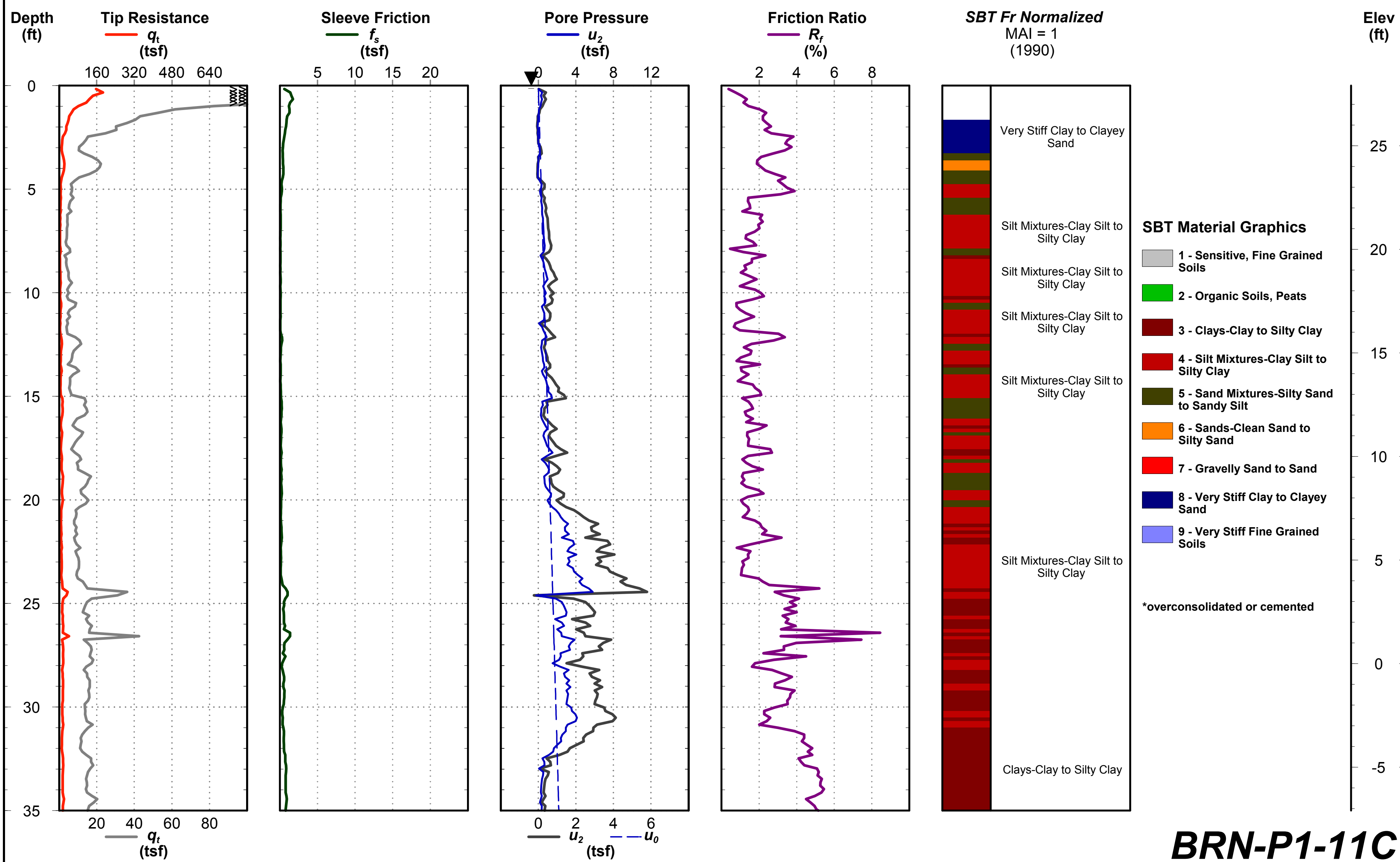
Cone Penetration Test BRN-P1-11C

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

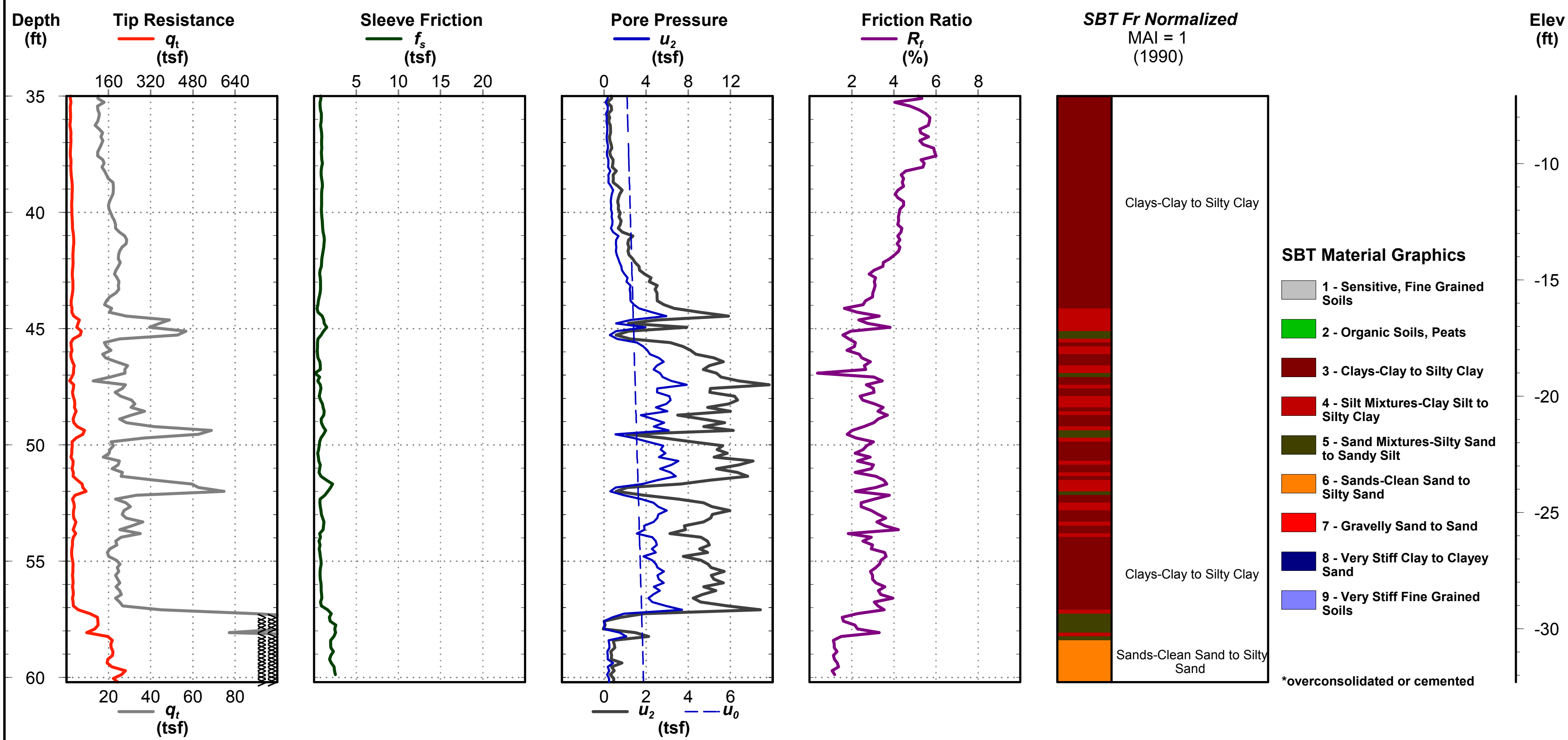
Cone Penetration Test BRN-P1-11C

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

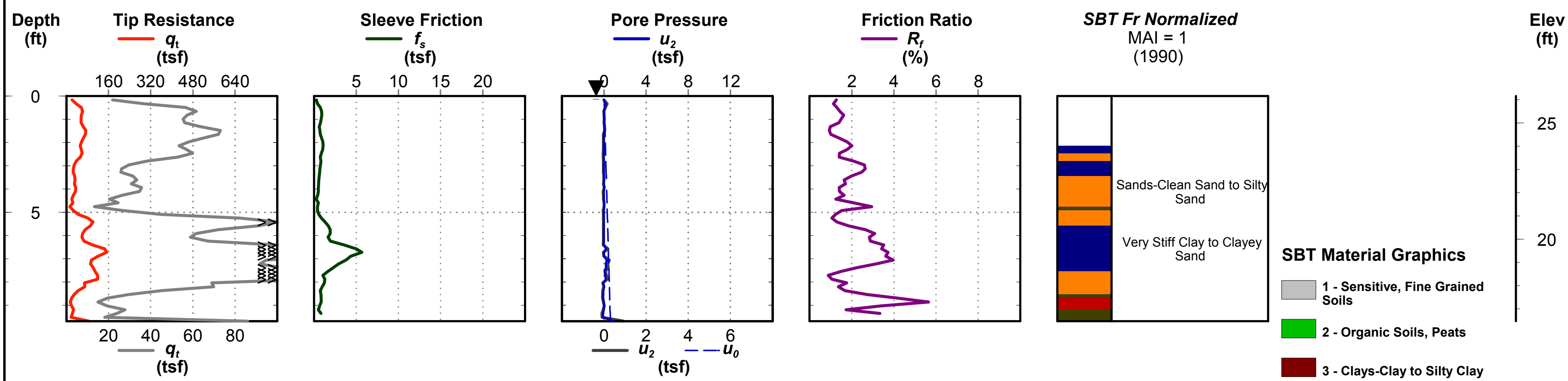
Cone Penetration Test BRN-P1-12C

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:



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-P1-12C

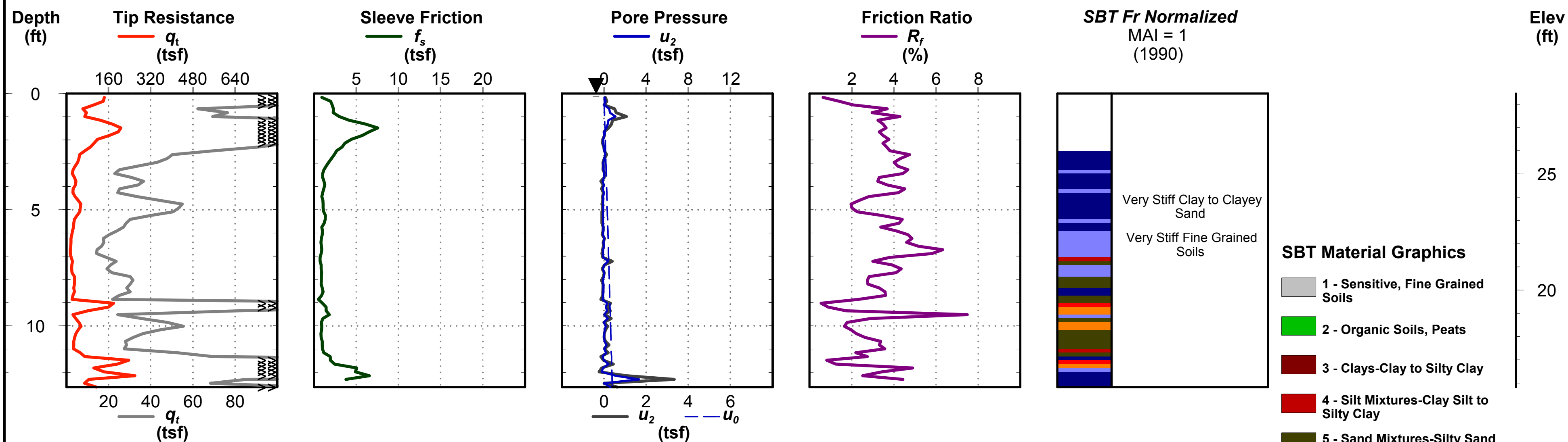
Cone Penetration Test BRN-P1-13C

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

BRN-P1-13C

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

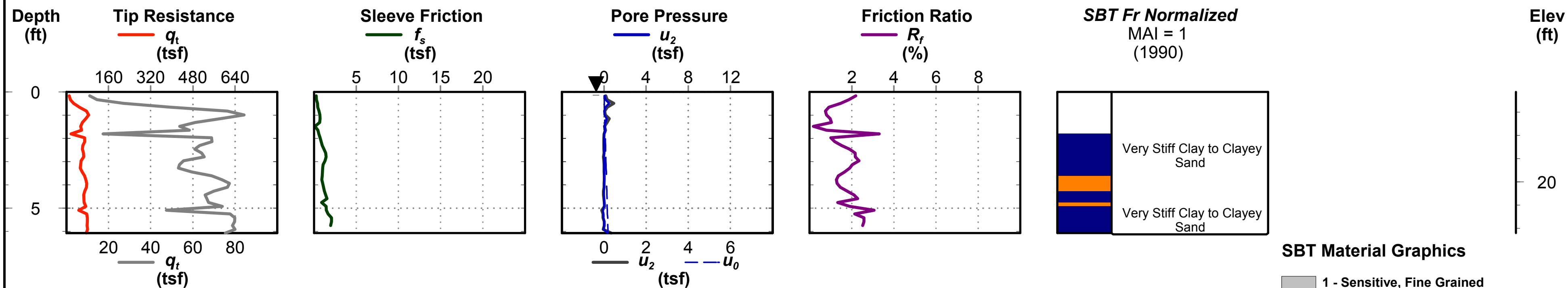
Cone Penetration Test BRN-P1-31C

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

BRN-P1-31C

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

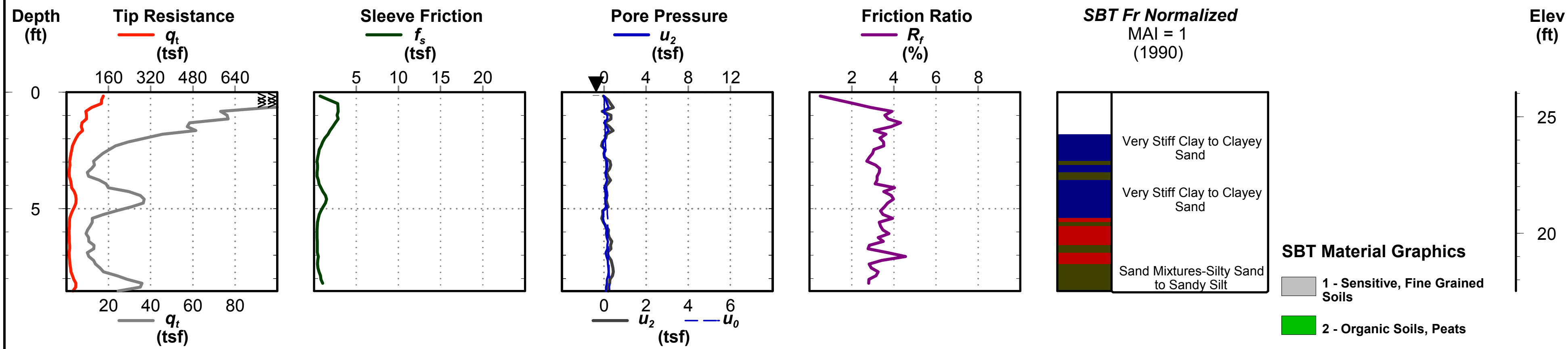
Cone Penetration Test BRN-P2-14C

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:



SBT Material Graphics

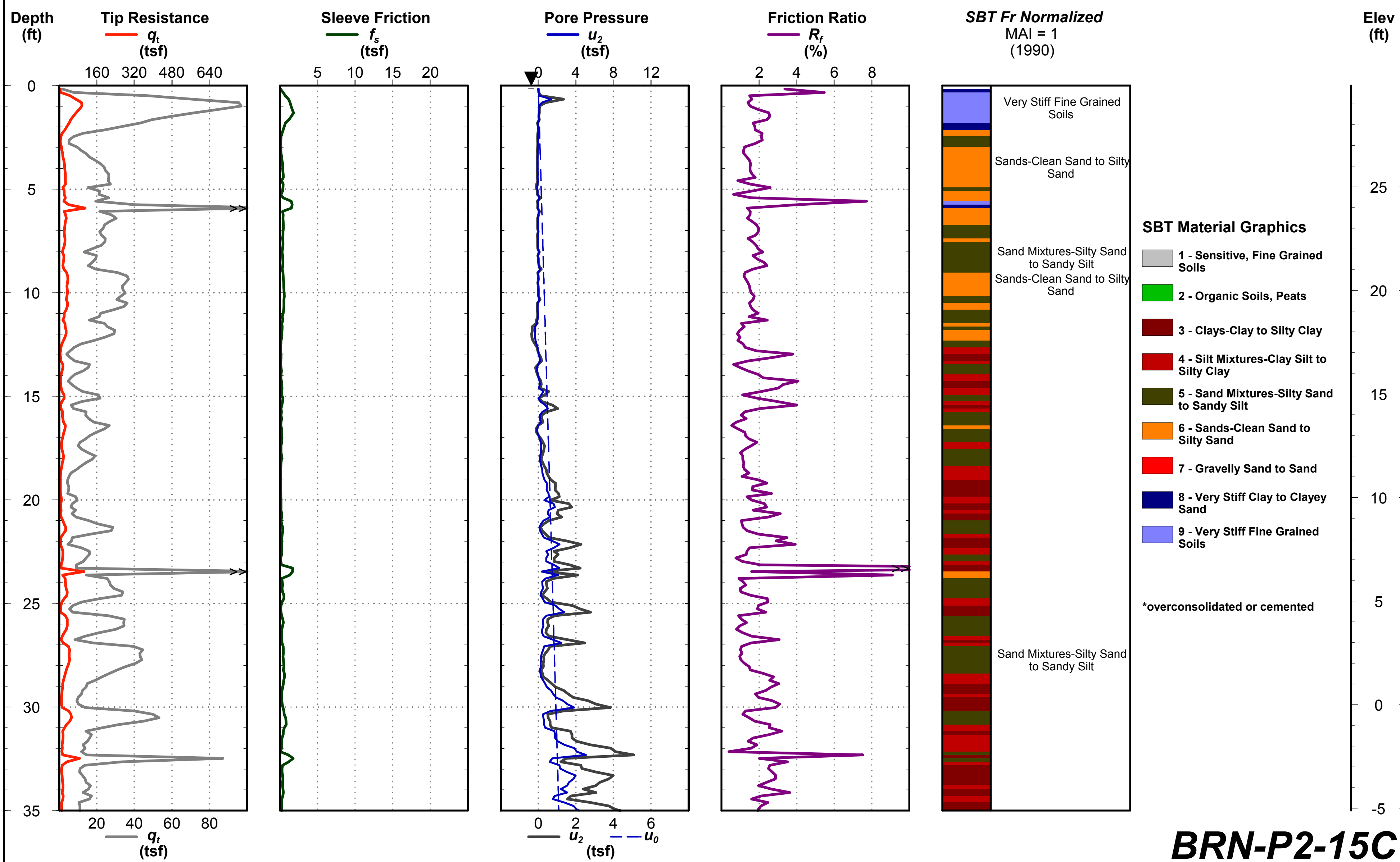
- 1 - Sensitive, Fine Grained Soils
- 2 - Organic Soils, Peats
- 3 - Clays-Clay to Silty Clay
- 4 - Silt Mixtures-Clay Silt to Silty Clay
- 5 - Sand Mixtures-Silty Sand to Sandy Silt
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff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

*overconsolidated or cemented

Cone Penetration Test BRN-P2-15C

Brownsville, Tx
Project Number :IBWC

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



BRN-P2-15C

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

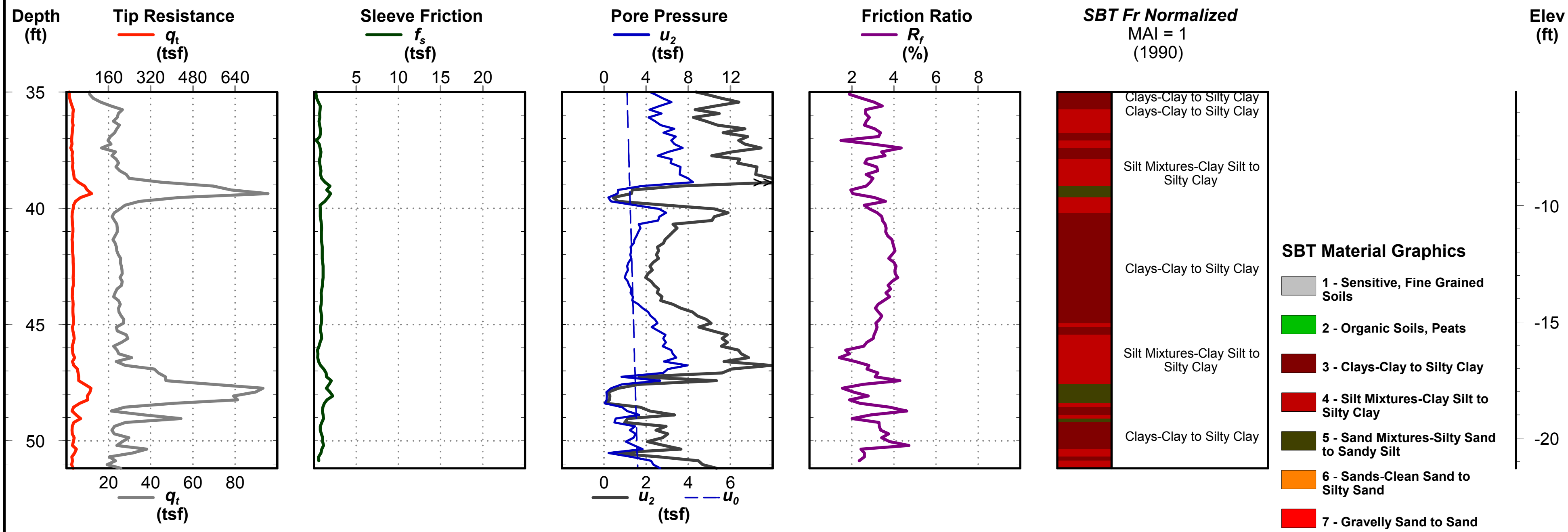
Cone Penetration Test BRN-P2-15C

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:



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

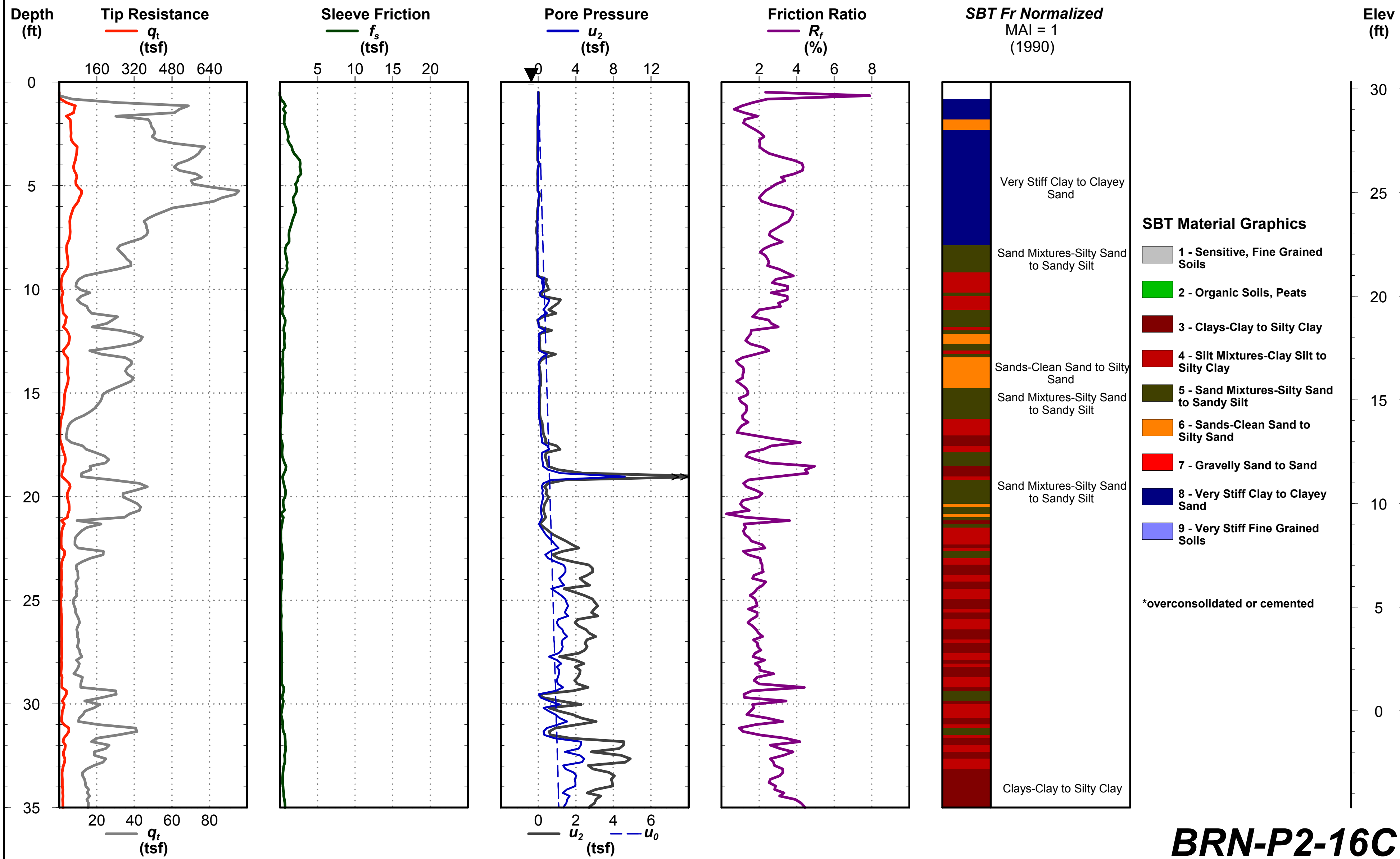
Cone Penetration Test BRN-P2-16C

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:



BRN-P2-16C

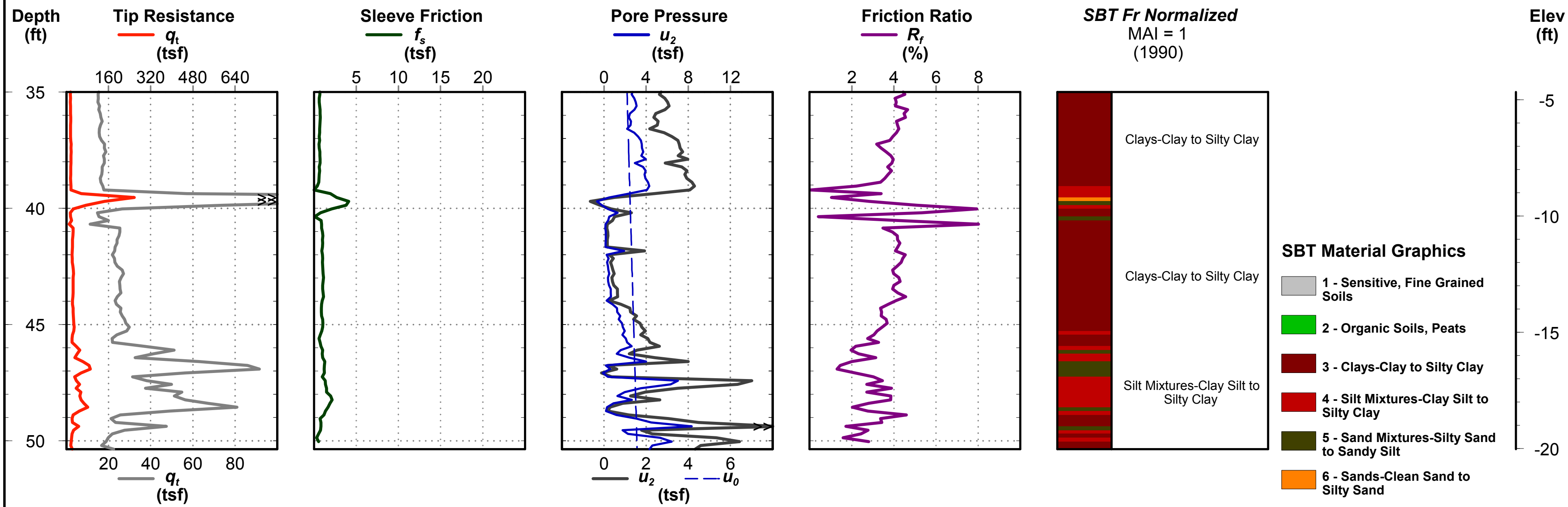
Cone Penetration Test BRN-P2-16C

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:



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

BRN-P2-16C

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

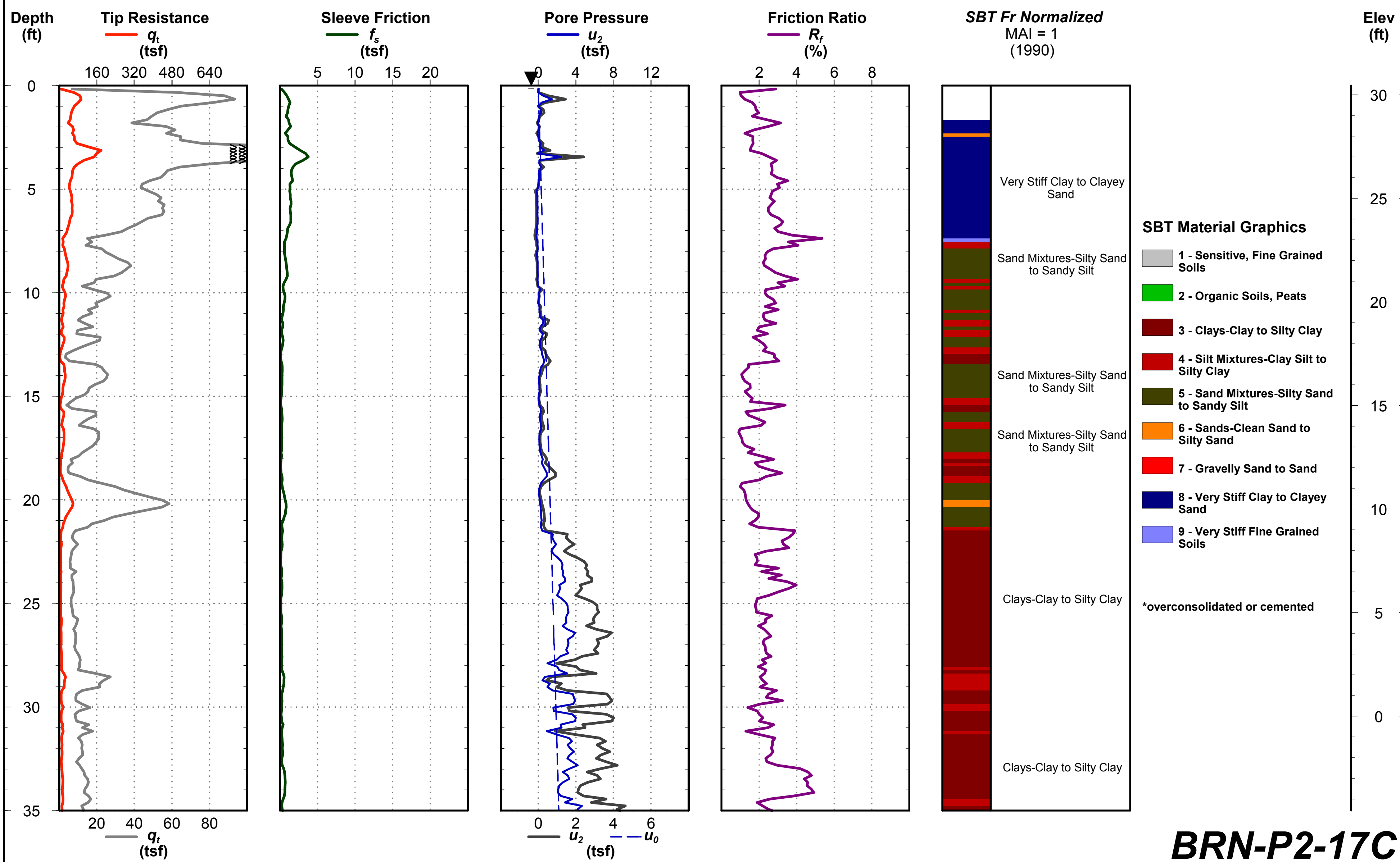
Brownsville, Tx
Project Number :IBWC

Cone Penetration Test BRN-P2-17C

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

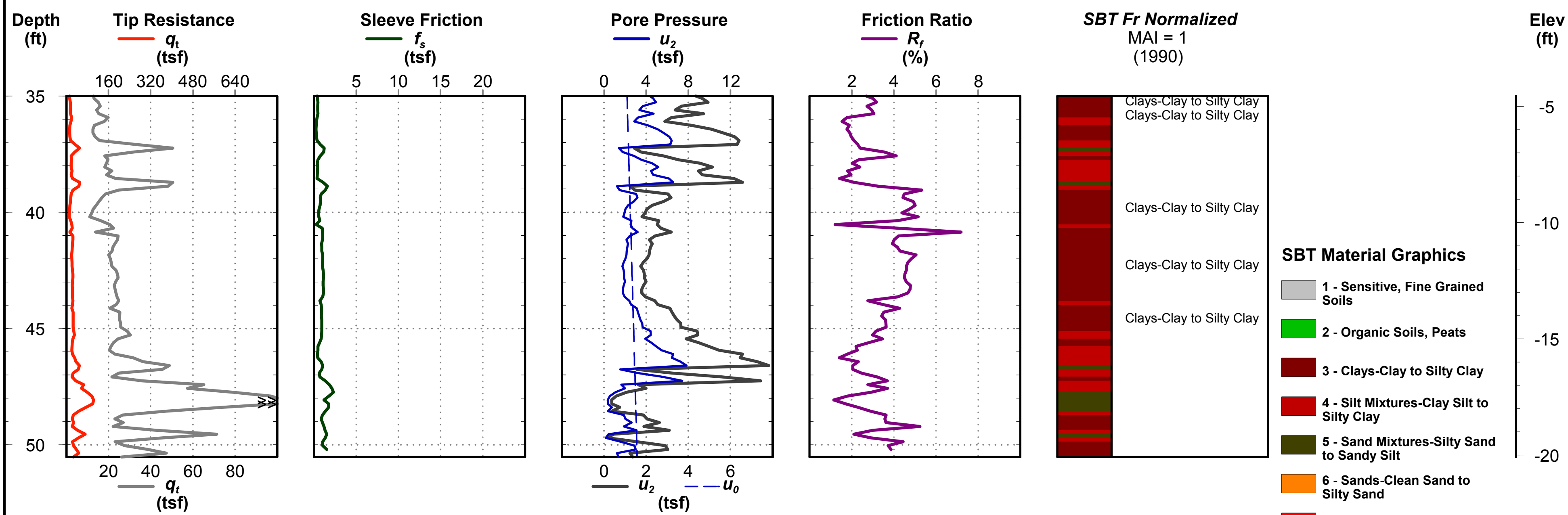
Cone Penetration Test BRN-P2-17C

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:



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

BRN-P2-17C

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

Cone Penetration Test BRN-P2-18C

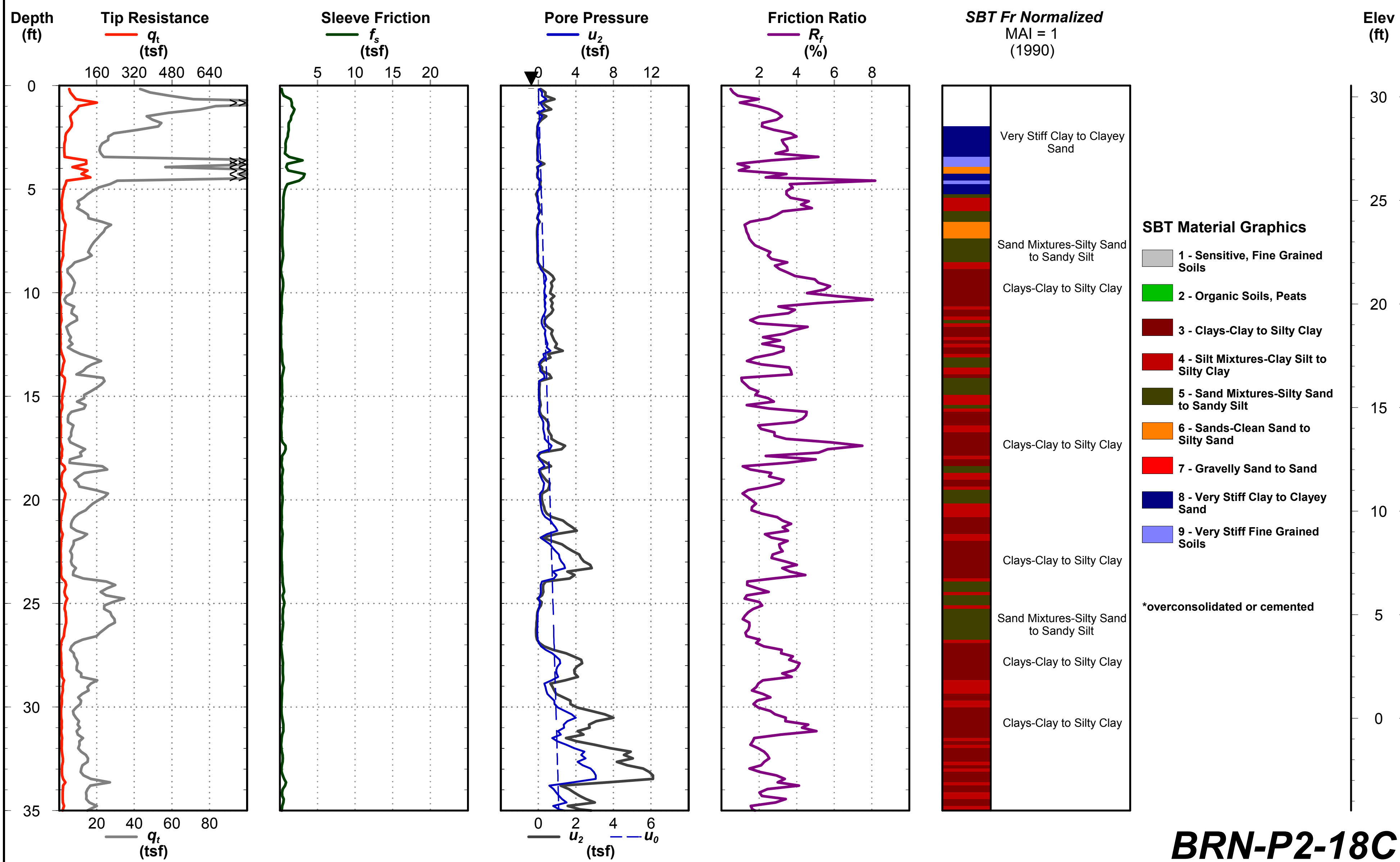
ERDC

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:



BRN-P2-18C

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

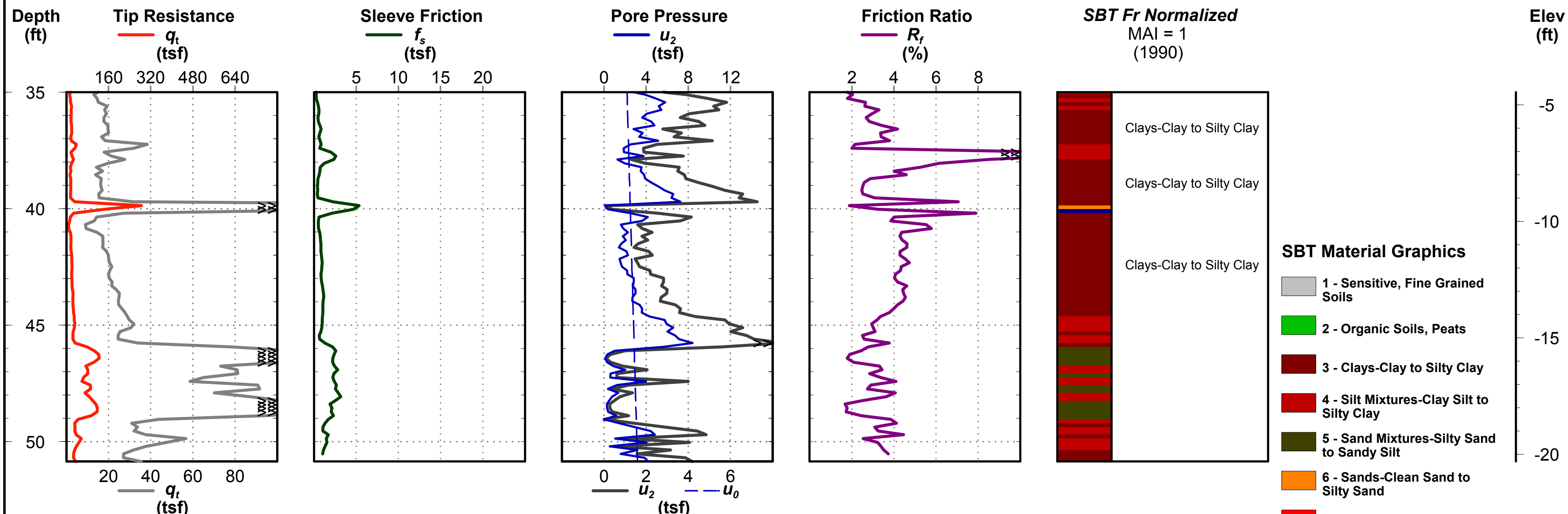
Cone Penetration Test BRN-P2-18C

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:



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

BRN-P2-18C

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

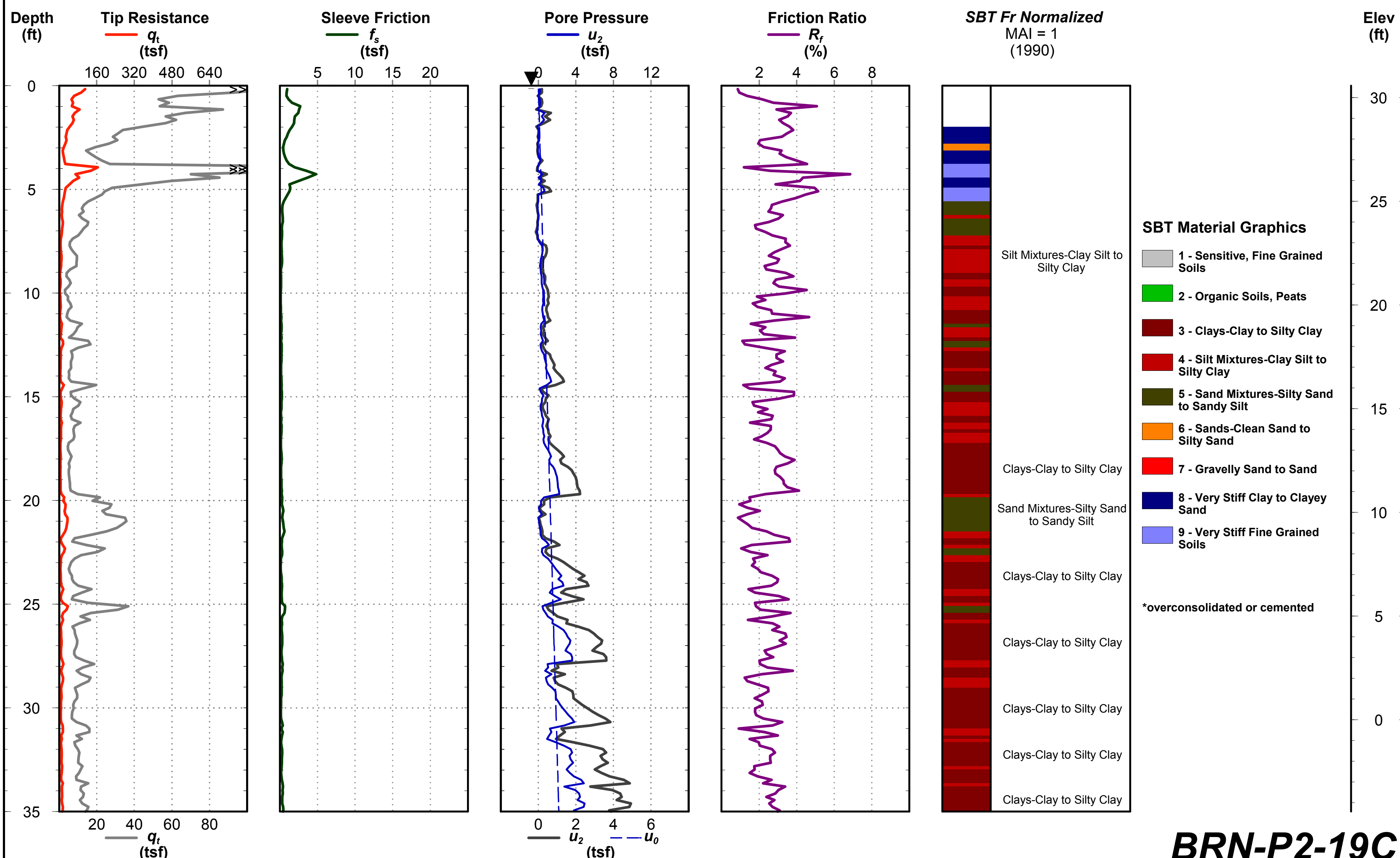
Cone Penetration Test BRN-P2-19C

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:



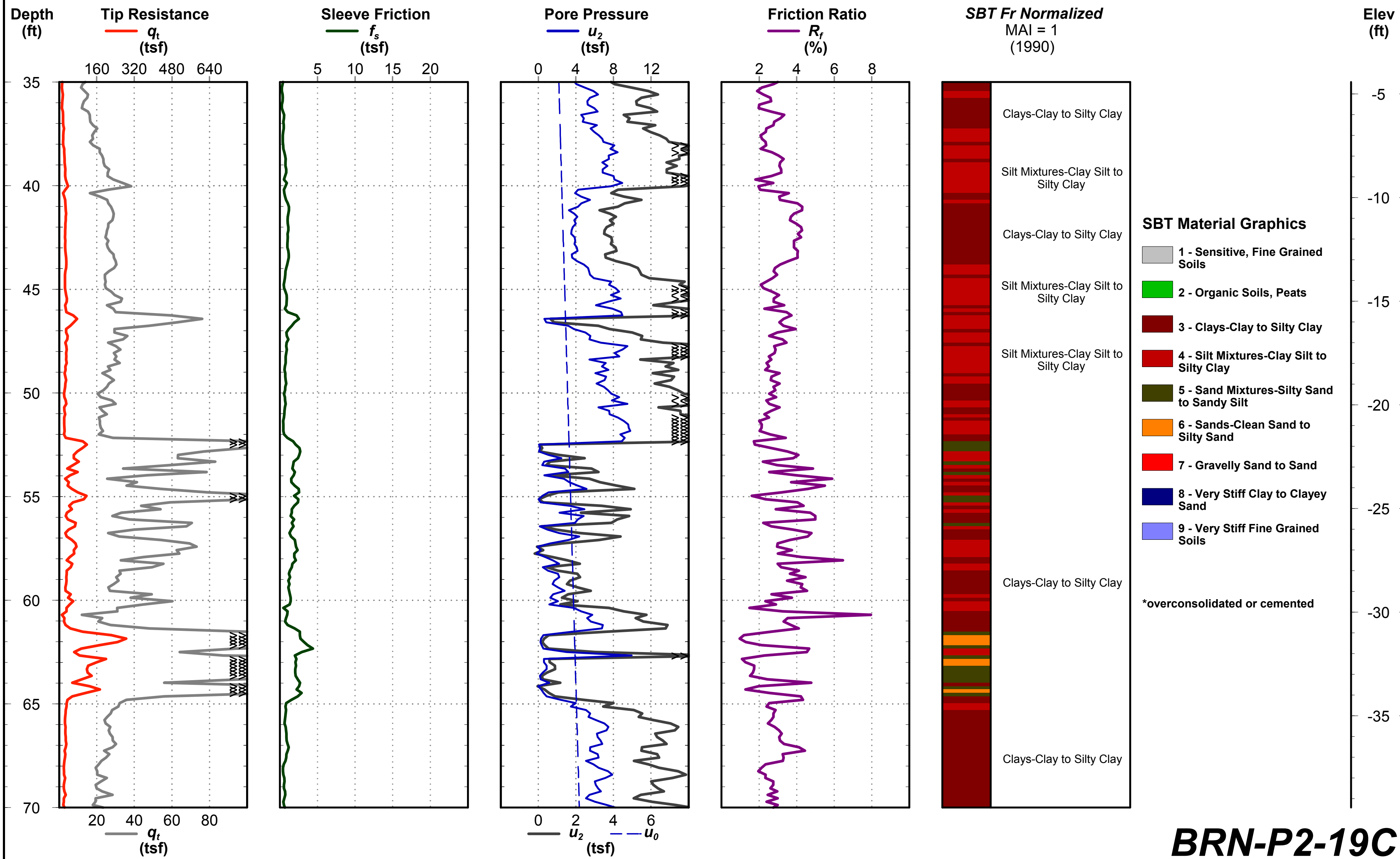
BRN-P2-19C

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

Cone Penetration Test BRN-P2-19C

Brownsville, Tx
Project Number :IBWC

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



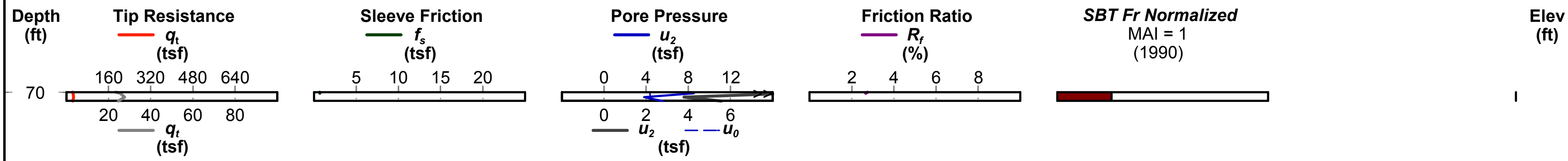
BRN-P2-19C

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

BRN-P2-19C

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

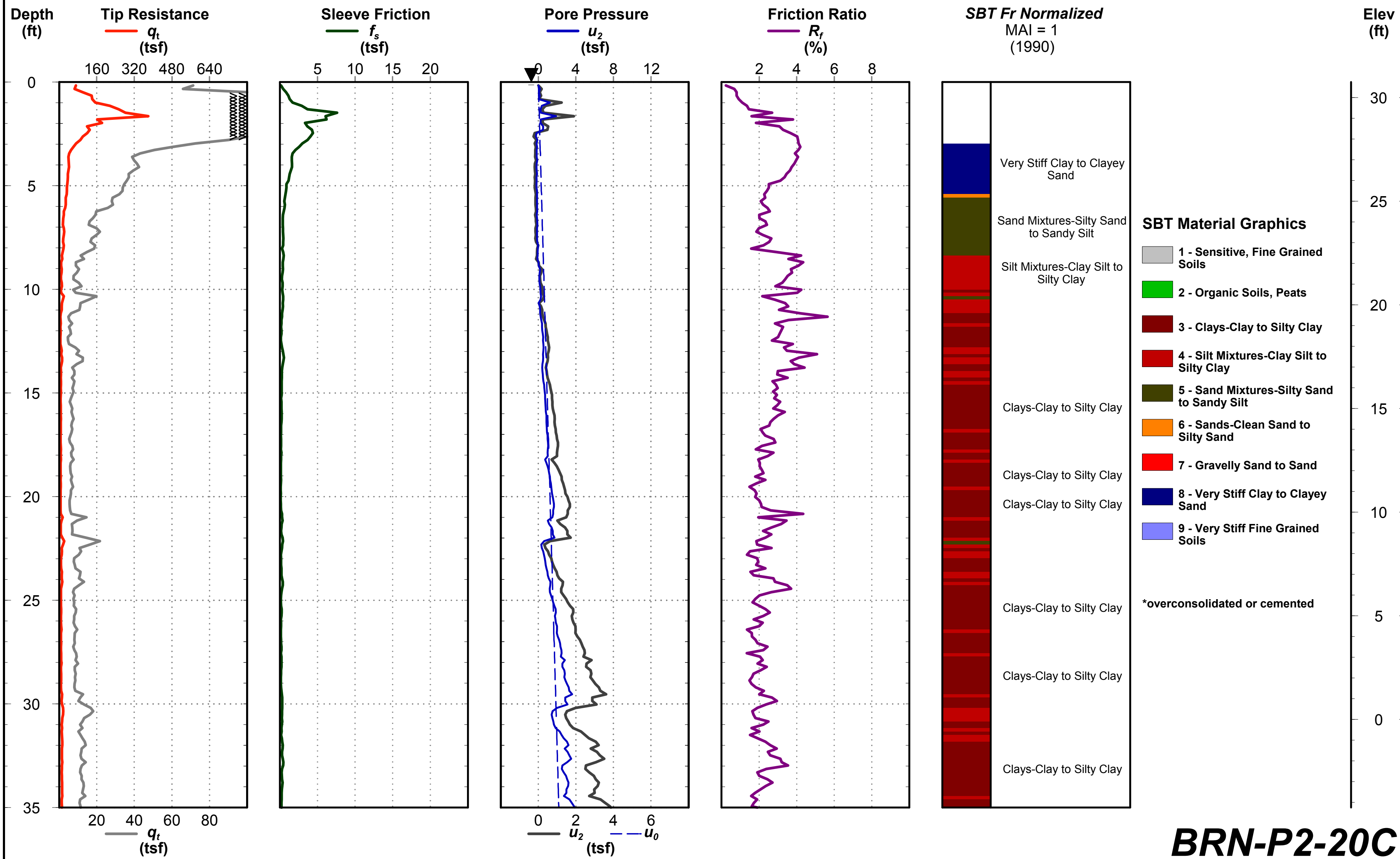
Cone Penetration Test BRN-P2-20C

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

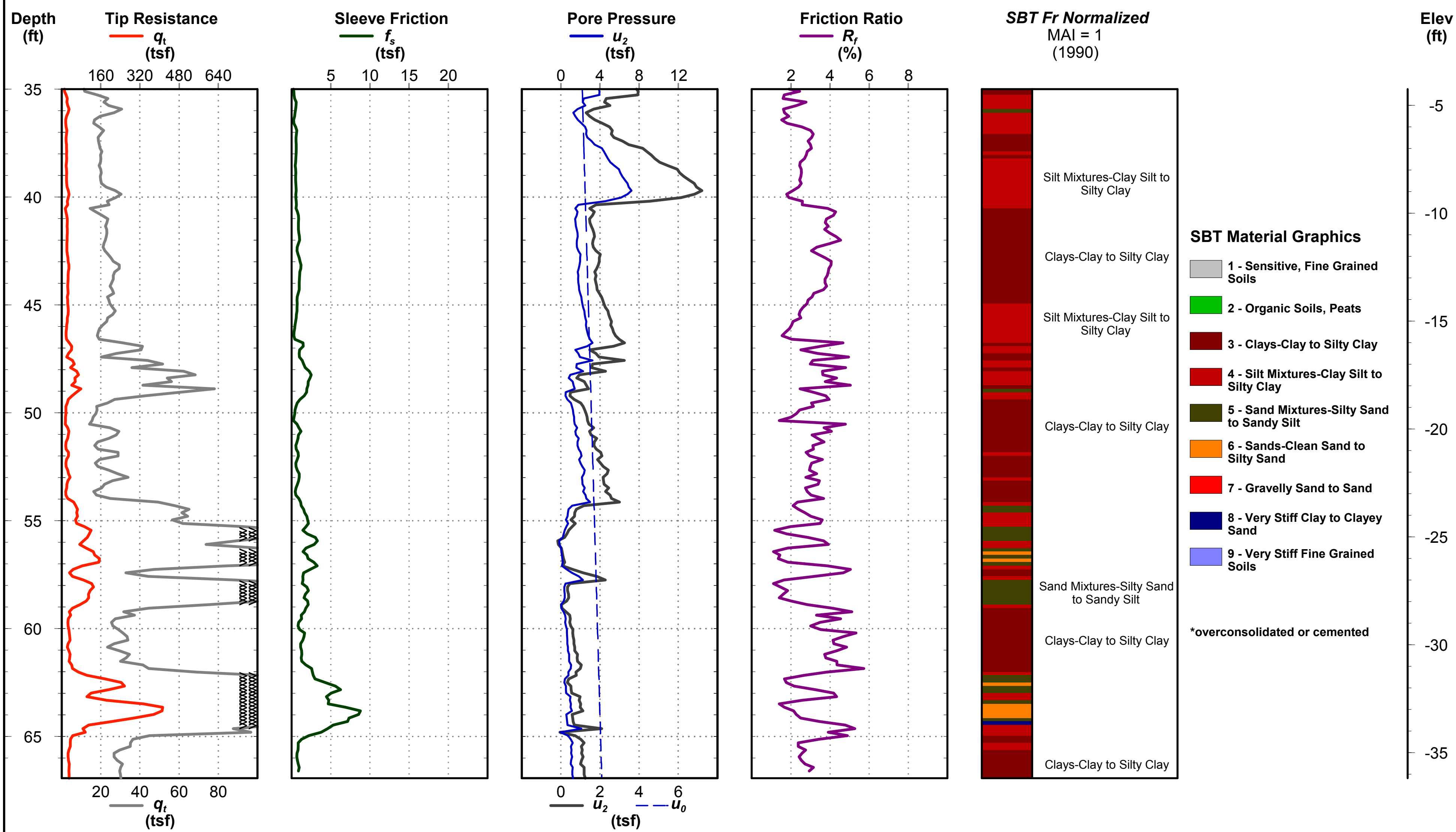
Cone Penetration Test BRN-P2-20C

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

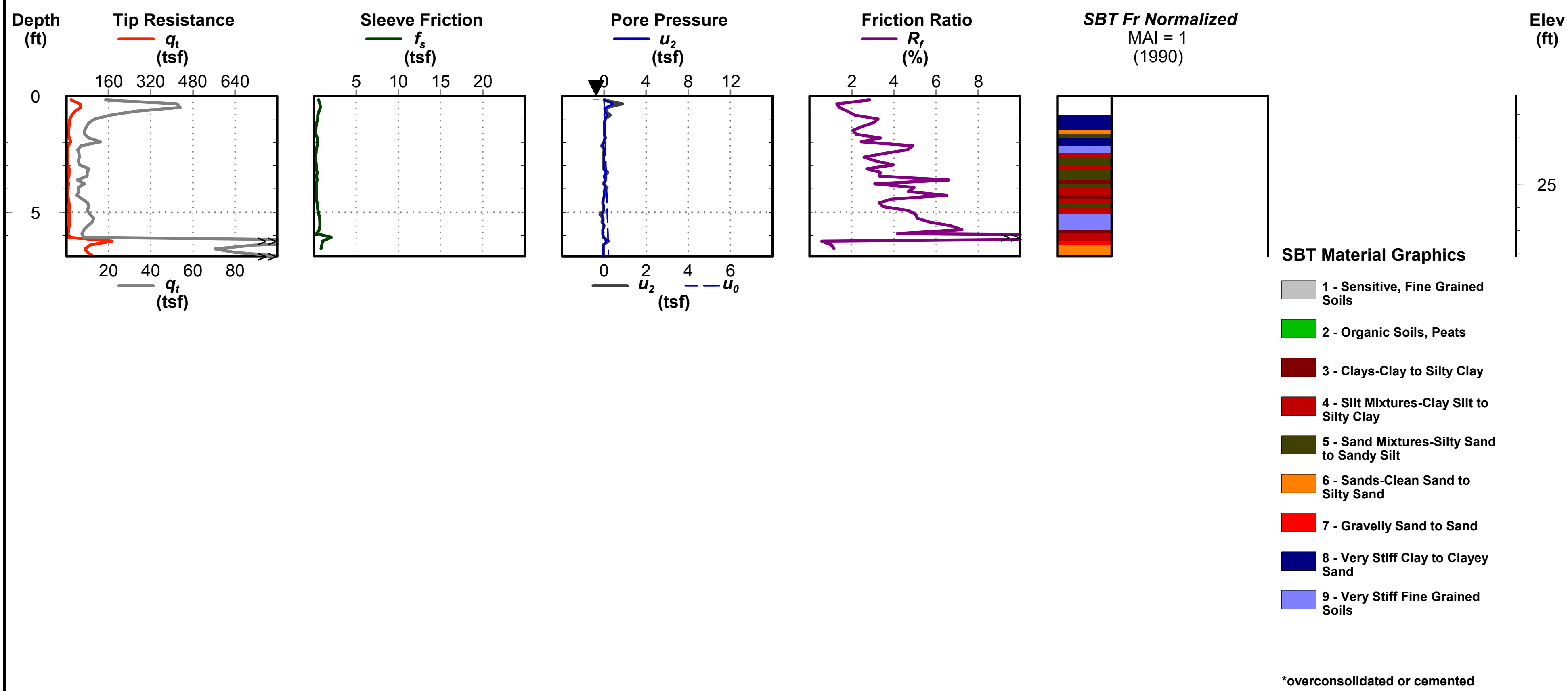
Cone Penetration Test BRN-P2-21C

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:



BRN-P2-21C

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

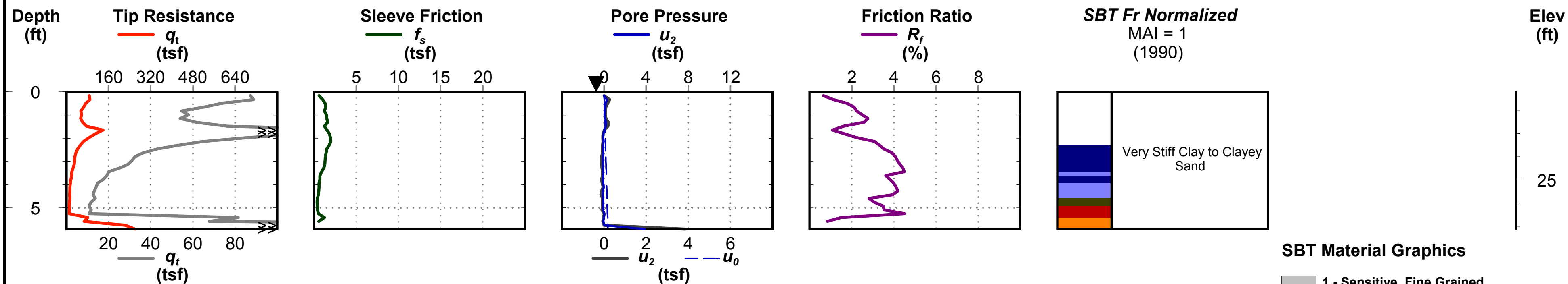
Cone Penetration Test BRN-P2-21C-a

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: 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

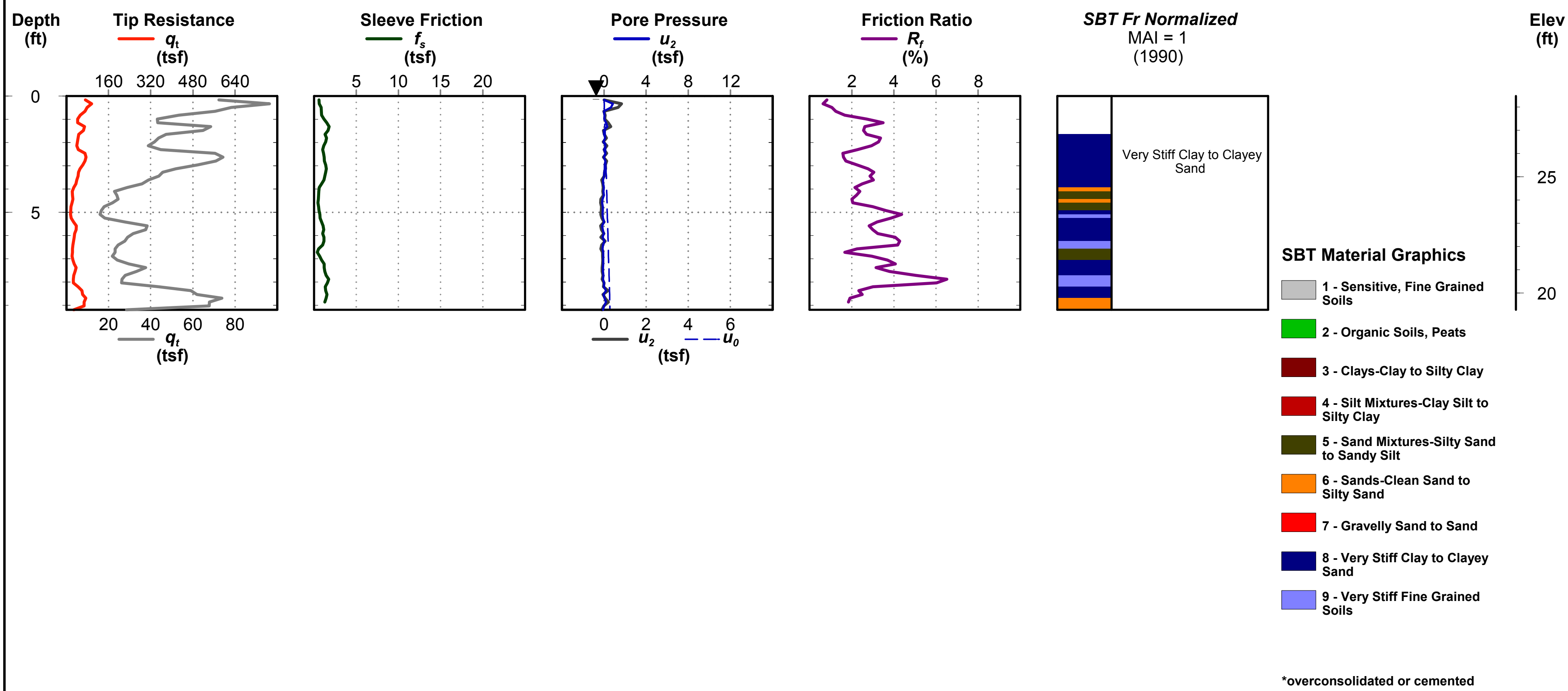
Cone Penetration Test BRN-P2-22C

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:



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

BRN-P2-22C

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

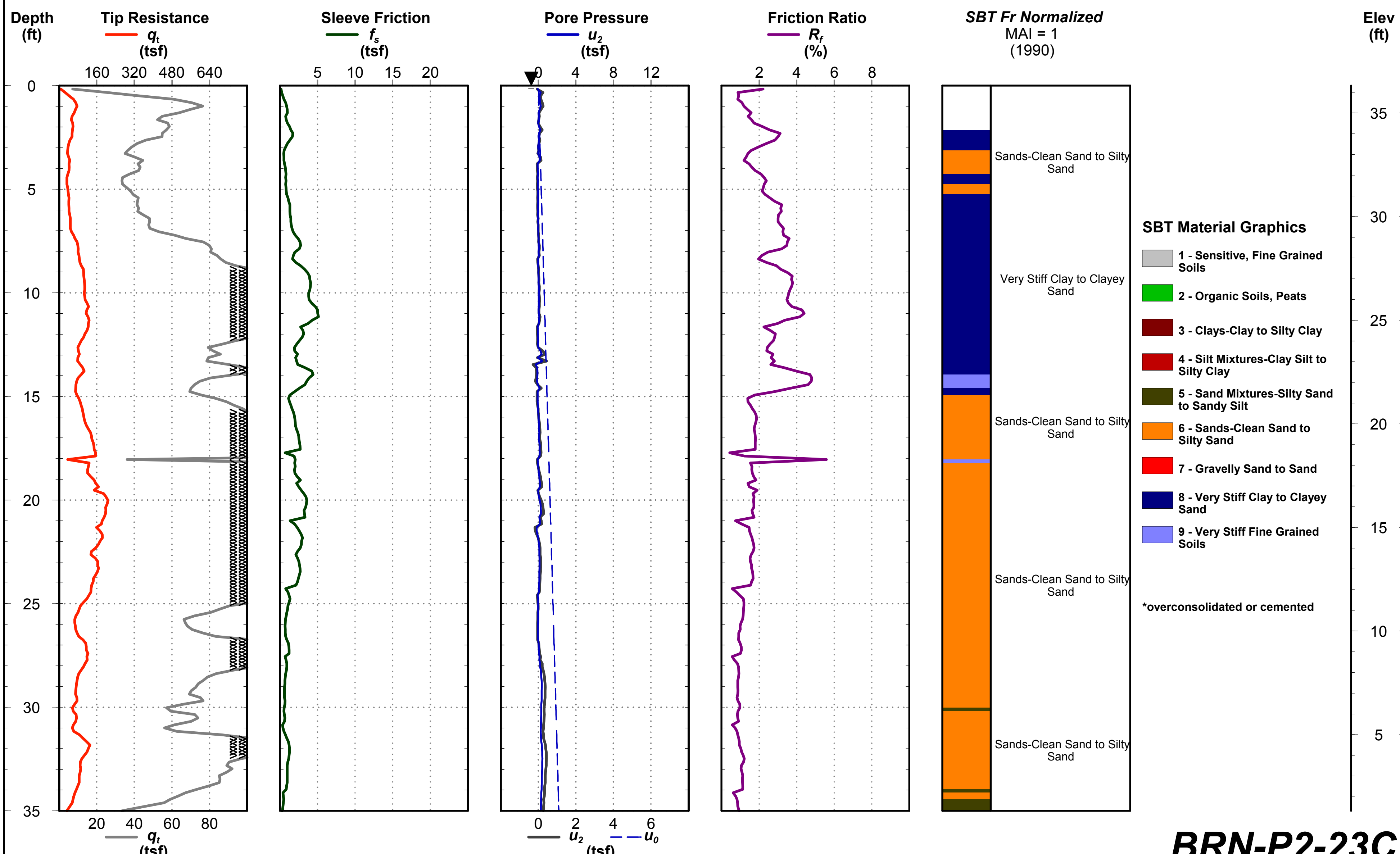
Cone Penetration Test BRN-P2-23C

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:



BRN-P2-23C

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

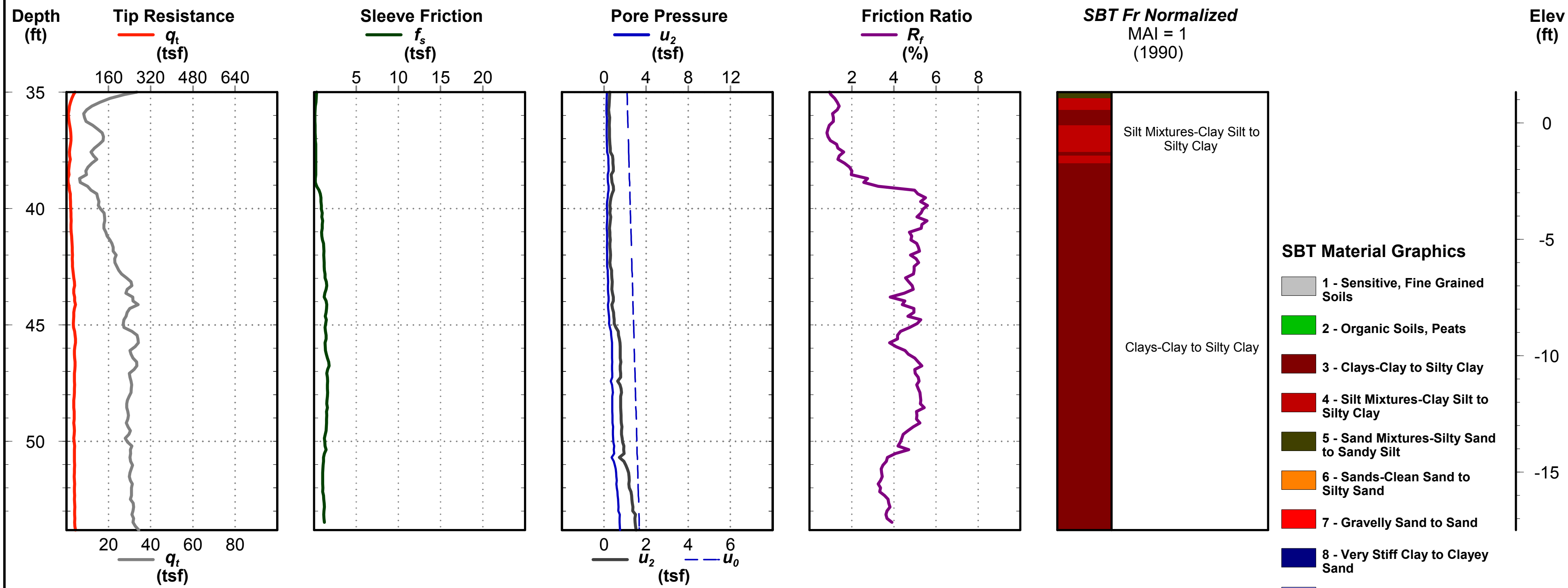
Cone Penetration Test BRN-P2-23C

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:



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

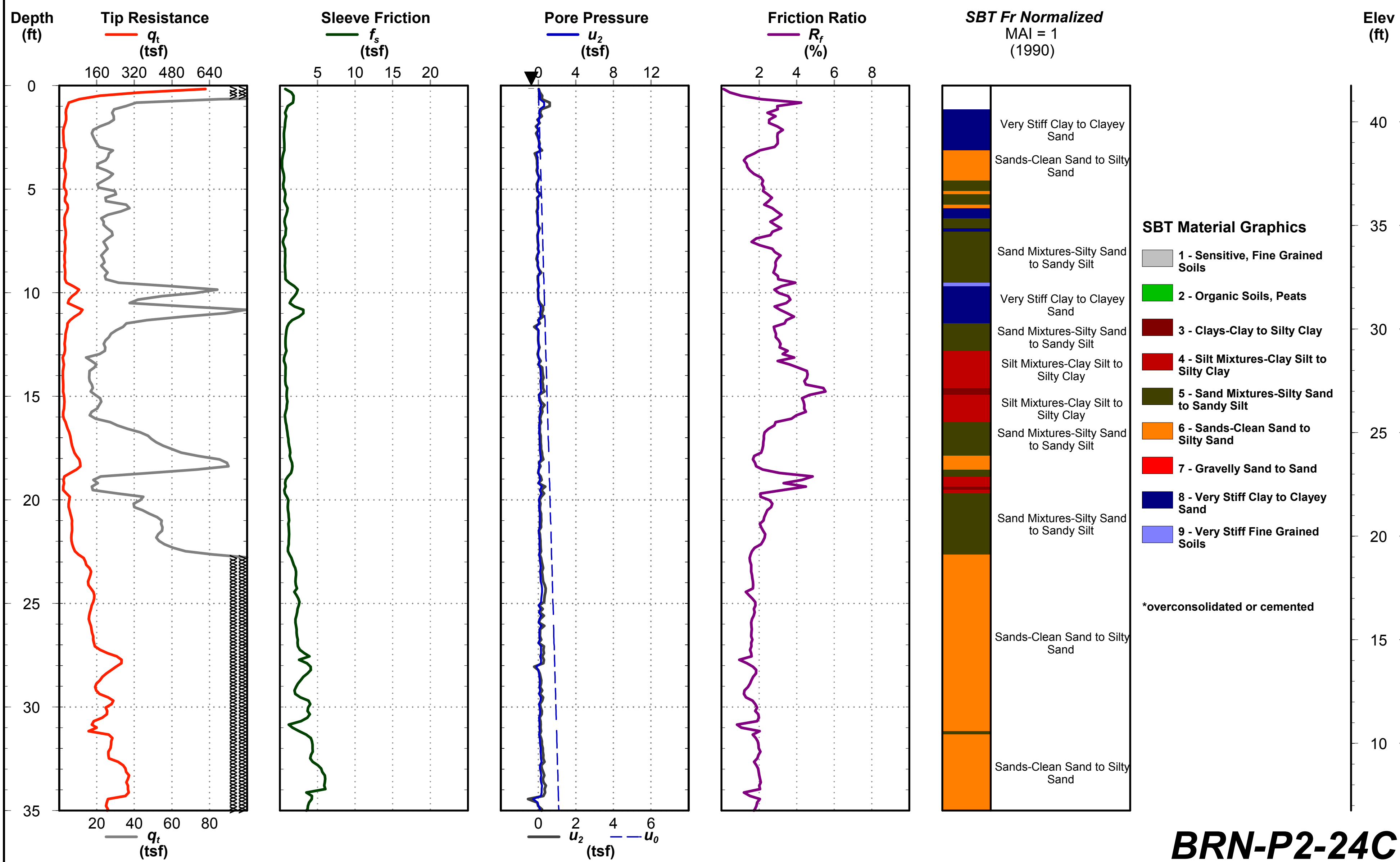
Cone Penetration Test BRN-P2-24C

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:



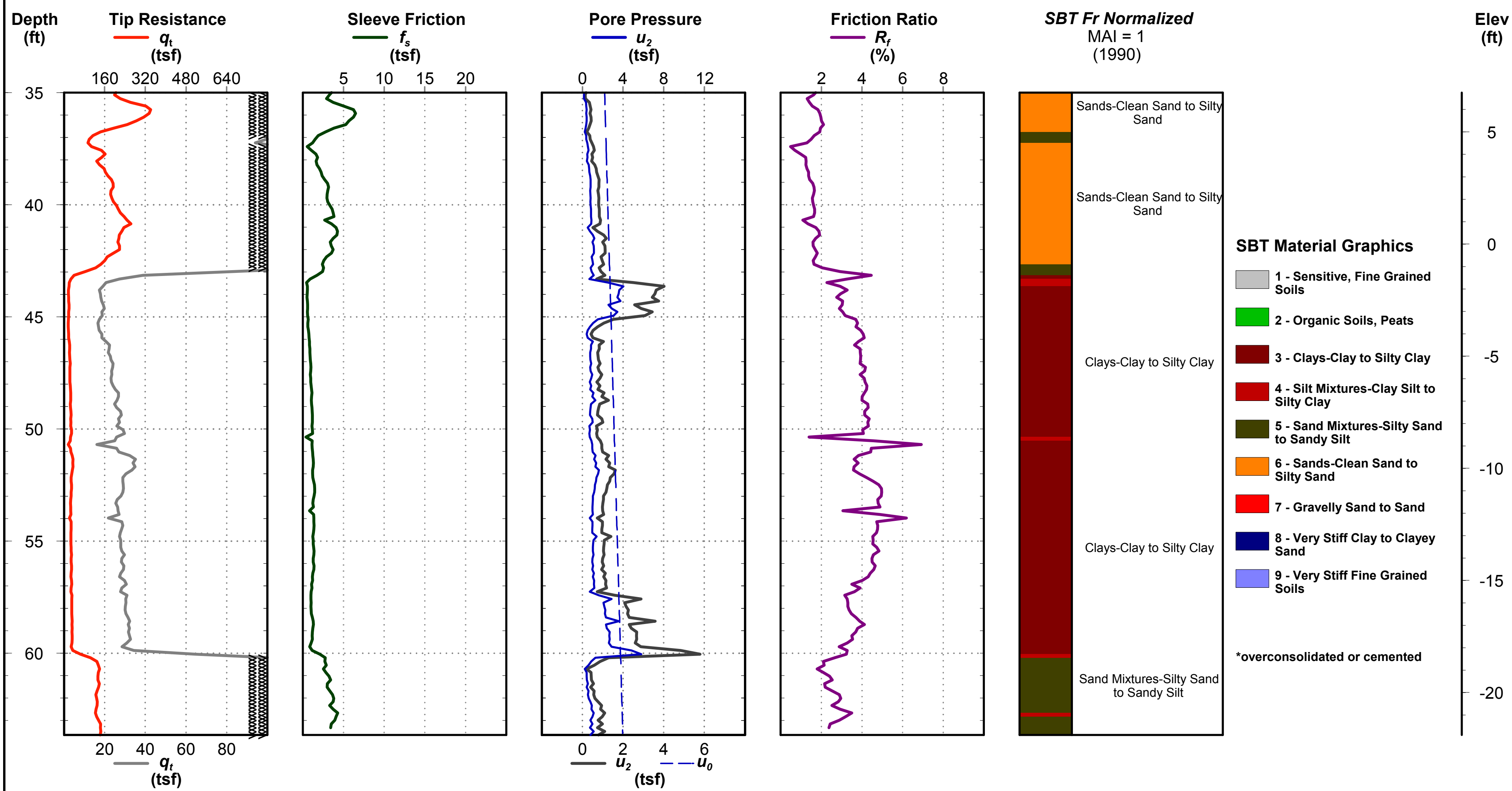
BRN-P2-24C

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

Cone Penetration Test BRN-P2-24C

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	**Cone Size:**	



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

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BRN-P2-24C

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

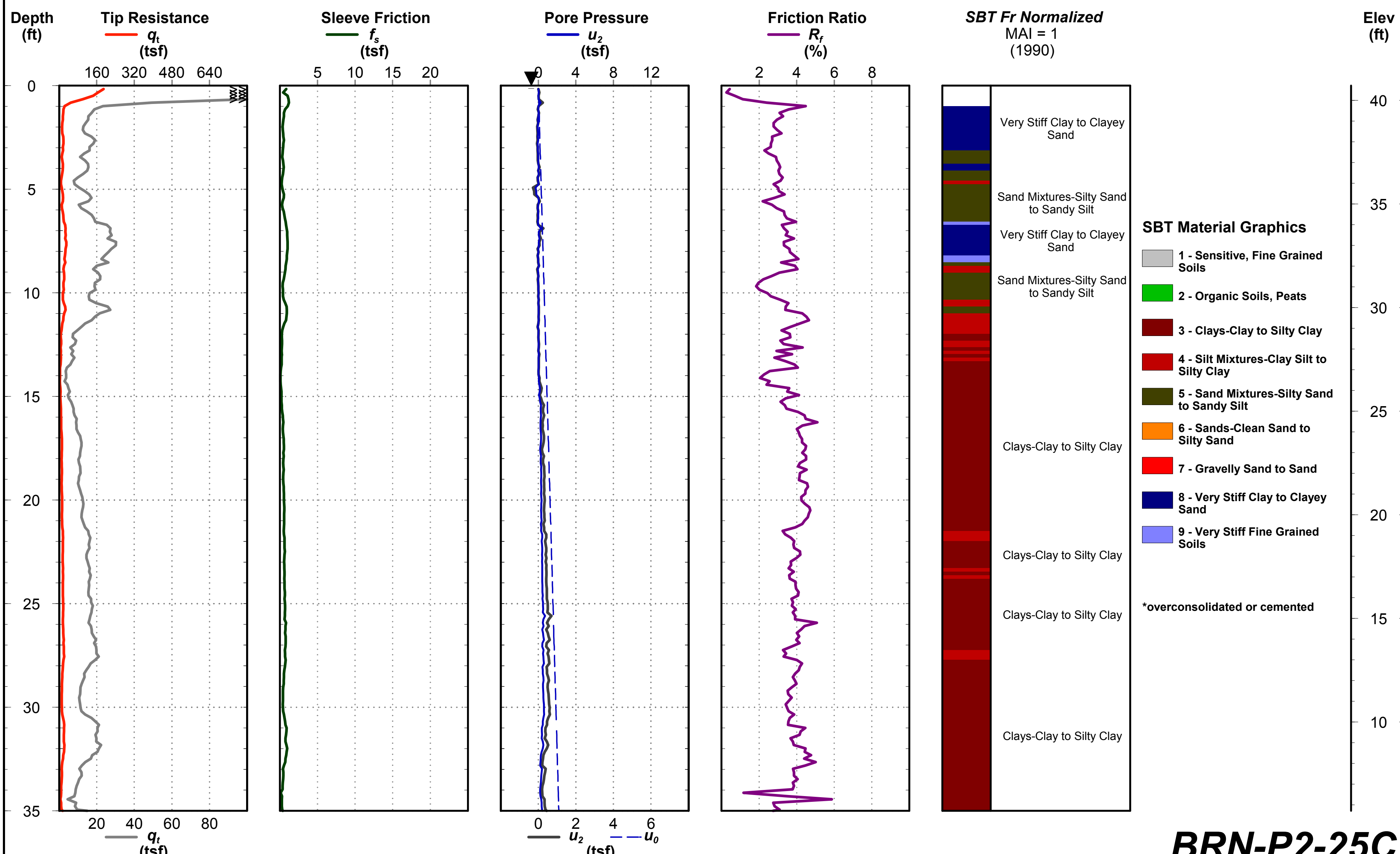
Cone Penetration Test BRN-P2-25C

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:



BRN-P2-25C

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

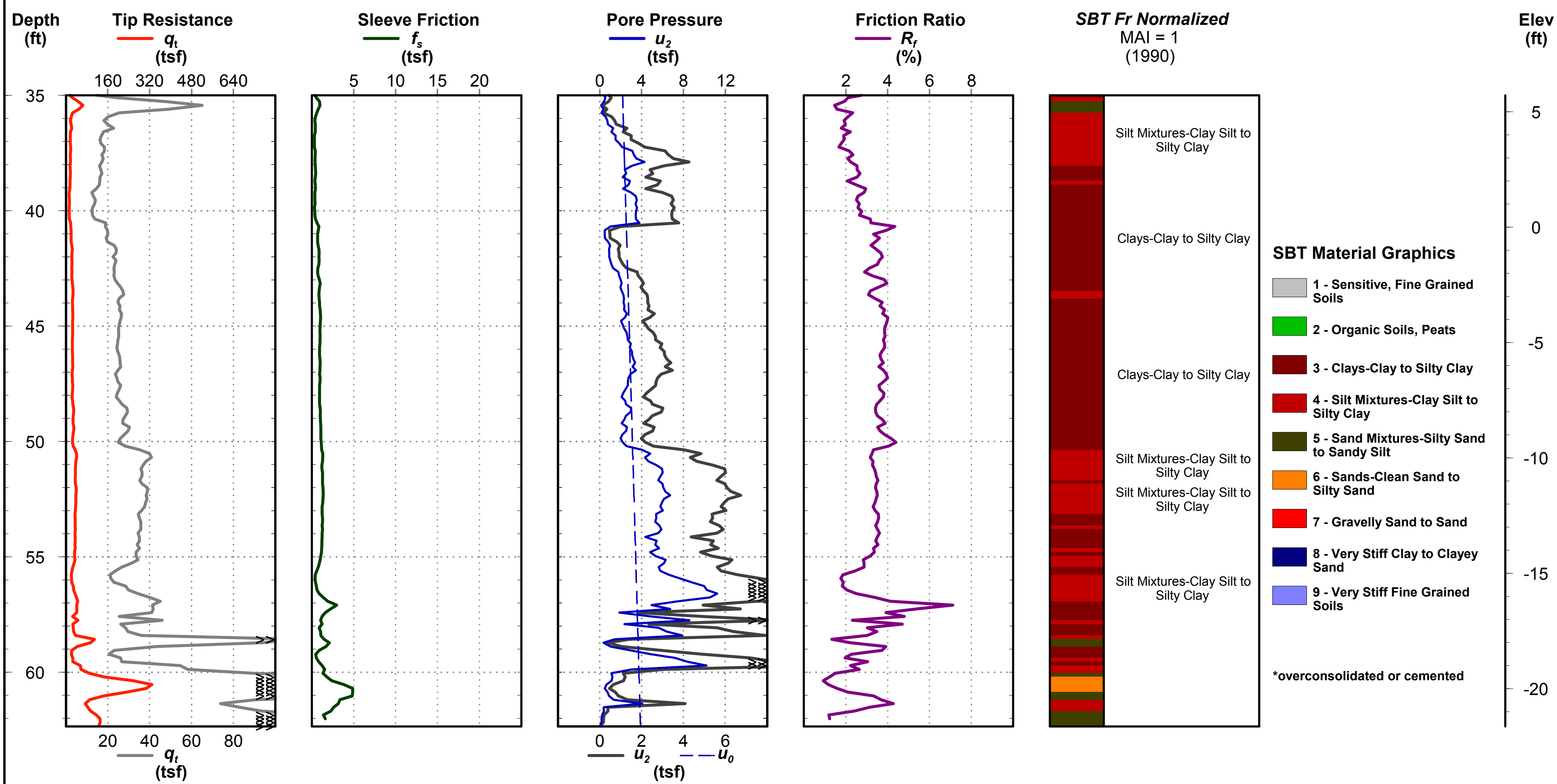
Cone Penetration Test BRN-P2-25C

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

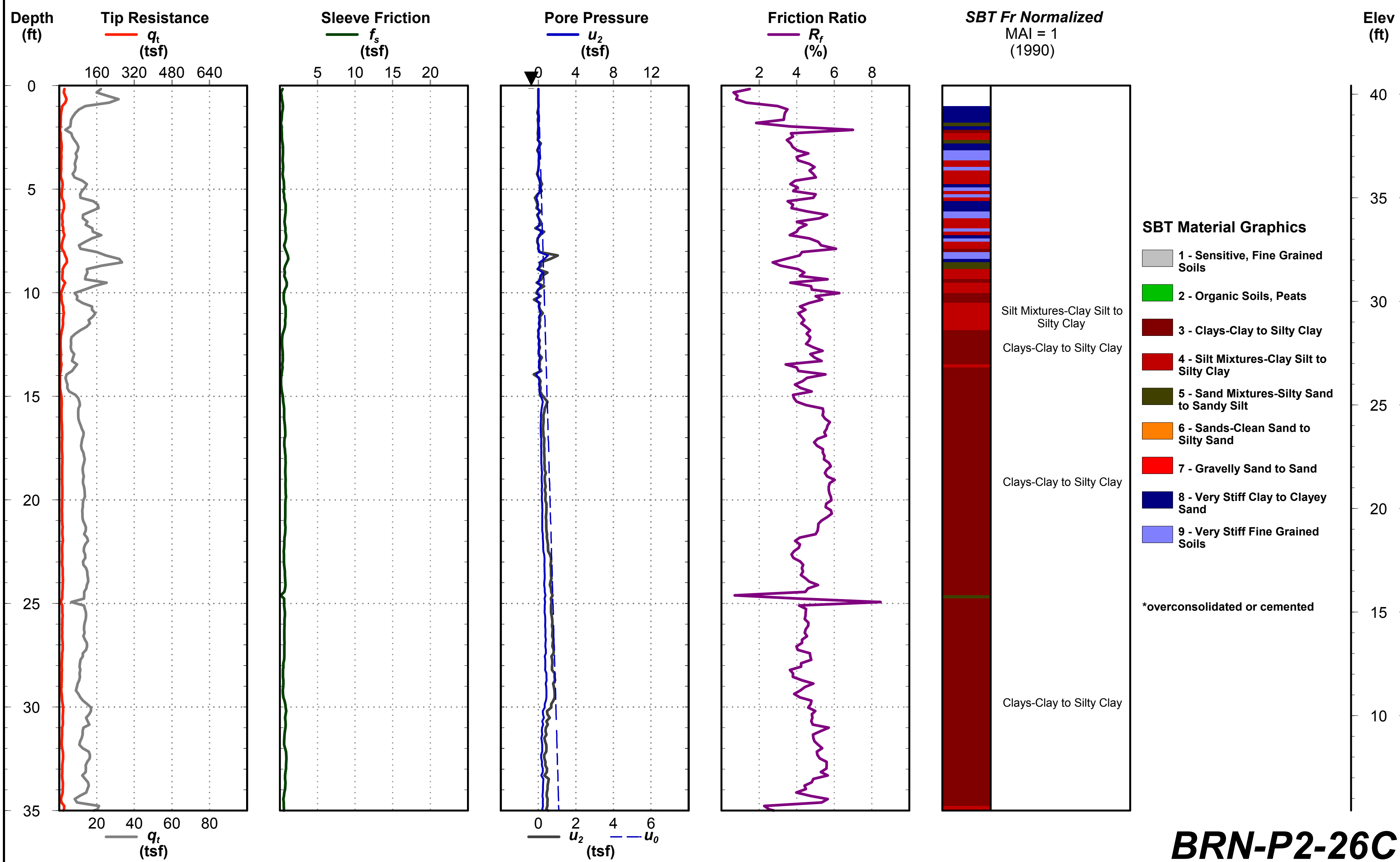
Cone Penetration Test BRN-P2-26C

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

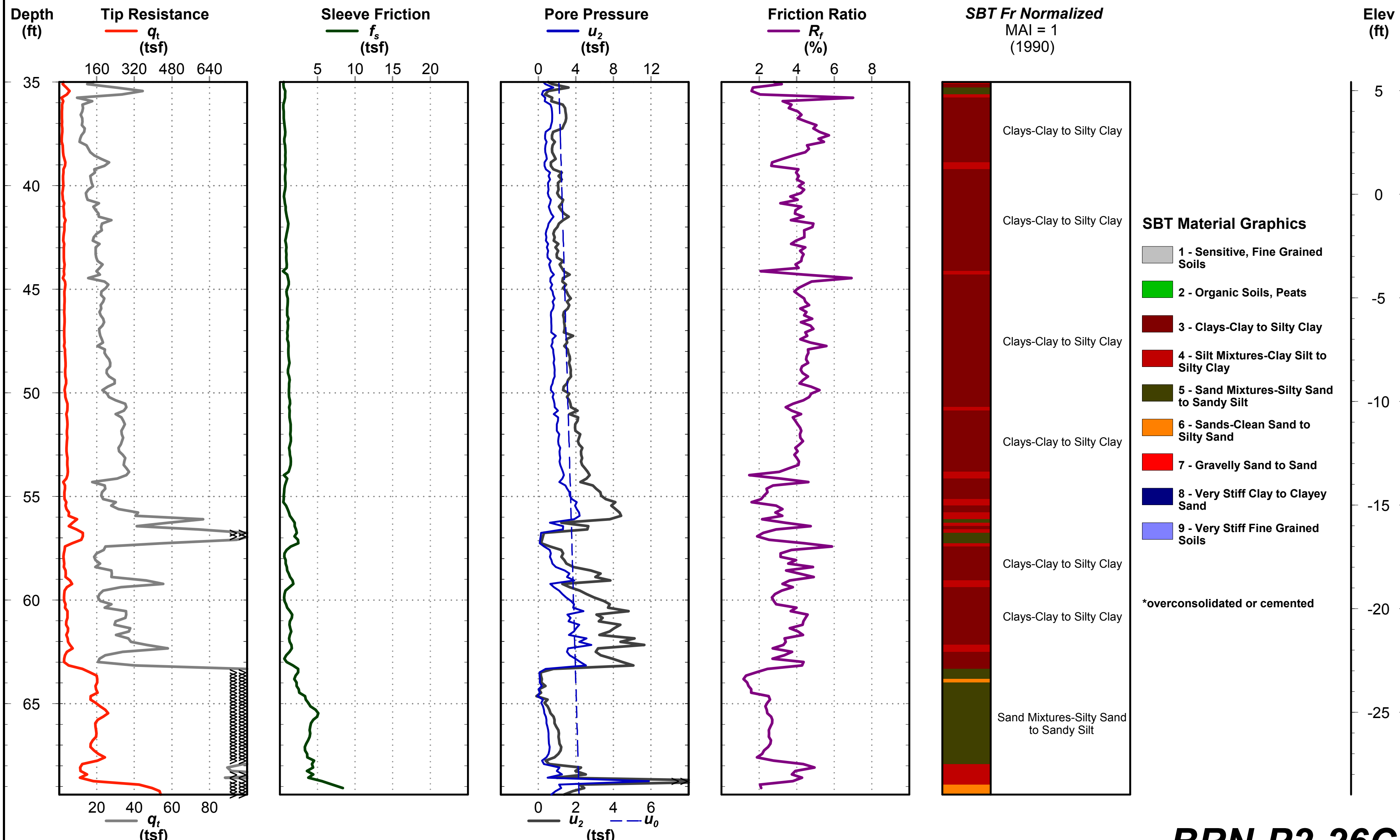
Cone Penetration Test BRN-P2-26C

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

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

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

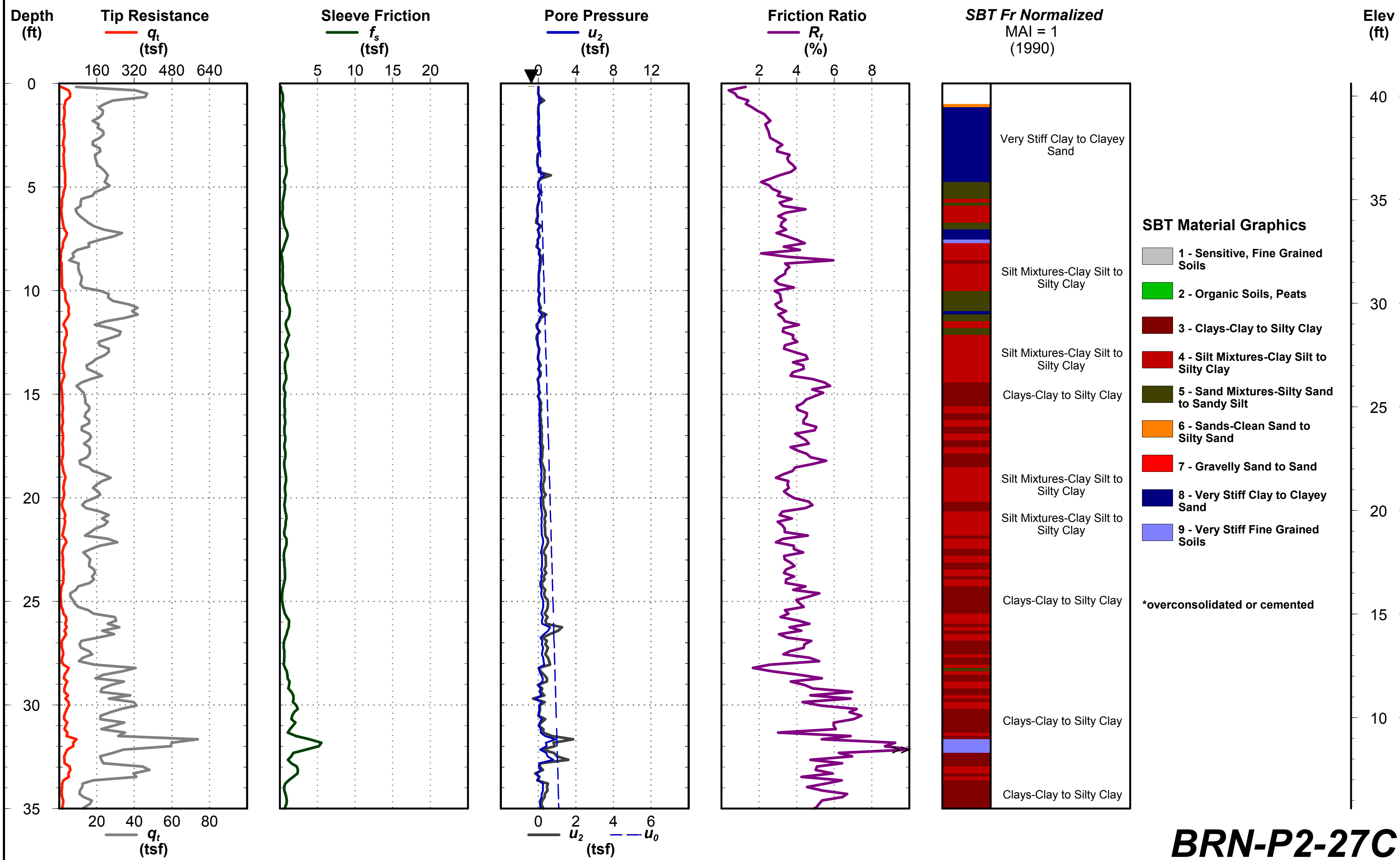
Cone Penetration Test BRN-P2-27C

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:



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

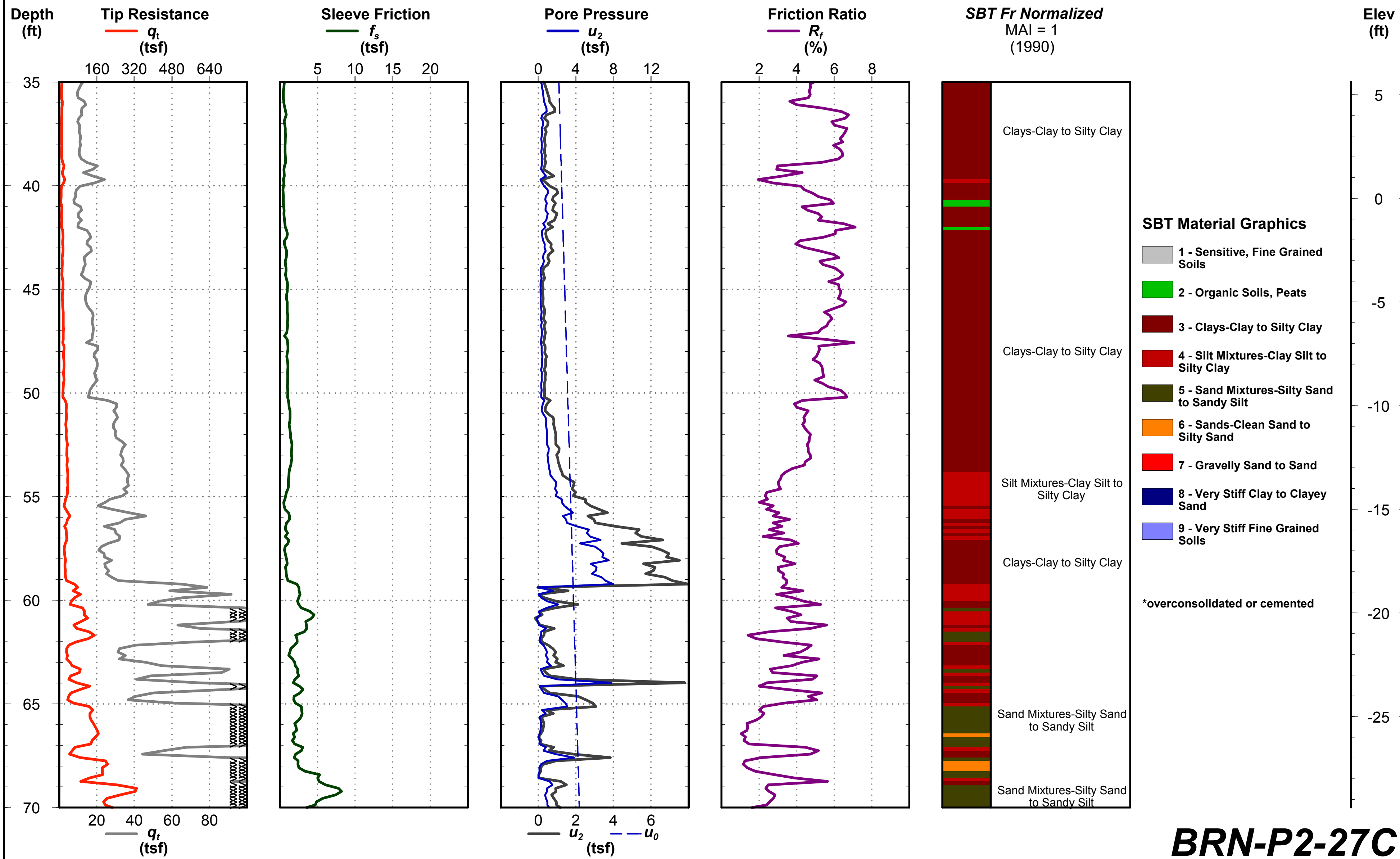
Cone Penetration Test BRN-P2-27C

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:



BRN-P2-27C

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

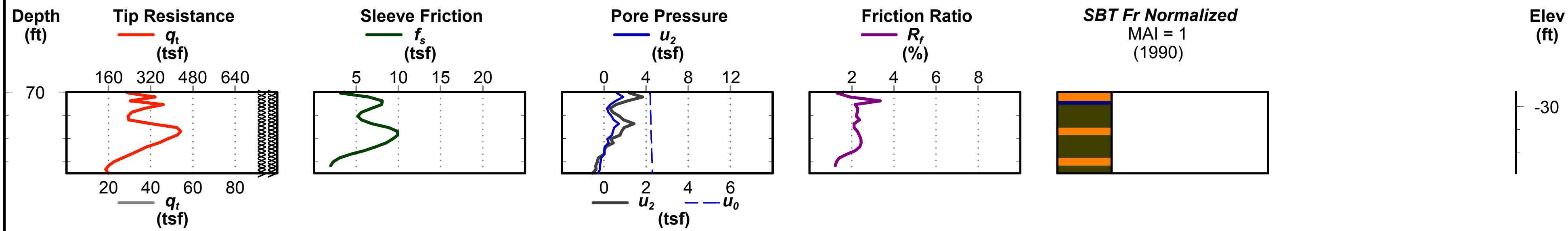
Cone Penetration Test BRN-P2-27C

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

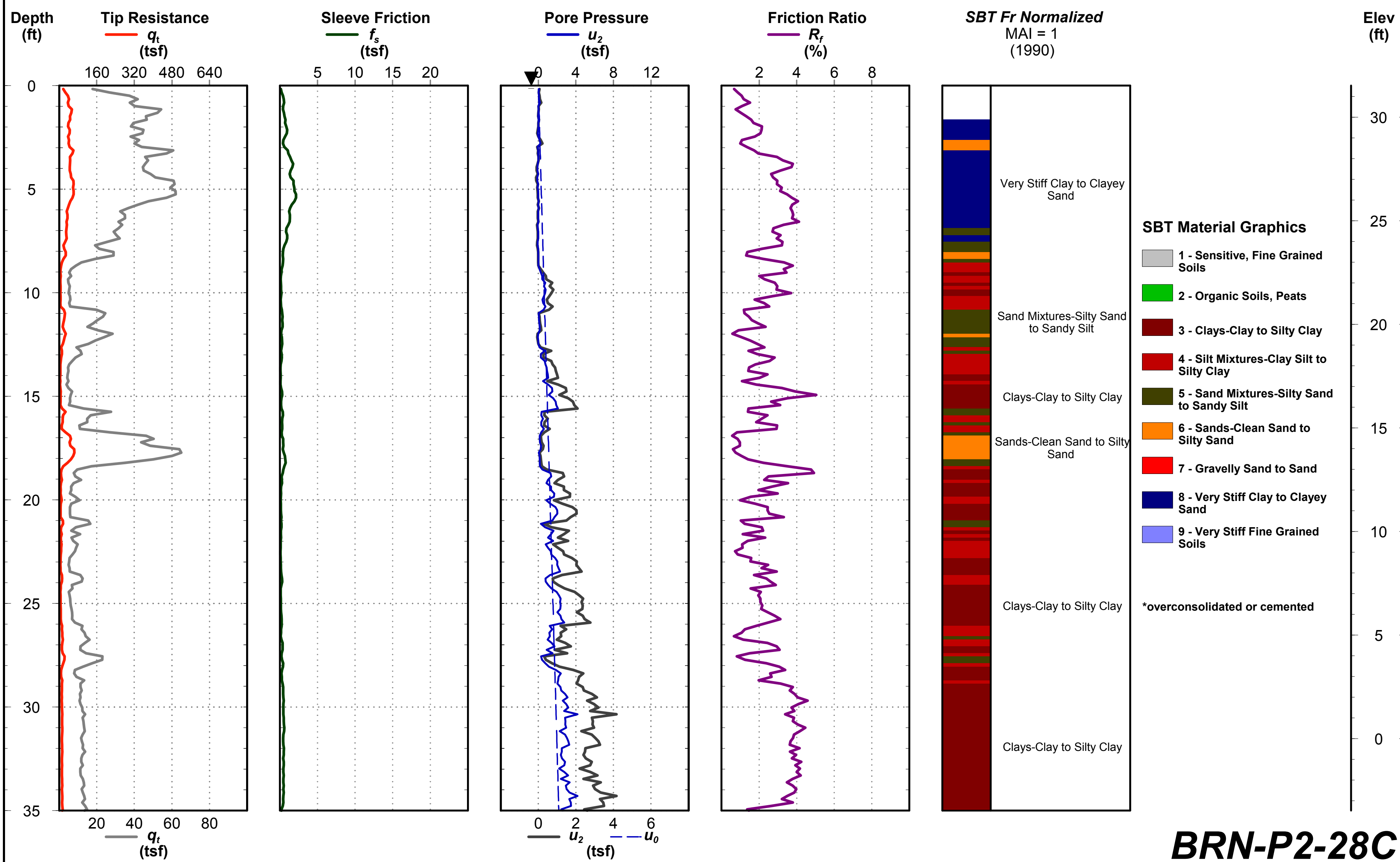
Cone Penetration Test BRN-P2-28C

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:



BRN-P2-28C

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

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

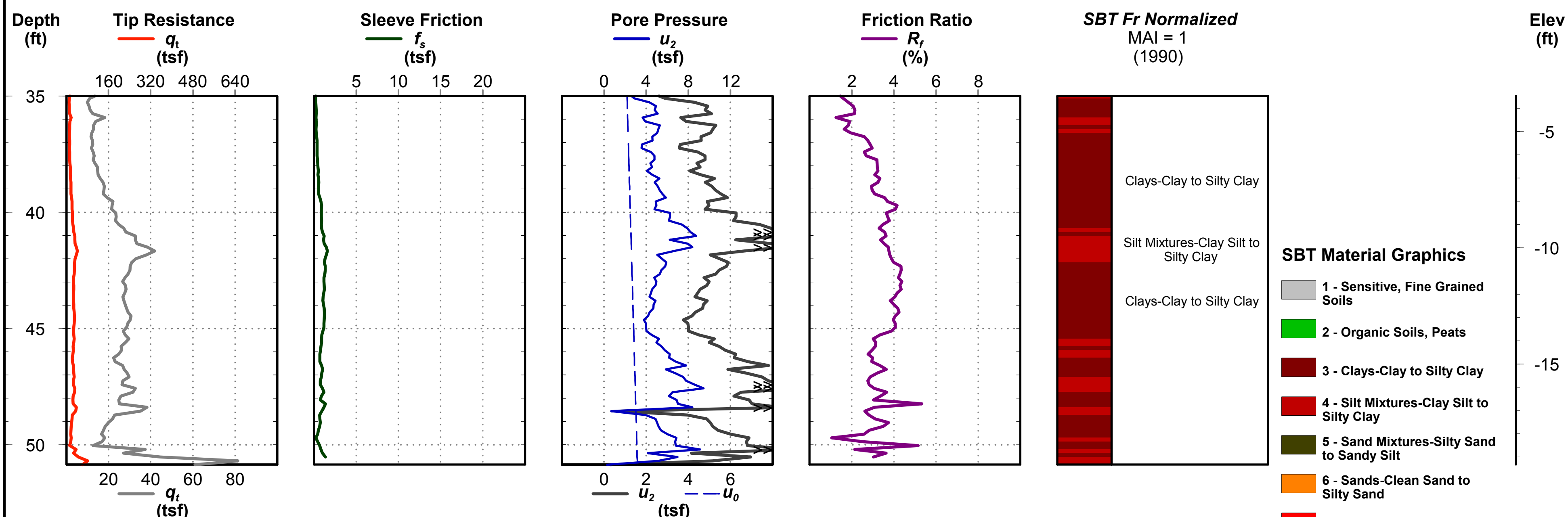
Cone Penetration Test BRN-P2-28C

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:



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-P2-28C

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

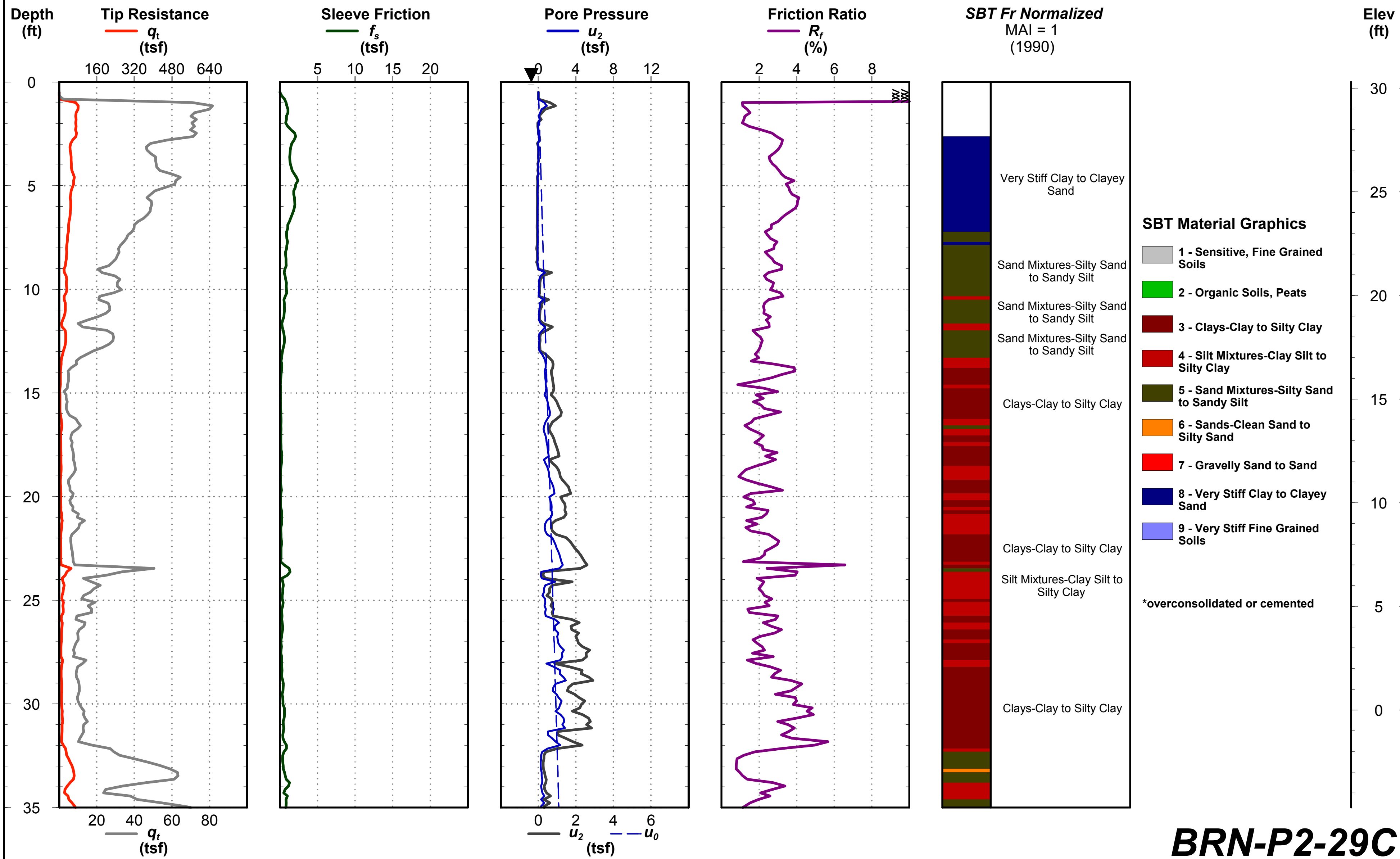
Cone Penetration Test BRN-P2-29C

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

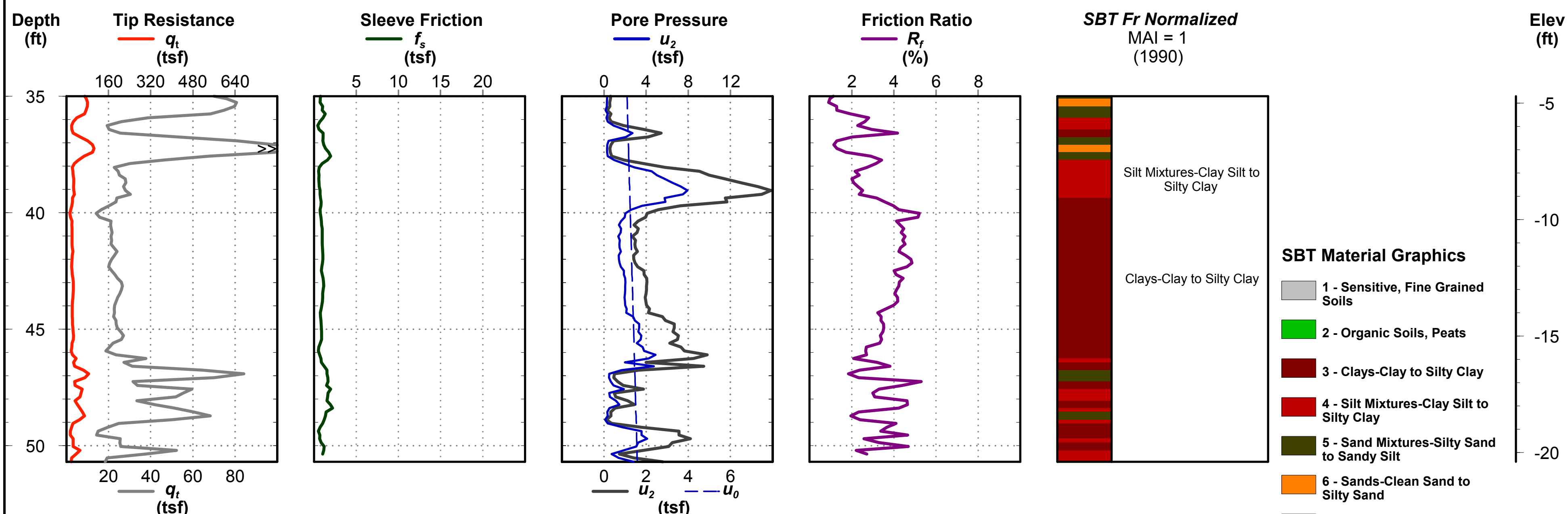
Brownsville, Tx
Project Number :IBWC

Cone Penetration Test BRN-P2-29C

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
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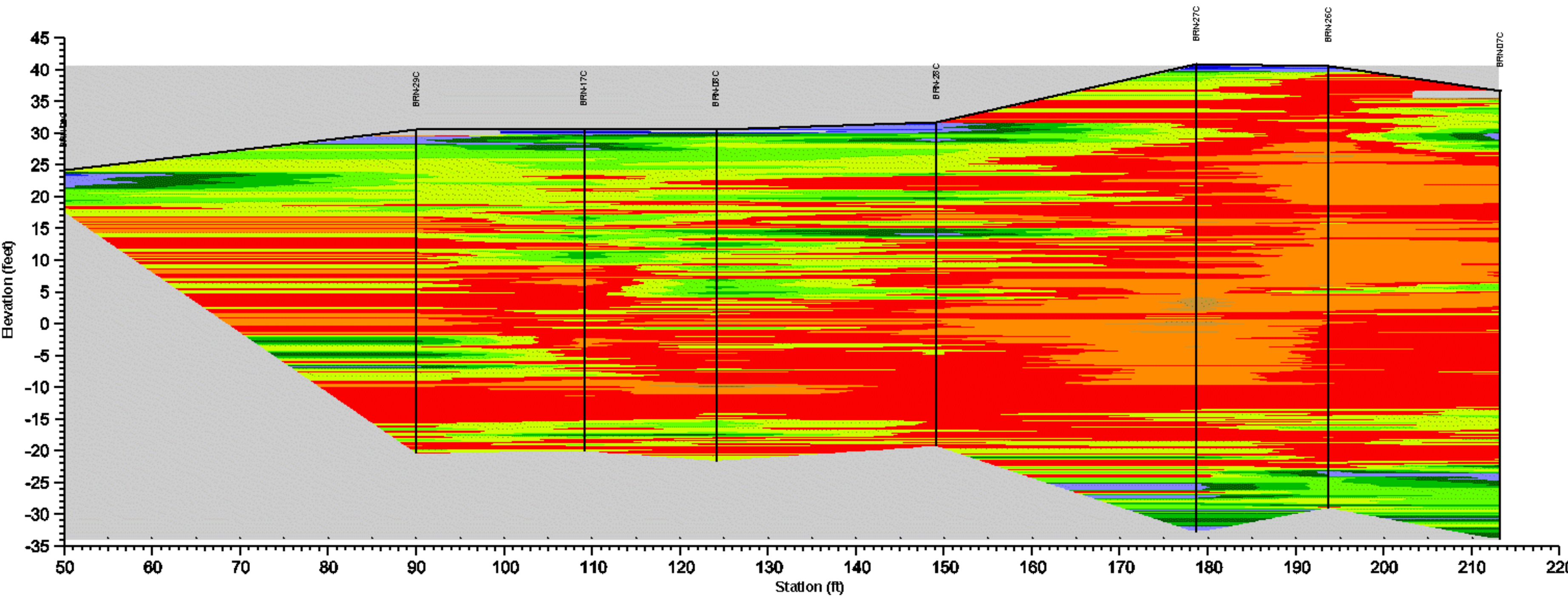
- 6 - Sands-Clean Sand to Silty Sand
- 7 - Gravelly Sand to Sand
- 8 - Very Stiff Clay to Clayey Sand
- 9 - Very Stiff Fine Grained Soils

*overconsolidated or cemented

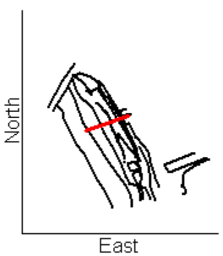
BRN-P2-29C

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

USACE 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

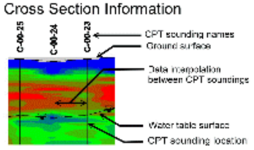
CPT Predicted Soil classification (SCN)

(non linear normalized ERDC 07 version)

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

In Situ anomaly causing data processing error

Cross Section Information

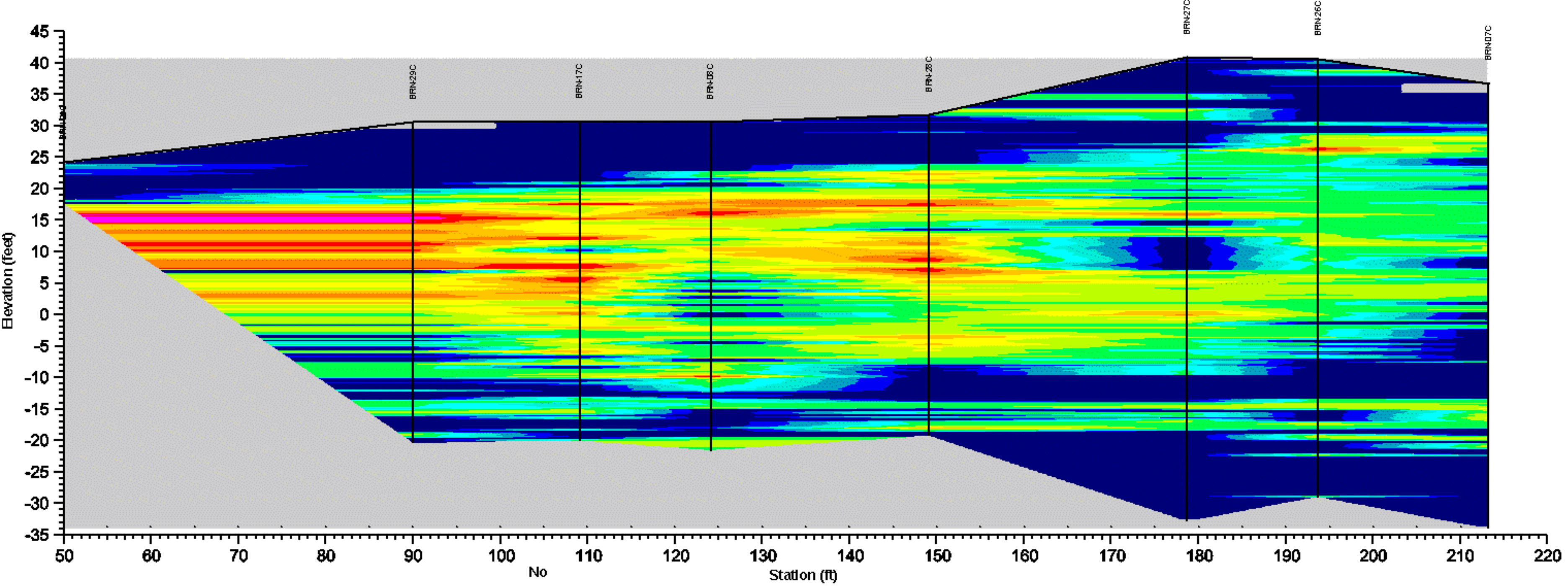


NOTES: CPT cross sections were generated using soil layer tracing to connect soil conditions between CPT soundings. For proper evaluation of a given section, the cross sections of CPT predicted soil type, strength, and normalized strength should be available for comprehensive interpretation. These CPT predicted techniques are based on advanced evaluation techniques developed at USACE ERDC over the last 25 years. Site based verifications 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.

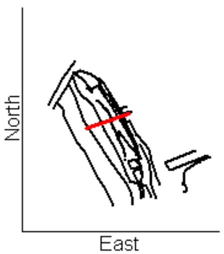
CPT Evaluation

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)

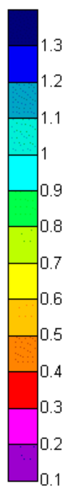


Cross Section:
R1

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

CPT predicted normalized Strength, Su1

(non linear normalized ERDC version)



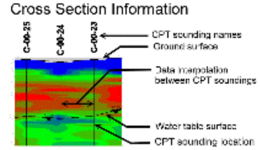
In Situ anomaly causing data processing error

Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information

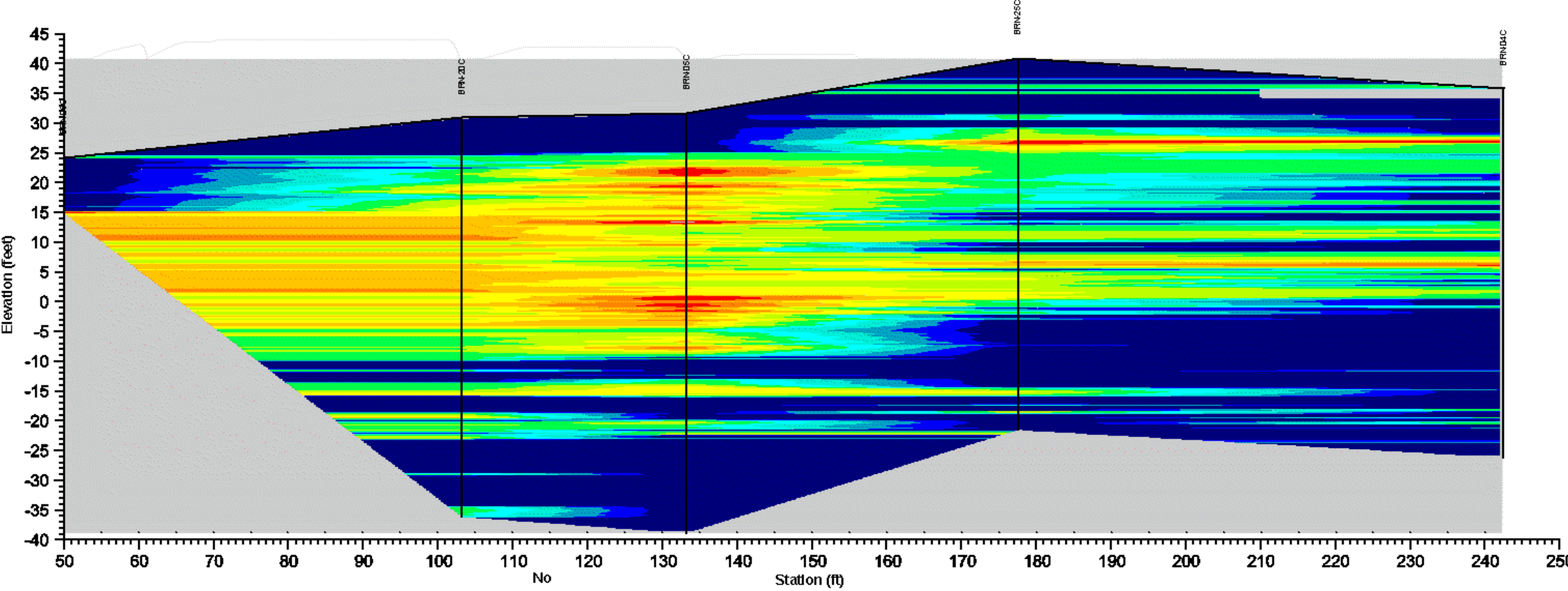


NOTES: CPT cross sections were generated using soil layer tracing to connect soil conditions between CPT soundings. For proper evaluation of a given section, the cross sections of CPT predicted soil type, strength, and normalized strength should be available for comprehensive interpretation. These CPT 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.

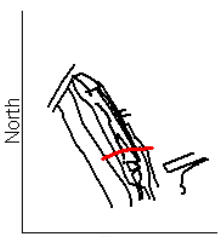
CPT Evaluation

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)



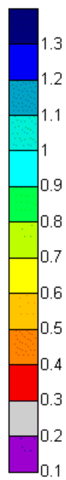
Cross Section:
R2

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

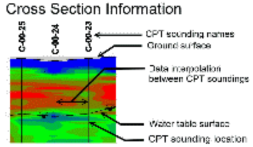


Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information

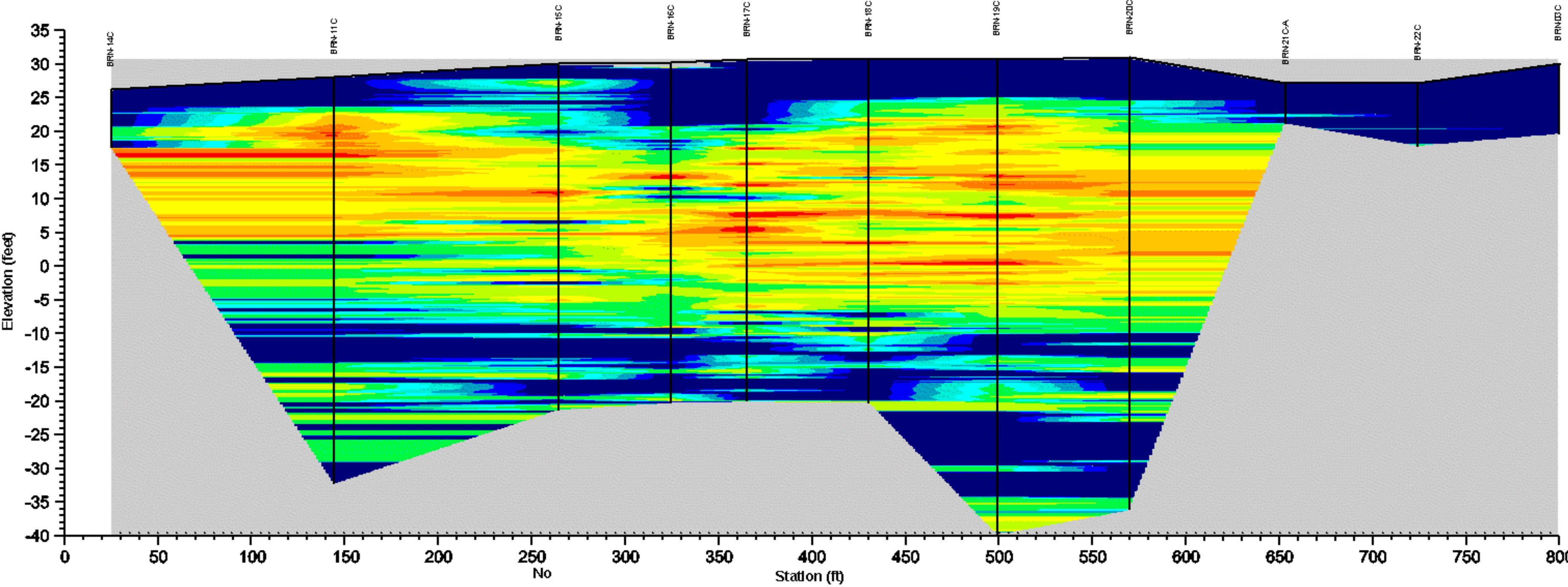


NOTES: CPT cross sections were generated using soil layer tracing to connect soil conditions between CPT soundings. For proper evaluation of a given section, the cross sections of CPT predicted soil type, strength, and normalized strength should be available for comprehensive interpretation. These CPT 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.

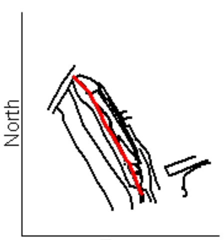
CPT Evaluation

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)

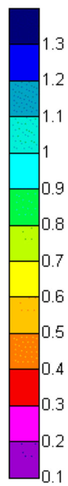


Cross Section:
P1

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

CPT predicted normalized Strength, Su1

(non linear normalized ERDC version)



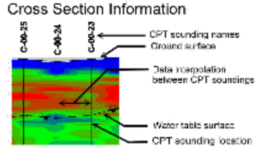
In Situ anomaly causing data processing error

Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information



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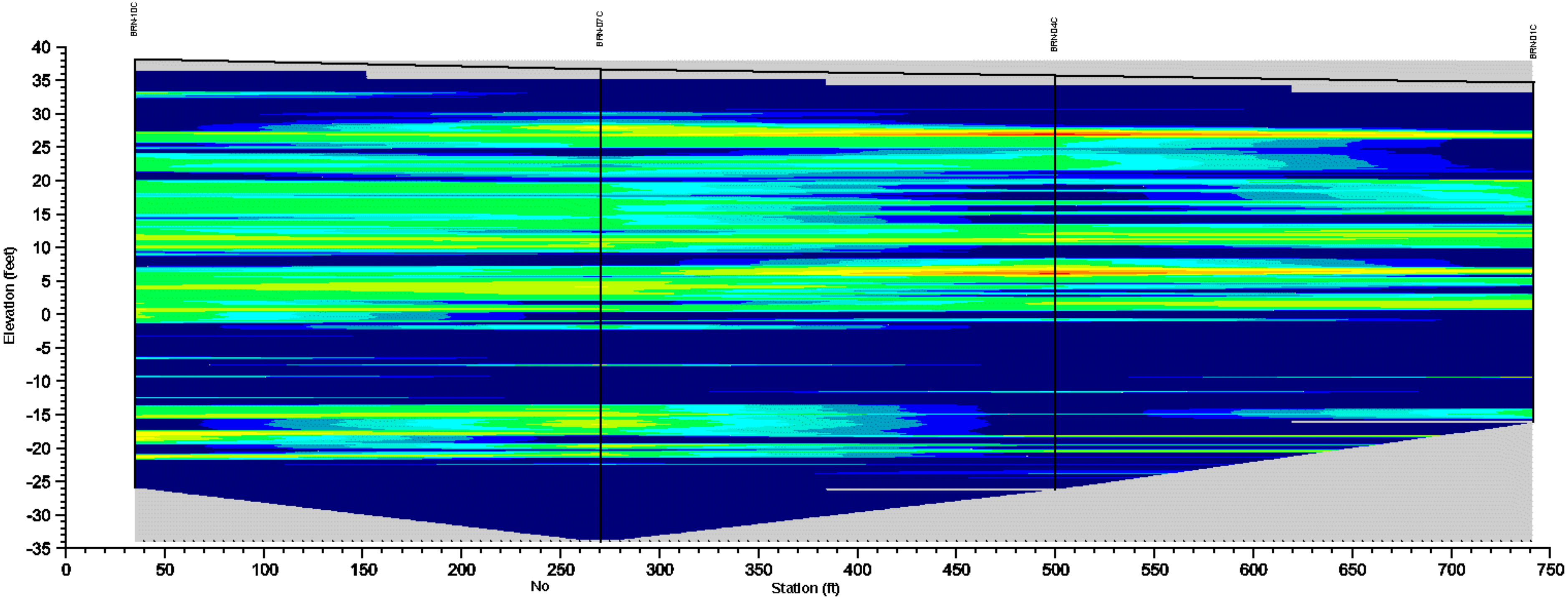
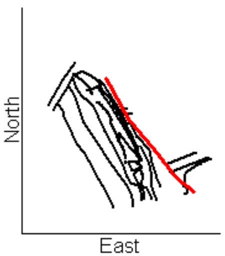
CPT Evaluation

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)

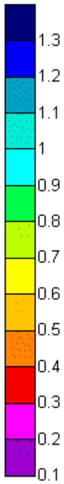


Cross Section:
P2

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

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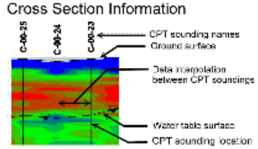
underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information



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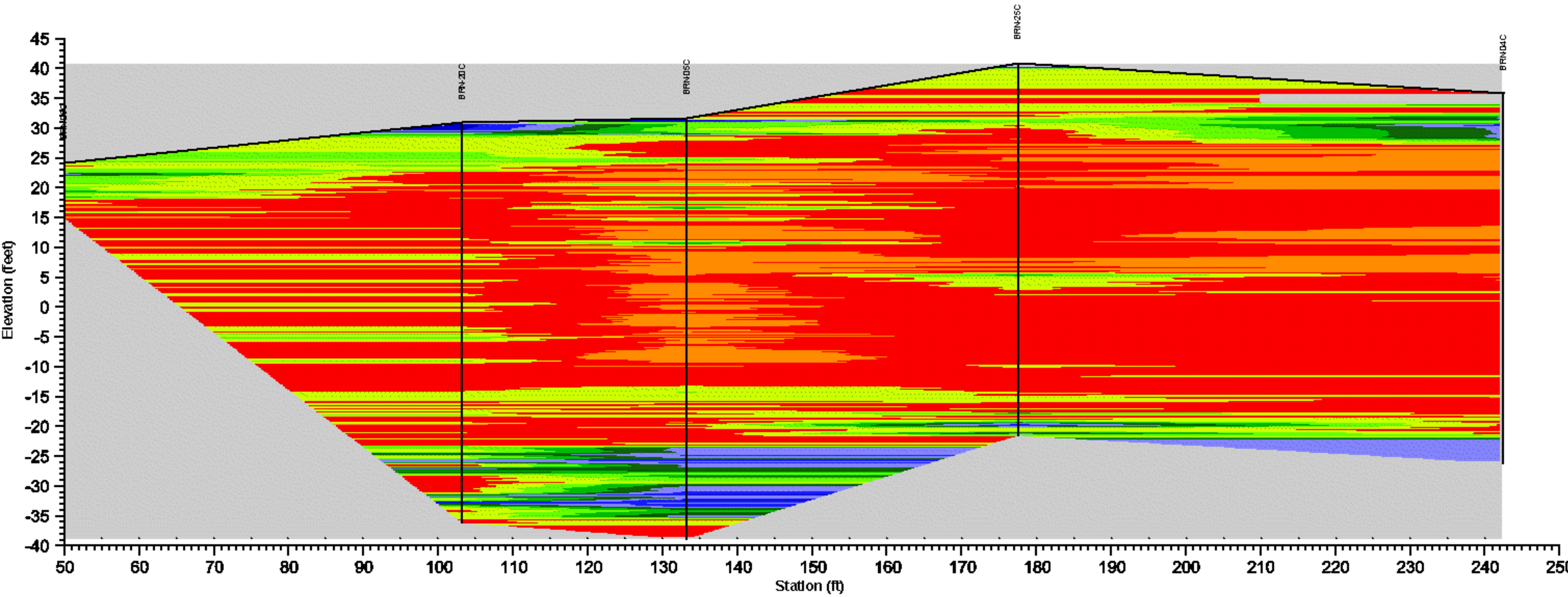
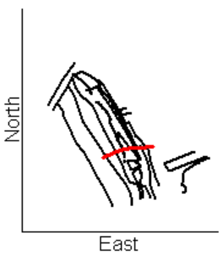
CPT Evaluation

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)

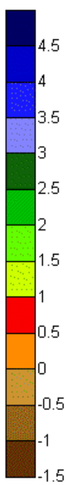


Cross Section:
R2

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

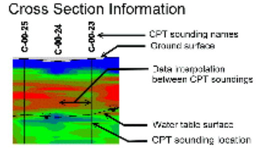
CH and CL clays (and NC organic clays)

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

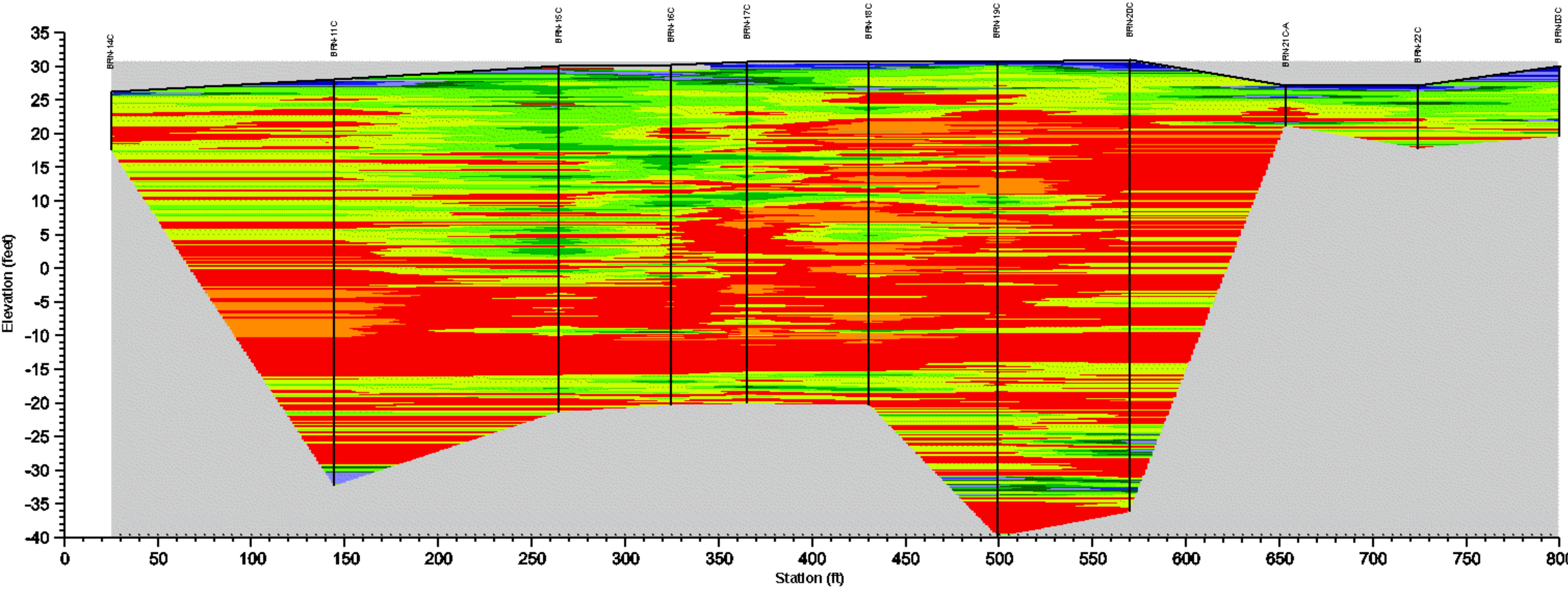
Cross Section of R2

CPT predicted Soil Classification (ERDC version)

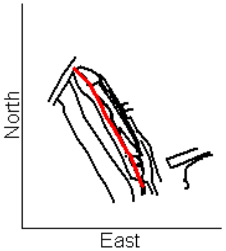
"Isaac Stephens PE, USACE ERDC GSL"

IWBC Brownsville TX

996



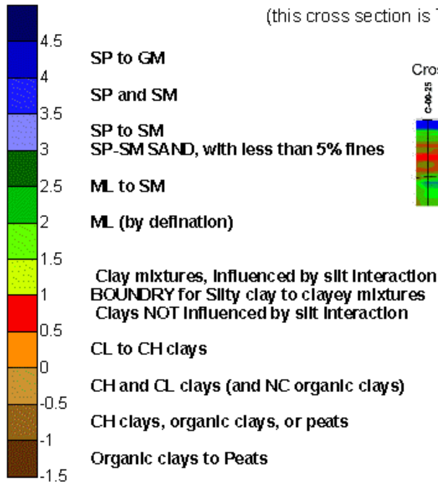
(this cross section is 774 feet long)



Cross Section:
P1

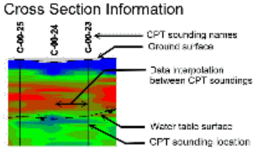
CPT Predicted Soil classification (SCN)

(non linear normalized ERDC 07 version)



In Situ anomaly causing data processing error

Cross Section Information



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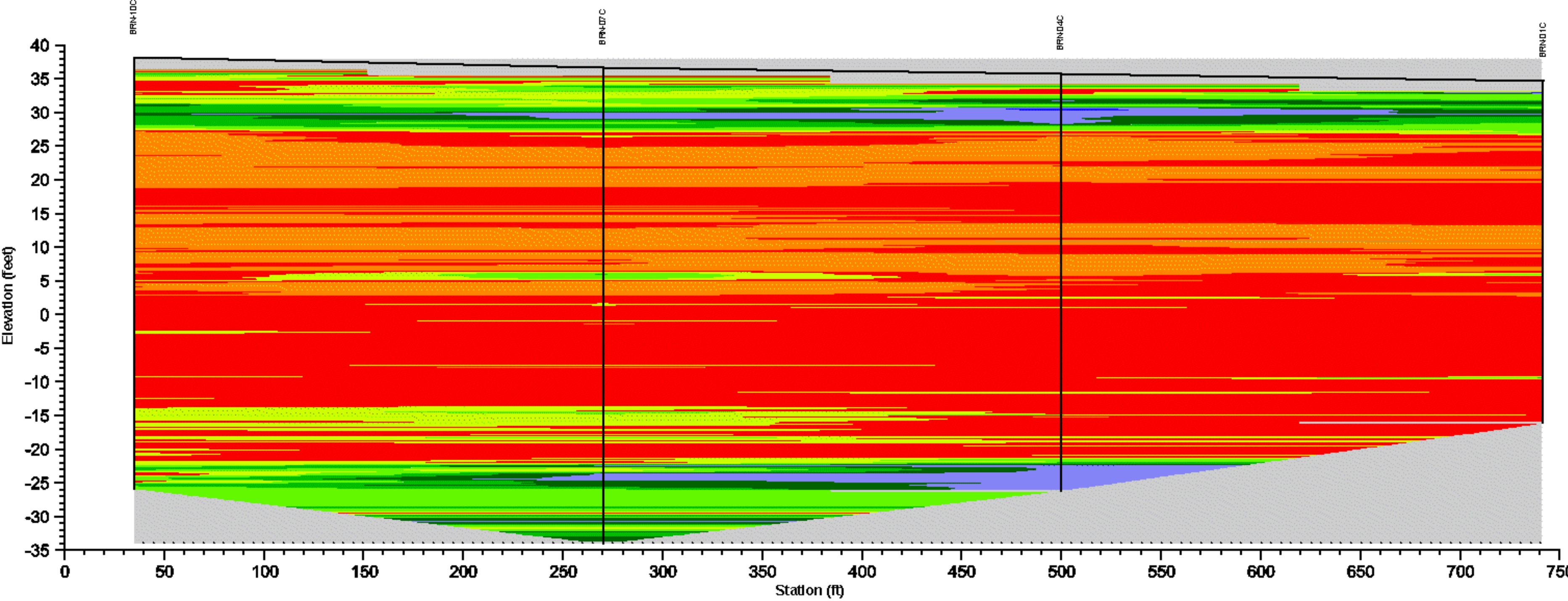
CPT Evaluation

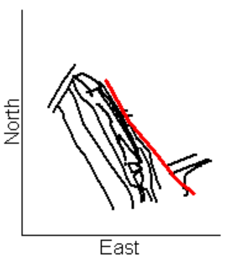
Cross Section of P1
CPT predicted Soil Classification (ERDC version)

"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)

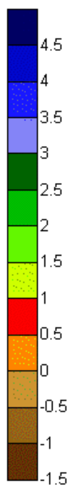


Cross Section:
P2

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)



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
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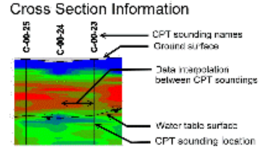
Organic clays to Peats

In Situ anomaly causing data processing error

Cross Section Information



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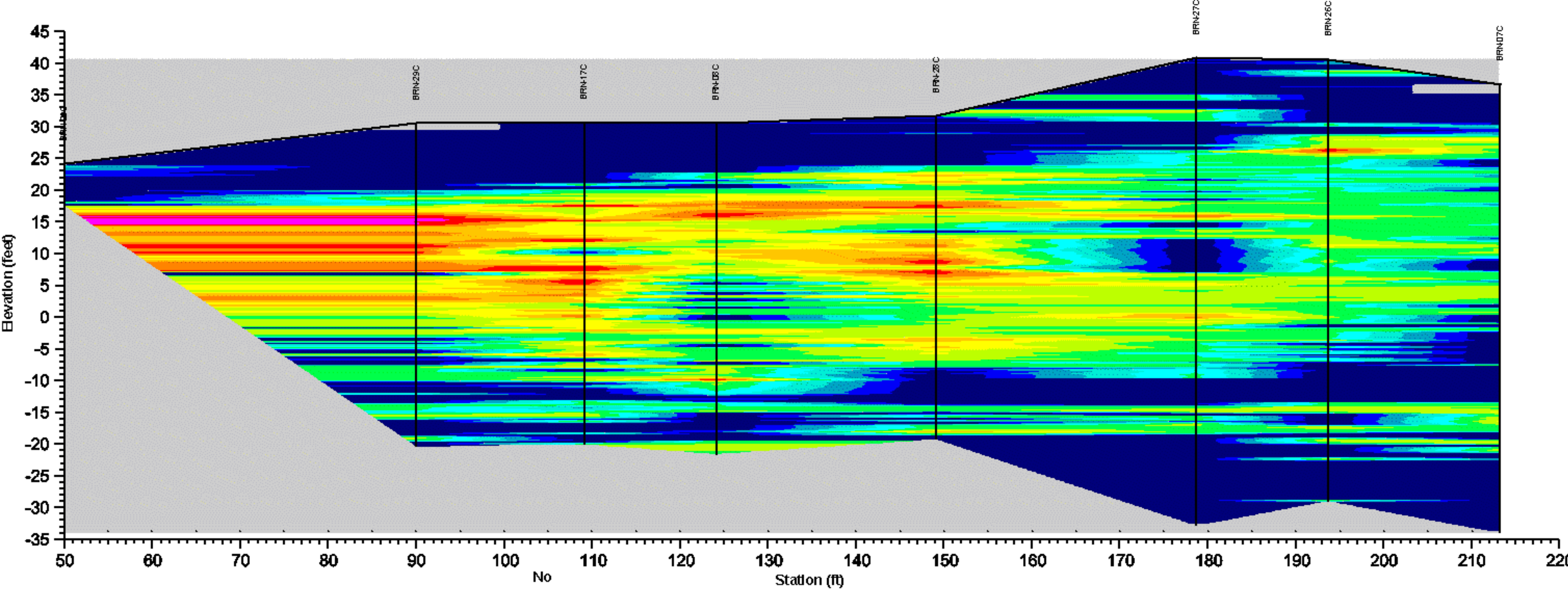
CPT Evaluation

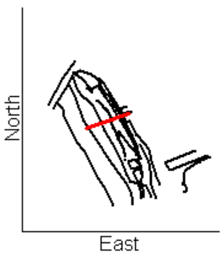
Cross Section of P2
CPT predicted Soil Classification (ERDC version)

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

USACE CPT Predicted Strength



(this cross section is 163 feet long)

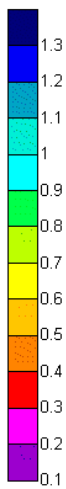


Cross Section:
R1

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

CPT predicted normalized Strength, Su1

(non linear normalized ERDC version)



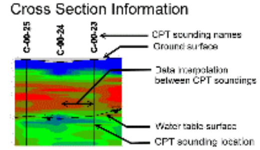
In Situ anomaly causing data processing error

Normally NC to slightly OC clay

underconsolidated or organic clay

highly underconsolidated or high organics

Cross Section Information



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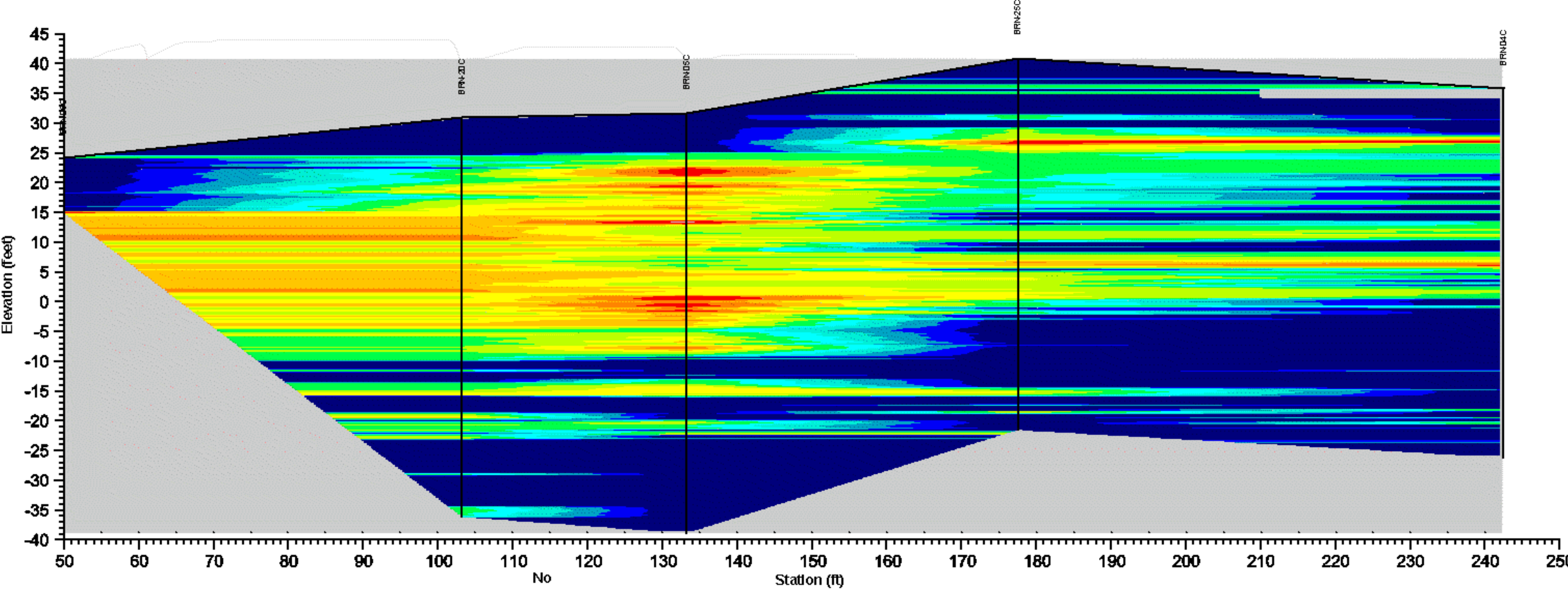
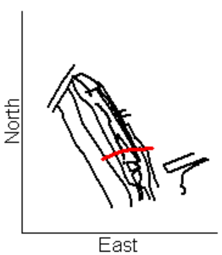
CPT Evaluation

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)

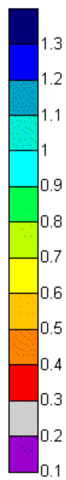


Cross Section:
R2

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

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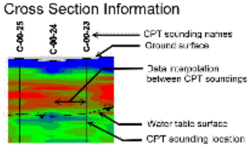
highly underconsolidated or high organics

Cross Section Information



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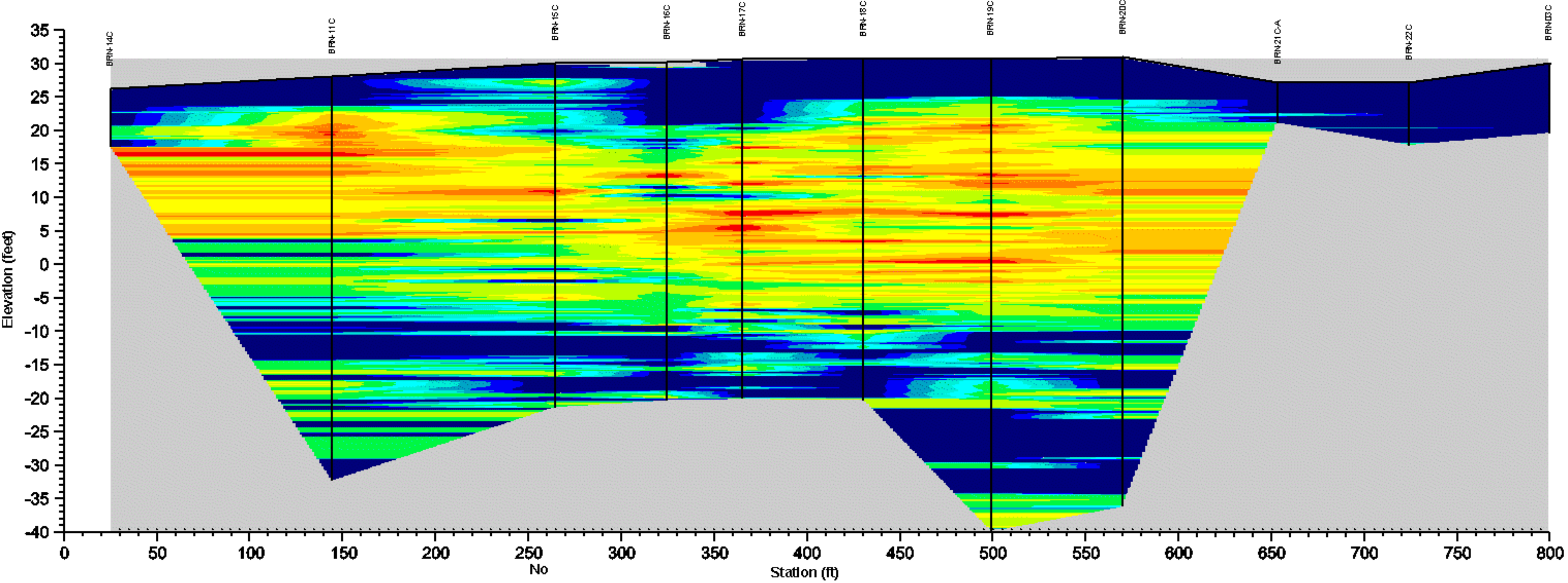
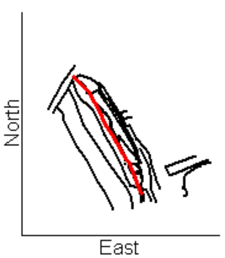
CPT Evaluation

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)



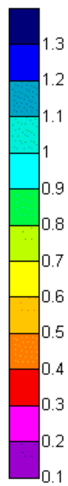
Cross Section:
P1

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

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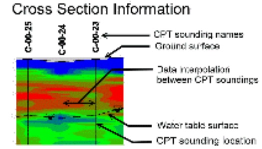
Cross Section Information



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CPT Evaluation

Cross Section of P1
CPT predicted Strength (ERDC version)

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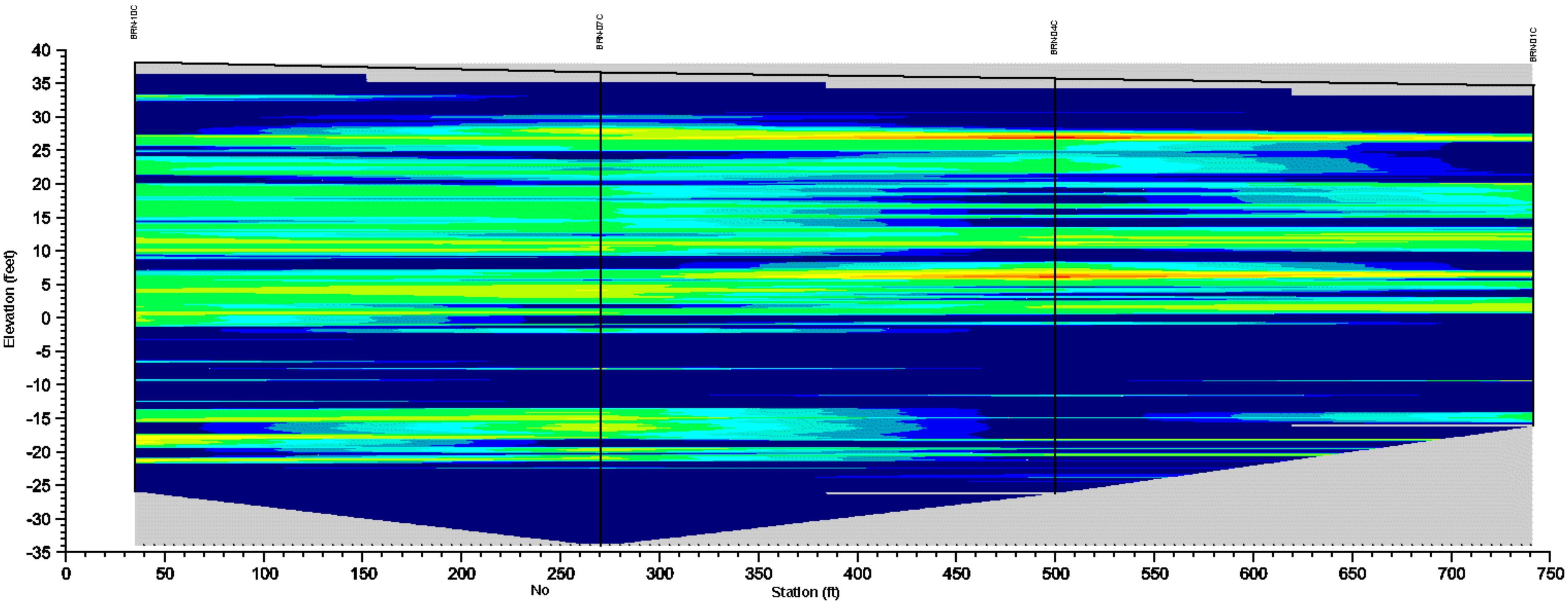
IWBC Brownsville TX



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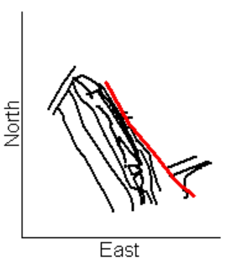
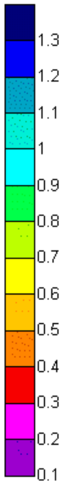


Cross Section:
P2

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

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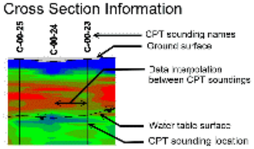
underconsolidated or organic clay

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

APPENDIX E

Geotechnical Laboratory Test Results

ARCADIS LABORATORY RESULTS

LAB TESTING

MOISTURE CONTENT, ATTEBERG LIMITS, #-200 (%) & TORVANE TEST

Upper Levee Segment Geotechnical Lab Testing Results Revised								
PSI Project No. 328-1663								
Boring	Sample Depth (ft)	Sample No.	MC (%)	#-200 (%)	Atterberg Limits			Torvane (tsf)
					LL (%)	PL (%)	PI (%)	
B-1	0 - 1.5	1	^	-	-	-	-	-
B-1	3.5 - 5	2	24.1	93	41	18	23	-
B-1	8 - 10	3	23.0	-	-	-	-	-
B-1	13 - 15	4	20.8	-	37	17	20	-
B-1	18 - 20	5	29.9	-	-	-	-	-
B-1	20 - 22	6	30.4	98	55	23	32	-
B-1	22 - 24	7	23.1	-	-	-	-	-
B-1	24 -26	8	34.1	77	48	32	16	0.275
B-1	26 - 28	9	44.9	-	-	-	-	-
B-1	28 - 30	10	35.0	-	-	-	-	-
B-1	30 - 32	11	30.2	-	52	20	33	0.9
B-1	32 - 34	12	48.9	94	71	21	50	-
B-1	34 - 36	13	22.0	95	51	20	31	-
B-1	36 - 38	14	27.0	-	-	-	-	-
B-1	38 - 40	15	26.7	-	-	-	-	-
B-1	40 - 42	16	26.4	-	-	-	-	-
B-1	44 - 46	17	25.8	99	48	20	29	-
B-1	46 - 48	18	26.6	-	-	-	-	-
B-1	48 - 49.5	19	24.0	-	-	-	-	-
B-1	53 - 55	20	27.7	-	-	-	-	-
B-1	58 - 60	21	23.1	99	30	15	15	-
B-1	68.5 - 70	22	31.5	96	54	15	40	-
B-1	73.5 - 75	23	32.9	59	28	17	11	-
B-1	78.5 - 80	24	28.4	-	-	-	-	-
B-1	83.5 - 85	25	26.0	84	27	17	10	-
B-1	88.5 - 90	26	25.6	-	-	-	-	-
B-1	93.5 - 95	27	25.5	73	23	17	6	-
B-1	98.5 - 100	28	24.5	-	-	-	-	-

Note: (*) No sample recovery; (-) Not tested; (^) Not enough sample for assigned test

Upper Levee Segment Geotechnical Lab Testing Results Revised								
PSI Project No. 328-1663								
Boring	Sample Depth (ft)	Sample No.	MC (%)	#-200 (%)	Atterberg Limits			Torvane (tsf)
					LL (%)	PL (%)	PI (%)	
B-2	0 - 1.5	1	10.0	-	-	-	-	-
B-2	4.5 - 6	2	16.5	-	-	-	-	-
B-2	8.5 - 10	3	31.4	88	27	17	10	-
B-2	13.5 - 15	4	34.4	-	-	-	-	-
B-2	18.5 - 20	5	27.0	62	-	-	-	-
B-2	24 - 25.5	6	31.7	54	28	13	16	-
B-2	26 - 27.5	7	31.4	-	-	-	-	-
B-2	28 - 29.5	8	27.0	78	30	14	17	-
B-2	30 - 31.5	9	27.6	-	-	-	-	-
B-2	32 - 33.5	10	36.3	-	54	27	28	-
B-2	34 - 36	11	31.4	97	63	22	41	-
B-2	36 - 38	12	30.1	99	58	13	45	-
B-2	38 - 40	13	32.2	80	39	16	23	-
B-2	40 - 42	14	34.6	94	62	23	39	-
B-2	42 - 44	15	25.9	99	72	31	41	-
B-2	44 - 46	16	26.5	-	-	-	-	-
B-2	46.5 - 48	17	11.0	-	-	-	-	-
B-2	48.5 - 50	18	26.2	100	31	17	14	-
B-2	53.5 - 55	19	38.3	-	-	-	-	-
B-2	58.5 - 60	20	25.5	-	-	-	-	-
B-2	63.5 - 65	21	29.6	99	47	20	26	-
B-2	68.5 - 70	22	27.0	-	-	-	-	-
B-2	73 - 75	23	23.3	99	38	16	23	-
B-2	78 - 80	24	25.7	-	-	-	-	-

Note: (*) No sample recovery; (-) Not tested; (^) Not enough sample for assigned test

Upper Levee Segment Geotechnical Lab Testing Results Revised								
PSI Project No. 328-1663								
Boring	Sample Depth (ft)	Sample No.	MC (%)	#-200 (%)	Atterberg Limits			Torvane (tsf)
					LL (%)	PL (%)	PI (%)	
B-3	0 - 1.5	1	26.0	-	-	-	-	-
B-3	3.5 - 5	2	1.5	-	-	-	-	-
B-3	8.5 - 10	3	22.9	76	28	15	13	-
B-3	13.5 - 15	4	33.0	-	-	-	-	-
B-3	18.5 - 20	5	28.4	75	28	15	13	-
B-3	20 - 22	6	31.9	-	-	-	-	-
B-3	22 - 23.5	7	0.0	67	-	-	-	-
B-3	24 - 25.5	8	60.6	-	53	20	33	-
B-3-2	26.5 - 28	9	42.5	-	-	-	-	-
B-3-2	28.5 - 30	10	33.4	85	-	-	-	-
B-3-2	30 - 32	11	33.1	98	55	22	32	-
B-3-2	32 - 34	12	38.7	99	55	25	31	-
B-3-2	36 - 38	13	26.0	99	44	17	27	-
B-3-3	38 - 40	14	25.4	-	-	-	-	-
B-3-3	40 - 42	15	26.8	100	66	20	46	-
B-3-3	42 - 44	16	26.1	-	-	-	-	-
B-3-3	44 - 46	17	24.9	-	36	15	21	-
B-3-3	48 - 50	18	25.8	100	-	-	-	-
B-3-3	53.5 - 55	19	25.7		54	24	30	-
B-3-3	58 - 60	20	11.0	33	-	-	-	-
B-3-3	63.5 - 65	21	28.7	-	-	-	-	-
B-3-3	68.5 - 70	22	25.6	99	65	26	39	-
B-3-3	73 - 75	23	23.8	-	-	-	-	-
B-3-3	78 - 80	24	28.9	-	-	-	-	-

Note: (*) No sample recovery; (-) Not tested; (^) Not enough sample for assigned test

Upper Levee Segment Geotechnical Lab Testing Results Revised								
PSI Project No. 328-1663								
Boring	Sample Depth (ft)	Sample No.	MC (%)	#-200 (%)	Atterberg Limits			Torvane (tsf)
					LL (%)	PL (%)	PI (%)	
B-4	0 - 1.5	1	11.9	-	-	-	-	-
B-4	3.5 - 5	2	14.8	-	32	16	18	-
B-4	8.5 - 10	3	19.2	75	-	-	-	-
B-4	13.5 - 15	4	24.0	-	-	-	-	-
B-4	18.5 - 20	5	27.7	64	28	20	8	-
B-4	20 - 21.5	6	23.4	-	-	-	-	-
B-4	22 - 23.5	7	27.2	-	-	-	-	-
B-4	24 - 25.5	8	31.2	-	-	-	-	-
B-4	26 - 27.5	9	31.1	99	42	18	23	-
B-4	28 - 30	10	25.4	-	-	-	-	-
B-4	30 - 32	11	24.6	-	-	-	-	0.94
B-4	32 - 34	12	*	*	*	*	*	*
B-4	34 - 36	13	28.5	98	70	20	50	-
B-4	36 - 38	14	26.4	-	-	-	-	-
B-4	38 - 40	15	39.4	-	-	-	-	-
B-4	40 - 42	16	26.0	-	-	-	-	-
B-4	42 - 44	17	22.6	98	47	17	31	-
B-4	44.5 - 46	18	24.6	-	-	-	-	-
B-4	46 - 48	19	26.9	-	-	-	-	-
B-4	48 - 50	20	27.7	-	52	20	32	-
B-4	53 - 55	21	34.8	-	-	-	-	-
B-4	58 - 60	22	24.3	99	53	17	36	1.25
B-4	63 - 65	23	24.0	-	-	-	-	-
B-4	68 - 70	24	25.0	-	59	14	45	-
B-4	73 - 75	25	25.4	99	-	-	-	-
B-4	78 - 80	26	*	*	*	*	*	*

Note: (*) No sample recovery; (-) Not tested; (^) Not enough sample for assigned test

LAB TESTING

UNCONFINED COMPRESSION TEST

Unconfined Compression Test ASTM D 2166

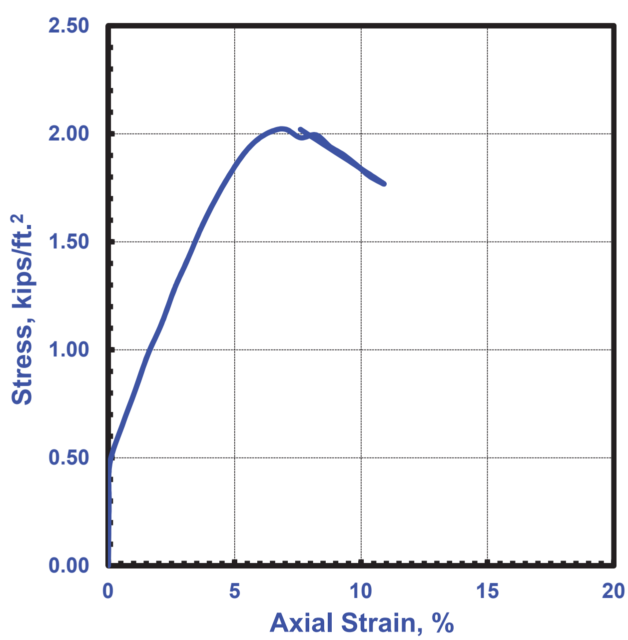
Test Date: *7/6/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-1*	**Depth, feet:**	*8-10'*	**Sample No./ID:**	*3*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen\Torvane:	*2.25*	**% Passing 200:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Dark Brown Silty Clay*			**Sample Condition:**	*Undisturbed*

Oven: 02OV328 **Load Cell:** NA	**Scale:** 01DS328 **DCDT** NA	**Calipers:** 2DC328 **Load Frame:** MG2092

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.640	2.670	2.700	2.670	1.22
Height (H), in.:	3.260	3.260	3.260	3.260	
Area, ft^2:	0.039	Volume, ft^3:	0.011	Seating:	

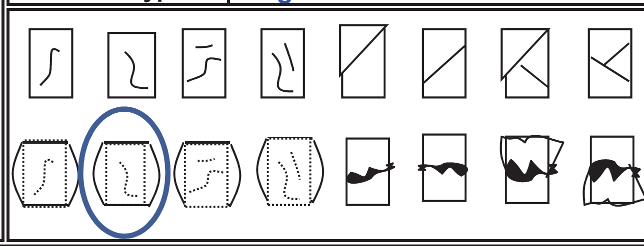
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	KK	NA	Wet Weight, gms:	568.70	568.70
Tare Wt., gm:	11.11	NA	Dry Weight, gms:	462.17	
Wet Wt+Tare, gm:	52.11	NA			
Dry Wt.+Tare, gm:	44.43	NA	Wet Unit Wt.,pcf	118.7	NA
Wt. of Water, gm:	7.68	NA	Dry Unit Wt, pcf:	96.5	NA
Dry Soil Wt., gm:	33.32	NA	Void Ratio, pcf:	0.747	NA
Moisture:	23.0%	NA	Saturation, %:	83.4%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	7.02
Max. Compressive Stress, ksf:	2.02
Max. Compressive Stress, tsf:	1.01
Max. Compressive Stress, psi:	14.0
Undrained Shear Strength (ksf):	**1.01**

Failure Type: Bulge



Remarks:					
Tested By:	PC	Computed By:	RM	Checked By:	HJL

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Unconfined Compression Test ASTM D 2166

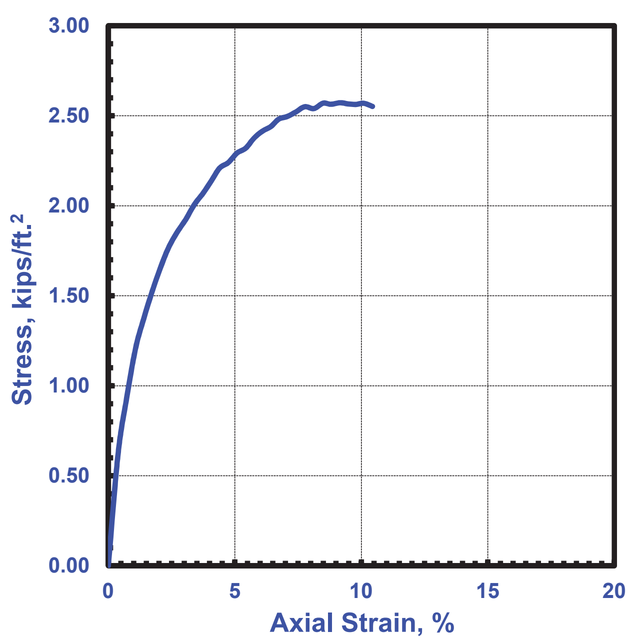
Test Date: *7/6/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-1*	**Depth, feet:**	*18-20'*	**Sample No./ID:**	*5*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen\Torvane:	*2.00*	**% Passing 200:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Dark Brown Silty Clay*			**Sample Condition:**	*Undisturbed*

Oven:	02OV328	**Scale:**	01DS328	**Calipers:**	2DC328
Load Cell:	NA	**DCDT**	NA	**Load Frame:**	MG2092

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.510	2.490	2.530	2.510	2.08
Height (H), in.:	5.230	5.230	5.230	5.230	
Area, ft^2:	0.034	Volume, ft^3:	0.015	Seating:	

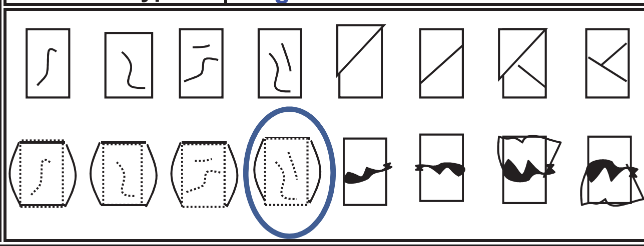
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	501	NA	Wet Weight, gms:	855.00	855.00
Tare Wt., gm:	15.74	NA	Dry Weight, gms:	658.18	
Wet Wt+Tare, gm:	64.48	NA			
Dry Wt.+Tare, gm:	53.26	NA	Wet Unit Wt.,pcf	125.9	NA
Wt. of Water, gm:	11.22	NA	Dry Unit Wt, pcf:	96.9	NA
Dry Soil Wt., gm:	37.52	NA	Void Ratio, pcf:	0.739	NA
Moisture:	29.9%	NA	Saturation, %:	109.3%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	9.16
Max. Compressive Stress, ksf:	2.57
Max. Compressive Stress, tsf:	1.29
Max. Compressive Stress, psi:	17.9
Undrained Shear Strength (ksf):	**1.29**

Failure Type: Bulge



Remarks:

Tested By:	PC	Computed By:	RM	Checked By:	HJL

Unconfined Compression Test ASTM D 2166

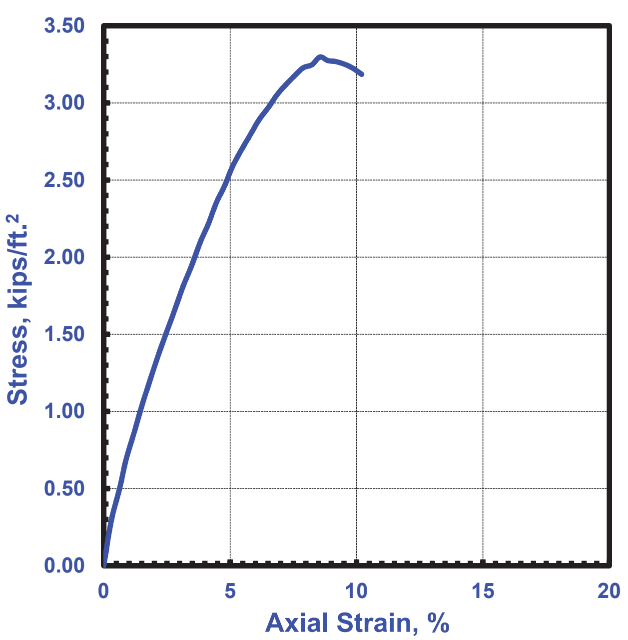
Test Date: *7/6/2016*

Project Name:	*Upper Levee Segment*			Project Number:	*0328-1663*
Boring Number:	*B-1*	Depth, feet:	*44-46'*	Sample No./ID:	*17*
Liquid Limit:	*48*	Plastic Limit:	*20*	Plasticity Index:	*29*
Hand Pen\Torvane:	*2.00*	% Passing 200:	*NA*	Specific Gravity:	*2.700*
Classification:	*Medium Brown Clay*			Sample Condition:	*Undisturbed*

Oven: 02OV328	Scale: 01DS328	Calipers: 2DC328
Load Cell: NA	DCDT NA	Load Frame: MG2092

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.660	2.670	2.720	2.683	2.03
Height (H), in.:	5.450	5.450	5.450	5.450	
Area, ft^2:	0.039	Volume, ft^3:	0.018	Seating:	

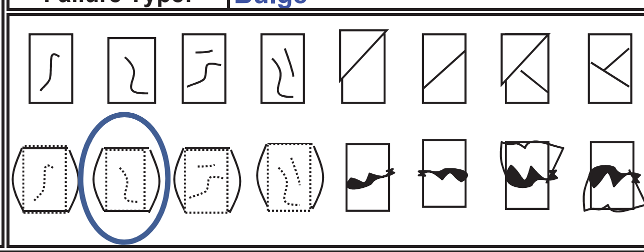
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	AAA	NA	Wet Weight, gms:	855.00	855.00
Tare Wt., gm:	11.05	NA	Dry Weight, gms:	679.80	
Wet Wt+Tare, gm:	67.27	NA			
Dry Wt.+Tare, gm:	55.75	NA	Wet Unit Wt.,pcf	105.7	NA
Wt. of Water, gm:	11.52	NA	Dry Unit Wt, pcf:	84.0	NA
Dry Soil Wt., gm:	44.7	NA	Void Ratio, pcf:	1.005	NA
Moisture:	25.8%	NA	Saturation, %:	69.2%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	8.55
Max. Compressive Stress, ksf:	3.30
Max. Compressive Stress, tsf:	1.65
Max. Compressive Stress, psi:	22.9
Undrained Shear Strength (ksf):	1.65

Failure Type: Bulge



Remarks:

Tested By:	PC	Computed By:	RM	Checked By:	HJL

1 of 1

Unconfined Compression Test ASTM D 2166

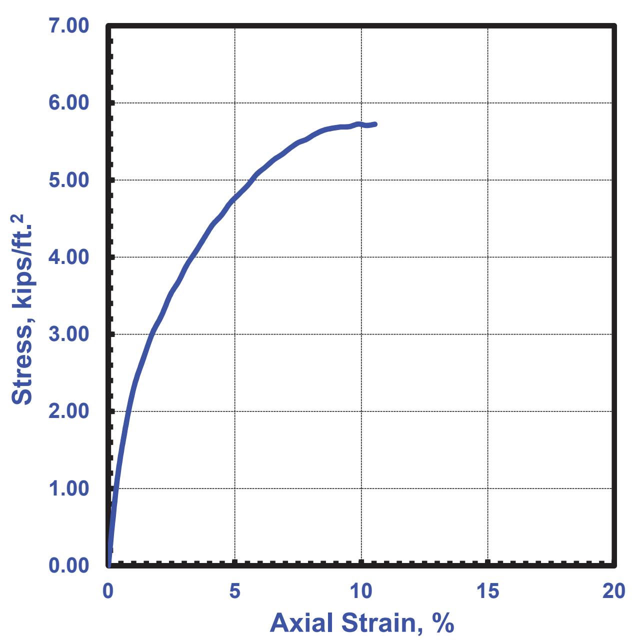
Test Date: *7/6/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-1*	**Depth, feet:**	*53-55'*	**Sample No./ID:**	*20*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen\Torvane:	*4.00*	**% Passing 200:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Clay*			**Sample Condition:**	*Undisturbed*

Oven:	02OV328	**Scale:**	01DS328	**Calipers:**	2DC328
Load Cell:	NA	**DCDT**	NA	**Load Frame:**	MG2092

Sample Data	**1**	**2**	**3**	**Average**	**Height/Dia. Ratio**
Diameter (D),in.:	2.690	2.650	2.660	2.667	1.89
Height (H), in.:	5.030	5.030	5.030	5.030	
Area, ft^2:	0.039	**Volume, ft^3:**	0.016	**Seating:**	

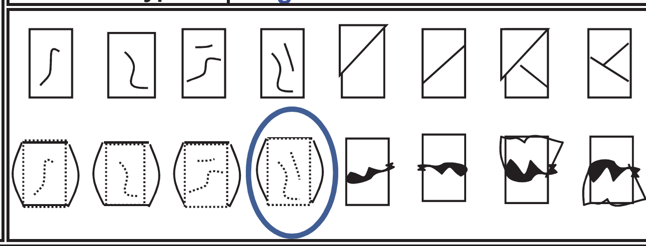
Mositure Data	**Before (Trimings)**	**After Test (Middle)**	**Specimen**	**Before**	**After**
Tare ID:	80	NA	**Wet Weight, gms:**	905.90	905.90
Tare Wt., gm:	15.6	NA	**Dry Weight, gms:**	709.59	
Wet Wt+Tare, gm:	60.5	NA			
Dry Wt.+Tare, gm:	50.77	NA	**Wet Unit Wt.,pcf**	122.8	NA
Wt. of Water, gm:	9.73	NA	**Dry Unit Wt, pcf:**	96.2	NA
Dry Soil Wt., gm:	35.17	NA	**Void Ratio, pcf:**	0.751	NA
Moisture:	27.7%	NA	**Saturation, %:**	99.5%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	9.88
Max. Compressive Stress, ksf:	5.73
Max. Compressive Stress, tsf:	2.86
Max. Compressive Stress, psi:	39.8
Undrained Shear Strength (ksf):	2.86

Failure Type: Bulge



Remarks:					
Tested By:	PC	Computed By:	RM	Checked By:	HJL

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Unconfined Compression Test ASTM D 2166

Test Date: 7/6/2016

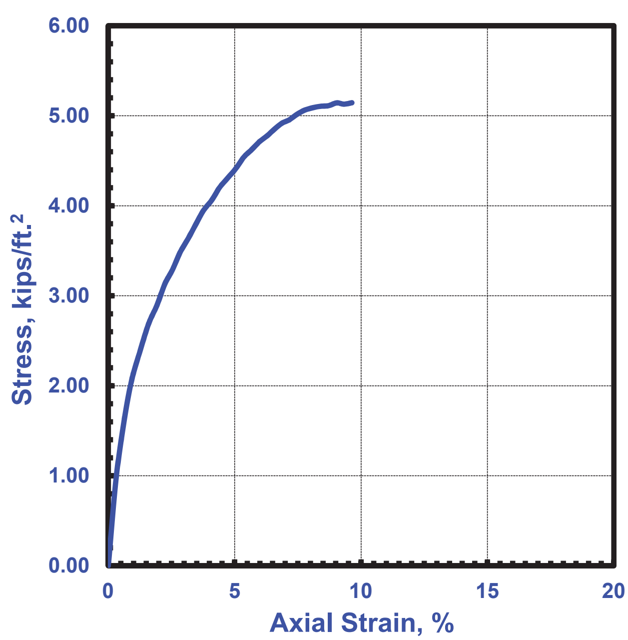
Project Name:	Upper Levee Segment			Project Number:	0328-1663
Boring Number:	B-2	Depth, feet:	42-44'	Sample No./ID:	15
Liquid Limit:	72	Plastic Limit:	31	Plasticity Index:	41
Hand Pen\Torvane:	4.00	% Passing 200:	94	Specific Gravity:	2.700
Classification:	Light Brown Clay			Sample Condition:	Undisturbed

Oven:	02OV328	Scale:	01DS328	Calipers:	2DC328
Load Cell:	NA	DCDT	NA	Load Frame:	MG2092

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.840	2.830	2.810	2.827	1.95
Height (H), in.:	5.500	5.500	5.500	5.500	
Area, ft^2:	0.044	Volume, ft^3:	0.020	Seating:	

Mositure Data	Before (Trimmings)	After Test (Middle)
Tare ID:	Z70	NA
Tare Wt., gm:	10.99	NA
Wet Wt+Tare, gm:	56.54	NA
Dry Wt.+Tare, gm:	47.16	NA
Wt. of Water, gm:	9.38	NA
Dry Soil Wt., gm:	36.17	NA
Moisture:	25.9%	NA

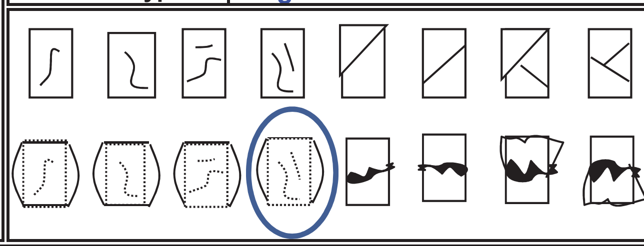
Specimen	Before	After
Wet Weight, gms:	1146.44	1146.44
Dry Weight, gms:	910.36	
Wet Unit Wt.,pcf	126.5	NA
Dry Unit Wt, pcf:	100.5	NA
Void Ratio, pcf:	0.677	NA
Saturation, %:	103.5%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	9.64
Max. Compressive Stress, ksf:	5.14
Max. Compressive Stress, tsf:	2.57
Max. Compressive Stress, psi:	35.7
Undrained Shear Strength (ksf):	2.57

Failure Type: Bulge



Remarks:

Tested By:	PC	Computed By:	RM	Checked By:	HJL

Unconfined Compression Test ASTM D 2166

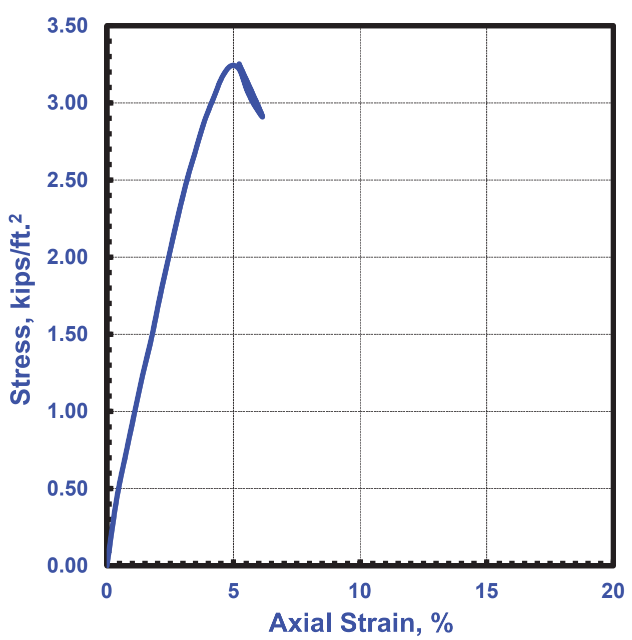
Test Date: *7/6/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-2*	**Depth, feet:**	*78-80'*	**Sample No./ID:**	*24*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen\Torvane:	*3.50*	**% Passing 200:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Silty Clay*			**Sample Condition:**	*Undisturbed*

Oven:	02OV328	**Scale:**	01DS328	**Calipers:**	2DC328
Load Cell:	NA	**DCDT**	NA	**Load Frame:**	MG2092

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.670	2.730	2.720	2.707	1.91
Height (H), in.:	5.170	5.170	5.170	5.170	
Area, ft^2:	0.040	Volume, ft^3:	0.017	Seating:	

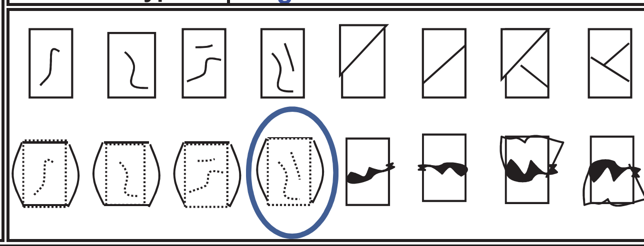
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	235	NA	Wet Weight, gms:	1078.90	1078.90
Tare Wt., gm:	15.92	NA	Dry Weight, gms:	858.02	
Wet Wt+Tare, gm:	59.88	NA			
Dry Wt.+Tare, gm:	50.88	NA	Wet Unit Wt.,pcf	138.2	NA
Wt. of Water, gm:	9	NA	Dry Unit Wt, pcf:	109.9	NA
Dry Soil Wt., gm:	34.96	NA	Void Ratio, pcf:	0.533	NA
Moisture:	25.7%	NA	Saturation, %:	130.3%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	5.22
Max. Compressive Stress, ksf:	3.25
Max. Compressive Stress, tsf:	1.63
Max. Compressive Stress, psi:	22.6
Undrained Shear Strength (ksf):	**1.63**

Failure Type: Bulge



Remarks:					
Tested By:	PC	Computed By:	RM	Checked By:	HJL

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1 of 1

Unconfined Compression Test ASTM D 2166

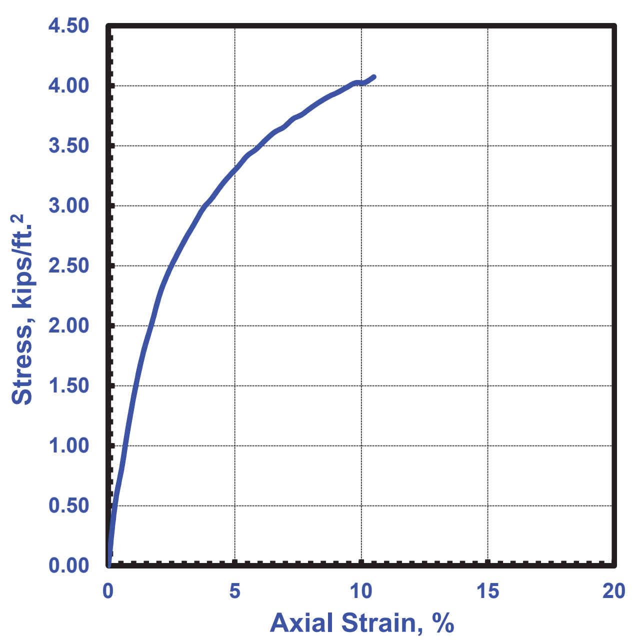
Test Date: *7/7/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-3*	**Depth, feet:**	*42-44'*	**Sample No./ID:**	*16*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen\Torvane:	*3.50*	**% Passing 200:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Clay*			**Sample Condition:**	*Undisturbed*

Oven: 02OV328	**Scale:** 01DS328	**Calipers:** 2DC328
Load Cell: NA	**DCDT** NA	**Load Frame:** MG2092

Sample Data	**1**	**2**	**3**	**Average**	**Height/Dia. Ratio**
Diameter (D),in.:	2.700	2.700	2.690	2.697	1.96
Height (H), in.:	5.290	5.290	5.290	5.290	
Area, ft^2:	0.040	**Volume, ft^3:**	0.017	**Seating:**	

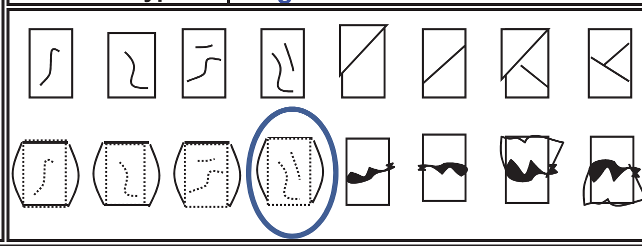
Mositure Data	**Before (Trimings)**	**After Test (Middle)**	**Specimen**	**Before**	**After**
Tare ID:	FF	NA	**Wet Weight, gms:**	980.90	980.90
Tare Wt., gm:	11.26	NA	**Dry Weight, gms:**	777.58	
Wet Wt+Tare, gm:	71.18	NA			
Dry Wt.+Tare, gm:	58.76	NA	**Wet Unit Wt.,pcf**	123.7	NA
Wt. of Water, gm:	12.42	NA	**Dry Unit Wt, pcf:**	98.0	NA
Dry Soil Wt., gm:	47.5	NA	**Void Ratio, pcf:**	0.718	NA
Moisture:	26.1%	NA	**Saturation, %:**	98.3%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	10.49
Max. Compressive Stress, ksf:	4.07
Max. Compressive Stress, tsf:	2.04
Max. Compressive Stress, psi:	28.3
Undrained Shear Strength (ksf):	2.04

Failure Type: Bulge



Remarks:					
Tested By:	PC	Computed By:	RM	Checked By:	HJL

psi

Unconfined Compression Test ASTM D 2166

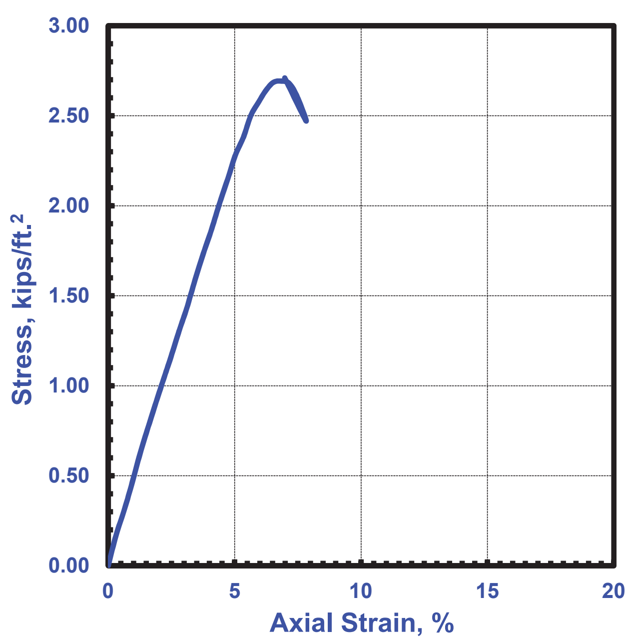
Test Date: *7/7/2016*

Project Name:	*Upper Levee Segment*			Project Number:	*0328-1663*
Boring Number:	*B-3*	Depth, feet:	*73-75'*	Sample No./ID:	*23*
Liquid Limit:	*NA*	Plastic Limit:	*NA*	Plasticity Index:	*NA*
Hand Pen\Torvane:	*2.75*	% Passing 200:	*NA*	Specific Gravity:	*2.700*
Classification:	*Medium Brown Silty Clay*			Sample Condition:	*Undisturbed*

Oven: 02OV328	Scale: 01DS328	Calipers: 2DC328
Load Cell: NA	DCDT NA	Load Frame: MG2092

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.680	2.680	2.700	2.687	2.05
Height (H), in.:	5.500	5.500	5.500	5.500	
Area, ft^2:	0.039	Volume, ft^3:	0.018	Seating:	

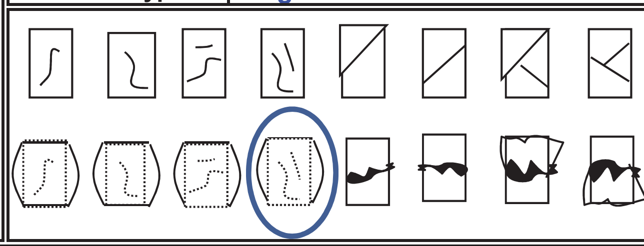
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	14-A	NA	Wet Weight, gms:	1046.31	1046.31
Tare Wt., gm:	10.49	NA	Dry Weight, gms:	844.99	
Wet Wt+Tare, gm:	58.98	NA			
Dry Wt.+Tare, gm:	49.65	NA	Wet Unit Wt.,pcf	127.8	NA
Wt. of Water, gm:	9.33	NA	Dry Unit Wt, pcf:	103.2	NA
Dry Soil Wt., gm:	39.16	NA	Void Ratio, pcf:	0.632	NA
Moisture:	23.8%	NA	Saturation, %:	101.8%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	6.98
Max. Compressive Stress, ksf:	2.71
Max. Compressive Stress, tsf:	1.36
Max. Compressive Stress, psi:	18.8
Undrained Shear Strength (ksf):	1.36

Failure Type: Bulge



Remarks:					
Tested By:	PC	Computed By:	RM	Checked By:	HJL

psi

1 of 1

Unconfined Compression Test ASTM D 2166

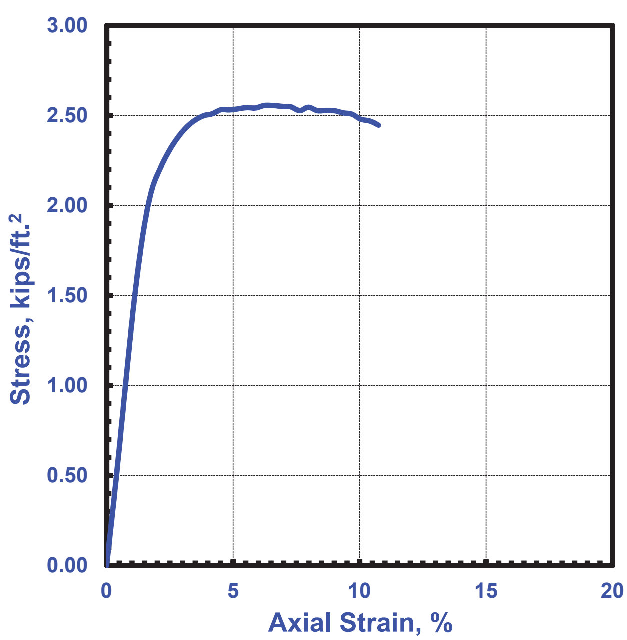
Test Date: *7/12/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-4*	**Depth, feet:**	*34-36'*	**Sample No./ID:**	*13*
Liquid Limit:	*70*	**Plastic Limit:**	*20*	**Plasticity Index:**	*50*
Hand Pen\Torvane:	*3.50*	**% Passing 200:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Clay*			**Sample Condition:**	*Undisturbed*

Oven: 02OV328	**Scale:** 01DS328	**Calipers:** 2DC328
Load Cell: NA	**DCDT** NA	**Load Frame:** MG2092

Sample Data	**1**	**2**	**3**	**Average**	**Height/Dia. Ratio**
Diameter (D),in.:	2.700	2.680	2.683	2.688	1.95
Height (H), in.:	5.240	5.240	5.240	5.240	
Area, ft^2:	0.039	**Volume, ft^3:**	0.017	**Seating:**	

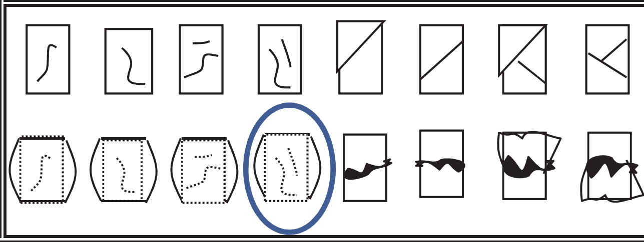
Mositure Data	**Before (Trimings)**	**After Test (Middle)**	**Specimen**	**Before**	**After**
Tare ID:	235	NA	**Wet Weight, gms:**	965.60	965.60
Tare Wt., gm:	15.93	NA	**Dry Weight, gms:**	751.37	
Wet Wt+Tare, gm:	58.84	NA			
Dry Wt.+Tare, gm:	49.32	NA	**Wet Unit Wt.,pcf**	123.7	NA
Wt. of Water, gm:	9.52	NA	**Dry Unit Wt, pcf:**	96.3	NA
Dry Soil Wt., gm:	33.39	NA	**Void Ratio, pcf:**	0.750	NA
Moisture:	28.5%	NA	**Saturation, %:**	102.7%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	6.24
Max. Compressive Stress, ksf:	2.56
Max. Compressive Stress, tsf:	1.28
Max. Compressive Stress, psi:	17.7
Undrained Shear Strength (ksf):	1.28

Failure Type: **Bulge W/ Shear Planes**



Remarks:					
Tested By:	PC	Computed By:	RM	Checked By:	HJL

psi

1 of 1

Unconfined Compression Test ASTM D 2166

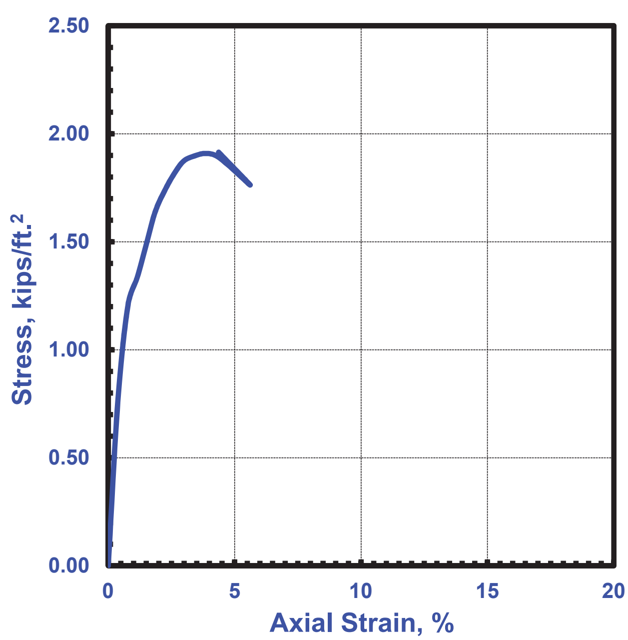
Test Date: *7/7/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-4*	**Depth, feet:**	*36-38'*	**Sample No./ID:**	*14*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen\Torvane:	*3.00*	**% Passing 200:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Clay*			**Sample Condition:**	*Undisturbed*

Oven:	02OV328	**Scale:**	01DS328	**Calipers:**	2DC328
Load Cell:	NA	**DCDT**	NA	**Load Frame:**	MG2092

Sample Data	**1**	**2**	**3**	**Average**	**Height/Dia. Ratio**
Diameter (D),in.:	2.700	2.690	2.700	2.697	1.67
Height (H), in.:	4.490	4.490	4.490	4.490	
Area, ft^2:	0.040	**Volume, ft^3:**	0.015	**Seating:**	

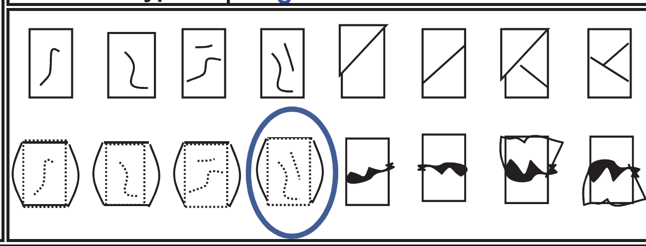
Mositure Data	**Before (Trimings)**	**After Test (Middle)**	**Specimen**	**Before**	**After**
Tare ID:	51	NA	**Wet Weight, gms:**	1027.43	1027.43
Tare Wt., gm:	11.21	NA	**Dry Weight, gms:**	812.56	
Wet Wt+Tare, gm:	53.05	NA			
Dry Wt.+Tare, gm:	44.3	NA	**Wet Unit Wt.,pcf**	152.6	NA
Wt. of Water, gm:	8.75	NA	**Dry Unit Wt, pcf:**	120.7	NA
Dry Soil Wt., gm:	33.09	NA	**Void Ratio, pcf:**	0.396	NA
Moisture:	26.4%	NA	**Saturation, %:**	180.4%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	4.37
Max. Compressive Stress, ksf:	1.91
Max. Compressive Stress, tsf:	0.96
Max. Compressive Stress, psi:	13.3
Undrained Shear Strength (ksf):	0.96

Failure Type: Bulge



Remarks:

Tested By:	PC	Computed By:	RM	Checked By:	HJL

psi

Unconfined Compression Test ASTM D 2166

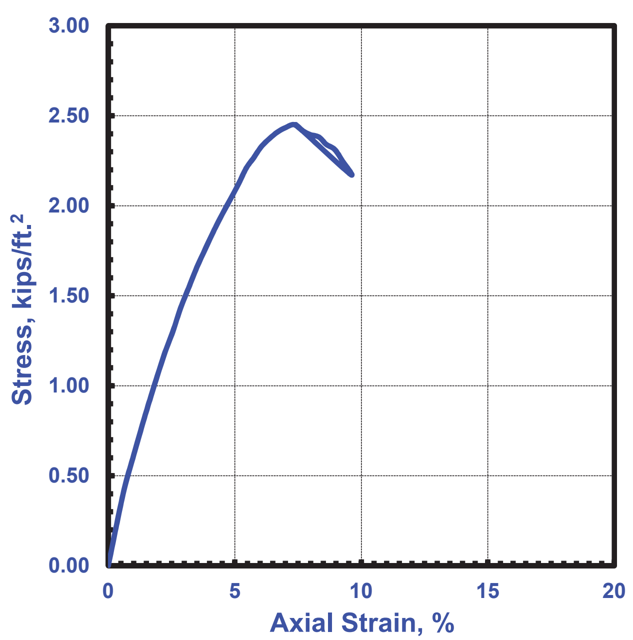
Test Date: *7/12/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-4*	**Depth, feet:**	*53-55'*	**Sample No./ID:**	*21*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen\Torvane:	*3.50*	**% Passing 200:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Clay*			**Sample Condition:**	*Undisturbed*

Oven:	02OV328	**Scale:**	01DS328	**Calipers:**	2DC328
Load Cell:	NA	**DCDT**	NA	**Load Frame:**	MG2092

Sample Data	**1**	**2**	**3**	**Average**	**Height/Dia. Ratio**
Diameter (D),in.:	2.640	2.660	2.670	2.657	2.07
Height (H), in.:	5.490	5.490	5.490	5.490	
Area, ft^2:	0.038	**Volume, ft^3:**	0.018	**Seating:**	

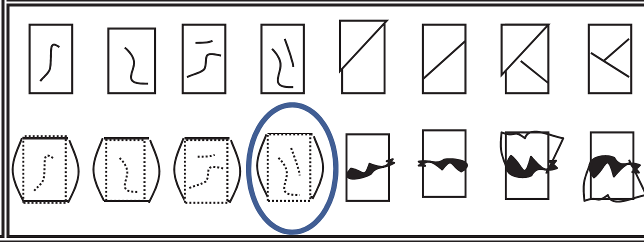
Mositure Data	**Before (Trimings)**	**After Test (Middle)**	**Specimen**	**Before**	**After**
Tare ID:	CC	NA	**Wet Weight, gms:**	1016.60	1016.60
Tare Wt., gm:	15.64	NA	**Dry Weight, gms:**	796.00	
Wet Wt+Tare, gm:	42.23	NA			
Dry Wt.+Tare, gm:	36.46	NA	**Wet Unit Wt.,pcf**	127.3	NA
Wt. of Water, gm:	5.77	NA	**Dry Unit Wt, pcf:**	99.6	NA
Dry Soil Wt., gm:	20.82	NA	**Void Ratio, pcf:**	0.691	NA
Moisture:	27.7%	NA	**Saturation, %:**	108.3%	NA



RESULTS

Actual Strain Rate, (%/min.):	0.38
Strain at Peak Stress, %:	7.41
Max. Compressive Stress, ksf:	2.45
Max. Compressive Stress, tsf:	1.23
Max. Compressive Stress, psi:	17.0
Undrained Shear Strength (ksf):	1.23

Failure Type: Bulge W/ Shear Planes



Remarks:

Tested By:	PC	Computed By:	RM	Checked By:	HJL

Unconfined Compression Test ASTM D 2166

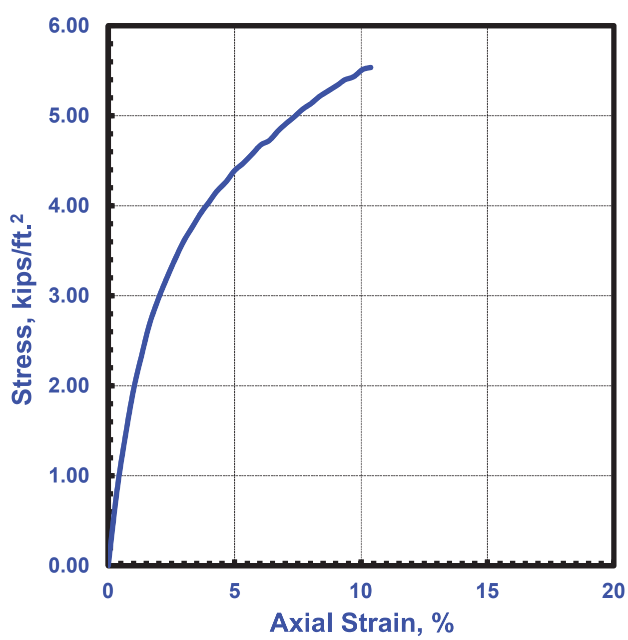
Test Date: *7/12/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-4*	**Depth, feet:**	*63-65'*	**Sample No./ID:**	*23*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen\Torvane:	*4.5+*	**% Passing 200:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Clay*			**Sample Condition:**	*Undisturbed*

Oven:	02OV328	**Scale:**	01DS328	**Calipers:**	2DC328
Load Cell:	NA	**DCDT**	NA	**Load Frame:**	MG2092

Sample Data	**1**	**2**	**3**	**Average**	**Height/Dia. Ratio**
Diameter (D),in.:	2.670	2.670	2.690	2.677	1.94
Height (H), in.:	5.190	5.190	5.190	5.190	
Area, ft^2:	0.039	**Volume, ft^3:**	0.017	**Seating:**	

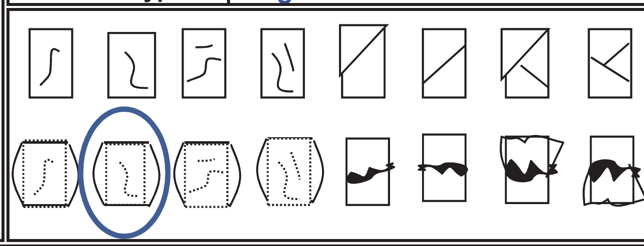
Mositure Data	**Before (Trimings)**	**After Test (Middle)**	**Specimen**	**Before**	**After**
Tare ID:	291	NA	**Wet Weight, gms:**	971.00	971.00
Tare Wt., gm:	15.54	NA	**Dry Weight, gms:**	781.27	
Wet Wt+Tare, gm:	48.09	NA			
Dry Wt.+Tare, gm:	41.73	NA	**Wet Unit Wt.,pcf**	126.7	NA
Wt. of Water, gm:	6.36	NA	**Dry Unit Wt, pcf:**	101.9	NA
Dry Soil Wt., gm:	26.19	NA	**Void Ratio, pcf:**	0.653	NA
Moisture:	24.3%	NA	**Saturation, %:**	100.4%	NA



RESULTS

Actual Strain Rate, (%/min.):	0.33
Strain at Peak Stress, %:	10.39
Max. Compressive Stress, ksf:	5.54
Max. Compressive Stress, tsf:	2.77
Max. Compressive Stress, psi:	38.4
Undrained Shear Strength (ksf):	2.77

Failure Type: Bulge



Remarks:					
Tested By:	PC	Computed By:	RM	Checked By:	HJL

Unconfined Compression Test ASTM D 2166

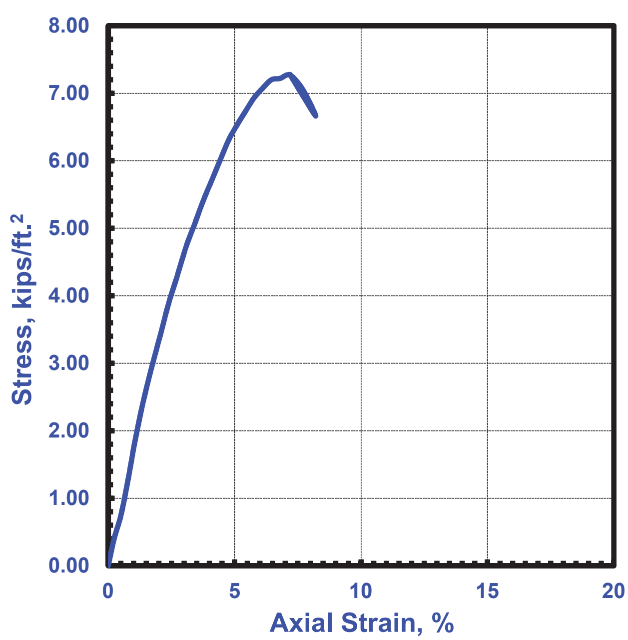
Test Date: *7/7/2016*

Project Name:	*Upper Levee Segment*	**Project Number:**	*0328-1663*
Boring Number:	*B-4* **Depth, feet:** *68-70'*	**Sample No./ID:**	*24*
Liquid Limit:	*59* **Plastic Limit:** *14*	**Plasticity Index:**	*45*
Hand Pen\Torvane:	*4.5+* **% Passing 200:** *NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Clay*	**Sample Condition:**	*Undisturbed*

Oven: 02OV328	**Scale:** 01DS328	**Calipers:** 2DC328
Load Cell: NA	**DCDT** NA	**Load Frame:** MG2092

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.700	2.660	2.680	2.680	1.94
Height (H), in.:	5.190	5.190	5.190	5.190	
Area, ft^2:	0.039	Volume, ft^3:	0.017	Seating:	

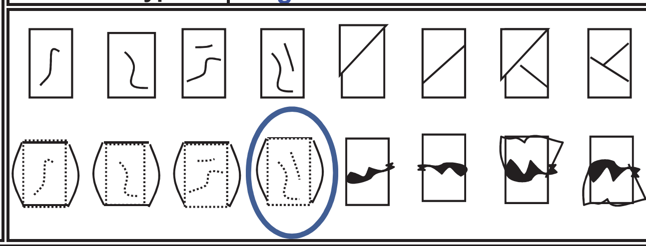
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	24	NA	Wet Weight, gms:	966.21	966.21
Tare Wt., gm:	11	NA	Dry Weight, gms:	779.14	
Wet Wt+Tare, gm:	35.74	NA			
Dry Wt.+Tare, gm:	30.95	NA	Wet Unit Wt.,pcf	125.7	NA
Wt. of Water, gm:	4.79	NA	Dry Unit Wt, pcf:	101.4	NA
Dry Soil Wt., gm:	19.95	NA	Void Ratio, pcf:	0.662	NA
Moisture:	24.0%	NA	Saturation, %:	98.0%	NA



RESULTS

Actual Strain Rate, (%/min.):	NA
Strain at Peak Stress, %:	7.19
Max. Compressive Stress, ksf:	7.28
Max. Compressive Stress, tsf:	3.64
Max. Compressive Stress, psi:	50.5
Undrained Shear Strength (ksf):	**3.64**

Failure Type: Bulge



Remarks:					
Tested By:	PC	Computed By:	RM	Checked By:	HJL

LAB TESTING

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

Unconsolidated Undrained Triaxial Test ASTM D 2850

Test Date: *7/7/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-1*	**Depth, feet:**	*22-24'*	**Sample No./ID:**	*7*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen:	*2.25*	**Torvane:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Dark Gray Silty Clay*			**Sample Condition:**	*Undisturbed*

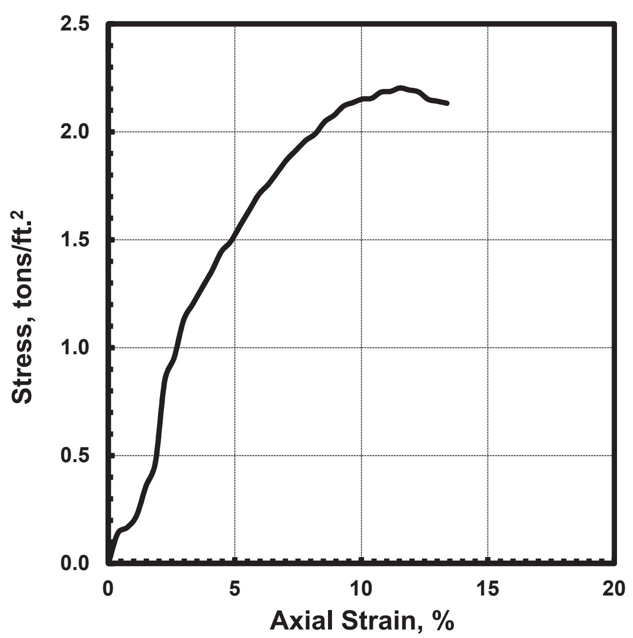
Oven:	02OV328	**Scale:**	03DS328	**Calipers:**	2DC328
Load Cell:	01LC328	**Cell Pressure:**	AIR	**Load Frame:**	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.560	2.560	2.560	2.560	2.10
Height (H), in.:	5.380	5.380	5.380	5.380	

Area, ft^2:	0.036	Volume, ft^3:	0.016	Piston Load, lbs:	2.42

Mositure Data	Before (Trimings)	After Test (Middle)
Tare ID:	Z73	-
Tare Wt., gm:	16.19	-
Wet Wt+Tare, gm:	60.29	-
Dry Wt.+Tare, gm:	52.01	-
Wt. of Water, gm:	8.28	-
Dry Soil Wt., gm:	35.82	-
Moisture:	23.1%	-

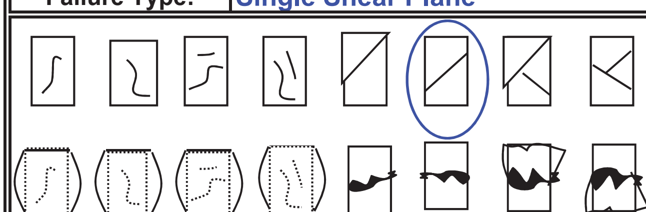
Specimen	Before	After
Wet Weight, gms:	952.90	952.90
Dry Weight, gms:	773.99	
Wet Unit Wt.,pcf	131.1	-
Dry Unit Wt, pcf:	106.5	-
Void Ratio, pcf:	0.582	-
Saturation, %:	107.2%	-



RESULTS

Actual Strain Rate, (%/min.):	0.81
Average Confining Pressure, psi:	20.0
Failure Strain at Peak Stress,%:	11.52
Max. Deviator Stress (Γϖ),tsf:	2.20
Minor Principle Stress (ϖ6),tsf:	1.44
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	3.64
Undrained Shear Strength (tsf):	1.10

Failure Type: Single Shear Plane



Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

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Unconsolidated Undrained Triaxial Test ASTM D 2850

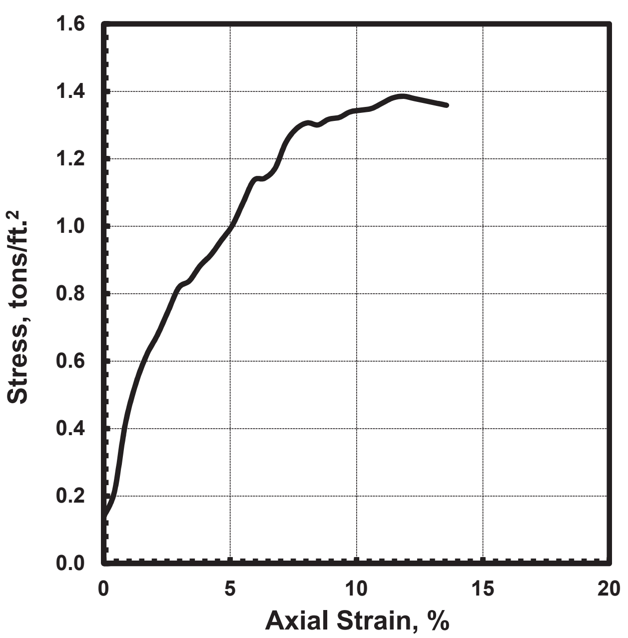
Test Date: *7/6/2016*

Project Name:	*Upper Levee Segment*			Project Number:	*0328-1663*
Boring Number:	*B-1*	Depth, feet:	*28-30'*	Sample No./ID:	*10*
Liquid Limit:	*NA*	Plastic Limit:	*NA*	Plasticity Index:	*NA*
Hand Pen:	*2*	Torvane:	*NA*	Specific Gravity:	*2.700*
Classification:	*Medium Brown Silty Clay*			Sample Condition:	*Undisturbed*

Oven:	02OV328	Scale:	03DS328	Calipers:	2DC328
Load Cell:	01LC328	Cell Pressure:	AIR	Load Frame:	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.700	2.700	2.700	2.700	1.75
Height (H), in.:	4.720	4.720	4.720	4.720	
Area, ft^2:	0.040	Volume, ft^3:	0.016	Piston Load, lbs:	2.42

Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	Z71	-	Wet Weight, gms:	865.92	865.92
Tare Wt., gm:	15.19	-	Dry Weight, gms:	641.52	
Wet Wt+Tare, gm:	49.34	-			
Dry Wt.+Tare, gm:	40.49	-	Wet Unit Wt.,pcf	122.1	-
Wt. of Water, gm:	8.85	-	Dry Unit Wt, pcf:	90.4	-
Dry Soil Wt., gm:	25.3	-	Void Ratio, pcf:	0.863	-
Moisture:	35.0%	-	Saturation, %:	109.4%	-

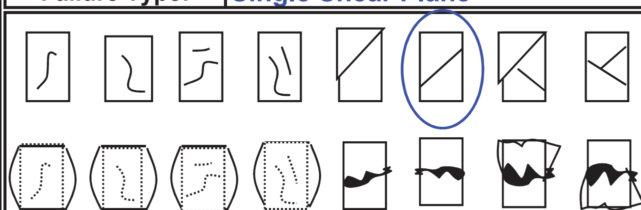


RESULTS

Actual Strain Rate, (%/min.):	0.87
Average Confining Pressure, psi:	22.0
Failure Strain at Peak Stress,%:	11.86
Max. Deviator Stress (Γϖ),tsf:	1.39
Minor Principle Stress (ϖ6),tsf:	1.58
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	2.97

Undrained Shear Strength (tsf):	0.69

Failure Type: Single Shear Plane



Remarks:

Tested By:	JOM	Computed By:	RM	Checked By:	HJL

psi

1 of 1

Unconsolidated Undrained Triaxial Test ASTM D 2850

Test Date: 7/8/2016

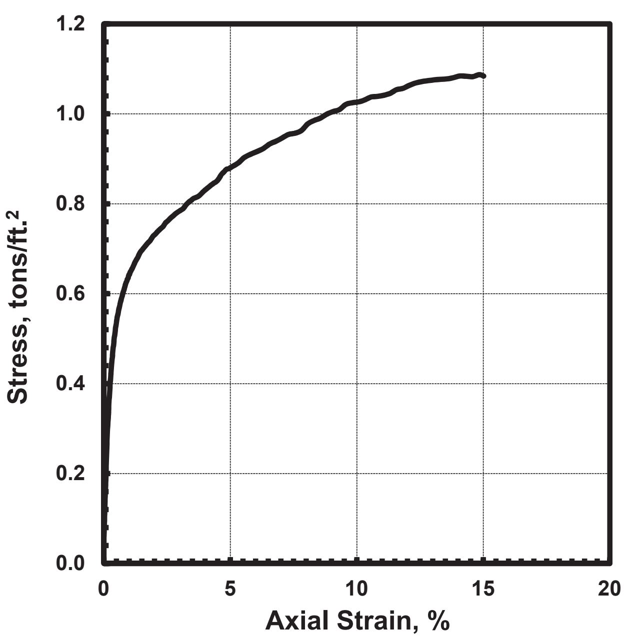
Project Name:	Upper Levee Segment			Project Number:	0328-1663
Boring Number:	B-1	Depth, feet:	32-34'	Sample No./ID:	12
Liquid Limit:	71	Plastic Limit:	21	Plasticity Index:	50
Hand Pen:	3	Torvane:	NA	Specific Gravity:	2.700
Classification:	Brown Clay			Sample Condition:	Undisturbed

Oven:	14-U-6	Scale:	E400 32358	Calipers:	01DC342
Load Cell:	02LC342	Cell Pressure:	PS2-111	Load Frame:	N/A

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.642	2.643	2.646	2.644	2.10
Height (H), in.:	5.539	5.539	5.541	5.540	

Area, ft^2:	0.038	Volume, ft^3:	0.018	Piston Load, lbs:	16.78

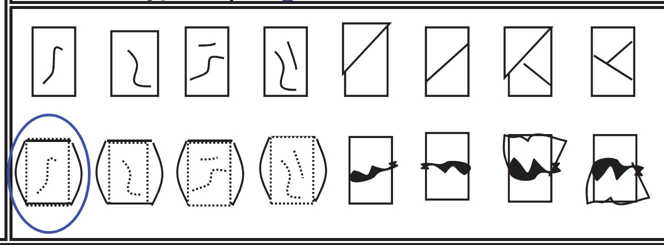
Mositure Data	Before (Trimmings)	After Test (Middle)	Specimen	Before	After
Tare ID:	Q22	NA	Wet Weight, gms:	983.90	983.90
Tare Wt., gm:	30.82	NA	Dry Weight, gms:	778.79	
Wet Wt+Tare, gm:	96.97	NA			
Dry Wt.+Tare, gm:	83.18	NA	Wet Unit Wt.,pcf	123.3	NA
Wt. of Water, gm:	13.79	NA	Dry Unit Wt, pcf:	97.6	NA
Dry Soil Wt., gm:	52.36	NA	Void Ratio, pcf:	0.727	NA
Moisture:	26.3%	NA	Saturation, %:	97.8%	NA



RESULTS

Actual Strain Rate, (%/min.):	1.00
Average Confining Pressure, psi:	24.0
Failure Strain at Peak Stress,%:	14.84
Max. Deviator Stress (Γϖ),tsf:	1.09
Minor Principle Stress (ϖ6),tsf:	1.73
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	2.82
Undrained Shear Strength (tsf):	0.54

Failure Type: Bulge



Remarks:					
Tested By:	JM	Computed By:	TW	Checked By:	KMV

Unconsolidated Undrained Triaxial Test ASTM D 2850

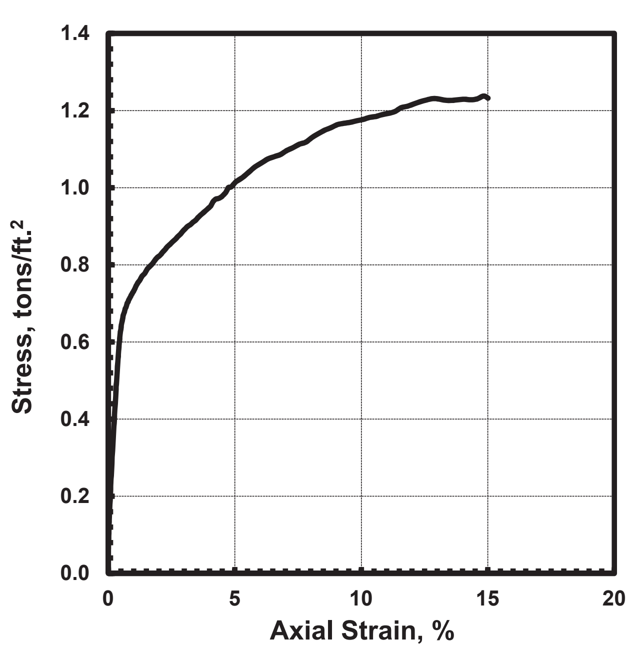
Test Date: *7/8/2016*

Project Name:	*Upper Levee Segment*			Project Number:	*0328-1663*
Boring Number:	*B1*	Depth, feet:	*34-36'*	Sample No./ID:	*13*
Liquid Limit:	*51*	Plastic Limit:	*30*	Plasticity Index:	*31*
Hand Pen:	*2*	Torvane:	*NA*	Specific Gravity:	*2.700*
Classification:	*Brown Clay*			Sample Condition:	*Undisturbed*

Oven:	14-U-6	Scale:	E400 32358	Calipers:	01DC342
Load Cell:	02LC342	Cell Pressure:	PS2-111	Load Frame:	N/A

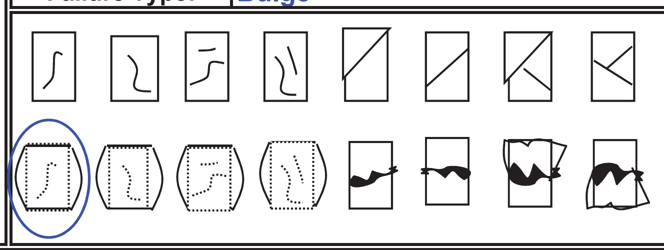
Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.665	2.663	2.664	2.664	2.08
Height (H), in.:	5.540	5.535	5.544	5.540	
Area, ft^2:	0.039	Volume, ft^3:	0.018	Piston Load, lbs:	15.39

Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	Q25	NA	Wet Weight, gms:	990.70	990.70
Tare Wt., gm:	31.22	NA	Dry Weight, gms:	786.67	
Wet Wt+Tare, gm:	83.66	NA			
Dry Wt.+Tare, gm:	72.86	NA	Wet Unit Wt.,pcf	122.2	NA
Wt. of Water, gm:	10.8	NA	Dry Unit Wt, pcf:	97.1	NA
Dry Soil Wt., gm:	41.64	NA	Void Ratio, pcf:	0.736	NA
Moisture:	25.9%	NA	Saturation, %:	95.2%	NA



RESULTS

Actual Strain Rate, (%/min.):	1.00
Average Confining Pressure, psi:	25.0
Failure Strain at Peak Stress,%:	14.83
Max. Deviator Stress (Γϖ),tsf:	1.24
Minor Principle Stress (ϖ6),tsf:	1.80
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	3.04
Undrained Shear Strength (tsf):	0.62
Failure Type:	Bulge



Remarks:					
Tested By:	JM	Computed By:	TW	Checked By:	KMV

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1 of 1

Unconsolidated Undrained Triaxial Test ASTM D 2850

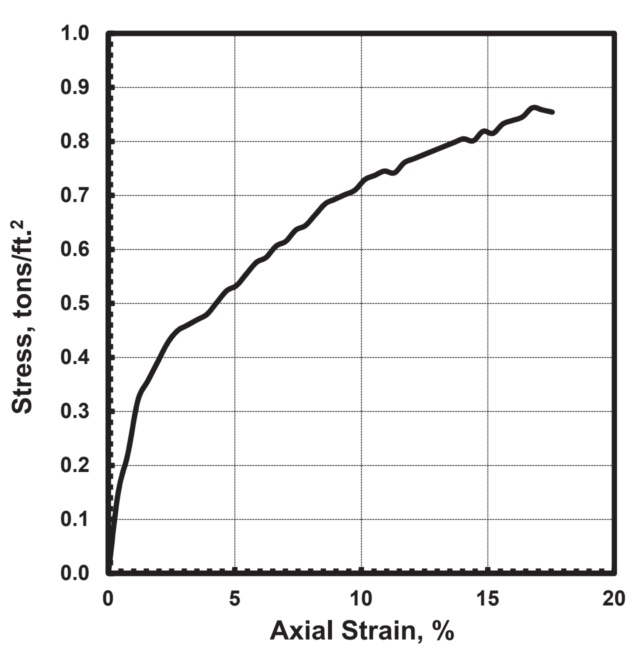
Test Date: *7/6/2016*

Project Name:	*Upper Levee Segment*			Project Number:	*0328-1663*
Boring Number:	*B-1*	Depth, feet:	*38-40'*	Sample No./ID:	*15*
Liquid Limit:	*NA*	Plastic Limit:	*NA*	Plasticity Index:	*NA*
Hand Pen:	*1.75*	Torvane:	*NA*	Specific Gravity:	*2.700*
Classification:	*Medium Brown Silty Clay*			Sample Condition:	*Undisturbed*

Oven:	02OV328	Scale:	03DS328	Calipers:	2DC328
Load Cell:	01LC328	Cell Pressure:	AIR	Load Frame:	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.710	2.710	2.710	2.710	1.89
Height (H), in.:	5.130	5.130	5.130	5.130	
Area, ft^2:	0.040	Volume, ft^3:	0.017	Piston Load, lbs:	2.42

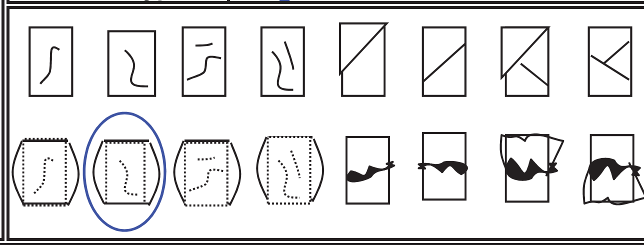
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	30	-	Wet Weight, gms:	963.30	963.30
Tare Wt., gm:	11.23	-	Dry Weight, gms:	760.27	
Wet Wt+Tare, gm:	57.11	-			
Dry Wt.+Tare, gm:	47.44	-	Wet Unit Wt.,pcf	124.0	-
Wt. of Water, gm:	9.67	-	Dry Unit Wt, pcf:	97.9	-
Dry Soil Wt., gm:	36.21	-	Void Ratio, pcf:	0.721	-
Moisture:	26.7%	-	Saturation, %:	100.0%	-



RESULTS

Actual Strain Rate, (%/min.):	0.78
Average Confining Pressure, psi:	26.0
Failure Strain at Peak Stress,%:	16.76
Max. Deviator Stress (Γϖ),tsf:	0.86
Minor Principle Stress (ϖ6),tsf:	1.87
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	2.73
Undrained Shear Strength (tsf):	**0.43**

Failure Type: Bulge



Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

Unconsolidated Undrained Triaxial Test ASTM D 2850

Test Date: *7/6/2016*

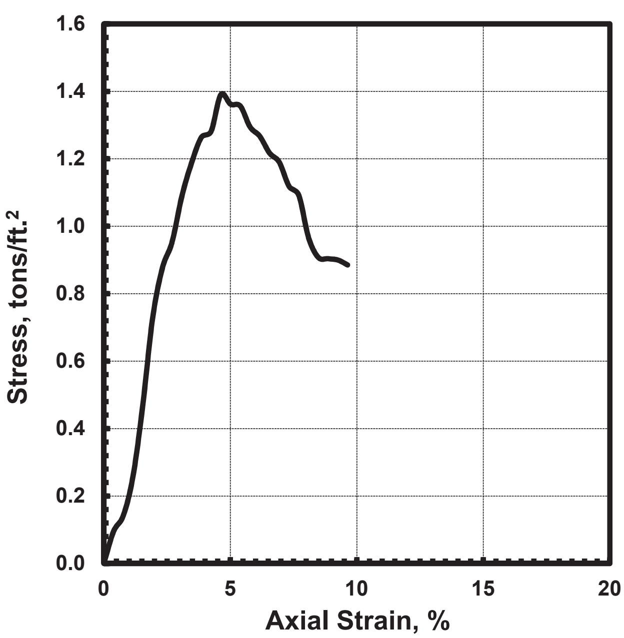
Project Name:	*Upper Levee Segment*			Project Number:	*0328-1663*
Boring Number:	*B-1*	Depth, feet:	*58-60'*	Sample No./ID:	*21*
Liquid Limit:	*30*	Plastic Limit:	*15*	Plasticity Index:	*15*
Hand Pen:	*2*	Torvane:	*NA*	Specific Gravity:	*2.700*
Classification:	*Medium Brown Silty Clay*			Sample Condition:	*Undisturbed*

Oven:	02OV328	Scale:	03DS328	Calipers:	2DC328
Load Cell:	01LC328	Cell Pressure:	AIR	Load Frame:	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.770	2.770	2.770	2.770	1.87
Height (H), in.:	5.190	5.190	5.190	5.190	

Area, ft^2:	0.042	Volume, ft^3:	0.018	Piston Load, lbs:	2.42

Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	XX	-	Wet Weight, gms:	999.90	999.90
Tare Wt., gm:	15.47	-	Dry Weight, gms:	812.51	
Wet Wt+Tare, gm:	62	-			
Dry Wt.+Tare, gm:	53.28	-	Wet Unit Wt.,pcf	121.8	-
Wt. of Water, gm:	8.72	-	Dry Unit Wt, pcf:	99.0	-
Dry Soil Wt., gm:	37.81	-	Void Ratio, pcf:	0.702	-
Moisture:	23.1%	-	Saturation, %:	88.7%	-



RESULTS

Actual Strain Rate, (%/min.):	0.73
Average Confining Pressure, psi:	34.0
Failure Strain at Peak Stress,%:	4.62
Max. Deviator Stress (Γϖ),tsf:	1.39
Minor Principle Stress (ϖ6),tsf:	2.45
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	3.84
Undrained Shear Strength (tsf):	**0.70**

Failure Type: Single Shear Plane

Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

Unconsolidated Undrained Triaxial Test ASTM D 2850

Test Date: *7/6/2016*

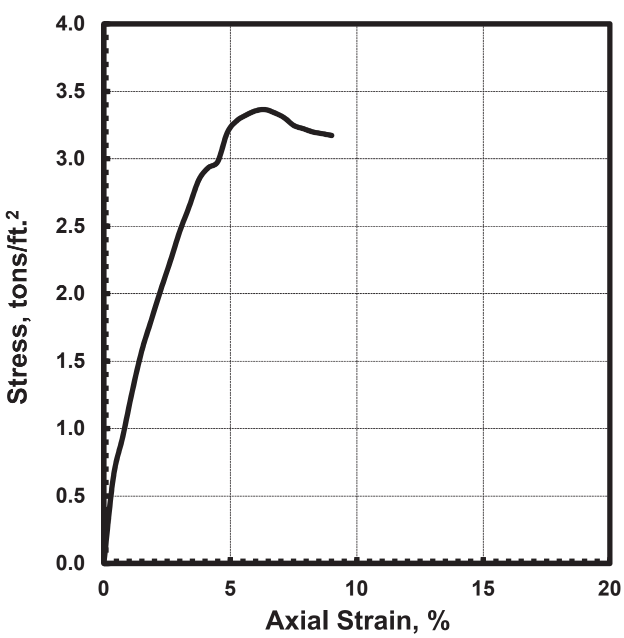
Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-2*	**Depth, feet:**	*40-42'*	**Sample No./ID:**	*14*
Liquid Limit:	*62*	**Plastic Limit:**	*23*	**Plasticity Index:**	*39*
Hand Pen:	*3.75*	**Torvane:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Silty Clay*			**Sample Condition:**	*Undisturbed*

Oven:	02OV328	**Scale:**	03DS328	**Calipers:**	2DC328
Load Cell:	01LC328	**Cell Pressure:**	AIR	**Load Frame:**	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.780	2.780	2.780	2.780	1.92
Height (H), in.:	5.330	5.330	5.330	5.330	

Area, ft^2:	0.042	**Volume, ft^3:**	0.019	**Piston Load, lbs:**	2.42

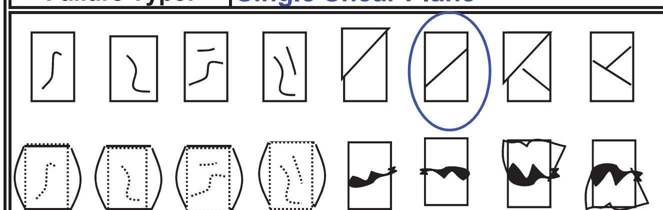
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	25-E	-	Wet Weight, gms:	1040.80	1040.80
Tare Wt., gm:	11.03	-	Dry Weight, gms:	781.83	
Wet Wt+Tare, gm:	57.45	-			
Dry Wt.+Tare, gm:	45.9	-	Wet Unit Wt.,pcf	122.6	-
Wt. of Water, gm:	11.55	-	Dry Unit Wt, pcf:	92.1	-
Dry Soil Wt., gm:	34.87	-	Void Ratio, pcf:	0.830	-
Moisture:	33.1%	-	Saturation, %:	107.7%	-



RESULTS

Actual Strain Rate, (%/min.):	0.96
Average Confining Pressure, psi:	22.0
Failure Strain at Peak Stress,%:	6.38
Max. Deviator Stress (Γϖ),tsf:	3.36
Minor Principle Stress (ϖ6),tsf:	1.58
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	4.95
Undrained Shear Strength (tsf):	**1.68**

Failure Type: Single Shear Plane



Remarks:							
Tested By:	JOL	Computed By:	RM	Checked By:	HJL		

psi

Unconsolidated Undrained Triaxial Test ASTM D 2850

Test Date: *7/6/2016*

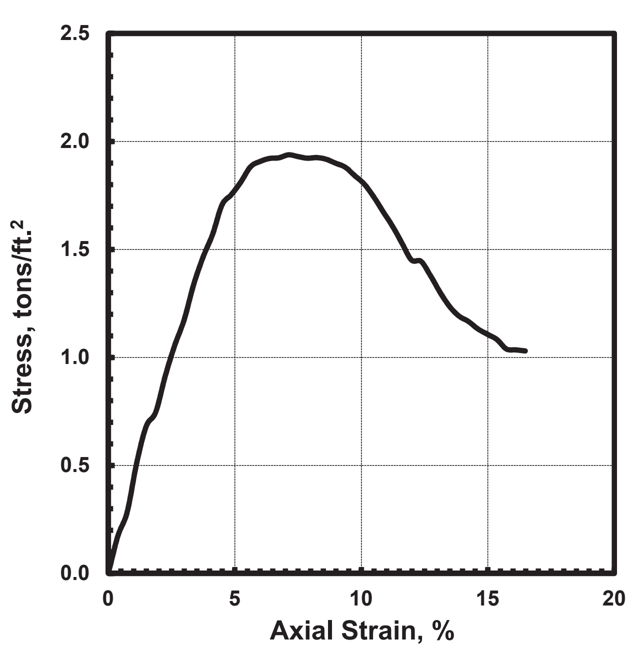
Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-2*	**Depth, feet:**	*44-46'*	**Sample No./ID:**	*16*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen:	*4*	**Torvane:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Silty Clay*			**Sample Condition:**	*Undisturbed*

Oven:	02OV328	**Scale:**	03DS328	**Calipers:**	2DC328
Load Cell:	01LC328	**Cell Pressure:**	AIR	**Load Frame:**	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.820	2.820	2.820	2.820	1.89
Height (H), in.:	5.340	5.340	5.340	5.340	

Area, ft^2:	0.043	**Volume, ft^3:**	0.019	**Piston Load, lbs:**	2.42

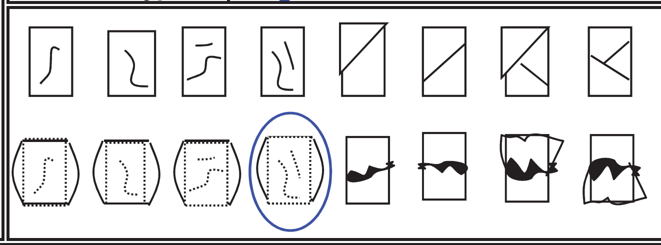
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	26	-	Wet Weight, gms:	1147.80	1147.80
Tare Wt., gm:	11.05	-	Dry Weight, gms:	907.23	
Wet Wt+Tare, gm:	53.18	-			
Dry Wt.+Tare, gm:	44.35	-	Wet Unit Wt.,pcf	131.1	-
Wt. of Water, gm:	8.83	-	Dry Unit Wt, pcf:	103.6	-
Dry Soil Wt., gm:	33.3	-	Void Ratio, pcf:	0.626	-
Moisture:	26.5%	-	Saturation, %:	114.4%	-



RESULTS

Actual Strain Rate, (%/min.):	0.67
Average Confining Pressure, psi:	23.0
Failure Strain at Peak Stress,%:	7.12
Max. Deviator Stress (Γϖ),tsf:	1.94
Minor Principle Stress (ϖ6),tsf:	1.66
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	3.59
Undrained Shear Strength (tsf):	**0.97**

Failure Type: Bulge W/ Shear Planes



Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

psi

Unconsolidated Undrained Triaxial Test ASTM D 2850

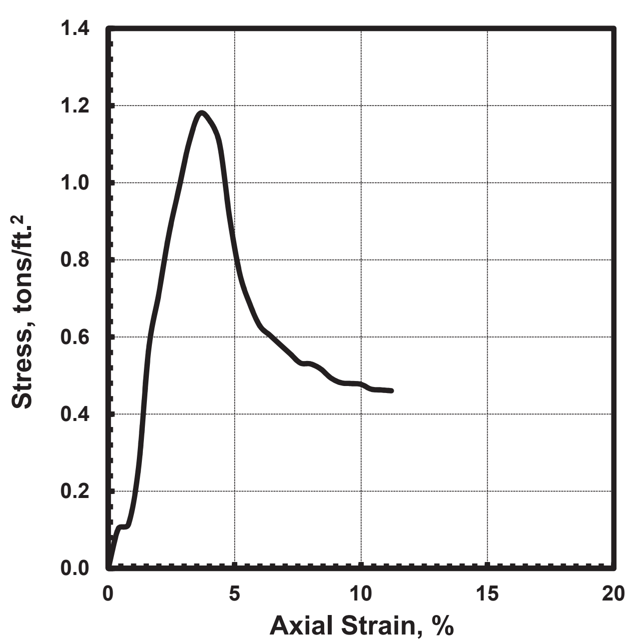
Test Date: *7/6/2016*

Project Name:	*Upper Levee Segment*			Project Number:	*0328-1663*
Boring Number:	*B-2*	Depth, feet:	*73-75'*	Sample No./ID:	*23*
Liquid Limit:	*38*	Plastic Limit:	*16*	Plasticity Index:	*23*
Hand Pen:	*3.25*	Torvane:	*NA*	Specific Gravity:	*2.700*
Classification:	*Medium Brown Silty Clay*			Sample Condition:	*Undisturbed*

Oven:	02OV328	Scale:	03DS328	Calipers:	2DC328
Load Cell:	01LC328	Cell Pressure:	AIR	Load Frame:	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.820	2.820	2.820	2.820	1.77
Height (H), in.:	5.000	5.000	5.000	5.000	
Area, ft^2:	0.043	Volume, ft^3:	0.018	Piston Load, lbs:	2.42

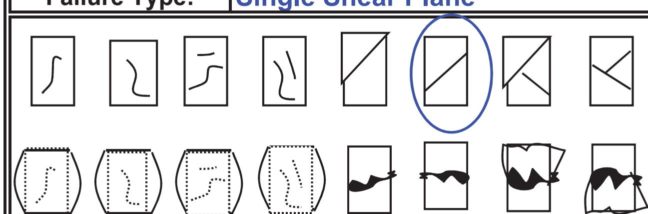
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	PP	-	Wet Weight, gms:	1037.62	1037.62
Tare Wt., gm:	15.59	-	Dry Weight, gms:	841.31	
Wet Wt+Tare, gm:	57.24	-			
Dry Wt.+Tare, gm:	49.36	-	Wet Unit Wt.,pcf	126.6	-
Wt. of Water, gm:	7.88	-	Dry Unit Wt, pcf:	102.6	-
Dry Soil Wt., gm:	33.77	-	Void Ratio, pcf:	0.642	-
Moisture:	23.3%	-	Saturation, %:	98.2%	-



RESULTS

Actual Strain Rate, (%/min.):	0.66
Average Confining Pressure, psi:	35.0
Failure Strain at Peak Stress,%:	3.60
Max. Deviator Stress (Γϖ),tsf:	1.18
Minor Principle Stress (ϖ6),tsf:	2.52
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	3.70
Undrained Shear Strength (tsf):	0.59

Failure Type: Single Shear Plane



Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

Unconsolidated Undrained Triaxial Test ASTM D 2850

Test Date: 7/6/2016

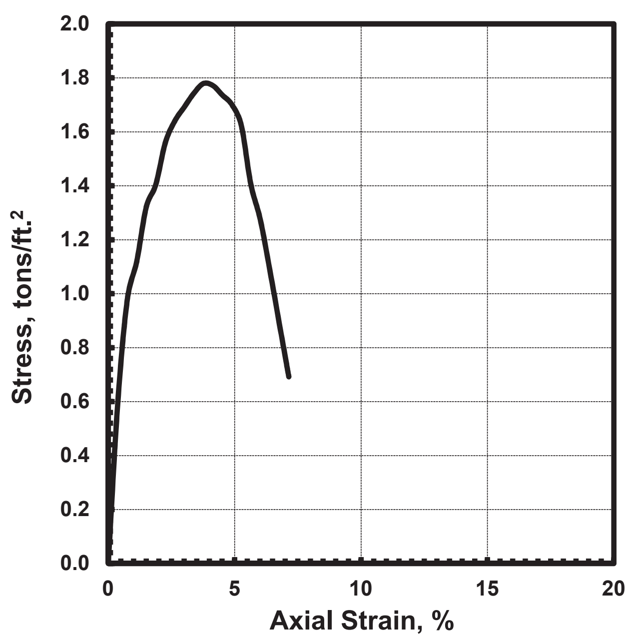
Project Name:	Upper Levee Segment			Project Number:	0328-1663
Boring Number:	B-3	Depth, feet:	38-40'	Sample No./ID:	14
Liquid Limit:	NA	Plastic Limit:	NA	Plasticity Index:	NA
Hand Pen:	3.5	Torvane:	NA	Specific Gravity:	2.700
Classification:	Medium Brown Clay			Sample Condition:	Undisturbed

Oven:	02OV328	Scale:	03DS328	Calipers:	2DC328
Load Cell:	01LC328	Cell Pressure:	AIR	Load Frame:	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.480	2.480	2.480	2.480	2.15
Height (H), in.:	5.320	5.320	5.320	5.320	

Area, ft^2:	0.034	Volume, ft^3:	0.015	Piston Load, lbs:	2.42

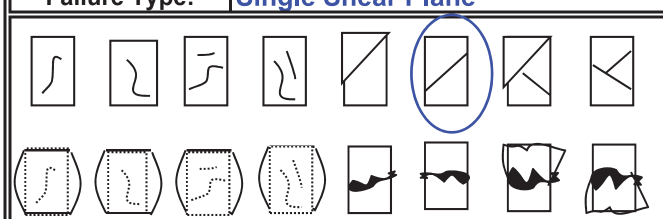
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	B3	-	Wet Weight, gms:	877.60	877.60
Tare Wt., gm:	11.36	-	Dry Weight, gms:	693.23	
Wet Wt+Tare, gm:	53.58	-			
Dry Wt.+Tare, gm:	44.71	-	Wet Unit Wt.,pcf	130.1	-
Wt. of Water, gm:	8.87	-	Dry Unit Wt, pcf:	102.8	-
Dry Soil Wt., gm:	33.35	-	Void Ratio, pcf:	0.639	-
Moisture:	26.6%	-	Saturation, %:	112.3%	-



RESULTS

Actual Strain Rate, (%/min.):	0.98
Average Confining Pressure, psi:	21.0
Failure Strain at Peak Stress,%:	3.76
Max. Deviator Stress (Γϖ),tsf:	1.78
Minor Principle Stress (ϖ6),tsf:	1.51
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	3.29
Undrained Shear Strength (tsf):	**0.89**

Failure Type: Single Shear Plane



Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

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Unconsolidated Undrained Triaxial Test ASTM D 2850

Test Date: *7/7/2016*

Project Name:	*Upper Levee Segment*			Project Number:	*0328-1663*
Boring Number:	*B-3*	Depth, feet:	*48-50'*	Sample No./ID:	*18*
Liquid Limit:	*NA*	Plastic Limit:	*NA*	Plasticity Index:	*NA*
Hand Pen:	*0.5*	Torvane:	*NA*	Specific Gravity:	*2.700*
Classification:	*Medium Brown Sandy Silt*			Sample Condition:	*Undisturbed*

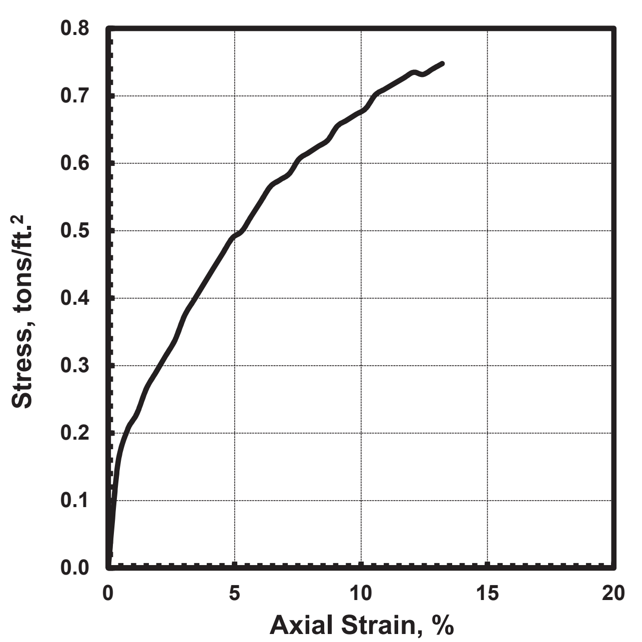
Oven:	02OV328	Scale:	03DS328	Calipers:	2DC328
Load Cell:	01LC328	Cell Pressure:	AIR	Load Frame:	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.670	2.670	2.670	2.670	1.99
Height (H), in.:	5.300	5.300	5.300	5.300	

Area, ft^2:	0.039	Volume, ft^3:	0.017	Piston Load, lbs:	2.42

Mositure Data	Before (Trimings)	After Test (Middle)
Tare ID:	15	-
Tare Wt., gm:	11.53	-
Wet Wt+Tare, gm:	50.73	-
Dry Wt.+Tare, gm:	42.7	-
Wt. of Water, gm:	8.03	-
Dry Soil Wt., gm:	31.17	-
Moisture:	25.8%	-

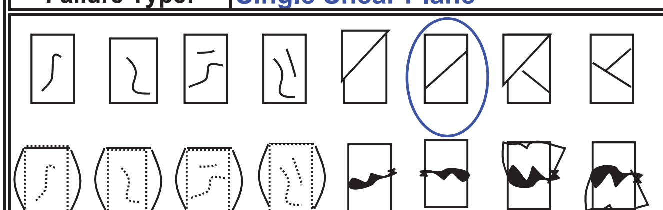
Specimen	Before	After
Wet Weight, gms:	1024.53	1024.53
Dry Weight, gms:	814.66	
Wet Unit Wt.,pcf	131.5	-
Dry Unit Wt, pcf:	104.6	-
Void Ratio, pcf:	0.611	-
Saturation, %:	113.9%	-



RESULTS

Actual Strain Rate, (%/min.):	0.78
Average Confining Pressure, psi:	25.0
Failure Strain at Peak Stress,%:	13.21
Max. Deviator Stress (Γϖ),tsf:	0.75
Minor Principle Stress (ϖ6),tsf:	1.80
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	2.55
Undrained Shear Strength (tsf):	0.37

Failure Type: Single Shear Plane



Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

Unconsolidated Undrained Triaxial Test ASTM D 2850

Test Date: *7/7/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-3*	**Depth, feet:**	*58-60'*	**Sample No./ID:**	*20*
Liquid Limit:	*NA*	**Plastic Limit:**	*NA*	**Plasticity Index:**	*NA*
Hand Pen:	*1.5*	**Torvane:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Silty Clay*			**Sample Condition:**	*Undisturbed*

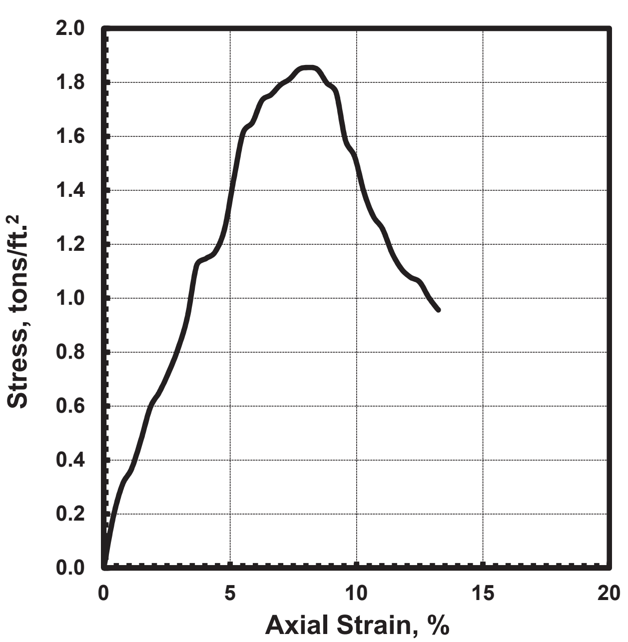
Oven:	02OV328	**Scale:**	03DS328	**Calipers:**	2DC328
Load Cell:	01LC328	**Cell Pressure:**	AIR	**Load Frame:**	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.430	2.430	2.430	2.430	2.24
Height (H), in.:	5.440	5.440	5.440	5.440	

Area, ft^2:	0.032	Volume, ft^3:	0.015	Piston Load, lbs:	2.42

Mositure Data	Before (Trimings)	After Test (Middle)
Tare ID:	11	-
Tare Wt., gm:	11.13	-
Wet Wt+Tare, gm:	61.4	-
Dry Wt.+Tare, gm:	51.66	-
Wt. of Water, gm:	9.74	-
Dry Soil Wt., gm:	40.53	-
Moisture:	24.0%	-

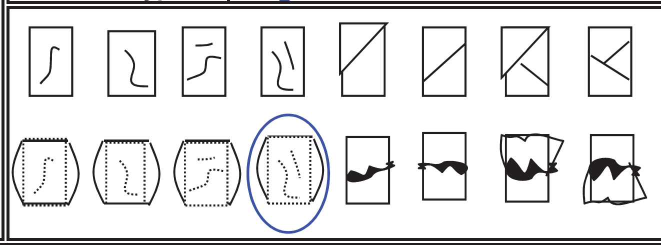
Specimen	Before	After
Wet Weight, gms:	908.28	908.28
Dry Weight, gms:	732.30	
Wet Unit Wt.,pcf	137.2	-
Dry Unit Wt, pcf:	110.6	-
Void Ratio, pcf:	0.524	-
Saturation, %:	123.9%	-



RESULTS

Actual Strain Rate, (%/min.):	0.86
Average Confining Pressure, psi:	21.0
Failure Strain at Peak Stress,%:	8.09
Max. Deviator Stress (Γϖ),tsf:	1.86
Minor Principle Stress (ϖ6),tsf:	1.51
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	3.37
Undrained Shear Strength (tsf):	**0.93**

Failure Type: Bulge W/ Shear Planes



Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

psi

Unconsolidated Undrained Triaxial Test ASTM D 2850

Test Date: *7/7/2016*

Project Name:	*Upper Levee Segment*			Project Number:	*0328-1663*
Boring Number:	*B-4*	Depth, feet:	*28-30'*	Sample No./ID:	*10*
Liquid Limit:	*NA*	Plastic Limit:	*NA*	Plasticity Index:	*NA*
Hand Pen:	*2*	Torvane:	*NA*	Specific Gravity:	*2.700*
Classification:	*Medium Brown Silty Clay*			Sample Condition:	*Undisturbed*

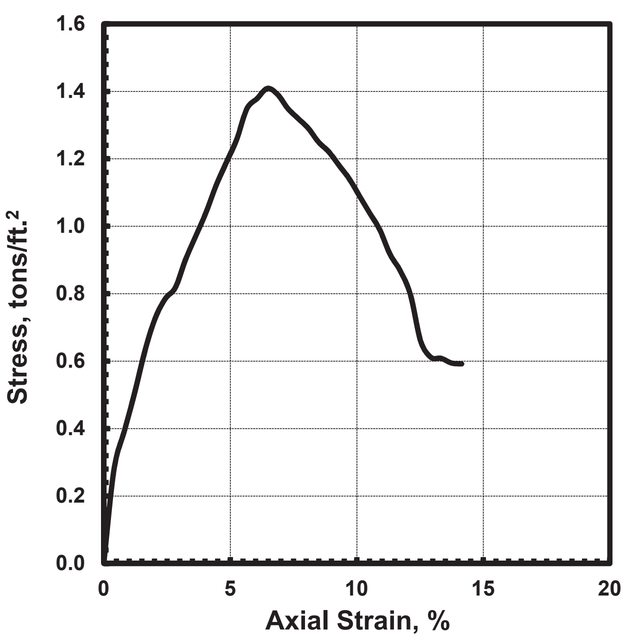
Oven:	02OV328	Scale:	03DS328	Calipers:	2DC328
Load Cell:	01LC328	Cell Pressure:	AIR	Load Frame:	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.680	2.680	2.680	2.680	1.85
Height (H), in.:	4.950	4.950	4.950	4.950	

Area, ft^2:	0.039	Volume, ft^3:	0.016	Piston Load, lbs:	2.42

Mositure Data	Before (Trimings)	After Test (Middle)
Tare ID:	110	-
Tare Wt., gm:	15.6	-
Wet Wt+Tare, gm:	53.17	-
Dry Wt.+Tare, gm:	45.99	-
Wt. of Water, gm:	7.18	-
Dry Soil Wt., gm:	30.39	-
Moisture:	23.6%	-

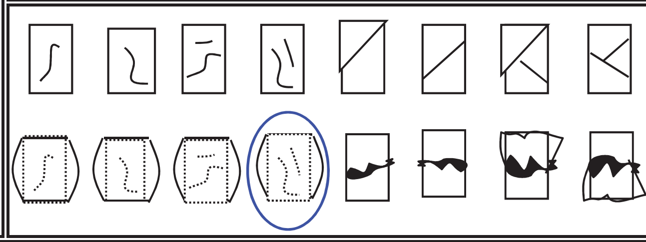
Specimen	Before	After
Wet Weight, gms:	905.62	905.62
Dry Weight, gms:	732.55	
Wet Unit Wt.,pcf	123.6	-
Dry Unit Wt, pcf:	99.9	-
Void Ratio, pcf:	0.686	-
Saturation, %:	93.0%	-



RESULTS

Actual Strain Rate, (%/min.):	0.69
Average Confining Pressure, psi:	22.0
Failure Strain at Peak Stress,%:	6.46
Max. Deviator Stress (Γϖ),tsf:	1.41
Minor Principle Stress (ϖ6),tsf:	1.58
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	2.99
Undrained Shear Strength (tsf):	0.70

Failure Type: Bulge W/ Shear Planes



Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

psi

Unconsolidated Undrained Triaxial Test ASTM D 2850

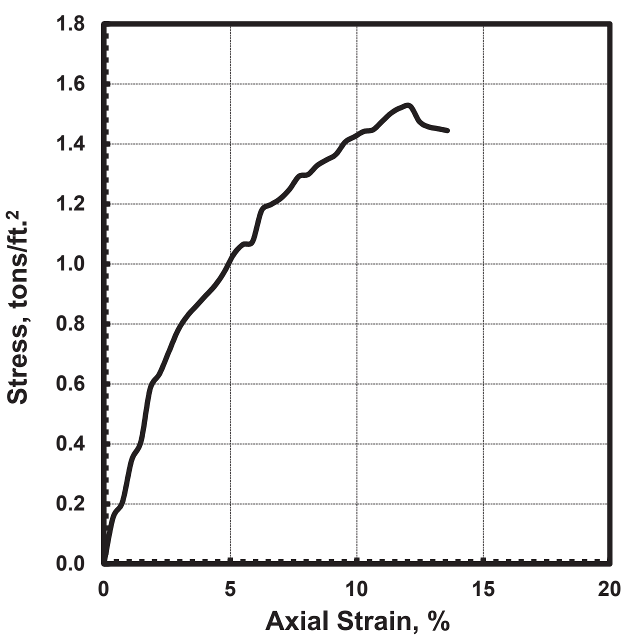
Test Date: *7/7/2016*

Project Name:	*Upper Levee Segment*			**Project Number:**	*0328-1663*
Boring Number:	*B-4*	**Depth, feet:**	*42-44'*	**Sample No./ID:**	*16*
Liquid Limit:	*47*	**Plastic Limit:**	*17*	**Plasticity Index:**	*31*
Hand Pen:	*1*	**Torvane:**	*NA*	**Specific Gravity:**	*2.700*
Classification:	*Medium Brown Silty Clay*			**Sample Condition:**	*Undisturbed*

Oven:	02OV328	**Scale:**	03DS328	**Calipers:**	2DC328
Load Cell:	01LC328	**Cell Pressure:**	AIR	**Load Frame:**	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.660	2.660	2.660	2.660	2.05
Height (H), in.:	5.450	5.450	5.450	5.450	
Area, ft^2:	0.039	Volume, ft^3:	0.018	Piston Load, lbs:	2.42

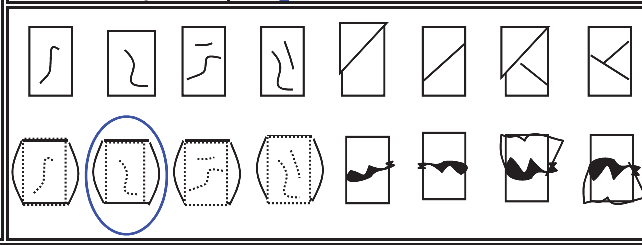
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	26	-	Wet Weight, gms:	905.62	905.62
Tare Wt., gm:	11.03	-	Dry Weight, gms:	738.75	
Wet Wt+Tare, gm:	66.93	-			
Dry Wt.+Tare, gm:	56.63	-	Wet Unit Wt.,pcf	113.9	-
Wt. of Water, gm:	10.3	-	Dry Unit Wt, pcf:	92.9	-
Dry Soil Wt., gm:	45.6	-	Void Ratio, pcf:	0.813	-
Moisture:	22.6%	-	Saturation, %:	75.0%	-



RESULTS

Actual Strain Rate, (%/min.):	0.69
Average Confining Pressure, psi:	27.0
Failure Strain at Peak Stress,%:	12.11
Max. Deviator Stress (Γϖ),tsf:	1.53
Minor Principle Stress (ϖ6),tsf:	1.94
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	3.47
Undrained Shear Strength (tsf):	**0.76**

Failure Type: Bulge



Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

psi

Unconsolidated Undrained Triaxial Test ASTM D 2850

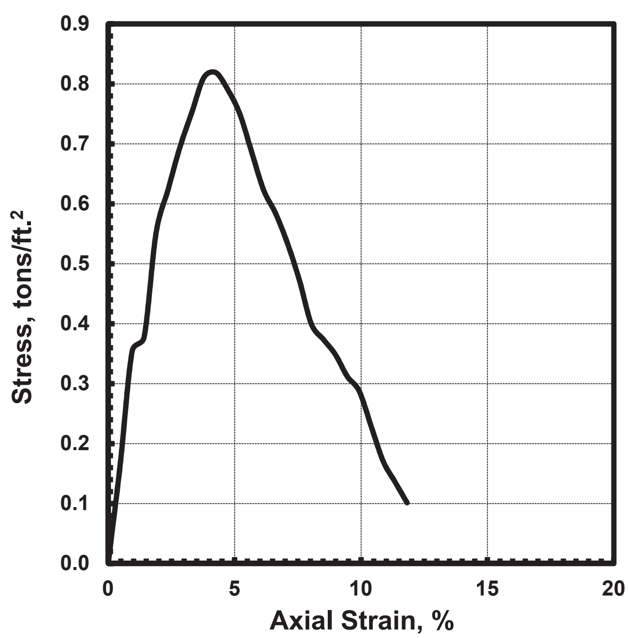
Test Date: *7/7/2016*

Project Name:	*Upper Levee Segment*			Project Number:	*0328-1663*
Boring Number:	*B-4*	Depth, feet:	*48-50'*	Sample No./ID:	*20*
Liquid Limit:	*52*	Plastic Limit:	*20*	Plasticity Index:	*32*
Hand Pen:	*4.5+*	Torvane:	*NA*	Specific Gravity:	*2.700*
Classification:	*Medium Brown Silty Clay*			Sample Condition:	*Undisturbed*

Oven:	02OV328	Scale:	03DS328	Calipers:	2DC328
Load Cell:	01LC328	Cell Pressure:	AIR	Load Frame:	02COM328

Sample Data	1	2	3	Average	Height/Dia. Ratio
Diameter (D),in.:	2.680	2.680	2.680	2.680	1.58
Height (H), in.:	4.230	4.230	4.230	4.230	
Area, ft^2:	0.039	Volume, ft^3:	0.014	Piston Load, lbs:	2.42

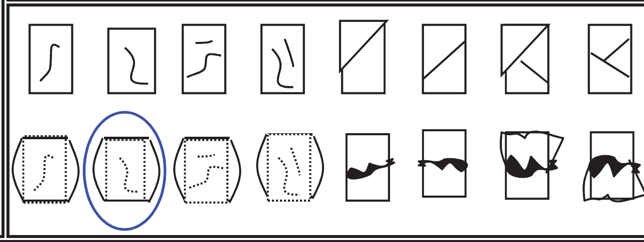
Mositure Data	Before (Trimings)	After Test (Middle)	Specimen	Before	After
Tare ID:	35	-	Wet Weight, gms:	787.64	787.64
Tare Wt., gm:	11.66	-	Dry Weight, gms:	620.72	
Wet Wt+Tare, gm:	52.57	-			
Dry Wt.+Tare, gm:	43.9	-	Wet Unit Wt.,pcf	125.7	-
Wt. of Water, gm:	8.67	-	Dry Unit Wt, pcf:	99.1	-
Dry Soil Wt., gm:	32.24	-	Void Ratio, pcf:	0.700	-
Moisture:	26.9%	-	Saturation, %:	103.7%	-



RESULTS

Actual Strain Rate, (%/min.):	0.66
Average Confining Pressure, psi:	30.0
Failure Strain at Peak Stress,%:	4.26
Max. Deviator Stress (Γϖ),tsf:	0.82
Minor Principle Stress (ϖ6),tsf:	2.16
Major Principle Strs (ϖ4≅Γϖ+ϖ6),tsf:	2.98
Undrained Shear Strength (tsf):	**0.41**

Failure Type: Bulge



Remarks:					
Tested By:	JOL	Computed By:	RM	Checked By:	HJL

psi

LAB TESTING

CONSOLIDATED UNDRAINED TRIAXIAL TEST

Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name: ***UPPER LEVEE SEGMENT*** ***Classification:*** ***Clay, Reddish Brown***

Project Number: ***03281663*** ***Boring Number:*** ***B-2*** ***Depth, feet:*** ***34-40*** ***Sample No./ID:***

Liquid Limit: ***63*** ***Plastic Limit:*** ***22*** ***Plasticity Index:*** ***41*** ***Percent Passing No. 200:*** ***97.0%***

Specimen/Stage Data	Before Test			After Consolidation or Shear		
Specimen/Stage No.	1	2	3	1	2	3
Diameter (D),in.:	2.817	2.861	2.880	2.829	2.841	2.795
Height (H), in.:	5.567	5.510	5.510	5.557	5.438	5.389
Cross-Sectional Area, in^2	6.233	6.429	6.514	6.286	6.341	6.137
Vol. (Vo, Vf), cm^3:	568.6	580.5	588.2	572.4	565.0	541.9
Moisture, {Wo, Wf} %:	30.5%	30.2%	26.7%	30.4%	29.7%	23.4%
Wet Soil Wt. {Mo, Mf}, ,gm:	1107.50	1102.48	1137.20	1106.54	1097.92	1106.87
Wet Unit Weight, pcf:	121.6	118.52	120.7	120.6	121.26	127.4
Dry Unit Weight, pcf:	93.1	91.0	95.2	92.5	93.5	103.3
Specific Gravity (Assumed):	2.7	2.7	2.7	2.7	2.7	2.7
Void Ratio, eo, ef:	0.81	0.85	0.77	0.82	0.80	0.63
Degree of Saturation, So, Sf:	1.02	0.96	0.94	1.00	1.00	1.00

Description	Saturation/Consolidation		
Specimen/Stage No.	1	2	3
Method	Wet Mounting Method		
Cell Pressure, lbs/in^2	64.1	73.0	116.8
Back Pressure, lbs/in^2	44.7	35.8	46.8
B-Parameter	0.95	0.95	0.95
Consolidation Pressure, lbs/in^2	19.4	37.2	70.0
Volume Change After (ΓV), cm^3	9.2	21.8	35.5
Time for Consolidation, min.	1440	1440	1560
Failure Type: *1*	*Single Shear*		
2		*Single Shear*	
3			*Single Shear*

Equipment	Specimen/Stage 1	2	3
Oven:			
Scale:			
Calipers:			
Digital Dial:			
Load Frame:	Load Frame	Load Frame	GeoJac Load Frame-APR 12
Load Cell ID:	05LC342	05LC342	02LC342
DCDT:	01PPS342	01PPS342	LPT-885
Cell Pressure Transducer:	03PG342	03PG342	01PG342
Pore Pressure Transducer:	04PG342	04PG342	02PG342
Radial Drainage Fliter Strip:	Yes	Yes	Yes

Shear Data	Specimen/Stage 1	2	3
Total Shearing Time, min	3598	3598	1800
Strain Rate, %/hr	0.14	0.14	0.50
Axial Strain at Failure, %	15.02	15.10	13.99
Deviator Stress, lbs/in^2 (Γϖ)	29.91	40.56	59.29
Excess Pore Pressure, lbs/in^2 (u)	8.67	18.91	32.68
A-Parameter, ∤u/Γϖ)	0.29	0.47	0.55
Total Major Pincipal Stress, lbs/in^2 (ϖ4#≅#ϖ6#.#Γ)ϖ	48.74	77.65	128.73
Total Minor Pincipal Stress, lbs/in^2#+ϖ6,	18.84	37.08	69.44
Effectivel Major Pincipal Stress, lbs/in^2 (χϖχ4#≅#ϖ4#0)#	40.08	58.74	96.05
Effectivel Minor Pincipal Stress, lbs/in^2#χϖχ6#≅#ϖ6#0#	10.17	18.18	36.76

Remarks:

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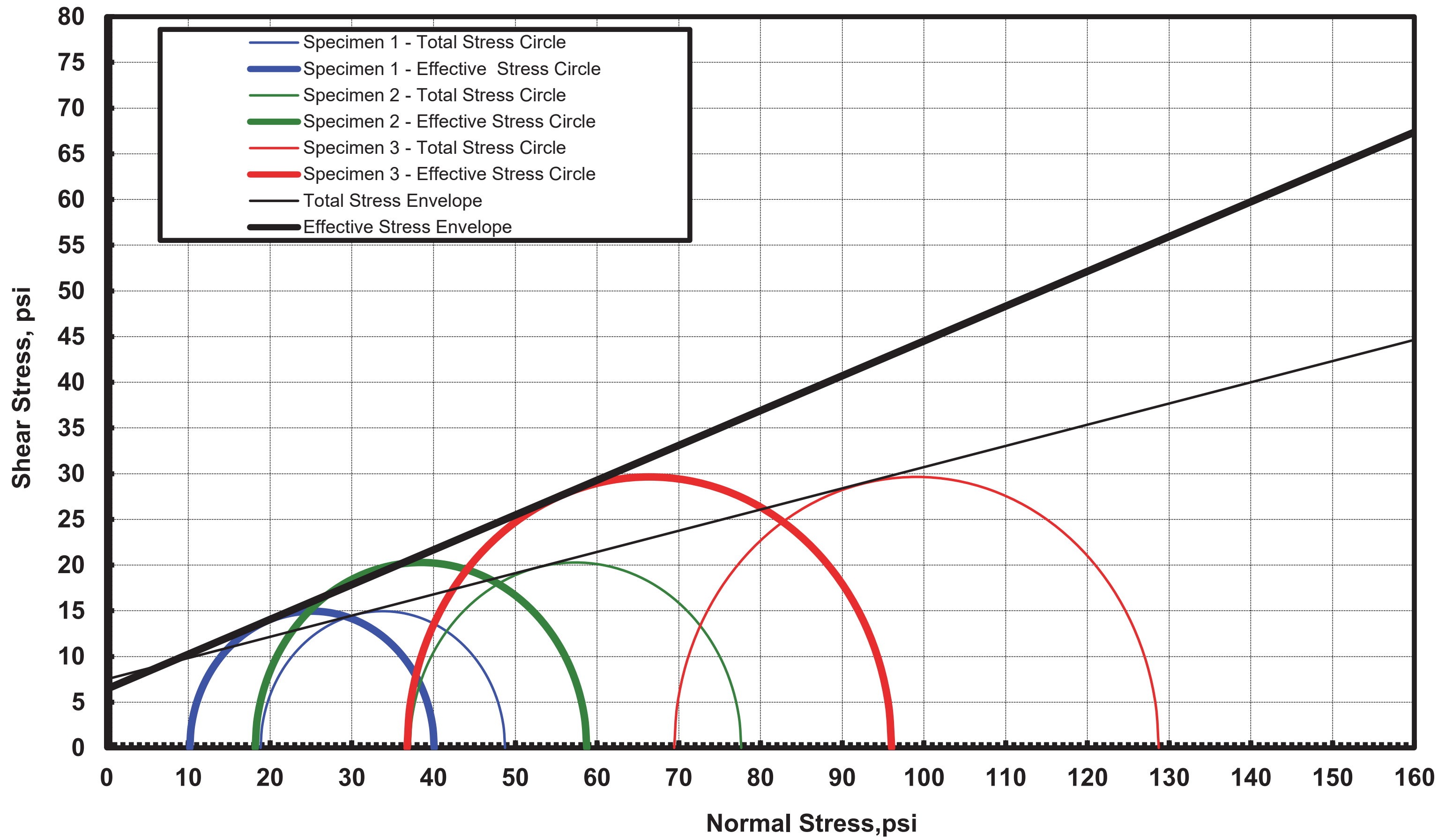
Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name:	UPPER LEVEE SEGMENT			***Classification:***	Clay, Reddish Brown		
Project Number:	0328-1663	***Boring Number:***	B2	***Depth, feet:***	34-40	***Sample No./ID:***	0
Cohesion (C_T), ksf:	1.08	***Friction Angle(ϕ_T), deg:***	13.1	***Cohesion (C_d), ksf:***	0.91	***Friction Angle(ϕ_d), deg:***	20.7

Remarks:



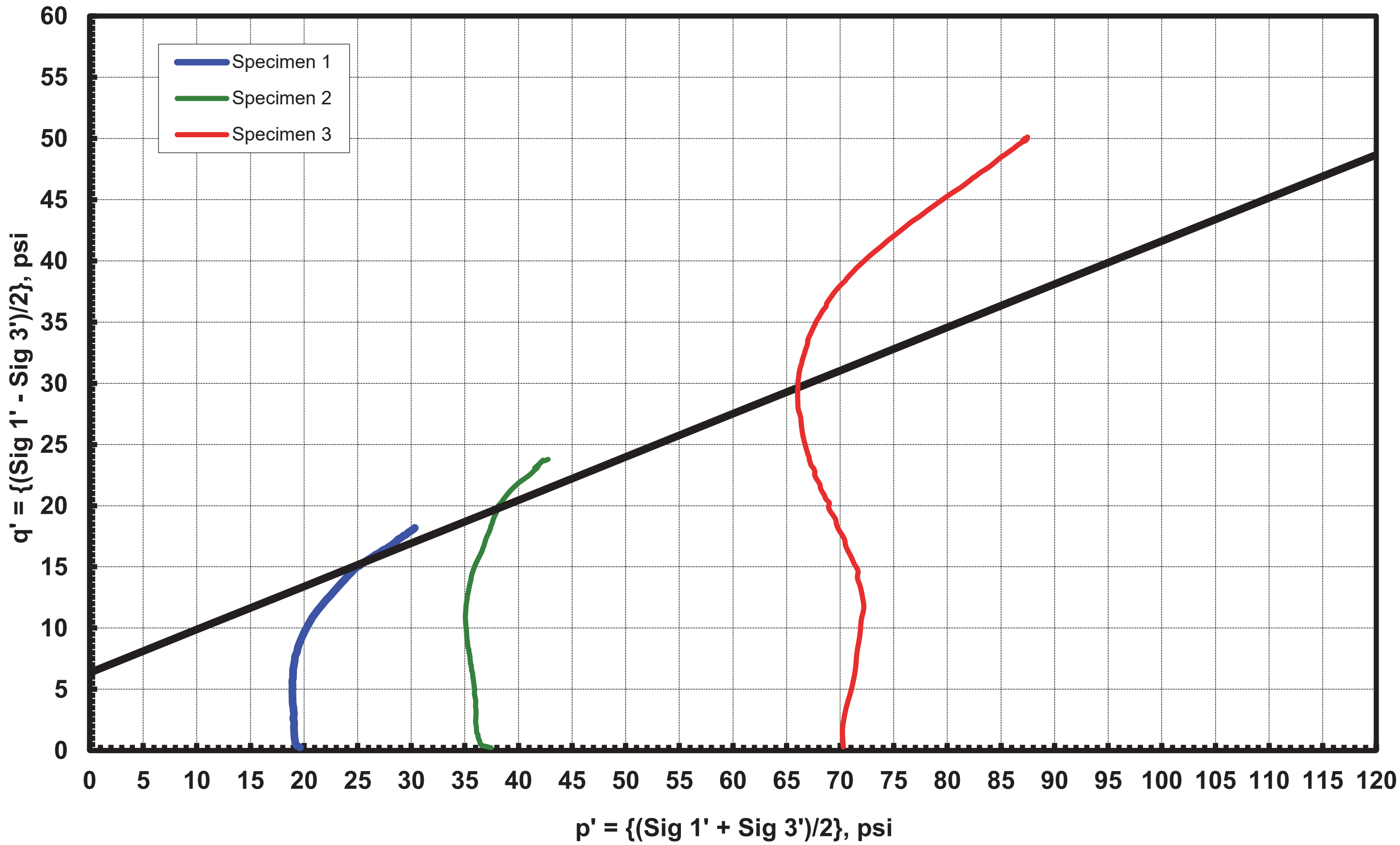
Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name: *UPPER LEVEE SEGMENT* ***Classification:*** *Clay, Reddish Brown*

Project Number: *0328-1663* ***Boring Number:*** *B2* ***Depth, feet:*** *34-40* ***Sample No./ID:*** *0*

Cohesion (C_d), ksf: *0.91* ***Friction Angle(ϕ_d), deg:*** *20.7*

Remarks:

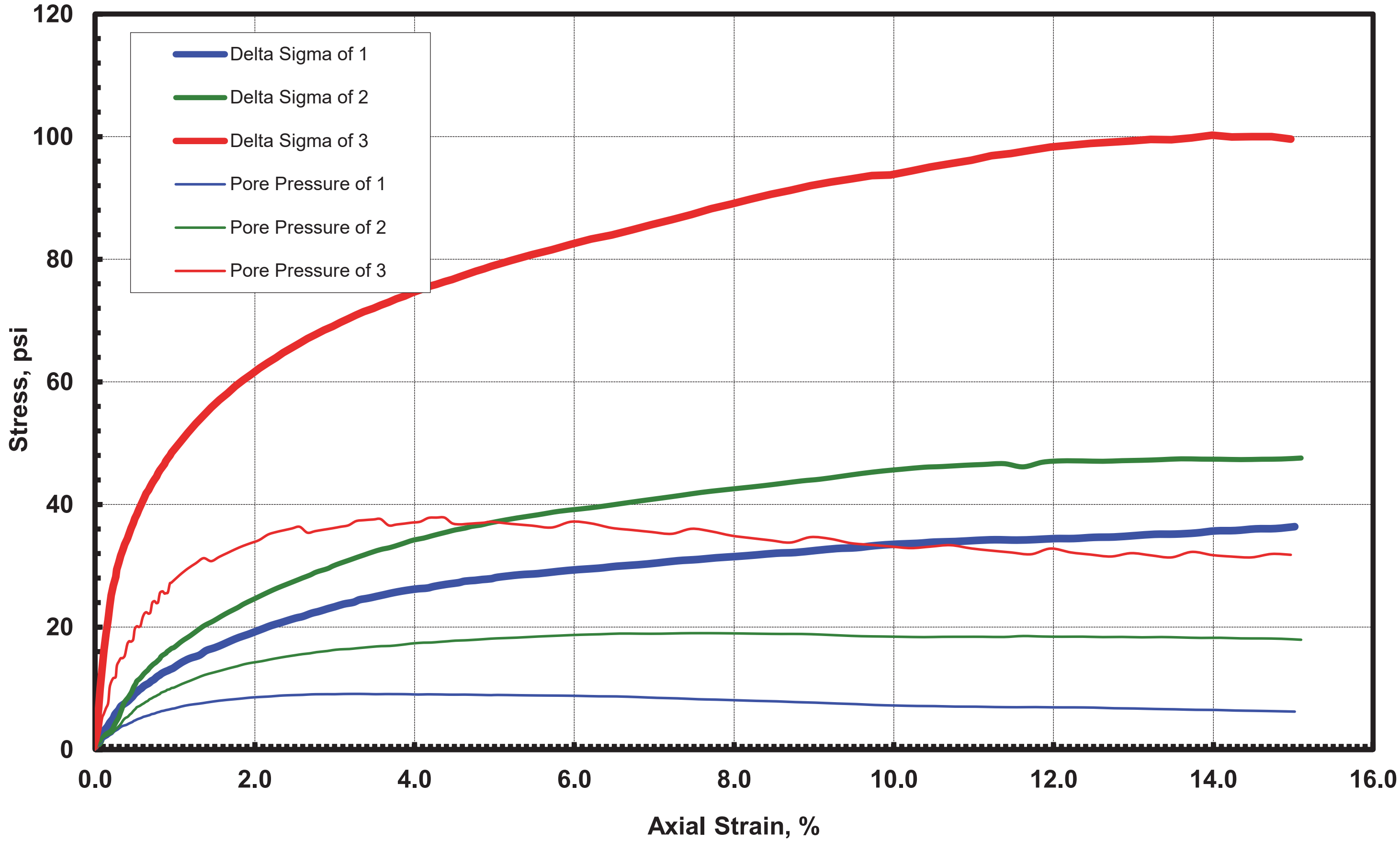


Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name: *UPPER LEVEE SEGMENT* ***Classification:*** *Clay, Reddish Brown*

Project Number: *0328-1663* ***Boring Number:*** *B2* ***Depth, feet:*** *34-40* ***Sample No./ID:*** *Sample1*

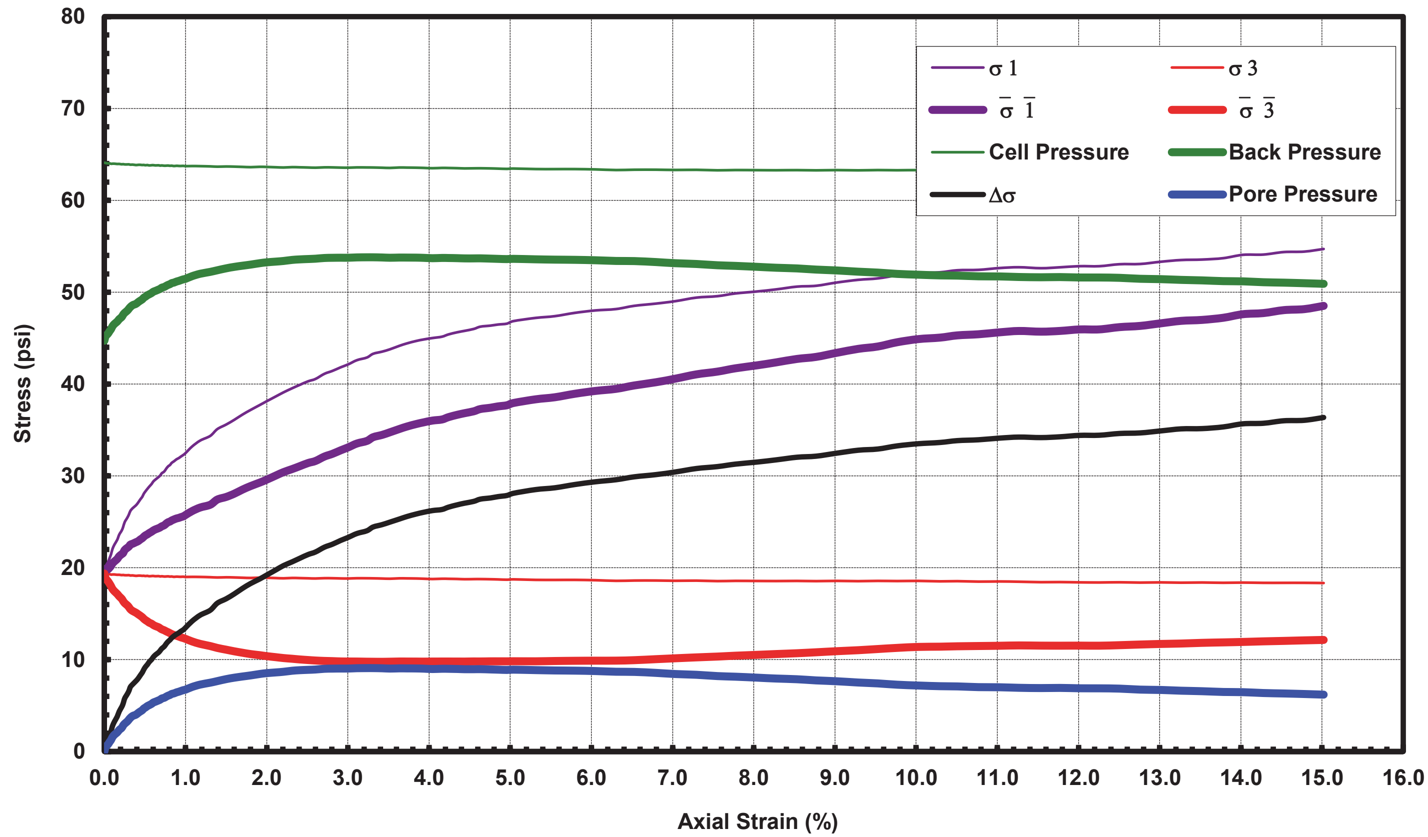
Remarks:



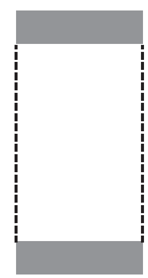
Sigma-1 CU B-2 34-40 ft.xlsm

Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name:	*UPPER LEVEE SEGMENT*			***Classification:***	*Clay, Reddish Brown*		
Project Number:	*0328-1663*	***Boring Number:***	*B2*	***Depth, feet:***	*36*	***Sample No./ID:***	*Sample1*
Specimen/Stage:	*Specimen 1*			***Effective Confining Pressure, psi:***	*19.4*	***Failure Type:***	*Single Shear*
Remarks:							



Before Shearing



After Shearing

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Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

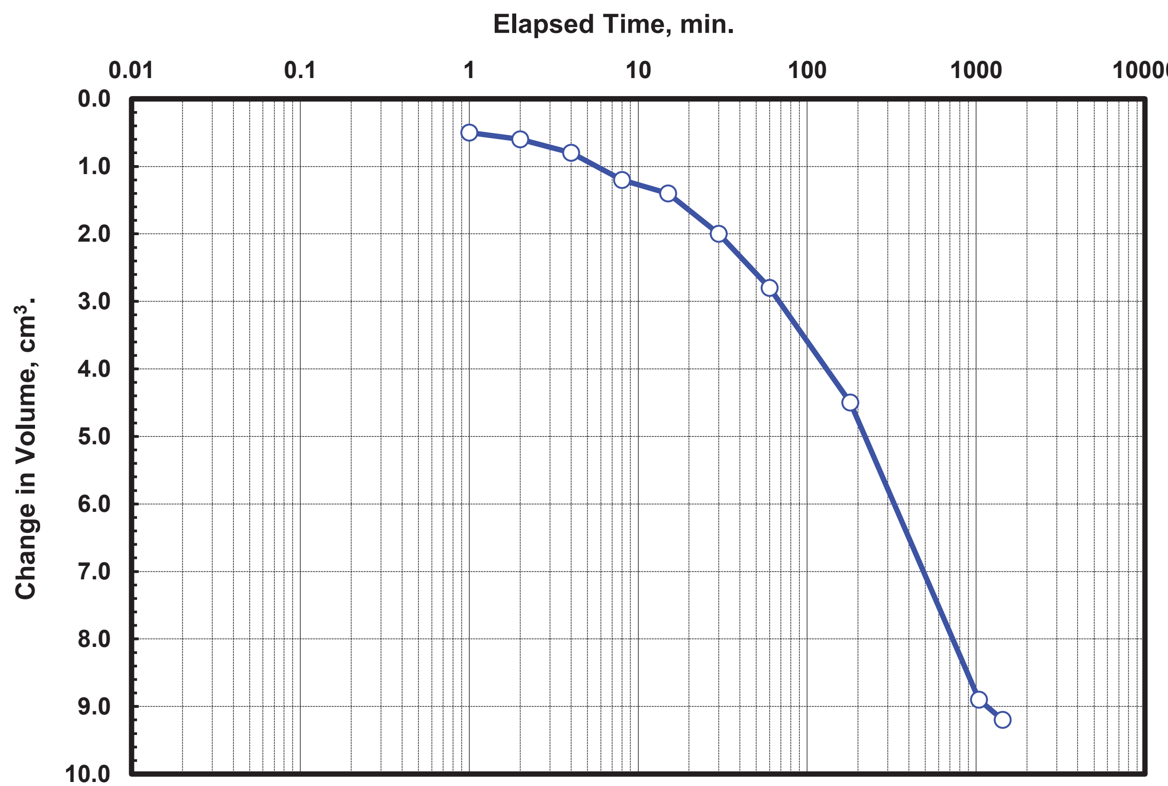
Project Name: UPPER LEVEE SEGMENT ***Classification:*** Clay, Reddish Brown

Project Number: 0328-1663 ***Boring Number:*** B2 ***Depth, feet:*** 36 ***Sample No./ID:*** Sample1

Specimen/Stage: Specimen 1 ***Effective Confining Pressure, psi:*** 19.4

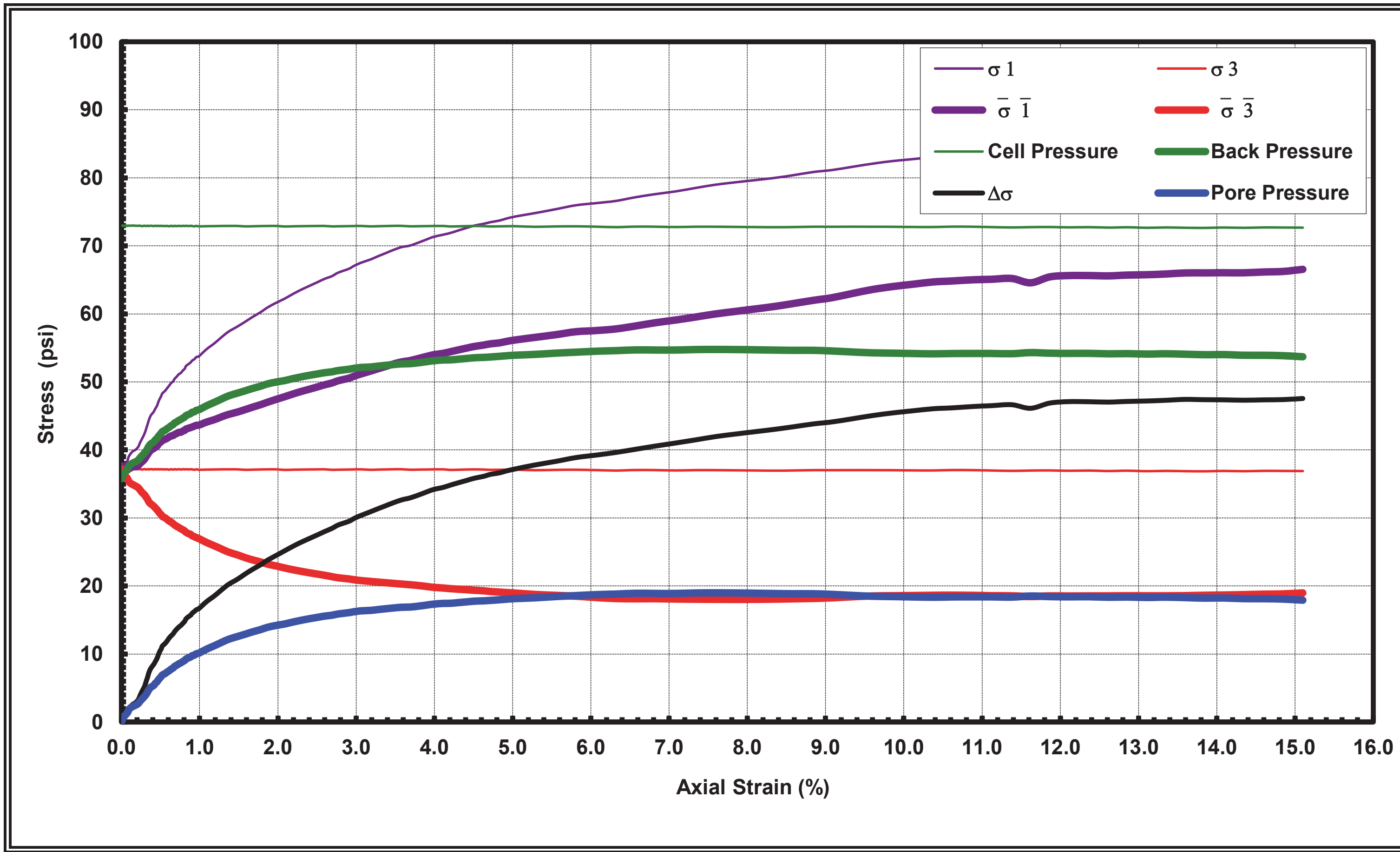
Remarks:

Date	Clock Time	Elaspsed Time, min.	Burrette Readings	Volume cm^3
7/5/2016	3:40:00 PM	0.0	23.0	0.0
7/5/2016	3:41:00 PM	1.0	22.5	0.5
7/5/2016	3:42:00 PM	2.0	22.4	0.6
7/5/2016	3:44:00 PM	4.0	22.2	0.8
7/5/2016	3:48:00 PM	8.0	21.8	1.2
7/5/2016	3:55:00 PM	15.0	21.6	1.4
7/5/2016	4:10:00 PM	30.0	21.0	2.0
7/5/2016	4:40:00 PM	60.0	20.2	2.8
7/5/2016	6:40:00 PM	180.0	18.5	4.5
7/6/2016	9:00:00 AM	1040.0	14.1	8.9
7/6/2016	3:40:00 PM	1440.0	13.8	9.2

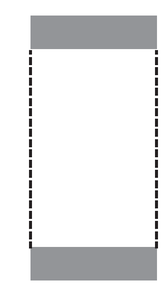


Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name:	*UPPER LEVEE SEGMENT*			***Classification:***	*Clay, Reddish Brown*		
Project Number:	*03281663-1*	***Boring Number:***	*B2*	***Depth, feet:***	*38*	***Sample No./ID:***	*Sample2*
Specimen/Stage:	*Specimen 2*			***Effective Confining Pressure, psi:***	*37.2*	***Failure Type:***	*Single Shear*
Remarks:							



Before Shear



After Shear

Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

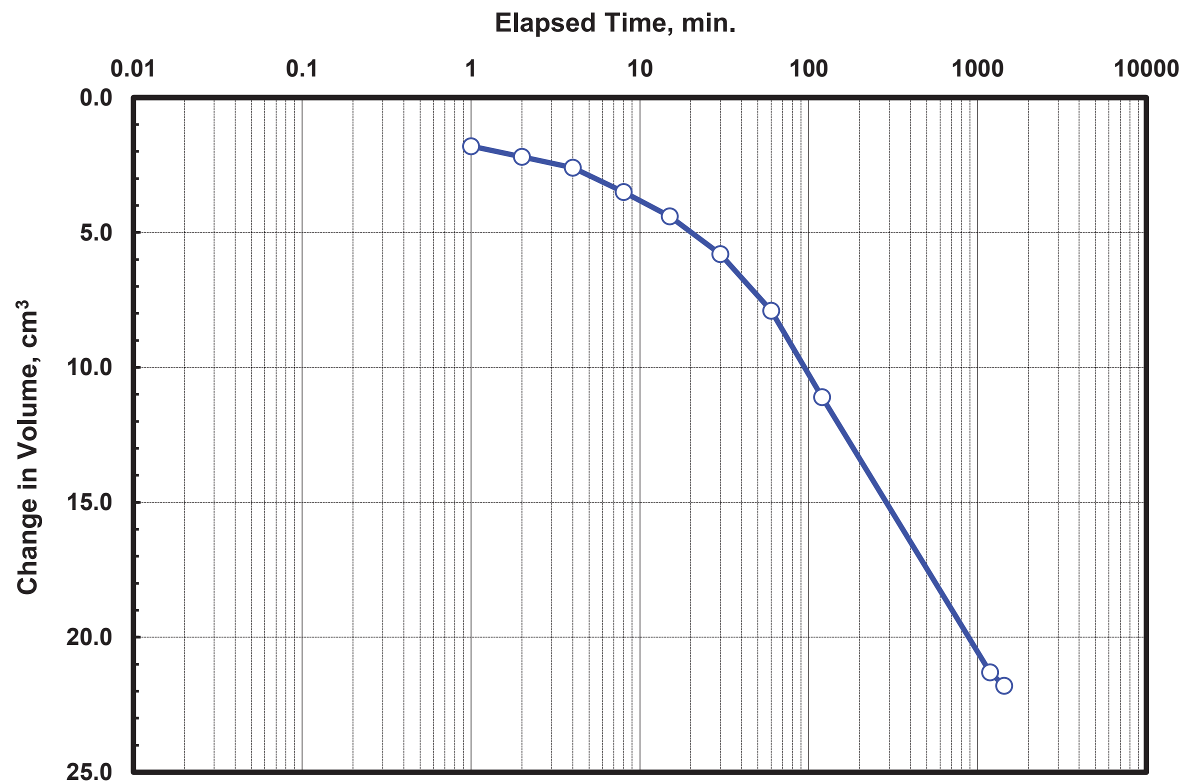
Project Name: *UPPER LEVEE SEGMENT* ***Classification:*** *Clay, Reddish Brown*

Project Number: *03281663-1* ***Boring Number:*** *B2* ***Depth, feet:*** *38* ***Sample No./ID:*** *Sample2*

Specimen/Stage: *Specimen 2* ***Effective Confining Pressure, psi:*** *37.2*

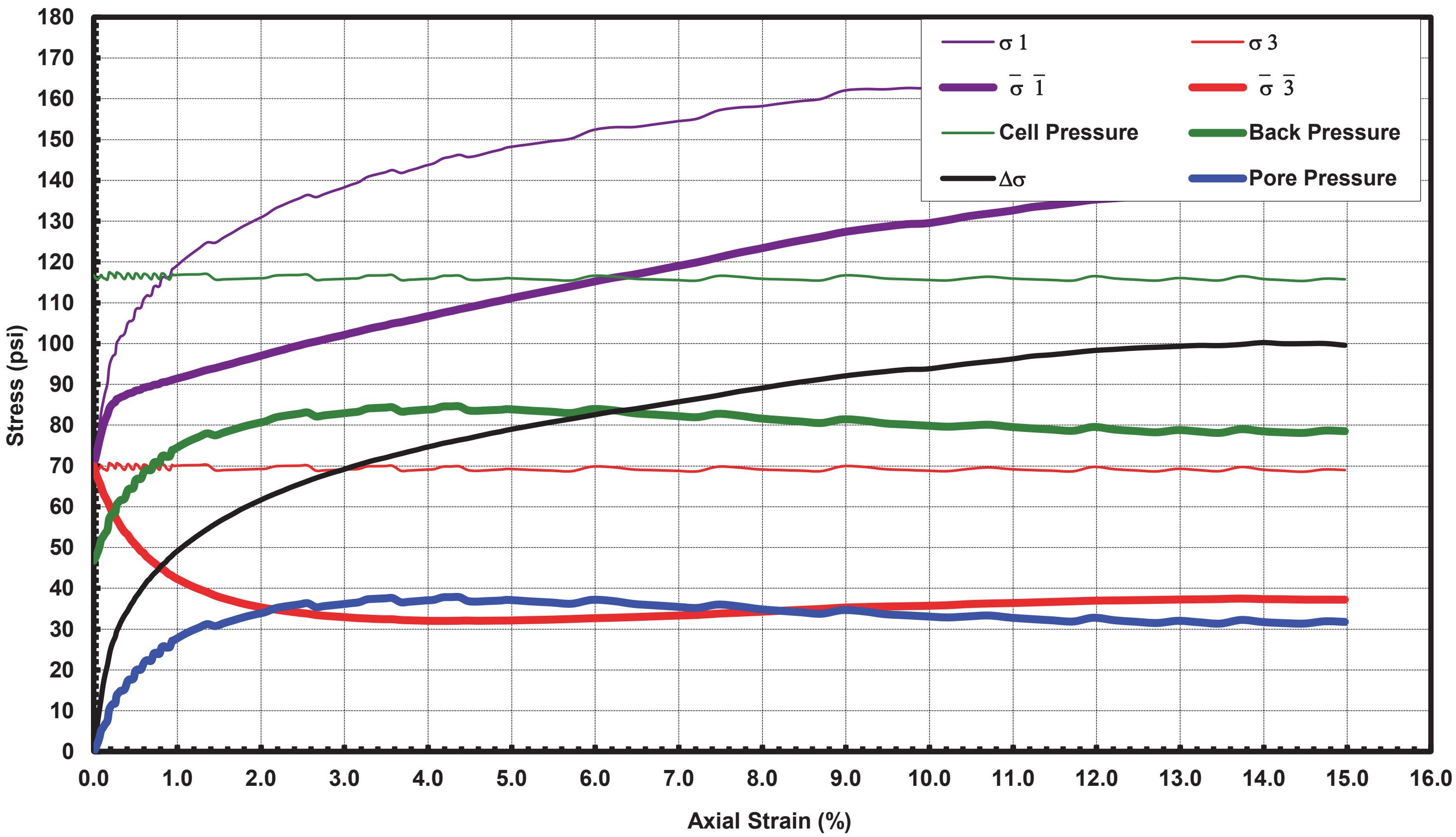
Remarks:

Date	Clock Time	Elaspsed Time, min.	Burrette Readings	Volume Cm^3
7/11/2016	1:00:00 PM	0.0	24.0	0.0
7/11/2016	1:01:00 PM	1.0	22.2	1.8
7/11/2016	1:02:00 PM	2.0	21.8	2.2
7/11/2016	1:04:00 PM	4.0	21.4	2.6
7/11/2016	1:08:00 PM	8.0	20.5	3.5
7/11/2016	1:15:00 PM	15.0	19.6	4.4
7/11/2016	1:30:00 PM	30.0	18.2	5.8
7/11/2016	2:00:00 PM	60.0	16.1	7.9
7/11/2016	3:00:00 PM	120.0	12.9	11.1
7/12/2016	8:45:00 AM	1185.0	2.7	21.3
7/12/2016	1:00:00 PM	1440.0	2.2	21.8

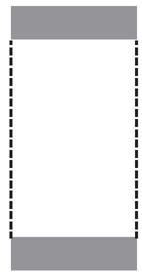


Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name:	*UPPER LEVEE SEGMENT*			***Classification:***	*Clay, Reddish Brown*		
Project Number:	*03281663-1*	***Boring Number:***	*B-2*	***Depth, feet:***	*40*	***Sample No./ID:***	*Sample3*
Specimen/Stage:	*Specimen 3*			***Effective Confining Pressure, psi:***	*70.0*	***Failure Type:***	*Single Shear*
Remarks:							



Before Shear



After Shear

Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

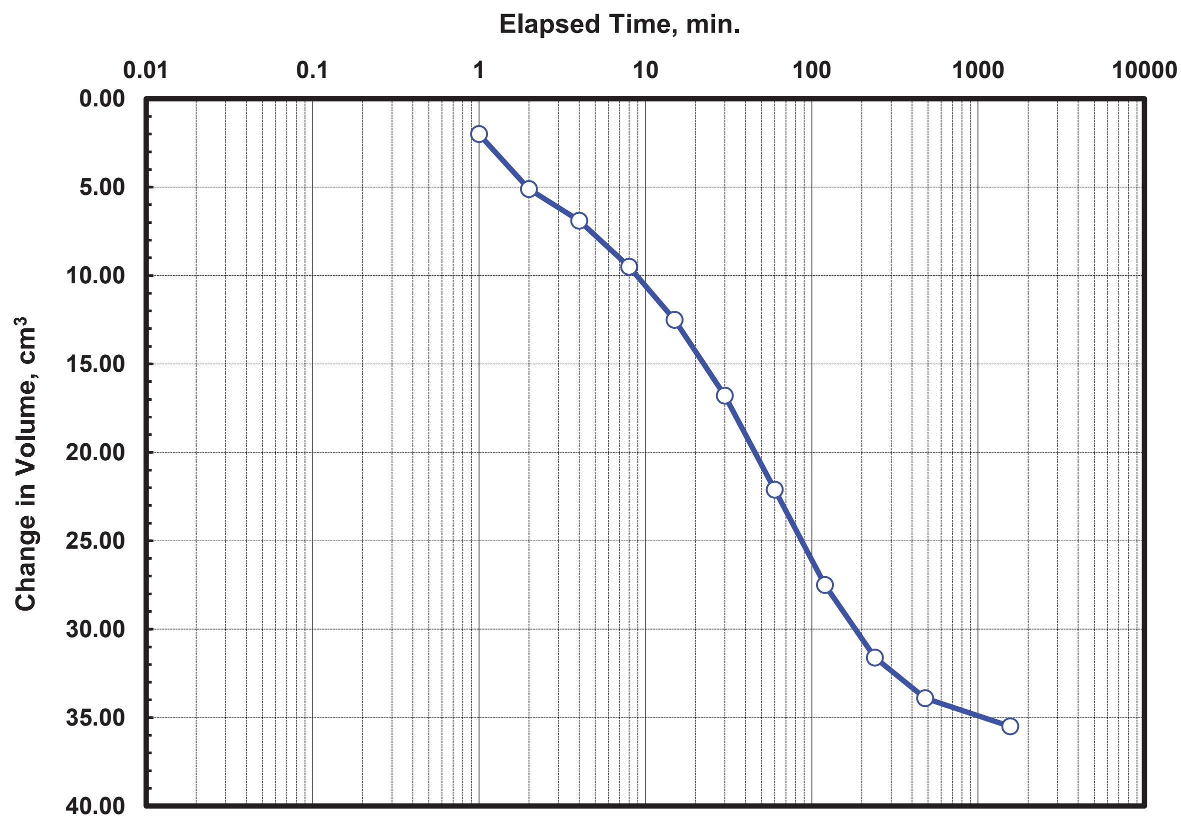
Project Name: *UPPER LEVEE SEGMENT* ***Classification:*** *Clay, Reddish Brown*

Project Number: *03281663-1* ***Boring Number:*** *B-2* ***Depth, feet:*** *40* ***Sample No./ID:*** *Sample3*

Specimen/Stage: *Specimen 3* ***Effective Confining Pressure, psi:*** *70.0*

Remarks:

Date	Clock Time	Elaspsed Time, min.	Burrette Readings	Volume cm^3
7/13/2016	9:00:00 AM	0.0	46.1	0.0
7/13/2016	9:01:00 AM	1.0	44.1	2.0
7/13/2016	9:02:00 AM	2.0	41.0	5.1
7/13/2016	9:04:00 AM	4.0	39.2	6.9
7/13/2016	9:08:00 AM	8.0	36.6	9.5
7/13/2016	9:15:00 AM	15.0	33.6	12.5
7/13/2016	9:30:00 AM	30.0	29.3	16.8
7/13/2016	10:00:00 AM	60.0	24.0	22.1
7/13/2016	11:00:00 AM	120.0	18.6	27.5
7/13/2016	1:00:00 PM	240.0	14.5	31.6
7/13/2016	5:00:00 PM	480.0	12.2	33.9
7/14/2016	11:00:00 AM	1560.0	10.6	35.5



Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name: *UPPER LEVEE SEGMENT* ***Classification:*** *Clay, Reddish Brown*

Project Number: *3281663* ***Boring Number:*** *B-3* ***Depth, feet:*** *30-38* ***Sample No./ID:***

Liquid Limit: *55* ***Plastic Limit:*** *22* ***Plasticity Index:*** *33* ***Percent Passing No. 200:*** *98.0%*

Specimen/Stage Data	Before Test			After Consolidation or Shear			Description	Saturation/Consolidation		
Specimen/Stage No.	1	2	3	1	2	3	Specimen/Stage No.	1	2	3
Diameter (D),in.:	2.815	2.864	2.868	2.831	2.831	2.800				
Height (H), in.:	5.501	5.511	5.512	5.481	5.451	5.441	Method	Wet Mounting Method		
Cross-Sectional Area, in^2	6.224	6.442	6.460	6.293	6.293	6.157	Cell Pressure, lbs/in^2	52.1	79.6	103.6
Vol. (Vo, Vf), cm^3:	561.0	581.8	583.5	565.2	562.1	549.0	Back Pressure, lbs/in^2	35.9	49.0	34.9
Moisture, {Wo, Wf} %:	31.8%	33.8%	22.6%	30.1%	31.8%	19.9%	B-Parameter	0.95	0.95	0.95
Wet Soil Wt. {Mo, Mf}, ,gm:	1109.40	1092.50	1181.70	1095.12	1076.27	1156.01	Consolidation Pressure, lbs/in^2	16.1	30.6	68.6
Wet Unit Weight, pcf:	123.4	117.18	126.4	120.9	119.48	131.4	Volume Change After (ΓV), cm^3	12.5	21.5	22.5
Dry Unit Weight, pcf:	93.6	87.6	103.1	92.9	90.7	109.6	Time for Consolidation, min.	1440	1440	1440
Specific Gravity (Assumed):	2.7	2.7	2.7	2.7	2.7	2.7	Failure Type: 1	Bulge		
Void Ratio, eo, ef:	0.80	0.92	0.63	0.81	0.86	0.54	2		Bulge	
Degree of Saturation, So, Sf:	1.07	0.99	0.96	1.00	1.00	1.00	3			Bulge

Equipment	Specimen/Stage			Shear Data	Specimen/Stage		
	1	2	3		1	2	3
Oven:				Total Shearing Time, min	1800	3336	1800
Scale:				Strain Rate, %/hr	0.50	0.27	0.50
Calipers:				Axial Strain at Failure, %	15.02	15.00	14.97
Digital Dial:				Deviator Stress, lbs/in^2 (Γϖ)	22.61	31.02	53.50
Load Frame:	GeoJac Load Frame-APR 12	GeoJac Load Frame-APR 12	GeoJac Load Frame-APR 12	Excess Pore Pressure, lbs/in^2 (u)	8.47	15.73	37.72
Load Cell ID:	02LC342	02LC342	02LC342	A-Parameter, ⊣u/Γϖ)	0.37	0.51	0.71
DCDT:	LPT-885	LPT-885	LPT-885	Total Major Pincipal Stress, lbs/in^2 (ϖ4#≅#ϖ6#.#Γ)ϖ	38.73	61.58	121.89
Cell Pressure Transducer:	01PG342	01PG342	01PG342	Total Minor Pincipal Stress, lbs/in^2#+ϖ6,	16.12	30.56	68.40
Pore Pressure Transducer:	02PG342	02PG342	02PG342	Effectivel Major Pincipal Stress, lbs/in^2 (χϖχ4#≅#ϖ4#0#	30.26	45.85	84.17
Radial Drainage Fliter Strip:	Yes	Yes	Yes	Effectivel Minor Pincipal Stress, lbs/in^2#χϖχ6#≅#ϖ6#0#	7.65	14.83	30.68

Remarks:

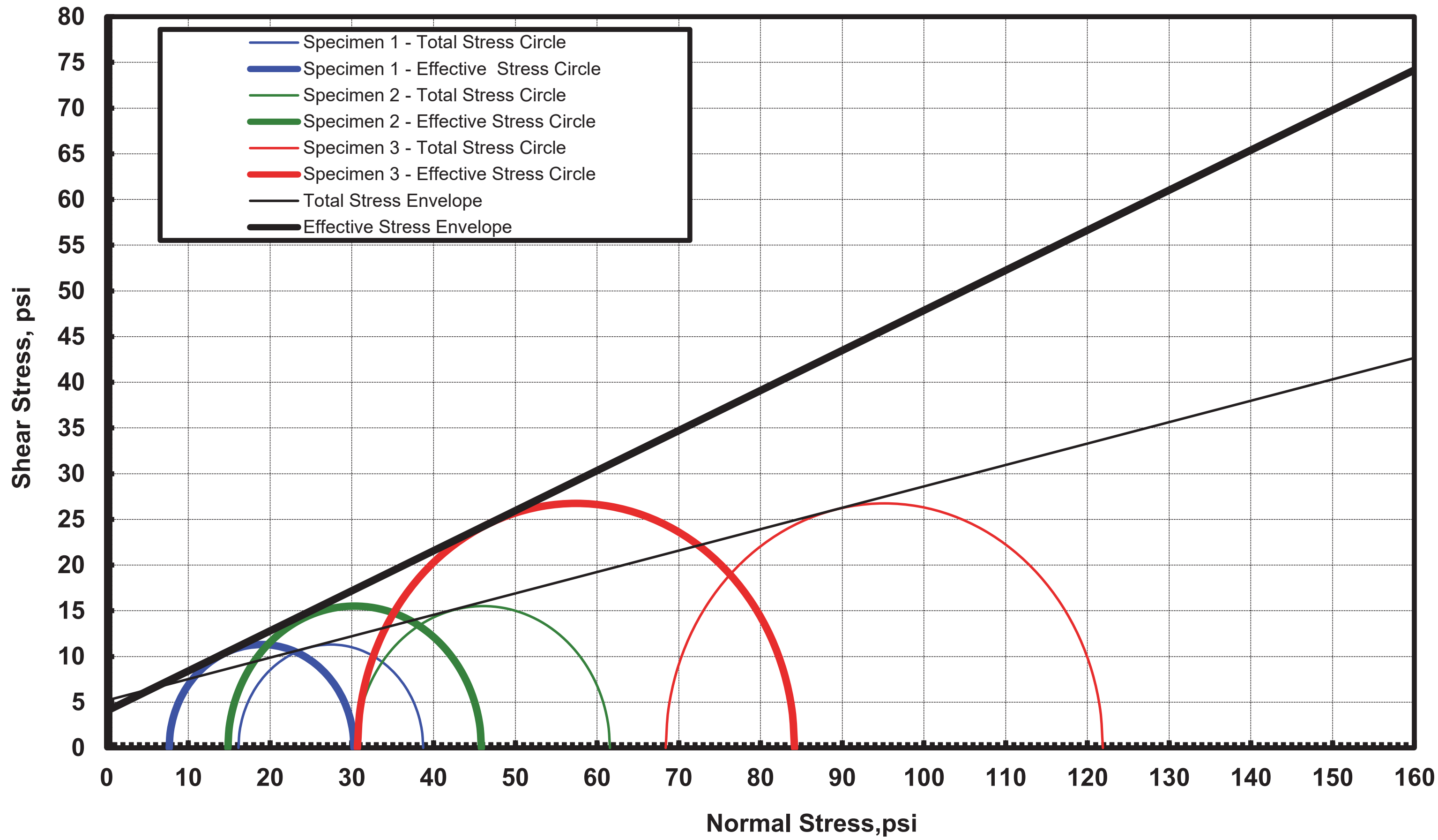
Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name: UPPER LEVEE SEGMENT **Classification:** Clay, Reddish Brown

Project Number: 0328-1663 **Boring Number:** B3 **Depth, feet:** 30-38 **Sample No./ID:** 0

Cohesion (C_T), ksf: 0.75 **Friction Angle(ϕ_T), deg:** 13.2 **Cohesion (C_d), ksf:** 0.50 **Friction Angle(ϕ_d), deg:** 23.7

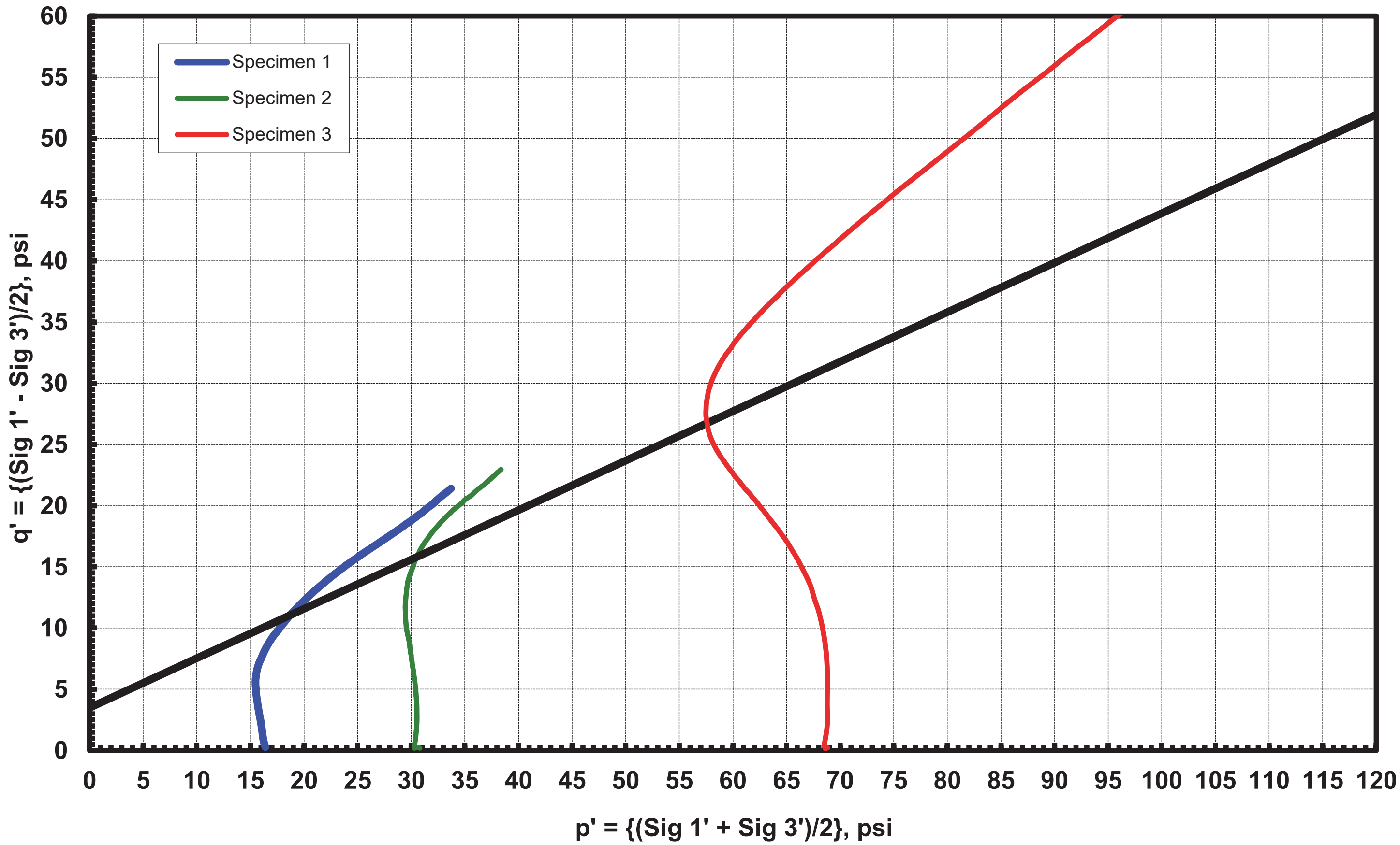
Remarks:



Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name:	*UPPER LEVEE SEGMENT*			***Classification:***	*Clay, Reddish Brown*		
Project Number:	*0328-1663*	***Boring Number:***	*B3*	***Depth, feet:***	*30-38*	***Sample No./ID:***	*0*
		Cohesion (C $_d$), ksf:	*0.50*	***Friction Angle(ϕ_d), deg:***	*23.7*		

Remarks:



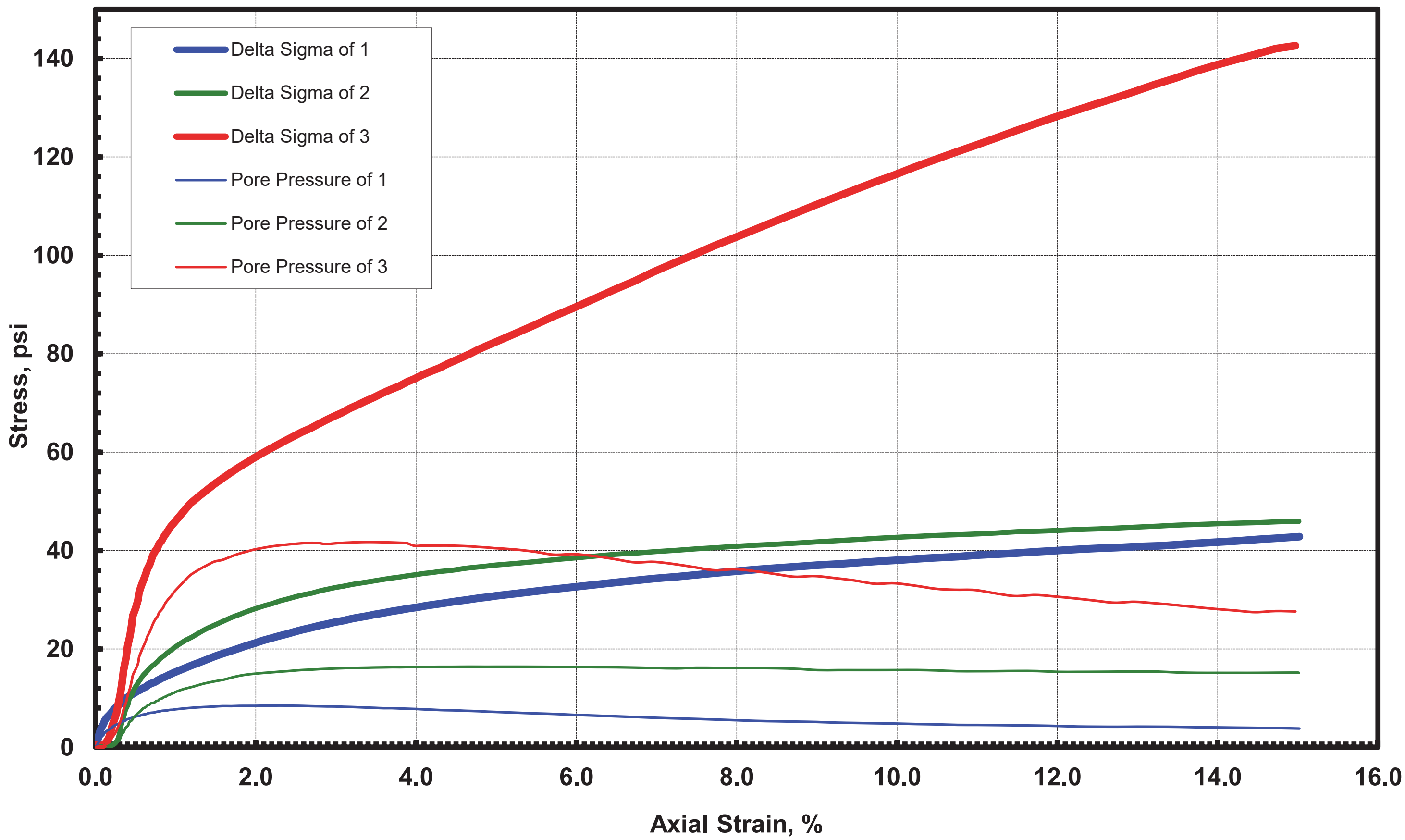
Sigma-1 CU B-3 30-38 ft.xlsm

Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name: UPPER LEVEE SEGMENT **Classification:** Clay, Reddish Brown

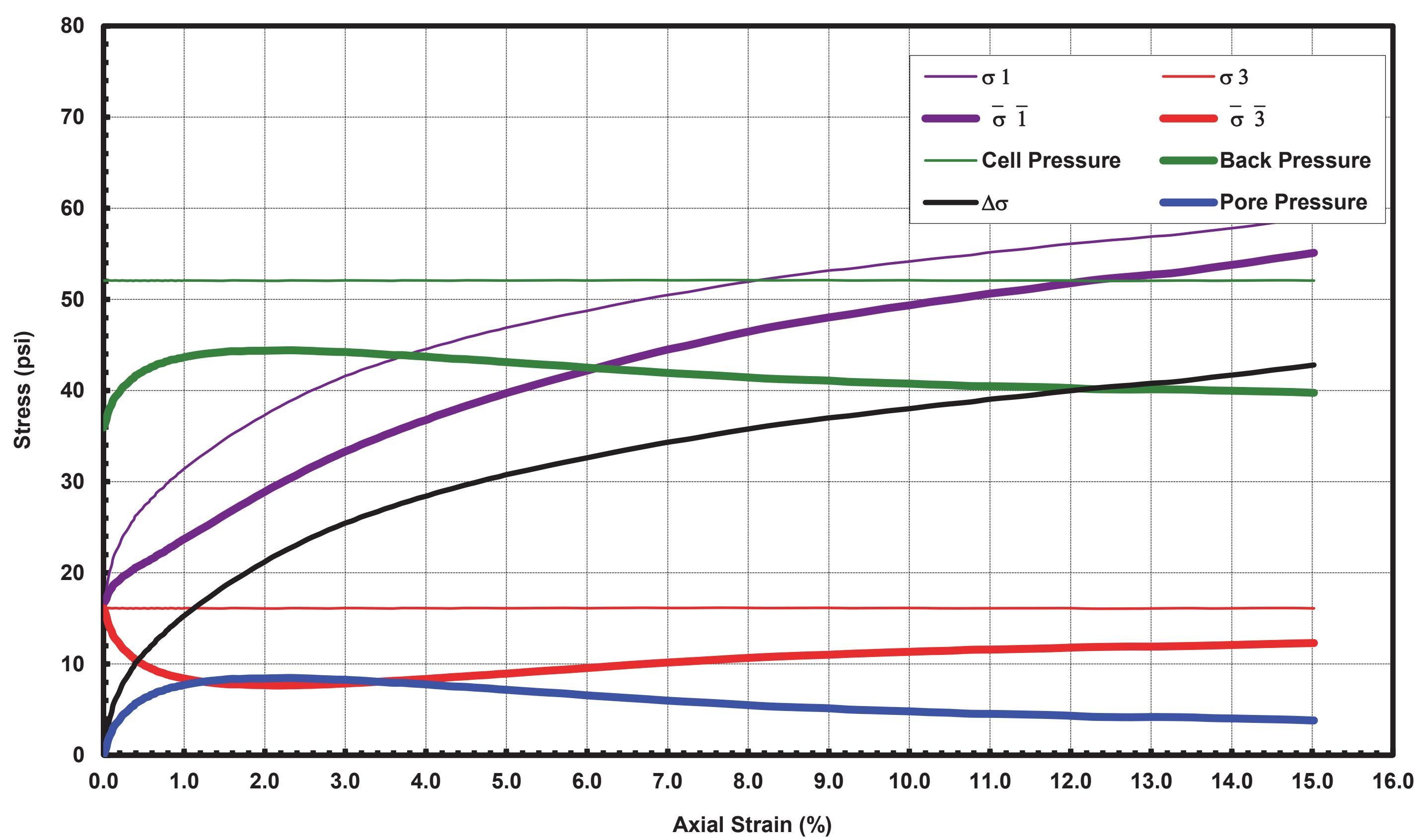
Project Number: 0328-1663 **Boring Number:** B3 **Depth, feet:** 30-38 **Sample No./ID:** Sample1

Remarks:

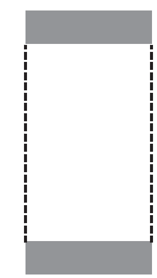


Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name:	*UPPER LEVEE SEGMENT*			***Classification:***	*Clay, Reddish Brown*		
Project Number:	*0328-1663*	***Boring Number:***	*B3*	***Depth, feet:***	*32*	***Sample No./ID:***	*Sample1*
Specimen/Stage:	*Specimen 1*			***Effective Confining Pressure, psi:***	*16.1*	***Failure Type:***	*Bulge*
Remarks:							



Before Shearing



After Shearing

Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name: *UPPER LEVEE SEGMENT* ***Classification:*** *Clay, Reddish Brown*

Project Number: *0328-1663* ***Boring Number:*** *B3* ***Depth, feet:*** *32* ***Sample No./ID:*** *Sample1*

Specimen/Stage: *Specimen 1* ***Effective Confining Pressure, psi:*** *16.1*

Remarks:

Date	Clock Time	Elaspsed Time, min.	Burrette Readings	Volume cm^3
7/5/2016	2:31:00 PM	0.0	23.0	0.0
7/5/2016	2:32:00 PM	1.0	22.0	1.0
7/5/2016	2:33:00 PM	2.0	21.7	1.3
7/5/2016	2:35:00 PM	4.0	21.2	1.8
7/5/2016	2:39:00 PM	8.0	20.4	2.6
7/5/2016	2:46:00 PM	15.0	19.8	3.2
7/5/2016	3:01:00 PM	30.0	17.8	5.2
7/5/2016	3:31:00 PM	60.0	15.9	7.1
7/5/2016	4:31:00 PM	120.0	13.8	9.2
7/5/2016	6:31:00 PM	240.0	12.4	10.6
7/6/2016	9:00:00 AM	1109.0	10.7	12.3
7/6/2016	2:31:00 PM	1440.0	10.5	12.5

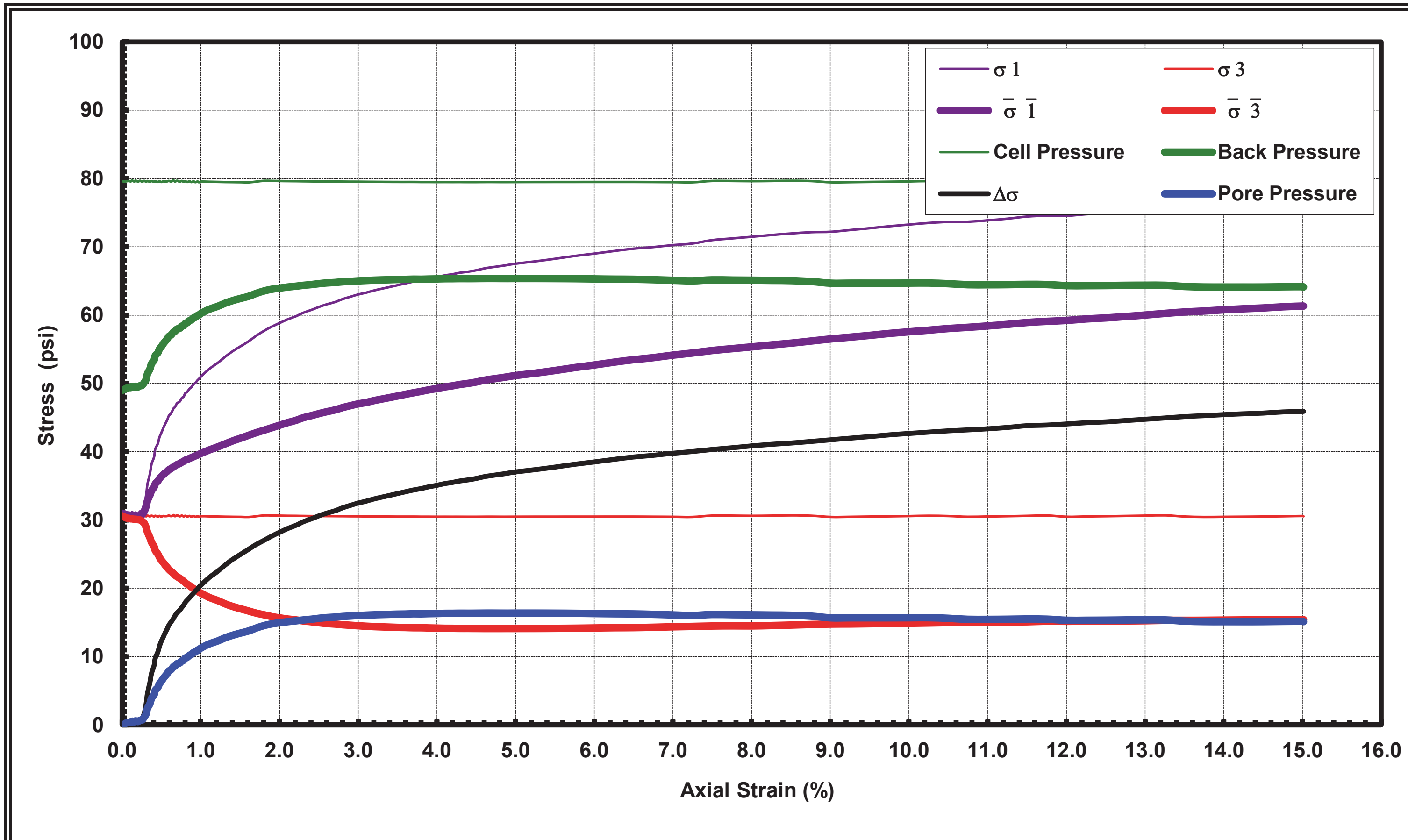
Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name: UPPER LEVEE SEGMENT | ***Classification:*** Clay, Reddish Brown

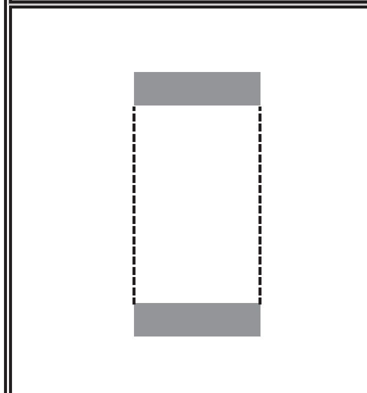
Project Number: 0328-1663 | ***Boring Number:*** B3 | ***Depth, feet:*** 34 | ***Sample No./ID:*** Sample2

Specimen/Stage: Specimen 2 | ***Effective Confining Pressure, psi:*** 30.6 | ***Failure Type:*** Bulge

Remarks:



Before Shear



After Shear

Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

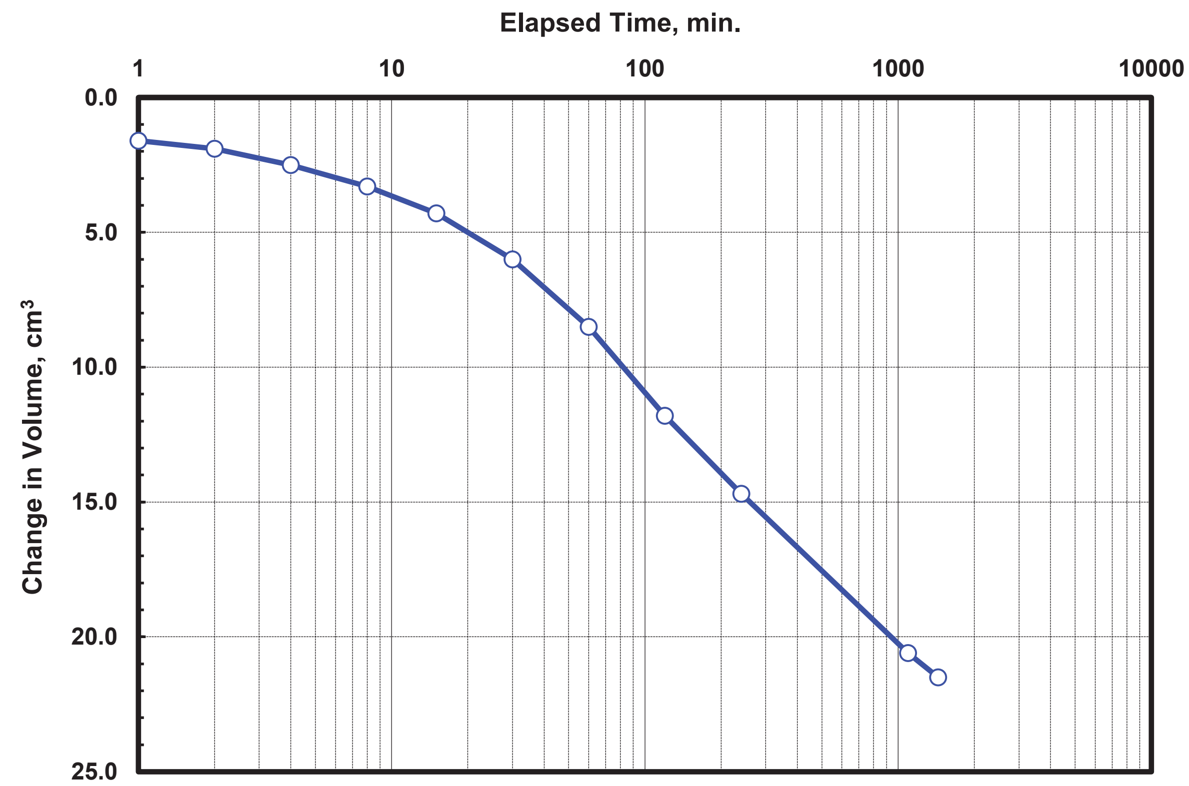
Project Name: *UPPER LEVEE SEGMENT* ***Classification:*** *Clay, Reddish Brown*

Project Number: *0328-1663* ***Boring Number:*** *B3* ***Depth, feet:*** *34* ***Sample No./ID:*** *Sample2*

Specimen/Stage: *Specimen 2* ***Effective Confining Pressure, psi:*** *30.6*

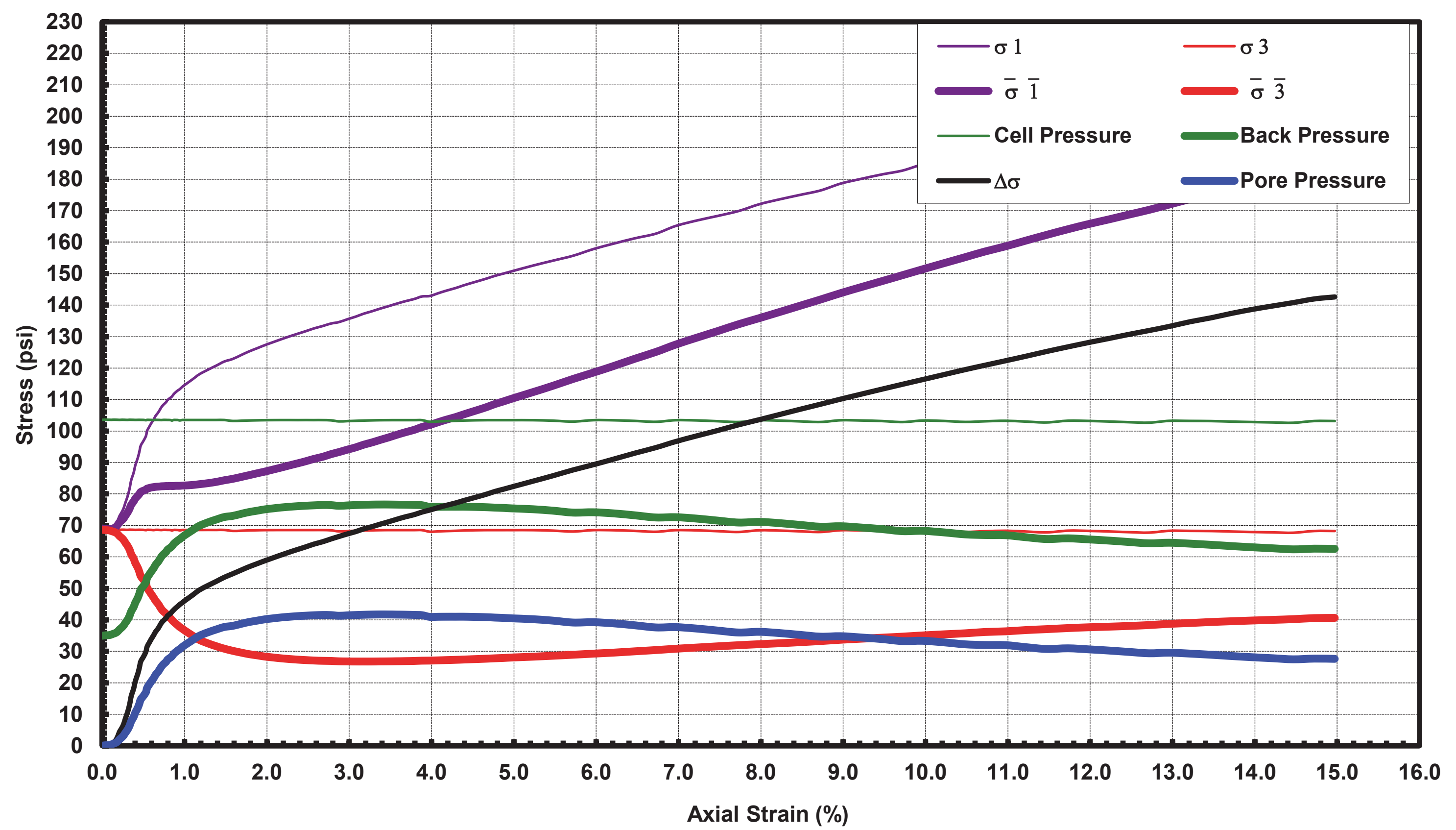
Remarks:

Date	Clock Time	Elaspsed Time, min.	Burrette Readings	Volume Cm^3
7/5/2016	2:45:00 PM	0.0	23.0	0.0
7/5/2016	2:46:00 PM	1.0	21.4	1.6
7/5/2016	2:47:00 PM	2.0	21.1	1.9
7/5/2016	2:49:00 PM	4.0	20.5	2.5
7/5/2016	2:53:00 PM	8.0	19.7	3.3
7/5/2016	3:00:00 PM	15.0	18.7	4.3
7/5/2016	3:15:00 PM	30.0	17.0	6.0
7/5/2016	3:45:00 PM	60.0	14.5	8.5
7/5/2016	4:45:00 PM	120.0	11.2	11.8
7/5/2016	6:45:00 PM	240.0	8.3	14.7
7/6/2016	9:00:00 AM	1095.0	2.4	20.6
7/6/2016	2:45:00 PM	1440.0	1.5	21.5

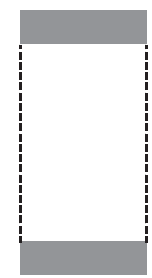


Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

Project Name:	***UPPER LEVEE SEGMENT***			***Classification:***	***Clay, Reddish Brown***		
Project Number:	***03281663-1***	***Boring Number:***	***B3***	***Depth, feet:***	***38***	***Sample No./ID:***	***3***
Specimen/Stage:	***Specimen 3***			***Effective Confining Pressure, psi:***	***68.6***	***Failure Type:***	***Bulge***
Remarks:							



Before Shear



After Shear

Sigma-1 CU B-3 30-38 ft.xlsm

Consolidated Undrained Triaxial Test with Pore Pressure Measurements (ASTM D 4767)

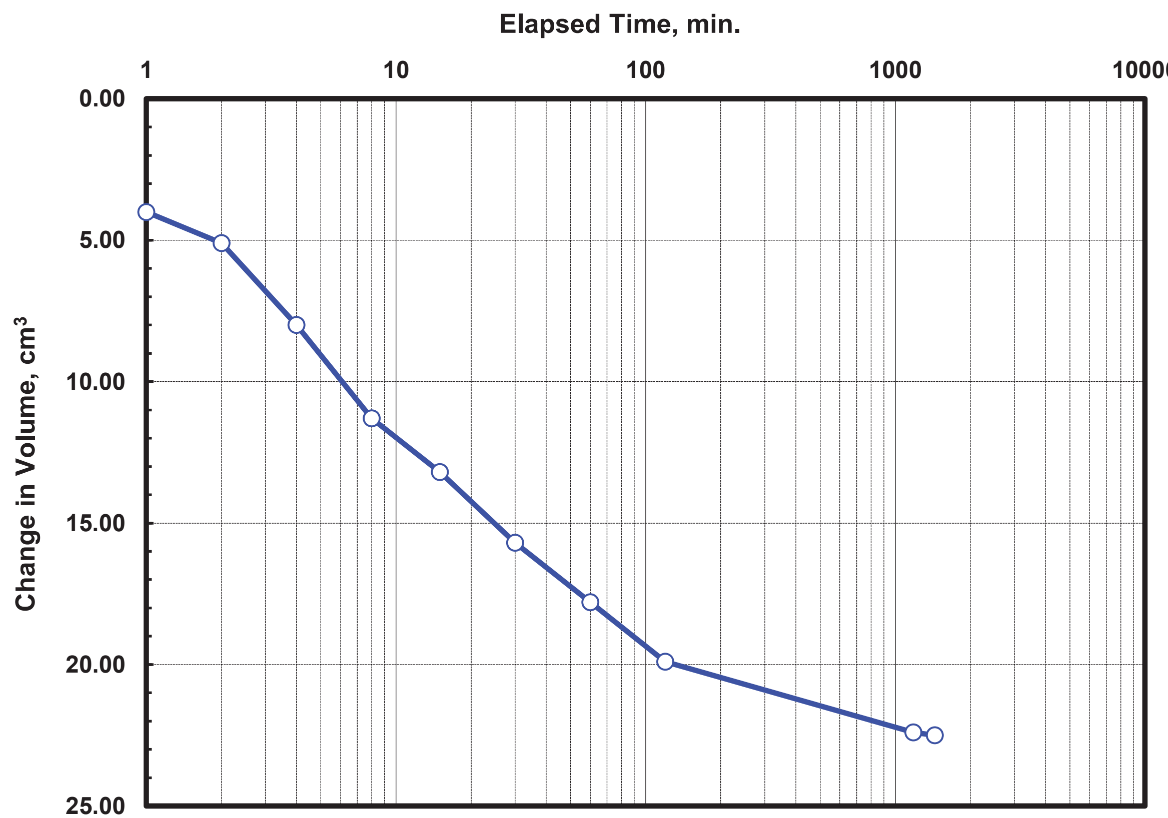
Project Name: UPPER LEVEE SEGMENT ***Classification:*** Clay, Reddish Brown

Project Number: 03281663-1 ***Boring Number:*** B3 ***Depth, feet:*** 38 ***Sample No./ID:*** 3

Specimen/Stage: Specimen 3 ***Effective Confining Pressure, psi:*** 68.6

Remarks:

Date	Clock Time	Elaspsed Time, min.	Burrette Readings	Volume cm^3
7/11/2016	1:10:00 PM	0.0	37.2	0.0
7/11/2016	1:11:00 PM	1.0	33.2	4.0
7/11/2016	1:12:00 PM	2.0	32.1	5.1
7/11/2016	1:14:00 PM	4.0	29.2	8.0
7/11/2016	1:18:00 PM	8.0	25.9	11.3
7/11/2016	1:25:00 PM	15.0	24.0	13.2
7/11/2016	1:40:00 PM	30.0	21.5	15.7
7/11/2016	2:10:00 PM	60.0	19.4	17.8
7/11/2016	3:10:00 PM	120.0	17.3	19.9
7/12/2016	8:50:00 AM	1180.0	14.8	22.4
7/12/2016	1:10:00 PM	1440.0	14.7	22.5



Sigma-1 CU B-3 30-38 ft.xlsm

USACE LABORATORY RESULTS

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

2970 S. Walton Walker, Suite 101 Dallas, TX 75211 (214) 331-4395 Fax (214) 331-4458
3101 Pleasant Valley, Suite 101 Arlington, TX 76015 (817) 467-5500 Fax (817) 468-9920

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

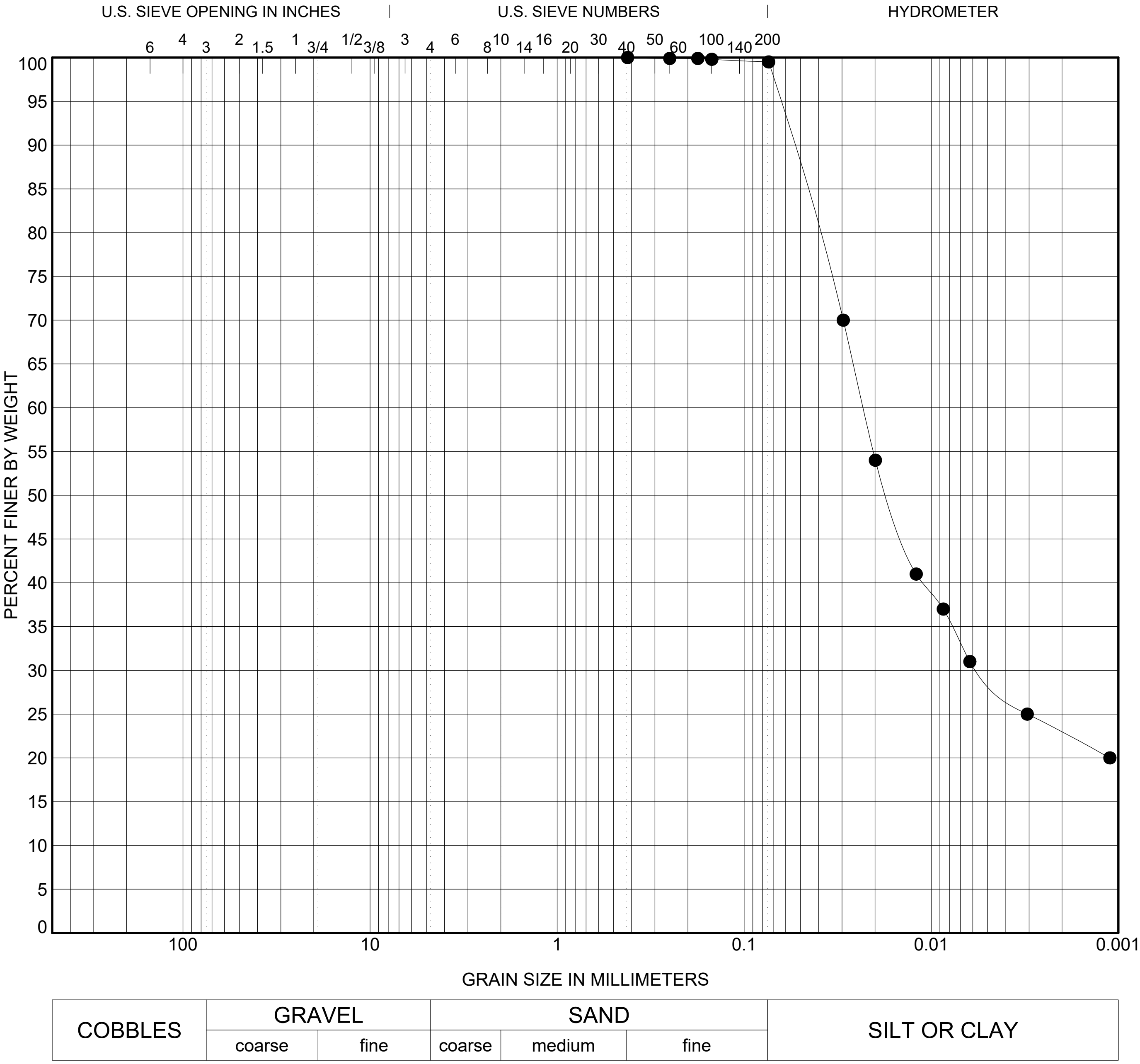
Boring No.	Sample No.	Sample Depth (ft.)	Visual Description & Unified Soil Classification (ASTM D-2487 & D-2488)		Moisture Content (%)	Unit Dry Weight (pcf)	Confining Pressure (tsf)	Q (tsf)	Strain @ Failure (%)	Type Failure
BRN-P3-33	--	2.0-4.0	Brown lean clay	CL	22.2	--	--	--	--	
	--	6.4-8.4	Brown lean 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	---	--	--	--	



						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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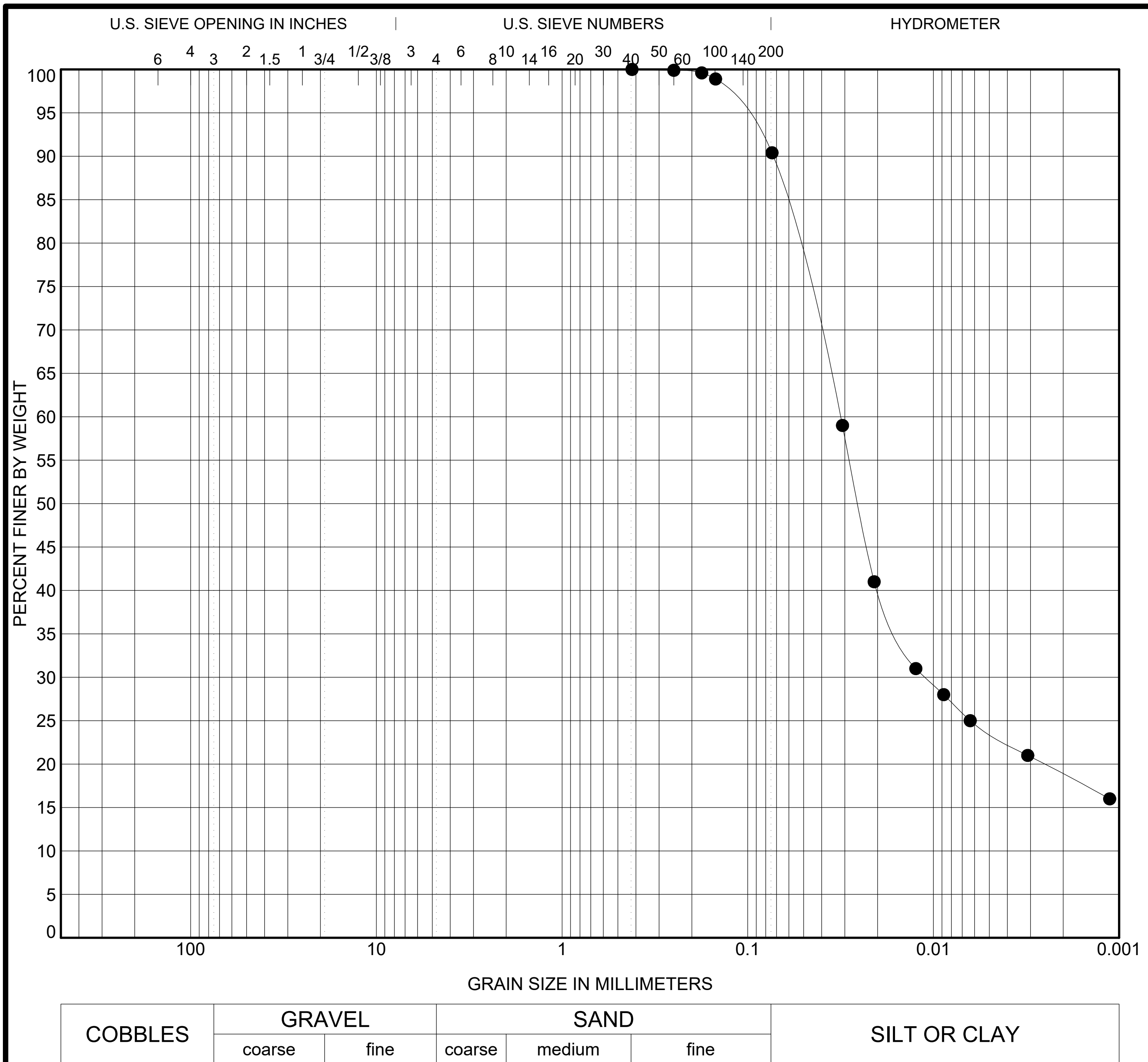
Arlington, Texas 76015
Telephone: (817) 467-5500
Fax: (817) 468-9920

GRAIN SIZE DISTRIBUTION

PROJECT: Brownsville Levee Subsurface Investigation

LOCATION: Brownsville, Texas

TEAM Job Number: 142086



						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

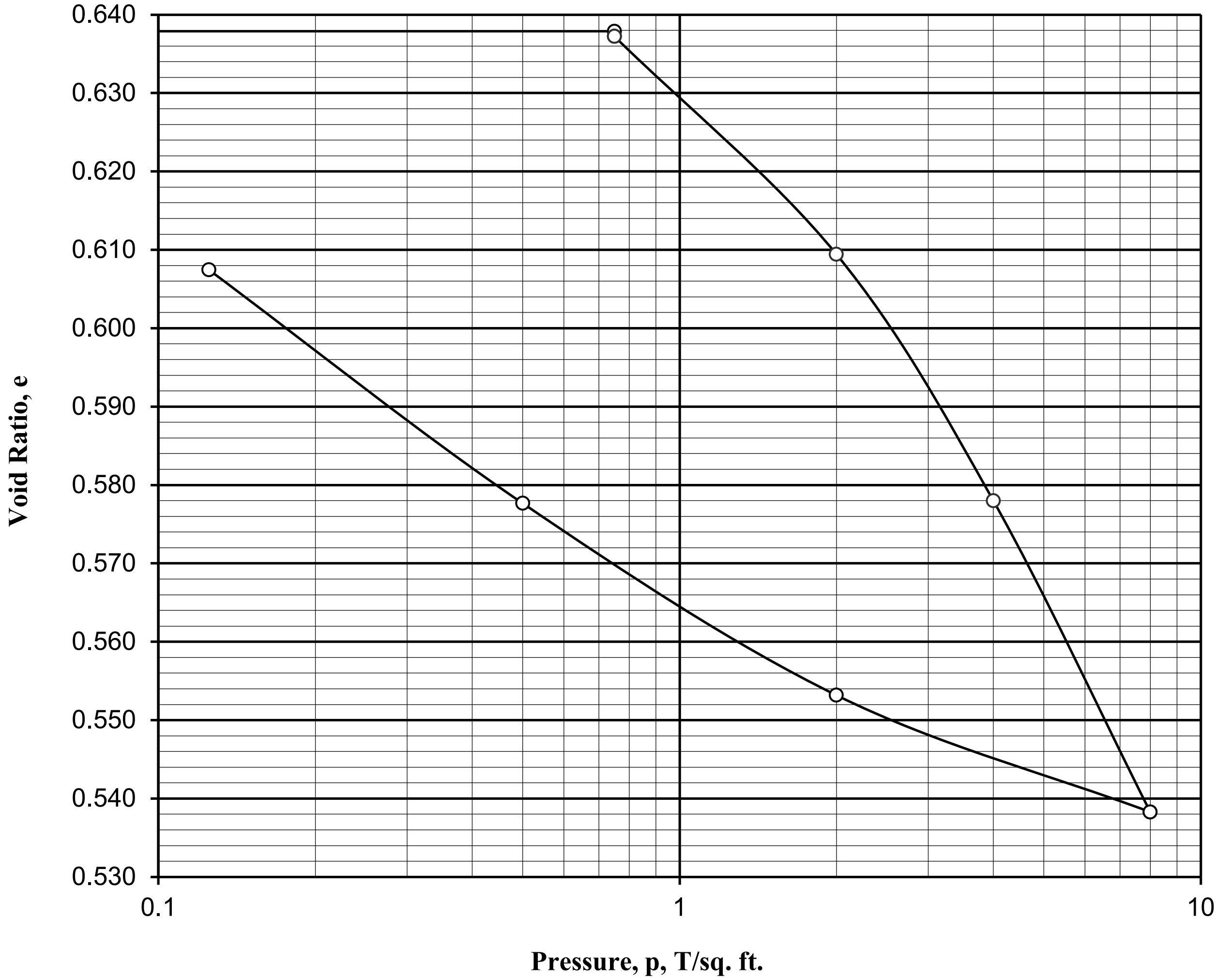
PROJECT: Brownsville Levee Subsurface Investigation

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					
Remarks			Team Project No.: 142086		
		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 $= \dfrac{W_S}{A \times G_s \times \gamma_W} = \dfrac{65.71}{31.67 \times 2.70 \times 1 \times 2.54} = 0.3022$ in.

Original height of water, $H_{WO} = \dfrac{W_{WO}}{A \times \gamma_W} = \dfrac{12.42}{31.67 \times 1 \times 2.54} = 0.1544$ in.

Final height of water, $H_{Wf} = \dfrac{W_{Wf}}{A \times \gamma_W} = \dfrac{14.58}{31.67 \times 1 \times 2.54} = 0.1812$ 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, $e_O = = \dfrac{H - H_S}{H_S} = \dfrac{0.495 - 0.3022}{0.3022} = 0.6379$

Void ratio after test, $e_f = \dfrac{H_f - H_S}{H_S} = \dfrac{0.4858 - 0.3022}{0.3022} = 0.6074$

Degree of saturation before test, $S_O = \dfrac{H_{WO}}{H - H_S} = \dfrac{0.1544}{0.4950 - 0.3022} = 80.1\%$

Degree of saturation after test, $S_f = \dfrac{H_{wf}}{H_f - H_S} = \dfrac{0.1812}{0.4858 - 0.3022} = 98.7\%$

Dry density before test, $\gamma_d = \dfrac{W_S}{H \times A} = \dfrac{65.71 \times 62.4}{0.495 \times 31.67 \times 2.54} = 103.0$ 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. °C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. °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:

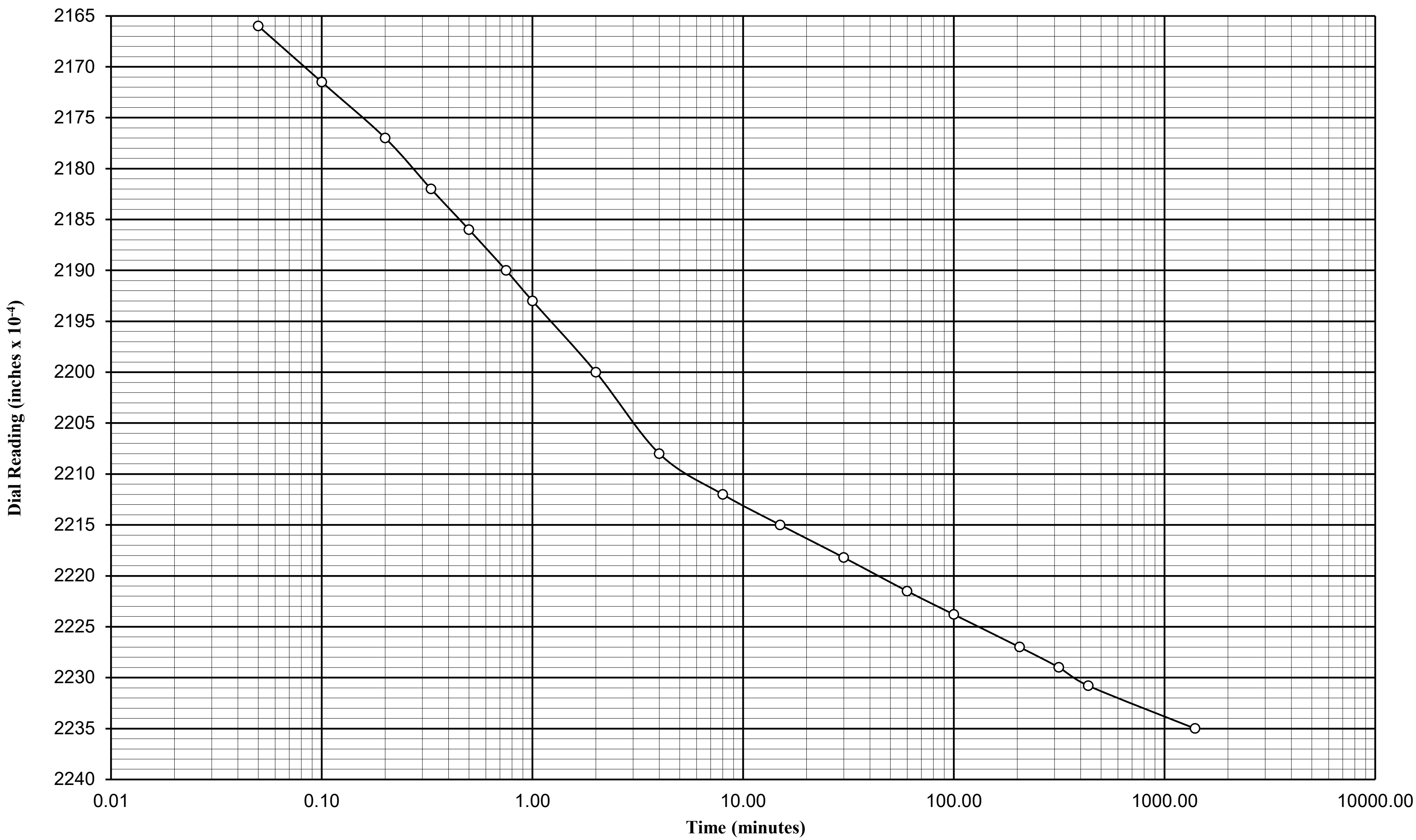
Height of voids, $H_V = (H - H_S) - \Delta H$ $H_S =$ 0.3022

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

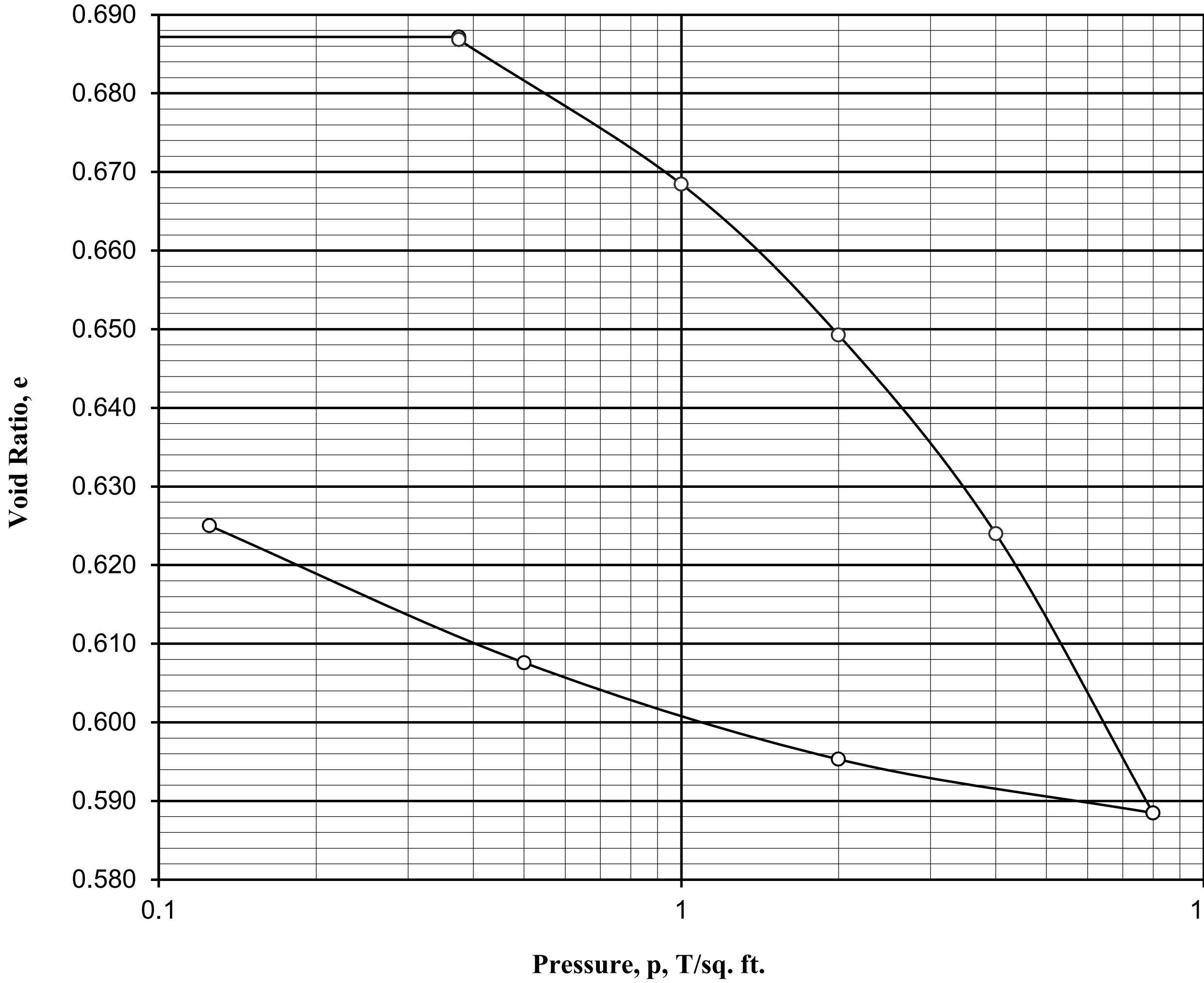
PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	25.6×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.21855	TEAM Project No.:	142086
BORING NO.:	BRN-P3-32b	t_{50} (min):	0.48	Date:	12/19/2014
DEPTH:	6.9-8.9	Load (tsf):	4	Remarks	
SAMPLE:	---	Thickness (inches)	0.495		

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		

TEAM Consultants, Inc.

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
PL	20		
Remarks			

Project: Brownsville Levee Repair

Team Project No.: 142086

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. ^{o}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{o}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. ^oC	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^oC
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

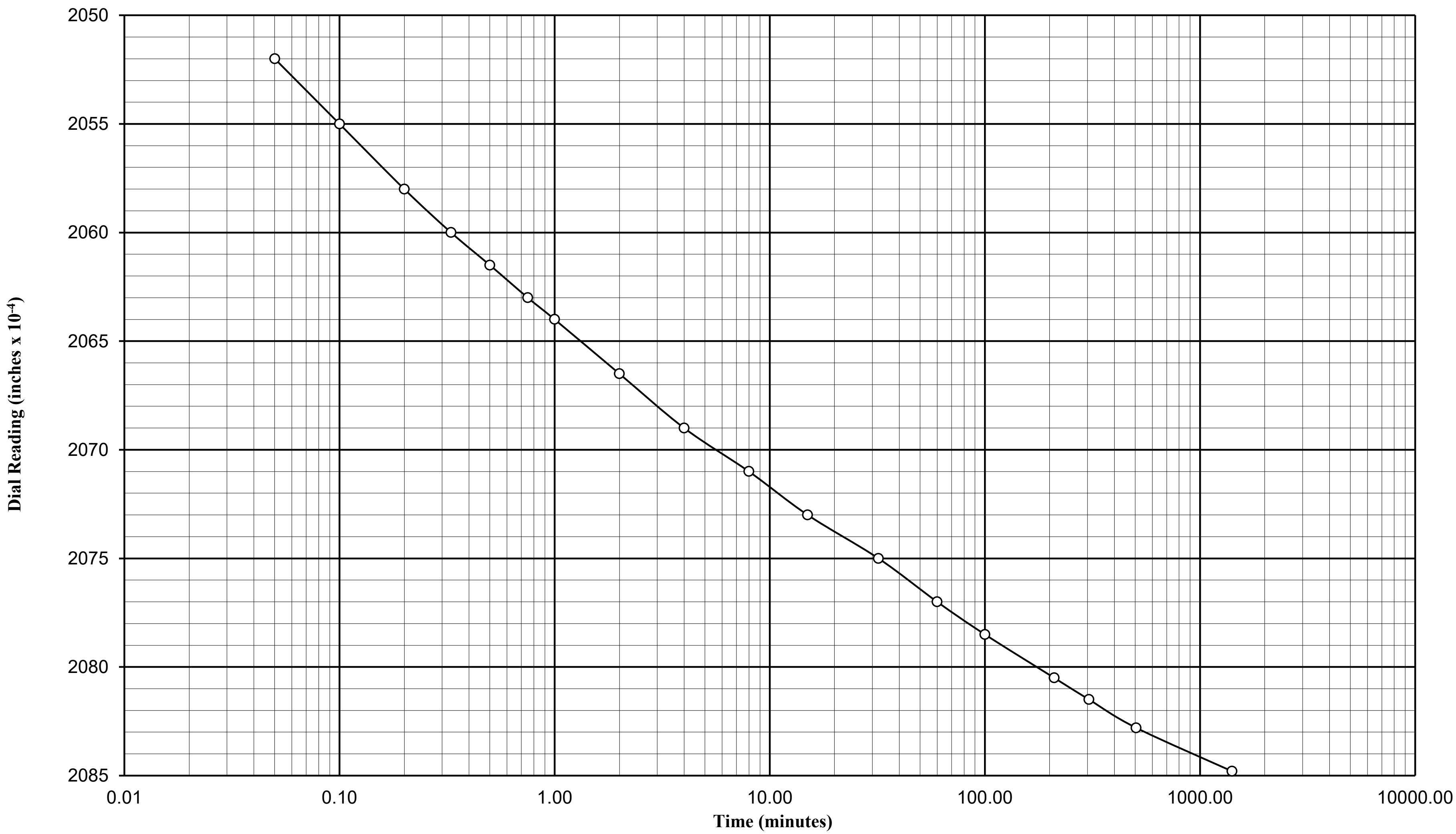
Pressure, P T./sq.ft.	Date Increment Applied	Time in Min. Increment Effective	Dial Reading 10^{-4} in.	Correction 10^{-4} in.	Change in Height, ΔH 10^{-4} in.	Height of Voids, H_V 10^{-4} in.	Void Ratio, e
0.1	12/19	Zero Point	2000	2000	0	2012	0.6872
0.4	12/19	Initial Load	2012	2012	0	2012	0.6872
0.4	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

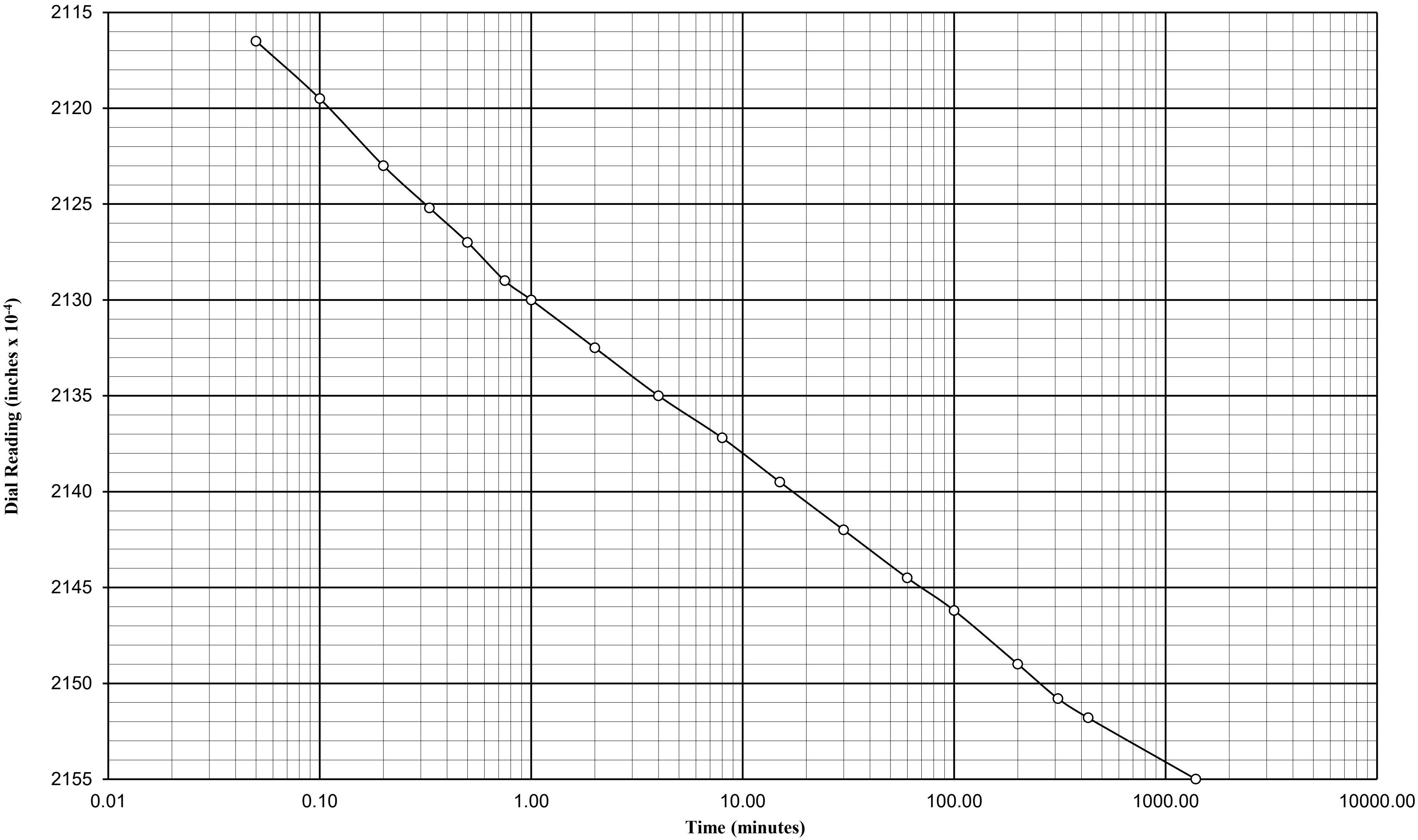
Void Ratio, $e = \frac{H_V}{H_S}$

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		

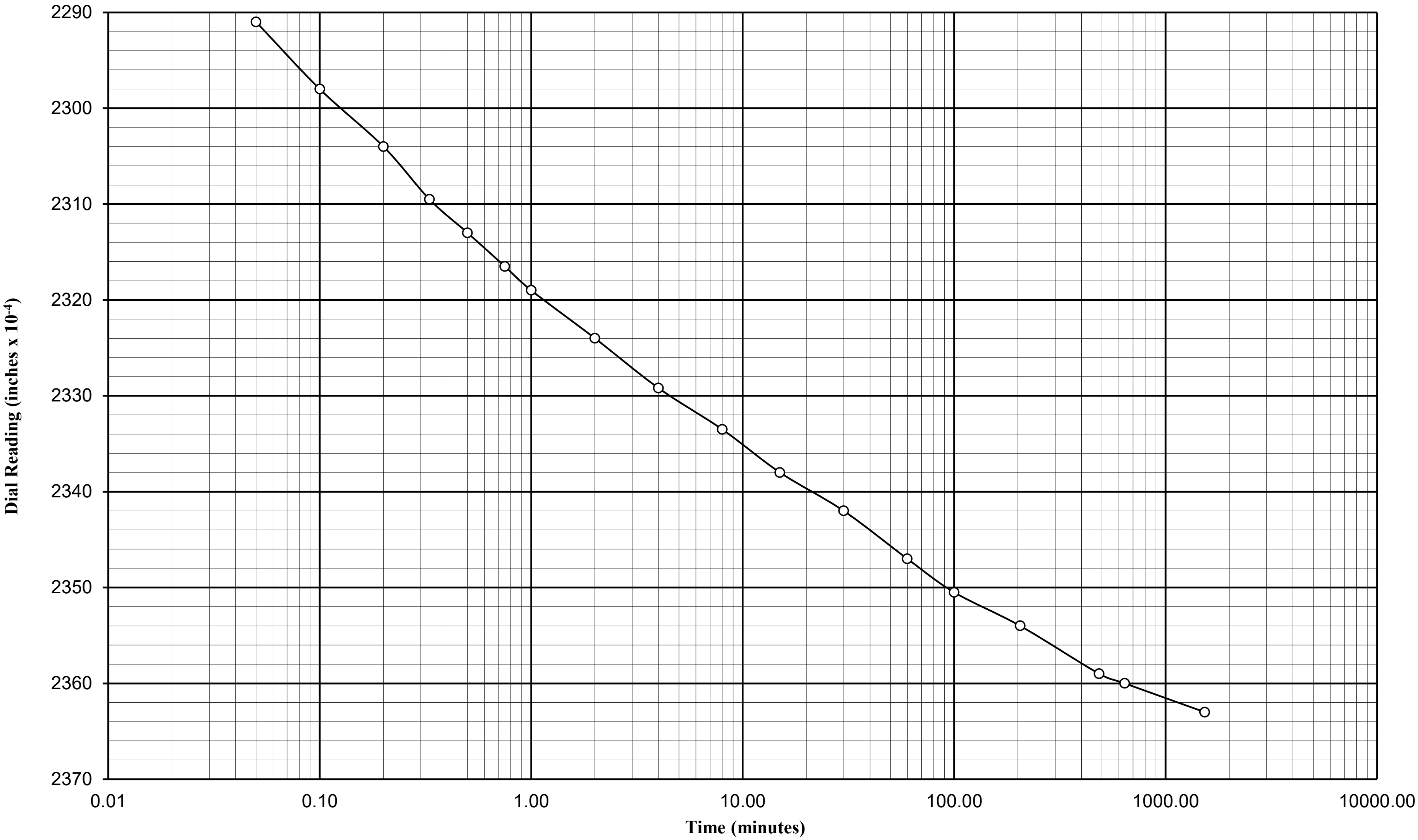


CONSOLIDATION TEST - DIAL READING TIME CURVE

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		

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		

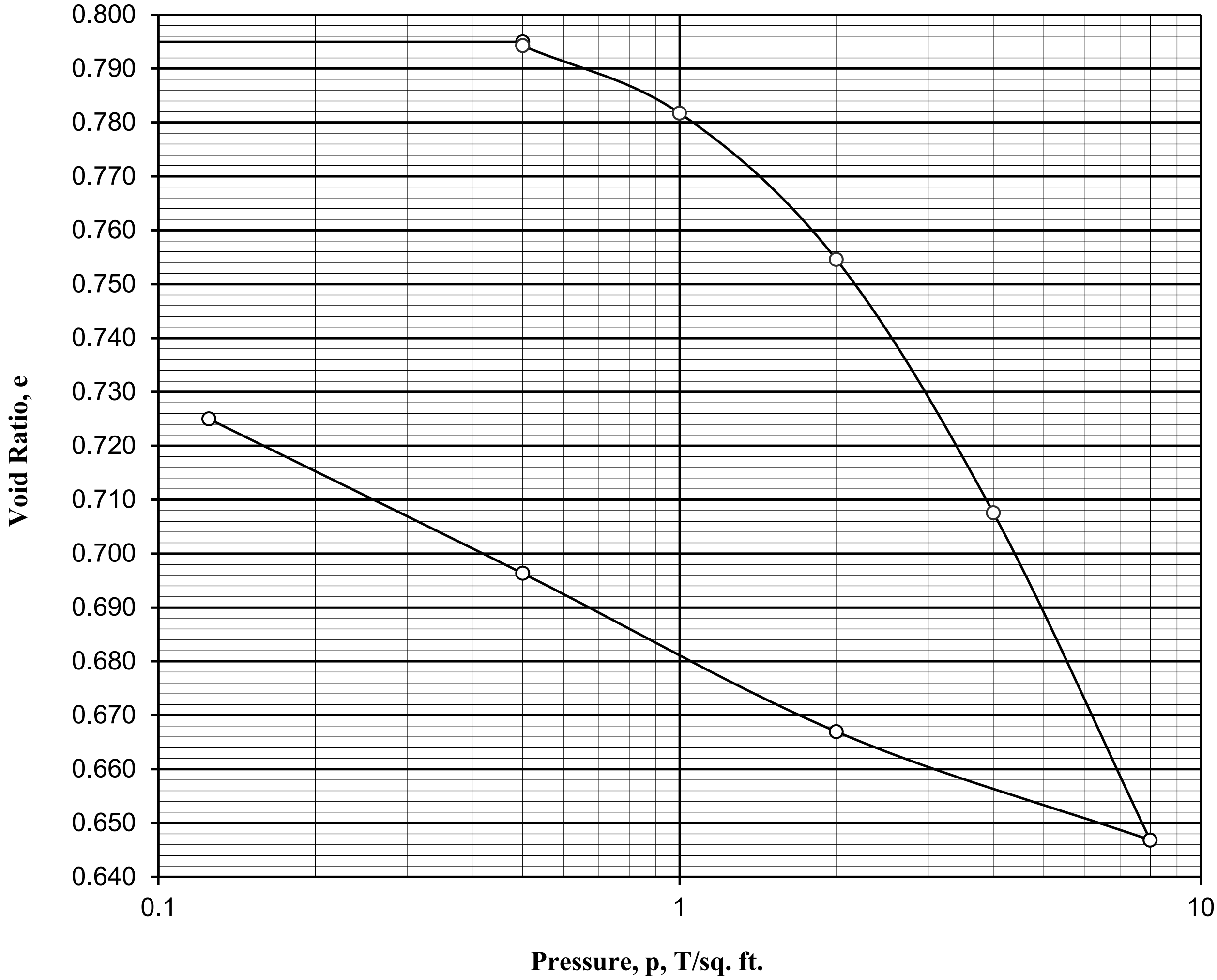


CONSOLIDATION TEST - DIAL READING TIME CURVE

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		

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	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				
Remarks				Team Project No.:	142086
				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. ^oC	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^oC
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

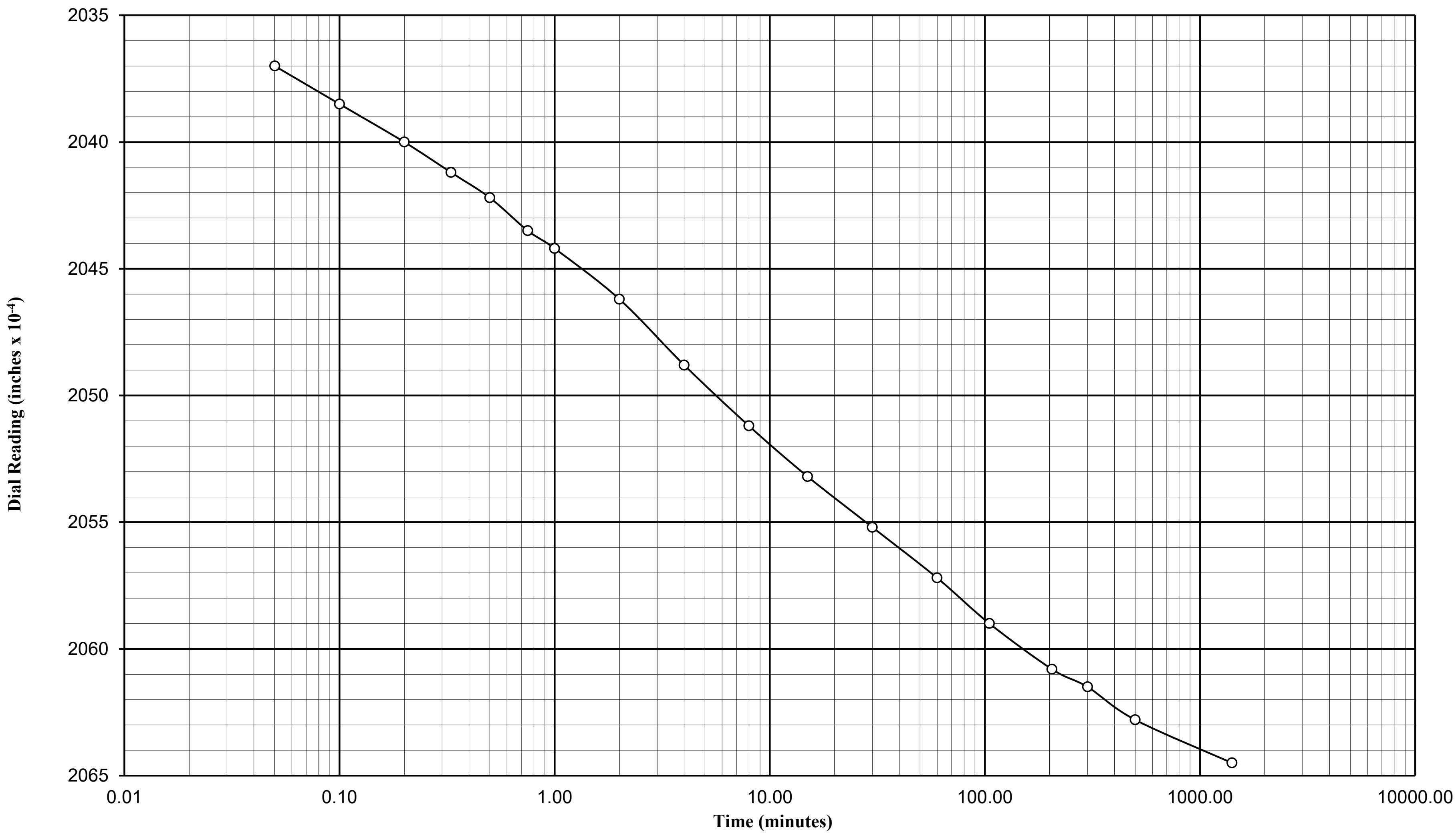
Pressure, P T./sq.ft.	Date Increment Applied	Time in Min. Increment Effective	Dial Reading 10^{-4} in.	Correction 10^{-4} in.	Change in Height, ΔH 10^{-4} in.	Height of Voids, H_V 10^{-4} in.	Void Ratio, e
0.1	12/19	Zero Point	2000	2000	0	2192	0.7950
0.5	12/19	Initial Load	2019	2019	0	2192	0.7950
0.5	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

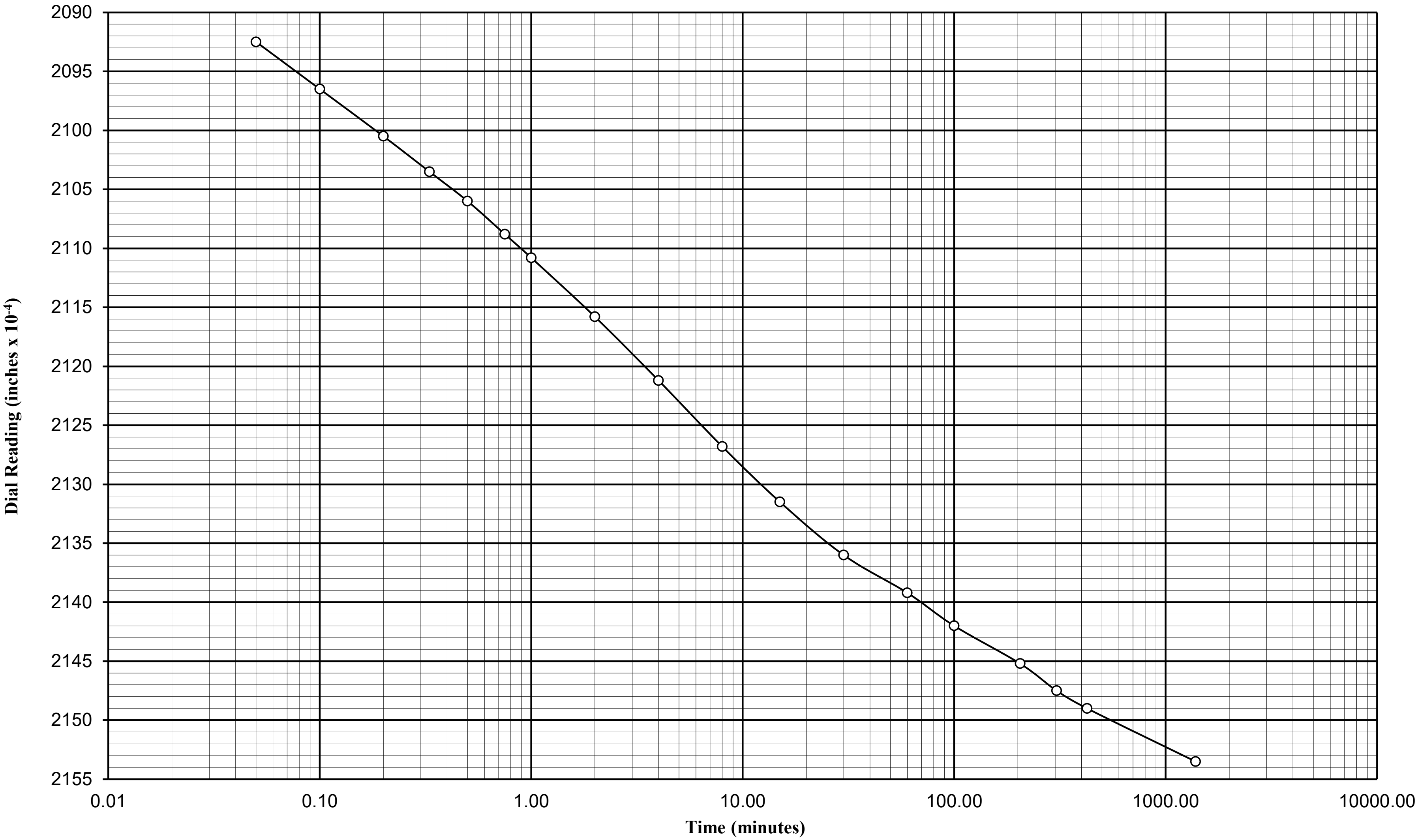
Void Ratio, $e = \dfrac{H_V}{H_S}$

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		

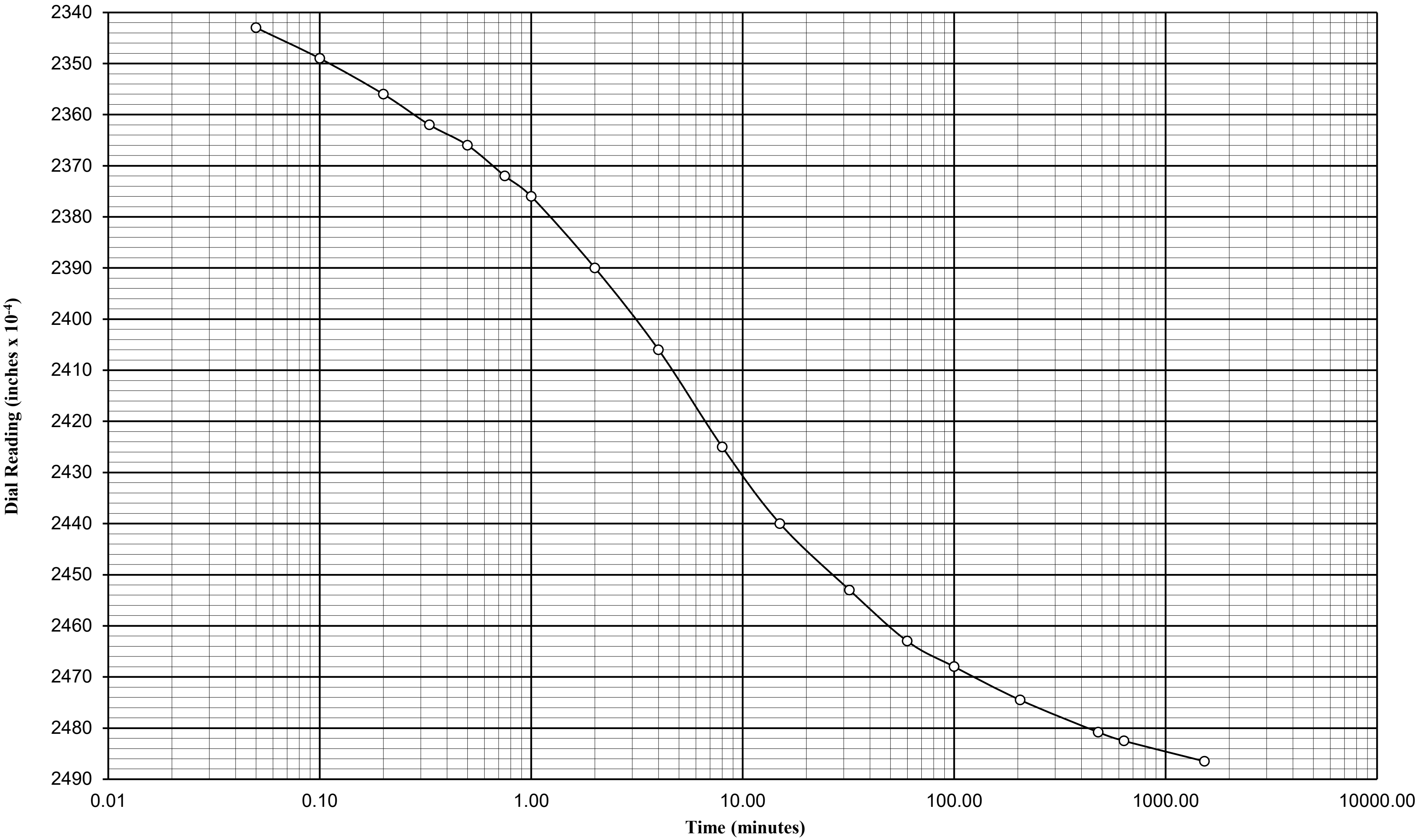


CONSOLIDATION TEST - DIAL READING TIME CURVE

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		



CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	3.78 x 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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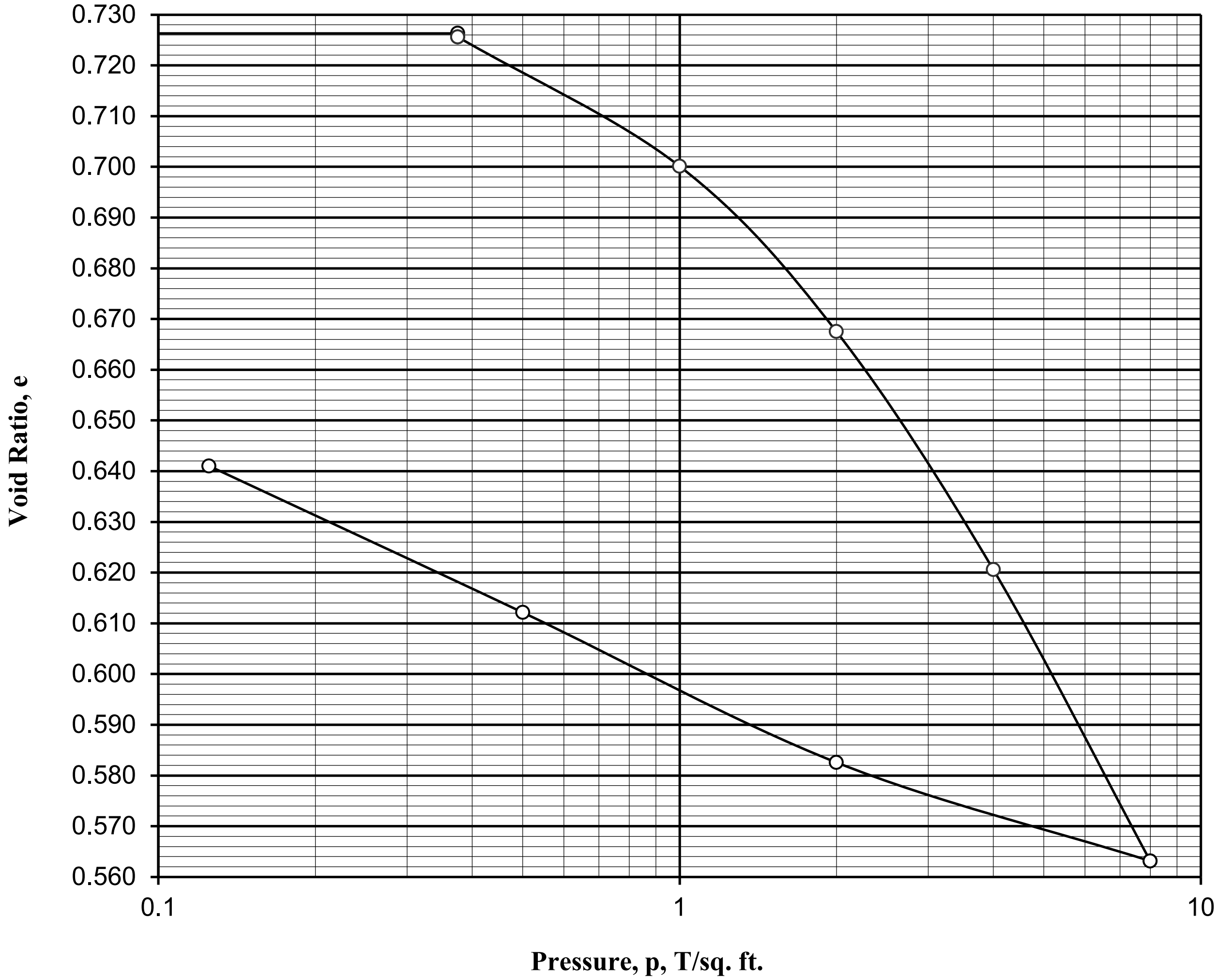
Geotechnical, Environmental, Construction Materials Testing



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, $H_s = \dfrac{W_S}{A \times G_s \times \gamma_W} = \dfrac{63.60}{31.67 \times 2.72 \times 1 \times 2.54} = 0.2908$ in.

Original height of water, $H_{WO} = \dfrac{W_{WO}}{A \times \gamma_W} = \dfrac{14.10}{31.67 \times 1 \times 2.54} = 0.1752$ in.

Final height of water, $H_{Wf} = \dfrac{W_{Wf}}{A \times \gamma_W} = \dfrac{14.90}{31.67 \times 1 \times 2.54} = 0.1852$ 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, $e_O = = \dfrac{H - H_S}{H_S} = \dfrac{0.502 - 0.2908}{0.2908} = 0.7263$

Void ratio after test, $e_f = \dfrac{H_f - H_S}{H_S} = \dfrac{0.4772 - 0.2908}{0.2908} = 0.6410$

Degree of saturation before test, $S_O = \dfrac{H_{WO}}{H - H_S} = \dfrac{0.1752}{0.5020 - 0.2908} = 83.0\%$

Degree of saturation after test, $S_f = \dfrac{H_{wf}}{H_f - H_S} = \dfrac{0.1852}{0.4772 - 0.2908} = 99.4\%$

Dry density before test, $\gamma_d = \dfrac{W_S}{H \times A} = \dfrac{63.60 \times 62.4}{0.502 \times 31.67 \times 2.54} = 98.3$ 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. ^{O}C	Date	Press. (tsf)	Time	Elapsed Time, (min)	Dial Reading (10^{-4} in.)	Temp. ^{O}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

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
						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

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-P3-33 SAMPLE NO. --- DEPTH 2-4' CONSOLIDOMETER NO. 5

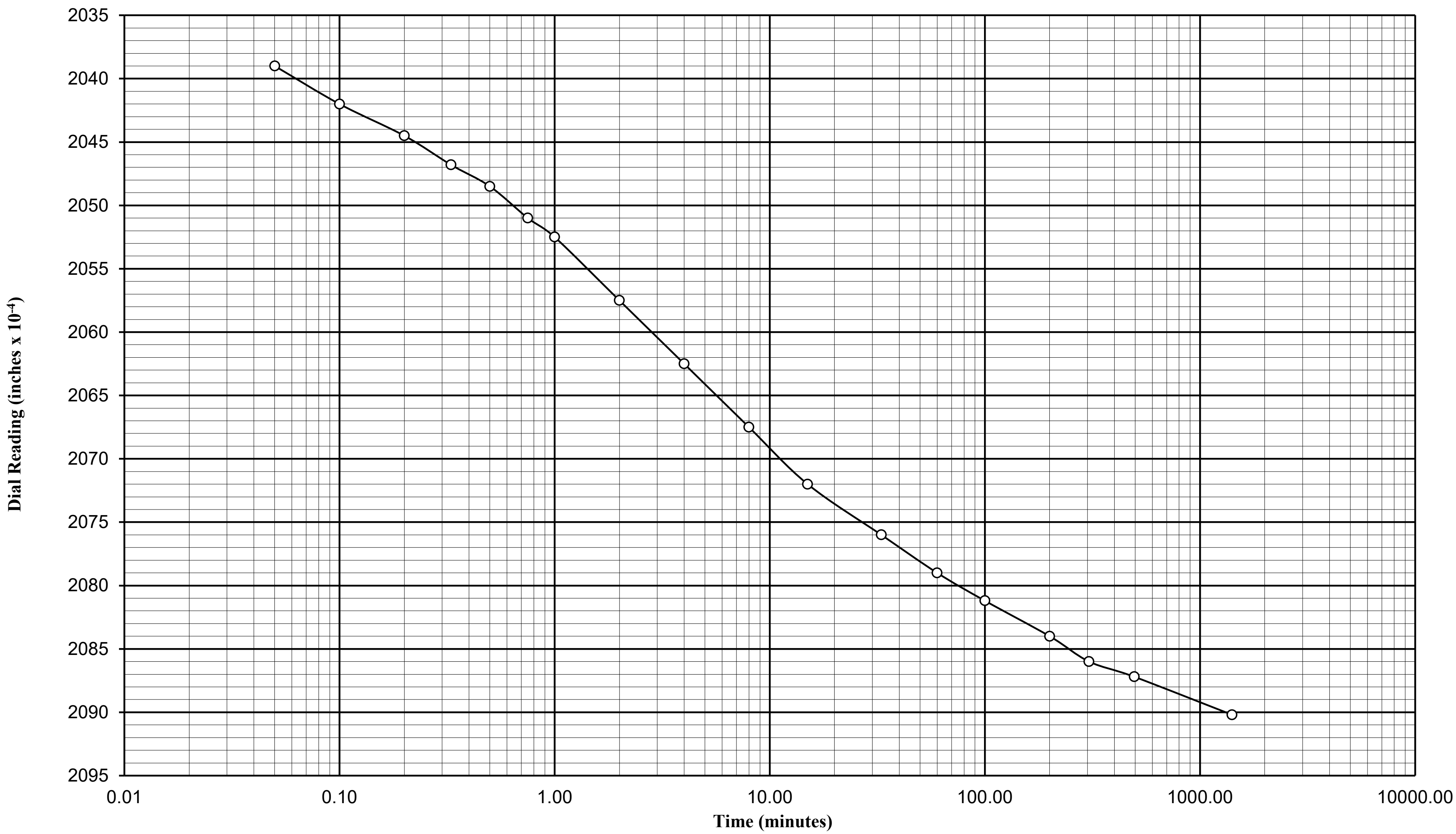
Pressure, P T./sq.ft.	Date Increment Applied	Time in Min. Increment Effective	Dial Reading 10^{-4} in.	Correction 10^{-4} in.	Change in Height, ΔH 10^{-4} in.	Height of Voids, H_V 10^{-4} in.	Void Ratio, e
0.1	12/19	Zero Point	2000	2000	0	2112	0.7263
0.375	12/19	Initial Load	2004	2004	0	2112	0.7263
0.375	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

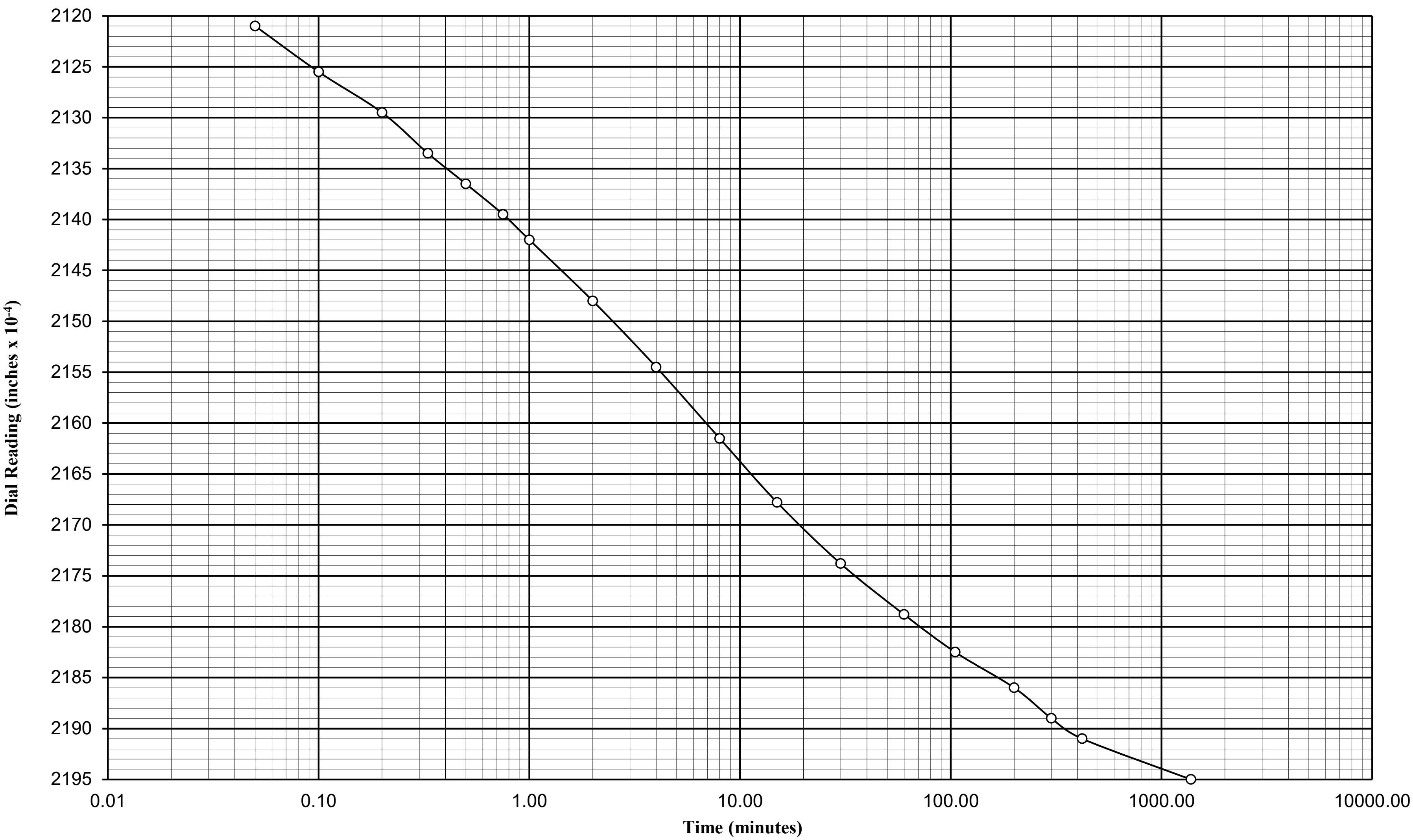
Void Ratio, $e = \frac{H_V}{H_S}$

Technician James Hutt Computed by James Hutt Checked by James Hutt



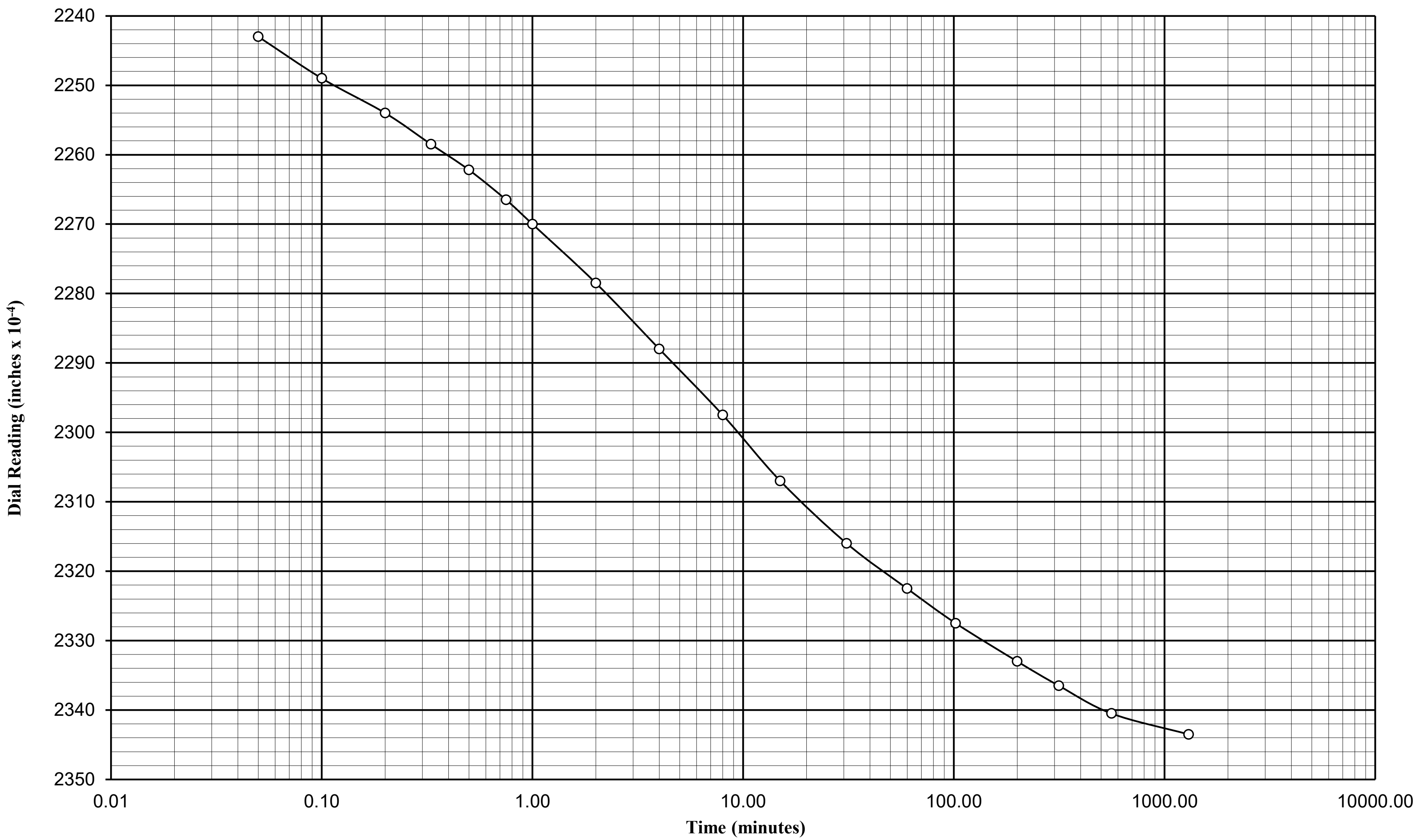
CONSOLIDATION TEST - DIAL READING TIME CURVE

PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	6.90×10^{-4} (cm^2/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		



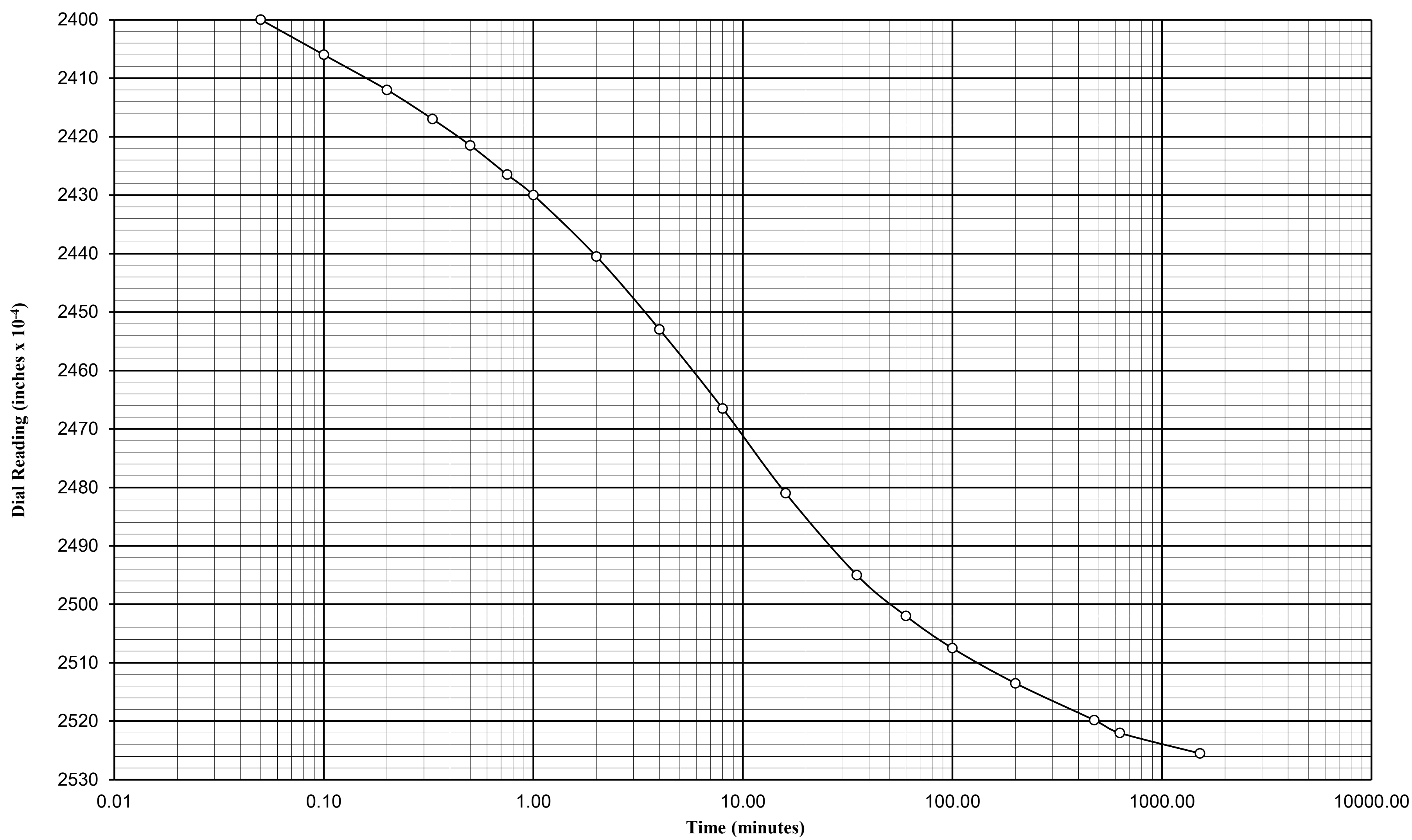
CONSOLIDATION TEST - DIAL READING TIME CURVE

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		



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		



CONSOLIDATION TEST - DIAL READING TIME CURVE

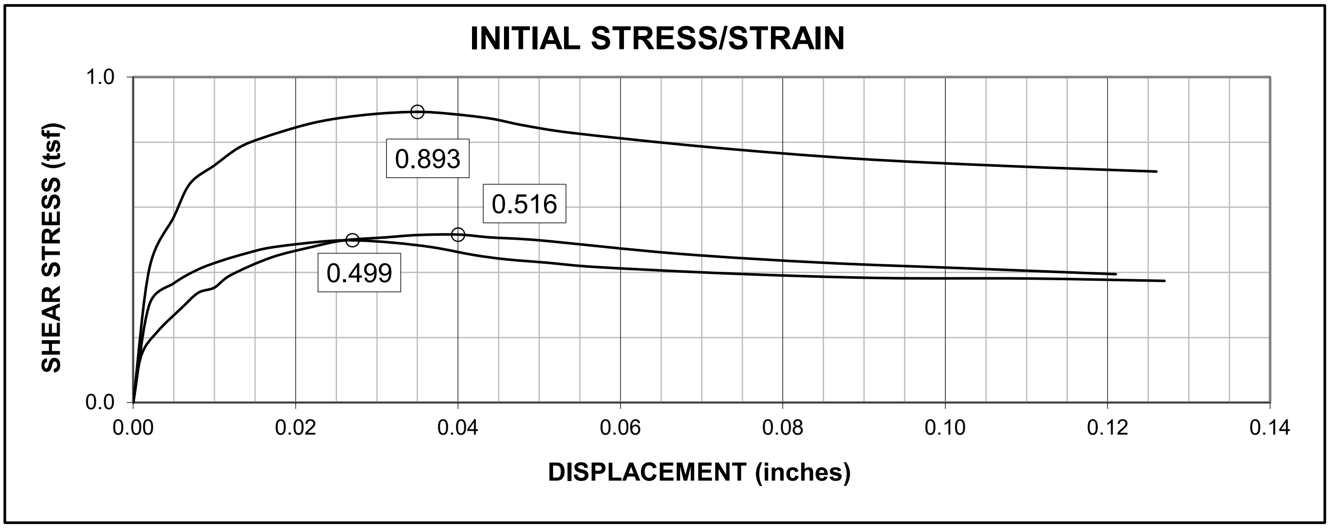
PROJECT:	Brownsville Levee Repair	Coefficient of Consolidation C_v	3.33×10^{-4} (cm^2/sec)		
		d_{50} (inches):	0.24500	TEAM Project No.:	142086
BORING NO.:	BRN-P3-33	t_{50} (min):	3.4	Date:	12/19/2014
DEPTH:	2-4'	Load (tsf):	8	Remarks	
SAMPLE:	---	Thickness (inches)	0.502		

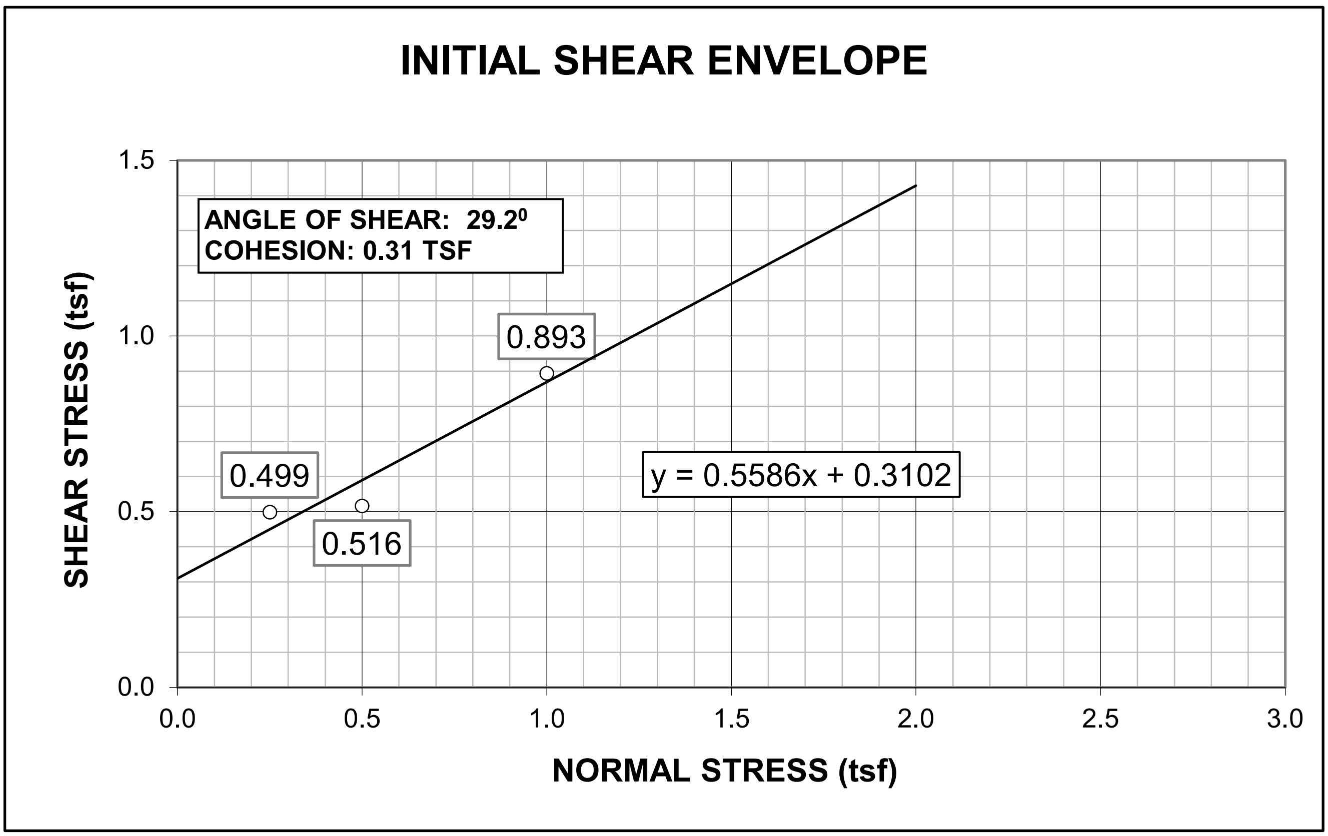
PROJECT: Brownsville Levee **DATE:** 12/30/2014 **JOB NO.:** 142086

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	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





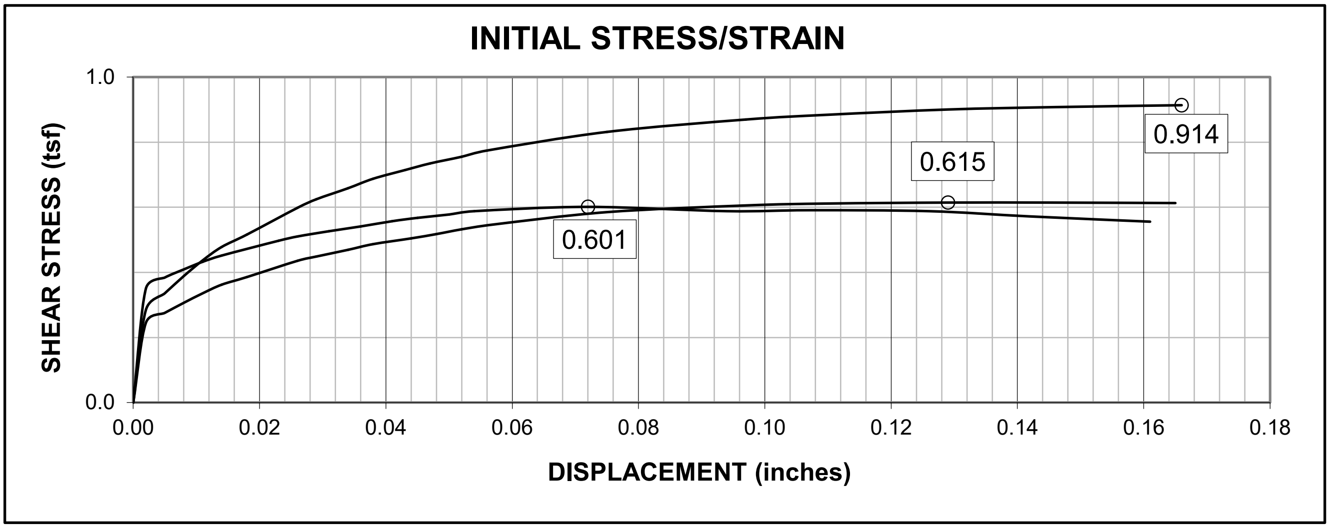
CONSOLIDATED DRAINED DIRECT SHEAR TEST

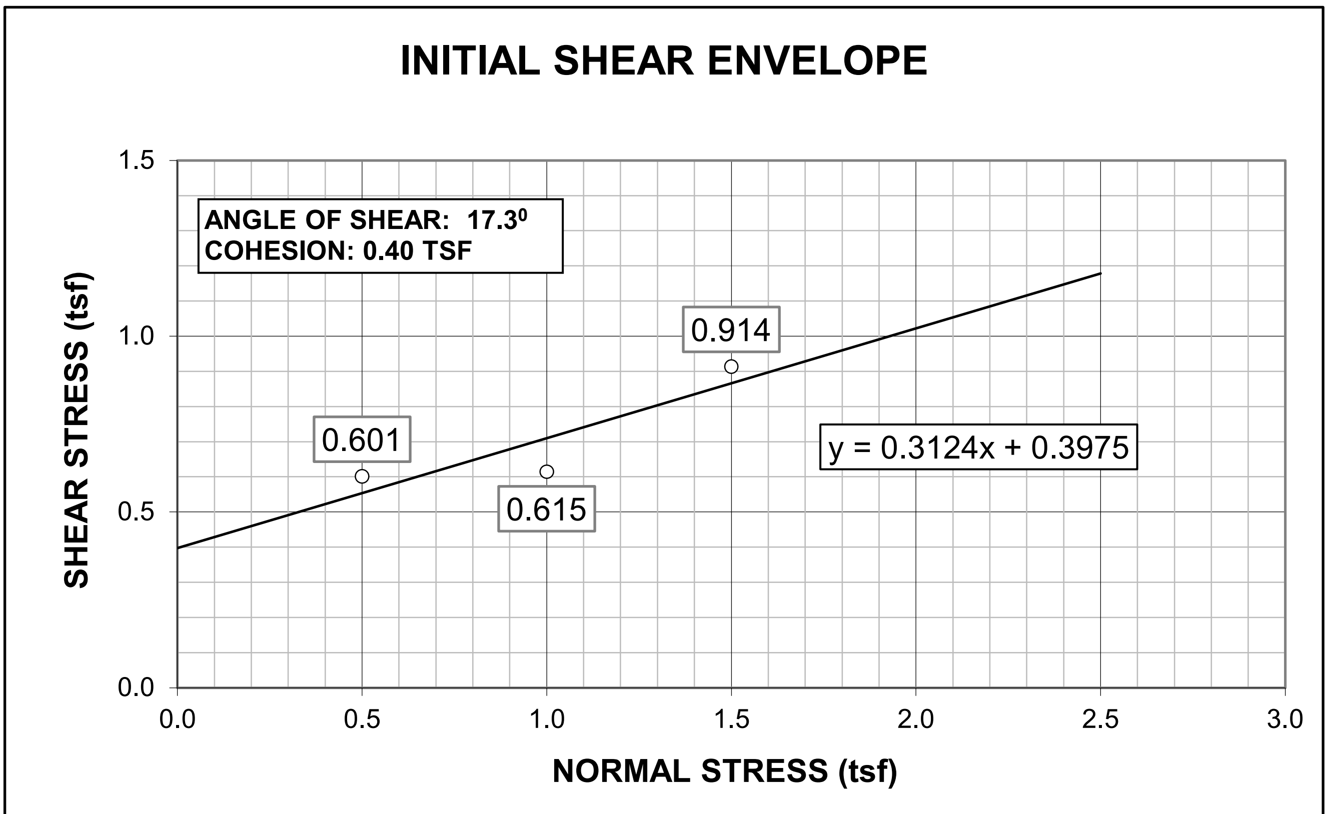
PROJECT: Brownsville Levee DATE: 1/8/2015 JOB NO.: 142086

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





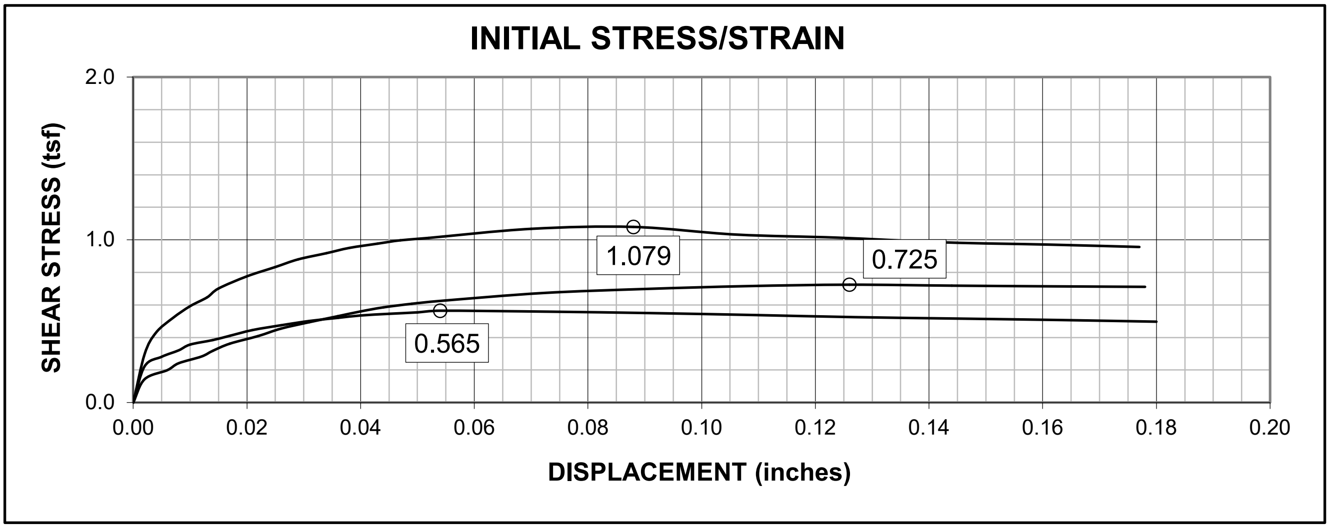
CONSOLIDATED DRAINED DIRECT SHEAR TEST

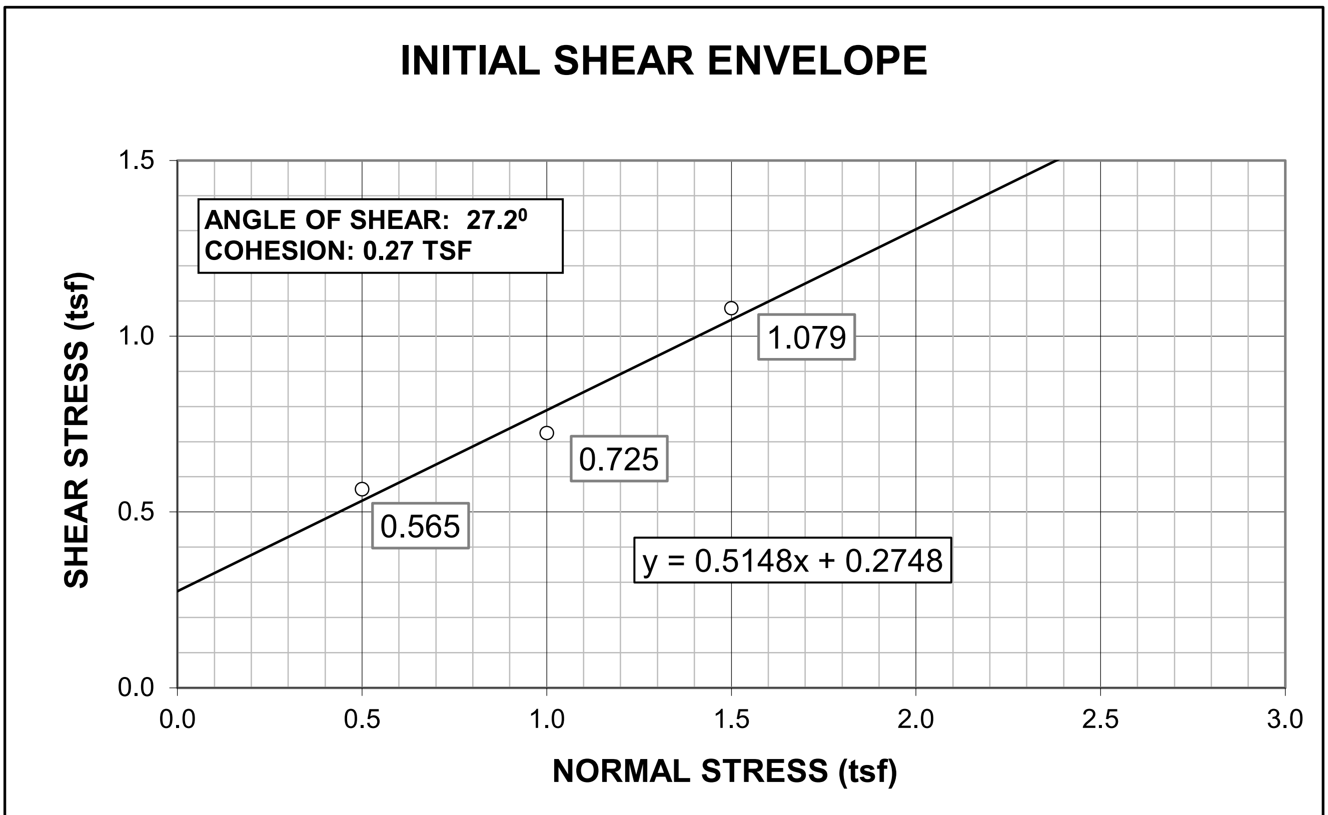
PROJECT: Brownsville Levee **DATE:** 1/5/2015 **JOB NO.:** 142086

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





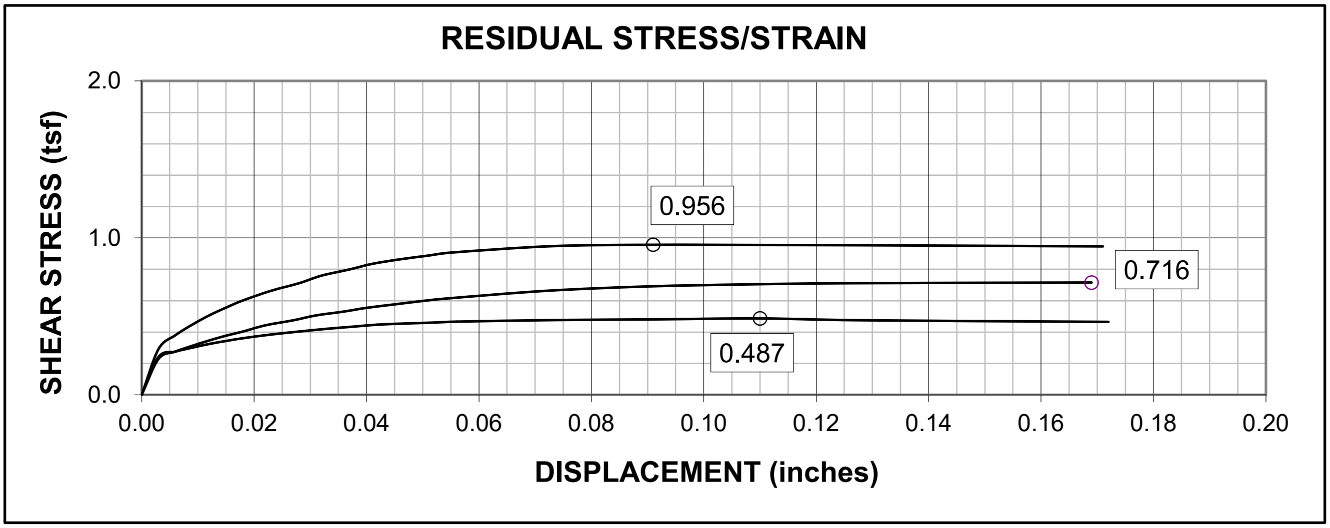
CONSOLIDATED DRAINED DIRECT SHEAR TEST

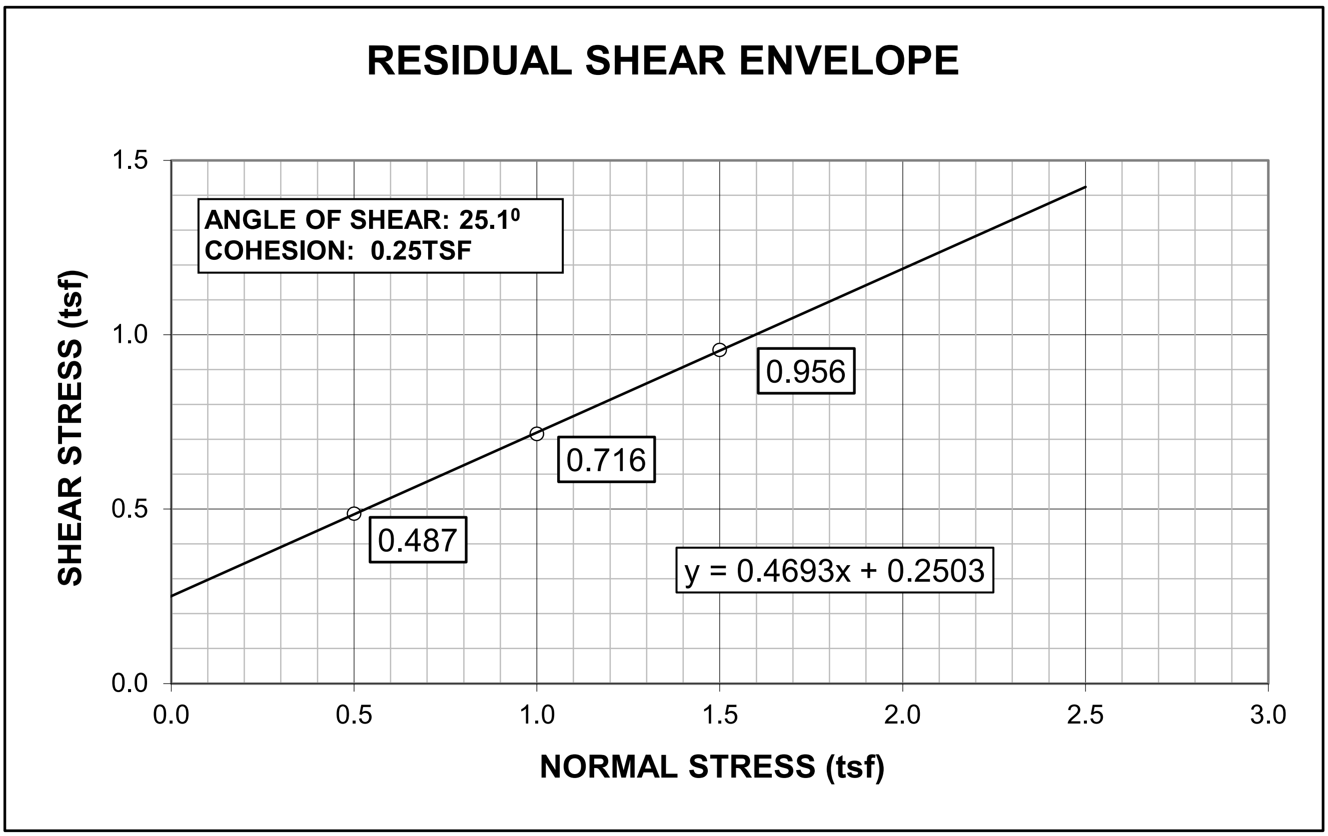
PROJECT: Brownsville Levee **DATE:** 1/5/2015 **JOB NO.:** 142086

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





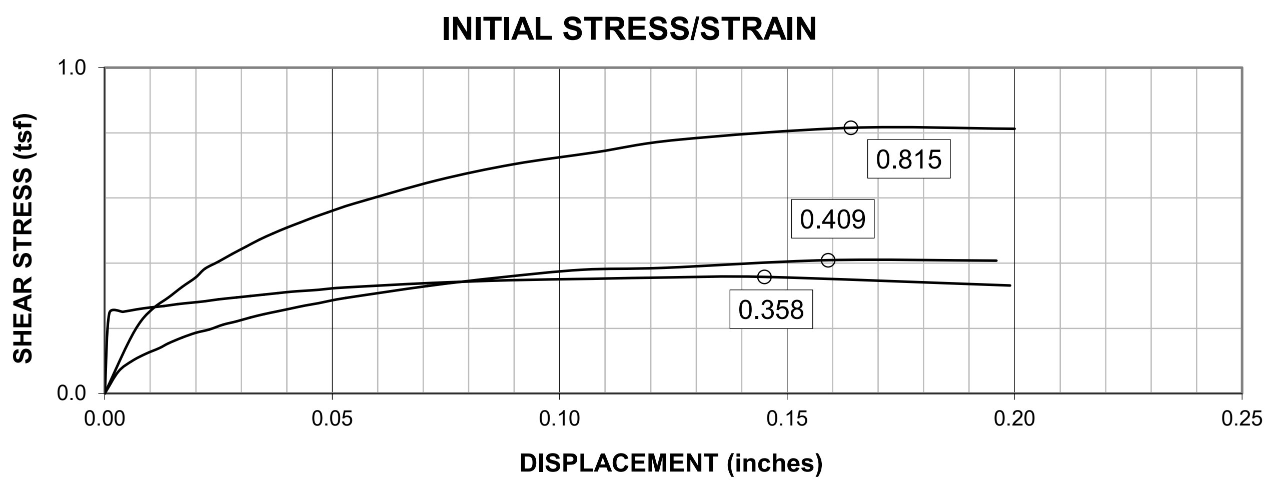
CONSOLIDATED DRAINED DIRECT SHEAR TEST

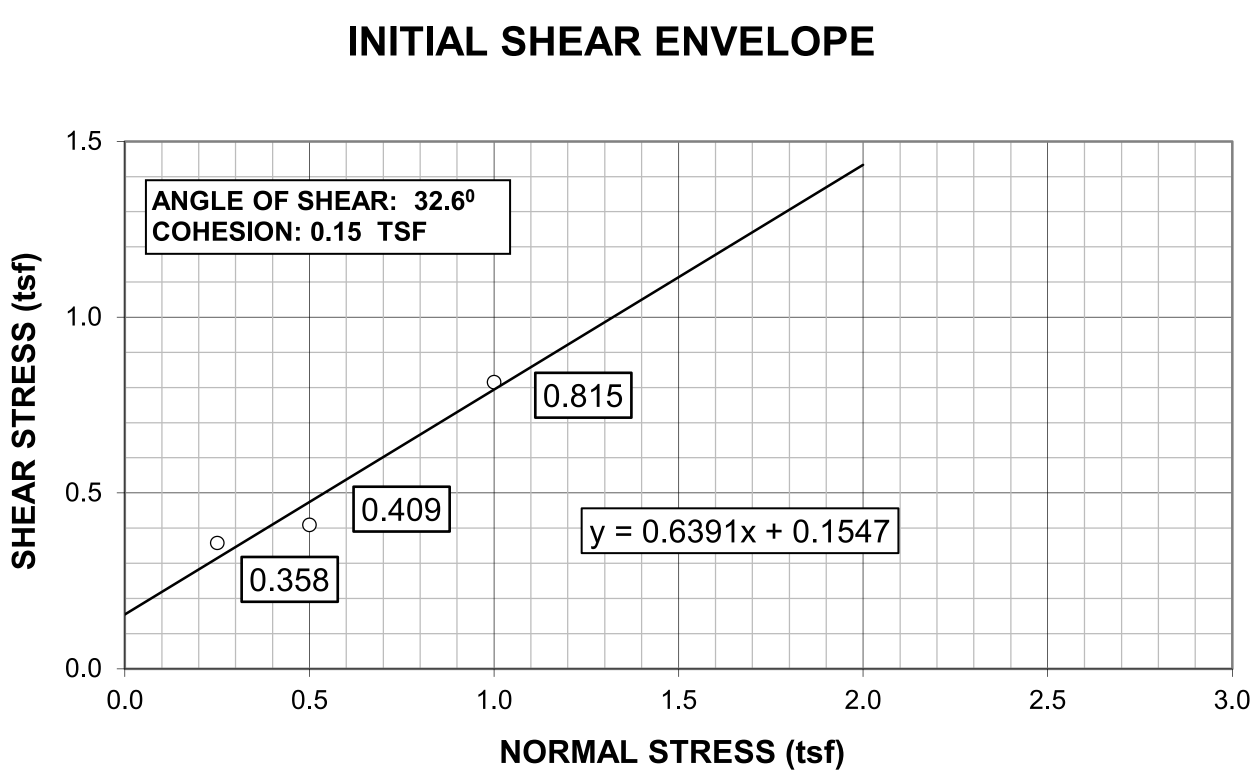
PROJECT: Brownsville Levee **DATE:** 12/19/2014 **JOB NO.:** 142086

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	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

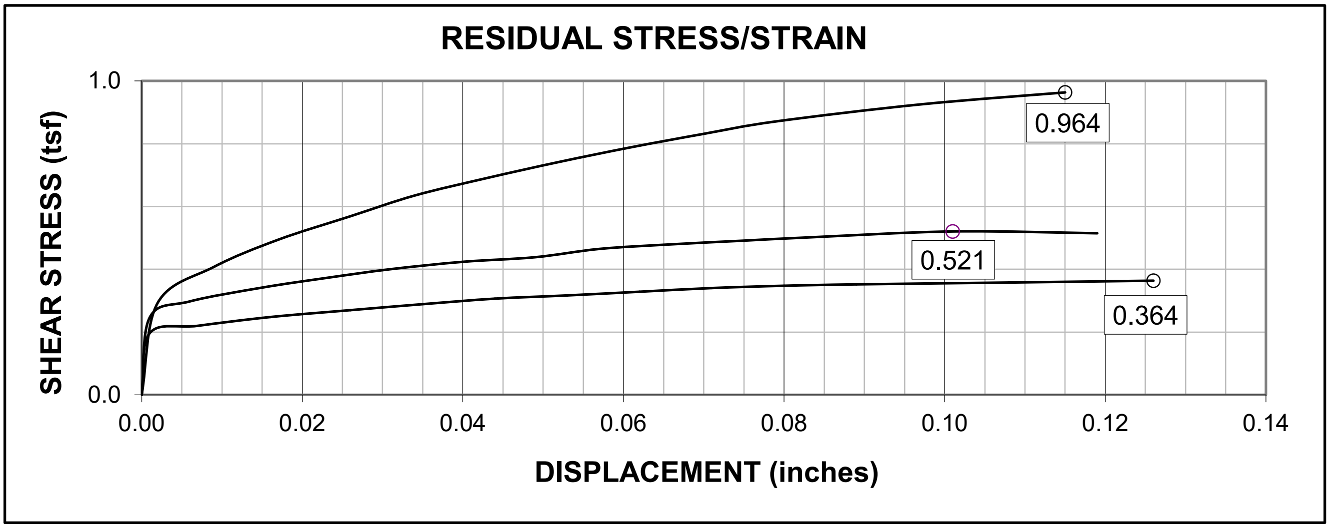


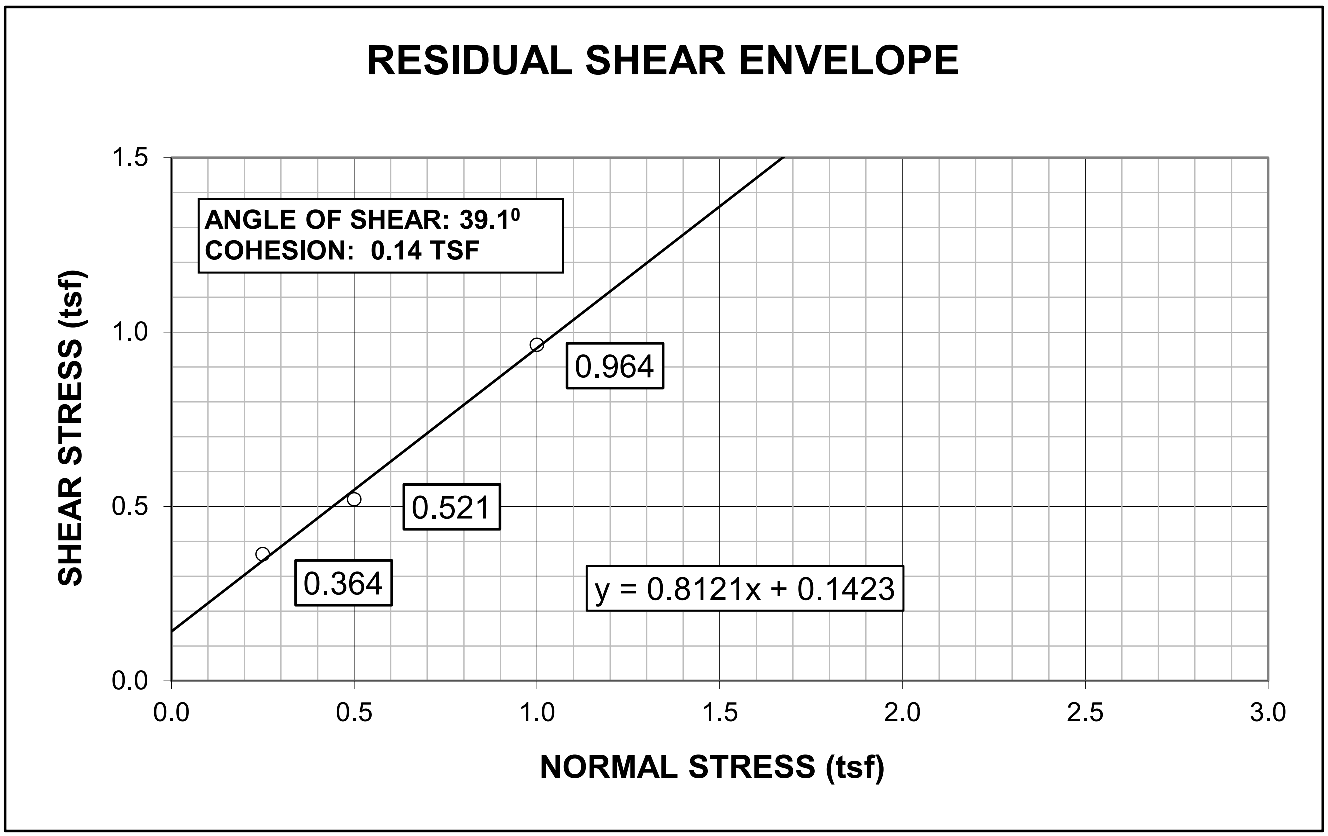


CONSOLIDATED DRAINED DIRECT SHEAR TEST

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**	**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		

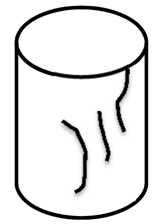
Height 1:	5.846 "	Dia.1:	2.855 "		
Height 2:	5.846 "	Dia.2:	2.867 "	Area:	6.447 In²
Height 3:	5.852 "	Dia.3:	2.873 "		
Young's Modulus for Membrane (tsf)	11.56				
Weight g:	1273.7	Strain Rate:	0.060	(Inches/Minute)	
Wet γ (pcf):	128.7	Strain Rate:	1.03	(%/Minute)	
Dry γ (pcf):	108.7	Length/Diameter 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

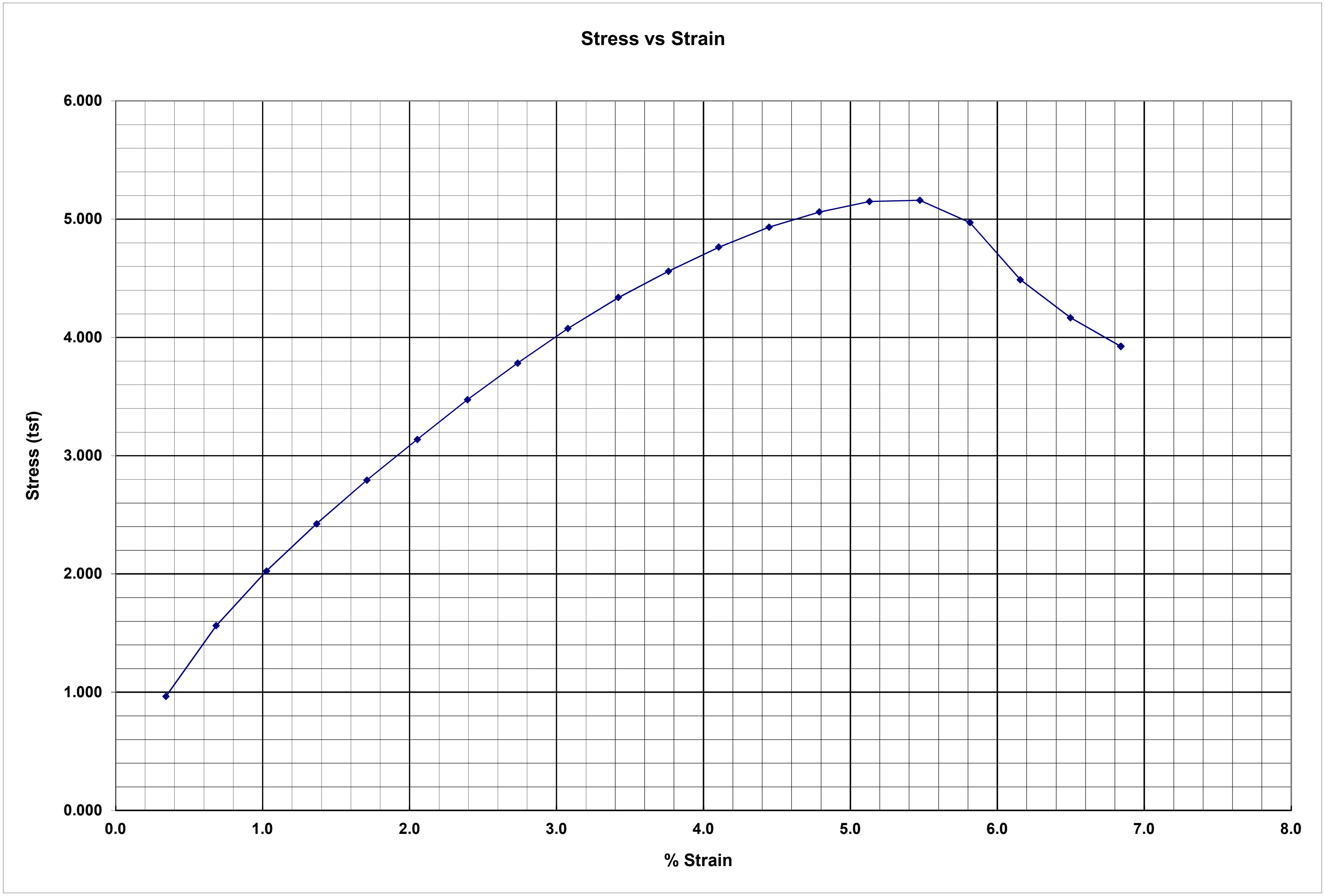
% Strain @ Maximum Strength = 5.47%

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 | **Hole :** P3-32b | **Sample :** | **Depth:** 13.5-15.5

TEAM Project No.: 142086 | **Date:** 1/6/15 | **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

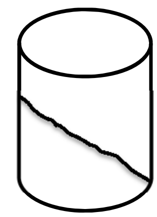
Proving Ring Constant: 1

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

Tested by: J. Young

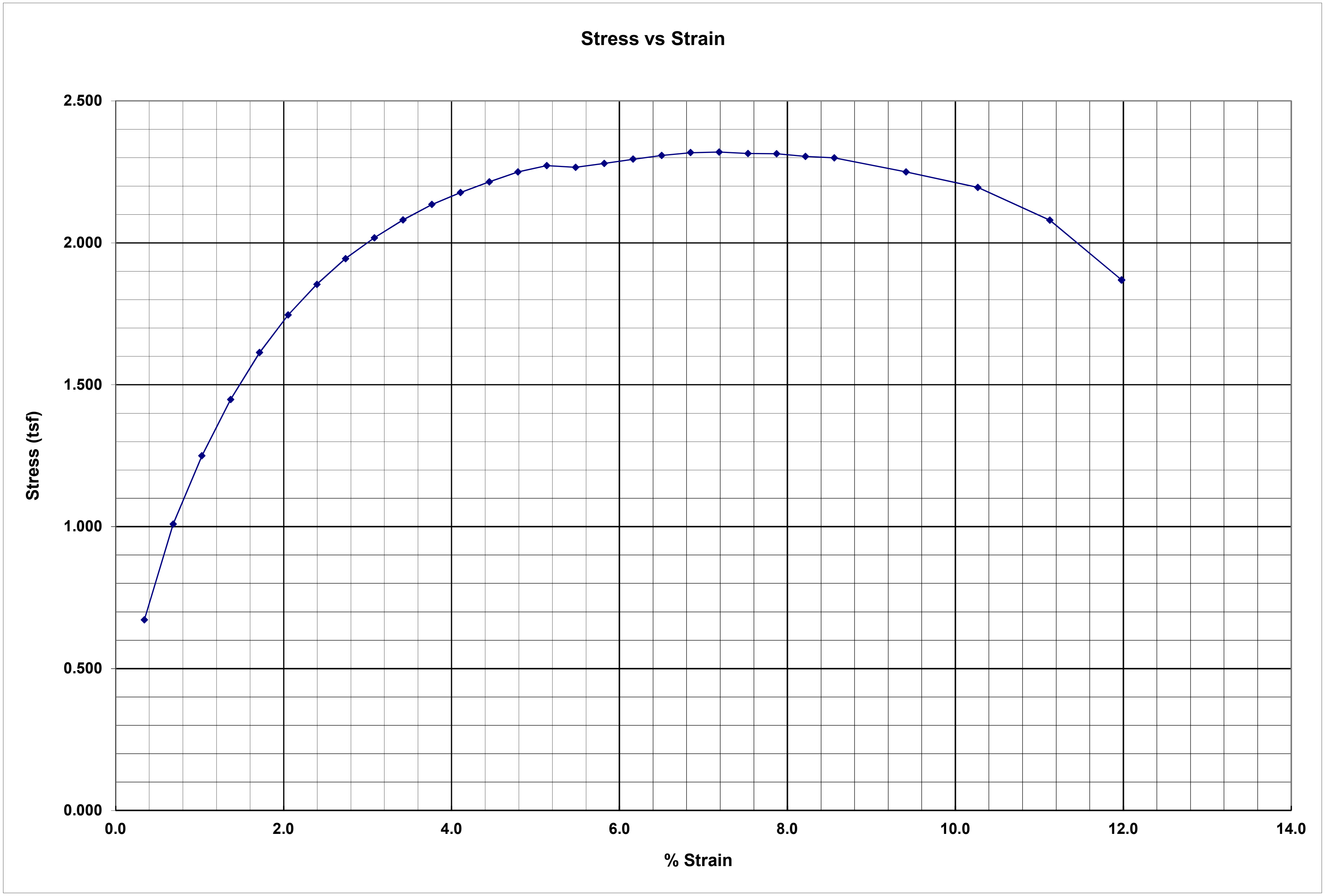
Maximum Compressive Strength (TSF)= 2.32

% 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.:**	P3-32b	**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 **Hole :** P3-32b **Sample :** **Depth:** 24.5-26.5

TEAM Project No.: 142086 **Date:** 1/6/15 **Material:** Dark brown lean clay

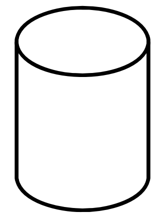
Height 1: 5.830 " **Dia.1:** 2.856 "
Height 2: 5.806 " **Dia.2:** 2.844 " **Area:** 6.402 in^2
Height 3: 5.846 " **Dia.3:** 2.865 "
Young's Modulus for Membrane (tsf) 11.56
Weight g: 1170.4 **Strain Rate:** 0.030 (Inches/Minute)
Wet γ (pcf): 119.5 **Strain Rate:** 0.51 (%/Minute)
Dry γ (pcf): 91.2 **Length/Diameter 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

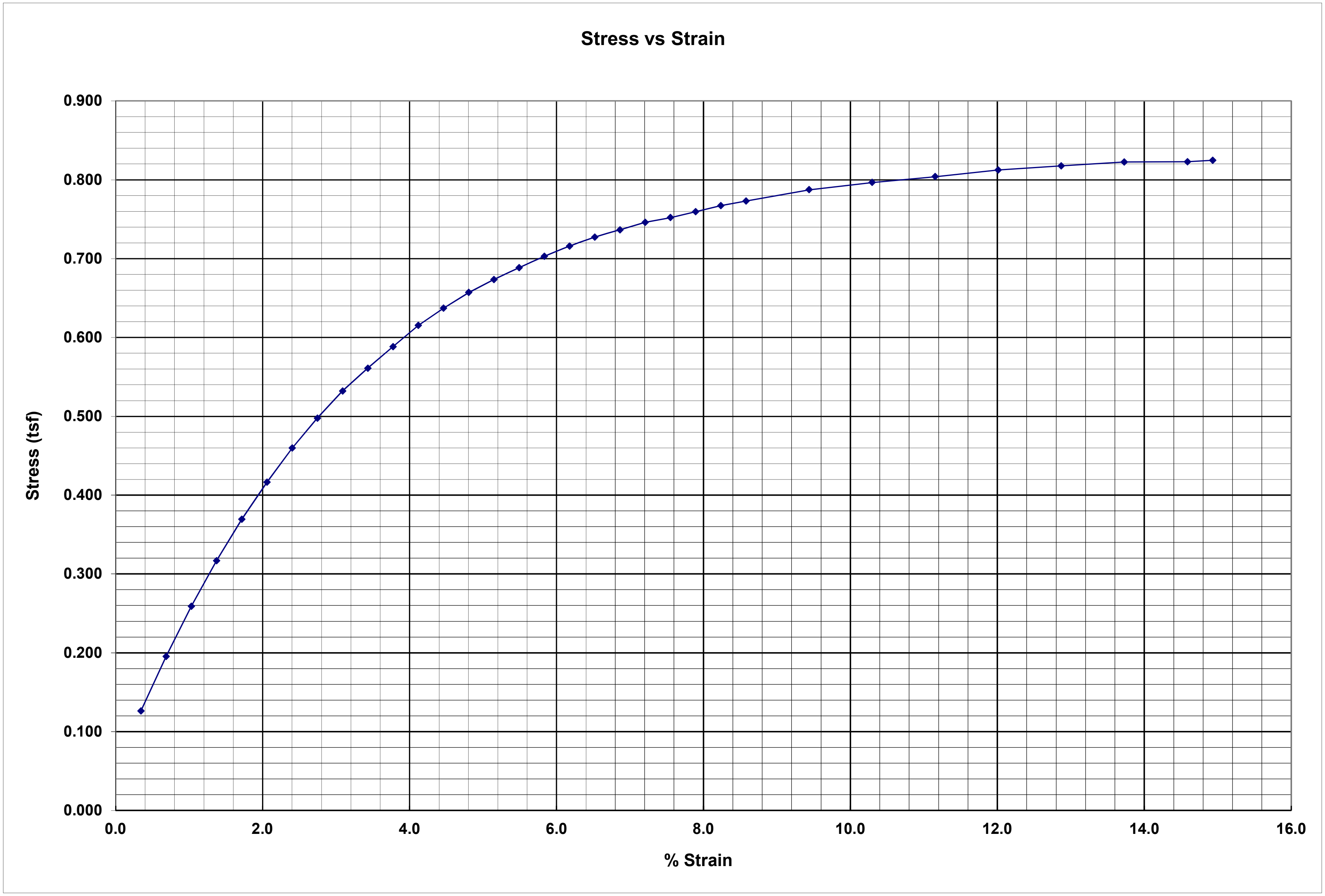
Confining Pressure (psi)	Dial Deflection	% Strain	Corrected Area (IN^2)	Load Dial Readings	Load Lbs	Deviator Stress (TSF) UNCORRECTED	Deviator Stress (TSF) CORRECTED*	Shearing Strength (cohesion)
22.1								
	0.020	0.343	6.424	11.3	11.3	0.127	0.126	0.063
	0.040	0.686	6.446	17.6	17.6	0.197	0.196	0.098
	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 **Tested by:** J. Young
Maximum Compressive Strength (TSF)= 0.82
% 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.:**	P3-32b	**Sample No.:**	0	**Depth:**	24.5-26.5
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 : P3-32b **Sample :** **Depth:** 26.7-28.7
Material: Dark brown lean clay

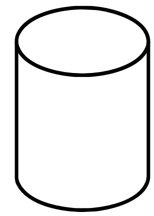
Height 1: 5.854 " **Dia.1:** 2.838 "
Height 2: 5.842 " **Dia.2:** 2.856 " **Area:** 6.421 In²
Height 3: 5.841 " **Dia.3:** 2.884 "
Young's Modulus for Membrane (tsf) 11.56
Weight g: 1211.7 **Strain Rate:** 0.060 (Inches/Minute)
Wet γ (pcf): 123.0 **Strain Rate:** 1.03 (%/Minute)
Dry γ (pcf): 97.4 **Length/Diameter 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 (IN^2)	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

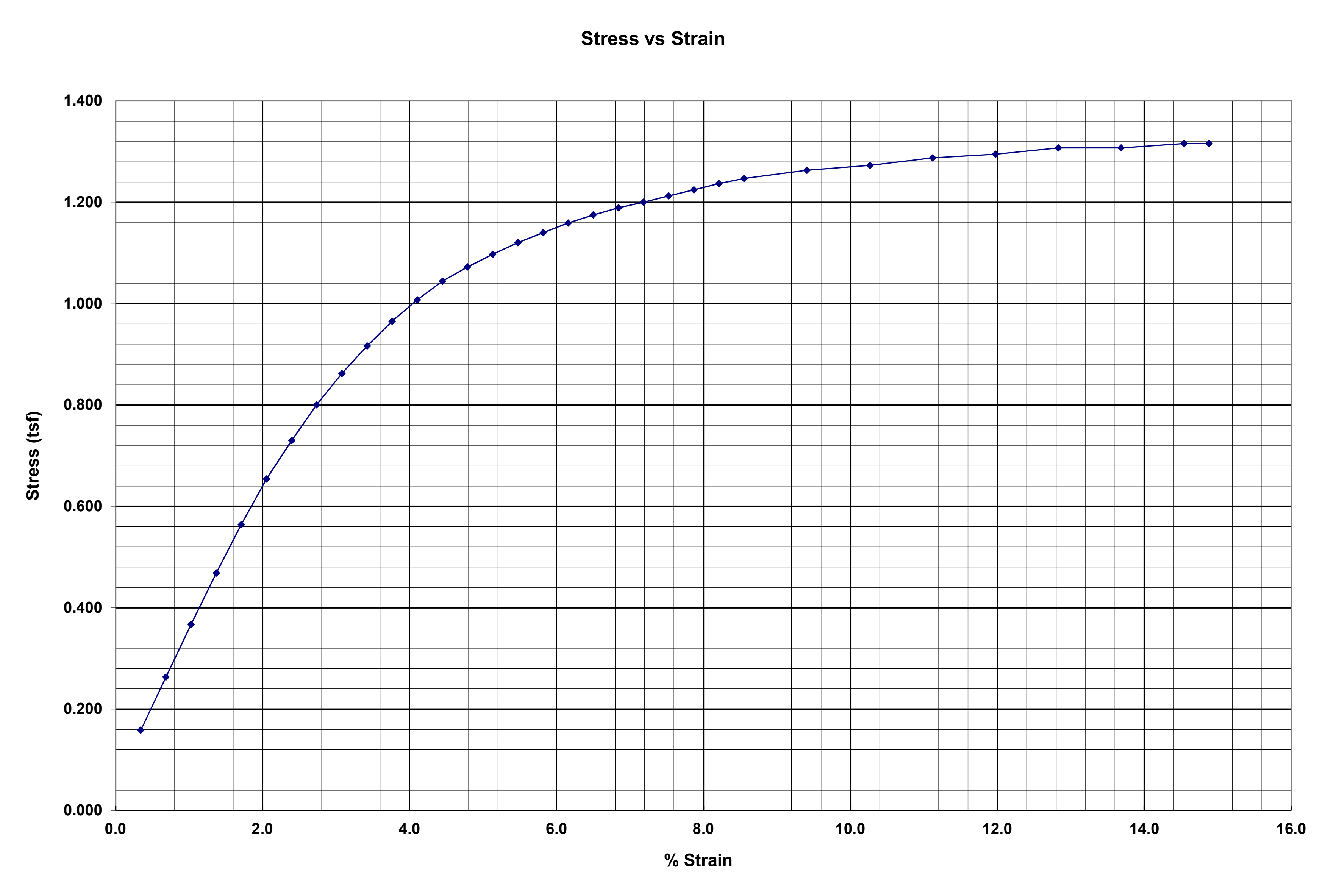
Strain (Inches/Inch) @ 50% Maximum Stress = 0.02069
Deformation @ 50% Maximum Stress (Inches)= 0.1209
Maximum Compressive Strength (TSF)= 1.32
% Strain @ Maximum Strength = 14.54%

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.: P3-32b **Sample No.:** 0 **Depth:** 26.7-28.7
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
TEAM Project No.: 142086 **Date:** 1/6/15
Hole : P3-32 **Sample :** **Depth:** 29-31
Material: Dark brown lean clay

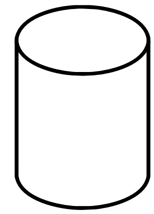
Height 1: 5.854 " **Dia.1:** 2.848 "
Height 2: 5.834 " **Dia.2:** 2.844 " **Area:** 6.384 In^2
Height 3: 5.847 " **Dia.3:** 2.861 "
Young's Modulus for Membrane (tsf) 11.56
Weight g: 1198.5 **Strain Rate:** 0.060 (Inches/Minute)
Wet γ (pcf): 122.4 **Strain Rate:** 1.03 (%/Minute)
Dry γ (pcf): 95.7 **Length/Diameter 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

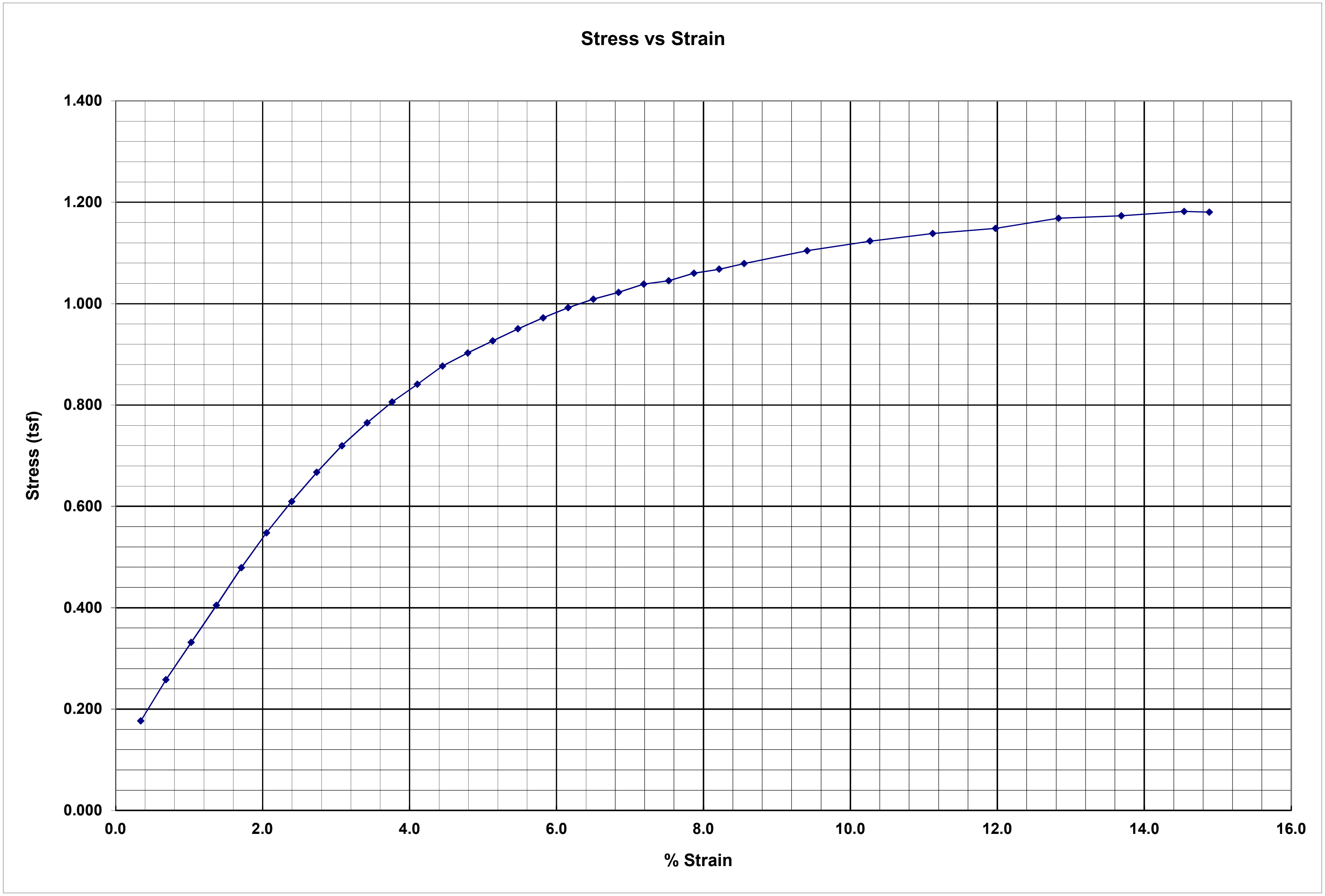
Strain (Inches/Inch) @ 50% Maximum Stress = 0.02291
Deformation @ 50% Maximum Stress (Inches)= 0.1337
Maximum Compressive Strength (TSF)= 1.18
% Strain @ Maximum Strength = 14.54%

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:** 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

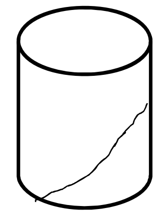
Height 1: 5.860 " **Dia.1:** 2.876 "
Height 2: 5.836 " **Dia.2:** 2.860 " **Area:** 6.456 In²
Height 3: 5.830 " **Dia.3:** 2.865 "
Young's Modulus for Membrane (tsf) 11.56
Weight g: 1221.5 **Strain Rate:** 0.060 (Inches/Minute)
Wet γ (pcf): 123.4 **Strain Rate:** 1.03 (%/Minute)
Dry γ (pcf): 95.5 **Length/Diameter 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%

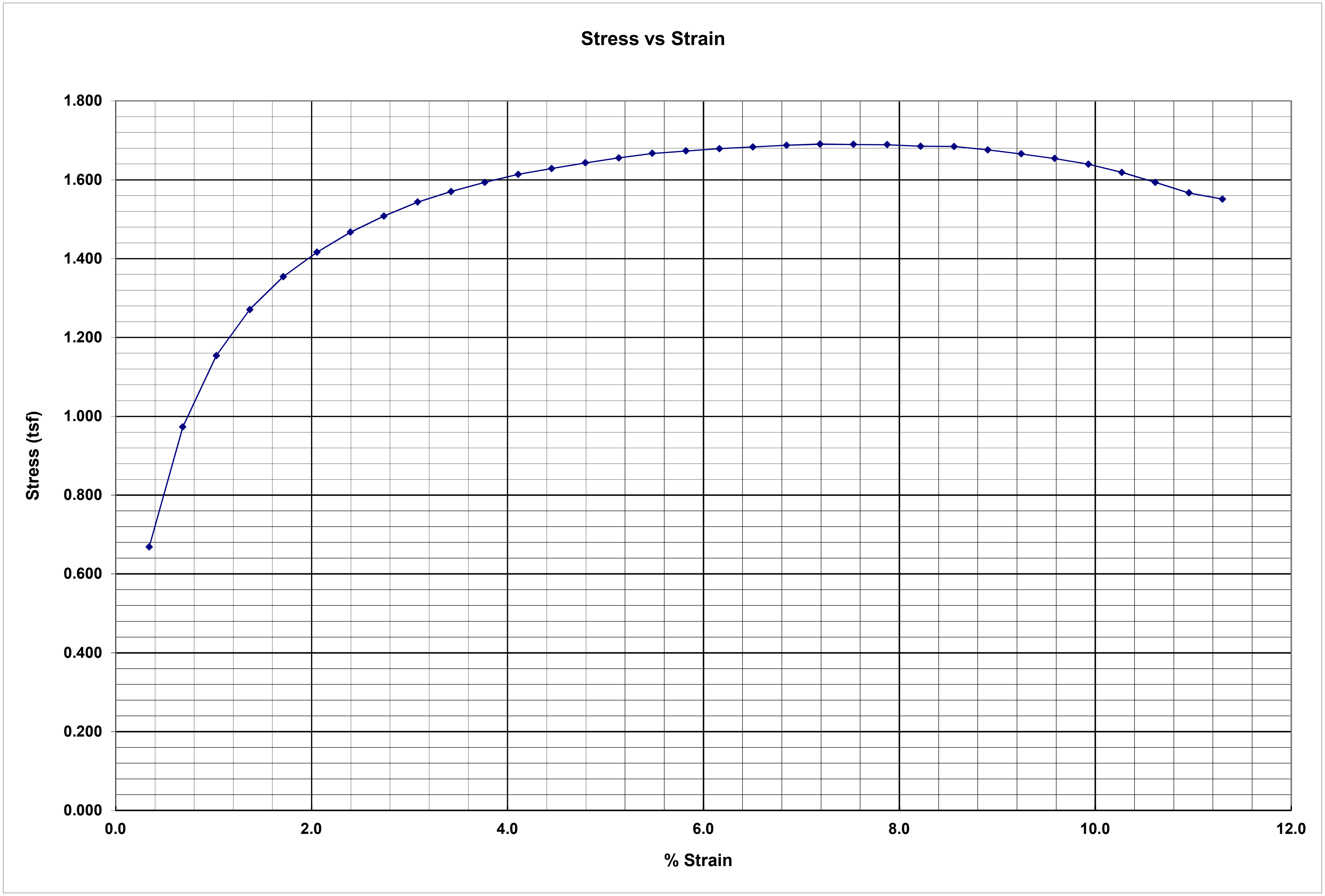
* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

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

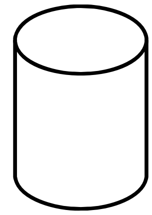
Height 1:	5.840 "	Dia.1:	2.862 "		
Height 2:	5.849 "	Dia.2:	2.875 "	Area:	6.471 In^2
Height 3:	5.849 "	Dia.3:	2.874 "		
Young's Modulus for Membrane (tsf)	11.56				
Weight g:	1227.4	Strain Rate:	0.060	(Inches/Minute)	
Wet γ (pcf):	123.6	Strain Rate:	1.03	(%/Minute)	
Dry γ (pcf):	98.3	Length/Diameter 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

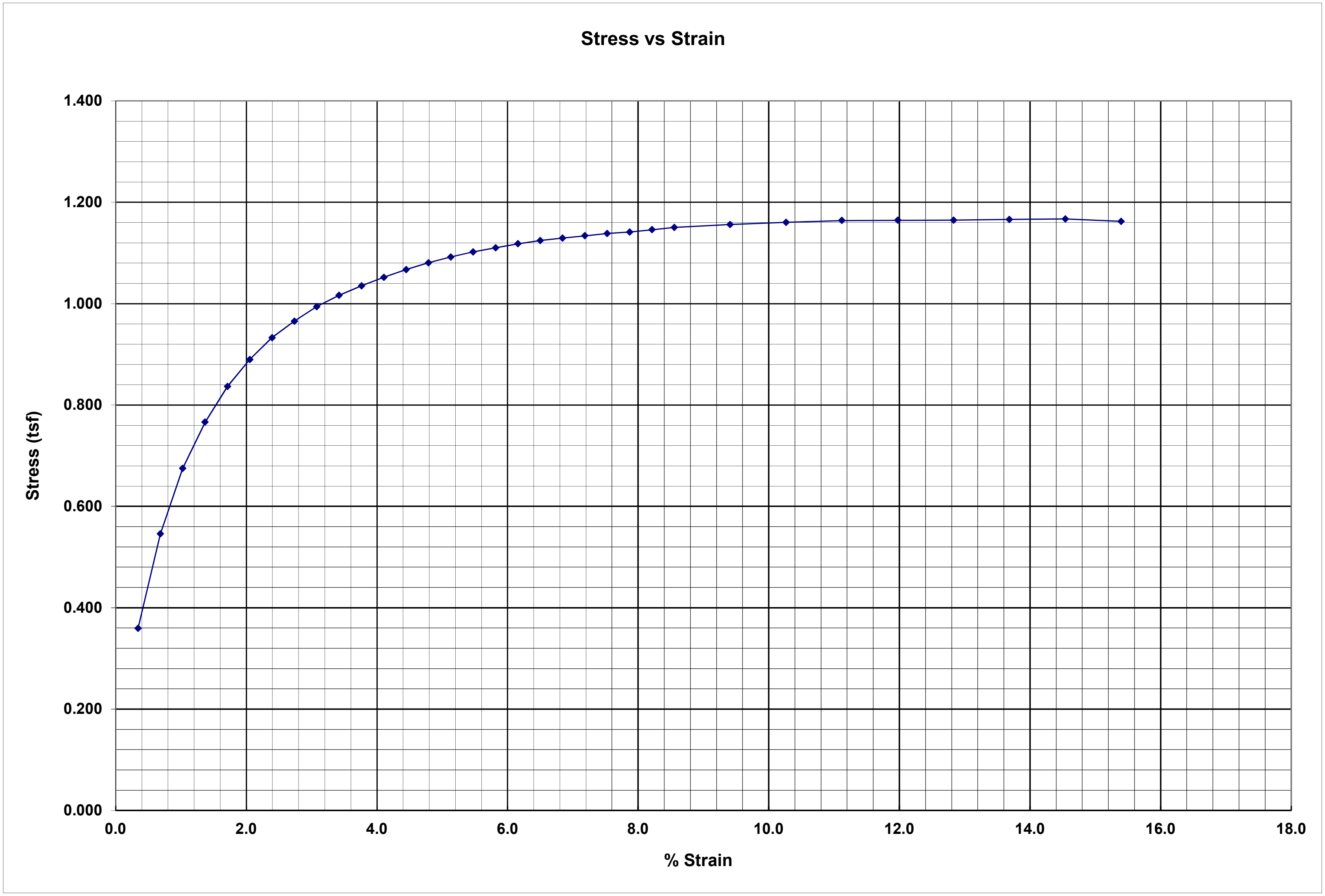
* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

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
TEAM Project No.: 142086 **Date:** 1/6/15
Hole : P3-33 **Sample :** **Depth:** 6.4-8.4
Material: Brown lean clay

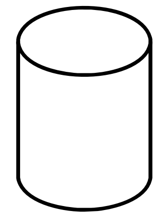
Height 1: 5.840 " **Dia.1:** 2.858 "
Height 2: 5.846 " **Dia.2:** 2.849 " **Area:** 6.351 In²
Height 3: 5.842 " **Dia.3:** 2.824 "
Young's Modulus for Membrane (tsf) 11.56
Weight g: 1188.3 **Strain Rate:** 0.060 (Inches/Minute)
Wet γ (pcf): 122.0 **Strain Rate:** 1.03 (%/Minute)
Dry γ (pcf): 97.9 **Length/Diameter 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

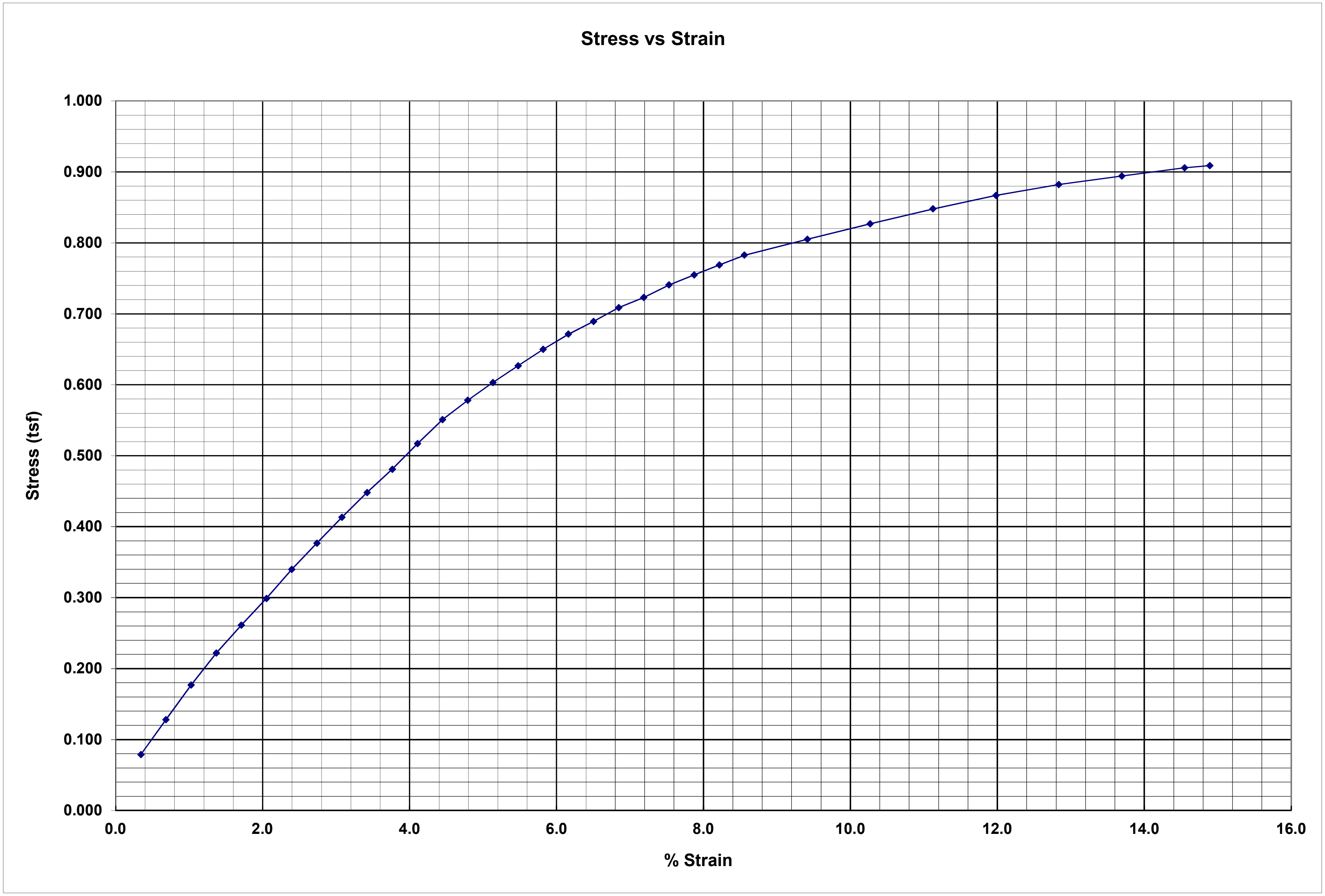
* A membrane correction has been applied in compliance with ASTM D-2850. Membrane thickness: 0.012 inches (Young's Modulus for Membrane = 11.56 tsf)

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

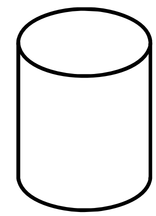
Height 1: 5.830 "	**Dia.1:** 2.783 "		
Height 2: 5.816 "	**Dia.2:** 2.757 "	**Area:** 6.028 In²	
Height 3: 5.840 "	**Dia.3:** 2.771 "		
Young's Modulus for Membrane (tsf) 11.56			
Weight g: 1123.7	**Strain Rate:** 0.060	(Inches/Minute)	
Wet γ (pcf): 121.8	**Strain Rate:** 1.03	(%/Minute)	
Dry γ (pcf): 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

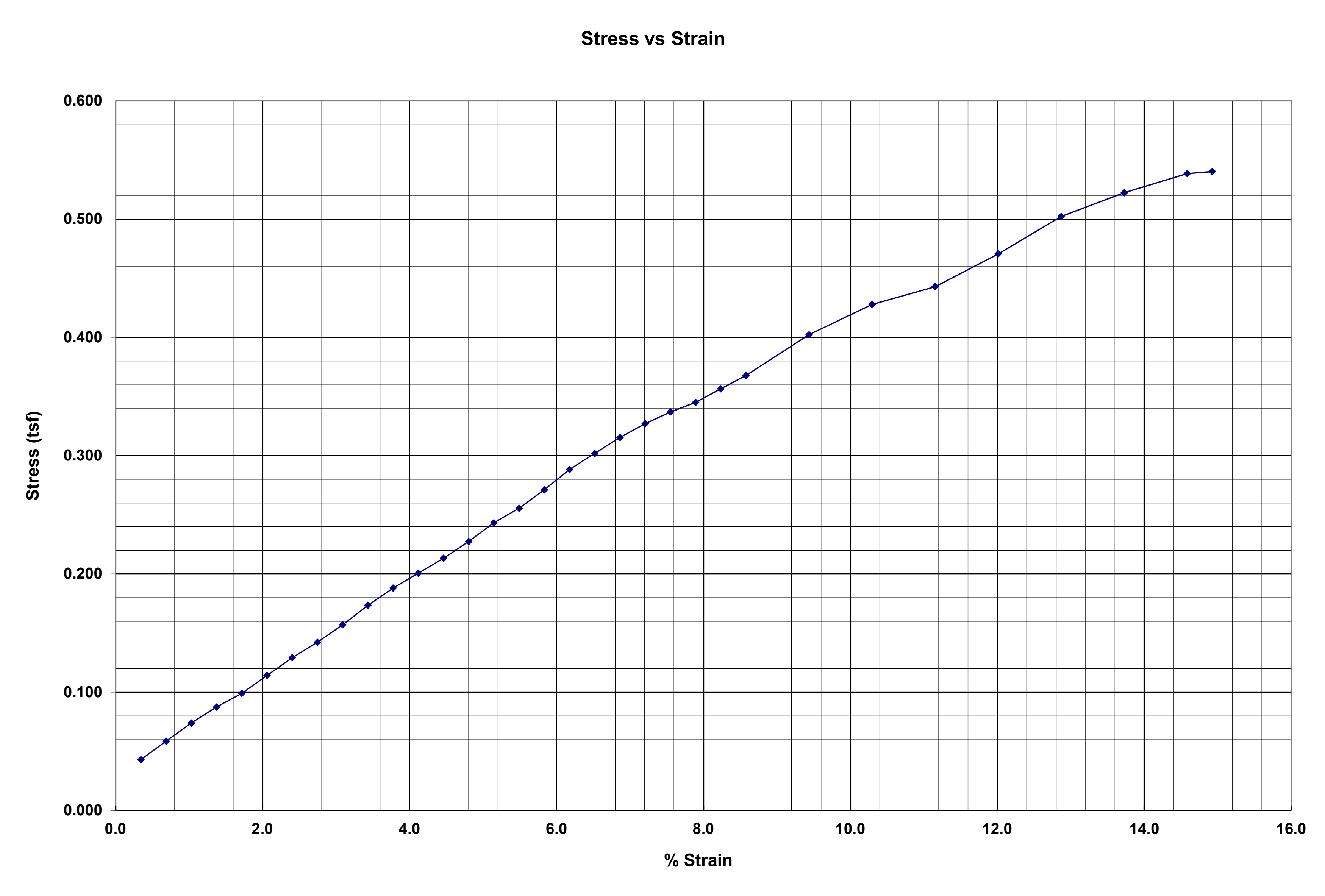
Maximum Compressive Strength (TSF)= 0.54

% 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

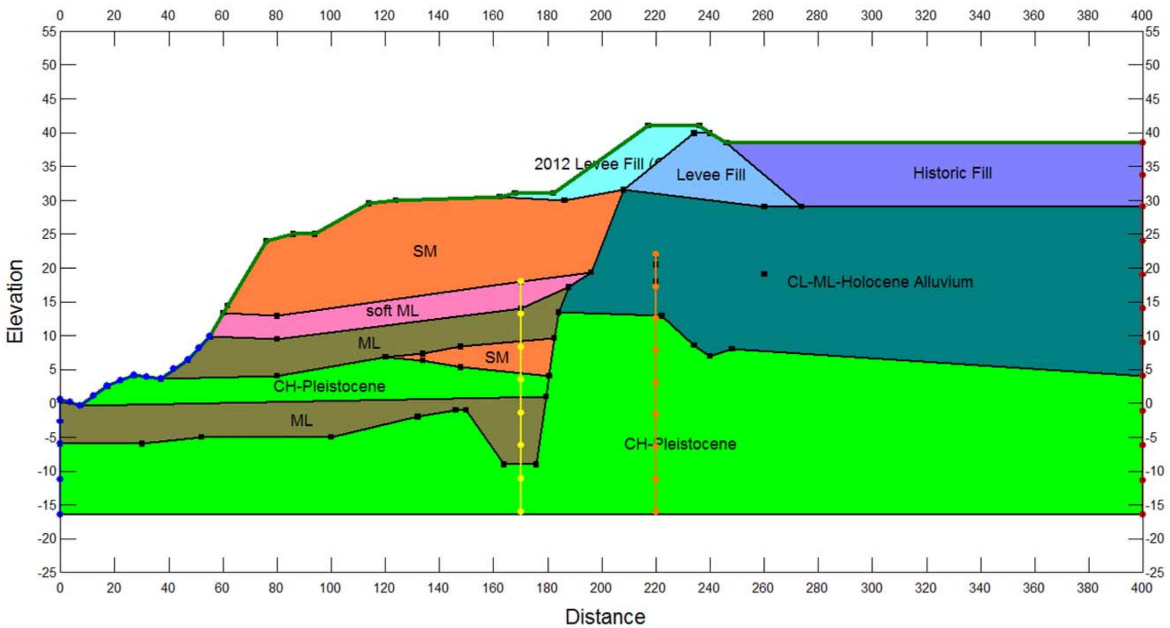
Project:	USACE-Brownsville Levee	**Hole No.:**	P3-33	**Sample No.:**	0	**Depth:**	10.8-12.8
TEAM Project No.:	142086	**Material:**	Brown silty clay with sand				
Date:	1/6/15						

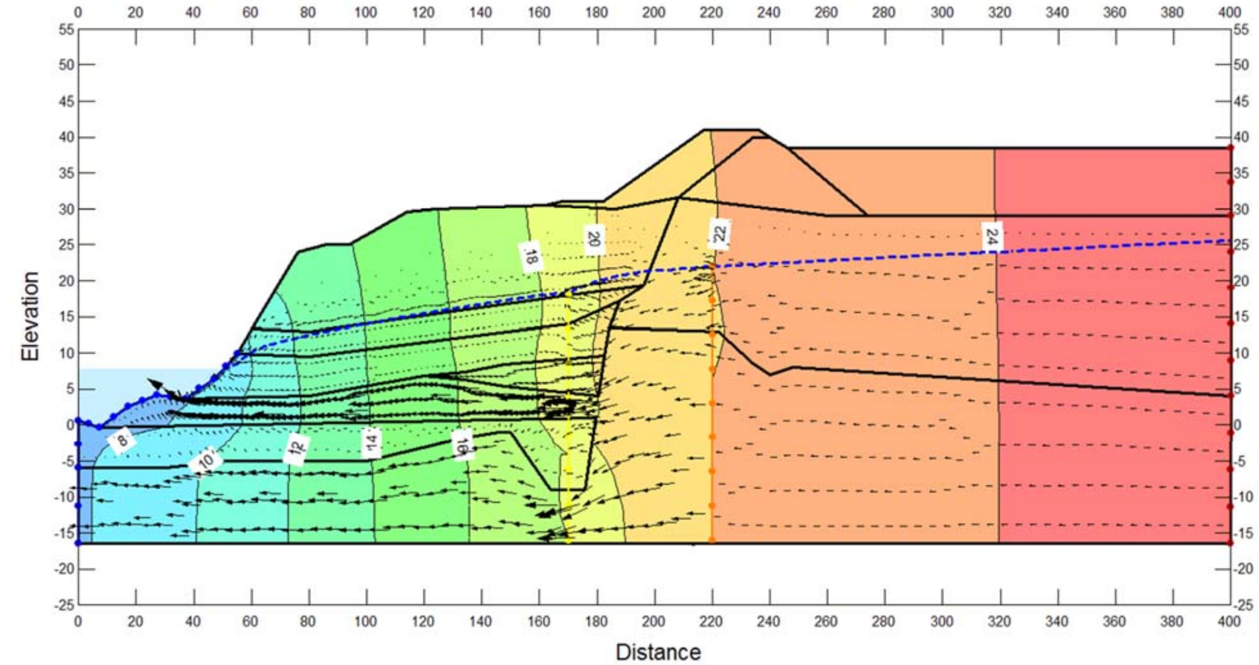


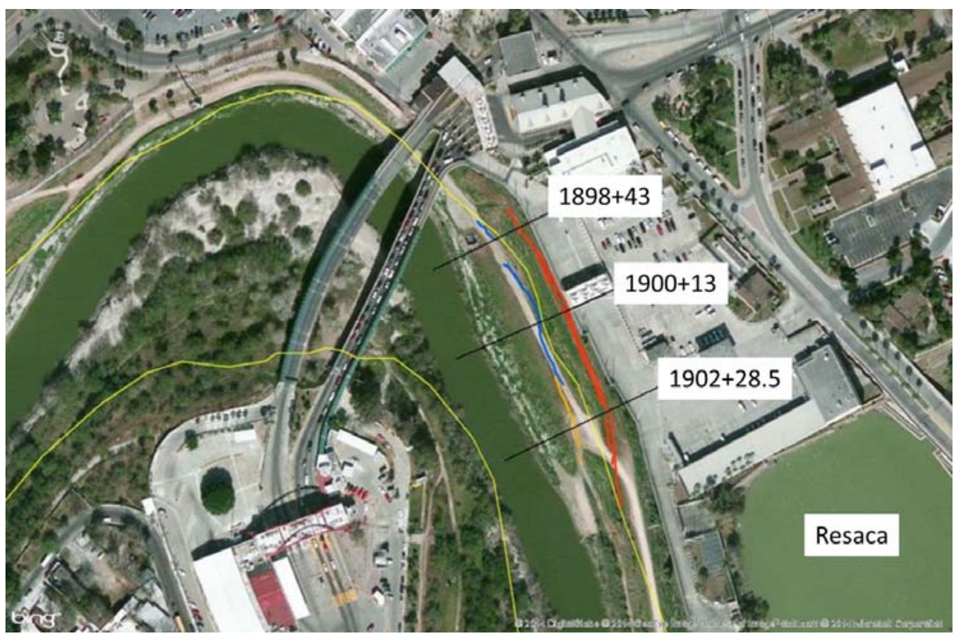
TEAM Consultants, Inc.

APPENDIX F

USACE Seepage and Slope Stability Model Plates







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

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

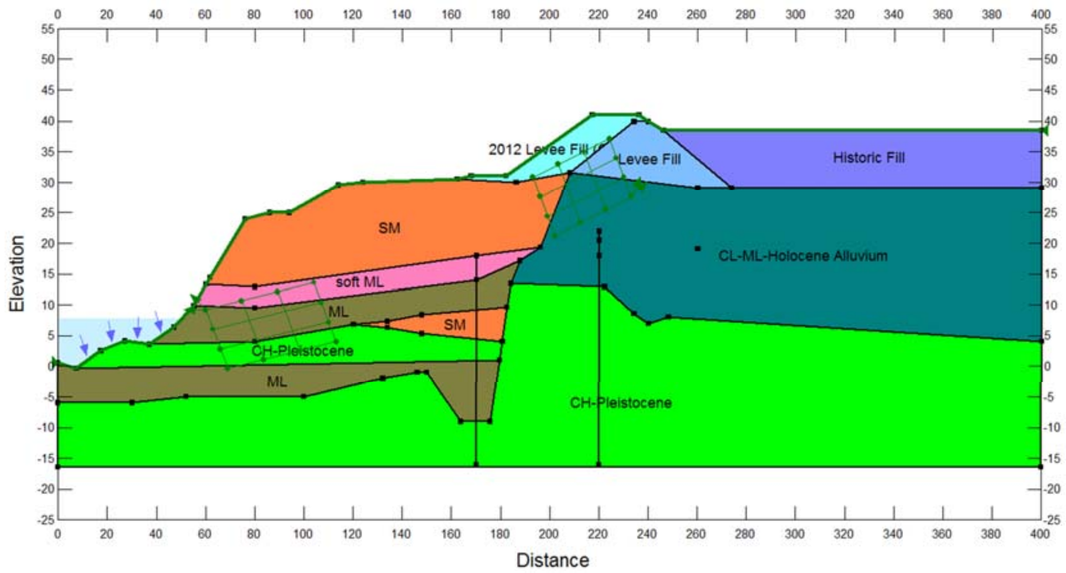
IBWC-BROWNSVILLE LEVEE

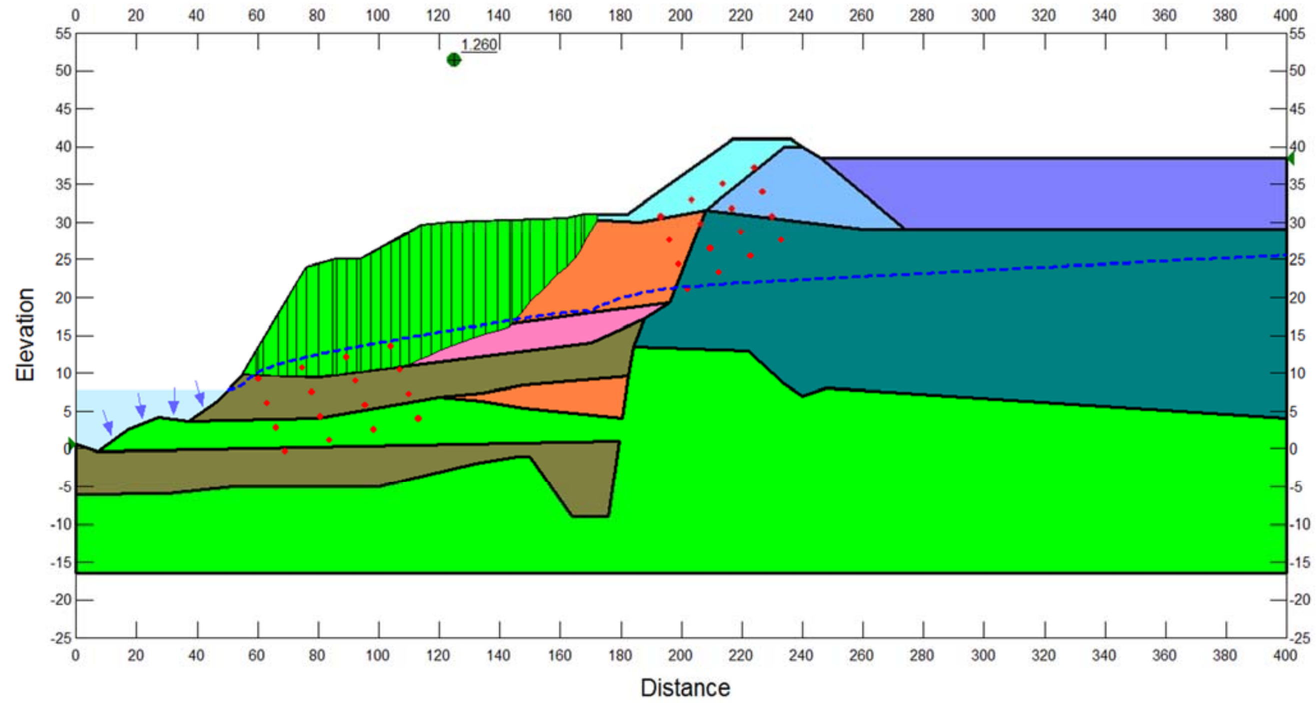
STEADY STATE SEEPAGE, SATURATED MODEL

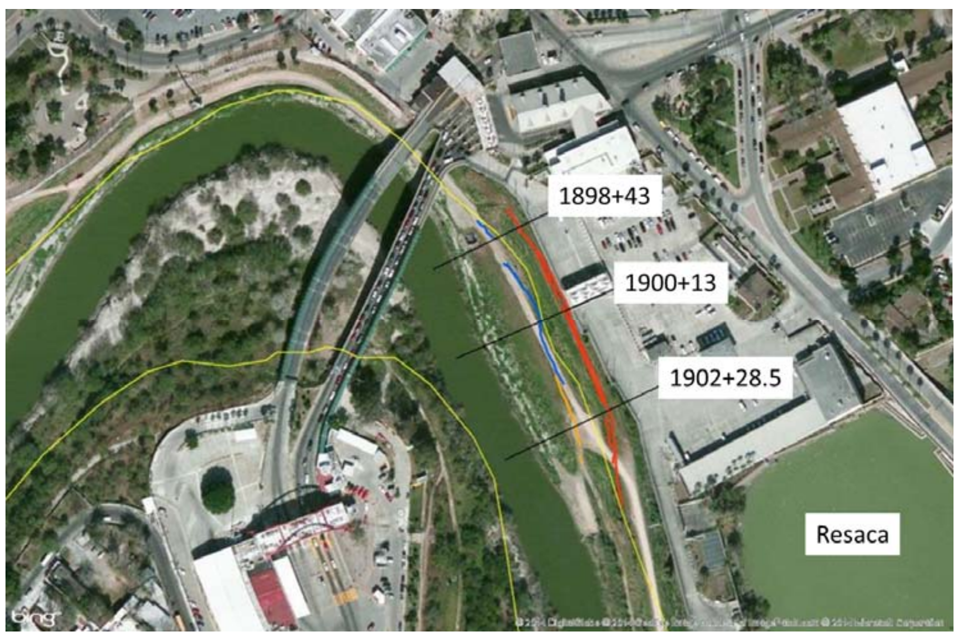
STEADY STATE SEEPAGE (WSE 7.77 FT)

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

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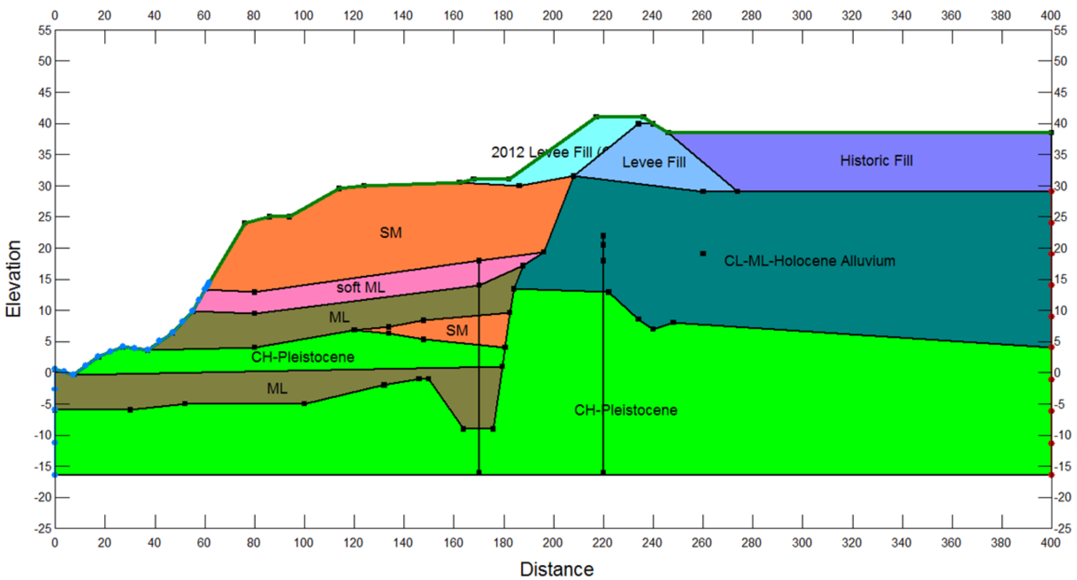
IBWC-BROWNSVILLE LEVEE

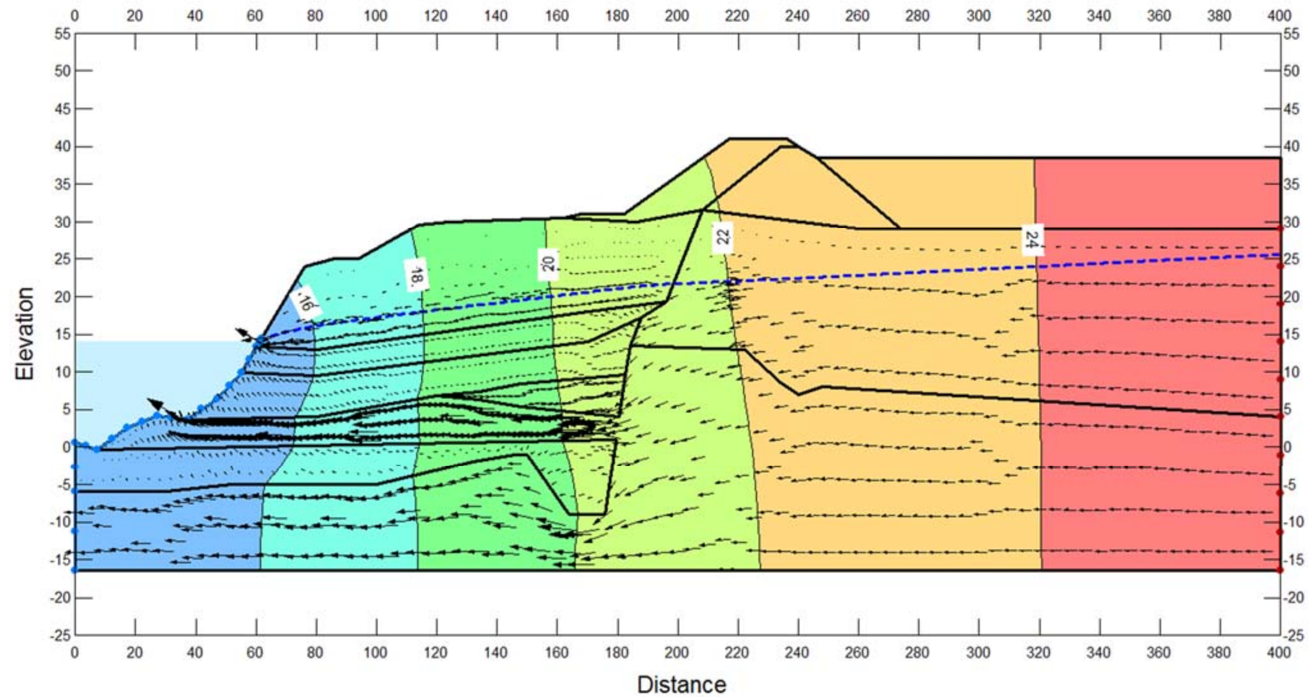
STABILITY MODEL

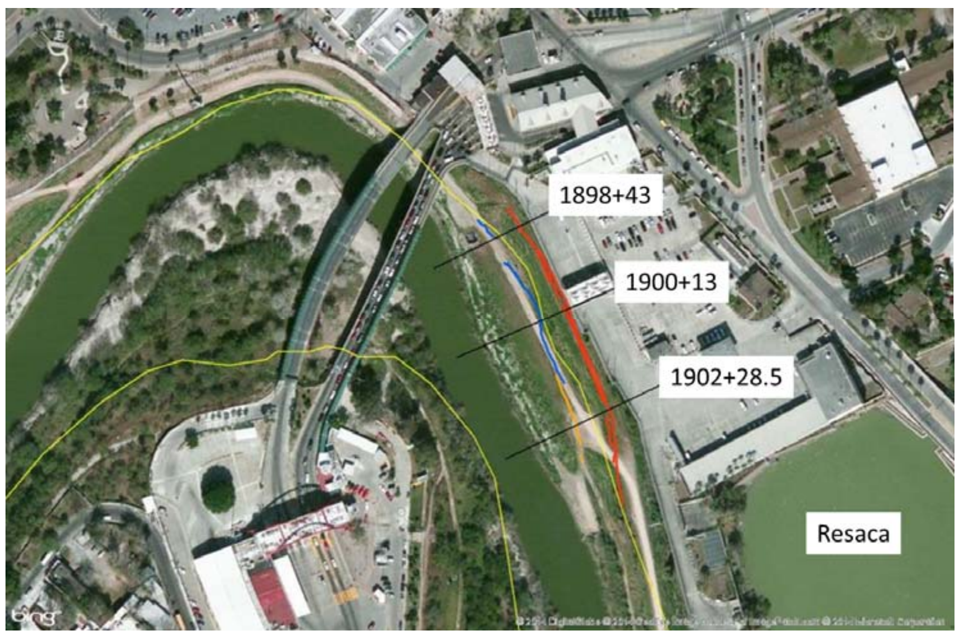
STEADY STATE FOS(WSE 7.77 FT)

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

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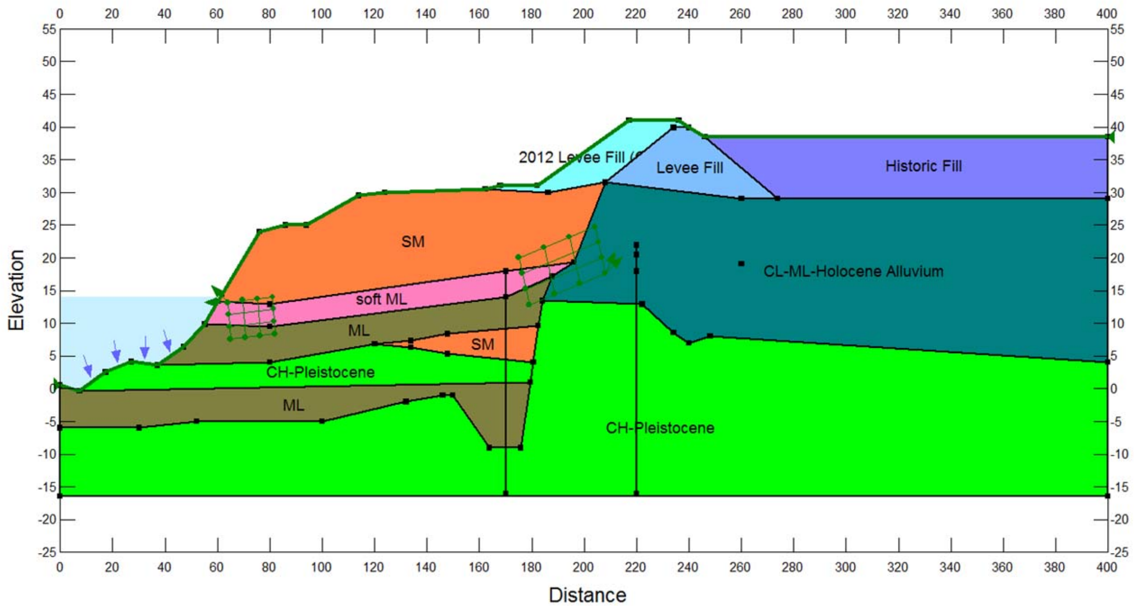
IBWC-BROWNSVILLE LEVEE

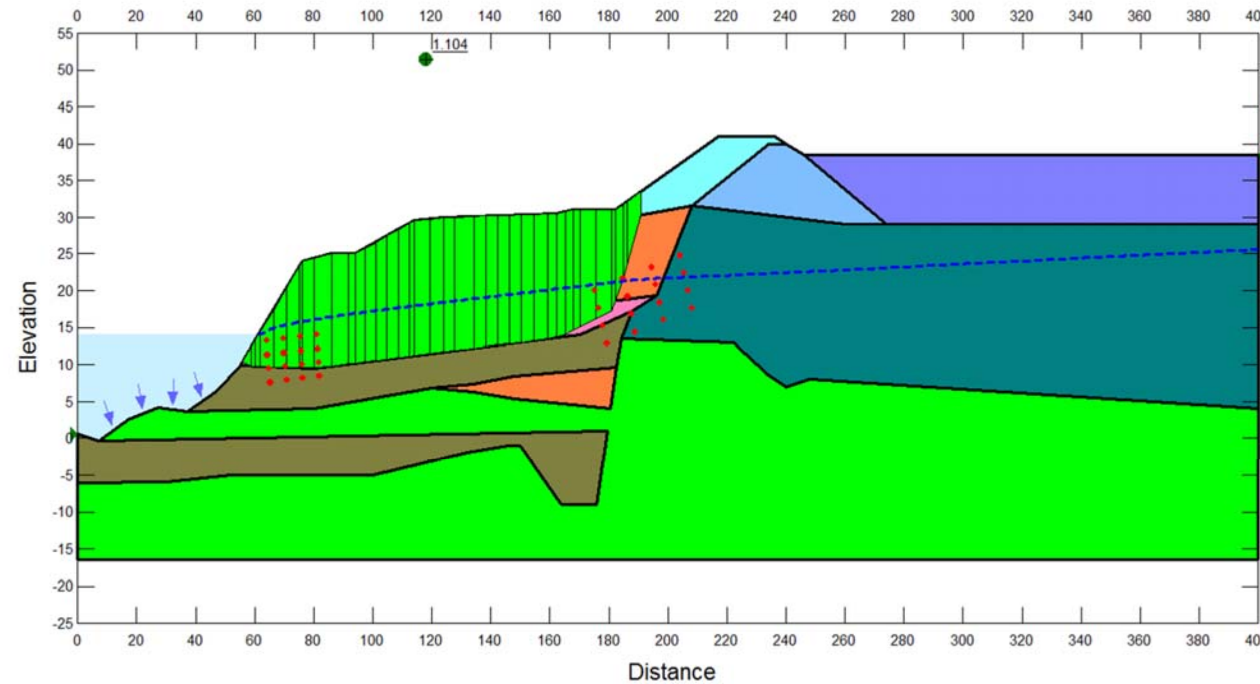
STEADY STATE SEEPAGE, SATURATED MODEL

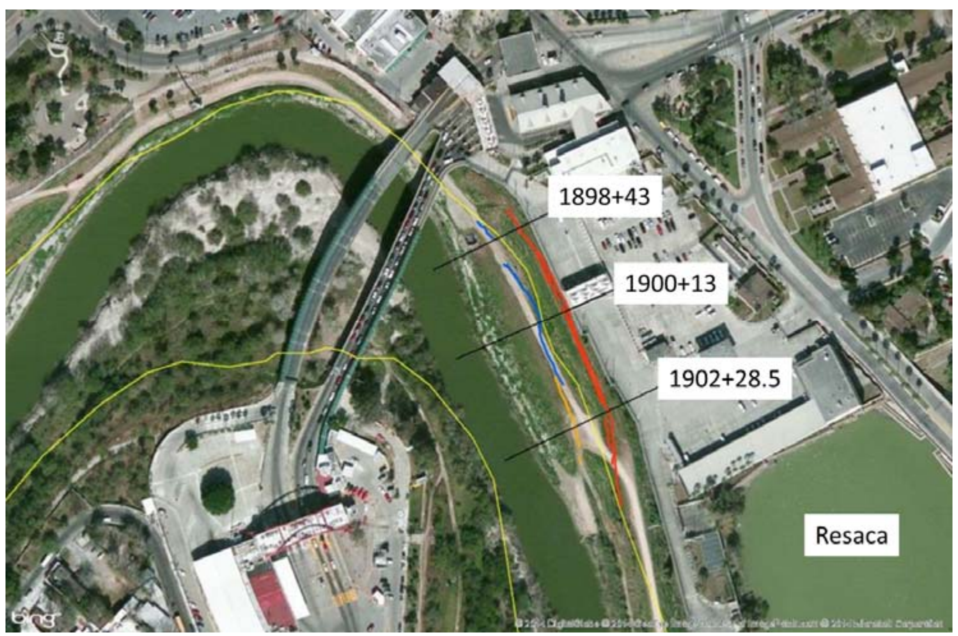
STEADY STATE SEEPAGE (WSE 14.31 FT)

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

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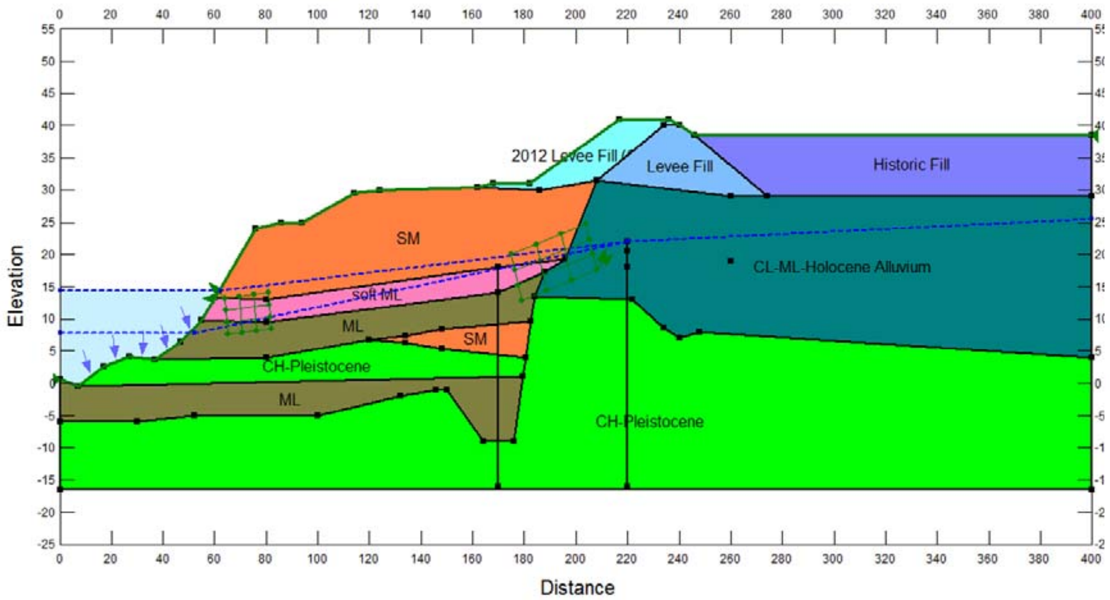
IBWC-BROWNSVILLE LEVEE

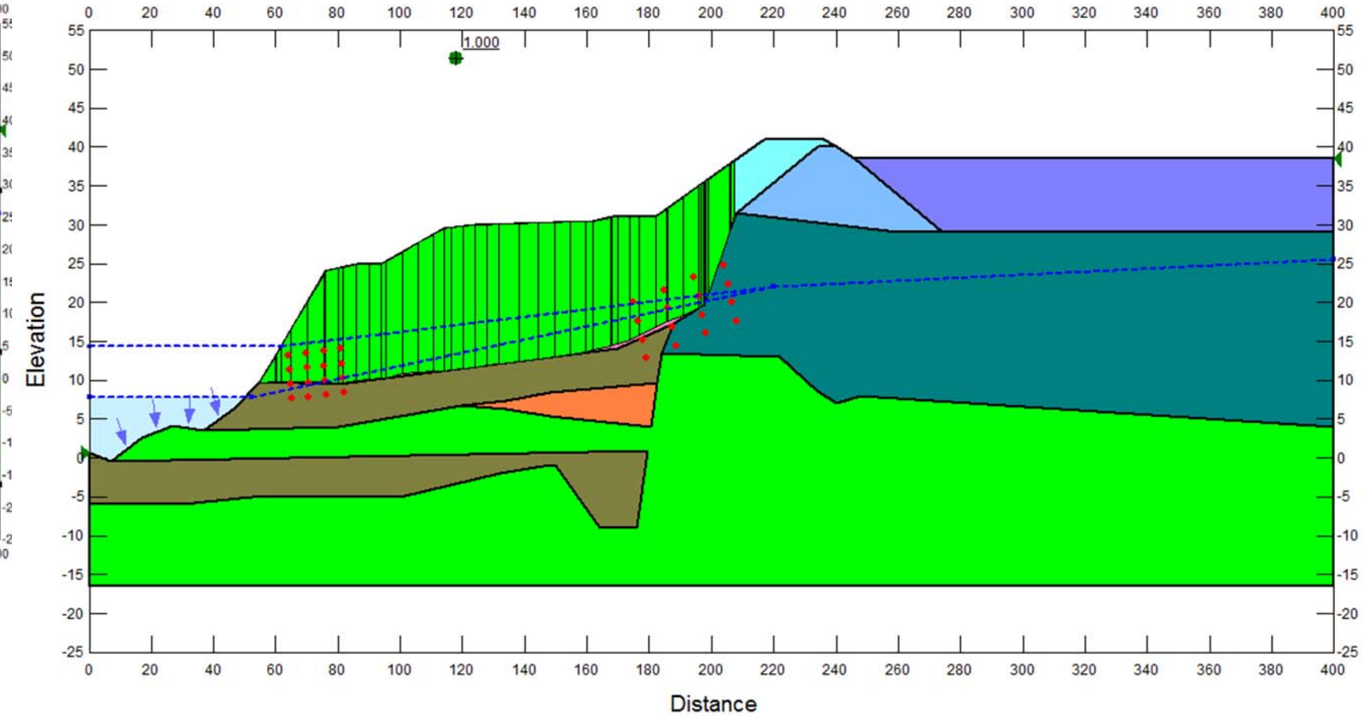
STABILITY MODEL

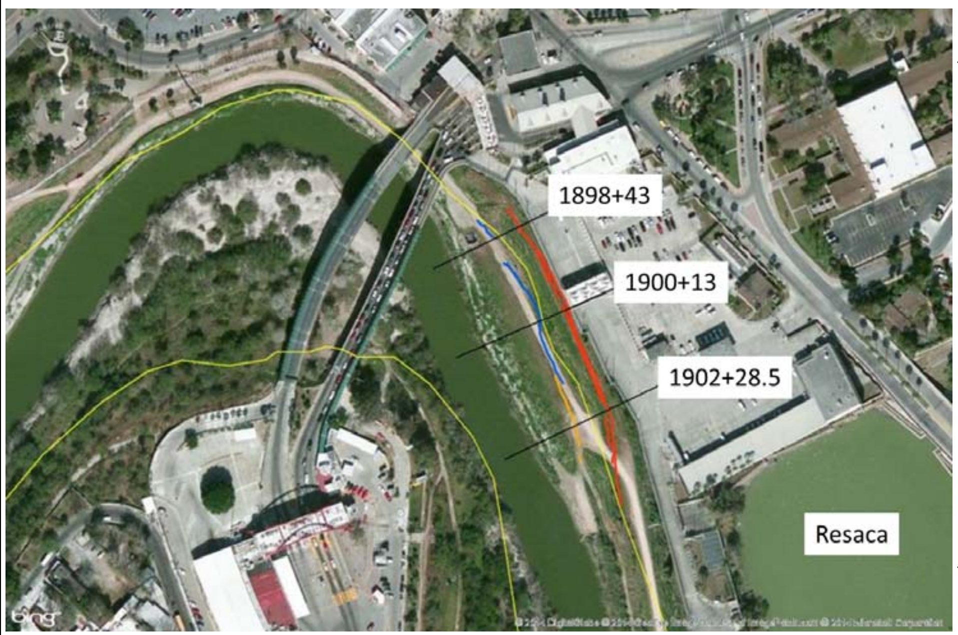
STEADY STATE FOS(WSE 14.31 FT)

STATION 1900+13

FEB-2015 **PLATE - 4**







				total stress	
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	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

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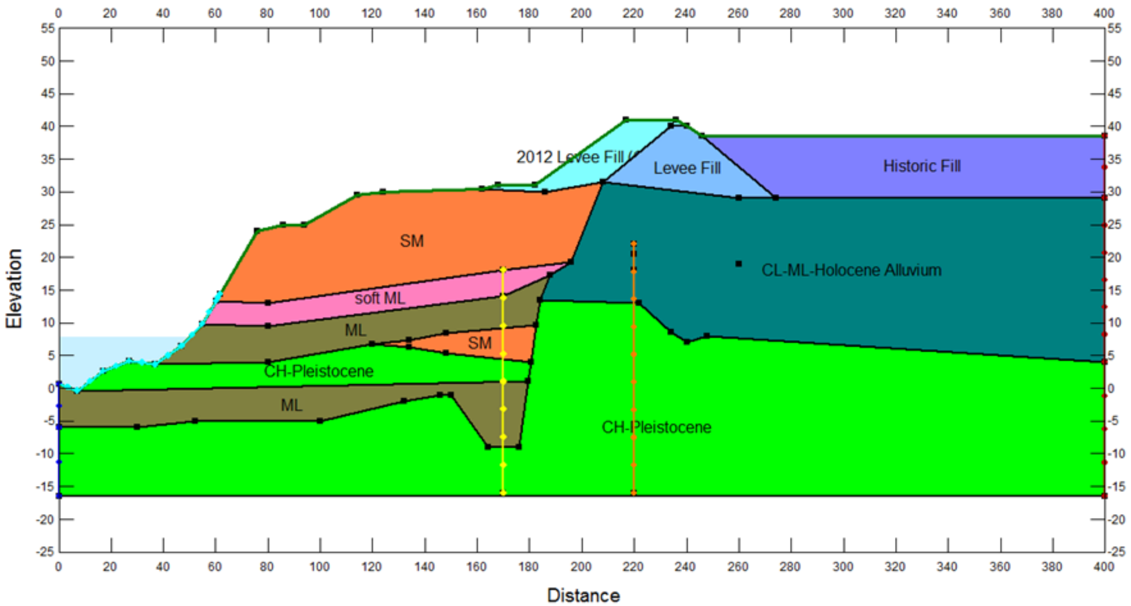
IBWC-BROWNSVILLE LEVEE

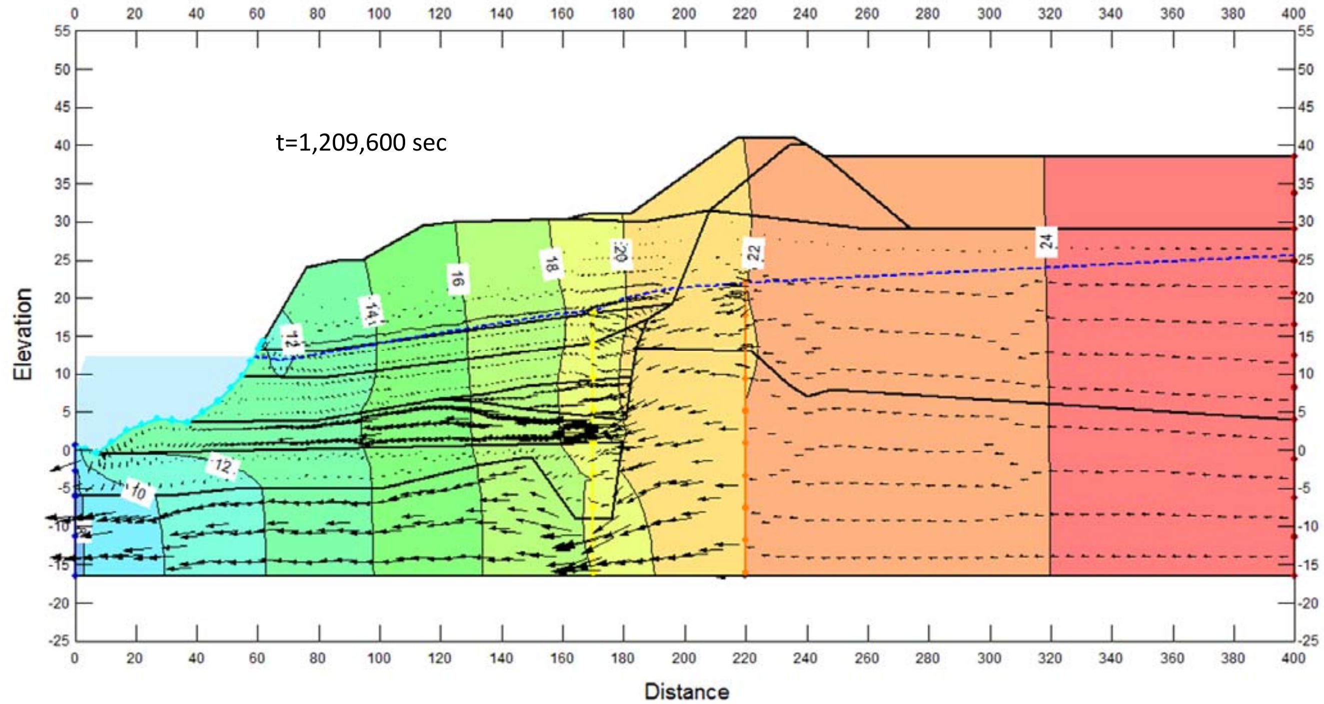
RAPID DRAWDOWN STABILITY

RAPID DRAWDOWN (WSE 7.77 & 14.31 FT)

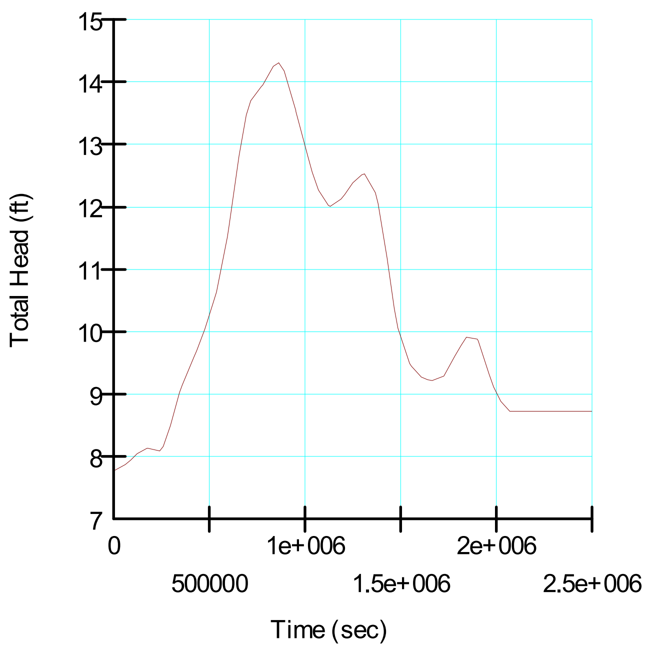
STATION 1900+13

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)

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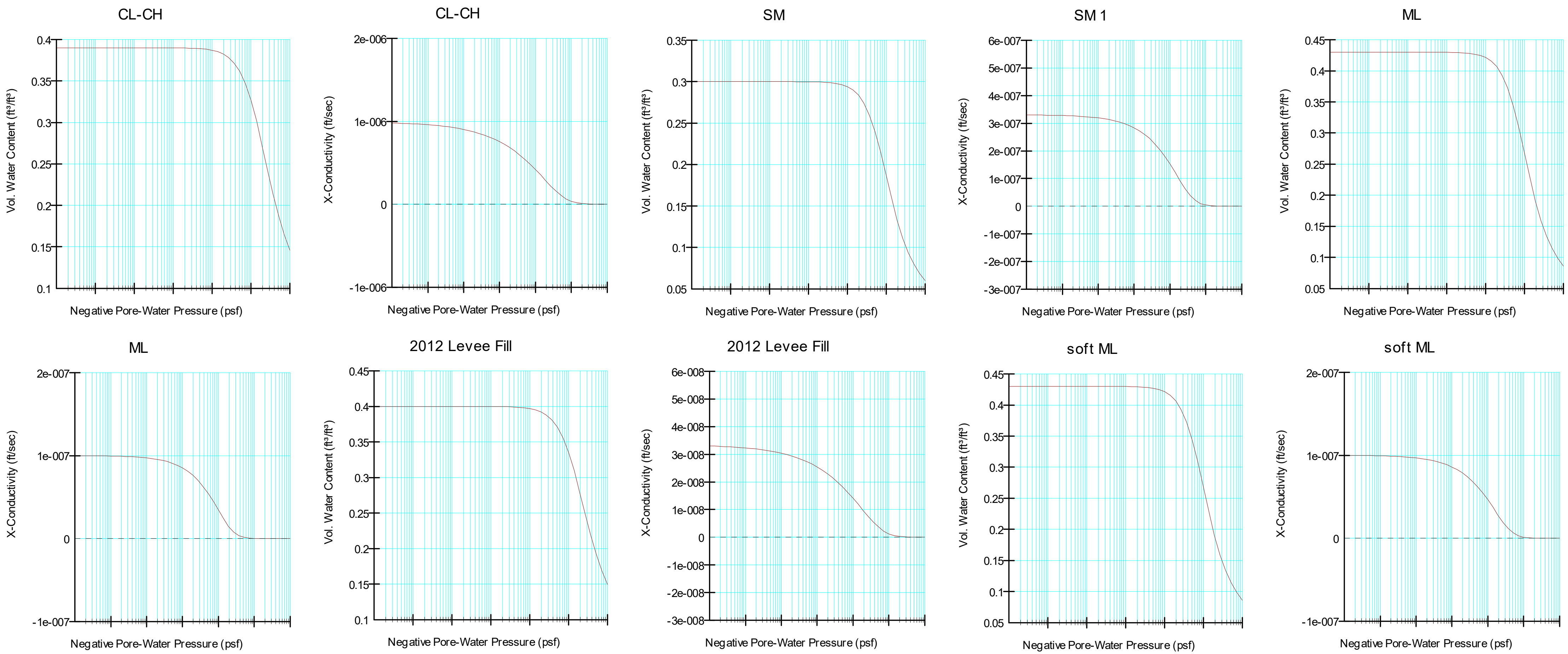
IBWC-BROWNSVILLE LEVEE

HYDROGRAPH, SATURATED MODEL

STEADY STATE SEEPAGE (WSE 14.31 FT)

STATION 1900+13

FEB-2015 **PLATE - 6**



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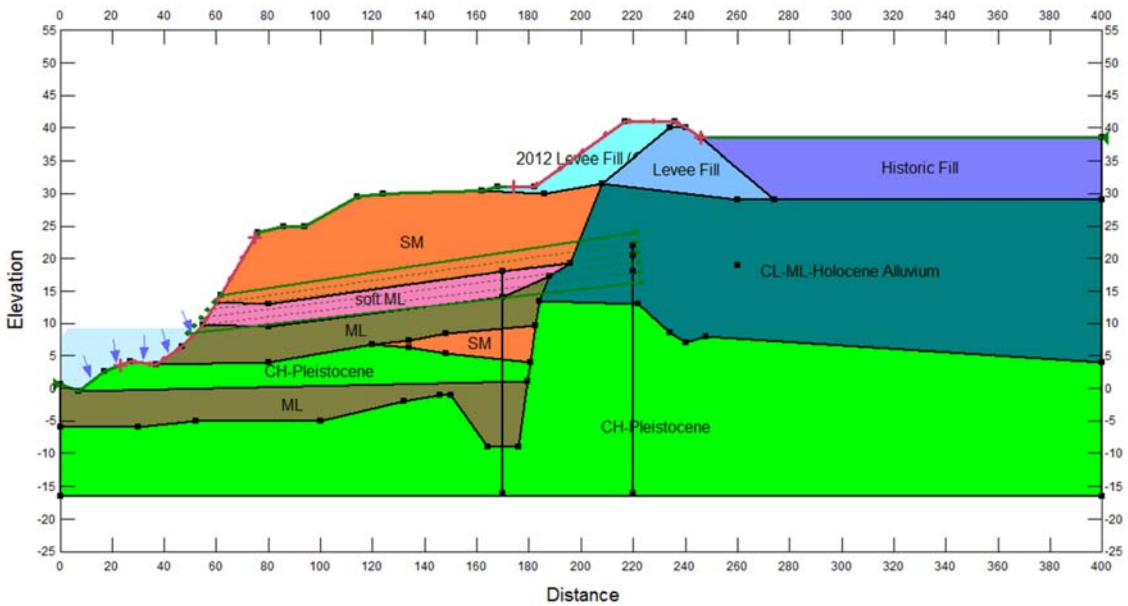
IBWC-BROWNSVILLE LEVEE

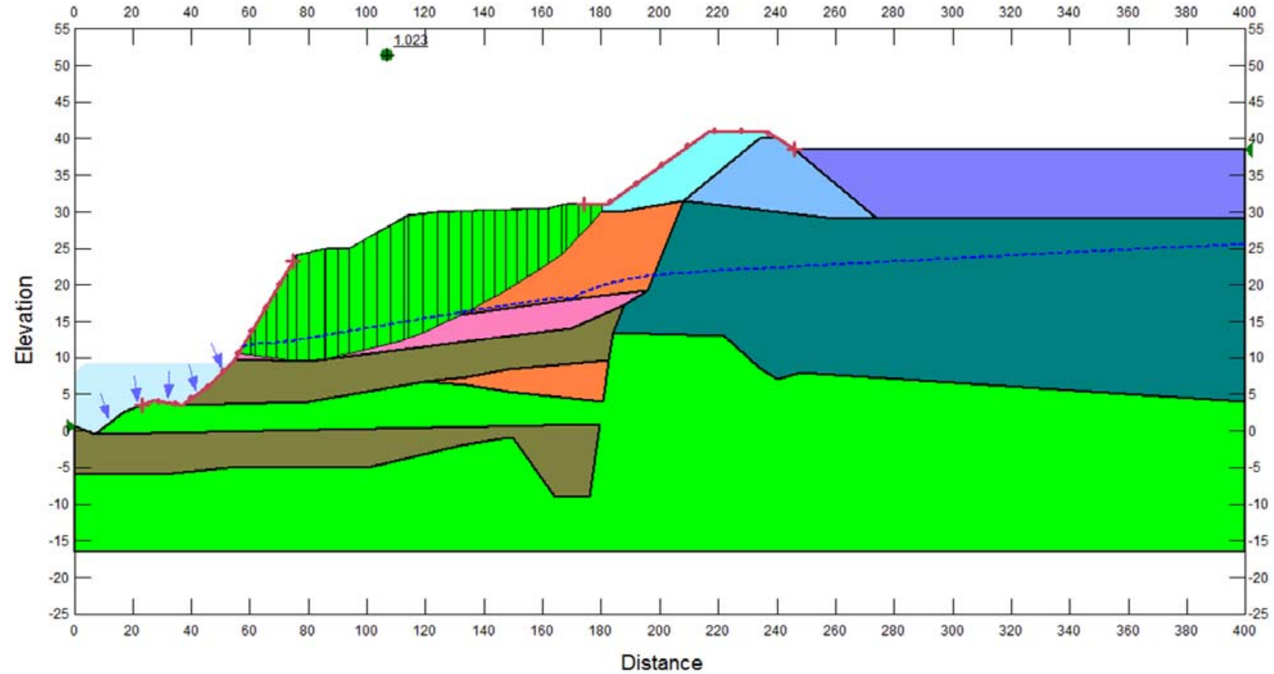
TRANSIENT HYDRAULIC PROPETIES

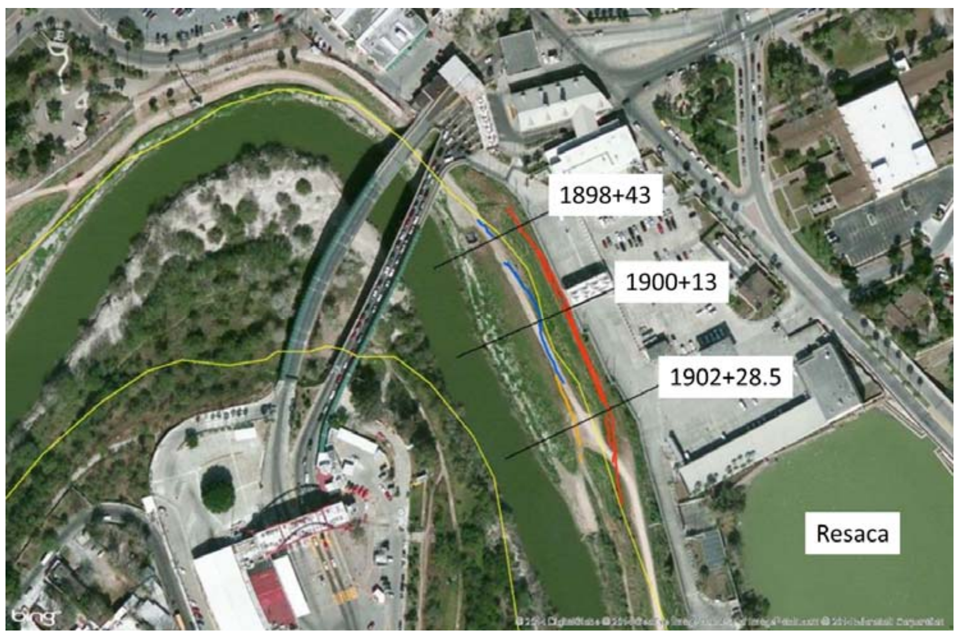
SWCC AND HCF'S

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			

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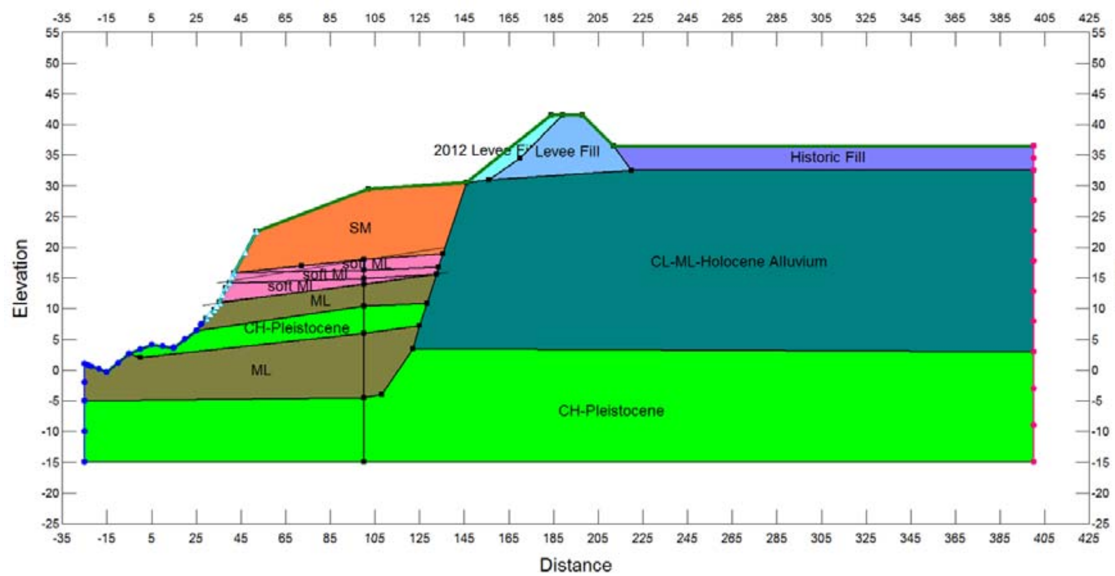
IBWC-BROWNSVILLE LEVEE

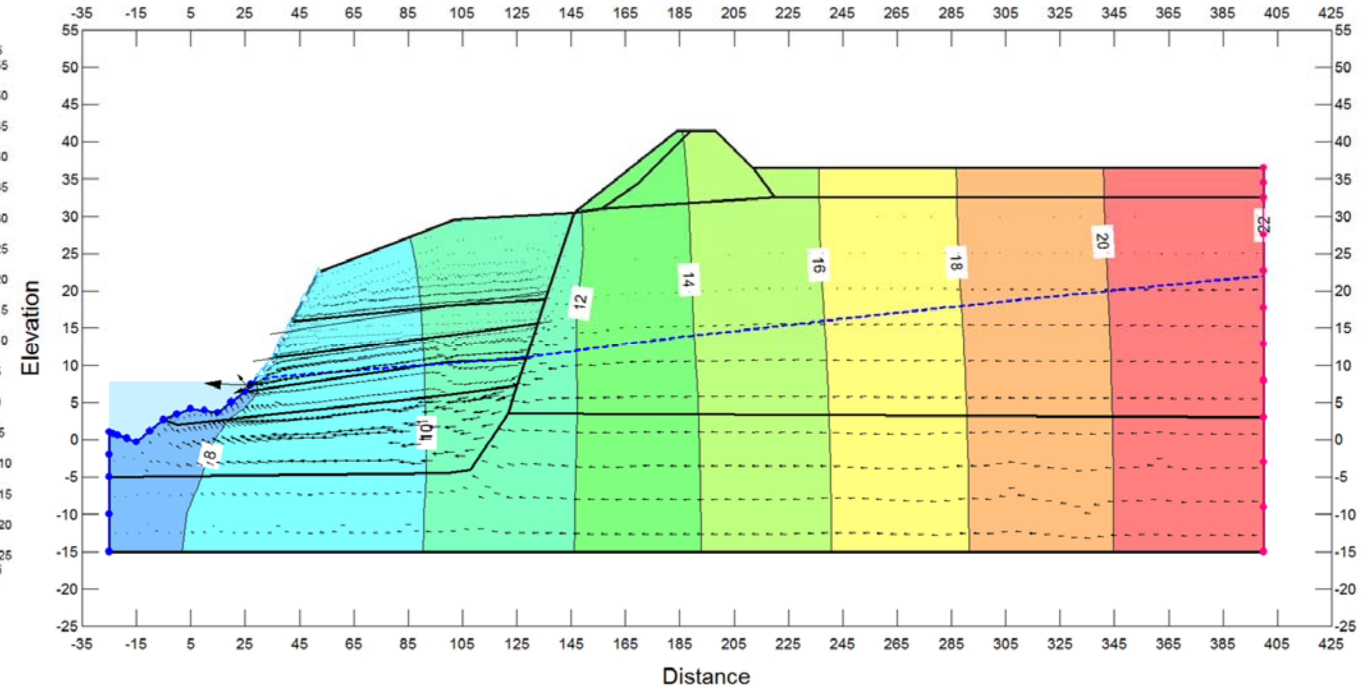
STABILITY MODEL

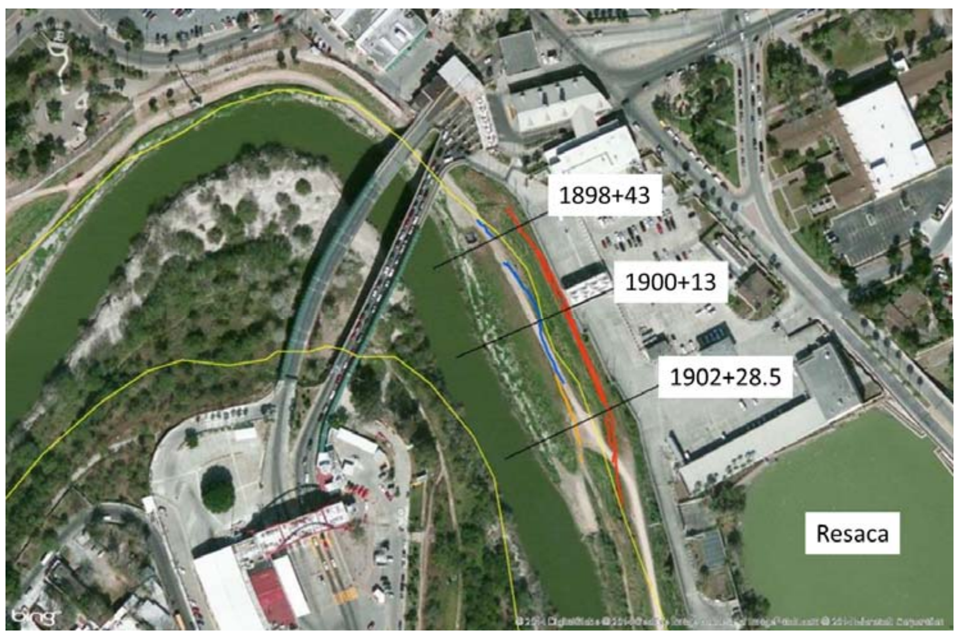
TRANSIENT FOS (HYDROGRAPH)

STATION 1900+13

FEB-2015 **PLATE - 8**







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	22.00

U.S. ARMY CORPS OF ENGINEERS

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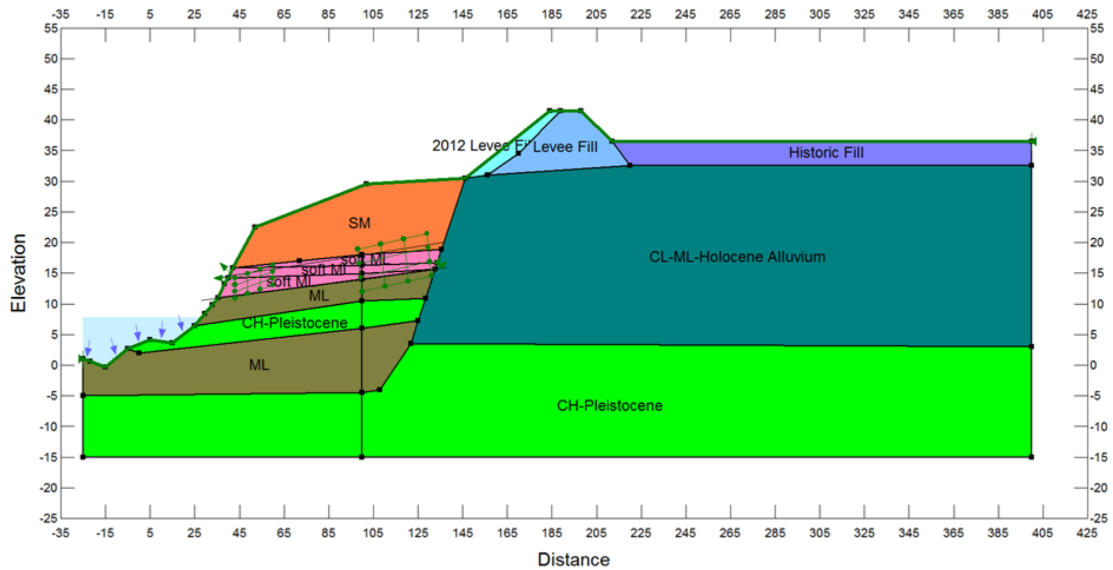
IBWC-BROWNSVILLE LEVEE

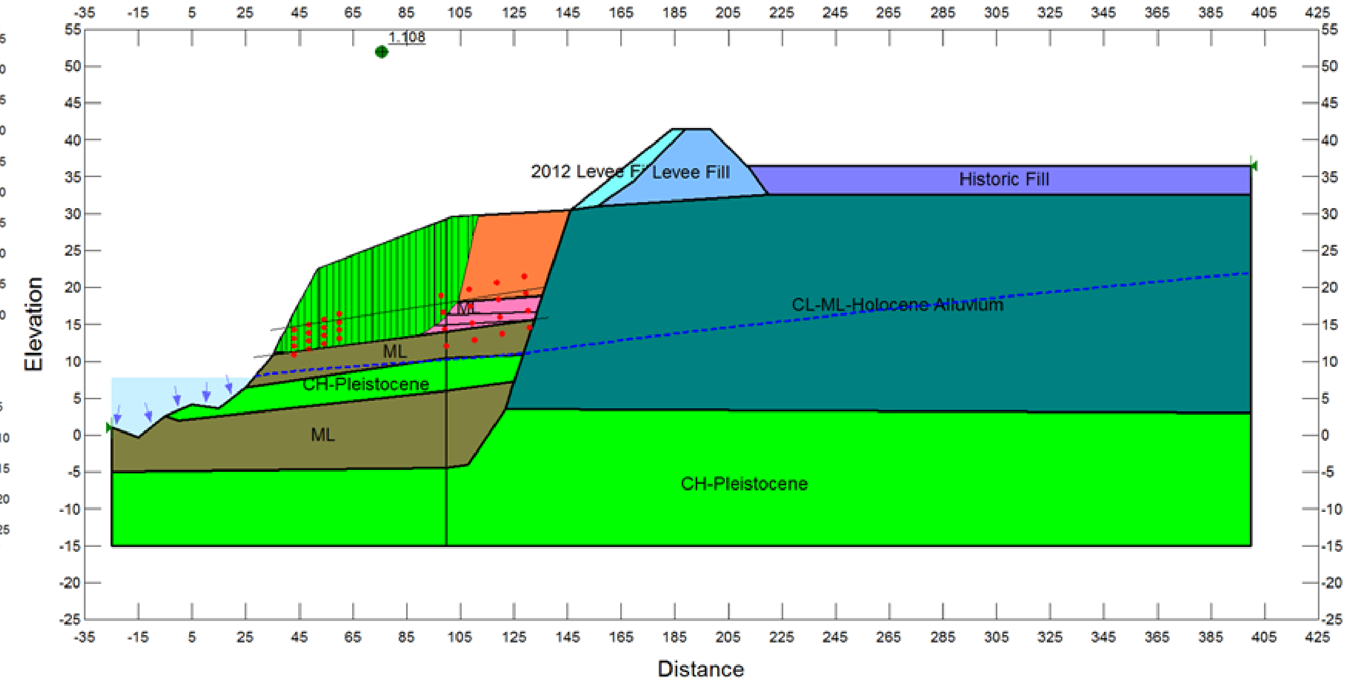
STEADY STATE SEEPAGE, SATURATED MODEL

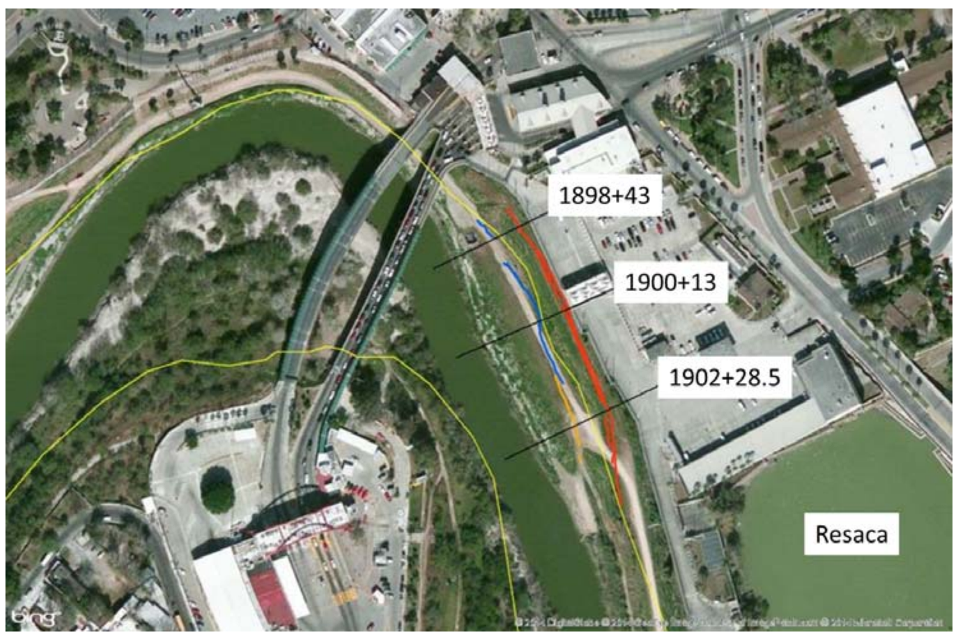
STEADY STATE SEEPAGE (WSE 7.77 FT)

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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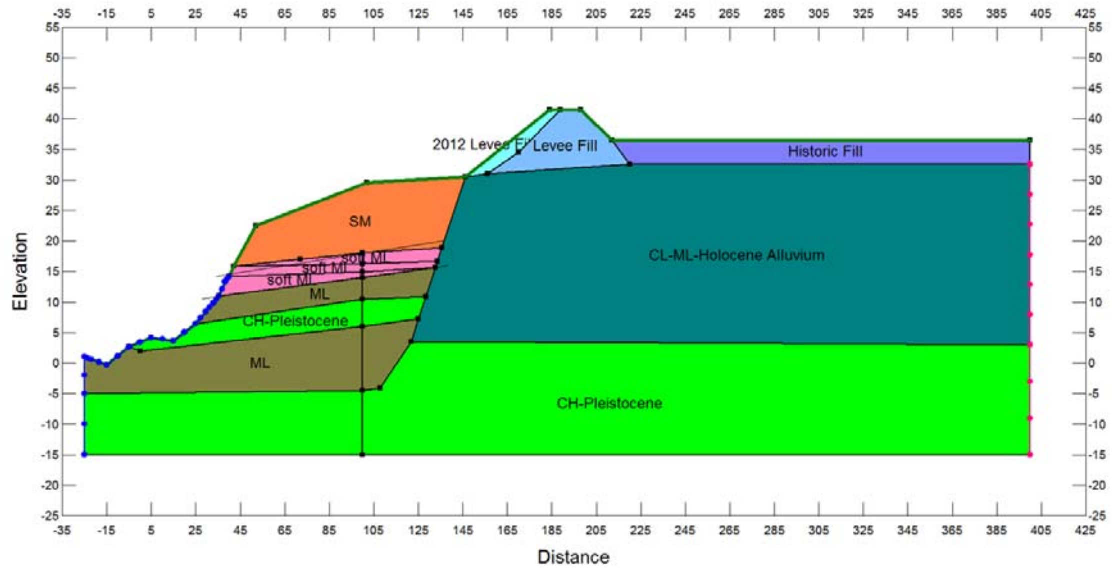
IBWC-BROWNSVILLE LEVEE

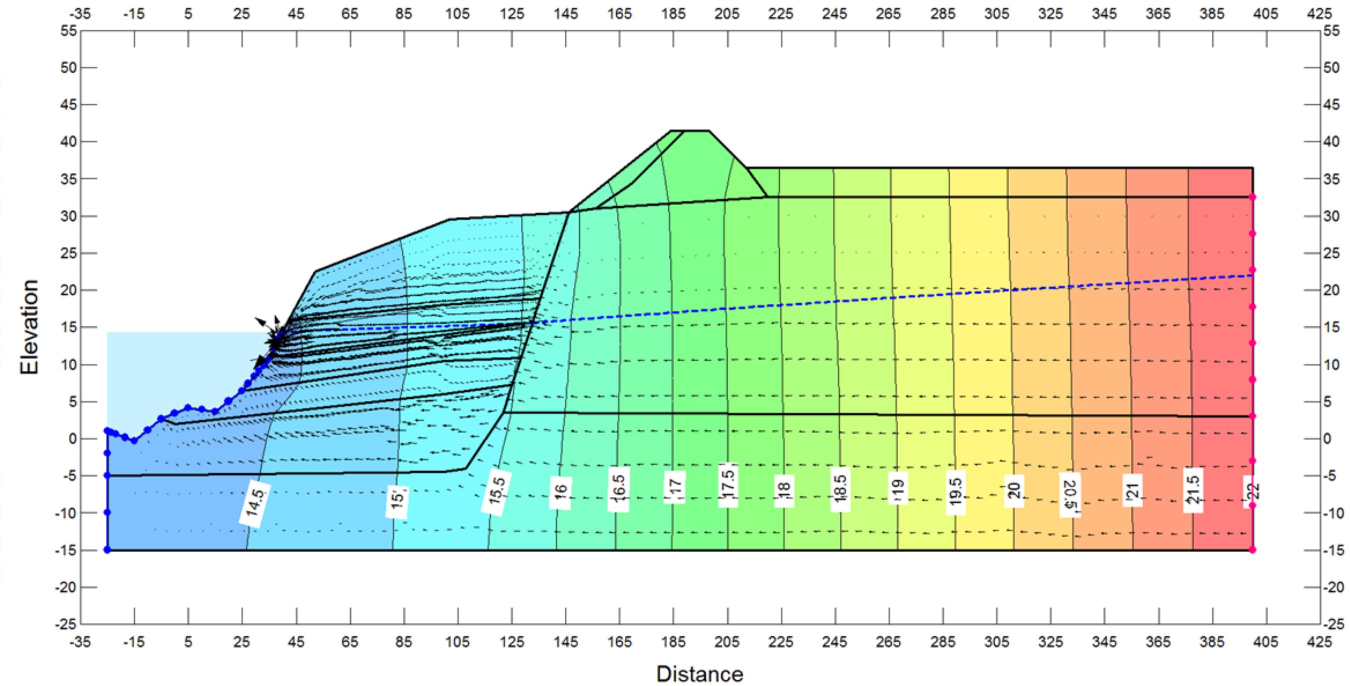
STABILITY MODEL

STEADY STATE FOS(WSE 7.77 FT)

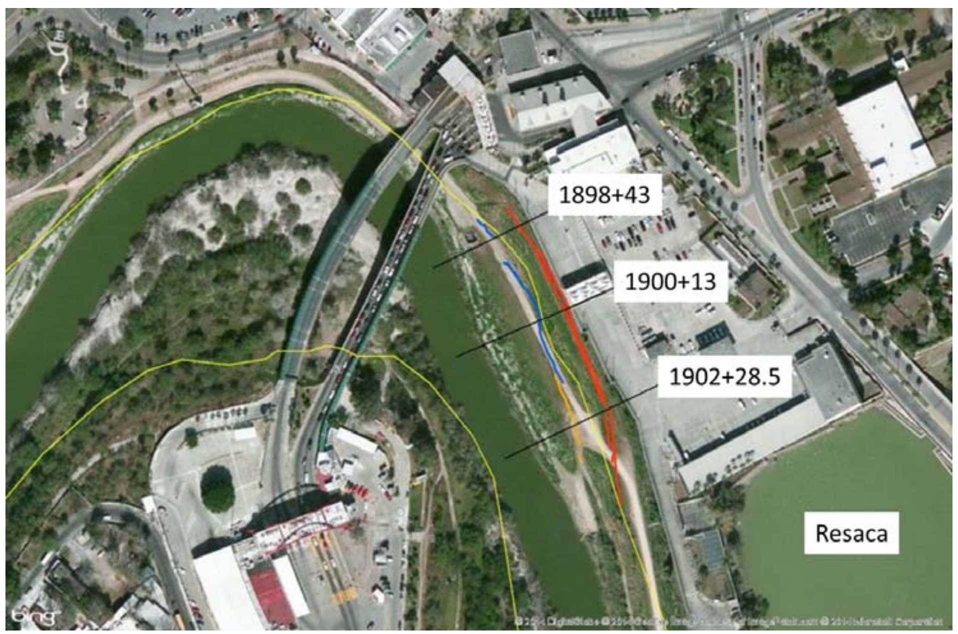
STATION 1898+43

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

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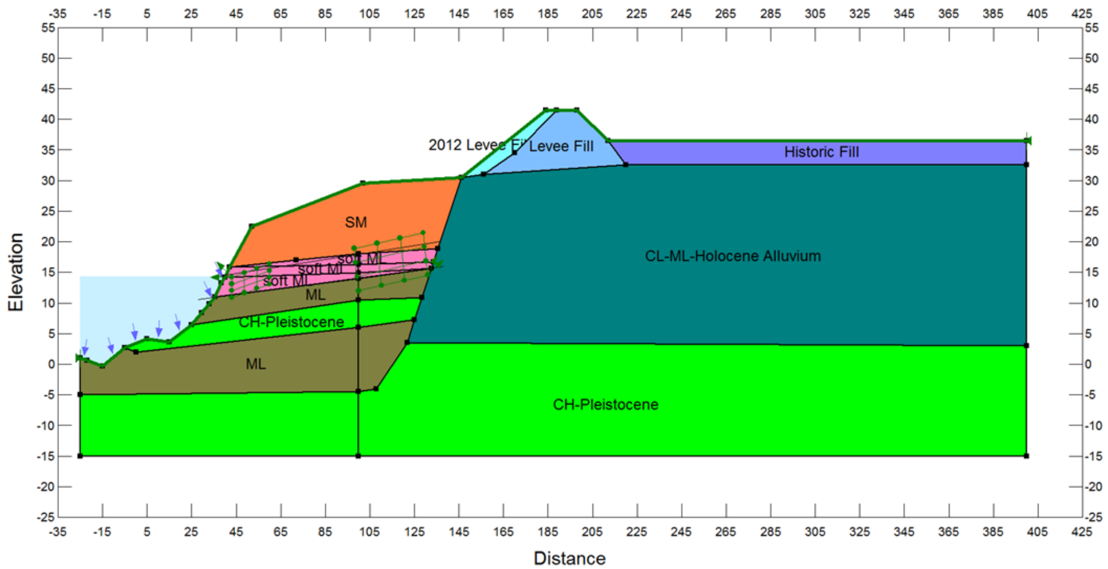
IBWC-BROWNSVILLE LEVEE

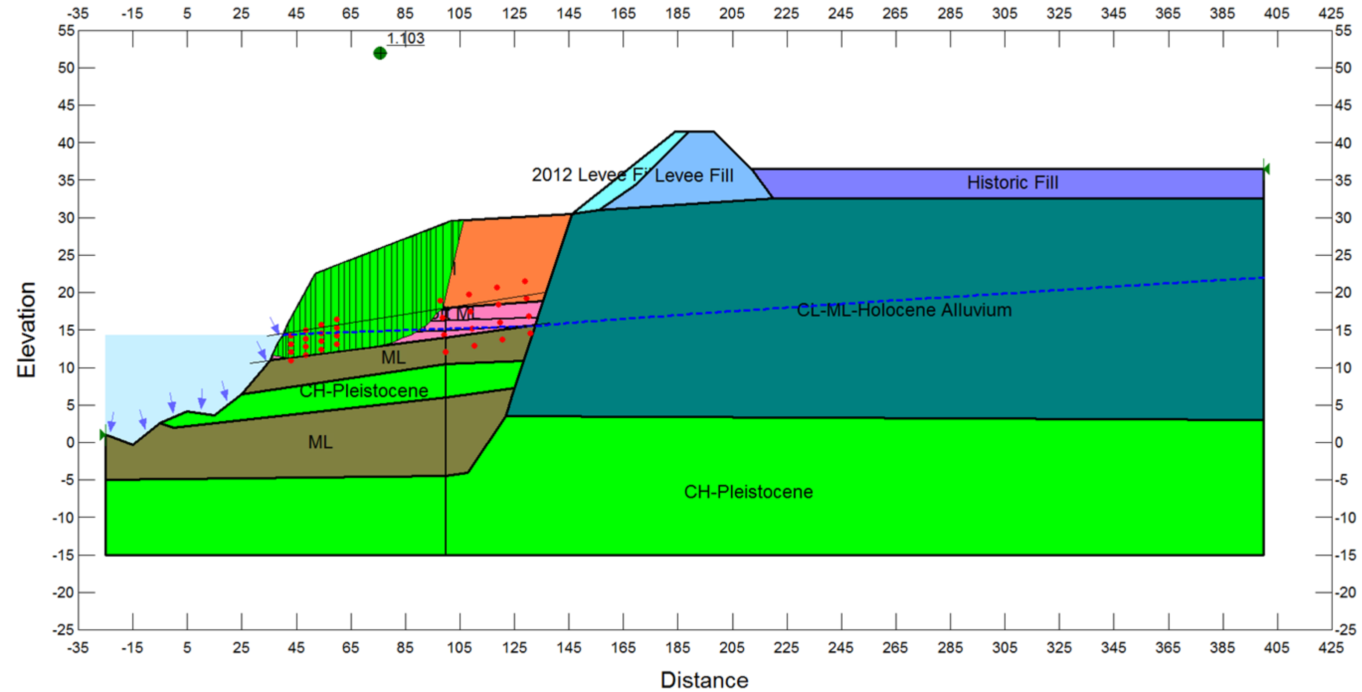
STEADY STATE SEEPAGE, SATURATED MODEL

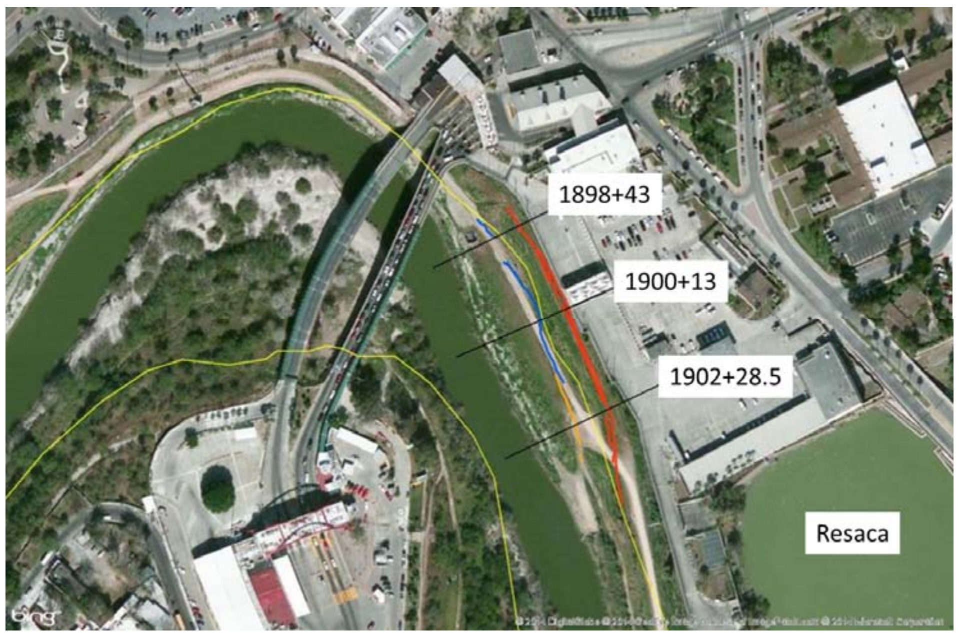
STEADY STATE SEEPAGE (WSE 14.31 FT)

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

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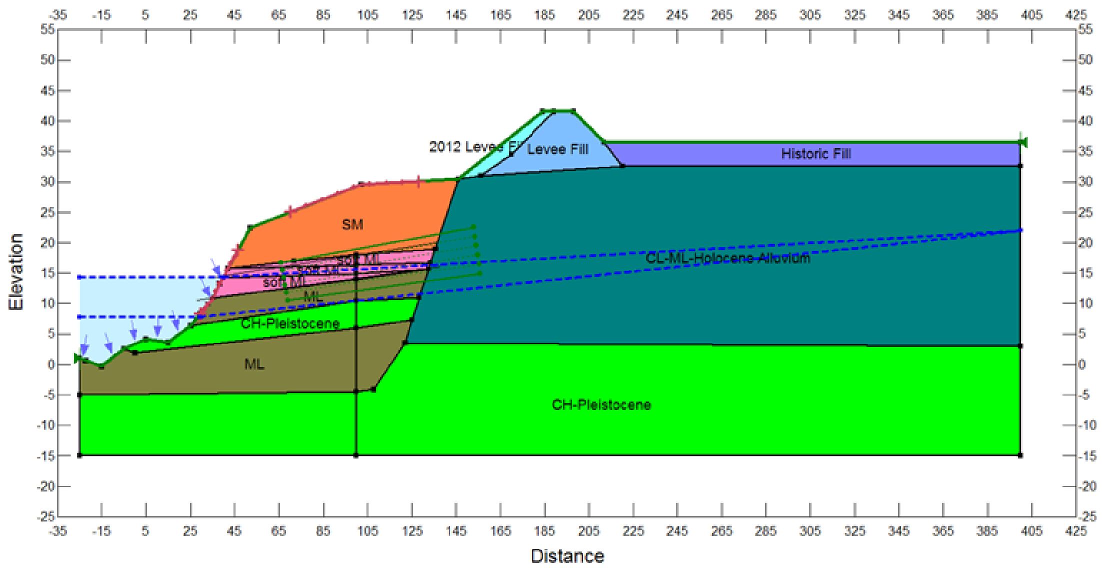
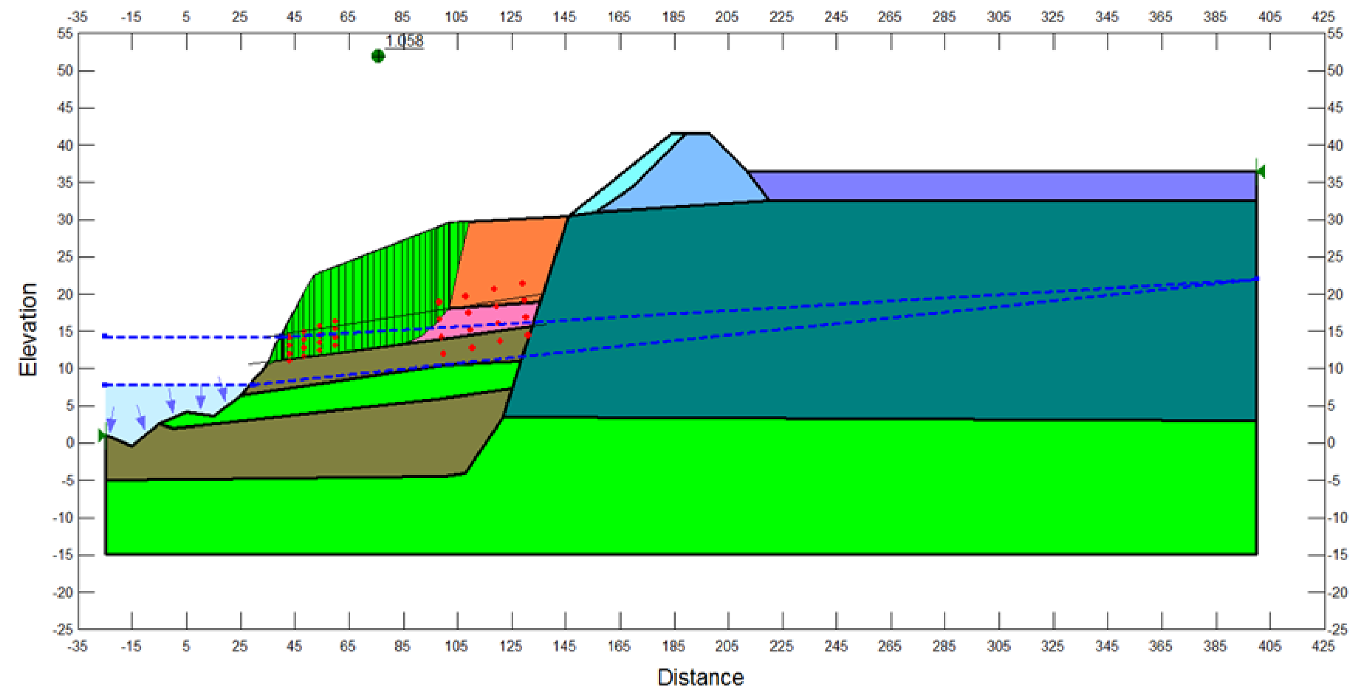
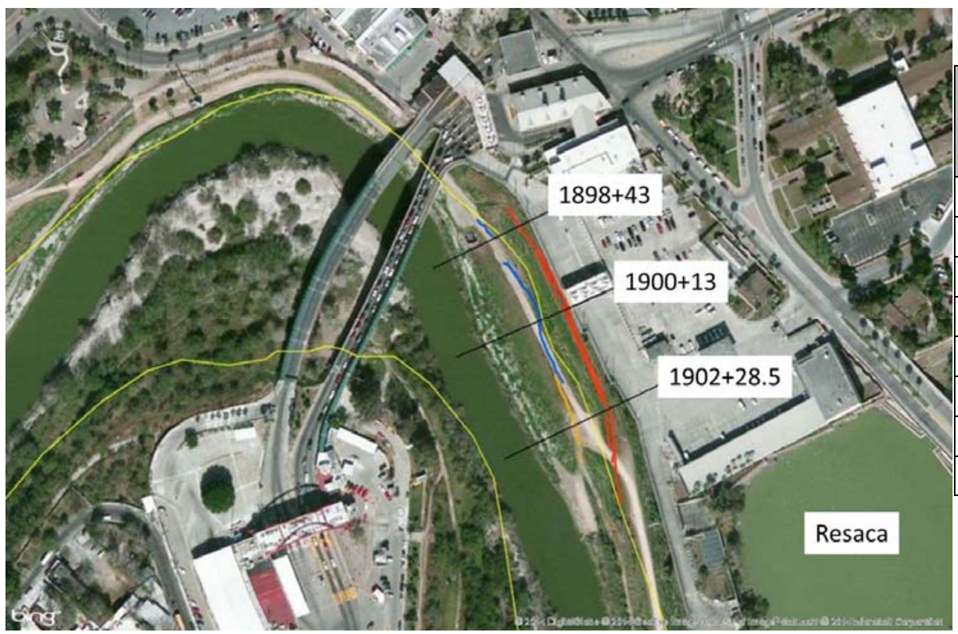
IBWC-BROWNSVILLE LEVEE
STABILITY MODEL

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

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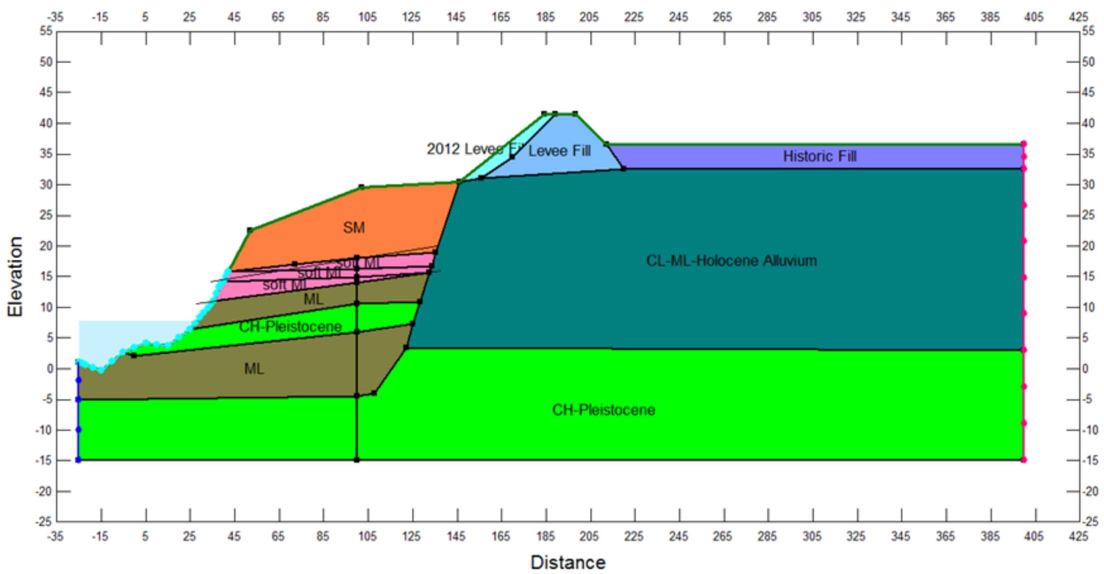
IBWC-BROWNSVILLE LEVEE

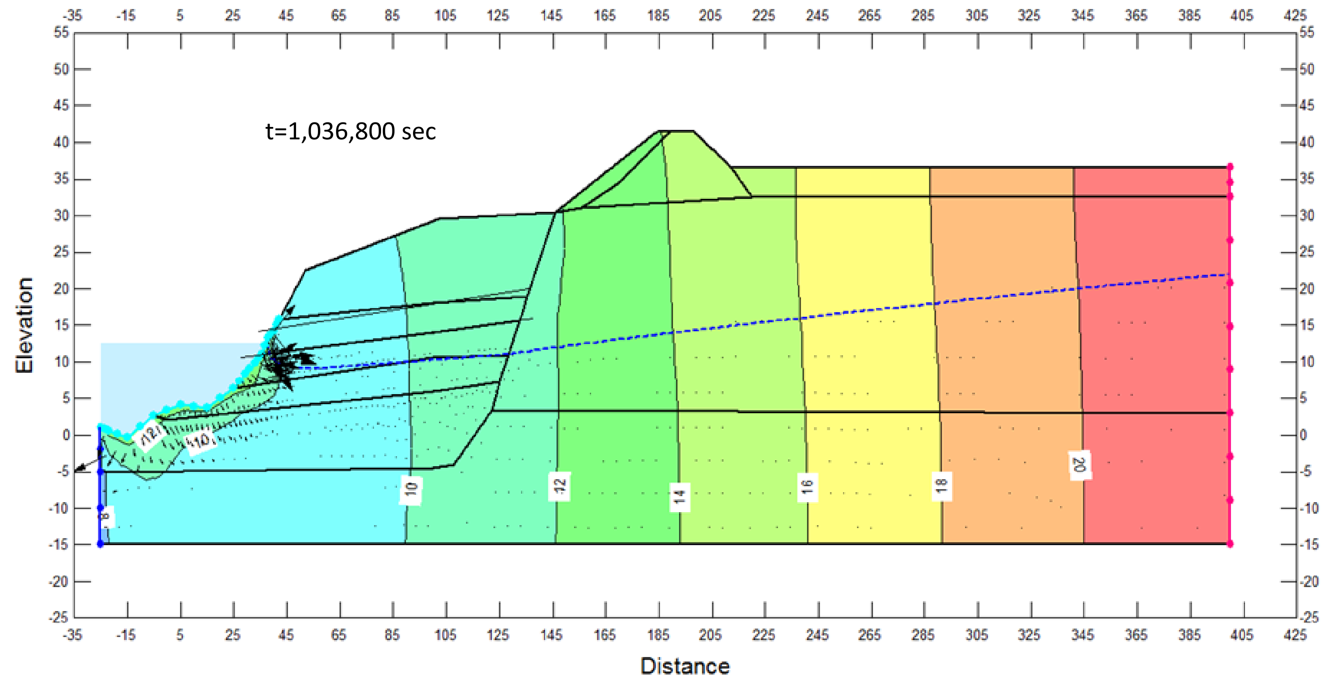
RAPID DRAWDOWN STABILITY

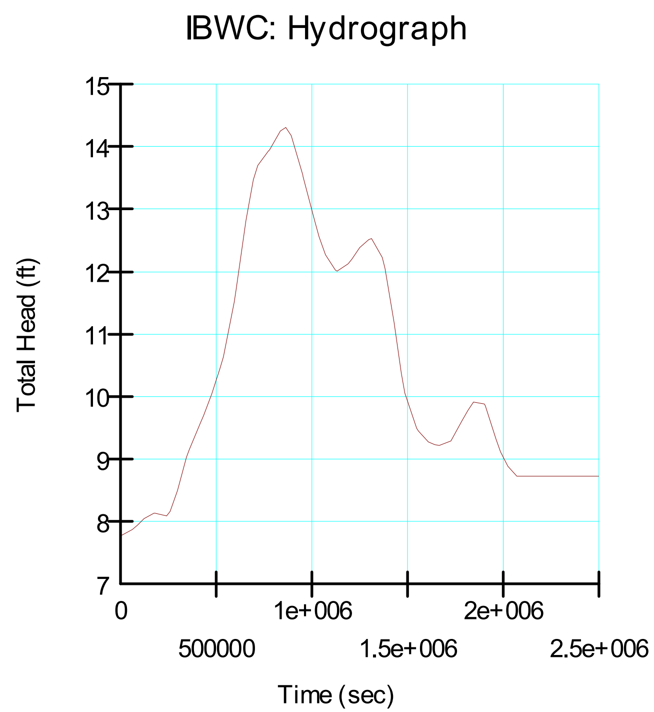
RAPID DRAWDOWN (WSE 7.77 & 14.31 FT)

STATION 1898+43

FEB-2015 **PLATE - 13**







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

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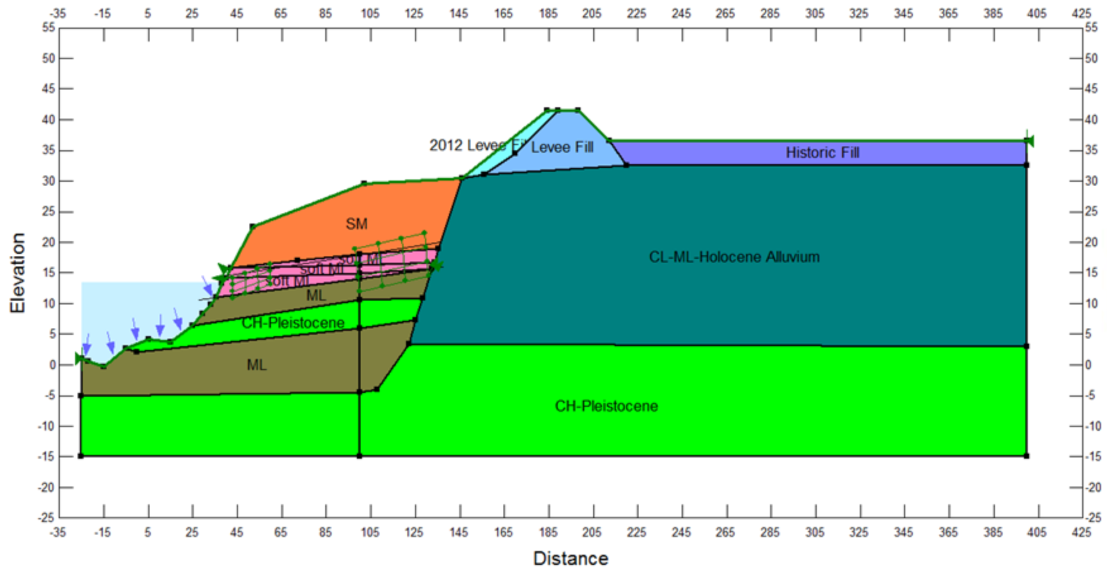
IBWC-BROWNSVILLE LEVEE

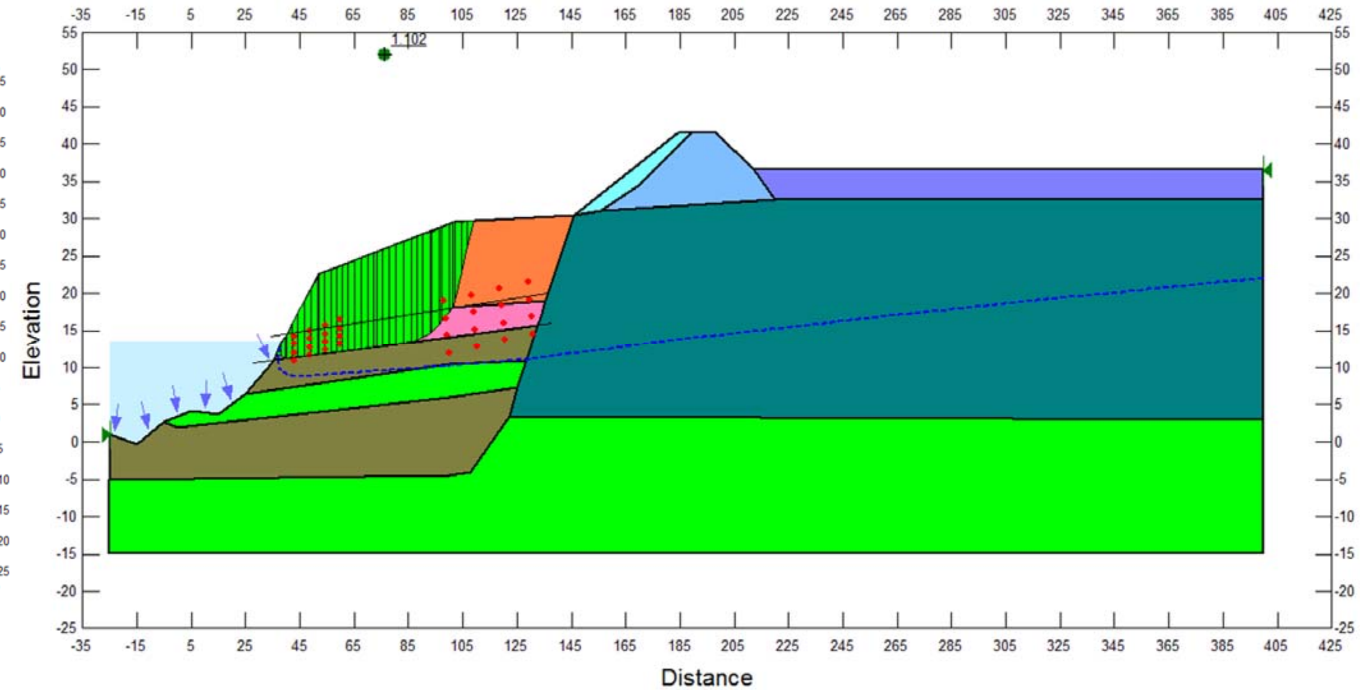
HYDROGRAPH, SATURATED MODEL

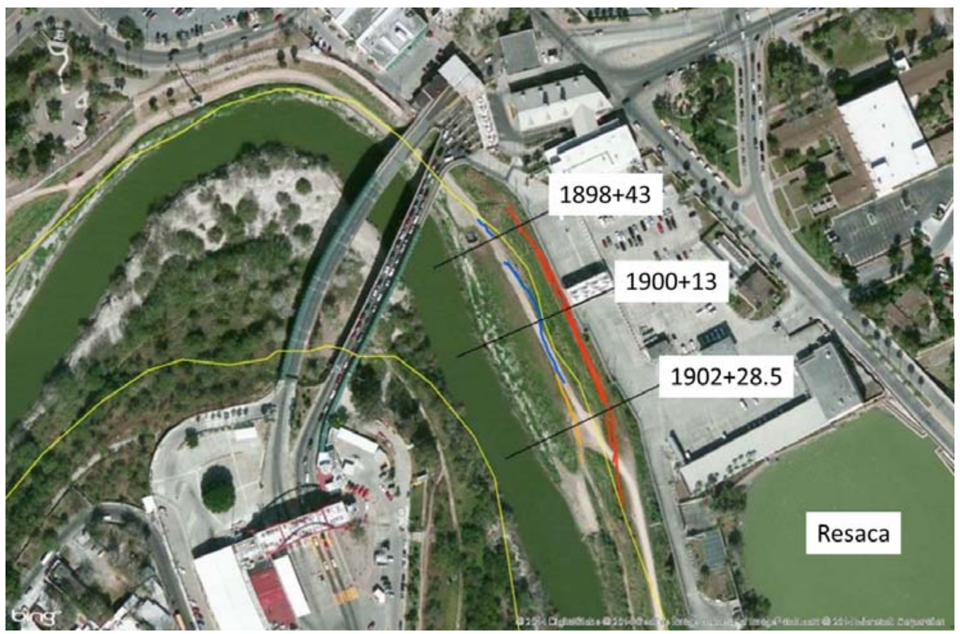
STEADY STATE SEEPAGE (WSE 14.31 FT)

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

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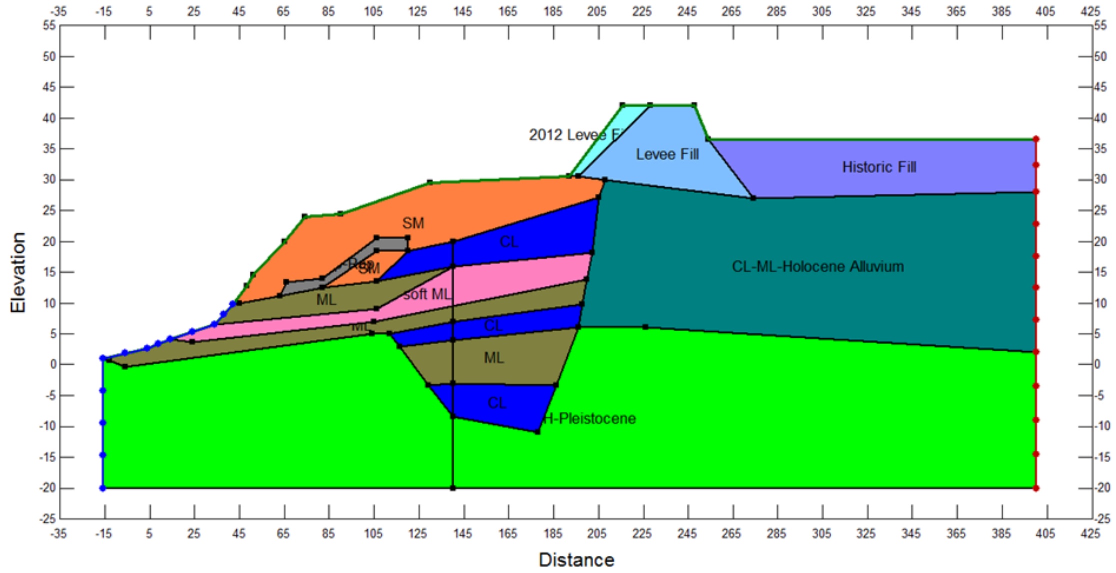
IBWC-BROWNSVILLE LEVEE

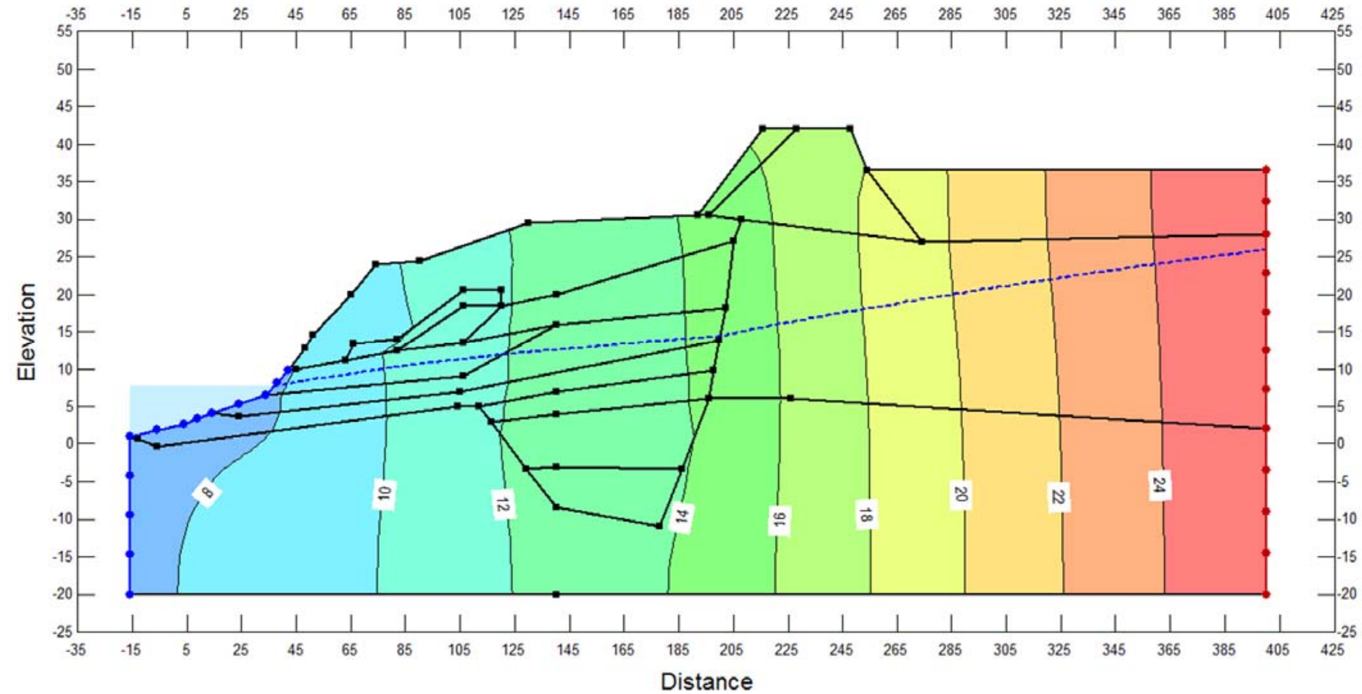
STABILITY MODEL

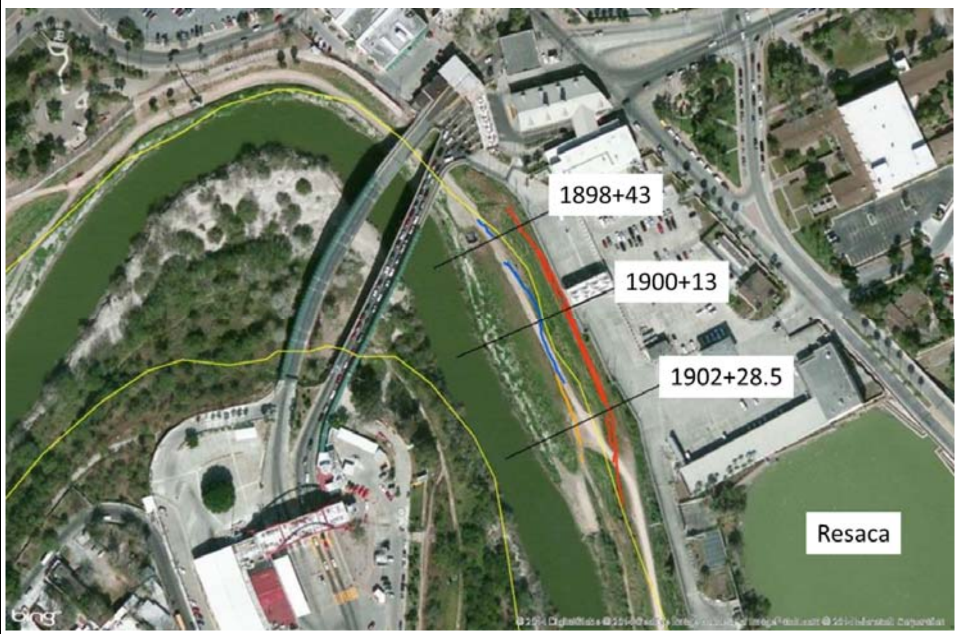
TRANSIENT FOS (HYDROGRAPH)

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

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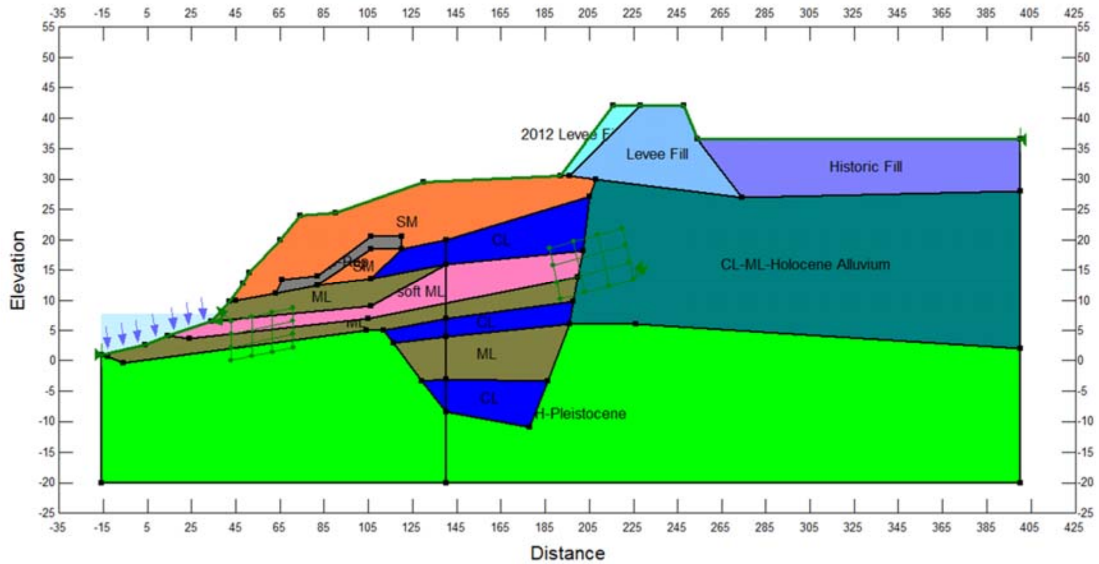
IBWC-BROWNSVILLE LEVEE

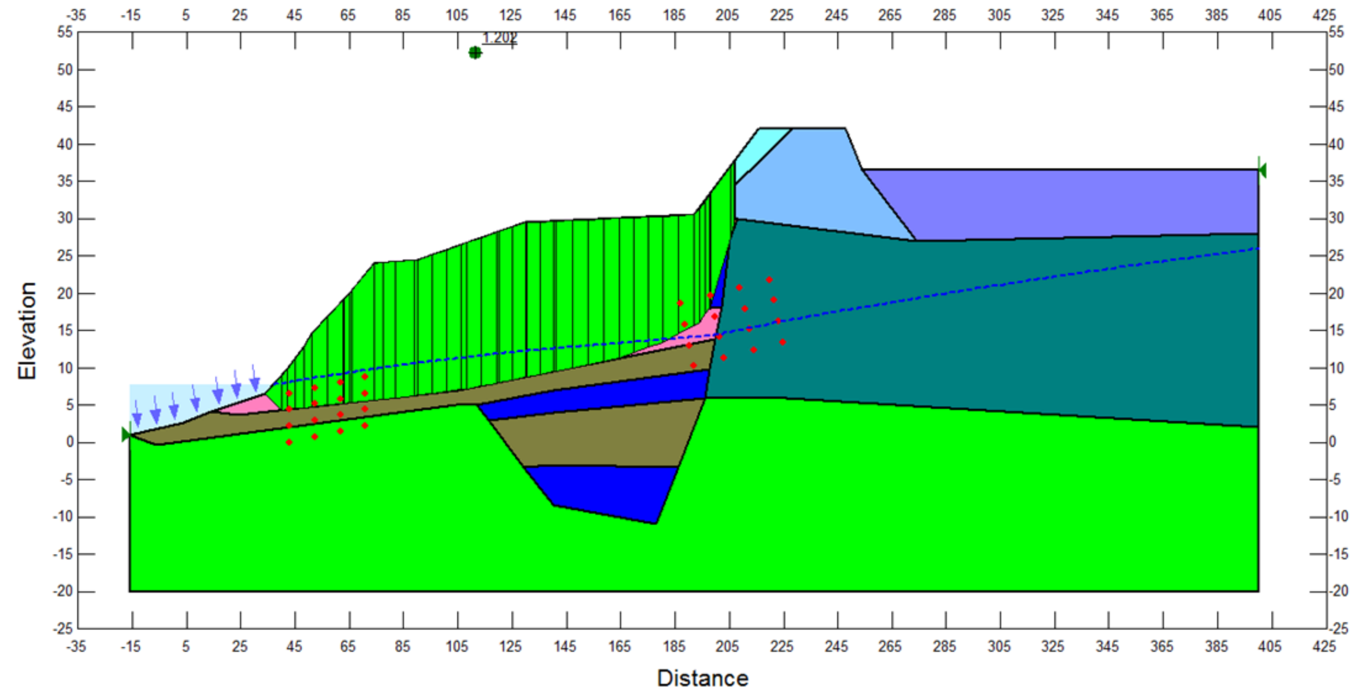
STEADY STATE SEEPAGE, SATURATED MODEL

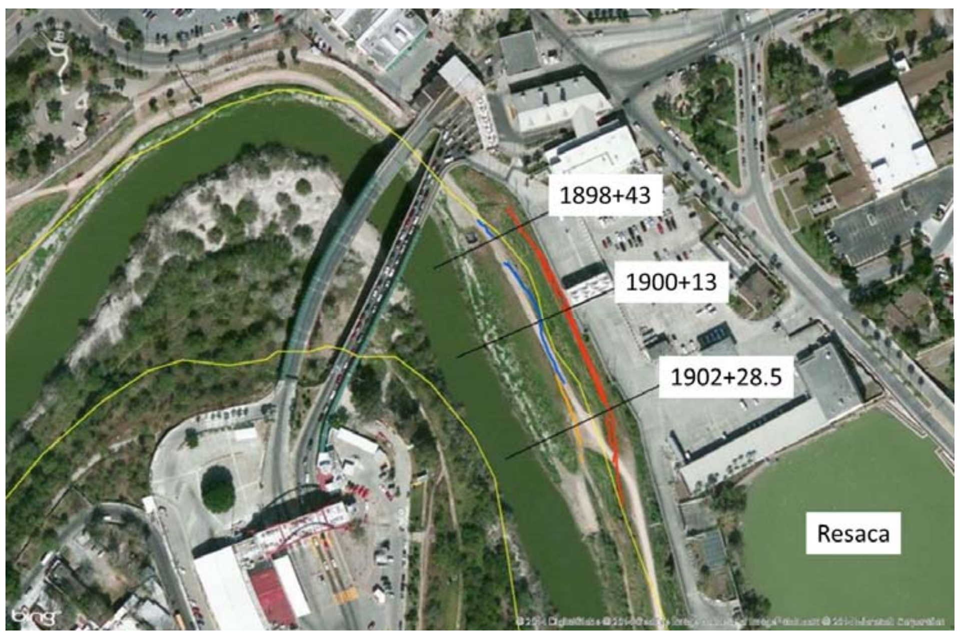
STEADY STATE SEEPAGE (WSE 7.77 FT)

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

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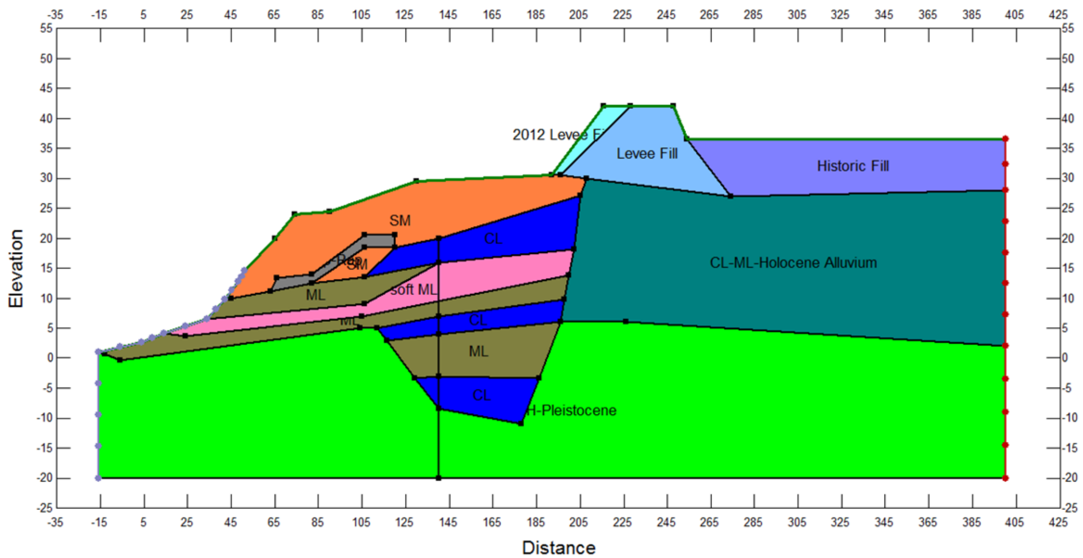
IBWC-BROWNSVILLE LEVEE

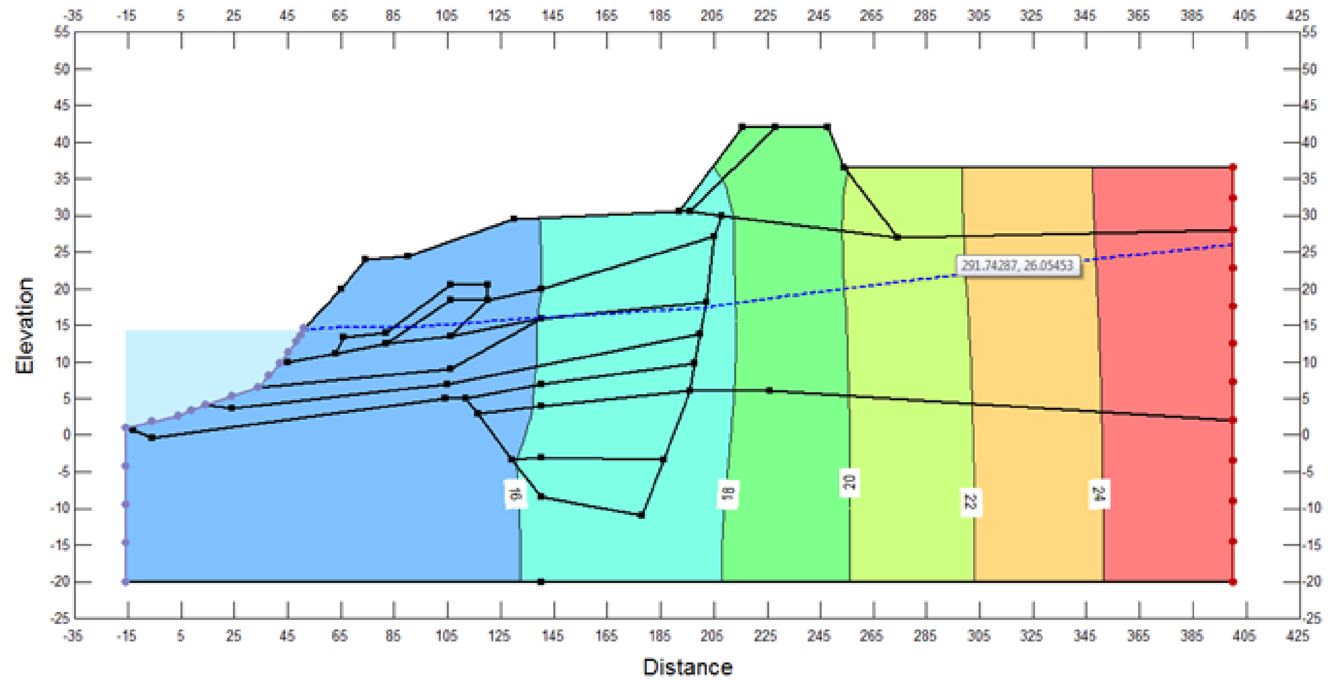
STABILITY MODEL

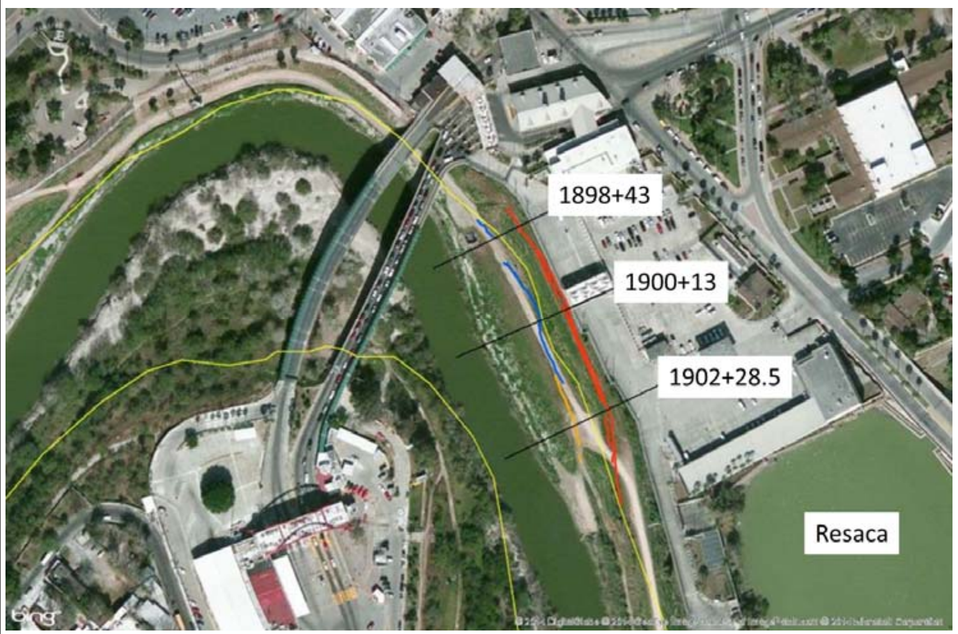
STEADY STATE FOS(WSE 7.77 FT)

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

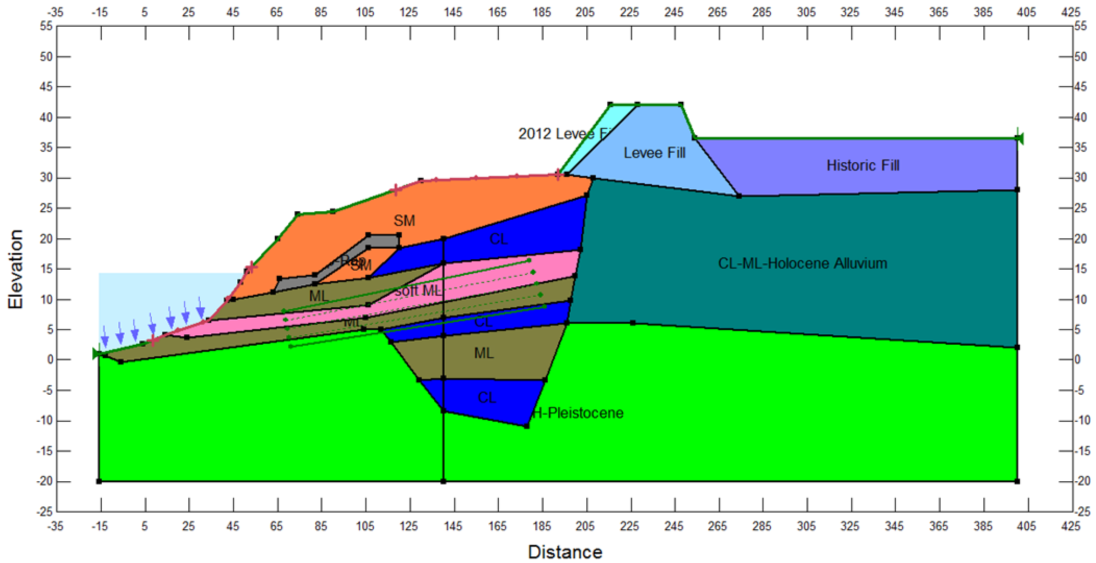
IBWC-BROWNSVILLE LEVEE

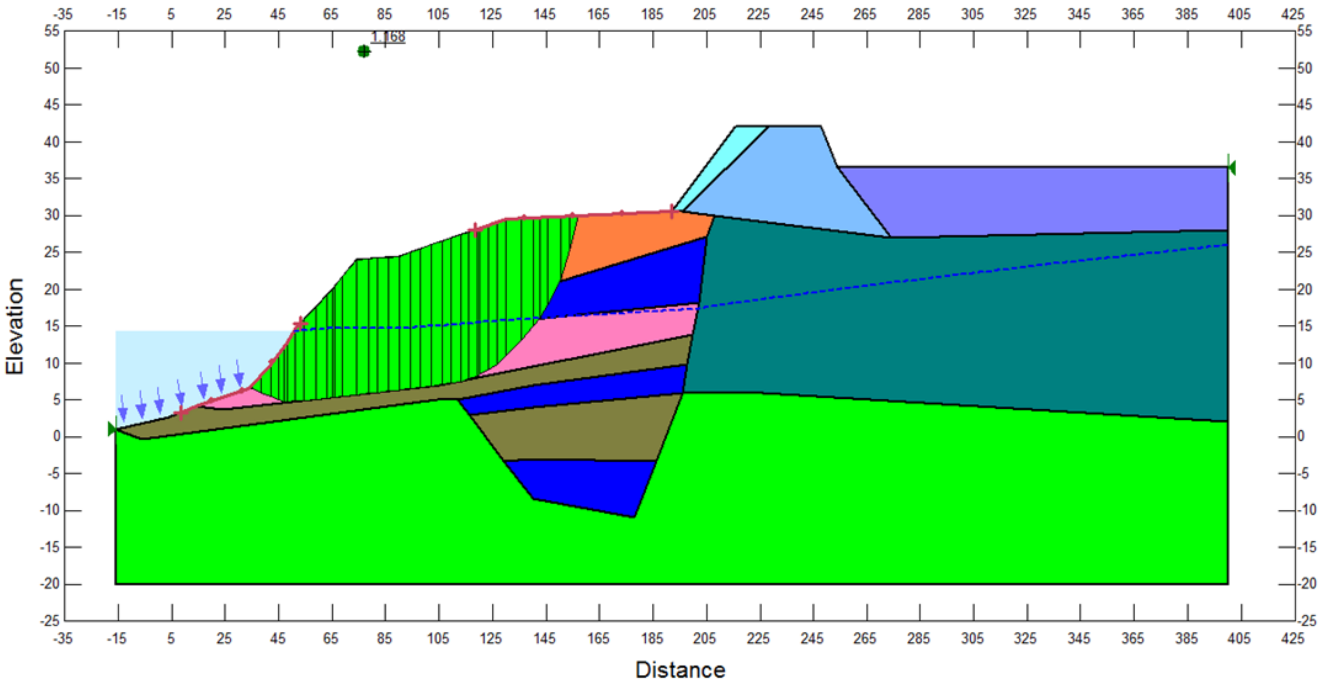
STEADY STATE SEEPAGE, SATURATED MODEL

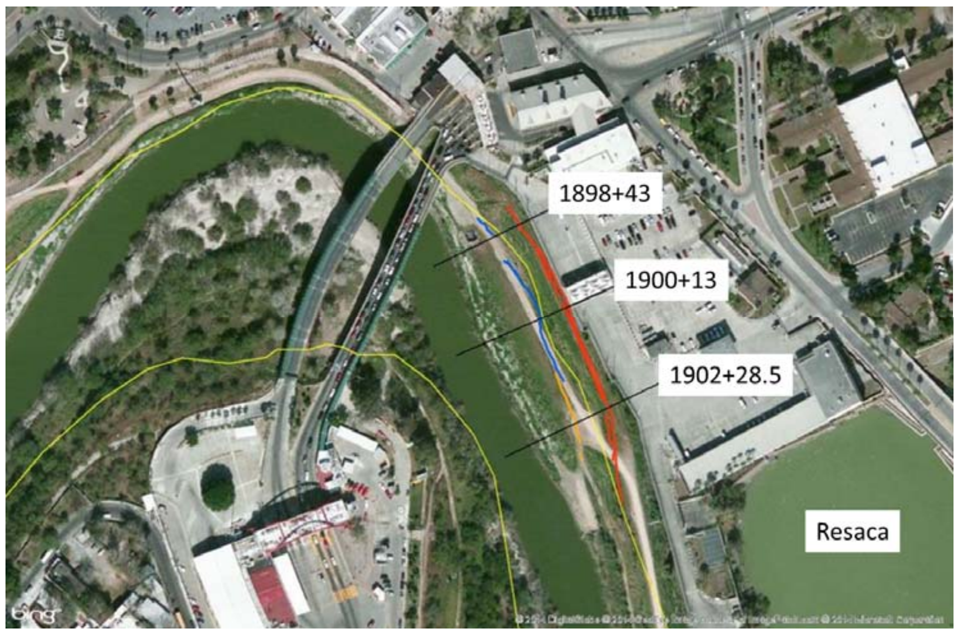
STEADY STATE SEEPAGE (WSE 14.31 FT)

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

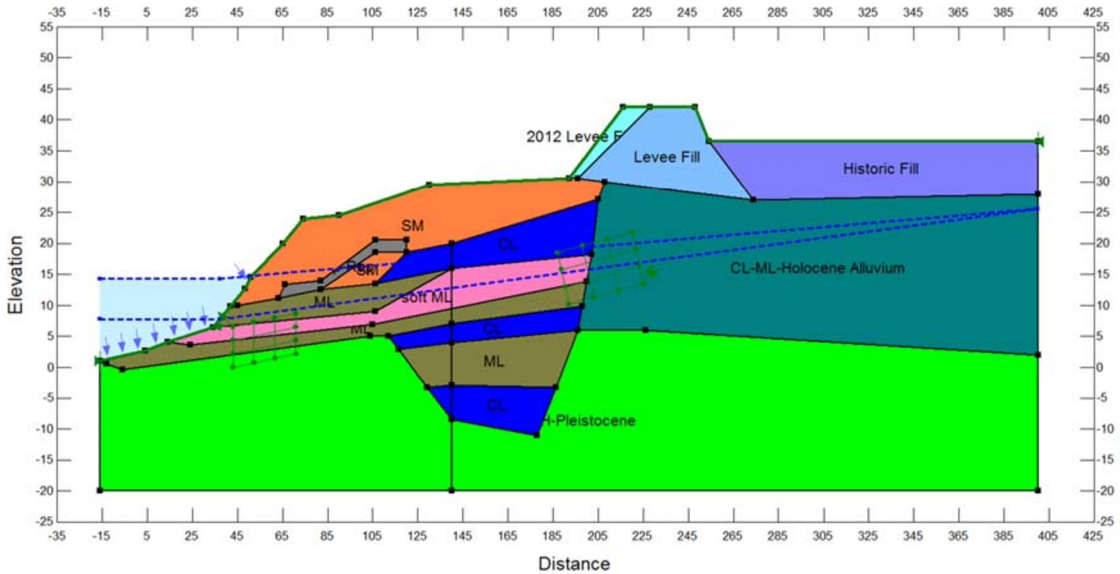
IBWC-BROWNSVILLE LEVEE

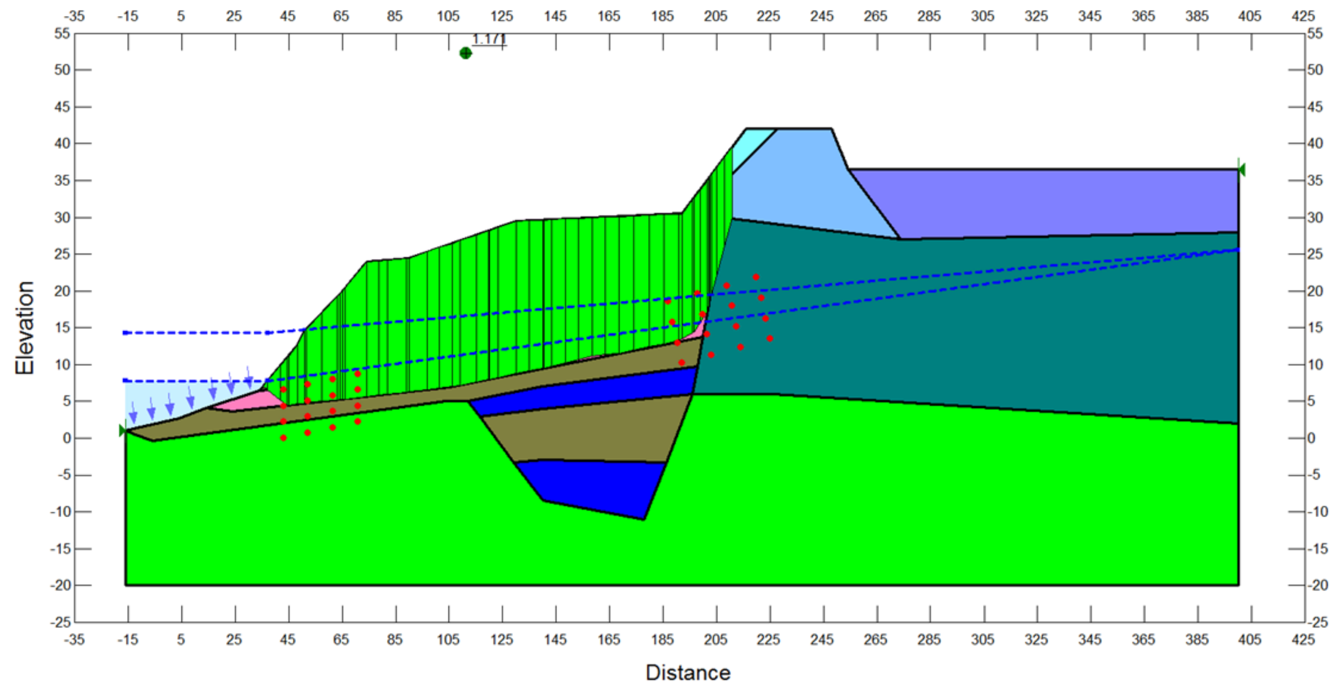
STABILITY MODEL

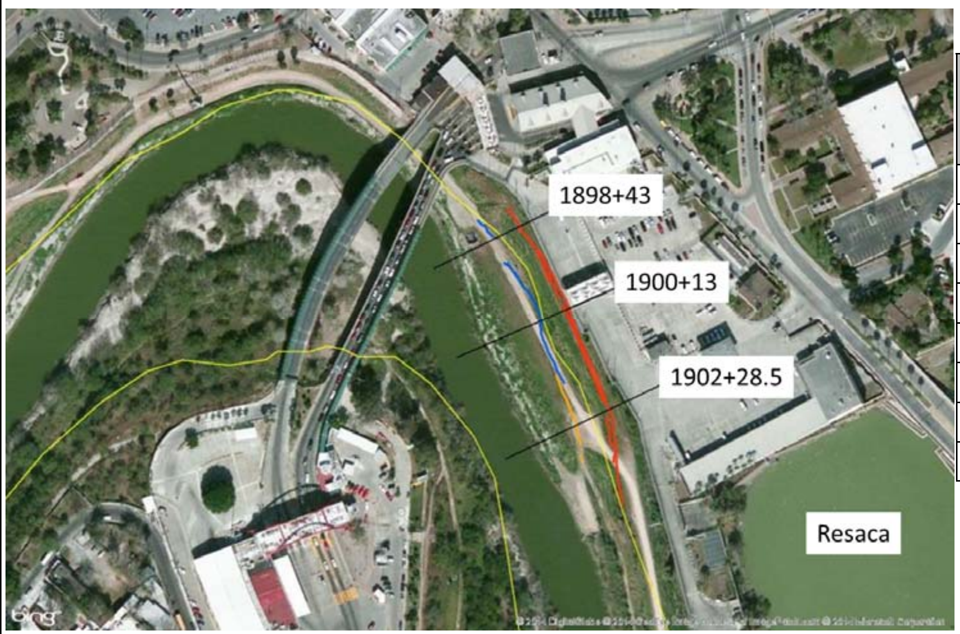
STEADY STATE FOS(WSE 14.31 FT)

STATION 1902+28.5

FEB-2015 **PLATE - 19**







				total stress	
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

Minimum factor of safety (FoS): 1.17

U.S. ARMY CORPS OF ENGINEERS

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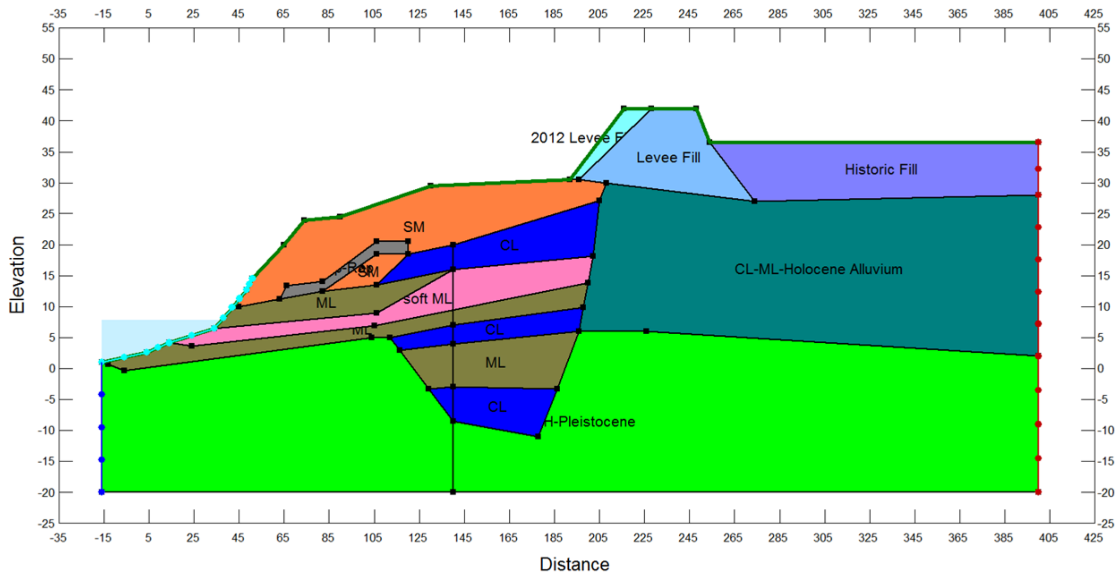
IBWC-BROWNSVILLE LEVEE

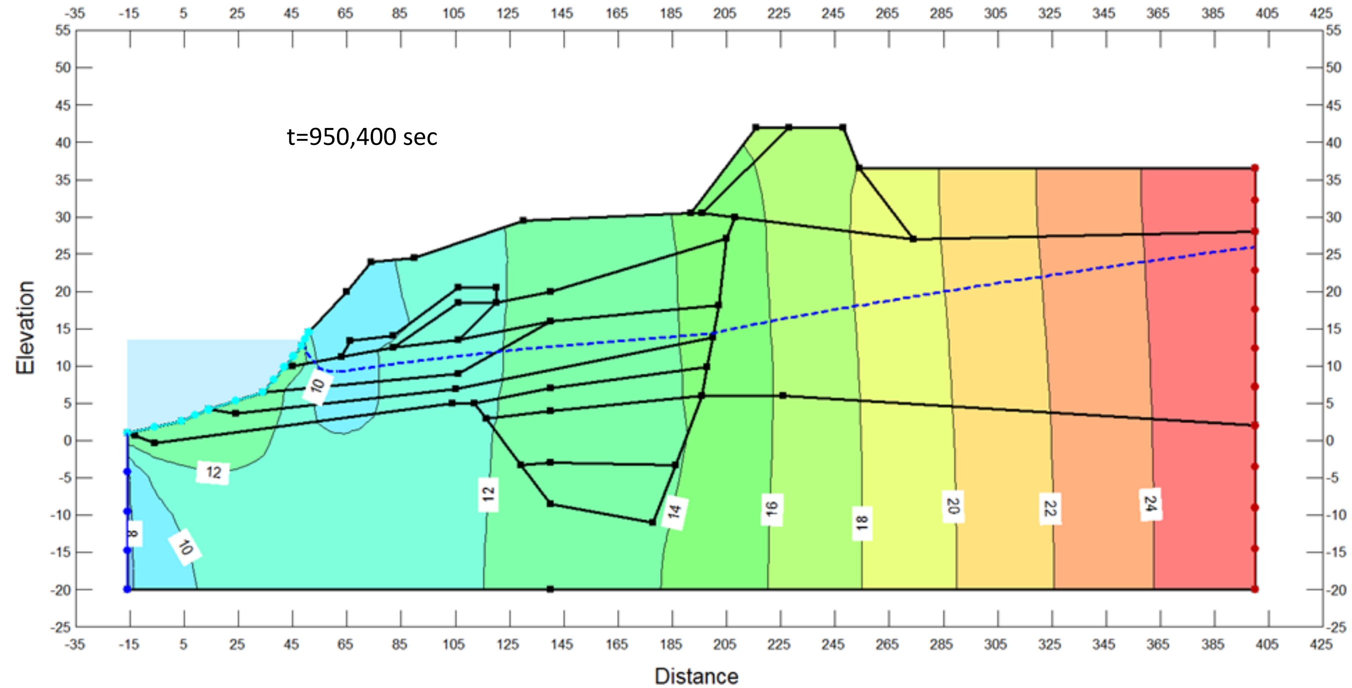
RAPID DRAWDOWN STABILITY

RAPID DRAWDOWN (WSE 7.77 & 14.31 FT)

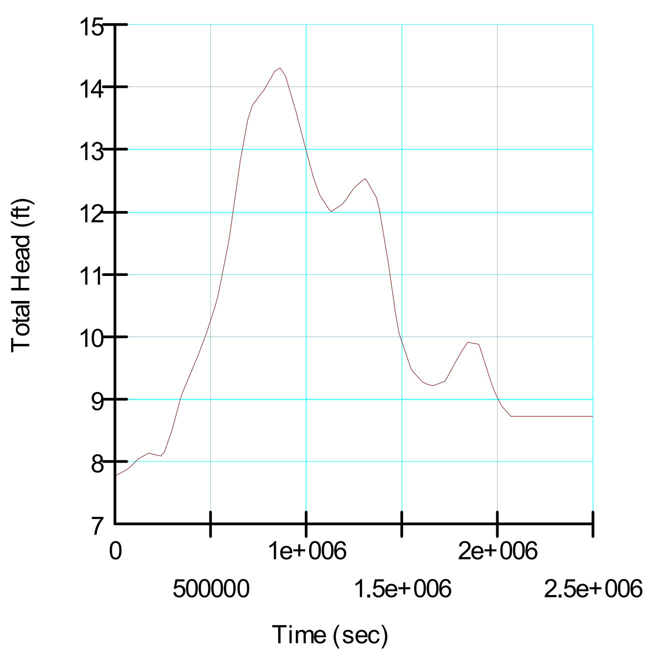
STATION 1902+28.5

FEB-2015 **PLATE - 20**





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
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)

U.S. ARMY CORPS OF ENGINEERS

ERDC-GSL

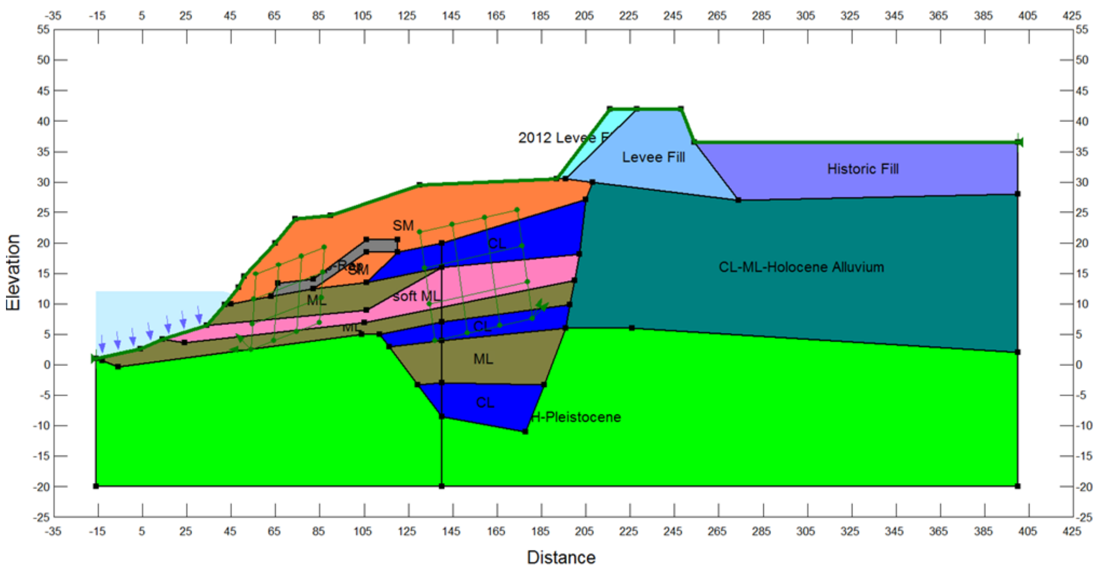
IBWC-BROWNSVILLE LEVEE

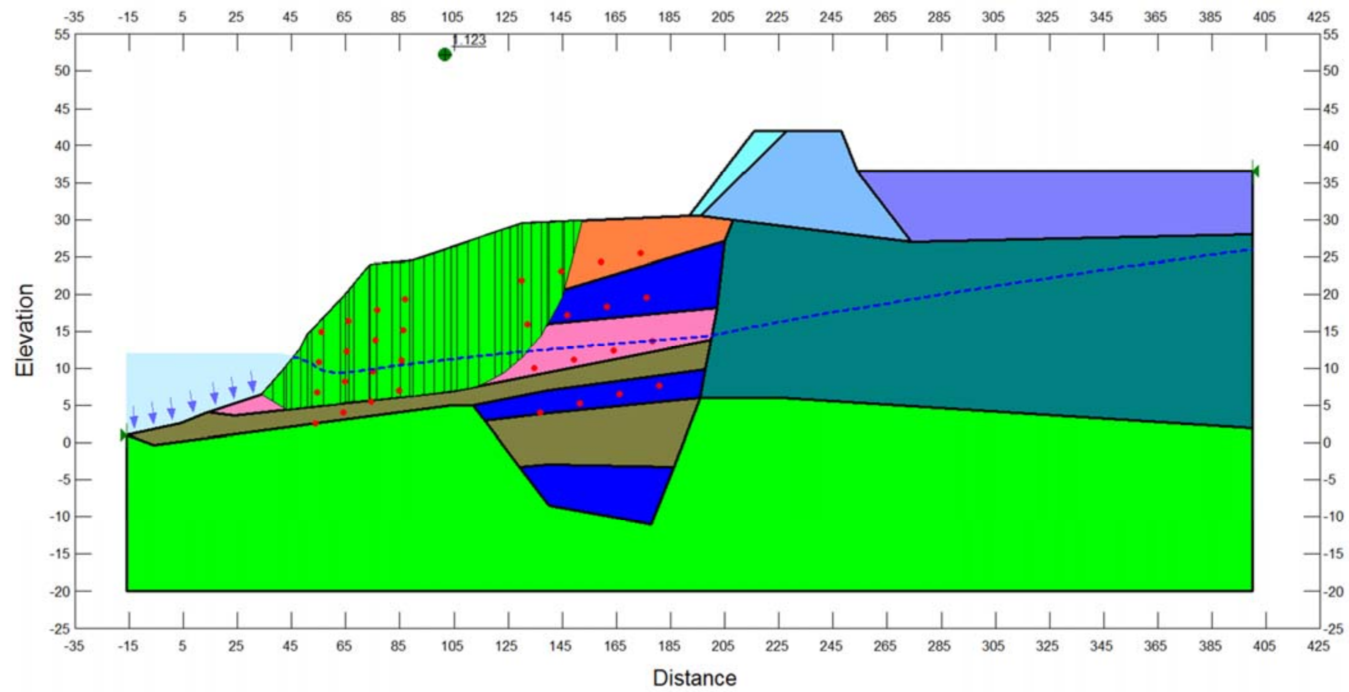
HYDROGRAPH, SATURATED MODEL

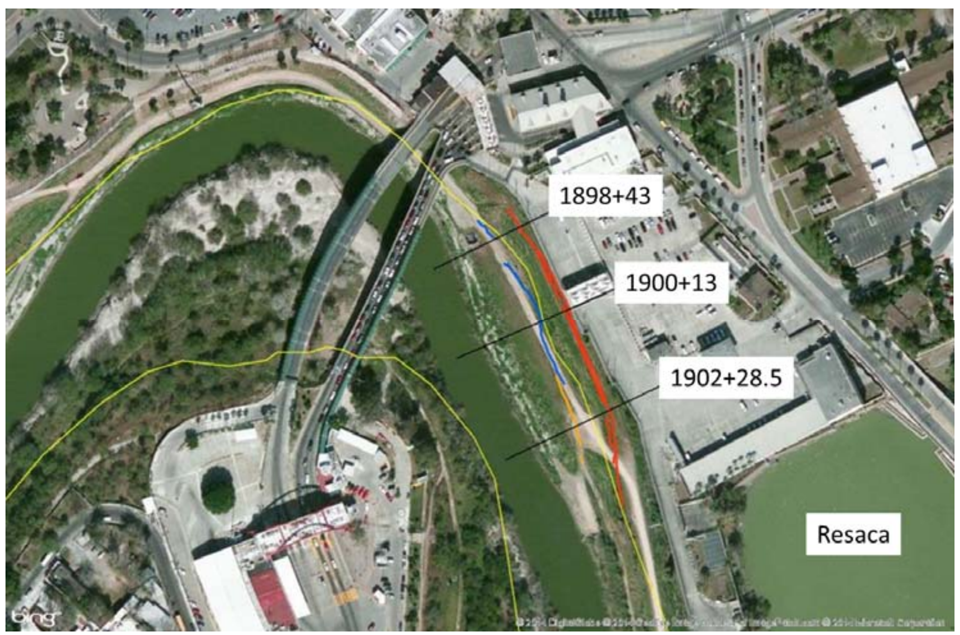
STEADY STATE SEEPAGE (WSE 14.31 FT)

STATION 1902+28.5

FEB-2015 **PLATE - 21**







Minimum factor of safety (FoS): 1.12

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

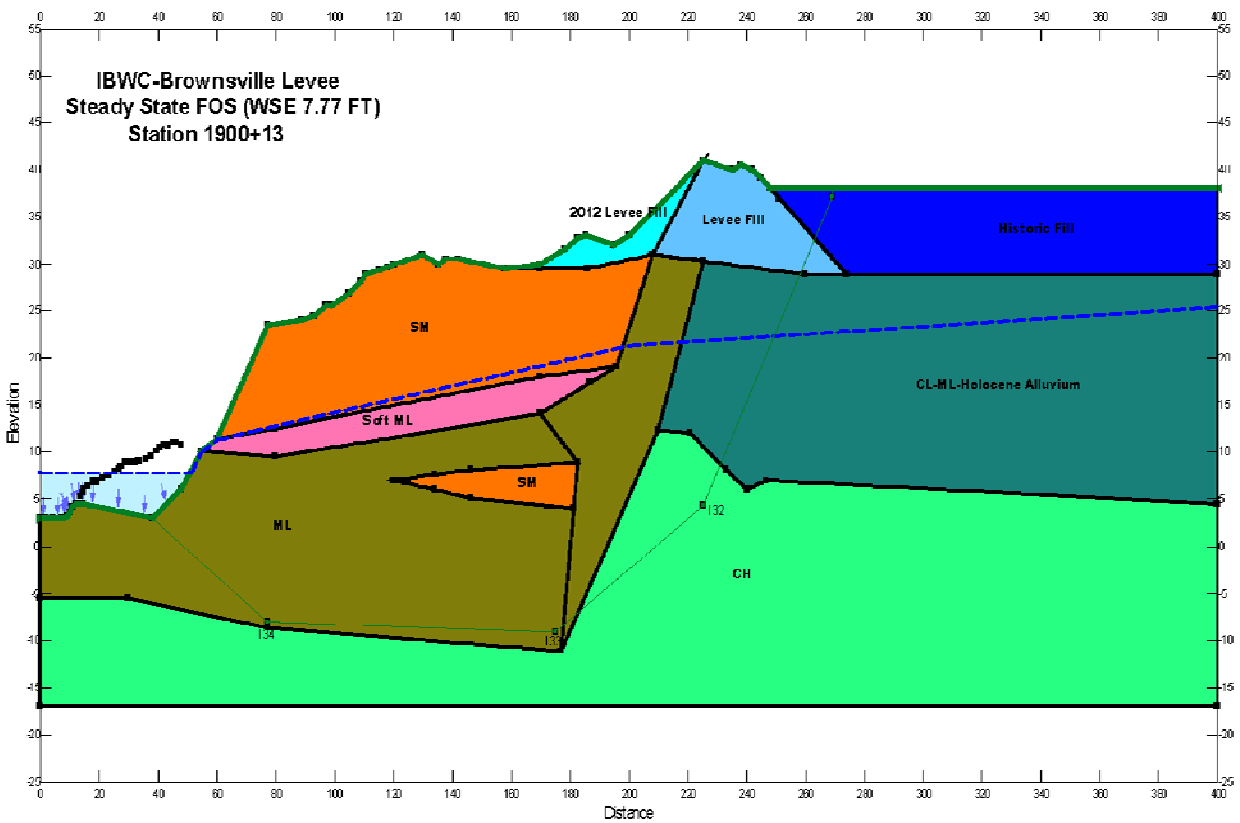
TRANSIENT FOS (HYDROGRAPH)

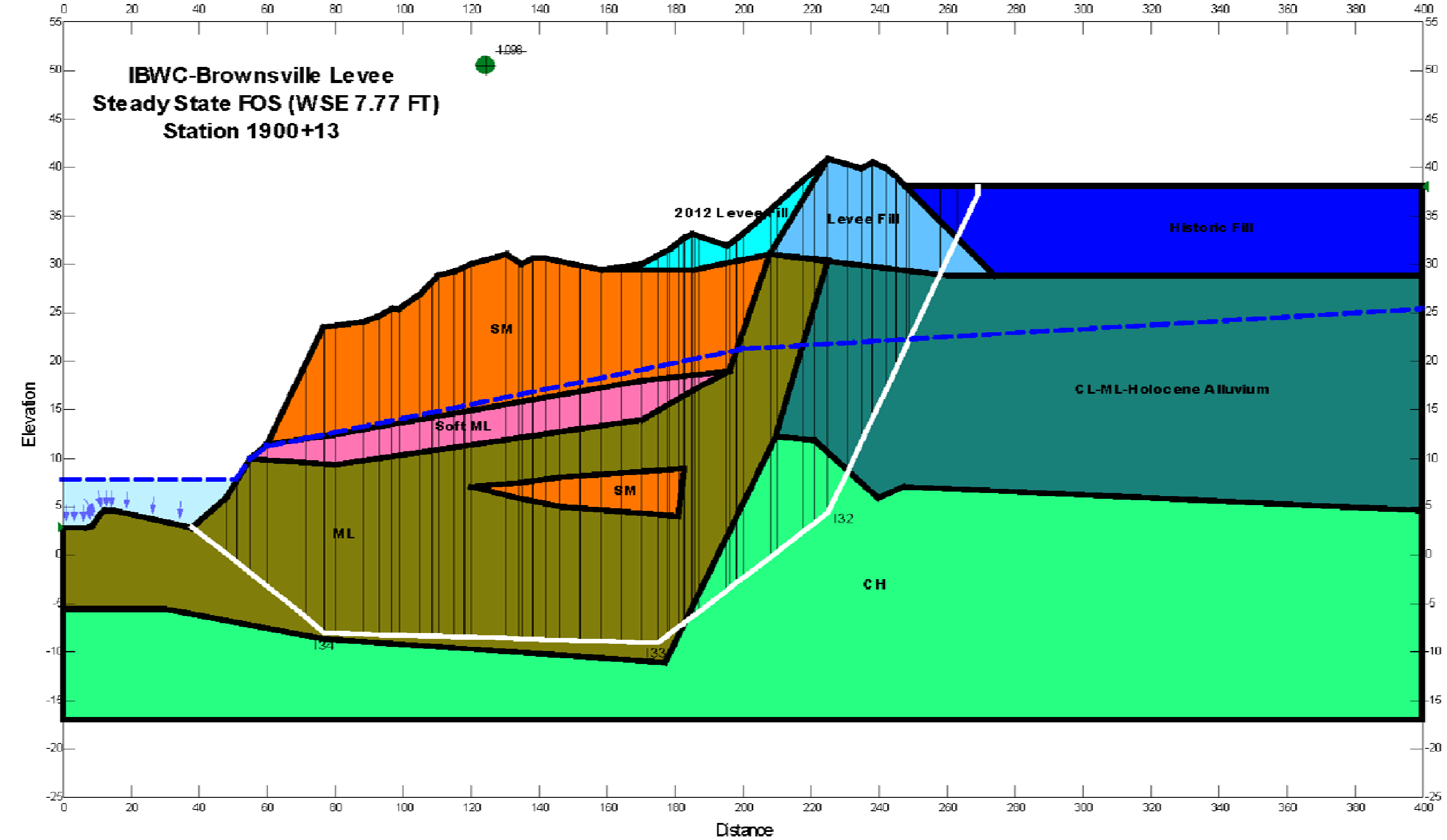
STATION 1902+28.5

FEB-2015 **PLATE - 22**

APPENDIX G

Assessment of USACE Alternative 1 Remediation and Back Calculation of Shear Strength Properties at STA 1900+13





Minimum Factor of Safety (FOS): 1.098

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1			

NOT TO SCALE

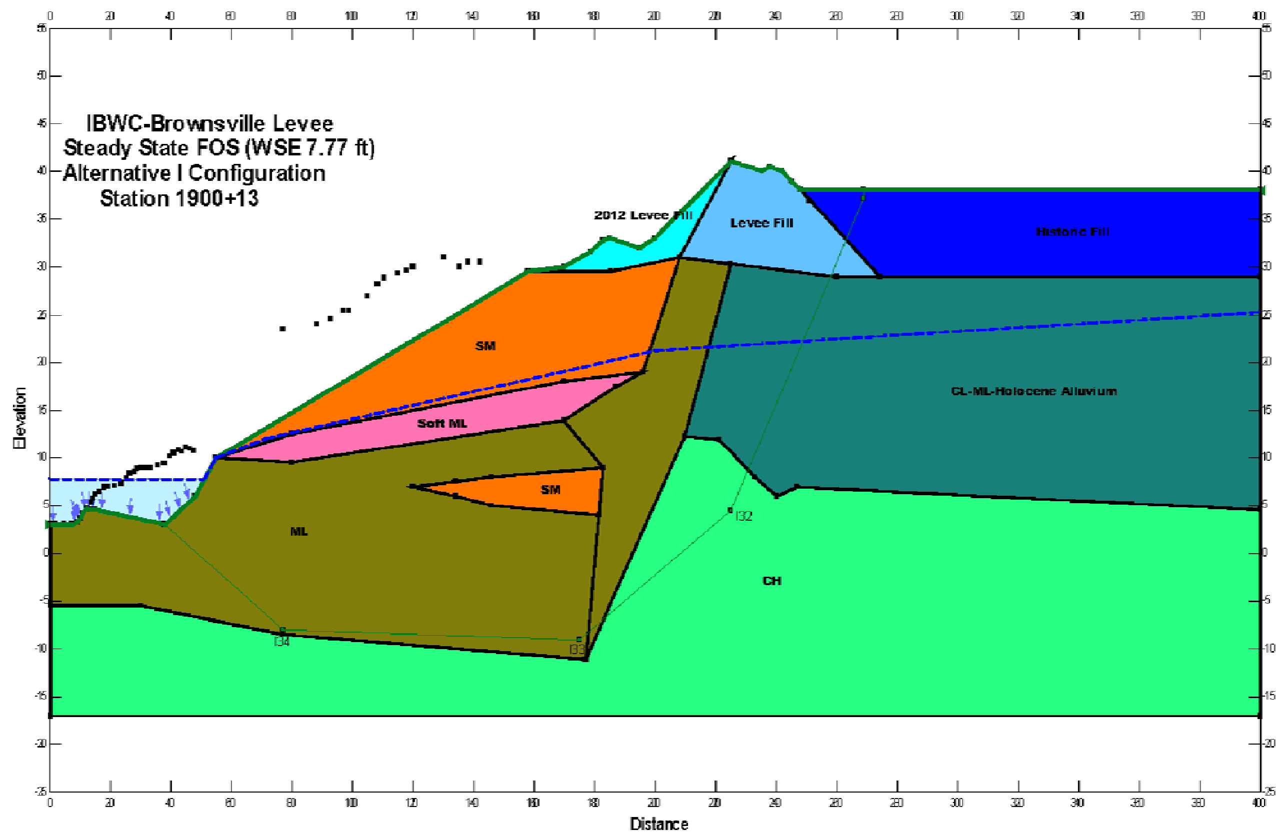
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

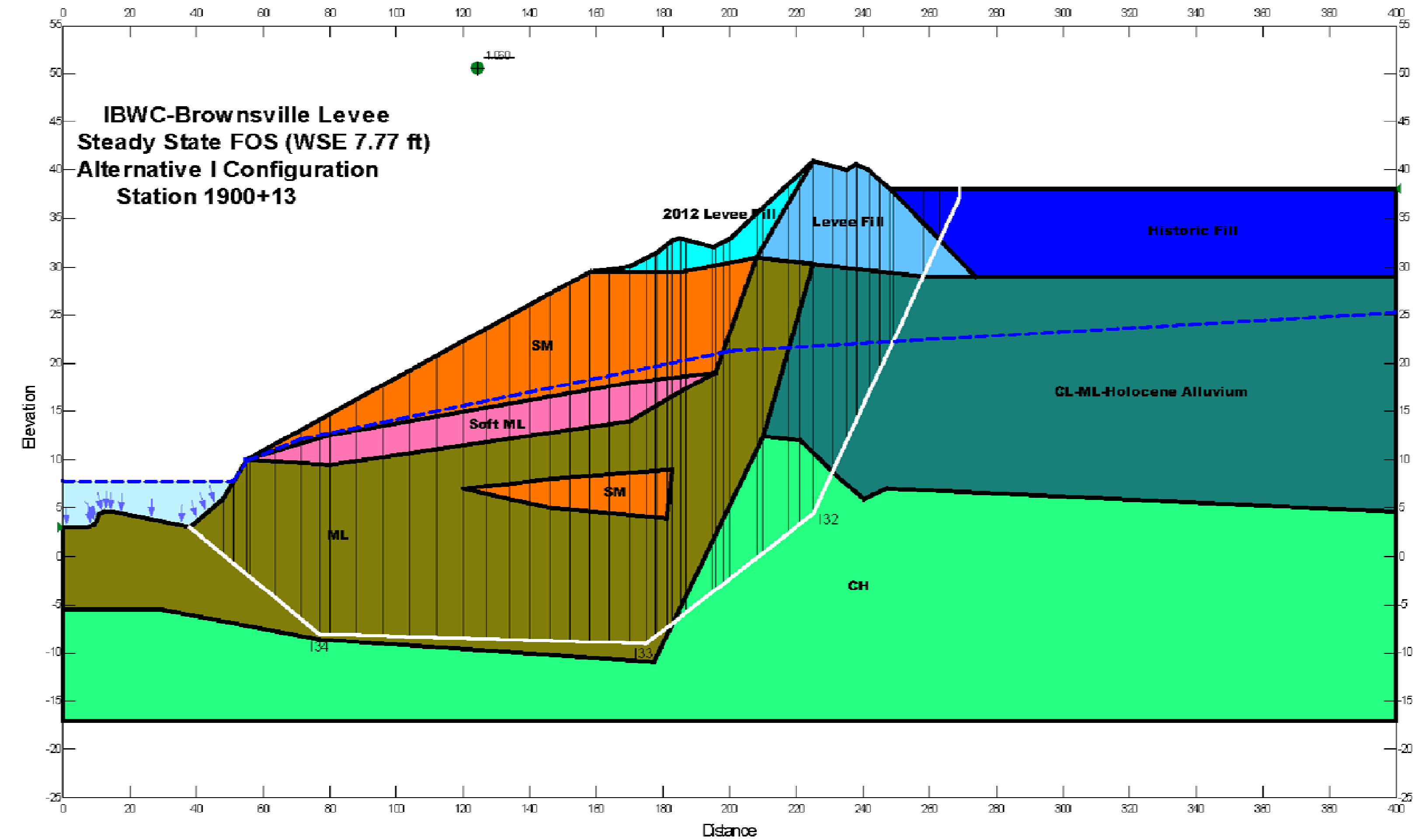
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - EXISTING SLOPE
STEADY STATE SEEPAGE

ARCADIS

SS-1





Minimum Factor of Safety (FOS): 1.060

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
SM	117	0	32
ML	119.38	230	0
2012 Levee Fill	127.34	300	12
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Soft ML	125.98	150	0
Combination 1			

NOT TO SCALE

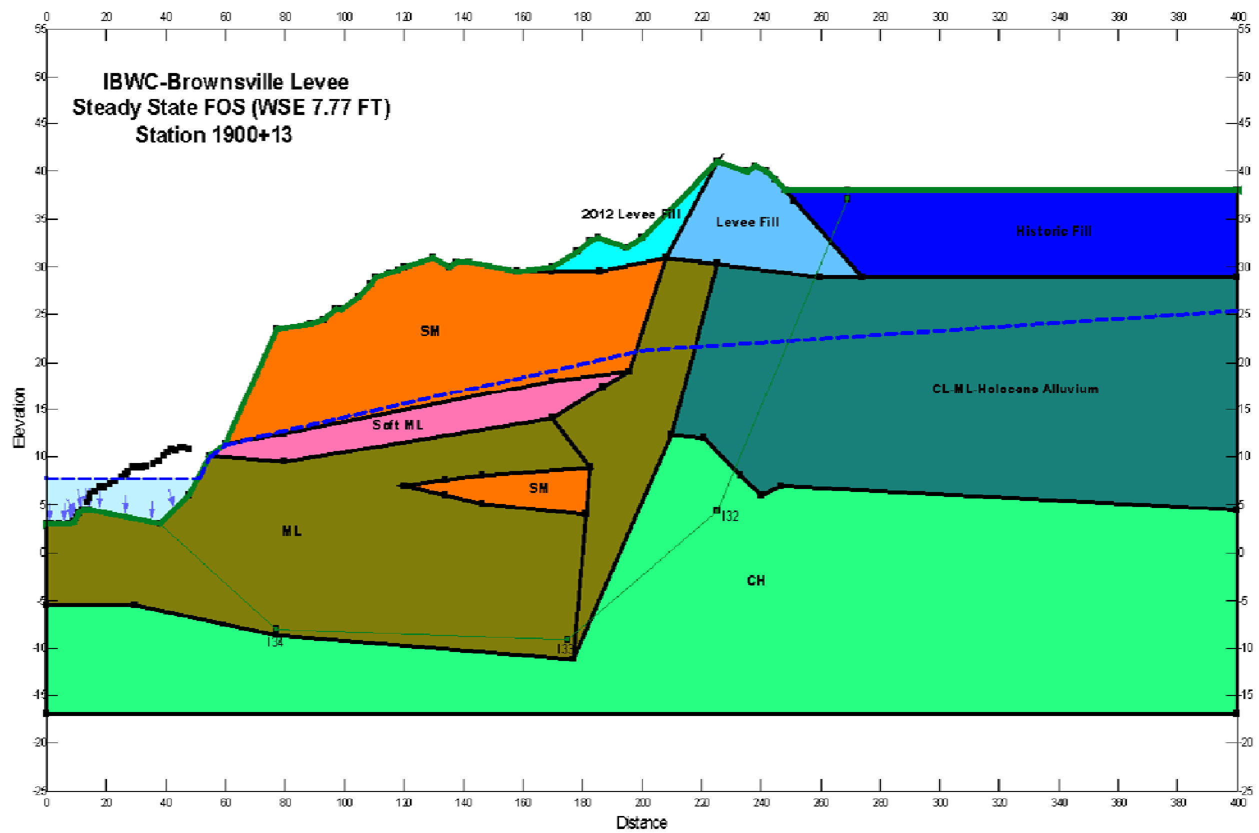
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

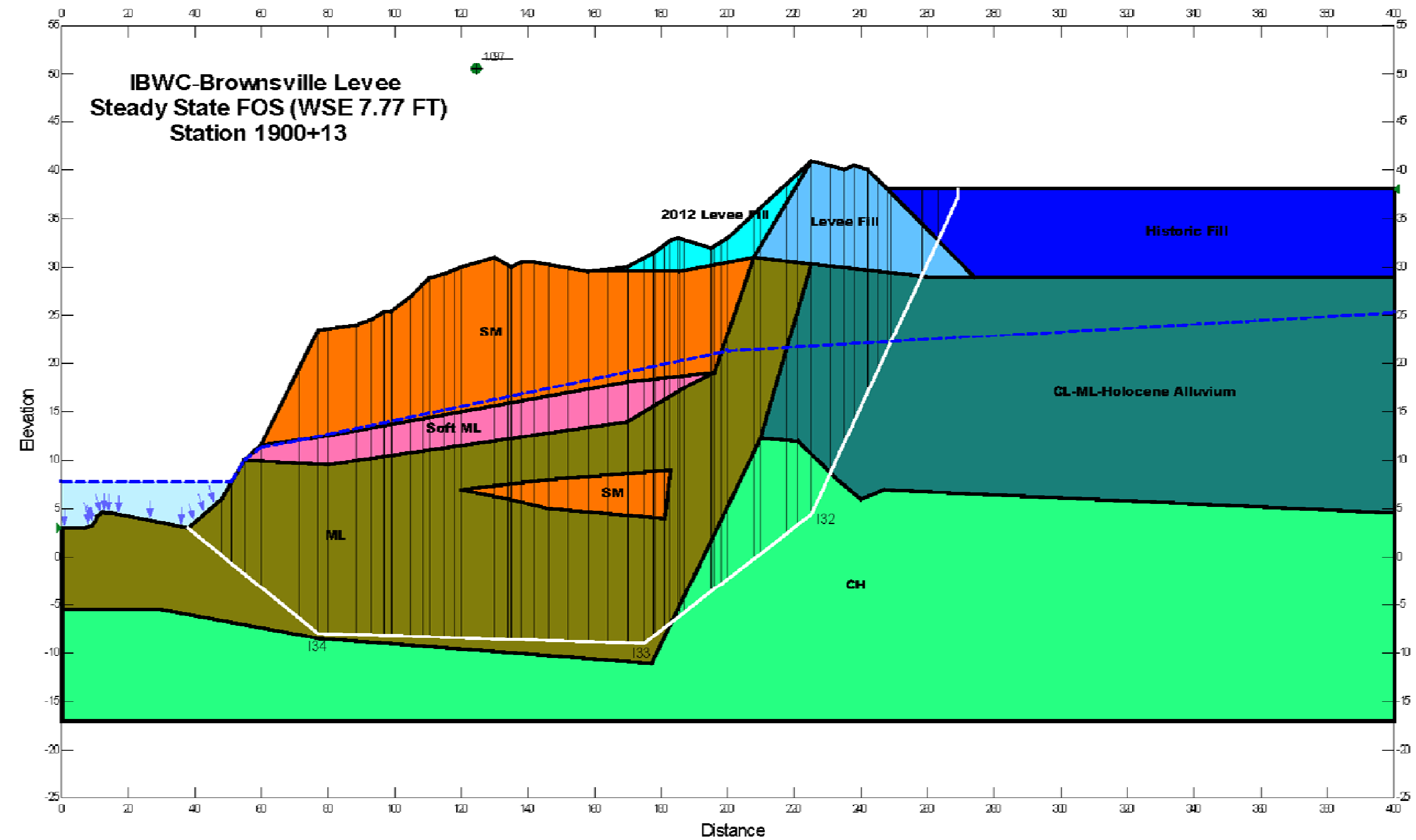
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - ALTERNATIVE 1
STEADY STATE SEEPAGE

ARCADIS

SS-2





Minimum Factor of Safety (FOS): 1.097

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
Combination 2: The shear strenght of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1			

NOT TO SCALE

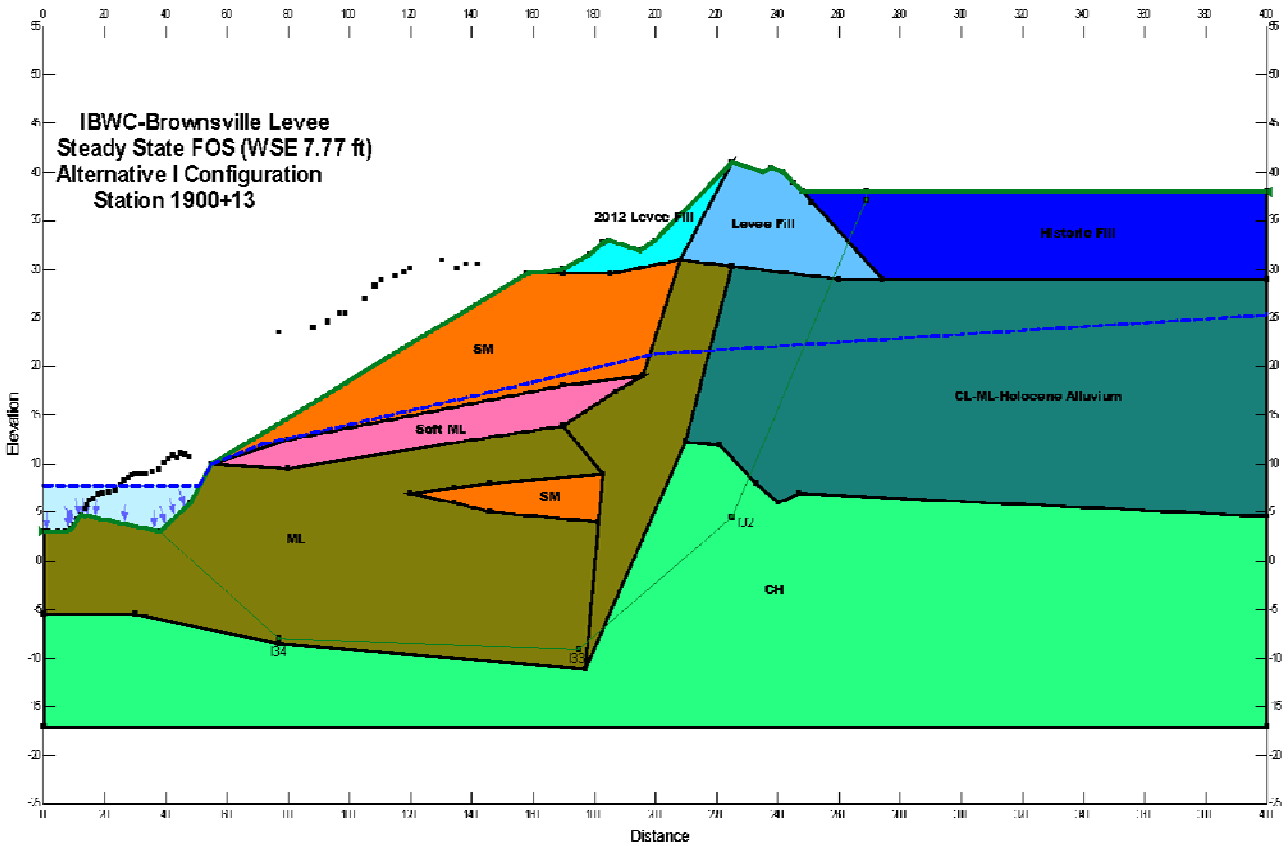
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

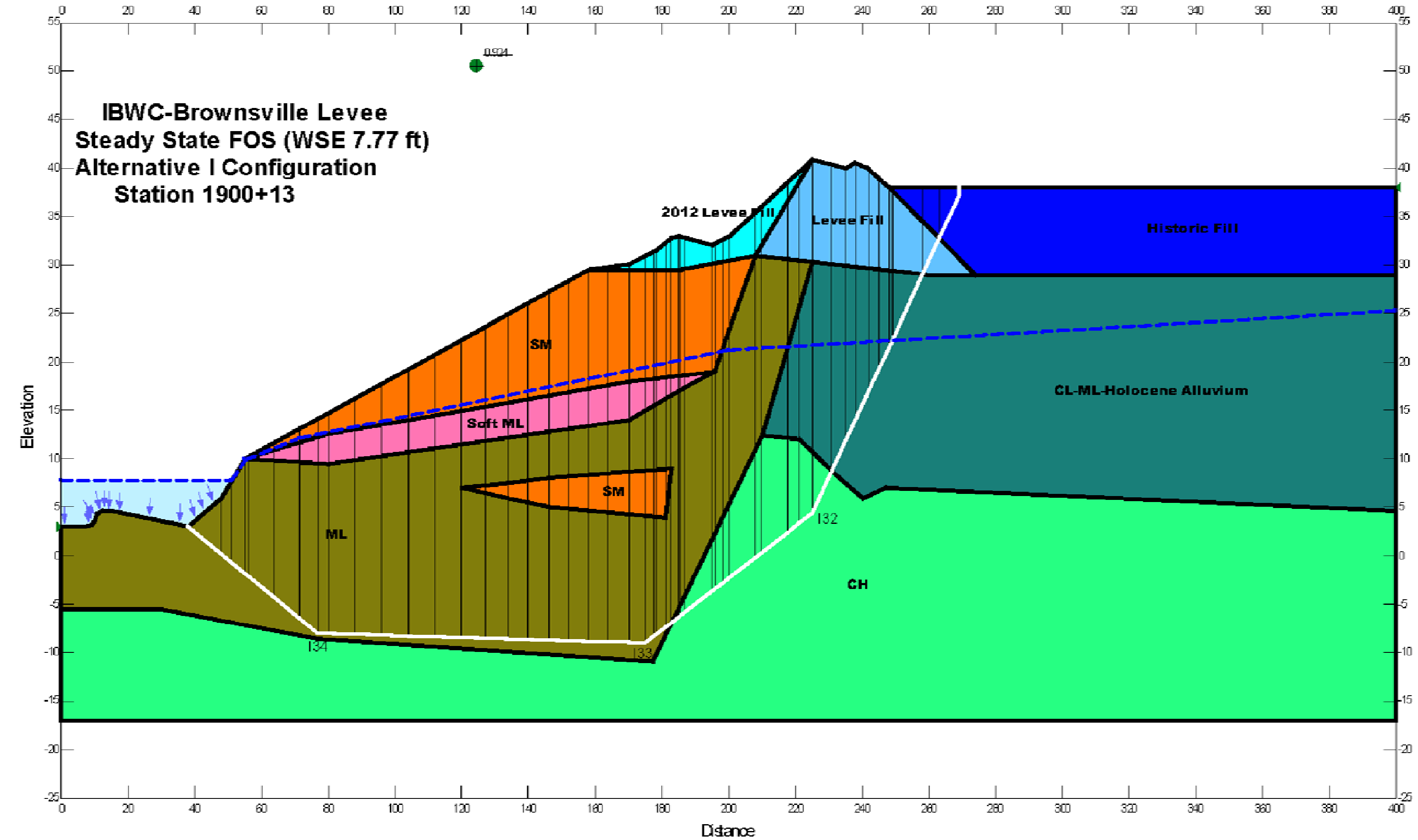
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - EXISTING SLOPE
STEADY STATE SEEPAGE

ARCADIS

SS-3





Minimum Factor of Safety (FOS): 0.924

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
SM	117	0	32
ML	119.38	0	11
2012 Levee Fill	127.34	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11
Soft ML	125.98	0	8
Combination 2			

NOT TO SCALE

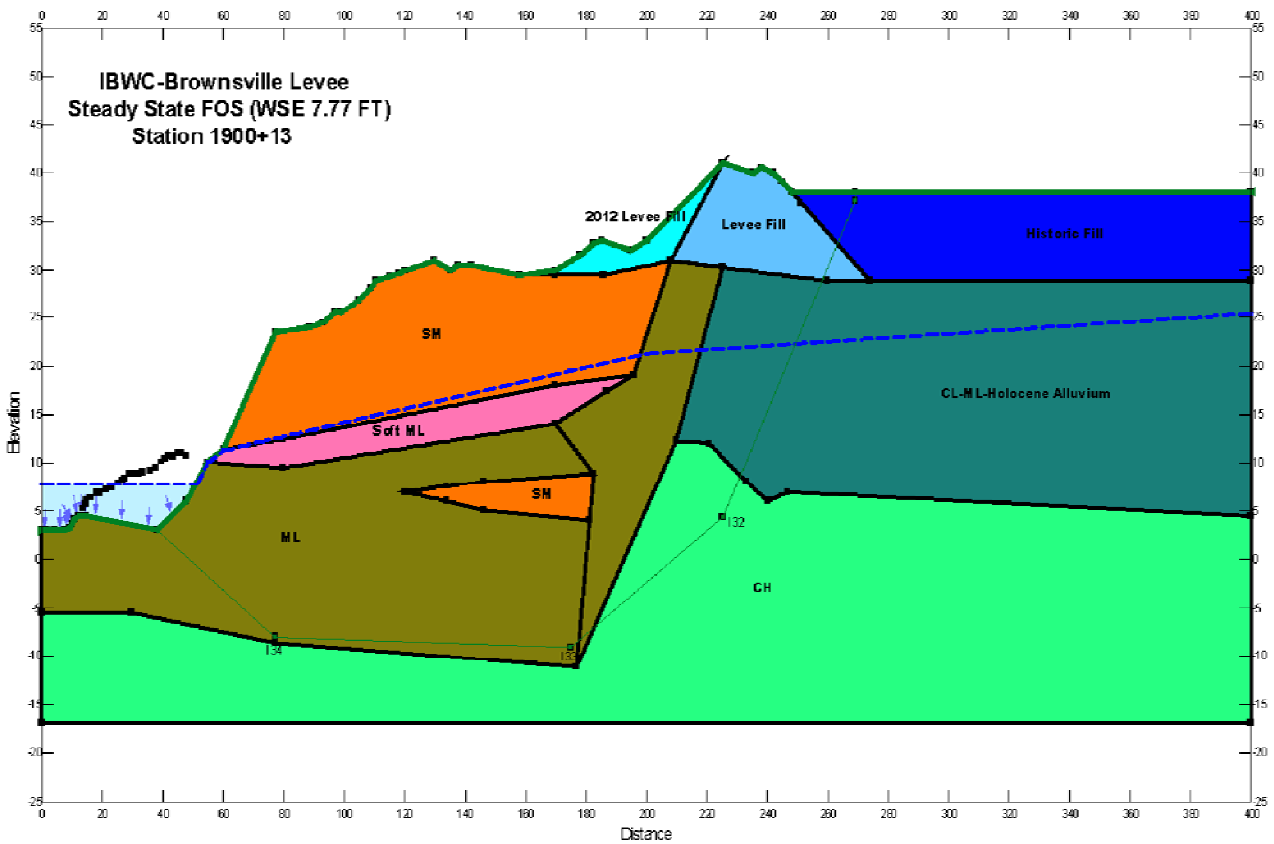
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

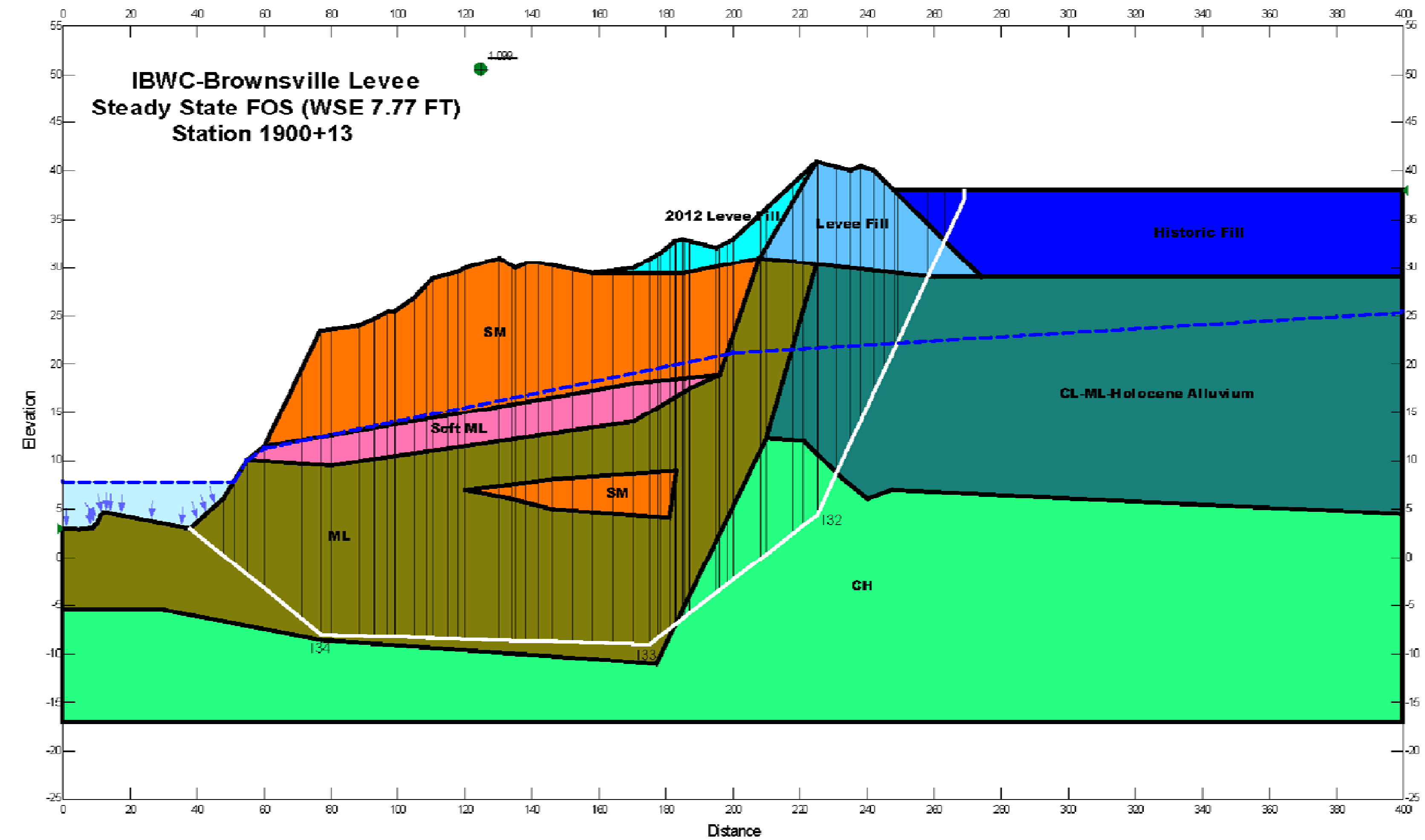
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - ALTERNATIVE 1
STEADY STATE SEEPAGE

ARCADIS

SS-4





Minimum Factor of Safety (FOS): 1.099

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1			

NOT TO SCALE

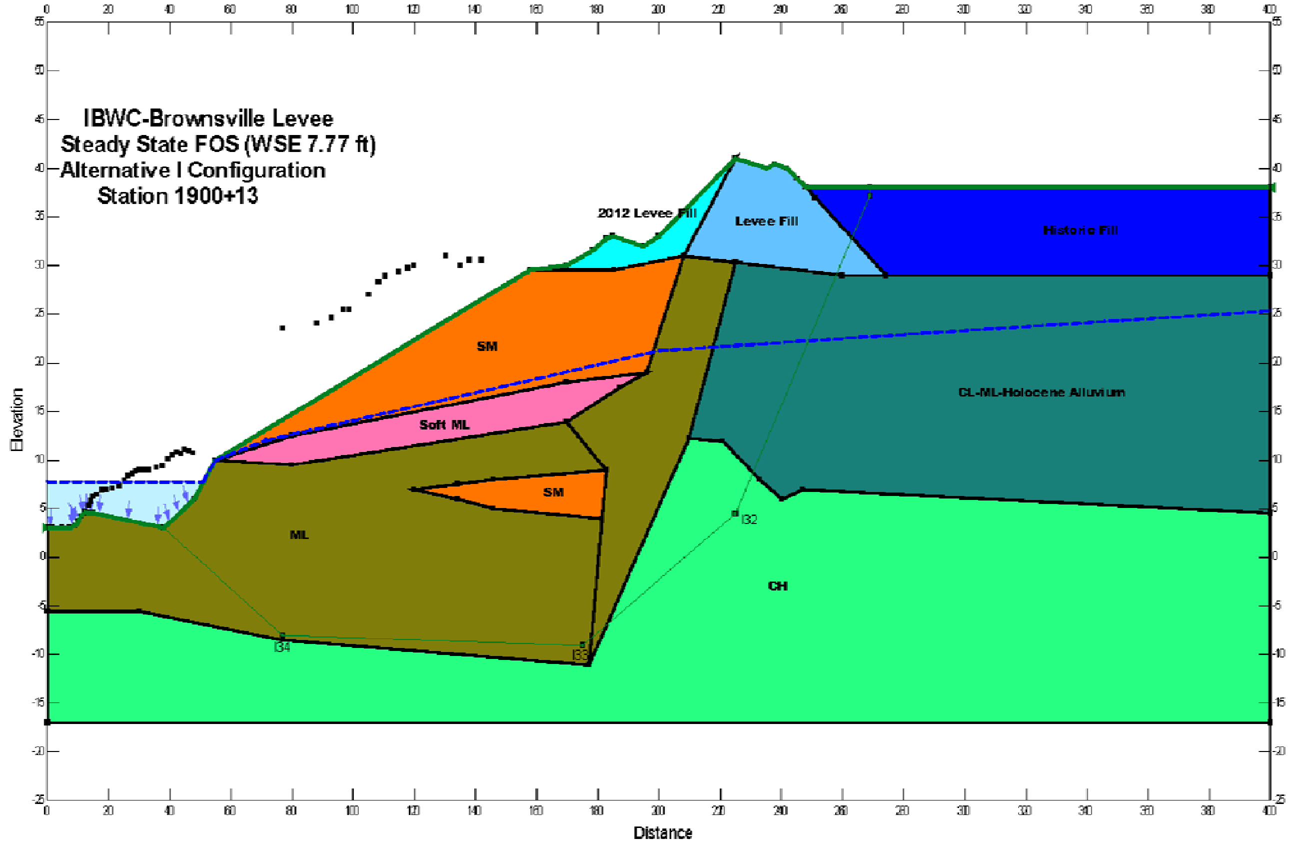
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

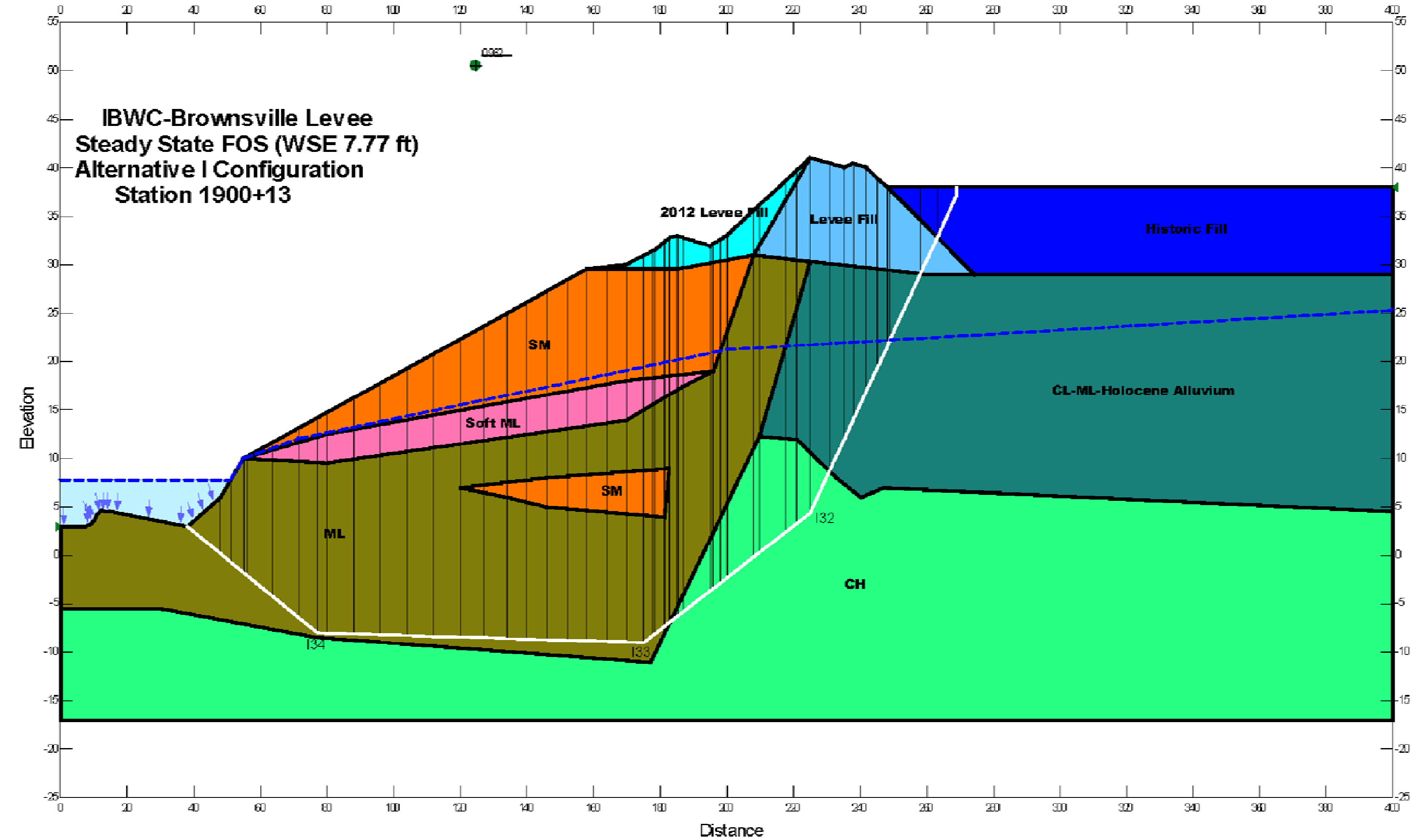
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - EXISTING SLOPE
STEADY STATE SEEPAGE

ARCADIS

SS-5





Minimum Factor of Safety (FOS): 0.962

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
Combination 3			

NOT TO SCALE

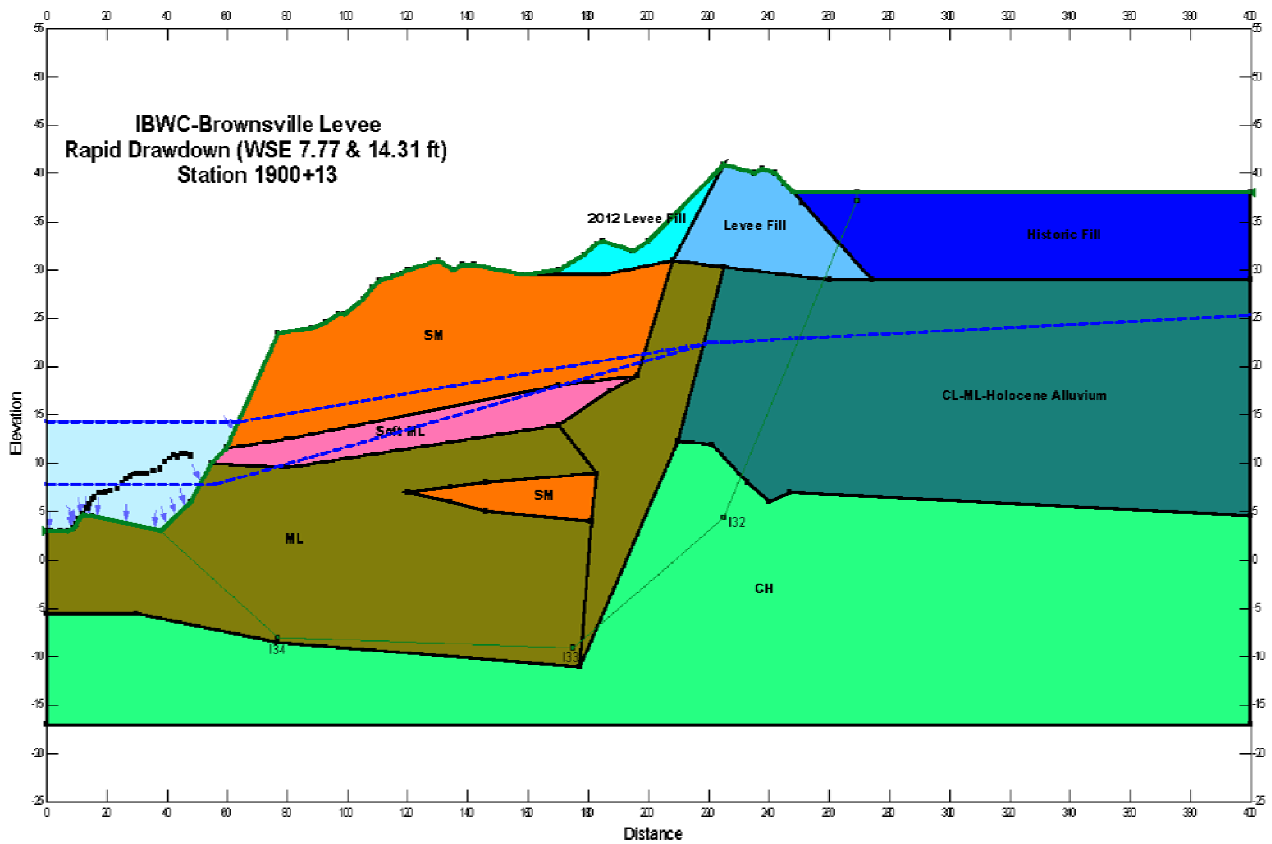
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

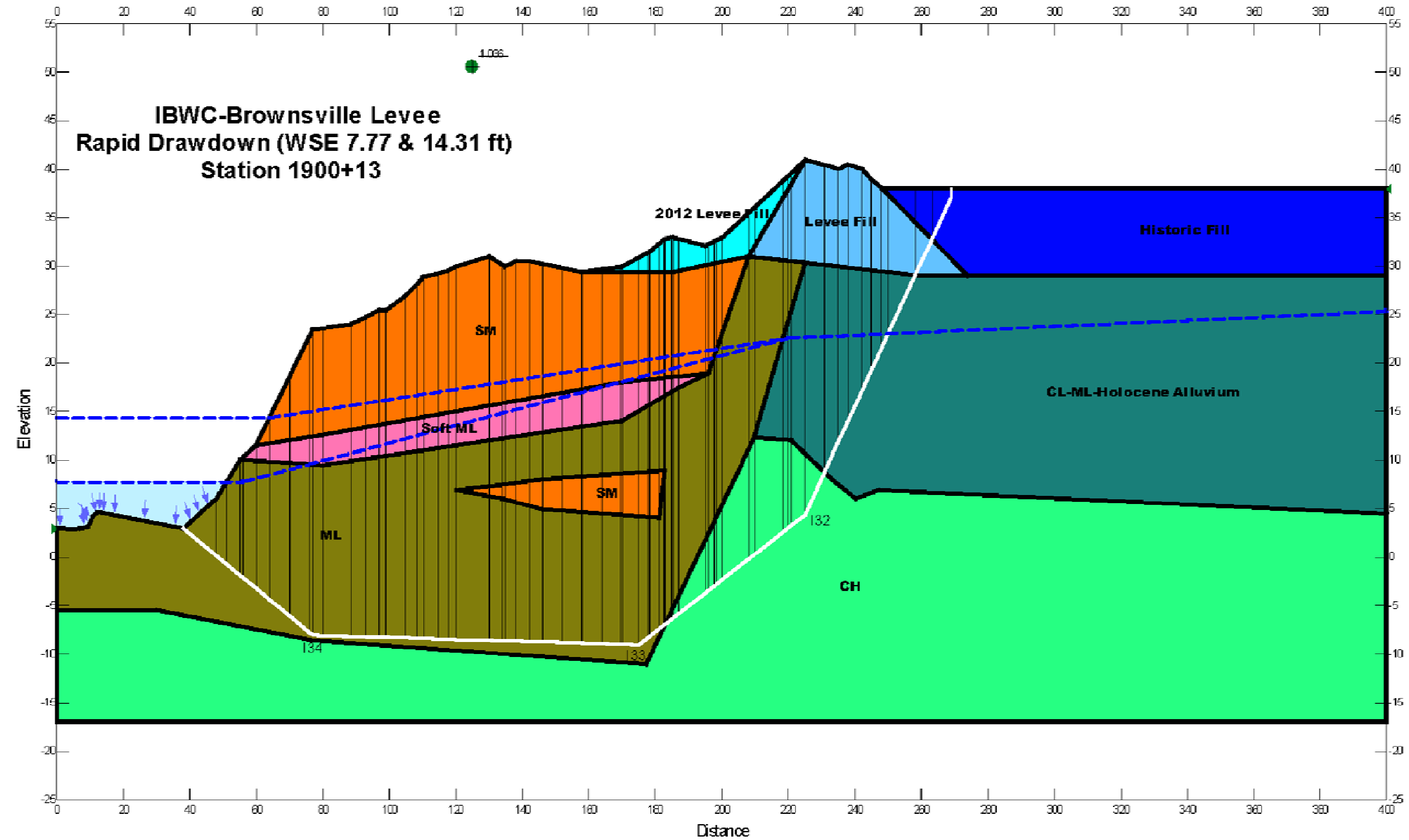
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - ALTERNATIVE 1
STEADY STATE SEEPAGE

ARCADIS

SS-6





Minimum Factor of Safety (FOS): 1.036

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

NOT TO SCALE

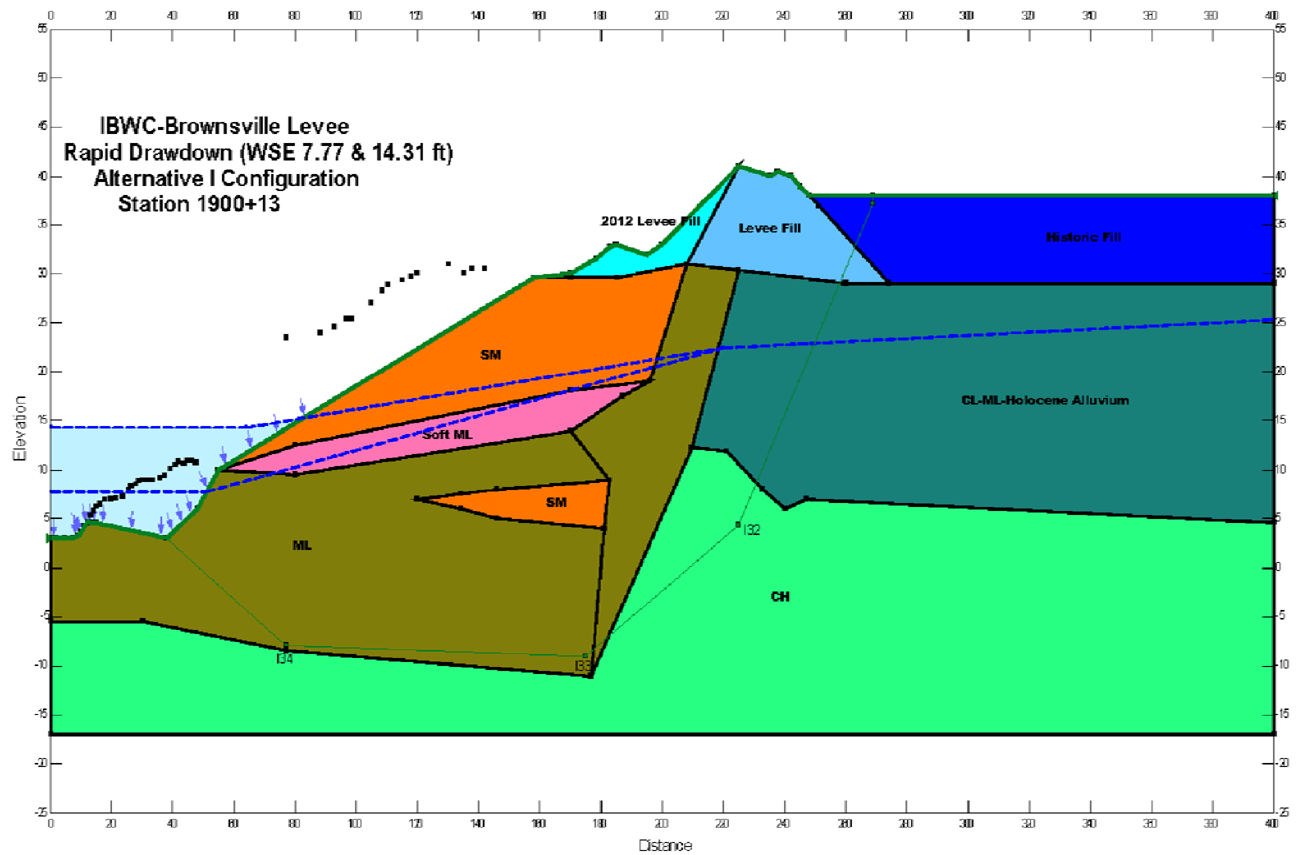
IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

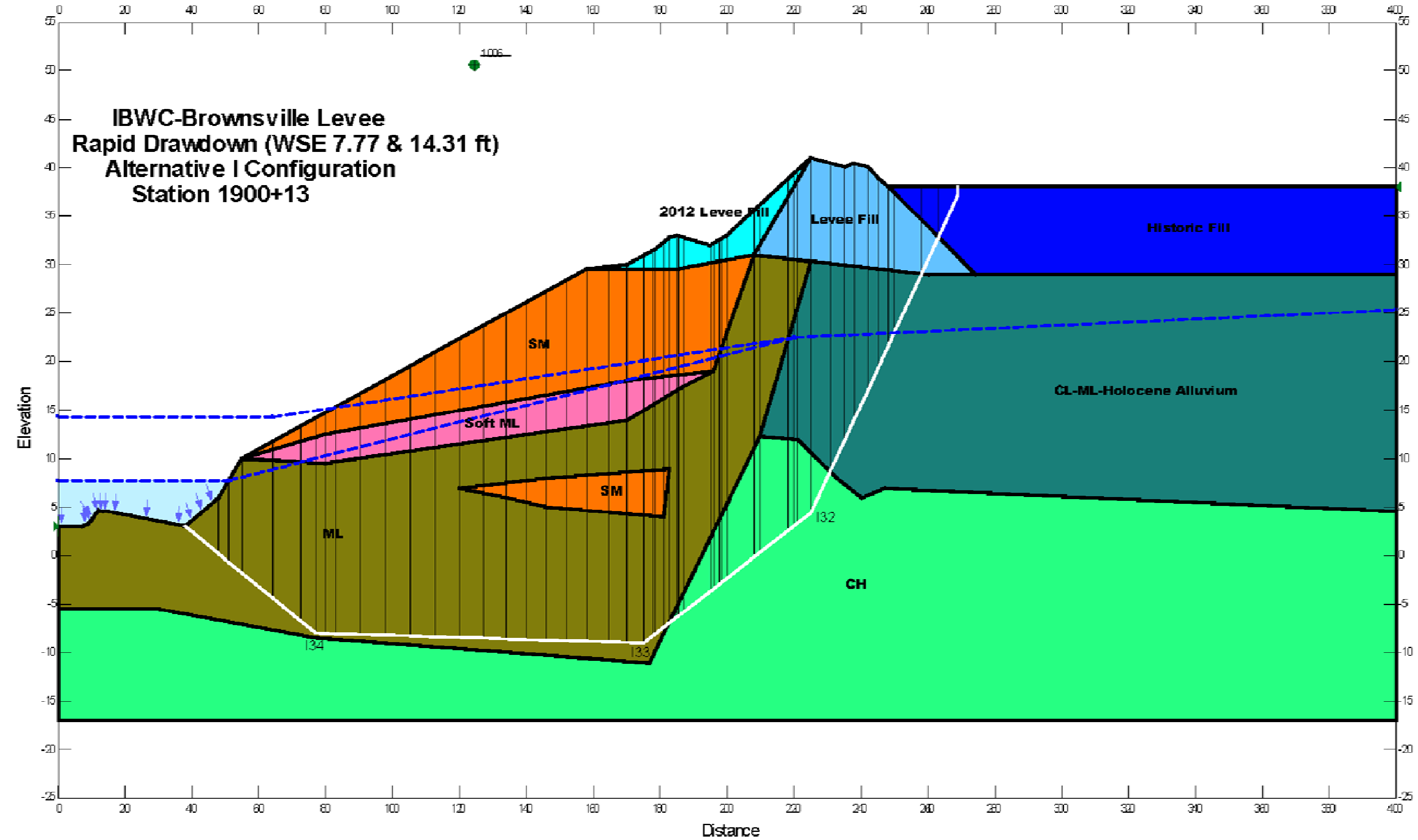
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - EXISTING SLOPE RAPID DRAWDOWN

ARCADIS

RD-1





Minimum Factor of Safety (FOS): 1.006

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
2012 Levee Fill	127.34	400	20	5000	0
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
Combination 4					

NOT TO SCALE

IBWC
PRELIMINARY GEOTECHNICAL MEMORANDUM

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

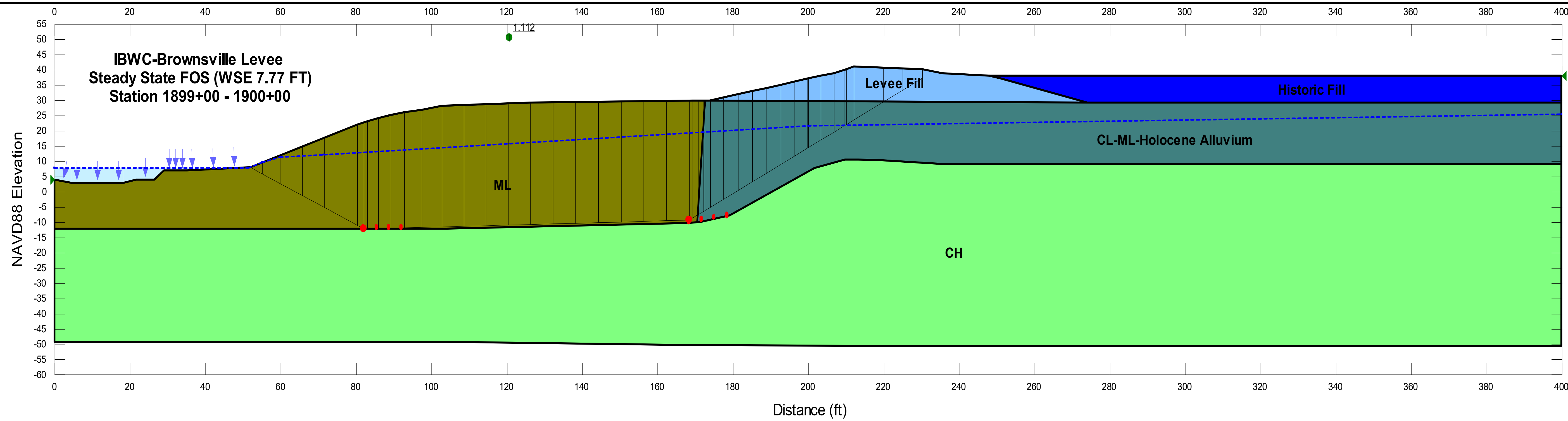
SLOPE STABILITY MODEL - ALTERNATIVE 1
RAPID DRAWDOWN

ARCADIS

RD-2

APPENDIX H

Station 1899+15 Slope Stability Analyses



Minimum Factor of Safety (FOS): 1.112

Potential Failure Surface			
Passive Block		Active Block	
Left Block		Right Block	
Projection Angle			
Starting Angle (Degree)	Ending Angle (Degree)	Starting Angle (Degree)	Ending Angle (Degree)
135	146.17	34.26	45

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	200	12
CL-Holocene	123.37	460	13
ML	119.38	230	0
Levee Fill	127.34	300	12
Historic Fill	127.34	200	24
Combination 1: The angle of internal friction (phi) of the silt (ML) was assessed as 0 degrees. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1			

IBWC
GEOTECHNICAL REPORT

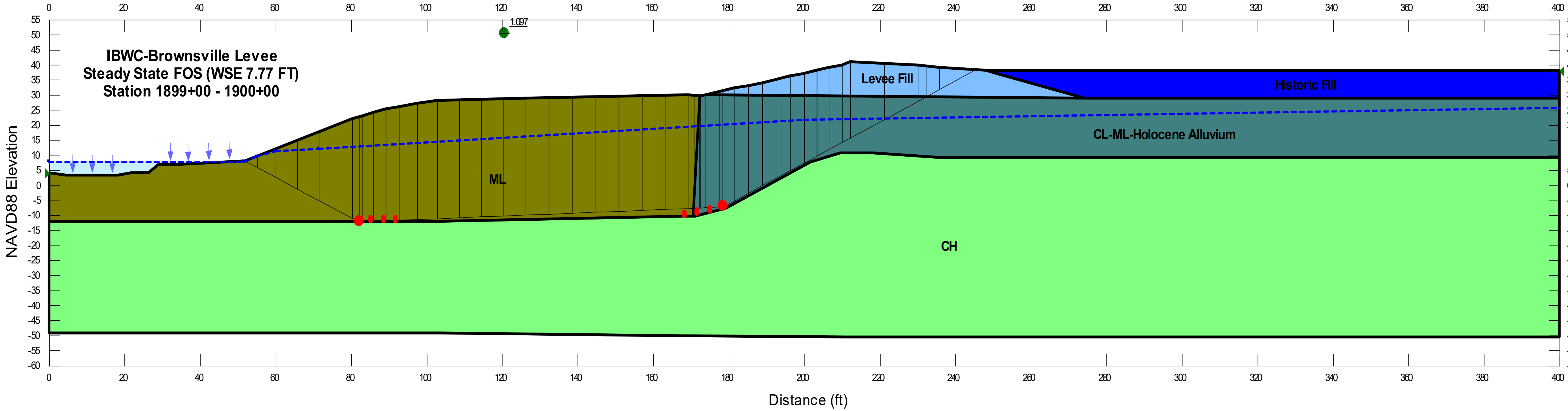
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY - NEW MODEL

STEADY STATE SEEPAGE

ARCADIS

APPENDIX



Minimum Factor of Safety (FOS): 1.097

Potential Failure Surface			
Passive Block		Active Block	
Left Block		Right Block	
Projection Angle			
Starting Angle (Degree)	Ending Angle (Degree)	Starting Angle (Degree)	Ending Angle (Degree)
140	146.17	34.26	50

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	0	10
CL-Holocene	123.37	0	10
ML	119.38	0	11
Levee Fill	127.34	0	11
Historic Fill	127.34	0	11

Combination 2: The shear strength of the soil was base on friction and the cohesion (c) of all the material was assessed as 0 psf. Varied the phi of all the materials to yield a FOS approximately of 1

IBWC
GEOTECHNICAL REPORT

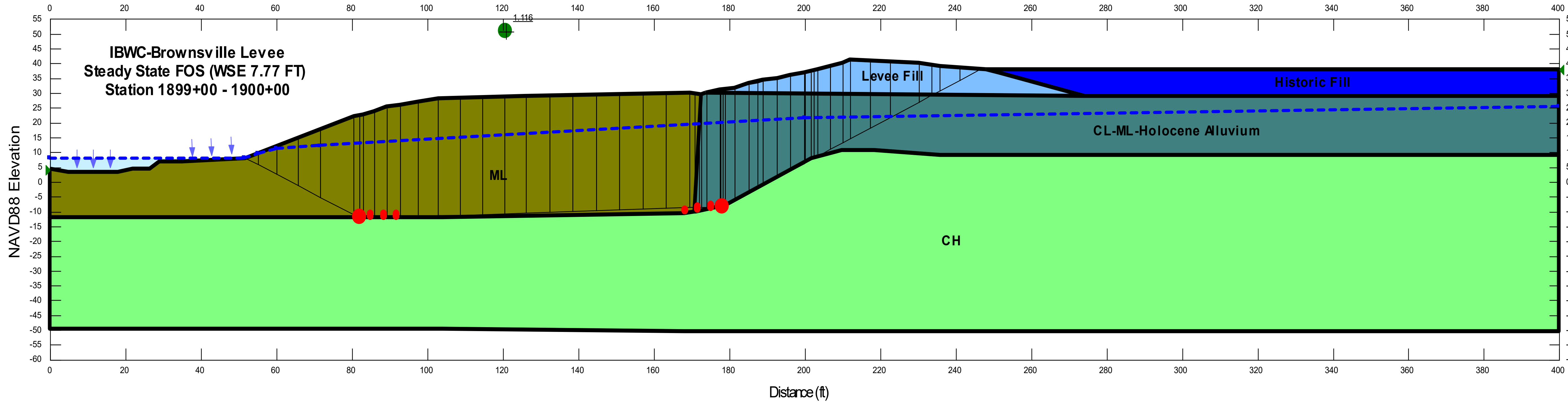
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY - NEW MODEL

STEADY STATE SEEPAGE

ARCADIS

APPENDIX



Minimum Factor of Safety (FOS): 1.116

Potential Failure Surface			
Passive Block		Active Block	
Left Block		Right Block	
Projection Angle			
Starting Angle (Degree)	Ending Angle (Degree)	Starting Angle (Degree)	Ending Angle (Degree)
139	146.17	34.26	49

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
ML	119.38	0	8
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12

Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1

IBWC
GEOTECHNICAL REPORT

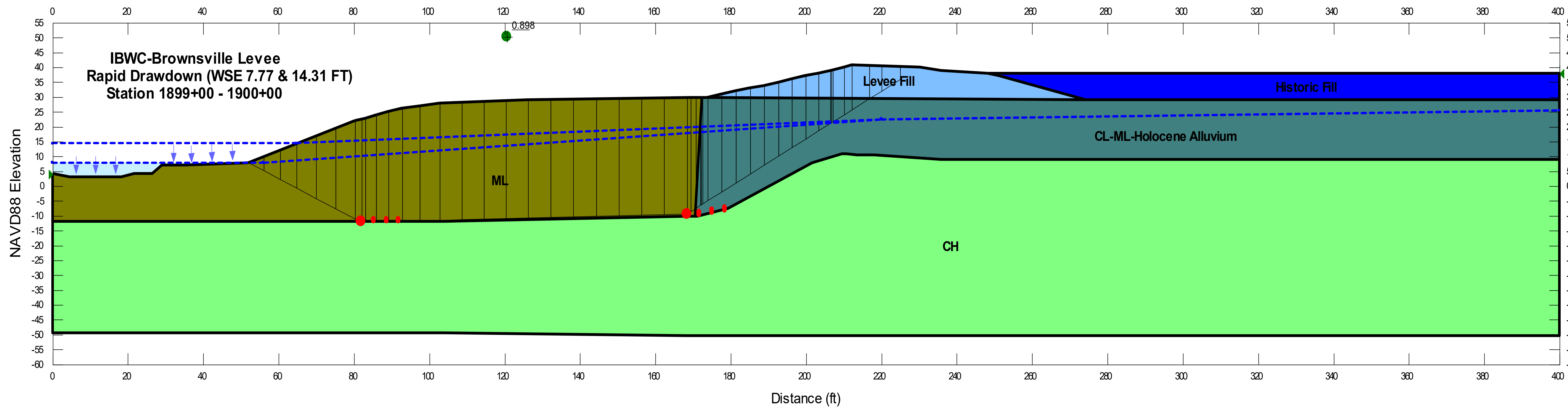
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY - NEW MODEL

STEADY STATE SEEPAGE

ARCADIS

APPENDIX



Minimum Factor of Safety (FOS): 0.898

Potential Failure Surface			
Passive Block		Active Block	
Left Block		Right Block	
Projection Angle			
Starting Angle (Degree)	Ending Angle (Degree)	Starting Angle (Degree)	Ending Angle (Degree)
142	146.17	34.26	52

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress c (psf)	Total Stress phi (degree)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
ML	119.38	190	0	0	29
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15

Combination 4: The angle of internal friction (phi') of the silt (ML) for the drained condition was assessed as 0 degrees. Varied the cohesion (c') and phi' of the other materials to yield a FOS of approximately 1

IBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

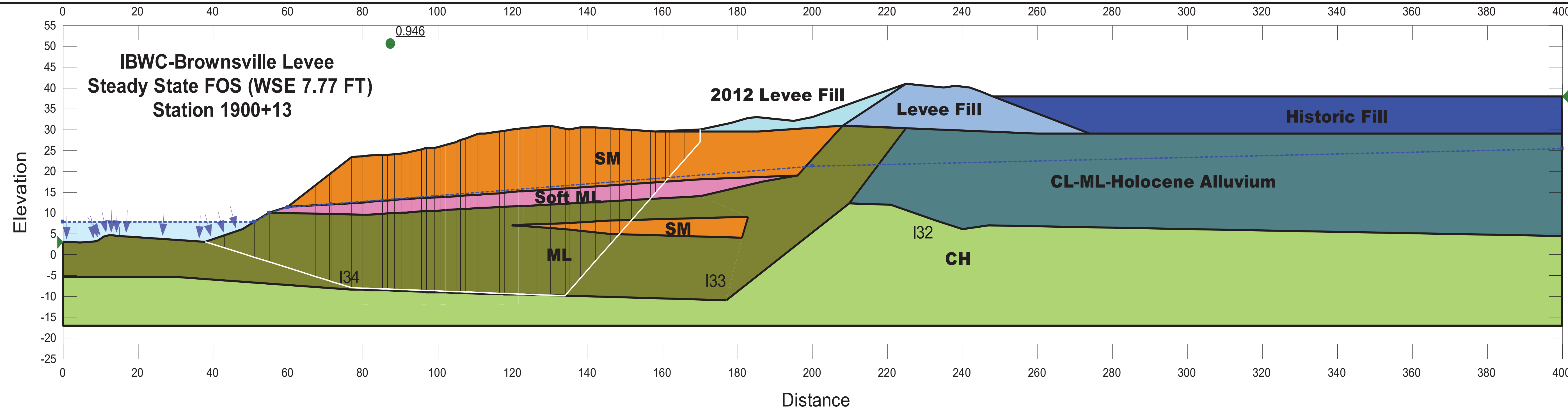
SLOPE STABILITY - NEW MODEL

RAPID DRAWDOWN

ARCADIS | **APPENDIX**

APPENDIX I

Stability Analyses at Toe of Levee, Station 1900+13



Minimum Factor of Safety (FOS): 0.946

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)
CH Pleistocene	121.98	90	12
CL-Holocene	123.37	225	12
SM	117	0	32
ML	119.38	0	8
2012 Levee Fill	127.34	105	12
Levee Fill	127.34	105	12
Historic Fill	127.34	95	12
Soft ML	125.98	150	0
DMZ	120	1536.8	0
Combination 3: The cohesion (c) of the silt (ML) was assessed as 0 psf. Varied the cohesion (c) and phi of the other materials to yield a FOS of approximately 1			

IBWC
GEOTECHNICAL REPORT

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE
WITHIN THE UPPER BROWNSVILLE LEVEE REACH
LOWER RIO GRANDE FLOOD CONTROL PROJECT

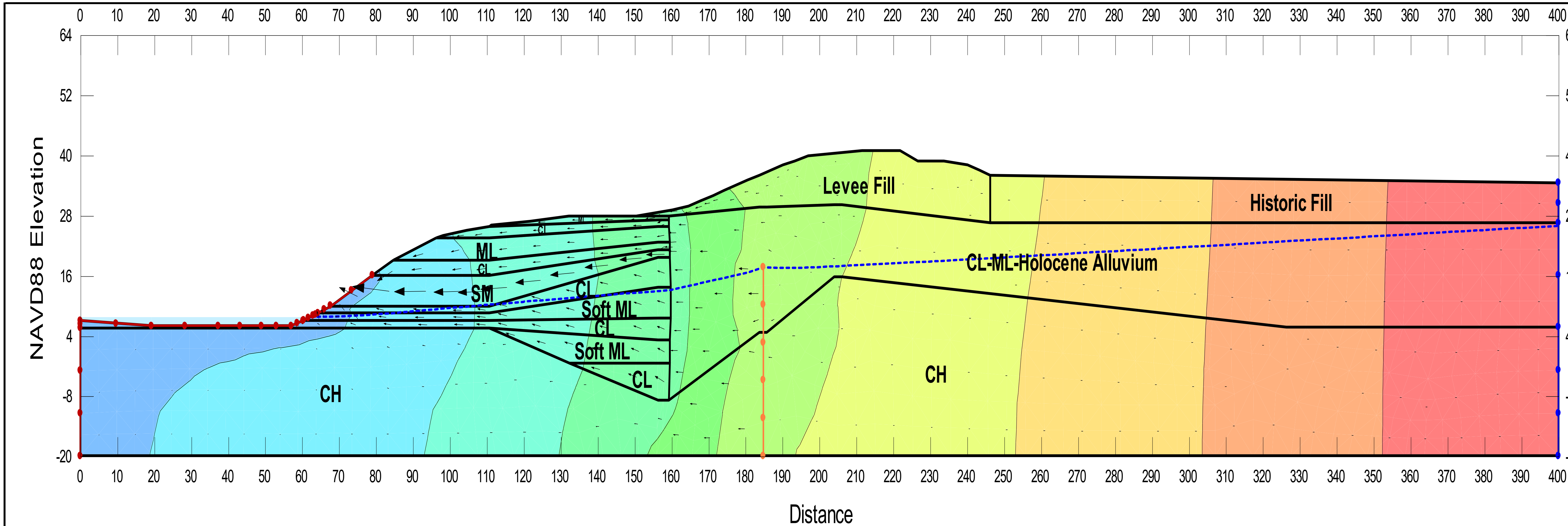
SLOPE STABILITY MODEL - EXISTING CONDITIONS RIVERWARD OF LEVEE

STEADY STATE SEEPAGE

ARCADIS | **APPENDIX**

APPENDIX J

Seepage Analysis at Station 1903+96



Boundary Conditions	Type	Magnitude (ft)
River	Head	7.77
Boring DP-202	Head	17.68
Protected Side	Head	25.98

Material	K_{sat} (ft/sec)	m_v (1/psf)	Ratio
CH Pleistocene	3.30E-08	3.60E-06	0.2
CL-Holocene	3.30E-08	2.50E-06	0.2
CL	1.00E-07	1.00E-06	1
SM	3.30E-07	5.00E-06	0.2
ML	1.00E-07	1.00E-05	0.2
Levee Fill	3.30E-08	3.74E-06	0.2
Historic Fill	3.30E-08	3.74E-06	0.2
Soft ML	1.00E-07	1.00E-05	1

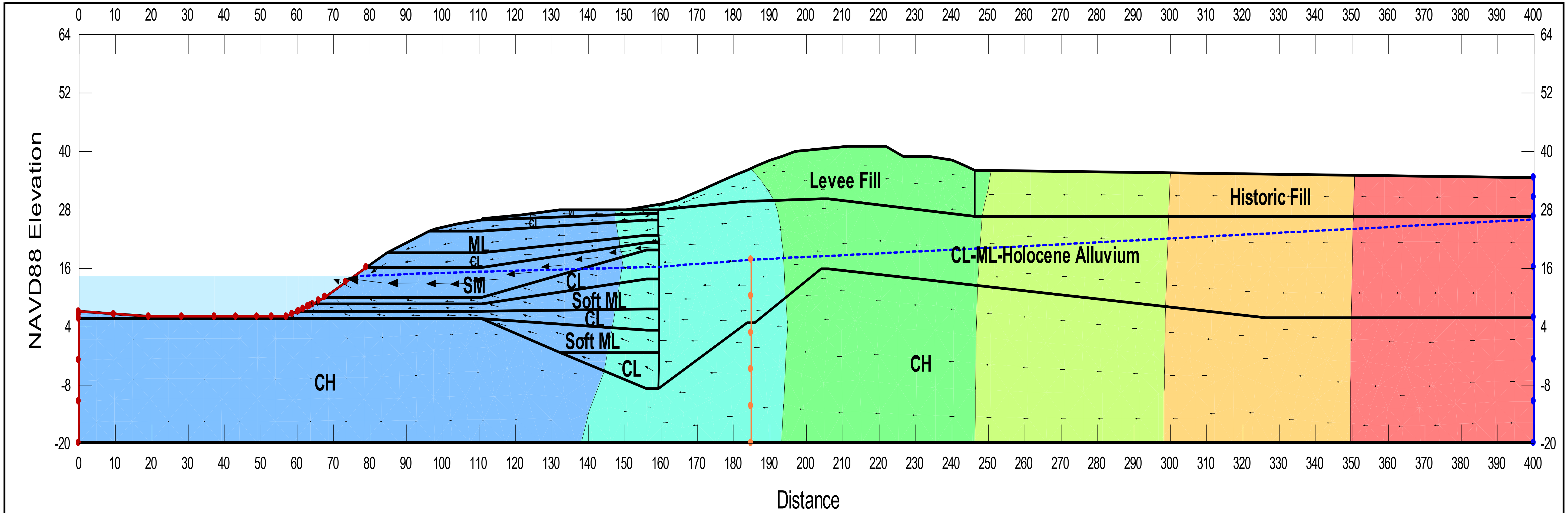
IBWC
STATION 1903+96

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SEEPAGE MODEL

STEADY STATE SEEPAGE (WSE 7.77 FT)

ARCADIS | **APPENDIX**



Boundary Conditions	Type	Magnitude (ft)
River	Head	14.31
Boring DP-202	Head	17.68
Protected Side	Head	25.98

Material	K_{sat} (ft/sec)	m_v (1/psf)	Ratio
CH Pleistocene	3.30E-08	3.60E-06	0.2
CL-Holocene	3.30E-08	2.50E-06	0.2
CL	1.00E-07	1.00E-06	1
SM	3.30E-07	5.00E-06	0.2
ML	1.00E-07	1.00E-05	0.2
Levee Fill	3.30E-08	3.74E-06	0.2
Historic Fill	3.30E-08	3.74E-06	0.2
Soft ML	1.00E-07	1.00E-05	1

IBWC
STATION 1903+96

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

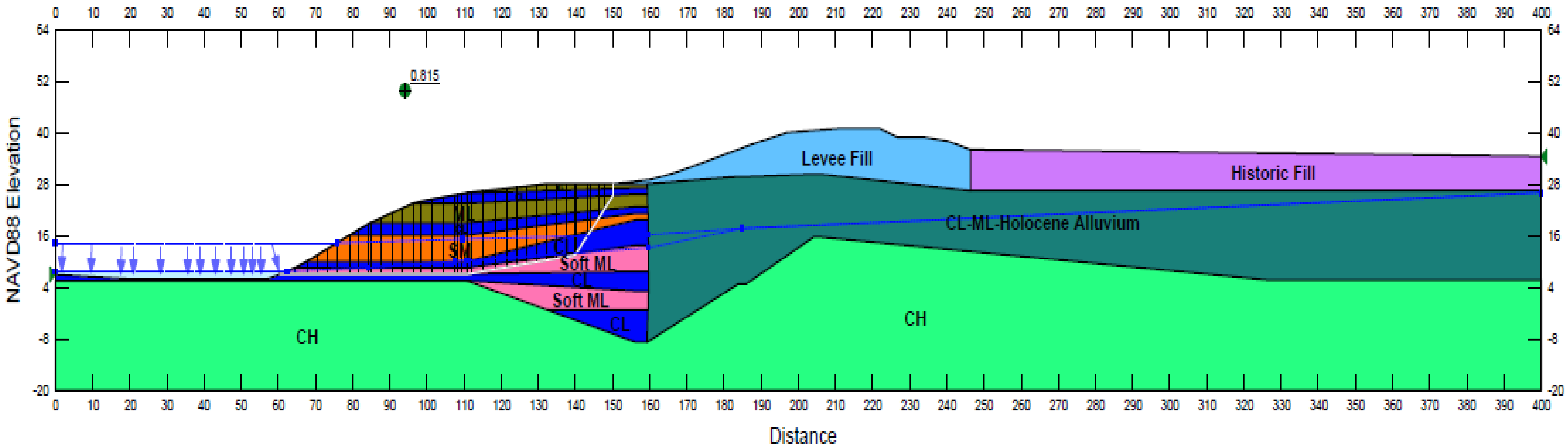
SEEPAGE MODEL

STEADY STATE SEEPAGE (WSE 14.31 FT)

ARCADIS | **APPENDIX**

APPENDIX K

Slope Stability Analyses at Station 1903+96



Minimum Factor of Safety (FOS): 0.815

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress	
				c (psf)	phi (deg.)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
CL	120	300	0	300	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0

Rapid Drawdown Conditions

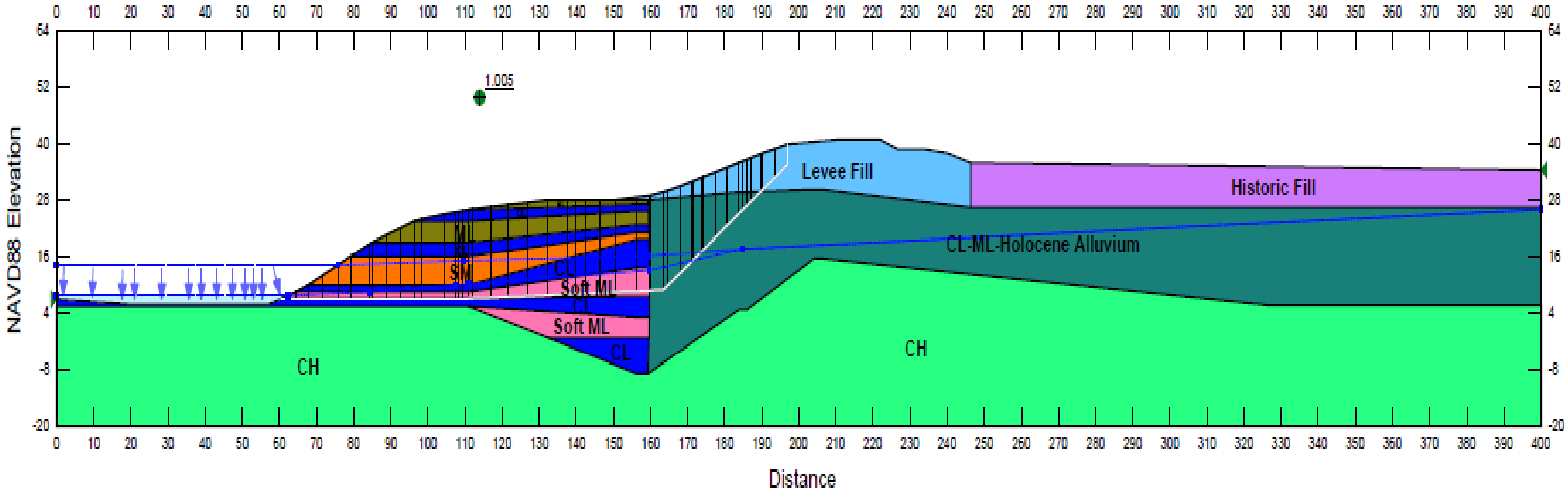
IBWC
STATION 1903+96

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - FAILURE NEAR LEVEE TOE RAPID DRAWDOWN (WSE 7.77 FT & 14.31 FT)

ARCADIS

APPENDIX



Minimum Factor of Safety (FOS): 1.005

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress	
				c (psf)	phi (deg.)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
CL	120	300	0	300	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0

Rapid Drawdown Conditions

IBWC
STATION 1903+96

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

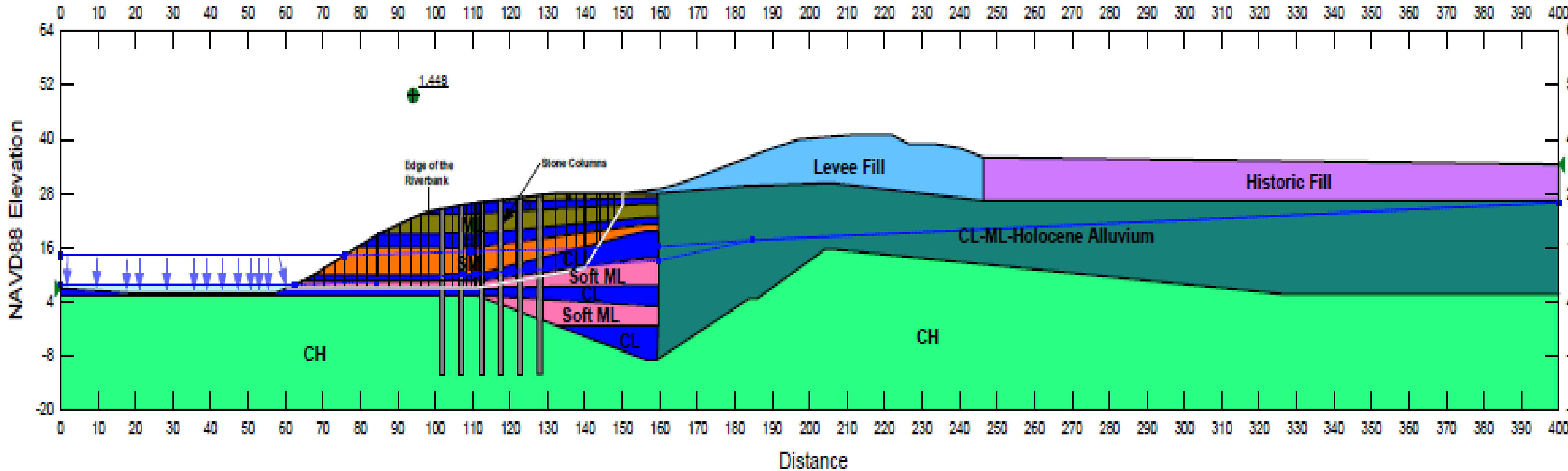
SLOPE STABILITY MODEL - FAILURE WITHIN LEVEE
RAPID DRAWDOWN (WSE 7.77 FT & 14.31 FT)

ARCADIS

APPENDIX

APPENDIX L

Stone Columns Analyses at STA 1903+96



Minimum Factor of Safety (FOS): 1.448

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress	
				c (psf)	phi (deg.)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
CL	120	300	0	300	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
Stone Columns	127	0	40	0	40

Rapid Drawdown Conditions

IBWC
STATION 1903+96

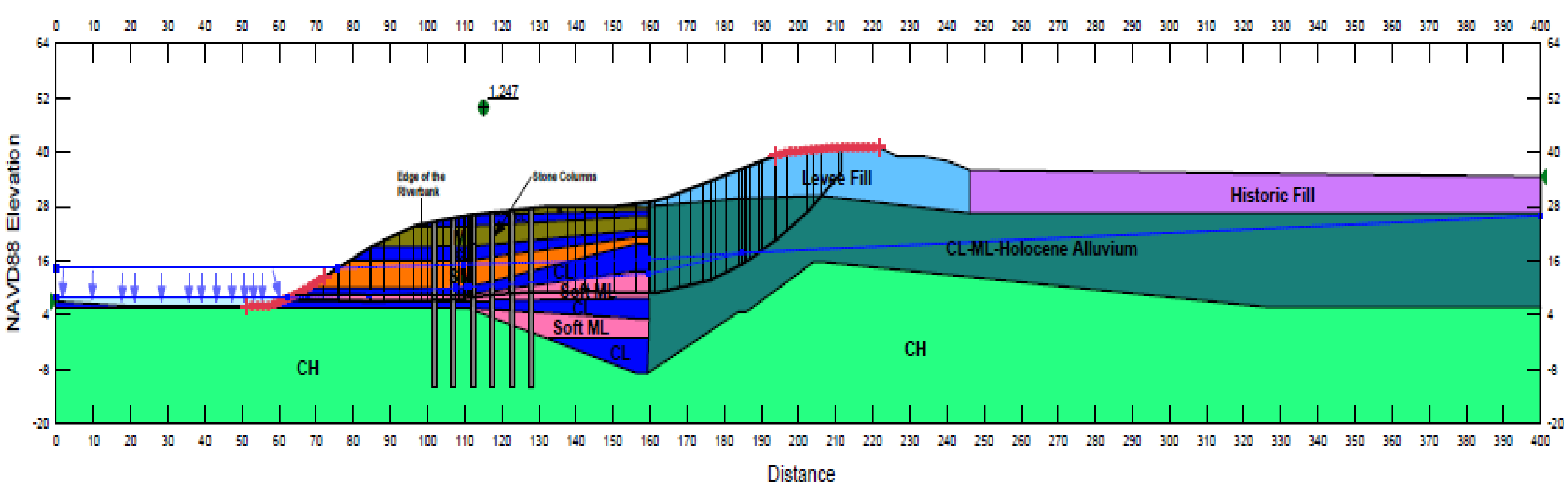
REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - FAILURE NEAR LEVEE TOE

RAPID DRAWDOWN (WSE 7.77 FT & 14.31 FT)

ARCADIS

APPENDIX



Minimum Factor of Safety (FOS): 1.247

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress	
				c (psf)	phi (deg.)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
CL	120	300	0	300	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
Stone Columns	127	0	40	0	40

Rapid Drawdown Conditions

IBWC
STATION 1903+96

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - FAILURE WITHIN LEVEE

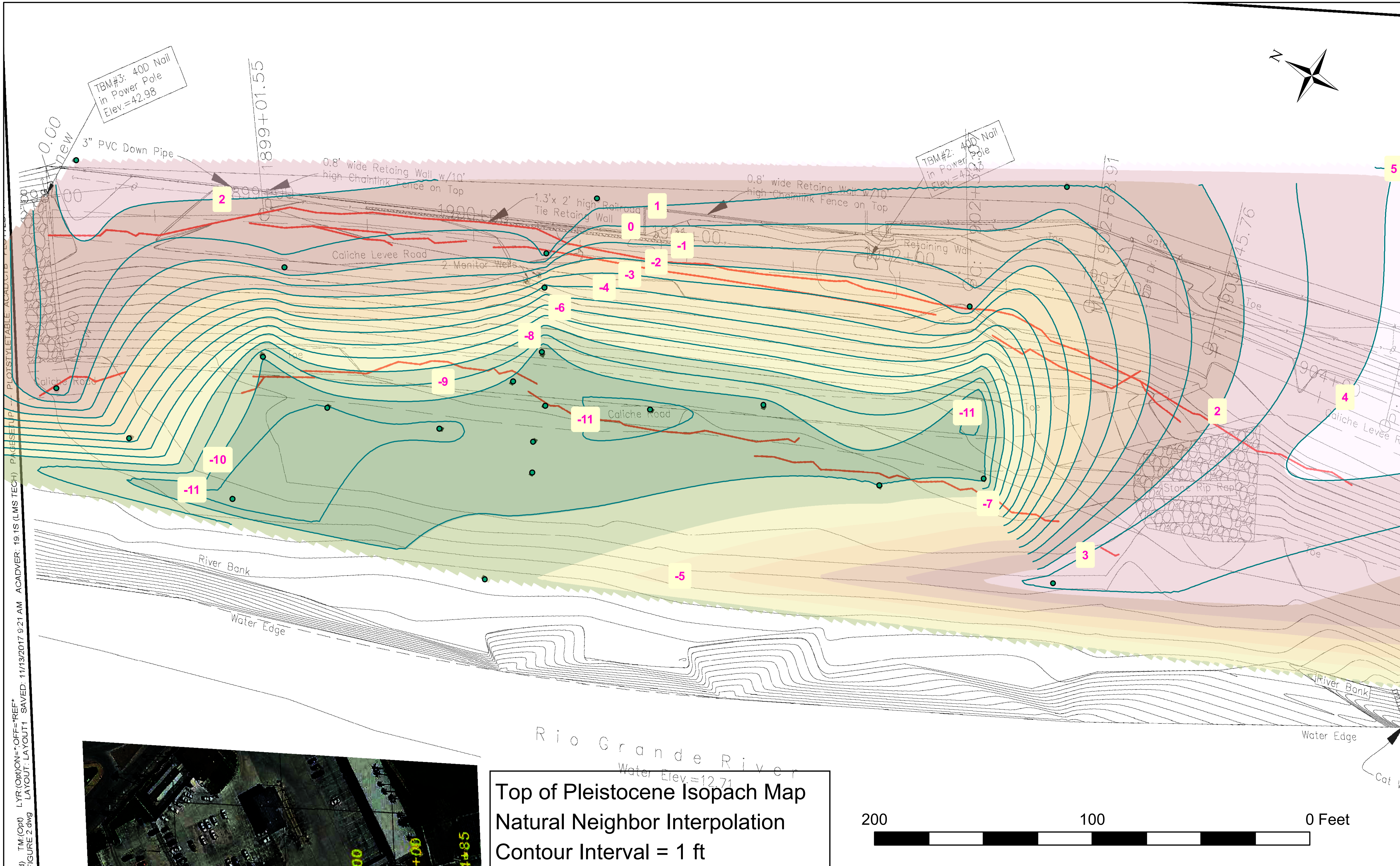
RAPID DRAWDOWN (WSE 7.77 FT & 14.31 FT)

ARCADIS

APPENDIX

APPENDIX M

Top of Pleistocene Clay Isopach Maps

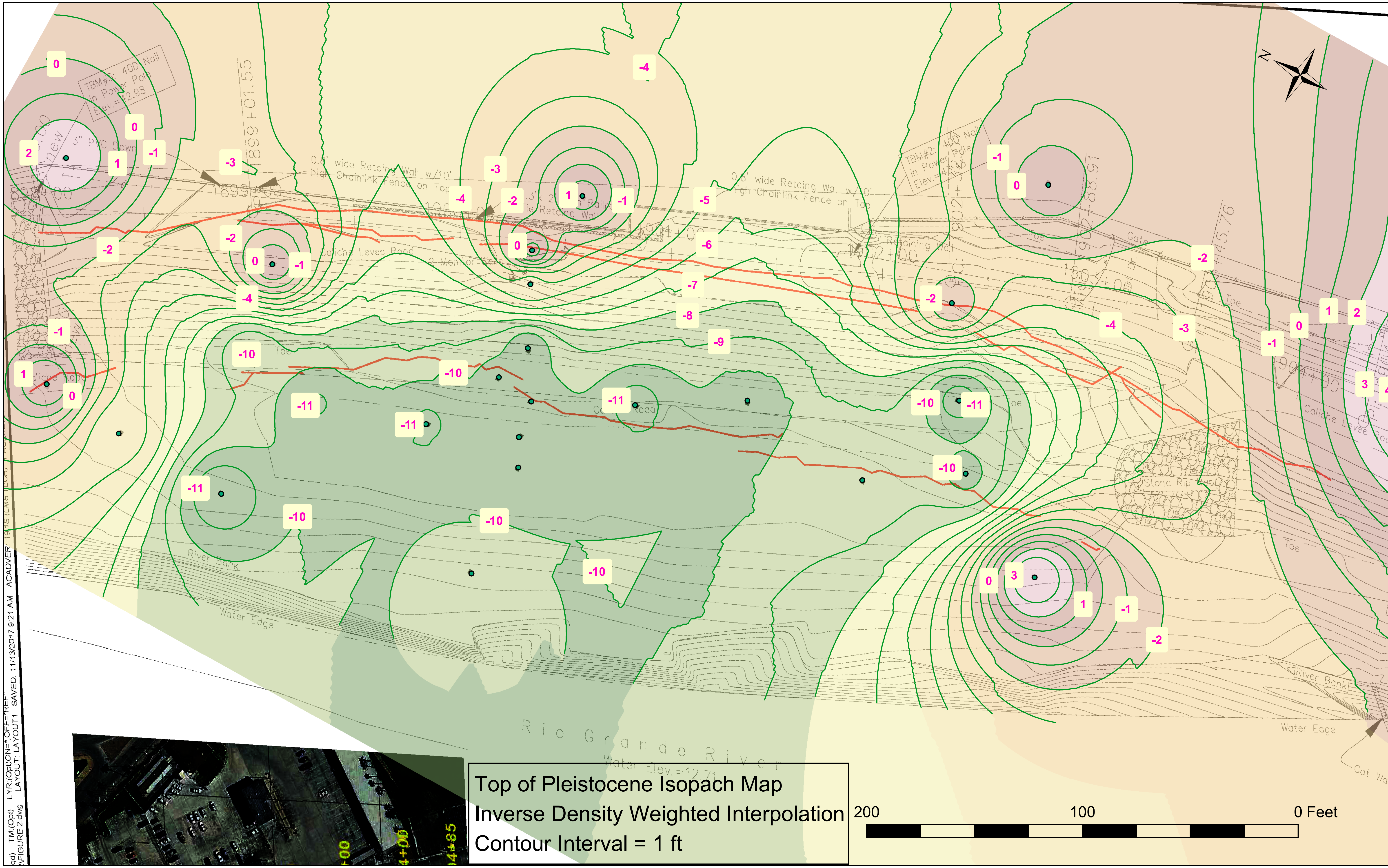


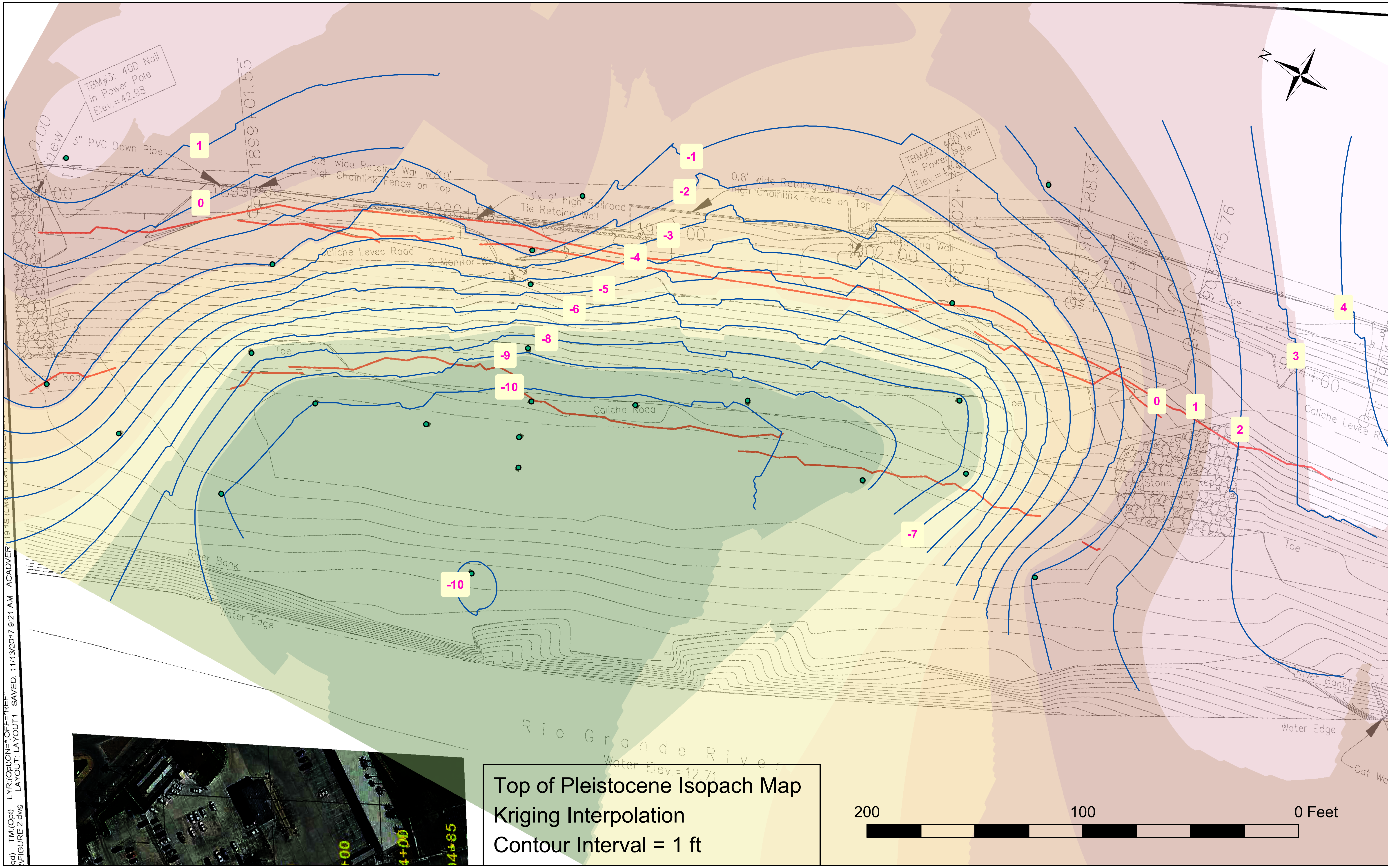
Top of Pleistocene Isopach Map
Natural Neighbor Interpolation
Contour Interval = 1 ft

200 100 0 Feet

LYR:(Opt)ON=*;OFF=*REF* LAYOUT: LAYOUT1 SAVED: 11/13/2017 9:21 AM ACADVER: 19.1S (LMS TECH) PAGESETUP: -- PLOTSTYLETABLE: ACADCTB

TM:(Opt) FIGURE 2.dwg


Top of Pleistocene Isopach Map
Inverse Density Weighted Interpolation
Contour Interval = 1 ft
200
100
0 Feet
Rio Grande River
Water Elev.=12.71
River Bank
Water Edge
Caliche Levee Road
Stone Rip Rap
Cat Walk


Top of Pleistocene Isopach Map
Kriging Interpolation
Contour Interval = 1 ft
200
100
0 Feet
TBM#3: 40D Nail in Power Pole Elev.=42.98
TBM#2: 40D Nail in Power Pole
3" PVC Down Pipe
0.8' wide Retaining Wall w/10' high Chainlink Fence on Top
1.3'x 2' high Railroad Tie Retaining Wall
Caliche Levee Road
2 Monitor Wells
Retaining Wall
Gate
Toe
Caliche Road
Stone Rip Rap
River Bank
Water Edge
Rio Grande River
Cat Walk
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
2
3
4

APPENDIX N

Revised DSM Shear Panel Calculations and Verification Model

For	IBWC - Brownsville Levee Rehab	Job No.	LA003315.0000	Sheet No.	1 of 7
Calc	Mixed Soil Shear Wall Design	Made By	CW	Checked by	KL
		Date	12/19/2017	Date	12/23/2017

ARCADIS

Key
Input
Calculation
Linked Cell
Design FOS
Check Cell

Geometric and Strength Considerations

H = Thickness of Deep Mixed (DM) Zone =	44.8	ft		
B = Width of DM Zone Normal to CL of Levee =	45	ft		
d = diameter of DM column =	72	in. =	6.0	ft
s = center-to-center column spacing =	20	ft		
e = column overlap distance =	12	in. =	1	ft

$A_{columnOL}$ = Area of overlapped column = $(\pi(d/2)^2)*(1-a_e)$ =	26.02	ft^2
A_{PP} = Area of panel in plan = $(Y_{int} - 1)A_{columnOL} + \pi(d/2)^2$ =	236.5	ft^2
A_{PS} = Area of panel in section = HB_{actual} =	2060.8	ft^2
A_{PN} = Area of panel normal to CL of levee = dH =	268.8	ft^2
Y = # of columns to achieve width B = B/(d-e) =	9.00	
Y_{int} = integer # of columns to achieve width B =	9	
$B_{actual} = (Y_{int} - 1)*(d-e) + d$ =	46.00	ft

q_{dm} = specified UCS of deep mixed ground = 110 psi

Column Overlap Calculations

e/d = overlap to diameter ratio =	0.167	(dimensionless)		
α= one-half cord angle = arcos(1-(e/d)) =	0.59	radians		
c = cord length = d(sinα) =	3.32	ft =	39.80	in.
a_e = overlap area ratio = $(2\alpha-\sin2\alpha)/\pi$ =	0.080	(dimensionless)		
a_s = area replacement ratio = $(\pi d(1-a_e))/(4s\cos\alpha)$ =	0.260	(dimensionless)		
b = effective panel width = $a_s s$ =	5.20	ft		

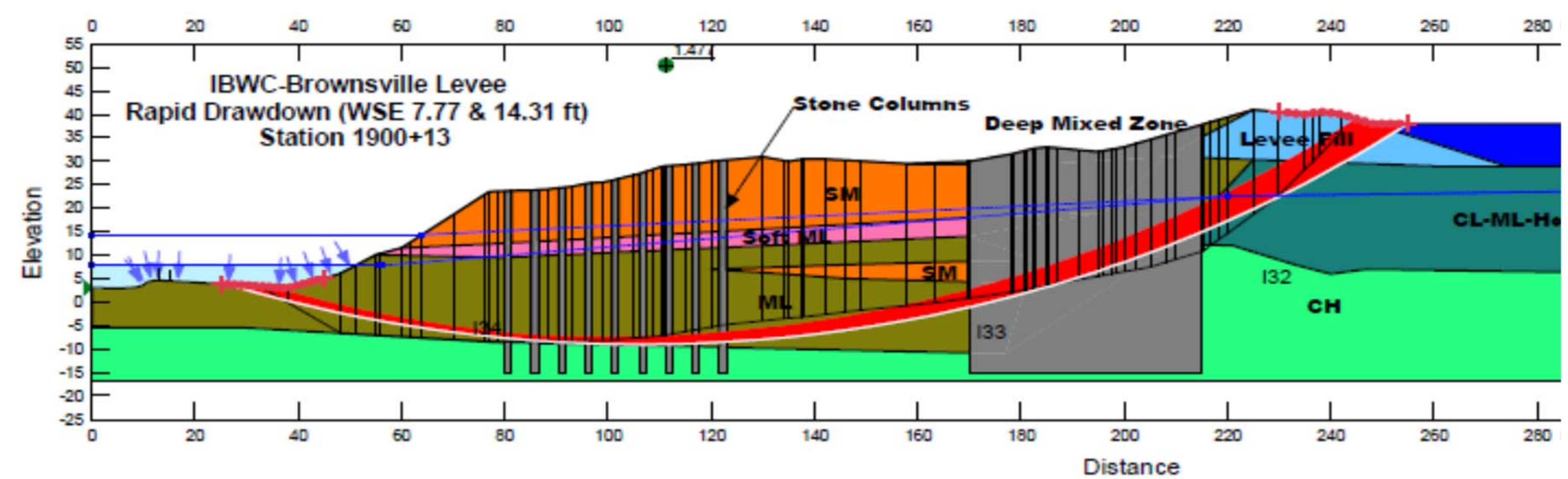
Design Strength of Deep-Mixed Ground

Time between construction and loading:	28 days			
f_c = Ratio of UCS as compared to 28-day UCS =	1	(dimensionless)		
f_d = Design Factor of Safety of Deep Mix Strength =	1.5			
p_{dm} = probability DM strength > specified strength =	80%			
f_v = Values of Variability factor =	0.87	(dimensionless)		
s_{dm} = design DM shear strength = $0.4 f_c f_v q_{dm}$ =	38.28	psi =	5512	psf
s_{dmz} = shear strength of DMZ = $a_s s_{dm}$ =	1434.502	psf		

Summary of Design Factors of Safety

Mode of Failure	Abbrev.	Req. FoS	Actual FoS
Sliding	F_s	1.3	1.56
Overturning	F_o	1.4	1.4
Crushing	F_c	1.4	1.71
Vertical Shearing	F_v	1.4	1.77
Extrusion	F_e	1.3	>10
Global Stability - Spencer's Method	F_g	1.4	1.477

Levee Geometry



Design Stratigraphy

Groundwater EL = 20.3 ft (include strata change at groundwater table)

Top EL DMZ = 32.8 ft

Top EL (ft.)	Thickness (ft)	Bottom Depth(ft)	Material	γ_{moist} (pcf)	γ_{Eff} (pcf)	σ'_{Layer} (psf)	σ'_{base} (psf)	Area within Central Block (ft^2)	Total Weight (lbs/ft)
32.8	2.7	2.7	Levee Fill	127	127	342.9	342.9	137	17399.0
30.1	9.8	12.5	SM Vadose	117	117	1146.6	1489.5	427.8	50052.6
20.3	1	13.5	SM Sat	117	55	54.6	1544.1	97.65	11425.1
19.3	0.3	13.8	Soft ML	126	64	19.08	1563.18	48.42	6100.9
19	17	30.8	ML Sat	119	57	962.2	2525.38	769	91511.0
2	14	44.8	CH	122	60	834.4	3359.78	550.4	67148.8
-12	-							System:	243637.4

Note: Area calculations from GeoStudio model. Area of unsaturated ML (ML Vadose) addedd to SM Vadose for consideration in system weight calculation

For	IBWC - Brownsville Levee Rehab	Job No.	LA003315.0000	Sheet No.	2 of 7
Calc	Mixed Soil Shear Wall Design	Made By	CW	Checked by	KL
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Linked Cell
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Check Cell

Sliding and Overturning Calculations

Assumed trial FoS = 1.4

Active Side Shear Forces

Groundwater EL = 21.5 ft (include strata change at groundwater table)

Material	Top EL (ft)	Thicknes (ft)	Bottom Depth(ft)	OCR (Max=10)	γ_{moist} (pcf)	γ_{Eff} (pcf)	σ'_{Layer} (psf)	σ'_{base} (psf)	c (psf)	c' (psf)	c_m (psf)	c'_m (psf)	ϕ (deg.)	ϕ' (deg.)	ϕ_m (deg.)	ϕ'_m (deg.)	K_o	V_a (lbs/ft)	h_{va} (ft above base)
Levee Fill	37.8	7.1	7.1	7.7	127	127	901.7	901.7	5000	400	3571.42857	285.7143	0	20	0.0	14.6	1.000	25357	45.07
ML Vadose	30.7	9.2	16.3	2.9	119	119	1094.8	1996.5	0	190	0	135.7143	29	0	21.6	0.0	0.935	4938	38.02
ML Sat	21.5	3.02	19.32	2.1	119	56.6	170.932	2167.432	0	190	0	135.7143	29	0	21.6	0.0	1.000	410	31.99
CL Sat	18.48	6.3	25.62	1.7	123	60.6	381.78	2549.212	400	200	285.714286	142.8571	0	14	0.0	10.1	0.905	3296	27.32
CH	12.18	24.18	49.8	1.1	122	59.6	1441.128	3990.34	2320	150	1657.14286	107.1429	0	16	0	11.6	0.817	15823	11.68
	-12	49.8															System:	49823	22.13

Active Side Weight Force

Groundwater EL = 22 ft (include strata change at groundwater table)

Material	Top EL (ft)	Area (ft^2)	Area %	γ_{moist} (pcf)	γ_{Eff} (pcf)	Total $W_{stratum}$ (lbs/ft)	c (psf)	c' (psf)	c_m (psf)	c'_m (psf)	ϕ (deg.)	ϕ' (deg.)	ϕ_m (deg.)	ϕ'_m (deg.)	$L_{failure\ plane}$ (ft)	$L_{failure\ plane}$ (%)	Weighted avg ϕ'_m	$H_{active\ face}$ (ft)	Weighted avg c'_m (psf/ft)
Hist. Fill	38	60.46	4.20	127	127	7678.42	400	200	285.7143	142.85714	15	20	10.8	14.6	8.2	11.0	1.2	0	0
Levee Fill	40.5	411.51	28.62	127	127	52261.77	5000	400	3571.429	285.71429	0	20	0.0	14.6	5.2	7.0	0.0	7.1	25357.14286
CL Vadose	30.3	273.92	19.05	123	123	33692.16	400	200	285.7143	142.85714	0	14	0.0	10.1	9.9	13.3	0.0	0	0
CL Sat	22	363.30	25.26	123	60.6	44685.9	400	200	285.7143	142.85714	0	14	0.0	10.1	22.5	30.3	3.1	6.3	900
ML Vadose	30.7	57.15	3.97	119	119	6800.85	0	190	0	135.71429	29	0	21.6	0.0	0	0.0	0.0	9.2	0
ML Sat	22	3.98	0.28	119	56.6	473.62	0	190	0	135.71429	29	0	21.6	0.0	0	0.0	0.0	3.02	409.8571429
SM Vadose	31	0.00	0.00	117	117	0	0	0	0	0	32	32	24.1	24.1	0	0.0	0.0	0	0
SM Sat	22	0.00	0.00	117	54.6	0	0	0	0	0	32	32	24.1	24.1	0	0.0	0.0	0	0
Soft ML	19	0.00	0.00	126	63.6	0	168	150	120	107.14286	0	0	0.0	0.0	0	0.0	0.0	0	0
CH	12	267.68	18.61	122	59.6	32656.96	2320	150	1657.143	107.14286	0	16	0.0	11.6	28.43	38.3	4.4	24.18	2590.714286
	A_{total} =	1438	ft^2		W_a =	178249.68	lbs/ft							System:	74.23		8.7	49.8	588

Note: Area calculations from GeoStudio model

α_a = measured angle from horizontal of base of active wedge = 42 °

D_a = active wedge driving force, considering no shear resistence = $W_a \tan\alpha_a$ = 160496.7 lbs/ft

U_a = active wedge uplift force = $(U_{Ha}/2)*L_{ua}$ = 39750.75 lbs/ft

where L_{ua} = measured length of active wedge uplift force = 37.75 ft

R_a = active wedge resisting force = $2(W_a - U_a \sin(90-\alpha_a))\tan\phi'_{m\ avg} + 2c'_{m\ avg} H_a \tan(90-\alpha_a)$ = 110451.5 lbs/ft

P_a = active side pressure considering shear resistence = $D_a - R_a$ = 50045.21 lbs/ft

For	IBWC - Brownsville Levee Rehab	Job No.	LA003315.0000	Sheet No.	3 of 7
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Check Cell

Sliding and Overturning Calculations (cont.)

Assumed trial FoS = 1.4

Passive Side Shear Forces

Groundwater EL : 19.1 ft (include strata change at groundwater table)

Material	Top EL (ft)	Thicknes	Bottom	OCR	γ_{moist} (pcf)	γ_{Eff} (pcf)	σ'_{Layer} (psf)	σ'_{base} (psf)	c (psf)	c' (psf)	c_m (psf)	c'_m (psf)	ϕ (deg.)	ϕ' (deg.)	ϕ_m (deg.)	ϕ'_m (deg.)	K_o	V_p (lbs/ft)	h_{vp} (ft above
Levee Fill	30	0.5	0.5	10.0	127	127	63.5	63.5	5000	400	3571.42857	285.7143	0	20	0.0	14.6	1.000	1786	41.67
SM Vadose	29.5	10.4	10.9	5.2	117	117	1216.8	1280.3	0	0	0	0	32	32	24.1	24.1	1.162	3625	35.45
SM Sat	19.1	1.1	12	3.0	117	54.6	60.06	1340.36	0	0	0	0	32	32	24.1	24.1	0.921	593	30.55
Soft ML	18	4	16	2.5	126	63.6	254.4	1594.76	168	150	120	107.1429	0	0	0.0	0.0	1.000	429	27.99
ML Sat	14	5.4	21.4	2.0	119	56.6	305.64	1900.4	0	190	0	135.7143	29	0	21.6	0.0	1.000	733	23.29
SM Sat	8.6	4.3	25.7	1.6	117	54.6	234.78	2135.18	0	0	0	0	32	32	24.05289	24.1	0.725	2806	18.43
ML Sat	4.3	15.1	40.8	1.2	119	56.6	854.66	2989.84	0	190	0	135.7143	29	0	21.60035	0.0	1.000	2049	8.66
CH	-10.8	1.2	42	1.0	122	59.6	71.52	3061.36	2320	150	1657.14286	107.1429	0	16	0	11.6	0.805	727	0.60
	-12	42															System:	12747	18.67

Passive Side Weight Force

Groundwater EL = 11.25 ft (include strata change at groundwater table)

Material	Top EL (ft)	Area (ft^2)	Area %	γ_{moist} (pcf)	γ_{Eff} (pcf)	Total $W_{stratum}$ (lbs/ft)	c (psf)	c' (psf)	c_m (psf)	c'_m (psf)	ϕ (deg.)	ϕ' (deg.)	ϕ_m (deg.)	ϕ'_m (deg.)	$L_{failure\ plane}$ (ft)	$L_{failure\ plane}$ (%)	Weighted avg ϕ'_m	$H_{passive\ face}$ (ft)	Weighted avg c'_m (psf/ft)
Levee Fill	30	3.00	0.08	127	127	381	5000	400	3571.429	285.71429	0	20	0.0	14.6	0	0.0	0.0	0.5	1785.714286
SM Vadose	29.5	1253.01	32.27	117	117	146602.17	0	0	0	0	32	32	24.1	24.1	0	0.0	0.0	10.4	0
SM Sat	11.25	59.96	1.54	117	54.6	7015.32	0	0	0	0	32	32	24.1	24.1	0	0.0	0.0	1.1	0
Soft ML	11.25	365.00	9.40	126	63.6	45990	168	150	120	107.14286	0	0	0.0	0.0	0	0.0	0.0	4	428.5714286
ML Sat	10	2048.22	52.75	119	56.6	243738.18	0	190	0	135.71429	29	0	21.6	0.0	113.3	85.3	0.0	5.4	732.8571429
SM Sat	9	125.50	3.23	117	54.6	14683.5	0	0	0	0	32	32	24.1	24.1	0	0.0	0.0	4.3	0
ML Sat	10	0.00	0.00	119	56.6	0	0	190	0	135.71429	29	0	21.6	0.0	0	0.0	0.0	15.1	2049.285714
CH	-10.8	28.50	0.73	122	59.6	3477	2320	150	1657.143	107.14286	0	16	0.0	11.6	19.5	14.7	1.7	1.2	128.5714286
	A_{total} =	3883.19	ft^2		W_p =	461887.17	lbs/ft							System:	132.8		1.7	42	122

Note: Area calculations from GeoStudio model

α_p = measured angle from horizontal of base of passive wedge = 6.5 °

D_p = passive wedge driving force, considering no shear resistence = $W_p \tan\alpha_p$ = 52625.4 lbs/ft

U_p = passive wedge uplift force = $(U_{Hp}/2) * L_{up}$ = 111917.5 lbs/ft

where L_{up} = measured length of active wedge uplift force = 132 ft

R_p = passive wedge resisting force = $2(W_p - U_p\cos\alpha_p)\tan\phi'_{m\ avg} + 2c'_{m\ avg} H_p \tan(90-\alpha_p)$ = 32196.04 lbs/ft

P_p = passive side pressure considering shear resistence = $D_p - R_p$ = 20429.36 lbs/ft

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Sliding and Overturning Calculations (cont.)

N = Resultant vertical force = W + V_a - V_p = 280713.4 lbs/ft

U = uplift force = H_{H20}*γ_{H20}*B_{actual} = 92713.92 lbs/ft

N' = Effective resultant vertical force = N - U = 187999.5 lbs/ft

x_W = location of DMZ weight force = 23.45 ft

h_p = height of action of passive force = 15.45 ft

h_a = height of action of active force = 18.30 ft

Note: See page 6 for x_w calculation; page 7 for h_a and h_p calculations

x_N = location of resultant N = B_{actual} - (P_ph_p + Wx_w + V_aB_{act} - P_ah_a) / N = 19.62 ft

x_U = location of uplift force = x center of hydrostatic gravity in central block = 23.28 ft

$x_{N'}$ = location of effective vertical force = (Nx_N - Ux_U)/N' = 17.82 ft

T = shear force along base = $2x_{N'}c'_m$ + N'tanϕ'_m = 42323.65 lbs/ft

Where:

Base Strata	c (psf)	c' (psf)	c_m (psf)	c'$_m$ (psf)	ϕ (deg.)	ϕ' (deg.)	ϕ_m (deg.)	ϕ'_m (deg.)
CH	2320	150	1657.143	107.1429	0	16	0.0	11.6

ΣP_H = Sum of horizontal Forces = P_p + T - P_a = 12708 lbs/ft

Check: If ΣP_H > 0 then trial FOS < $F_{s\ Actual}$ TRUE? TRUE

Use Goal Seek to determine FOS such that ΣP_H = 0

$FOS_{\Sigma Ph=0}$ = 1.56

From page 1:

a_s = area replacement ratio = (πd(1-a_e))/(4scosα) = 0.260236 (dimensionless)

b = effective panel width = a_ss = 5.20 ft

s = center-to-center column spacing = 20 ft

B_{actual} = (Y_{int} - 1)*(d-e) + d = 46.00 ft

B_{actual}/3 = 15.33 ft

B_{actual}/2 = 23.00 ft

q_{toe} = (N'/b)*[(2s/3$x_{N'}$)-((s-b)/B_{actual})] for $x_{N'}$ ≤ B_{actual}/3

q_{toe} = (N'/B_{actual})*[(3s/b)*(1 - (2$x_{N'}$/B_{actual})) + 1] for B_{actual}/3 ≤ $x_{N'}$ ≤ B_{actual}/2

q_{toe} = 14703.89 psf

Bearing Capacity Analysis (Hansen Method)

Bottom Elevation of DSM = -12 ft

Average Ground Elevation = 32.8 ft

Trial FoS = 1.4

γ_{above} = weighted average of soil unit weight above DMZ = 75.0 pcf

γ_{below} = unit weight of soil below DMZ = 59.6 pcf

D = depth of embedment to base of DMZ = 44.8 ft

e_B = eccentricity normal to CL of levee = M_B/Q = (B_{actual}/2) - $x_{N'}$ = 5.2 ft

L' = Effective panel length = B_{actual} - 2e_B = 35.6 ft

From Tables 4-4 and 4-5 EM 1110-1-1905:

N_c =Hansen cohesion factor = 9.08

N_γ = Hansen soil wedge weight factor = 0.58

N_q =Hansen surcharge factor = 2.86

ζ_{sc} = Hansen shape with eccentricity cohesion factor = 1.05

0.2b/L' for ϕ'_m = 0 1 + (b/L')(N_q/N_c) for ϕ'_m > 0

$\zeta_{s\gamma}$ = Hansen shape with eccentricity soil wedge factor = 0.942

1 for ϕ'_m = 0 1 - (0.4)(b/L') for ϕ'_m > 0

ζ_{sq} = Hansen shape with eccentricity surcharge factor = 1.03

1 for ϕ'_m = 0 1 + (b/L')tanϕ'_m for ϕ'_m > 0

k = depth reference factor = 0.974

D/B_{actual} for D/B_{actual} ≤ 1 $\tan^{-1}$(D/B_{actual}) for D/B_{actual} > 1

ζ_{dc} = Hansen depth cohesion factor = 1.39

0.4k for ϕ'_m = 0 1 + 0.4k for ϕ'_m > 0

$\zeta_{d\gamma}$ = Hansen depth soil wedge factor = 1

for all conditions

ζ_{dq} = Hansen depth surcharge factor = 1.25

1 for ϕ'_m = 0 1+2tanϕ*(1-sinϕ)^{2}k for ϕ'_m > 0

q_u = q_{all} = ultimate bearing capacity = $c'_mN_c\zeta_{sc}\zeta_{dc}$ + 0.5L'$\gamma_{below}N_\gamma\zeta_{s\gamma}\zeta_{d\gamma}$ + $D\gamma_{above}N_q\zeta_{sq}\zeta_{dq}$ =

q_u = q_{all} = ultimate bearing capacity = 14429 psf

Check: If q_{toe} > q_{all} then trial FOS > $F_{O\ Actual}$ TRUE? TRUE

Use Goal Seek to determine FOS such that q_{toe} - q_{all} = 0

$FOS_{q\ toe\ -\ q\ all\ =\ 0}$ = 1.40

Note: Bearing capacity calculations do not consider slope, base tilt, or inclined load. Assumes DMZ is below groundwater level and assumes linear relation between bearing capacity correction factors listed in EM-1110-1-1905 Table 4-4

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Input
Calculation
Linked Cell
Design FOS
Check Cell

Crushing of the Deep Mixed Ground

Trial FOS =	1.4	
ϕ'_m =	11.6	°
K_o = at-rest earth pressure = $(K_{o\,a} + K_{o\,p})/2$ =	0.811	psf
σ'_v = effective vertical stress at base of DMZ = $(\sigma'_{v\,a} + \sigma'_{v\,p})/2$ =	3526	psf
σ'_h = effective horizontal stress at base of DMZ = $K_o\sigma'_{v=}$	2859	psf
s_{dm} = design DM shear strength = $0.4f_cf_vq_{dm}$ =	5512	psf
$q_{toe\,eq2} = (2s_{dm}/F_c) + \sigma_h$ =	10734	psf

Check: if $q_{toe} > q_{toe\,eq2}$ then trial FOS < $F_{c\,Actual}$ TRUE? TRUE

Use Goal Seek to determine FOS such that $q_{toe} - q_{toe\,eq2} = 0$

$FOS_{q\,toe - q\,toe\,eq2 = 0}$ = 1.705

Shearing on Vertical Planes

τ_v = $(V_p/H) + (N/H)(1 - (3x_N/2B_{actual}))^2$ for $x_N \le B_{actual}/3$

$(V_p/H) + (3N/4H)(1 - (2x_N/B_{actual}))$ for $B_{actual}/3 \le x_N \le B_{actual}/2$

B_{actual} =	46.00	ft		
$B_{actual}/3$ =	15.33	ft		
$B_{actual}/2$ =	23.00	ft		
x_N =	19.62	ft		
H =	44.8	ft		
V_p =	12747	lbs/ft =	12.7	kips/ft
N =	280713.4	lbs/ft =	280.7	kips/ft
τ_v =	0.975	ksf =	975	psf

F_v = FoS against vertical shear = $s_{dmz,\,v}/\tau_v$

where $s_{dmz,\,v} = (c/s)s_{dm}$ =	914.1149	psf
c = cord length = d(sinα) =	3.32	ft
s = center-to-center column spacing =	20	ft
$s_{dmz,\,v}/FOS_{trial}$ =	652.9392	psf

Check: if $s_{dmz,\,v}/FOS_{trial} < \tau_v$, then trial FOS < $F_{v\,Actual}$ TRUE? TRUE

Use Goal Seek to determine FOS such that $\tau_v - (s_{dmz,\,v}/FOS) = 0$

$FOS_{\tau v - (sdmz,\,v/FOS) = 0}$ = 1.77

Extrusion of Soft Ground Between Shear Walls

F_e = Factor of Safety against extrusion = $(2c_e(2+B_{actual}((1/(s-b))+(1/H_e)))/(\sigma_{vf} - \sigma_{vp})$

c_e = average value of total stress cohesion intercept =	CH	2320	psf
H_e = thickness of layer of soft clay to be analyzed for extrusion =		24.18	ft
σ_{vf} = average total vertical stress at base on active side of DMZ =		6080.74	psf
σ_{vp} = average total vertical stress at base on passive side of DMZ =		5002	psf
F_e = Factor of Safety against extrusion =		30	
F_e = Factor of Safety against extrusion for infinite spacing =		17	

Note: Internal formulas for σ_{vf} and σ_{vp} are correct only for the given stratigraphy. If stratigraphy is altered, the internal *formulas must be hand-altered, as well.*

For	IBWC - Brownsville Levee Rehab	Job No.	LA003315.0000	Sheet No.	6 of 7
Calc	Mixed Soil Shear Wall Design	Made By	CW	Checked by	KL
		Date	12/19/2017	Date	12/23/2017

ARCADIS

Key
Input
Calculation
Linked Cell
Design FOS
Check Cell

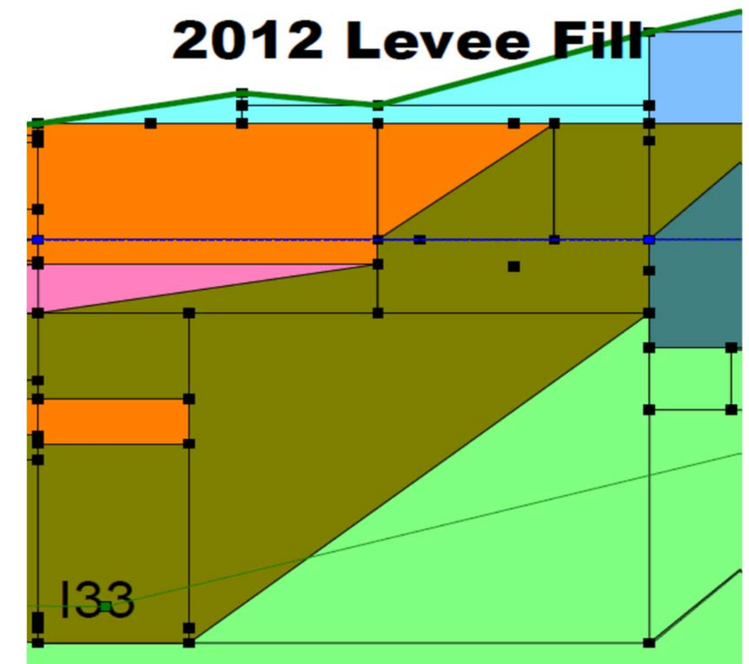
DMZ Center of Gravity Calculations

Groundwater EL = 21 ft (include strata change at groundwater table) $X_{point\ O}$ = 170

i	Material	Top EL (ft)	Area (ft^2)	Area %	γ_{moist} (pcf)	γ_{Eff} (pcf)	W'$_i$ (lbs/ft)	x $_{i1}$	x $_{i2}$	Shape	$x_{COG\ i}$	x $_i$	W_ix_i
1	Levee Fill	33	18.75	0.92	127	127	2381.25	170	185	Left Tri	10.005	10.005	23824.406
2	Levee Fill	33	5.00	0.25	127	127	635	185	195	Right Tri	3.33	18.33	11639.55
3	Levee Fill	32	45.00	2.21	127	127	5715	185	215	Rectangle	15	30	171450
4	Levee Fill	37.8	58.00	2.84	127	127	7366	195	215	Left Tri	13.34	38.34	282412.44
5	SM Vadose	30.5	237.45	11.64	117	117	27781.65	170	195	Rectangle	12.5	12.5	347270.63
6	SM Vadose	30.5	61.70	3.03	117	117	7218.9	195	208	Right Tri	4.329	29.329	211723.12
7	SM Sat	21	49.99	2.45	117	54.6	2729.454	170	195	Rectangle	12.5	12.5	34118.175
8	SM Sat	8	41.07	2.01	117	54.6	2242.422	170	181.1	Rectangle	5.55	5.55	12445.442
9	Soft ML	19	50.00	2.45	126	63.6	3180	170	195	Right Tri	8.325	8.325	26473.5
10	ML Vadose	30.5	61.75	3.03	119	119	7348.25	195	208	Left Tri	8.671	33.671	247422.93
11	ML Vadose	30.5	66.50	3.26	119	119	7913.5	208	215	Rectangle	3.5	41.5	328410.25
12	ML Sat	19	50.00	2.45	119	56.6	2830	170	195	Left Tri	16.675	16.675	47190.25
13	ML Sat	21	120.00	5.88	119	56.6	6792	195	215	Rectangle	10	35	237720
14	ML Sat	15	77.70	3.81	119	56.6	4397.82	170	181.1	Rectangle	5.55	5.55	24407.901
15	ML Sat	15	457.65	22.44	119	56.6	25902.99	181.1	215	Right Tri	11.2887	22.3887	579934.27
16	ML Sat	4.3	180.90	8.87	119	56.6	10238.94	170	181.1	Rectangle	5.55	5.55	56826.117
17	CH	15	457.65	22.44	122	59.6	27275.94	181.1	215	Left Tri	22.6113	33.7113	919507.4
		A_{total} =	2039.11	ft^2		W_{system} =	151949.1	lbs/ft				ΣW_ix_i =	3562776.4

$x_w = \Sigma w_ix_i / W_{system}$ = 23.44717

Generalized DMZ Geometry for Center of Gravity Calculations



For	IBWC - Brownsville Levee Rehab	Job No.	LA003315.0000	Sheet No.	7 of 7
Calc	Mixed Soil Shear Wall Design	Made By	CW	Checked by	KL
		Date	12/19/2017	Date	12/23/2017

ARCADIS

Key
Input
Calculation
Linked Cell
Design FOS
Check Cell

Active Side Vertical Stress Distribution

Groundwater 21.5 (include strata change at groundwater table)

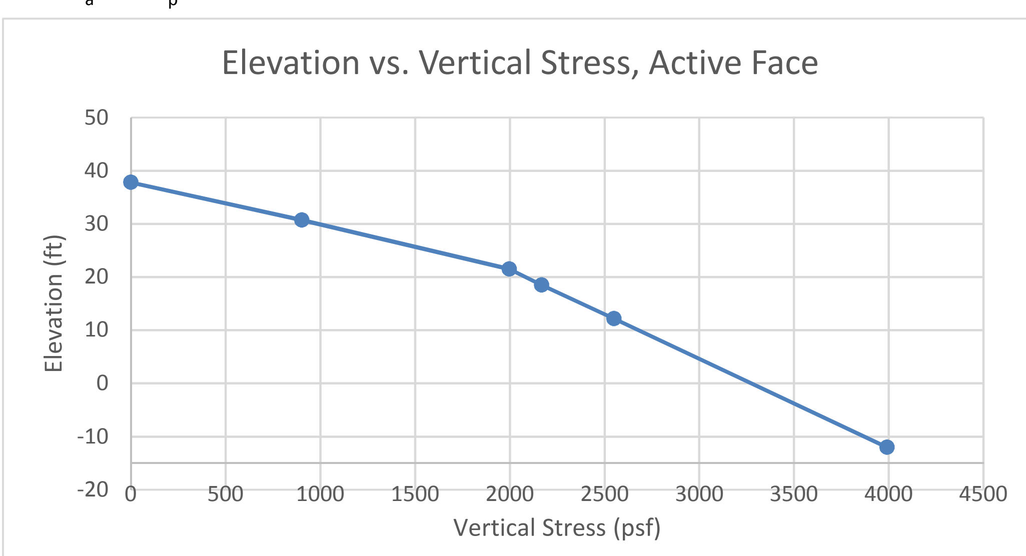
	Top EL (ft)	Thicknes (ft)	Bottom Depth(ft)	γ_{moist} (pcf)	γ_{Eff} (pcf)	σ'_{Layer} (psf)	σ'_{base} (psf)	AUC ob (lbs/ft)	AUC I (lbs/ft)	y COG ob (ft)	y COG i (ft)	AUC t (psf)	% area (%/100)	y i (ft)	Wt avg y i gam (ft)
Levee Fill	37.8	7.1	7.1	127	127	901.7	901.7	0	3201	0.00	45.07	3201	0.03	45.0667	1.235728
ML Vadose	30.7	9.2	16.3	119	119	1094.8	1996.5	8295.64	5036	38.10	36.57	13332	0.11	37.5208	4.284844
ML Sat	21.5	3.02	19.32	119	56.6	170.932	2167.432	6029.43	258	31.99	31.49	6288	0.05	31.9693	1.721834
CL Sat	18.48	6.3	25.62	123	60.6	381.78	2549.212	13654.82	1203	27.33	26.28	14857	0.13	27.2450	3.467429
CH	12.18	24.18	49.8	122	59.6	1441.128	3990.34	61639.95	17423	12.09	8.06	79063	0.68	11.2019	7.586529
	-12	49.8										116741			18.29636

Passive Vertical Stress Distribuiton (include strata change at groundwater table)

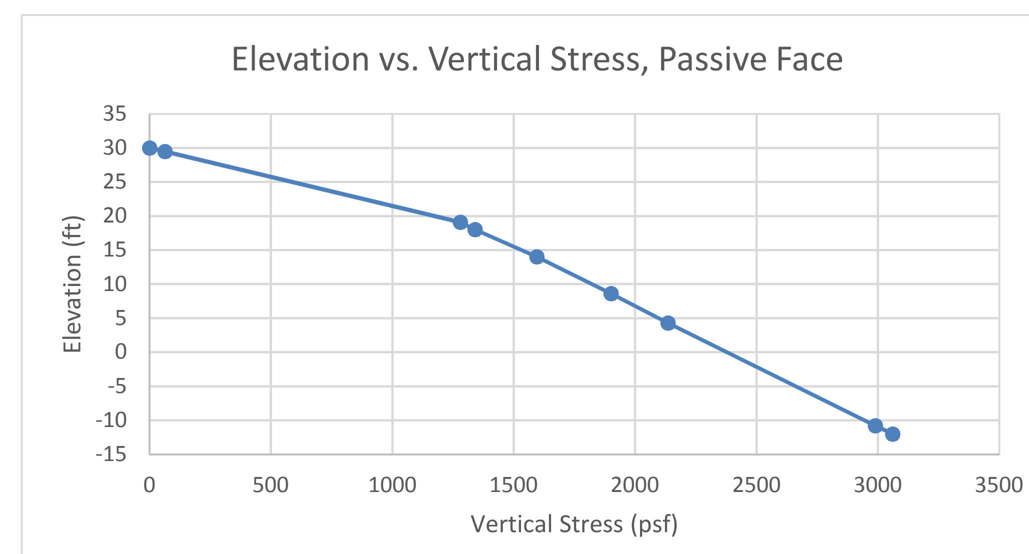
Groundwater EL : 19.1 ft

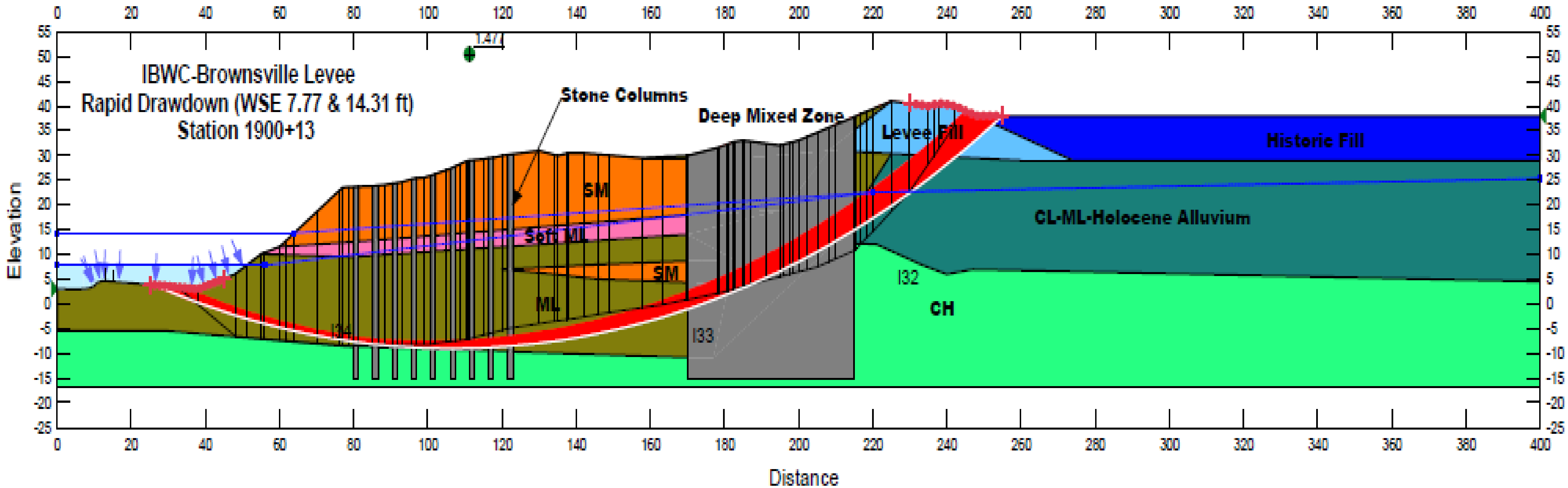
Material	Top EL (ft)	Thicknes (ft)	Bottom Depth(ft)	γ_{moist} (pcf)	γ_{Eff} (pcf)	σ'_{Layer} (psf)	σ'_{base} (psf)	AUC ob (lbs/ft)	AUC i (lbs/ft)	y COG ob (ft)	y COG i (ft)	AUC t (lbs/ft)	% area (%/100)	y i (ft)	Wt avg y i gam (ft)
Levee Fill	30	0.5	0.5	127	127	63.5	63.5	0	16	0.00	41.67	15.9	0.00	41.6665	0.008849
SM Vadose	29.5	10.4	10.9	117	117	1216.8	1280.3	660.4	6327	36.30	34.56	6987.8	0.09	34.7273	3.24623
SM Sat	19.1	1.1	12	117	54.6	60.06	1340.36	1408.33	33	30.55	30.37	1441.4	0.02	30.5458	0.588972
Soft ML	18	4	16	126	63.6	254.4	1594.76	5361.44	509	28.00	27.33	5870.2	0.08	27.9421	2.194243
ML Sat	14	5.4	21.4	119	56.6	305.64	1900.4	8611.704	825	23.30	22.40	9436.9	0.13	23.2211	2.931461
SM Sat	8.6	4.3	25.7	117	54.6	234.78	2135.18	8171.72	505	18.45	17.73	8676.5	0.12	18.4082	2.136614
ML Sat	4.3	15.1	40.8	119	56.6	854.66	2989.84	32241.22	6453	8.75	6.23	38693.9	0.52	8.3295	4.311515
CH	-10.8	1.2	42	122	59.6	71.52	3061.36	3587.808	43	0.60	0.40	3630.7	0.05	0.5976	0.029027
	-12	42										74753.3			15.44691

Note: h_a and h_p were assessed as vertical distance from base of DMZ to center of vertical stress distribution of active and passive faces, respectively, considering appropriate (effective vs. total) overburden stresses.



Nomenclature
AUC = Area Under Curve
ob = due to overburden
i = over depth interval
COG = center of gravity
t = total
gam = considering unit weight





Minimum Factor of Safety (FOS): 1.477

Material	Unit Weight (pcf)	c' (psf)	phi' (degree)	Total Stress	
				c (psf)	phi (deg.)
CH Pleistocene	121.98	150	16	2320	0
CL-Holocene	123.37	200	14	400	0
CL	120	300	0	300	0
SM	117	0	32	0	32
ML	119.38	190	0	0	29
Levee Fill	127.34	400	20	5000	0
Historic Fill	127.34	200	20	400	15
Soft ML	125.98	150	0	168	0
Stone Columns	127	0	40	0	40
Deep Mixed Zone	120	1435	0	1435	0

Rapid Drawdown Conditions

IBWC
STATION 1903+96

REMEDIATION DESIGN OF LEVEE FLOODPLAIN FAILURE WITHIN THE UPPER BROWNSVILLE LEVEE REACH LOWER RIO GRANDE FLOOD CONTROL PROJECT

SLOPE STABILITY MODEL - FAILURE WITHIN LEVEE

RAPID DRAWDOWN (WSE 7.77 FT & 14.31 FT)

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APPENDIX

ATTACHMENT C

DESIGN CALCULATIONS

C.1 GEOTEXTILE
C.2 SOIL FILTER
C.3 ARTICULATED CONCRETE BLOCK MATRESS (ACBM)
C.4 ROCK RIPRAP
C.5 GATEWELL STRUCTURE 205

ATTACHMENT C

DESIGN CALCULATIONS

C.1 GEOTEXTILE
C.2 SOIL FILTER
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Geotextile Calculations

Reference: Designing with Geotextiles, Second Edition
Robert M. Koerner

ASTM E 11

Geotextile Beneath Rip Rap and PreCast Concrete Blocks

Insitu Soil Conditions

Base Soil - Category 1 [from filter calculations, 1]

$d_{10} < 0.001$ mm $\quad d_{60} = 0.03$ mm $\quad d_{85} = 0.05$ mm

$CU = \frac{d_{60}}{d_{10}} = \frac{0.03}{0.001} = 30 \qquad d_{50} = 0.02$ mm

medium dense condition - relative density $D_R \approx 80\%$

porosity, $n = 0.42$

% Passing #200 sieve = 90%

note that CU does not really work for Clay soils

SUBJECT: Levee Geotextile
PREPARED BY LSMorr DATE 17Dec15 CHECKED BY APPROVED BY JOB NUMBER 4839-01

Base Soil - Category 2 [from filter calculations, 2]

$d_{10} = 0.06$ mm $d_{60} = 0.1$ mm $d_{85} = 0.19$ mm

$CU = \frac{d_{60}}{d_{10}} = \frac{0.1}{0.06} = 1.7$ ✓ $d_{50} = 0.085$ mm

medium dense condition $D_R \approx 80\%$

porosity, $\eta = 0.41$

% Passing #200 sieve = 40%

Base Soil - Category 3 [from filter calculations, 3A]

$d_{10} = 0.06$ mm $d_{60} = 0.1$ mm $d_{85} = 0.4$ mm

$CU = \frac{d_{60}}{d_{10}} = \frac{0.1}{0.06} = 1.7$ ✓ $d_{50} = 0.085$ mm

medium dense condition $D_R \approx 80\%$

porosity, $\eta = 0.39$

% Passing #200 sieve = 50%

DESIGN NOTES AND COMPUTATIONS

SUBJECT: Levee Geotextile

PREPARED BY LSMorr | DATE 17Dec15 | CHECKED BY [illegible] | APPROVED BY | JOB NUMBER 4839-01

Filter Soil

$d_{10} = 0.2\,mm$ $d_{60} = 1.0\,mm$ $d_{85} = 6\,mm$

$CU = \frac{d_{60}}{d_{10}} = \frac{1.0}{0.2} = 5$ ✓ $d_{50} = 0.7\,mm$

medium dense condition $D_R \approx 70\%$

porosity, $n = 0.35$ estimate

% Passing #200 Sieve = 5%

SUBJECT: Levee Geotextile				
PREPARED BY LSMorr	DATE 18 Dec 15	CHECKED BY	APPROVED BY	JOB NUMBER 4839-01
				SHEET NO. 4 OF

Determine Geotextile Placed on Exposed Soil [Category 1, 2, & 3]

Requirements: Adequate flow through the Geotextile A.

Adequate soil retention by the Geotextile B.

Downstream of the East International Bridge

See Page 4A

Lake Brown Water Elevation = 25 ft
100-year flood Elevation = 36.5 ft
High Water River Elevation = 15 ft
Typical Water River Elevation = 7.5 ft
Typical distance from Lake to River ≈ 500 ft

Time: River water Level to recede from El 36.5 to 7.5 1-2 wks

River water Level to recede from El 15 to 7.5 1-7 days

Assume: 1 hour

Geotextile placed on exposed / graded to 5:1 natural soil

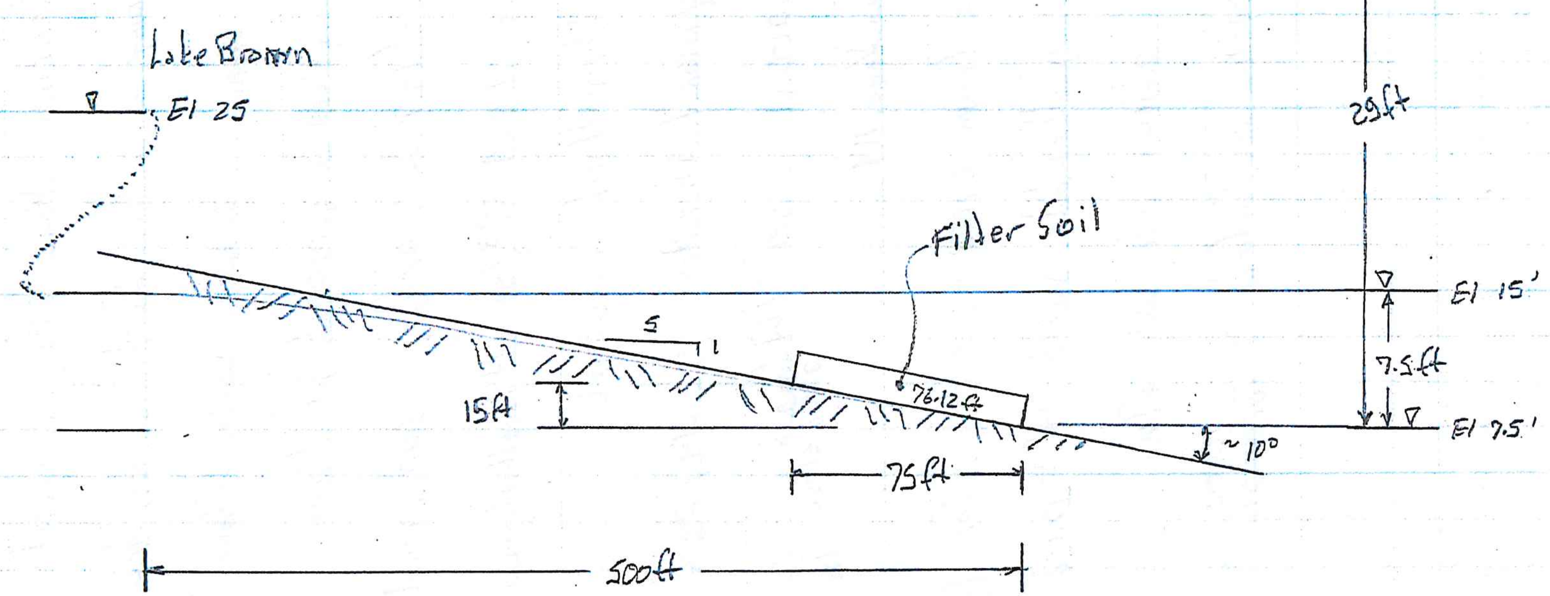
DESIGN NOTES AND COMPUTATIONS

SUBJECT: Levee Geotextile
PREPARED BY: LSMarr
DATE: 18Dec15
CHECKED BY:
APPROVED BY:
JOB NUMBER: 4839.01
SHEET NO. 9A OF 12

Area per foot length $= 500 \times 7.5 \times 1 = 3750$ sf/ft of length

Area per foot length $= 500.8 \times 29 \times 1 = 14523$ sf/ft of length

Area per foot length $= 76.12 \times 15 \times 1 = 1142$ sf/ft of length



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SUBJECT: Levee Geotextile				
PREPARED BY LSMarr	DATE 18Dec15	CHECKED BY [illegible]	APPROVED BY	JOB NUMBER 4839-01
				SHEET NO. 5 OF 12

Reference: Koerner Page 218 Example Problem

Use Category 1 2 + 3 Soils [existing soils] + 100-year flood

A. Determine q_{max} seepage water flow toward river

q_{max} = (Area x porosity) / time see Page 4A

q_{max} = (14523 sf x 0.40) / 3600 seconds = 1 hour

q_{max} = 1.614 ft³/sec - ft. length along slope

Determine Permittivity required

$\Psi_{ult} = q_{max} / \Delta h \times Area = 1.614 / 29 \times 1500$

$\Psi_{ult} = 0.00011$ sec⁻¹/ft length along slope assuming all seepage occurs in 1 hour

Table 2.11 on Page 119

Ψ ranges from 0.02 - 2.2 sec⁻¹ for typical geotextiles

Therefore: geotextile with $\Psi = 0.02$ sec⁻¹ or greater will allow adequate flow

Determine Factor of Safety = $\frac{0.02}{0.00011}$ = 182

SUBJECT: Levee Geotextile				
PREPARED BY LSMarr	DATE 18 Dec 15	CHECKED BY [illegible]	APPROVED BY	JOB NUMBER 4839-01
				SHEET NO. 6 OF 12

∴ Category 1, 2, +3 soils and 7.5 foot change in river level

A Determine q_{max} seepage of water flow toward river

$$q_{max} = \left(\frac{\text{Area}}{\text{ft length}} \times \text{porosity}\right) / \text{time}$$

$$q_{max} = \left(\frac{3750\ cf}{\text{ft length}} \times 0.40\right) / 3600 \text{ seconds} \qquad = 1 \text{ hour}$$

$q_{max} = 0.417\ ft^3/\text{sec - ft. length along slope for 1 hour}$

Determine Permittivity required

$\Psi_{ult} = q_{max} / \Delta h \times \text{Area} = 0.417 / 7.5 \times 500$

$\Psi_{ult} = 0.00011\ sec^{-1}/\text{ft length along slope}$, assuming all seepage occurs in 1 hour

Table 2.11 on Page 114

Ψ ranges from 0.02 - 2.2 sec^{-1} for typical geotextiles

Therefore: geotextile with $\Psi = 0.02\ sec^{-1}$ or greater will allow adequate flow

$$\text{Factor of Safety} = \frac{0.02}{0.00011} = 182.$$

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DESIGN NOTES AND COMPUTATIONS

SUBJECT: Levee Geotextile
PREPARED BY: LSMarr DATE: 18 Dec 15 CHECKED BY: NF APPROVED BY: JOB NUMBER: 1839-01
SHEET NO. 7 OF 12

B. Determine Soil Retention see Page 121

Method 1

Category 1 90% passing #200 sieve AOS ≥ #50 sieve $O_{95} < 0.297$ mm

Category 2 40% " "

Category 3 50% " " AOS ≥ #30 sieve $O_{95} < 0.59$ mm

Method 2 $O_{95} < (2 \text{ or } 3)\, d_{85}$

Category 1 $d_{85} = 0.05$ m $O_{95} < 0.1$ to 0.15 mm

Category 2 $d_{85} = 0.14$ mm $O_{95} < 0.28$ to 0.42 mm

Category 3 $d_{85} = 0.4$ mm $O_{95} < 0.8$ to 1.2 mm

Method 3 use relative density $D_R = 80\%$

Category 1 $d_{50} = 0.02$ $CU = 30?$ $O_{95} < \frac{13.5 \times 0.02}{30} = 0.009$ mm?

Category 2 $d_{50} = 0.085$ $CU = 1.7$

Category 3 $d_{50} = 0.085$ $CU = 1.7$ $O_{95} < 1.5 \times 1.7 \times 0.085 = 0.217$ mm

Category 1 O_{95} varies from $0.009^{?}$ mm to 0.297 mm

Category 2 O_{95} varies from 0.217 mm to 0.59 mm

Category 3 O_{95} varies from 0.217 mm to 1.2 mm

use #70 sieve → 0.212 mm #16 sieve → 1.18 mm Table 1 ASTM E11.

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DESIGN NOTES AND COMPUTATIONS

SUBJECT: Levee Geotextile

PREPARED BY: LSMarr | DATE: 21Dec15 | CHECKED BY: W | APPROVED BY: | JOB NUMBER: 4839-01

SHEET NO. 8 OF 12

Candidate Geotextile [IBWC Spec 31.05.19 Type 2]

Weight: 6 oz/sy min

Permittivity Ψ = 0.5 sec^{-1} minimum

Flow Rate: 75 gpm/sf

Apparent Opening Size AOS: 80-120 sievesize (0.18 - 0.125 mm)

Permittivity Ψ

Typical geotextiles have permittivity values ranging from Ψ = 0.02 to 2.2 sec^{-1} Ref. Koerner Page 88

Permeability kn: range from 0.0016 to 0.46 ft/min

ft/min / 60secs 0.000027 - 0.0077 ft/sec

from Page 6 Ψ_{ult} = 0.00011 sec^{-1}/ft length along slope

assuming all seepage occurs in 1 hour

required permittivity

IBWC Type 2 permittivity 0.5 sec^{-1} is greater than 0.00011 sec^{-1}

Therefore: Type 2 is acceptable

Flow Rate 75 gpm/sf X 0.002228 = 0.1671 cf/sec/sf

for a river elevation drop of 7.5 ft in 1 hour

q_{max} = 0.417 cf/sec - ft length along the slope from Page 6

$$\frac{0.1671 \times 7.5\text{ ft}}{0.417} = \frac{1.253}{0.417} = 3 \times 75 = 225 \text{ gpm/sf}$$

225 gpm/sf needed for 1 hour

for a river elevation drop of 29 ft in 1 hour

q_{max} = 1.614 cf/sec - ft of length along slope

$$\frac{0.1671 \times 29\text{ ft}}{1.614} = \frac{4.85}{1.614} = 3 \times 75 = 225 \text{ gpm/sf}$$

225 gpm/sf needed for 1 hour

DESIGN NOTES AND COMPUTATIONS

SUBJECT: Levee Geotextile

PREPARED BY: LSMorr | DATE: 21 Dec 15 | CHECKED BY: KV | APPROVED BY: | JOB NUMBER: 4839-01

Apparent Opening Size "AOS" #80-#120 sieve size

#80 sieve = 0.18 mm

#120 sieve = 0.125 mm

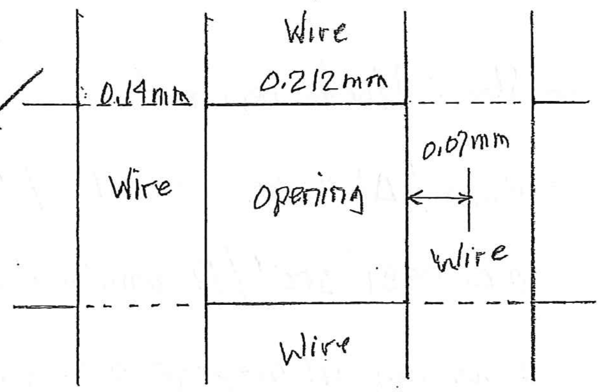
required retention 0.217mm from Page 7

#70 sieve = 0.212 mm

use #70 sieve or smaller sieve [smaller ⇒ < 0.212 mm]

Porosity of #70 sieve

$$\eta = \frac{0.212}{(0.07 \times 4) + 0.212} = 0.43$$



η of the sieve is greater than η of the soil

geotextile with AOS = 0.212mm will allow more

q than the soil can produce

SUBJECT: Levee Geotextile

PREPARED BY LSMarr | DATE 21Dec15 | CHECKED BY | APPROVED BY | JOB NUMBER 4839-01

SHEET NO. 11 OF 12

Reference Koerner Page 218 Example Problem

USE Filter Soil

A. Determine seepage water through filter Soil

q_{max} = (Area x porosity)/time

q_{max} = (1142 sf x 0.35)/3600 sec = 1 hour

q_{max} = 0.111 cf/sec - ft length along slope

Determine Permittivity required

Ψ = q_{max} / Δh x Area = 0.111 / 15 x 76.12

Ψ = 0.000097 sec⁻¹/ft length along slope

assuming all seepage occurs in 1 hour

Ψ ranges from 0.02 - 2.2 sec⁻¹ for typical geotextiles

Therefore geotextile with Ψ > 0.02 sec⁻¹ will allow adequate flow

Factor of Safety = $\frac{0.02}{0.00019}$ = 105

SUBJECT: Levee Geotextile

PREPARED BY LSMarr | DATE 21 Dec 15 | CHECKED BY [initials] | APPROVED BY | JOB NUMBER 4839-01

B Determine Soil Retention see Page 121

Method 1 5% passing #200 sieve AOS ≥ #30 sieve $O_{95} < 0.59mm$

Method 2 $O_{95} < (2 \text{ or } 3)\, d_{85}$ $O_{95} <$ 12mm to 18 mm

Method 3 use relative density = 70% CU = 5

$$O_{95} < \frac{19.5 d_{50}}{CU} = \frac{13.5 \times 0.7}{5} < 1.9\,mm$$

Therefore O_{95} = AOS < 0.59 to 18 mm #30 sieve to 5/8 inch sieve

#30 sieve = 0.6mm

Required Geotextile between ACBM + Filter Soil

Permittivity $\Psi > 0.02\ sec^{-1}$

AOS = 0.6mm = #30 sieve

TABLE 1 Nominal Dimensions, Permissible Variations for Wire Cloth of Standard Test Sieves (U.S.A.) Standard Series

Sieve Designation		Nominal Sieve Opening, in.[B]	Permissible Variation of Average Opening from the Standard Sieve Designation	Opening Dimension Exceeded By Not More Than 5 % of the Openings	Maximum Individual Opening	Nominal Wire Diameter, mm[C]
Standard[A]	Alternative					
(1)	(2)	(3)	(4)	(5)	(6)	(7)
125 mm	5 in.	5	±3.70 mm	130.0 mm	130.9 mm	8.00
106 mm	4.24 in.	4.24	±3.20 mm	110.2 mm	111.1 mm	6.30
100 mm[D]	4 in.[D]	4	±3.00 mm	104.0 mm	104.8 mm	6.30
90 mm	3½ in.	3.5	±2.70 mm	93.6 mm	94.4 mm	6.30
75 mm	3 in.	3	±2.20 mm	78.1 mm	78.7 mm	6.30
63 mm	2½ in.	2.5	±1.90 mm	65.6 mm	66.2 mm	5.60
53 mm	2.12 in.	2.12	±1.60 mm	55.2 mm	55.7 mm	5.00
50 mm[D]	2 in.[D]	2	±1.50 mm	52.1 mm	52.6 mm	5.00
45 mm	1¾ in.	1.75	±1.40 mm	46.9 mm	47.4 mm	4.50
37.5 mm	1½ in.	1.5	±1.10 mm	39.1 mm	39.5 mm	4.50
31.5 mm	1¼ in.	1.25	±1.00 mm	32.9 mm	33.2 mm	4.00
26.5 mm	1.06 in.	1.06	±.800 mm	27.7 mm	28.0 mm	3.55
25.0 mm[D]	1.00 in.[D]	1	±.800 mm	26.1 mm	26.4 mm	3.55
22.4 mm	7/8 in.	0.875	±.700 mm	23.4 mm	23.7 mm	3.55
19.0 mm	3/4 in.	0.750	±.600 mm	19.9 mm	20.1 mm	3.15
16.0 mm	5/8 in.	0.625	±.500 mm	16.7 mm	17.0 mm	3.15
13.2 mm	0.530 in.	0.530	±.410 mm	13.83 mm	14.05 mm	2.80
12.5 mm[D]	½ in.[D]	0.500	±.390 mm	13.10 mm	13.31 mm	2.50
11.2 mm	7/16 in.	0.438	±.350 mm	11.75 mm	11.94 mm	2.50
9.5 mm	3/8 in.	0.375	±.300 mm	9.97 mm	10.16 mm	2.24
8.0 mm	5/16 in.	0.312	±.250 mm	8.41 mm	8.58 mm	2.00
6.7 mm	0.265 in.	0.265	±.210 mm	7.05 mm	7.20 mm	1.80
6.3 mm[D]	¼ in.[D]	0.250	±.200 mm	6.64 mm	6.78 mm	1.80
5.6 mm	No. 3½[E]	0.223	±.180 mm	5.90 mm	6.04 mm	1.60
4.75 mm	No. 4	0.187	±.150 mm	5.02 mm	5.14 mm	1.60
4.00 mm	No. 5	0.157	±.130 mm	4.23 mm	4.35 mm	1.40
3.35 mm	No. 6	0.132	±.110 mm	3.55 mm	3.66 mm	1.25
2.80 mm	No. 7	0.110	±.095 mm	2.975 mm	3.070 mm	1.12
2.36 mm	No. 8	0.0937	±.080 mm	2.515 mm	2.600 mm	1.00
2.00 mm	No. 10	0.0787	±.070 mm	2.135 mm	2.215 mm	0.900
1.7 mm	No. 12	0.0661	±.060 mm	1.820 mm	1.890 mm	0.800
1.4 mm	No. 14	0.0555	±.050 mm	1.505 mm	1.565 mm	0.710
1.18 mm	No. 16	0.0469	±.045 mm	1.270 mm	1.330 mm	0.630
1.00 mm	No. 18	0.0394	±.040 mm	1.080 mm	1.135 mm	0.560
850 µm[F]	No. 20	0.0331	±35 µm	925 µm	970 µm	0.500
710 µm	No. 25	0.0278	±30 µm	775 µm	815 µm	0.450
600 µm	No. 30	0.0234	±25 µm	660 µm	695 µm	0.400
500 µm	No. 35	0.0197	±20 µm	550 µm	585 µm	0.315
425 µm	No. 40	0.0165	±19 µm	471 µm	502 µm	0.280
355 µm	No. 45	0.0139	±16 µm	396 µm	426 µm	0.224
300 µm	No. 50	0.0117	±14 µm	337 µm	363 µm	0.200
250 µm	No. 60	0.0098	±12 µm	283 µm	306 µm	0.160
212 µm	No. 70	0.0083	±10 µm	242 µm	263 µm	0.140
180 µm	No. 80	0.0070	±9 µm	207 µm	227 µm	0.125
150 µm	No. 100	0.0059	±8 µm	174 µm	192 µm	0.100
125 µm	No. 120	0.0049	±7 µm	147 µm	163 µm	0.090
106 µm	No. 140	0.0041	±6 µm	126 µm	141 µm	0.071
90 µm	No. 170	0.0035	±5 µm	108 µm	122 µm	0.063
75 µm	No. 200	0.0029	±5 µm	91 µm	103 µm	0.050
63 µm	No. 230	0.0025	±4 µm	77 µm	89 µm	0.045
53 µm	No. 270	0.0021	±4 µm	66 µm	76 µm	0.036
45 µm	No. 325	0.0017	±3 µm	57 µm	66 µm	0.032
38 µm	No. 400	0.0015	±3 µm	48 µm	57 µm	0.030
32 µm	No. 450	0.0012	±3 µm	42 µm	50 µm	0.028
25 µm[D]	No. 500	0.0010	±3 µm	34 µm	41 µm	0.025
20 µm[D]	No. 635	0.0008	±3 µm	29 µm	35 µm	0.020

[A] These standard designations correspond to the values for test sieve openings recommended by the International Standards Organization, Geneva, Switzerland, except where noted.

[B] Only approximately equivalent to the metric values in Column 1.

[C] The average diameter of the wires in the x and y direction, measured separately, of any wire cloth shall not deviate from the nominal values by more than ±15 %.

[D] These sieves are not in the standard series but they have been included because they are in common usage.

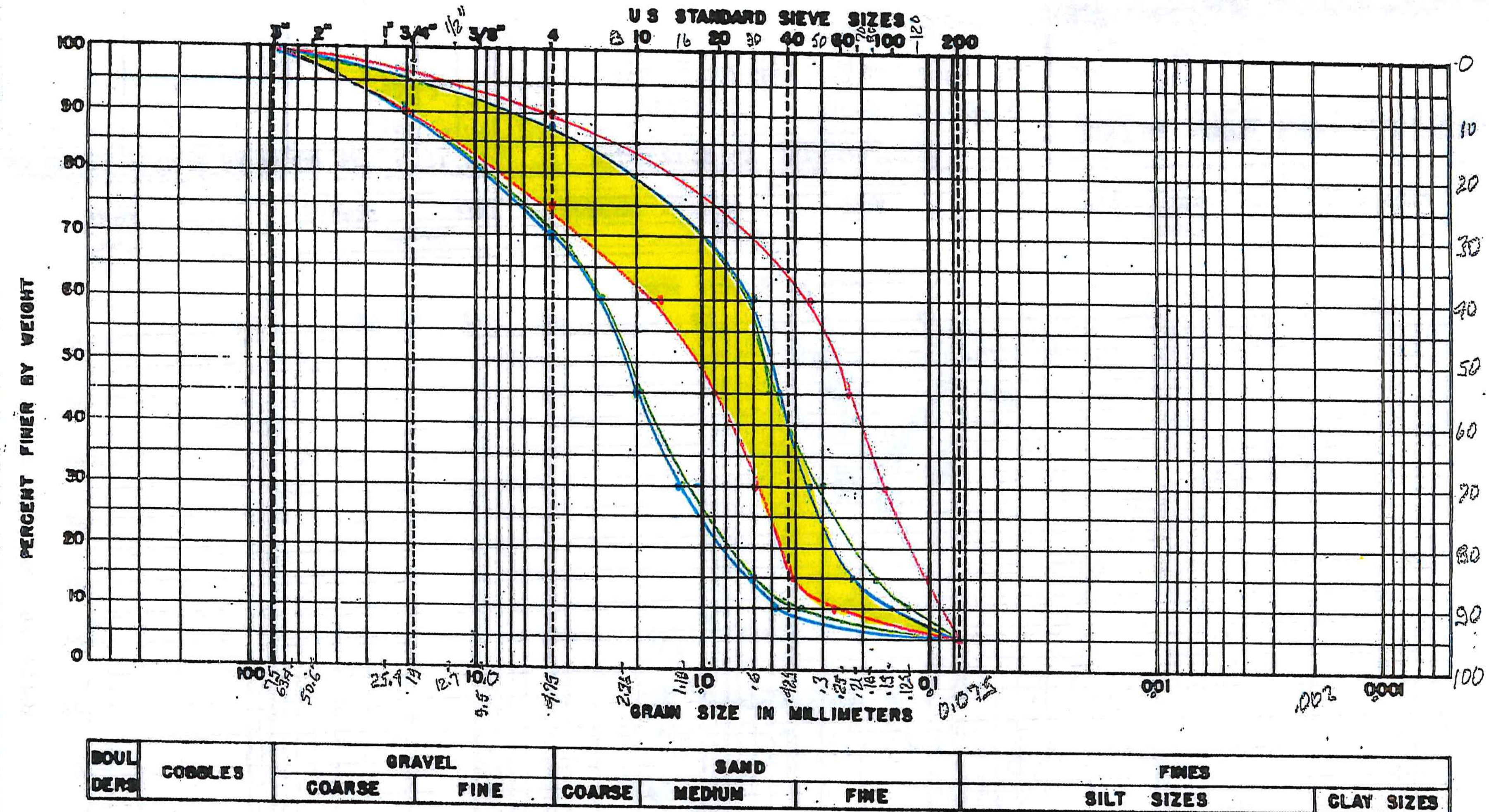
[E] These numbers (3½ to 635) are the approximate number of openings per linear in. but it is preferred that the sieve be identified by the standard designation in millimetres or micrometres.

[F] 1000 µm—1 mm.

shall be made along the midpoints of the opening as shown in Fig. 1.

4.4 Sieve cloth shall conform to the dimensional requirements of Table 1. The average opening (distance between parallel wires measured at the center of the opening), in the x (horizontal) and y (vertical) directions measured separately, shall conform to the values in Column 1, within the permissible variation in average opening size shown in Column 4. Not more than 5 % of the openings shall exceed the value shown in Column 5. The maximum individual

FILTER DESIGN BROWNSVILLE LEVEE



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
FILTER 1						
FILTER 2						
FILTER 3A						

GRAIN SIZE DISTRIBUTION

JOB NO. ____________

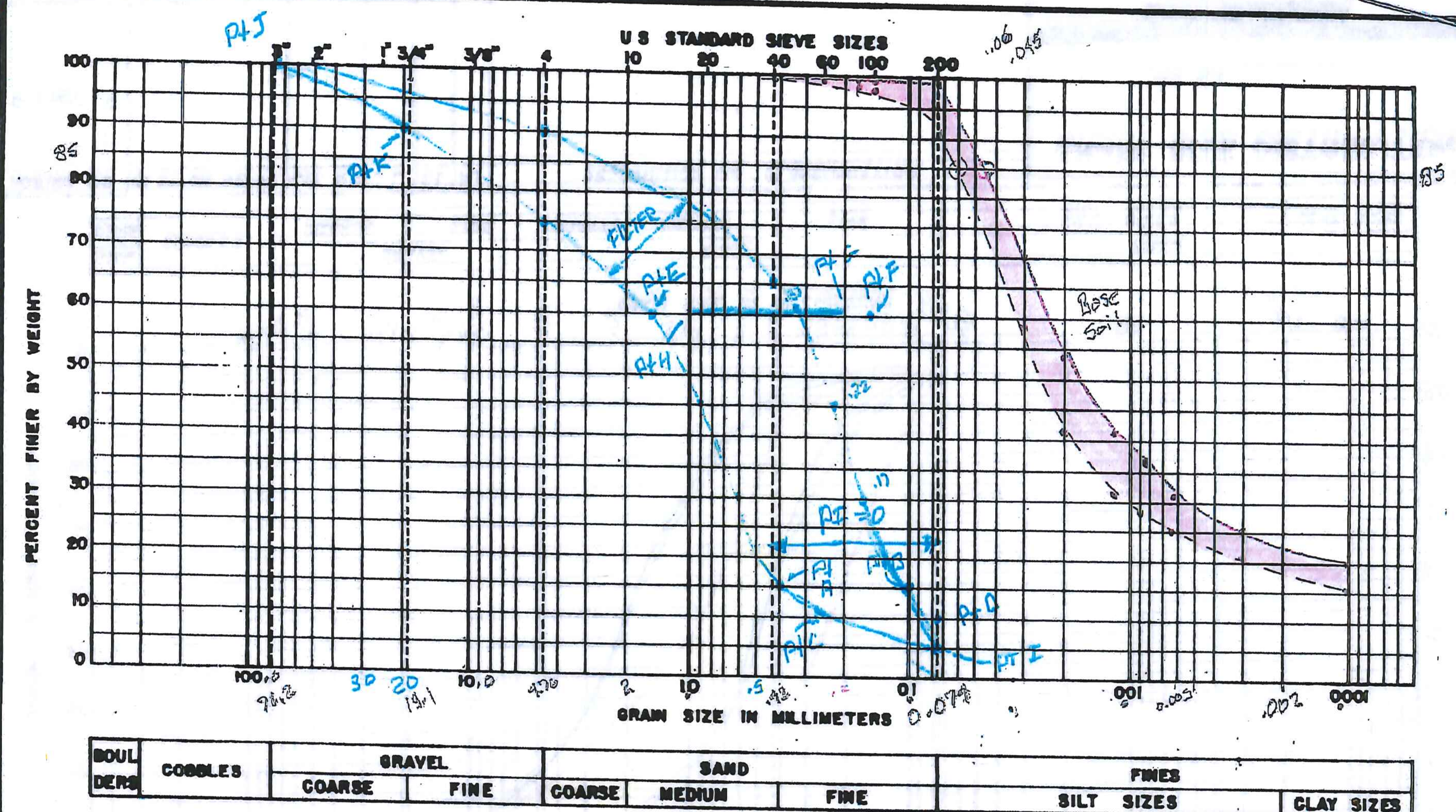
[redacted]

Perm $D_{15} = 0.54$ $0.54^2 = 0.2916$ mm/sec
$d_{15} = 0.001$ $0.001^2 = 0.000001$ mm/sec

CL silty clay Hydrometers

1

% Passing
100 3"
90-96 3/4"
85
75-90 #4
15-45 #40
5 #200
100 narrow



Base Soil "d"

Filter Soil "D"

BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
P3-33 —	6.4-8.4		36	22	19	CL Category 1 % Passing 200 > 85%
P3-33---	13-15		33	22	11	CL $D_{15} \leq 9 \times d_{85}$ $d_{85} = 0.06$ $D_{15} = 0.54$ $D_{90} = 25$.
						D $d_{85} = 0.045$ $D_{15} = 0.405$ $D_{90} = 20$

GRAIN SIZE DISTRIBUTION

JOB NO.

[redacted]

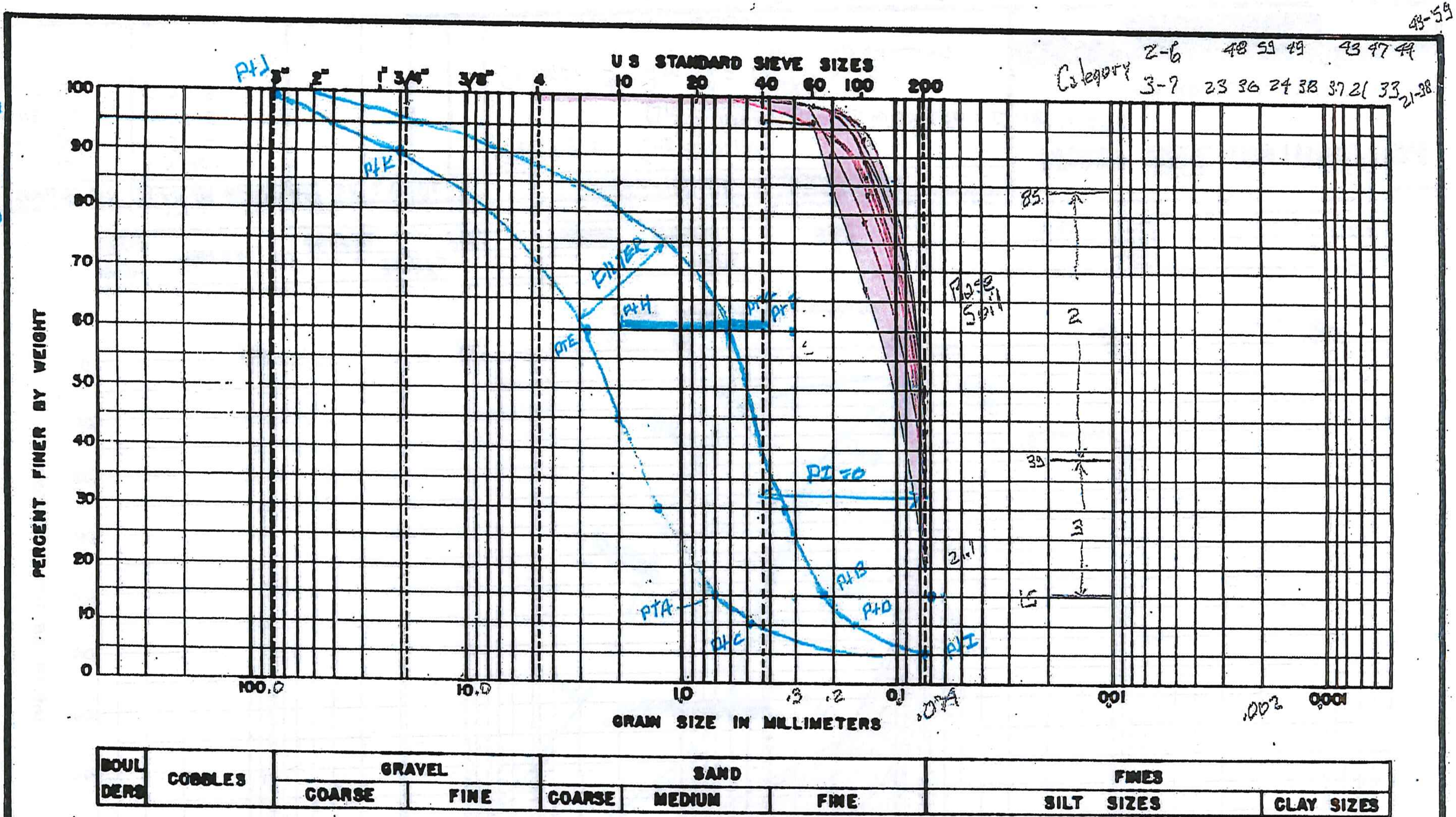
$D_{15} \geq 3\text{ to }5 \times d_{15}$
$D_{15} = 3\text{ to }5 \times 0.001$

Permeability $\frac{D_{15}}{d_{15}} \geq 3\text{ to }5$ $\frac{0.54}{0.001} = 540$ $\frac{0.405}{0.001} = 405$

Compute filter as Cat 2 ②

Filter
% Passing

100 3"
90-96 3/4"
70-88 #4
10-40 #40
0-5 #200



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
						Category 2 $D_{15} \leq 0.7mm$ $D_{15} \geq 3 to 5 \times d_{15} = 3 to 5 \times 0.06 = 0.18 to 0.30$ Category 3 $d_{85} \times 4 = 0.2 \times 4 = 0.8 mm$ $0.1 \times 4 = 0.4 mm \to 0.7m$ $D_{15} \leq \left(\frac{40-A}{40-15}\right)$ A = 30%

GRAIN SIZE DISTRIBUTION

JOB NO. ____________

[redacted]

$D_{15} \leq \left(\frac{40-30}{40-15}\right) = \frac{10}{25} = 0.4 mm \to 0.7 mm$

Perm ok $\frac{D_{15}}{d_{15}} \geq 3 to 5$

GC
SM w/gravel
Corrected Curves

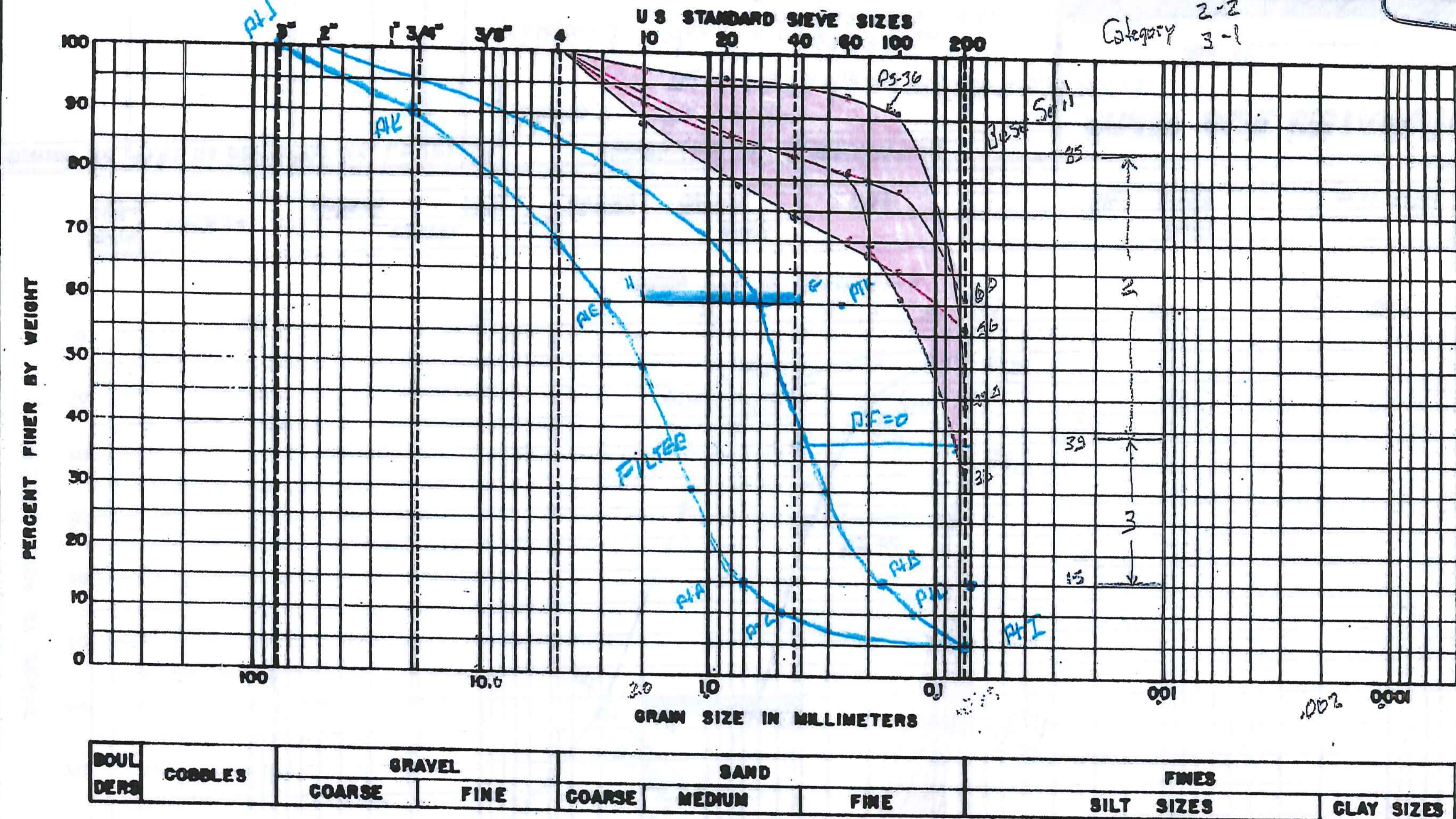
Compute Filter as Cats

(3A)

Filter % Passing

100 3"
90-96 3/4"
20-82 #4
10-43 #40
-5 #200

Category 2-2
3-1



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
P3-39	9.5-10.5					Category 2 $D_{15} \leq 0.7$ mm
P3-39	10.5-12					$D_{15} \geq 3$ to $5 \times d_{15} = 3$ to $5 \times 0.06 = 0.18$ to 0.30
P3-36	16.5-18					Category 3 $d_{85} \times 4 = 1.3 \times 4 = 5.2$
P3-36	13.5-15					$0.11 \times 4 = 0.44 \rightarrow 0.7$

GRAIN SIZE DISTRIBUTION

JOB NO.________

[redacted]

Perm ok $\frac{D_{15}}{d_{15}} \geq 3$ to 5

$D_{15} \leq \left(\frac{40-A}{40-15}\right)$ $A = 33$ $\left(\frac{40-33}{40-15}\right) = \frac{7}{25} = 0.28$ mm

5. The filter fabric for this project shall meet all requirements of Type 2 in Table 24, when sampled and tested.

B. Packaging and Labeling
All material shall be packaged and shipped with suitable wrapping to protect against damage to the fabric, ultraviolet light, and moisture during shipping, handling, and storage at the job site. Each roll must be identified with a tag or label affixed to the outside of the roll and have the following information:

1. Name of fabric manufacturer.
2. Manufacturer's lot number or control numbers, if any

Table 24 - Filter Fabric Requirements

Physical Properties	Test Method	Type 1	Type 2
Fabric Weight on an ambient temperature air-dried tension-free sample.	ASTM D3776	136 g/m² (4 oz/yd²). minimum	203 g/m² (6 oz/yd²), minimum
Permittivity, 1/sec.	ASTM D4991	1.0, minimum	0.5, minimum
Flow Rate	ASTM D4991	N/A	75 gpm/ft²
Tensile Strength (Grab)	ASTM D4632	445 N (100 lbs) minimum	890 N (200 lbs) minimum
Apparent Opening Size	ASTM D4751	70-100	80-120
Elongation at yield,%	ASTM D4632	20-100	20-100
Trapezoidal Tear	ASTM D4533	156 N (35 lbs) minimum	334 N (75 lbs) minimum
Static (CBR) Puncture	ASTM D6241	N/A	3226 N (725 lbs)
UV Resistance, minimum	ASTM D4355	70%	70%

3. Unique roll number, serially designed
4. Brand name of the product
5. Manufacturer's style or catalog designated of the fabric, if any
6. Row width and length size.

C. Provide fabric that is free of defects, rips, holes, or flaws. Replace damaged materials at no expense to the USIBWC.

D. Geotextile shall be shipped and maintained in a heavy-duty protective cover until it is placed. During all periods of shipment and storage, the geotextile shall be protected from direct sunlight, ultraviolet rays, temperatures greater than one hundred forty degrees Fahrenheit (140°F), mud, dirt, and other contaminants.

1. Do not expose the fabric to direct sunlight for more than two (2) weeks during the storage and installation.

ATTACHMENT C

DESIGN CALCULATIONS

C.1 GEOTEXTILE
C.2 SOIL FILTER
C.3 ARTICULATED CONCRETE BLOCK MATRESS (ACBM)
C.4 ROCK RIPRAP
C.5 GATEWELL STRUCTURE 205

SUBJECT: Filter Design Brownsville Levee				
PREPARED BY LSMarr	DATE 23 Nov 15	CHECKED BY [illegible]	APPROVED BY	JOB NUMBER 4839-01
				SHEET NO. 1 OF

References: Design and Construction of Levees CECW-EC
Manual No. 1110-2-1913 April 2000
Section II Seepage Through Embankments
Para. 5.7 - 5.12
Appendix D Filter Design

Filters for Embankment Dams Design and Construction
FEMA October 2011
Chapter 5+6+7 Attachment A Design Example 5.3.1

Geotechnical Evaluation of the Brownsville Levee Cracking and Partial Slope Failure
USACE ERDC July 2015
Drilling Logs and Lab Data and Cross Sections

DESIGN NOTES AND COMPUTATIONS

SUBJECT: Filter Design Brownsville Levee
PREPARED BY: LSM
DATE: 24 Nov 15
CHECKED BY: W
APPROVED BY:
JOB NUMBER: 4839-01
SHEET NO. 2 OF

Attachment A [FEMA] Base Soil Selection

This project → Foundation Materials [Fig A-2]

based upon Grain Size Curves, our soils classify as Category 1, 2, +3

Base soils are categorized according to their fines content [-#200]

Figure A-15 Selection Process and Page 235

1. Are base soils categorized correctly? Table 5-1

Reference Grain Size Distribution 1, 2, +3

1 CL soils, hydrometer data

% Passing #200 sieve > 85% Category 1

2 SM Soils, sieve analyses

% Passing #200 sieve between 85% and 40% Category 2

% Passing #200 sieve between 39% and 15% Category 3

3 GC and SM with Gravel, sieve analyses - corrected

% Passing #200 sieve between 85% and 40% Category 2

% Passing #200 sieve between 39% and 15% Category 3

∴ more than one category is present

determine if a continuous seepage path is present

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SUBJECT: Filter Design	Brownsville Levee			
PREPARED BY LSM	DATE 24 Nov 15	CHECKED BY [illegible]	APPROVED BY	JOB NUMBER 4839-01
				SHEET NO. 3 OF

2. Continuous Seepage Path   Reference: USACE Report

USACE Cross Sections A B C + D

USACE Drilling logs P3-31 P3-32 P3-33 P3-34 P3-35 P3-36

USACE Water level Readings Section 4.3.5 and 5.3

P3-31 Water level about 13.5 ft depth, El 16.5 23.5 ft [guess]

P3-32 Water level about 17 ft depth, El 23 ft [shallow screen]

P3-33 Water level about 13 ft depth, El 18 ft

P3-34 Water level about 11 ft depth El 12 ft

P3-35 Water level about 9 ft depth El 23 ft

P3-36 Water level about 6 ft depth El 25 ft [guess]

∴ Soil strata below these water level are probably seepage paths more so the more granular layers

∴ use grain size distribution data for these strata per Page 235 determine [trial design] using the fine side of the lowest numbered category - Category 1

DESIGN NOTES AND COMPUTATIONS

SUBJECT: Filter Design   Brownsville levee

PREPARED BY: LSM   DATE: 24 Nov 15   CHECKED BY: APPROVED BY: JOB NUMBER: 4839-01

SHEET NO. 4 OF

Filter Design Procedure FEMA 5.2 and Design Example 5.3.1

Step 1 Plot grain size distribution, See ①

Step 2 No correction needed - no sizes larger than #4

Step 3 not needed

Step 4 Determine base soil category Table 5-1

① is Category 1

Step 5 Satisfy stability Category 1 F - filter B - base

Maximum $D_{15}F \leq 9 \times D_{85}B$ not less than 0.2mm

$D_{85}B = 0.06$ mm and 0.045 mm

$D_{15}F \leq 9 \times 0.06 = 0.54$ mm

Point A $D_{15}F \leq 9 \times 0.045 = 0.405$ mm use this one, finer side

Step 6 Satisfy permeability

Minimum $D_{15}F \geq 3$ to $5 \times$ Maximum $D_{15}B$

$D_{15}B = 0.001$

$D_{15}F = 3 \times 0.001 = 0.003$ per USACE use $D_{15}F = 0.1$ mm

$D_{15}F = 5 \times 0.001 = 0.005$

DESIGN NOTES AND COMPUTATIONS

SUBJECT: Filter Design Brownsville Levee
PREPARED BY LSM DATE 24Nov15 CHECKED BY WF APPROVED BY JOB NUMBER 4839-01
SHEET NO. 5 OF

Step 7 Max $D_{15}F$ / Min $D_{15}F$ < 5

$$\frac{\text{Max } D_{15}F}{\text{Min } D_{15}F} = \frac{0.405}{0.1} = 4.05 < 5$$

Step 8 Determine D_5F and $D_{100}F$

Point I D_5F = 0.075 mm   USACE uses D_5F = 0.075 [#200]

Point J $D_{100}F$ ≤ 2 inches   USACE uses $D_{100}F$ = 3 inches

USACE uses material between #40 and #200 → PI = 0

back to Step 7 Min $D_{15}F$ / $D_{15}B$ = 4 to 5   Page 88

$$\frac{\text{Min } D_{15}F}{D_{15}B} = \frac{0.005}{0.001} = 5$$

however Min $D_{15}F$ = 0.1 mm per USACE

Check Coefficient of Uniformity C_u = 2 to 6

Point C = Point A X 0.7 = 0.405 X 0.7 = 0.28 mm $D_{10}F$

Point D = Point B X 0.7 = 0.1 X 0.7 = 0.07 mm → 0.075 mm $D_{10}F$

Point E = Point C X 6 = 0.28 X 6 = 1.68 mm $D_{60}F$

Point F = Point D X 2 = 0.075 X 2 = 0.15 mm $D_{60}F$

Sliding Bar F < G < H   H = G X 5

let G = 0.2 mm   H = 5 X 0.2 = 1.0 mm

DESIGN NOTES AND COMPUTATIONS

SUBJECT: Filter Design Brownsville Levee
PREPARED BY LSM DATE 24 Nov 15 CHECKED BY [initials] APPROVED BY JOB NUMBER 4839-01
SHEET NO: 6 OF

Step 9 Maximum $D_{90}F$ Table 5-4

Minimum $D_{10}F = 0.075mm < 0.5mm$ $D_{90}F = 20mm$ Point K

Step 10 Determine filter gradation band

Band Width Ratio is 5 or less in mm below $D_{60}F$

Check $D_{15}F$ Max $D_{15}F = 0.405mm$ Min $D_{15}F = 0.1mm$

$$\frac{0.405}{0.1} = 4.05 < 5$$

Try $D_{30}F$ use Max $D_{30}F = 0.6mm$ use Min $D_{30}F = 0.17mm$

$$\frac{0.6}{0.17} = 3.5 < 5$$

Try $D_{45}F$ Use Max $D_{45}F = 0.9mm$ Use Min $D_{45}F = 0.22mm$

$$\frac{0.9}{0.22} = 4.1 < 5$$

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DESIGN NOTES AND COMPUTATIONS

SUBJECT: Filter Design Brownsville Levee

PREPARED BY: LSH | DATE: 24 Nov 15 | CHECKED BY: KP | APPROVED BY: | JOB NUMBER: 4839-01

SHEET NO. 7 OF

Filter Design Procedure FEMA 52 and Design Example 5.3.1

Step 1 Plot grain size distribution See (2)

Step 2 no correction needed - no sizes larger than #4

Step 3 not needed

Step 4 Base Soil Category - use Category 2 Table 5-1

Step 5 Satisfy Stability

Maximum $D_{15}F \le 0.7$mm Point A

Step 6 Minimum $D_{15}F \ge 3$ to $5 \times$ Max $D_{15}B$ use $D_{15}B = 0.07$mm

Min $D_{15}F \ge 3 \times 0.07 = 0.21$mm $\ge 5 \times 0.07 = 0.35$mm

use 0.21mm finer Point B

Step 7 Max $D_{15}F$ / Min $D_{15}F$ $\frac{0.7}{0.21} = 3.3 < 5$

Check Coefficient of Uniformity $C_u = 2$ to 6

Point C = Point A $\times 0.7 = 0.7 \times 0.7 = 0.49$ mm $D_{10}F$

Point D = Point B $\times 0.7 = 0.21 \times 0.7 = 0.15$ mm $D_{10}F$

Point E = Point C $\times 6 = 0.49 \times 6 = 2.94$ mm $D_{60}F$

Point F = Point D $\times 2 = 0.15 \times 2 = 0.3$ mm $D_{60}F$

Sliding Bar F < G < E H = G × 5 let G = 0.4mm H = 5 × 0.4 = 2.0mm

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DESIGN NOTES AND COMPUTATIONS

SUBJECT: Filter Design   Brownsville Levee

PREPARED BY: LSM   DATE: 24 Nov 15   CHECKED BY: KK   APPROVED BY: JOB NUMBER: 4839-01

SHEET NO. 8 OF

Step 8   Determine D_5F and $D_{100}F$ Table 5-4

Point I $D_5F = 0.075$ mm

Point J $D_{100}F \leq 2$ inches   USACE uses $D_{100}F = 3$ inches

Step 9   Maximum $D_{90}F$ Table 5-4

Minimum $D_{10}F = 0.15$ mm < 0.5 mm $D_{90}F = 20$ mm   Point K

Step 10   Determine Filter gradation band

Band Width Ratio = 5 or less in mm   Below $D_{60}F$

Check $D_{15}F$ Max $D_{15}F = 0.7$mm   Min $D_{15}F = 0.21$mm

$$\frac{0.7}{0.21} = 3.3 < 5$$

Try $D_{30}F$ use Max $D_{30}F = 1.3$mm   use Min $D_{30}F = 0.32$mm

$$\frac{1.3}{0.32} = 4 < 5$$

Try $D_{45}F$ use Max $D_{45}F = 2$mm   use Min $D_{45}F = 0.45$mm

$$\frac{2.0}{0.45} = 4.4 < 5$$

Check $D_{60}F$ Max $D_{60}F = 2.9$mm   Min $D_{10}F = 0.6$

$$\frac{2.9}{0.6} = 4.8 < 5$$

DESIGN NOTES AND COMPUTATIONS

SUBJECT: Filter Design   Brownsville Levee
PREPARED BY: LSM   DATE: 24Nov15   CHECKED BY: KF   APPROVED BY: JOB NUMBER: 4839-01
SHEET NO. 9 OF

Filter Design Procedure FEMA 5.2 and Design Example 5.3.1

Step 1 Plot grain size distribution See (3)

Step 2 Correction needed

Correction Factor $= \frac{100}{\%\text{Passing \#4}} = \frac{100}{61.1\%} = 1.64$ P3-34 9.5'-10.5'

Correction Factor $= \frac{100}{74.5} = 1.34$ P3-34 10.5'-12'

Correction Factor $= \frac{100}{81.1} = 1.23$ P3-36 16.5'-18'

Correction Factor $= \frac{100}{88.8} = 1.13$ P3-36 13.5'-15'

Step 3 Corrected grain size distribution See (3A)

Step 4 Base Soil Category - use Category 3 Table 5-4

Step 5 Satisfy Stability

Max $D_{15}F \leq \left[\frac{40-A}{25}\right]\left[(4 \times D_{85}B) - 0.7\text{mm}\right] + 0.7\text{mm}$

$A = (56 + 33 + 60 + 46)/4 = 49$ use 33

$D_{85}B = (1.3 + 0.55 + 0.55 + 0.12)/4 = 0.63$ use 0.60

Max $D_{15}F \leq \left[\frac{40-33}{25}\right](4 \times 0.6) = 0.672\text{ mm}$ use 0.7mm Point A

Step 6 Minimum $D_{15}F \geq 3$ to 5 X Maximum $D_{85}B$ use $D_{85}B = 0.06$ mm

Min $D_{15}F > 3 \times 0.06 = 0.18$ mm $= 5 \times 0.06 = 0.3$ mm

use 0.18 mm Point B

Step 7 Max $D_{15}F$ / Min $D_{15}F$ $\frac{0.7}{0.18} = 3.9 < 5$

Check Coefficient of Uniformity $C_u = 2$ to 6

Point C = Point A X 0.7 = 0.7 X 0.7 = 0.49 mm $D_{10}F$

Point D = Point B X 0.7 = 0.18 X 0.7 = 0.13 mm $D_{10}F$

Point E = Point C X 6 = 0.49 X 6 = 2.94 mm $D_{60}F$

Point F = Point D X 2 = 0.13 X 2 = 0.26 mm $D_{60}F$

Sliding B F < G < H H = G X 5 let G = 0.4 mm H = 5 X 0.4 = 2 mm

Step 8 Determine D_5F and $D_{100}F$ Table 5-3

Point I D_5F = 0.075 mm

Point J $D_{100}F$ = 2 inches USACE uses $D_{100}F$ = 3 inches

Step 9 Maximum $D_{90}F$ Table 5-4

Minimum $D_{10}F$ = 0.13 mm < 0.5 mm $D_{90}F$ = 20 mm Point K

DESIGN NOTES AND COMPUTATIONS

SUBJECT: Filter Design Brownsville Levee

PREPARED BY LSM | DATE 24 Nov 15 | CHECKED BY | APPROVED BY | JOB NUMBER 1839-01

SHEET NO. 11 OF

Step 10 Determine Filter gradation band

Band Width Ratio = 5 or less in mm below $D_{60}F$

Check $D_{15}F$ Max $D_{15}F$ = 0.7mm Min $D_{15}F$ = 0.18mm

$$\frac{0.7}{0.18} = 3.9 < 5$$

Try $D_{30}F$ use Max $D_{30}F$ = 1.2 mm use Min $D_{30}F$ = 0.3 mm

$$\frac{1.2}{0.3} = 4 < 5$$

Try $D_{45}F$ use Max $D_{45}F$ = 2.0mm use Min $D_{45}F$ = 0.5mm

$$\frac{2.0}{0.5} = 4 < 5$$

Check $D_{60}F$ Max $D_{60}F$ = 2.9 mm Min $D_{60}F$ = 0.6mm

$$\frac{2.9}{0.6} = 4.8 < 5$$

DESIGN NOTES AND COMPUTATIONS

SUBJECT: Filter Design
PREPARED BY: LSMarr DATE: 25 Nov 15 CHECKED BY: [illegible] APPROVED BY: JOB NUMBER: 4839-01
SHEET NO. 12 OF

Results of Filter Design + Gradation Requirements

Sieve Size	Percent Passing	Cummulative Percent Retained	Each Sieve Percent Retained
3 inch	100	0	0
3/4 inch	90-95	5-10	5-10
#4	75-90	10-25	5-16
#8	65-80	20-35	10-10
#16	55-75	25-45	5 15-20 10
#40	15-40	60-85	35-40
#100	5-10	90-95	30 10-30 10
#200	0-5	95-100	5 15-35 5
			✓ 15 - 10

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DESIGN NOTES AND COMPUTATIONS

SUBJECT: Filter Design Brownsville Levee

PREPARED BY: LSMarr DATE: 30Nov15 CHECKED BY: APPROVED BY: JOB NUMBER: 4839-01

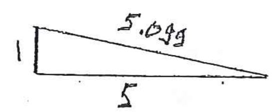
SHEET NO. 13 OF

Filter Thickness

Section 6.3 Page 111 & 112 and Section 6.5 Page 116

Minimum Thickness = 18 inches

Desired Thickness = 36 inches



$\frac{1}{5.099} = 0.196$

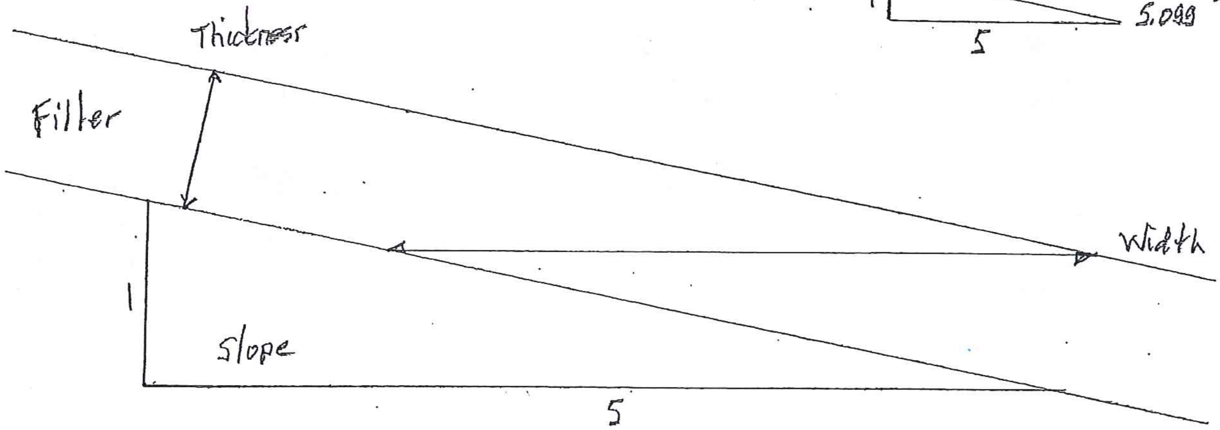


Figure 6.3 [modified] for 5:1 slope thickness to width ratio = 0.196

thickness = 18 inches width = 18/0.196 = 92 inches = 7.65 ft

thickness = 36 inches width = 36/0.196 = 184 inches = 15.3 ft

width = 8 ft thickness = 8X12 = 96 X 0.196 = 18.8 inches

maximum particle size is 3 inches

use 19 inches thickness with a width of 8 ft "not a dam"
20 inches 8.5 ft

Filter width is defined as the horizontal measurement across the filter. The filter thickness is defined as the measurement normal to the slope, sometimes known as the tangential thickness. Both definitions are illustrated in Figure 6-2. For vertical filters, the thickness equals the width.

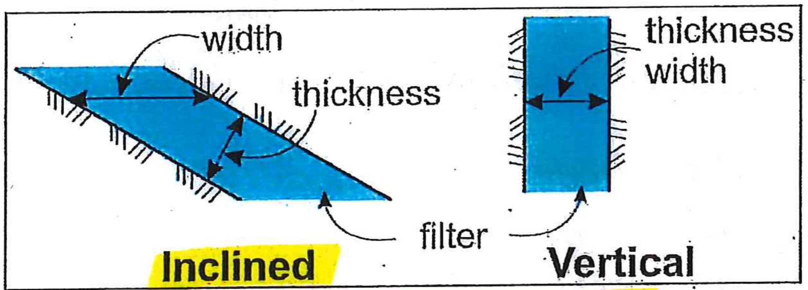


Figure 6-2. Definition of filter width and thickness.

When filters are placed against a slope, the width is always greater than the thickness. The difference between width and thickness increases as the slope becomes flatter, as shown in Figure 6-3. Narrow widths on flat slopes can lead to small thickness, which can be problematic due to the "Christmas tree" effect described later.

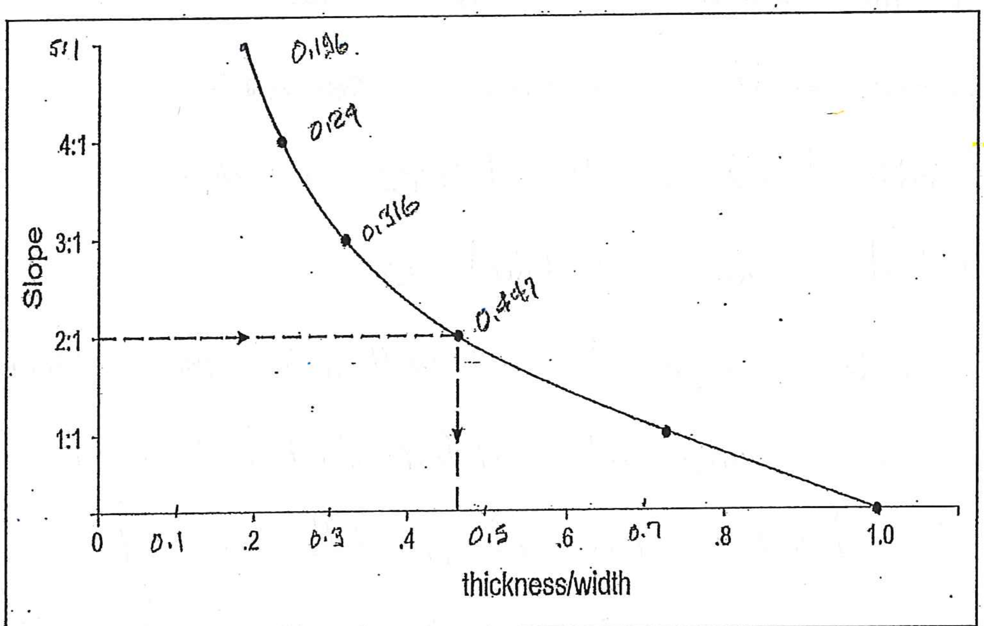


Figure 6-3. Effect of slope on filter width (e.g., a 10-ft-wide filter on a 2H:1V slope will have a 4.5-ft thickness).

SUBJECT: Filter Design	Brownsville Levee			
PREPARED BY LSMorr	DATE 30Nov15	CHECKED BY	APPROVED BY	JOB NUMBER 4839-01
				SHEET NO. 19 OF

Benching of the natural slope not needed for compacting purposes

Factor of Safety against sliding $= \frac{\tan\phi}{\tan\beta}$

ϕ = internal friction angle of filter → say 35°

β = slope angle = 11°

$$FS = \frac{\tan 35^\circ}{\tan 11^\circ} = \frac{0.7}{0.194} = 3.6$$

Placing and Compacting Filter

Reference Chapter 7 FEMA

Reference Section 7.9.4 Page 159-160

Construction Recommendations - see attached

Relative Density: ASTM D 4253 + D 4254

19 inches compacted thickness

1st Lift 9 inches compacted - 10 to 11 inches loose thickness

2nd Lift 10 inches compacted - 11 to 12 inches loose thickness

Geotextile [between base soils and filter material]

non woven with Apparent Opening Size ≈ #100 Sieve

Compaction of contacts with adjacent materials

Contacts between the filter/drain and adjacent materials, such as between the filter/drain and the impervious core, must be adequately compacted. If left uncompacted, an area of low shear strength and high compressibility could develop along the contact. Compaction of zonal contacts can be overlooked rather easily since the filter/drain is compacted by smooth-drum vibratory rollers and the impervious core is normally compacted by a tamping (sheepsfoot) or a rubber-tired (pneumatic) type roller. Equipment operators of each type of roller are often given instructions to avoid tracking on adjacent zones. Each operator working in accordance with his instructions may result in the area around the contacts not receiving adequate compaction.

Proper compaction of the contacts is accomplished by overlapping the vibratory roller onto the adjacent material rather than overlapping the tamping roller onto the filter/drain. However, roller operators and inspectors should be taught that a minor amount of mixing of the two adjacent materials is less a detriment than leaving the contact uncompacted. An overlap of 1 ft is usually specified. To facilitate compaction of contacts, all grade stakes used to mark zonal contacts prior to compaction should be removed so that operators do not drive around the stakes. Density testing should be conducted at or near zonal boundaries to verify that adequate compaction is being achieved in these critical areas. An example of rolling a sand filter/drain contact is shown in Figure 7-24 (Hammer 2003).

7.9.4 Recommendations

The following recommendations are made concerning moisture and compaction of filter and transition zone materials:

- Filter materials should be wetted prior to handling to facilitate handling as well as to help minimize segregation.
- The designer should consider all implications when specifying a desired density for the filter/drain material. A relative density of 70% is often used as a criterion value for minimum acceptable density.
- A method-type specification is usually recommended.
- Compaction of filter materials should be by means of vibratory rollers with the minimum required effort specified that will attain the desired density.

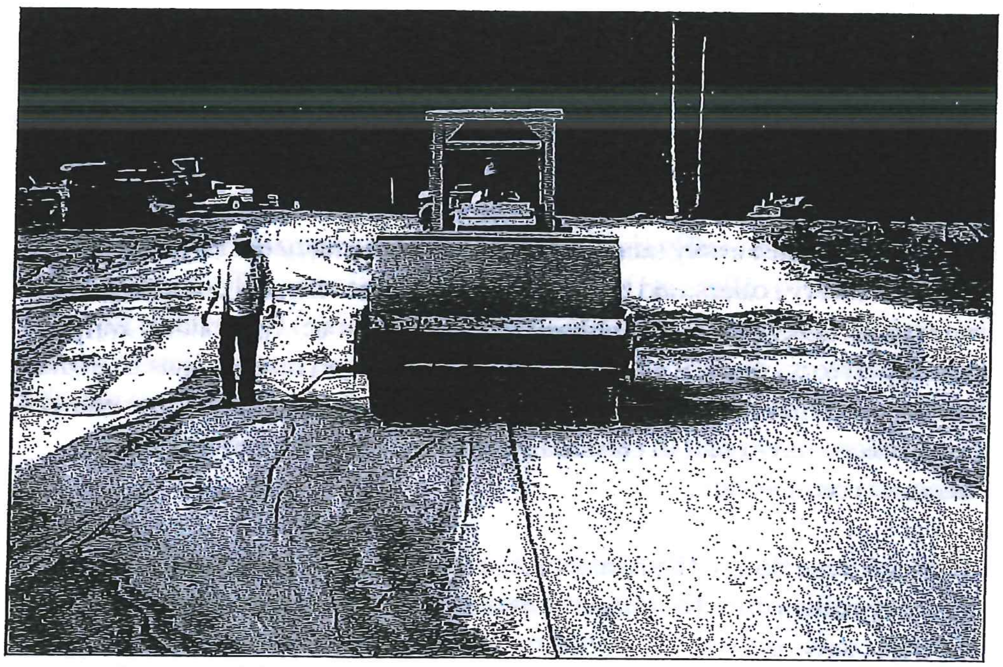


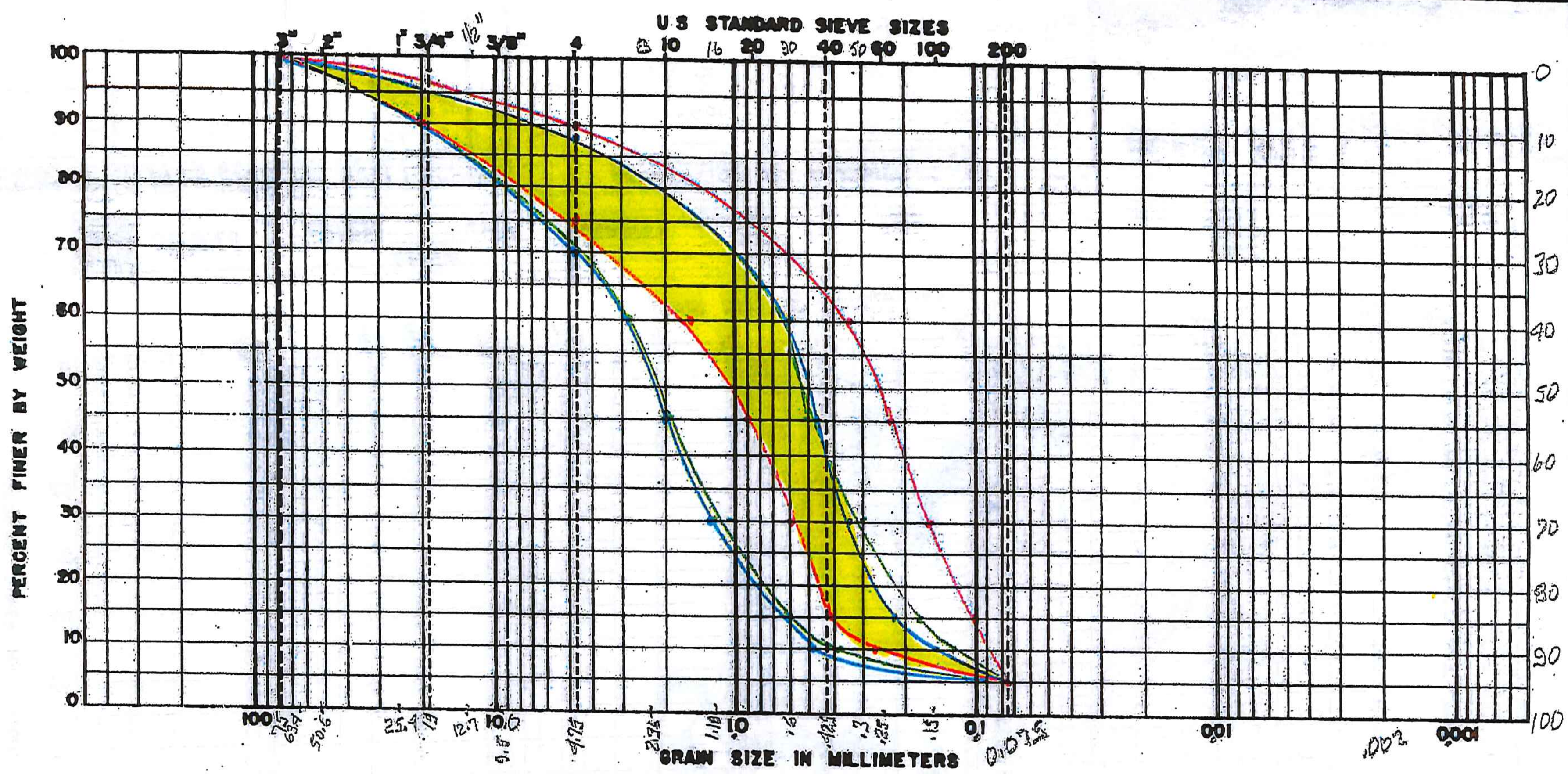
Figure 7-24. Compacting a joint between two zones by a vibratory roller.

- Avoid over-compaction, which can lead to particle breakage.

7.10 Horizontal and vertical control

Horizontal and vertical control of sand filter/drain and transition zones in an embankment is vitally important during construction. See section 6.4 for additional explanation of errors, such as the "Christmas tree" effect, that can be introduced by poor survey control as it relates to chimneys. Each lift of these zones must be accurately surveyed and staked or otherwise marked with temporary control points to ensure proper geometry is maintained at all times. Surface locations of previously placed lifts should not be used to establish the location of subsequent lifts due to possible errors resulting from over or under-building, spreading, or surface unevenness. To facilitate proper horizontal and vertical control, the surface of the filter/drain (and the entire embankment for that matter) must be maintained at the same longitudinal elevation. Every effort must be made to accomplish this during construction, even to the extent of building partial sections of the filter/drain in low areas in order to ensure a level surface is maintained. It should be noted that this type of control is applicable to construction on an embankment surface. Other areas that may be

FILTER DESIGN BROWNSVILLE LEVEE



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
FILTER 1						
FILTER 2						
FILTER 3A						

GRAIN SIZE DISTRIBUTION

JOB NO. ________

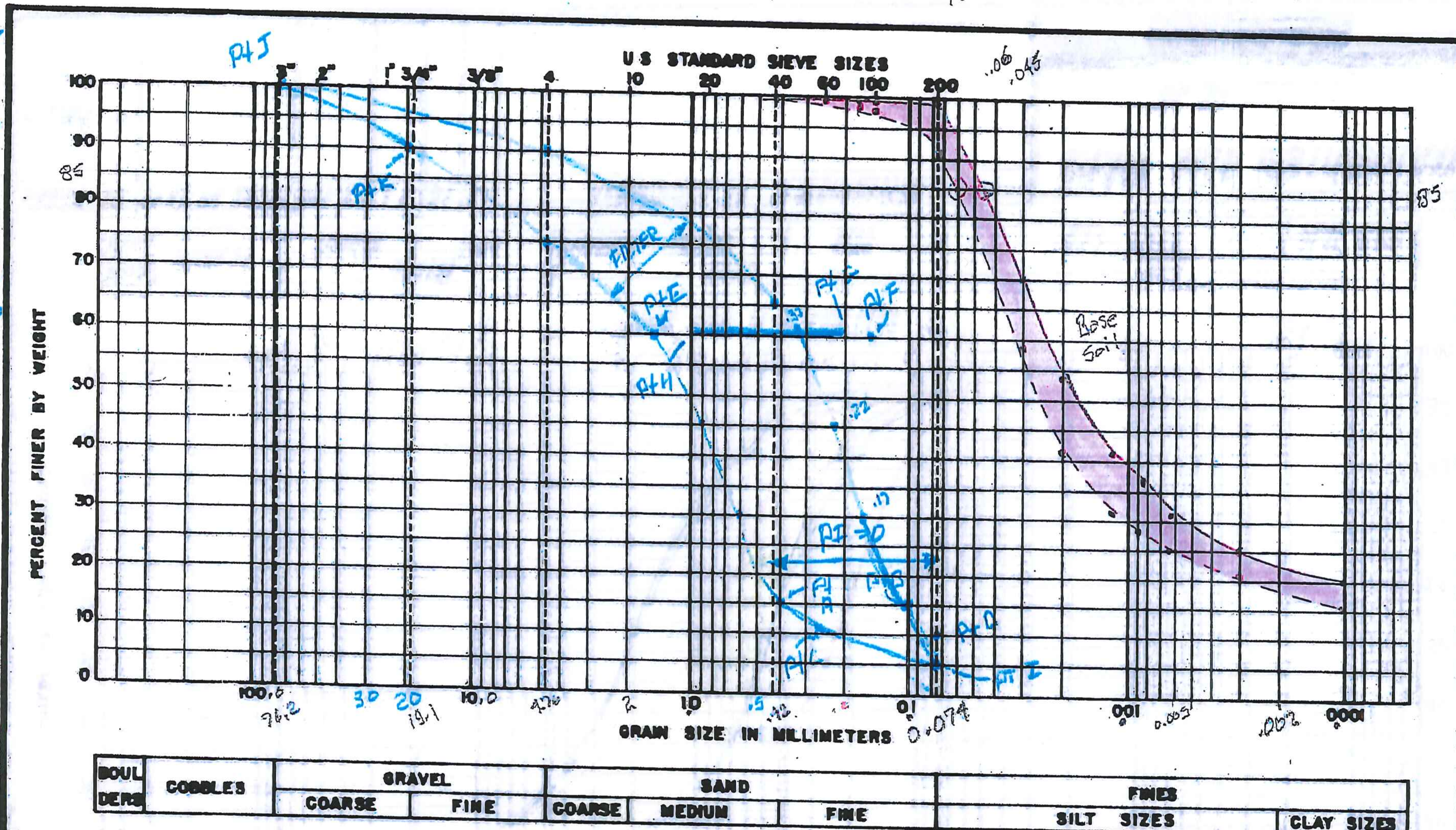
[redacted]

Perm $D_{15} = 0.54$ $0.54^2 = 0.2916$ mm/sec
$d_{15} = 0.001$ $0.001^2 = 0.000001$ mm/sec

CL silty clay Hydrometers

①

% Passing
100 3"
90-96 3/4"
75-90 #4
15-65 #40
5 #200
100 maximum



Base Soil "d"

Filter Soil "D"

BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
P3-39 —	6.4-8.4		36	22	19	CL Category 1 % Passing 200 > 85%
P3-33 ---	13-15		33	22	11	CL $D_{15} \le 9 \times d_{85}$ $d_{85} = 0.06$ $D_{15} = 0.54$ $D_{90} = 25$
						D $d_{85} = 0.045$ $D_{15} = 0.405$ $D_{90} = 20$

GRAIN SIZE DISTRIBUTION

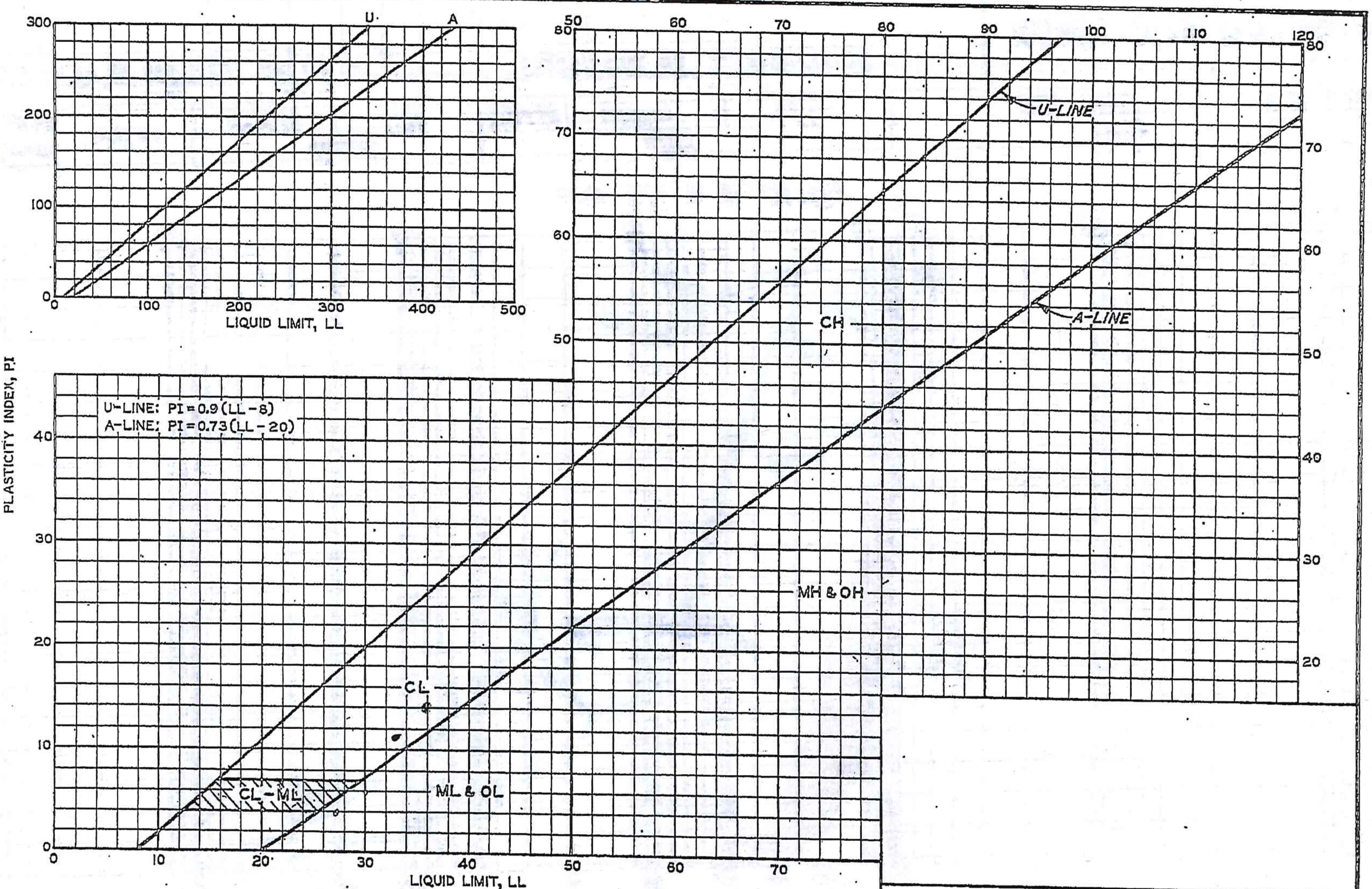
JOB NO.______

[redacted]

$D_{15} \ge 3$ to $5 \times d_{15}$
$D_{15} = 3$ to 5×0.001
$D_{15} = 0.003$ to $0.005 \rightarrow 0.1$ mm

Permeability $\frac{D_{15}}{d_{15}} \ge 3$ to 5 $\frac{0.54}{0.001} = 540$ $\frac{0.405}{0.001} = 405$

CL



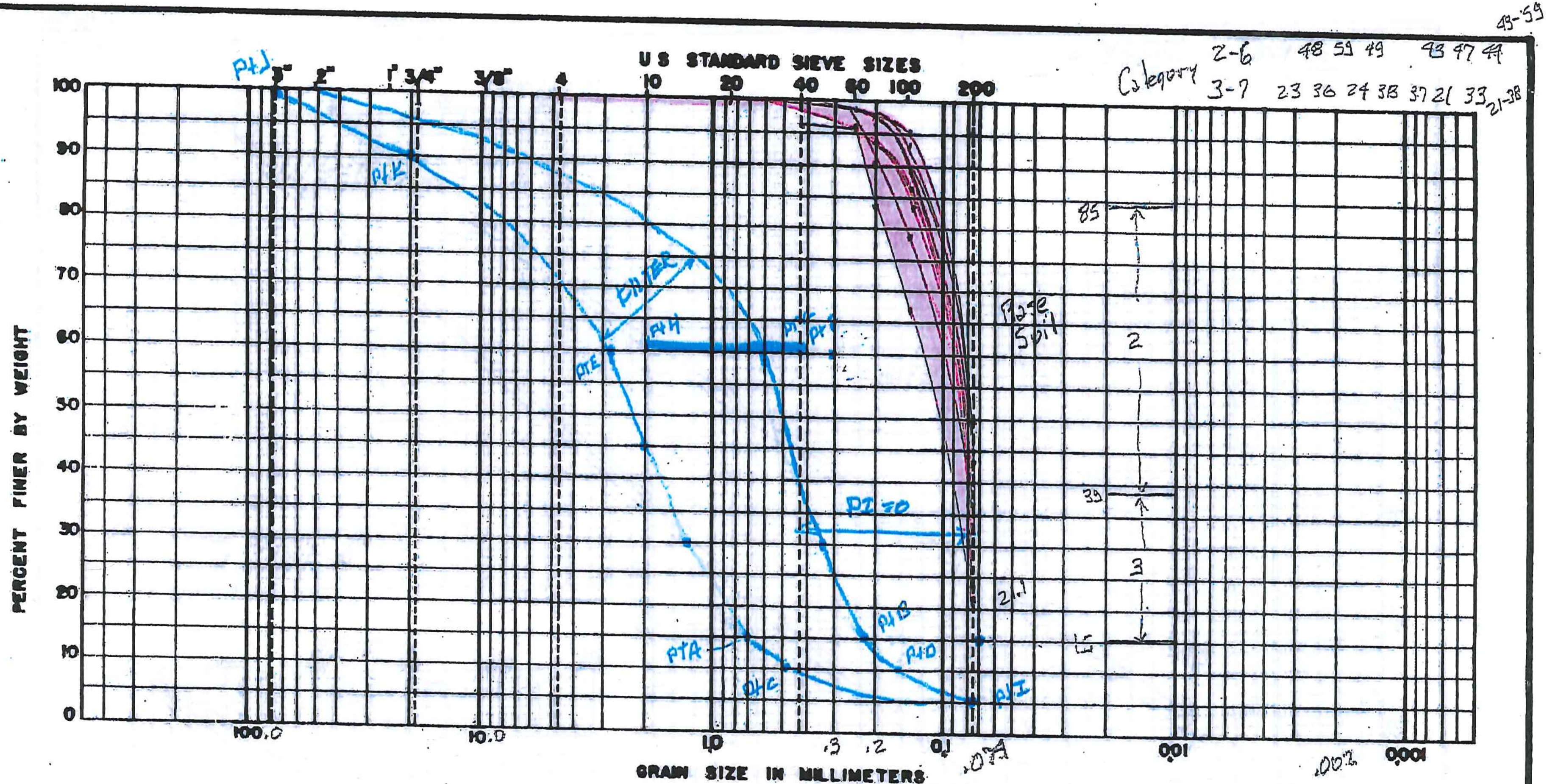
PLASTICITY CHART

ENG FORM JUNE 1970 4334

TRANSLUCENT

SM Compute filter as Cat 2 ②

Filter
% Passing
100 3"
90-96 3/4"
00-88 #4
10-40 #40
0-5 #200



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
						Category 2 $D_{15} \leq 0.7mm$ $D_{15} \geq 3 to 5 \times d_{15} = 3 to 5 \times 0.06 = 0.18 to 0.30$ Category 3 $d_{85} \times 4 = 0.2 \times 4 = 0.8mm$ $0.1 \times 4 = 0.4mm \rightarrow 0.7m$ $D_{15} \leq \left(\frac{40-A}{40-15}\right)$ $A = 30\%$

GRAIN SIZE DISTRIBUTION

JOB NO. ____________

[redacted]

Perm ok $\frac{D_{15}}{d_{15}} \geq 3 to 5$

$D_{15} \leq \left(\frac{40-30}{40-15}\right) = \frac{10}{25} = 0.4mm \rightarrow 0.7mm$

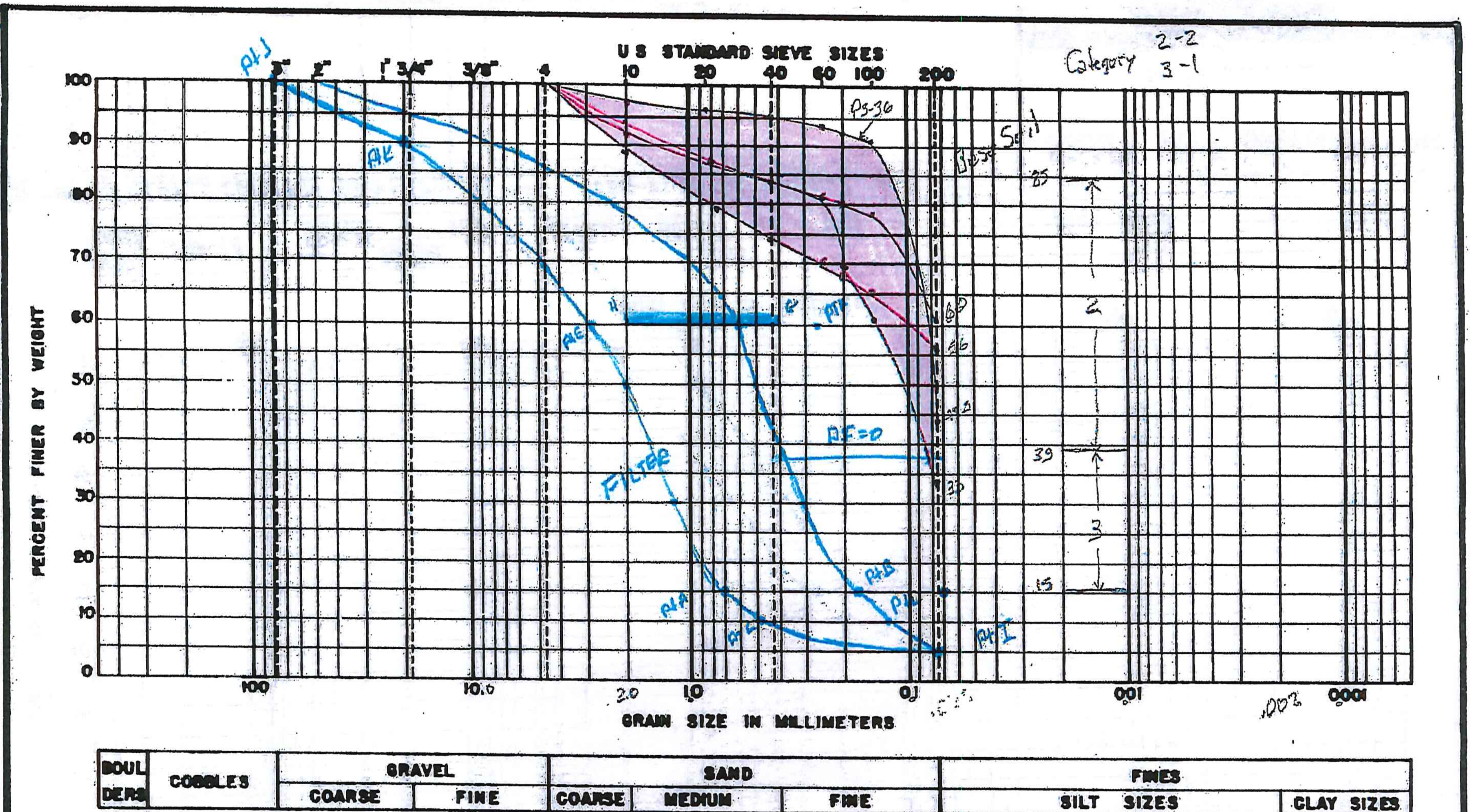
GC
SM w/gravel
Corrected Curves

Compute filter as Cat 3

3A

Filter % Passing

100 3"
90-06 3/4"
70-82 #4
10-43 #40
-5 #200



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
P3-39	9.5-10.5					Category 2 $D_{15} \le 0.7$ mm
P3-39	10.5-12					$D_{15} \ge 3$ to $5 \times d_{15} = 3$ to $5 \times 0.06 = 0.18$ to 0.30
P3-36	16.5-18					
P3-36	13.5-15					Category 3 $d_{85} \times 4 = 1.3 \times 4 = 5.2$
						$0.11 \times 4 = 0.44 \rightarrow 0.7$

GRAIN SIZE DISTRIBUTION

JOB NO. ________

[redacted]

Perm ok $\frac{D_{15}}{d_{15}} \ge 3$ to 5

$D_{15} \le \left(\frac{40-A}{40-15}\right)$ $A = 33$ $\left(\frac{40-33}{40-15}\right) = \frac{7}{25} = 0.28$ mm

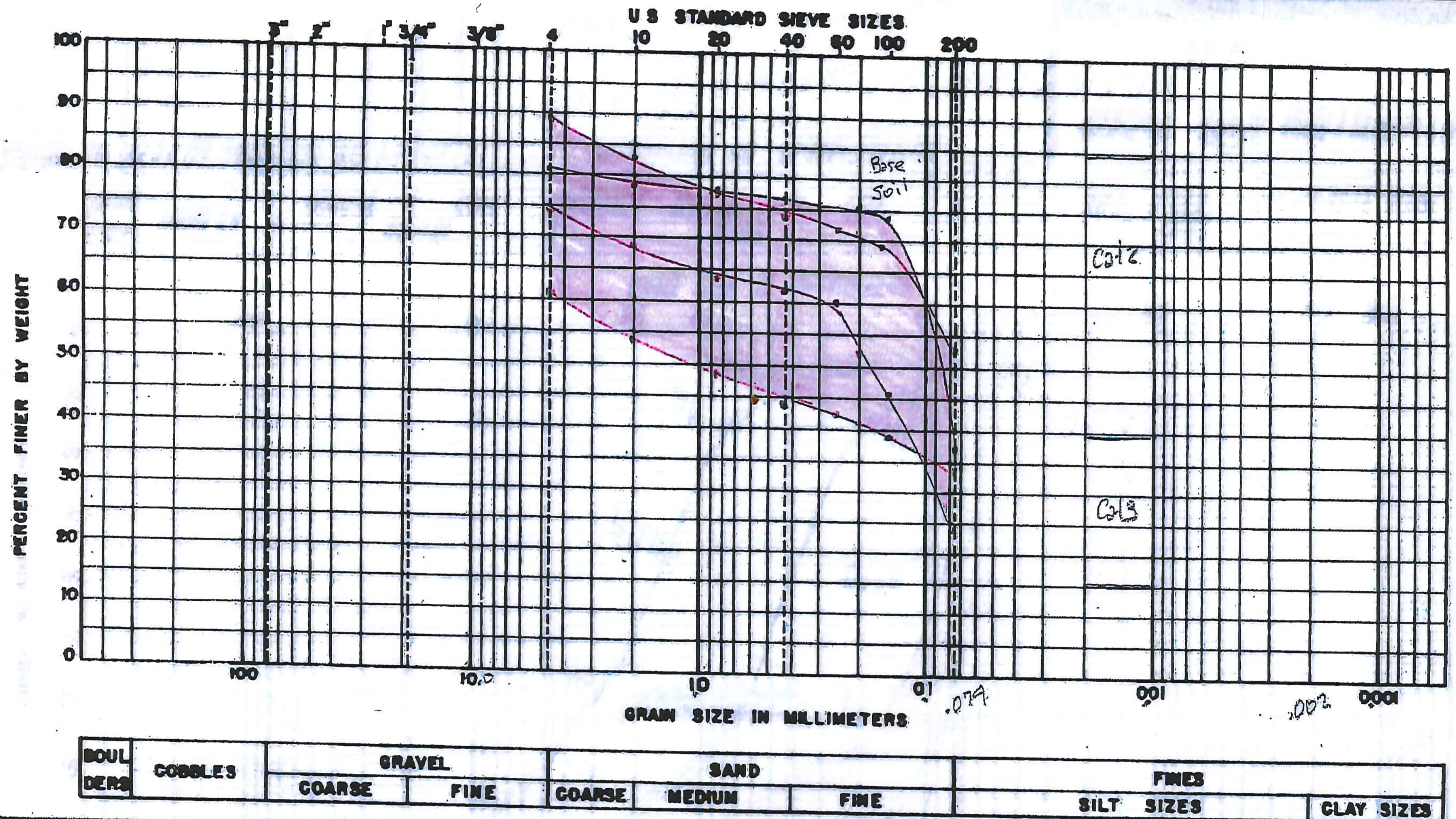
Correction Factor $\frac{100}{\% \#4}$

61 → 1.64
74.5 → 1.34
81.1 → 1.23
88.2 → 1.13

GC
SM w/gravel

uncorrected

3



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
P3-34	9.5-10.5					
P3-34	10.5-12					
P3-36	16.5-18					
P3-36	13.5-15					

GRAIN SIZE DISTRIBUTION

JOB NO. ________

[redacted]

ATTACHMENT C

DESIGN CALCULATIONS

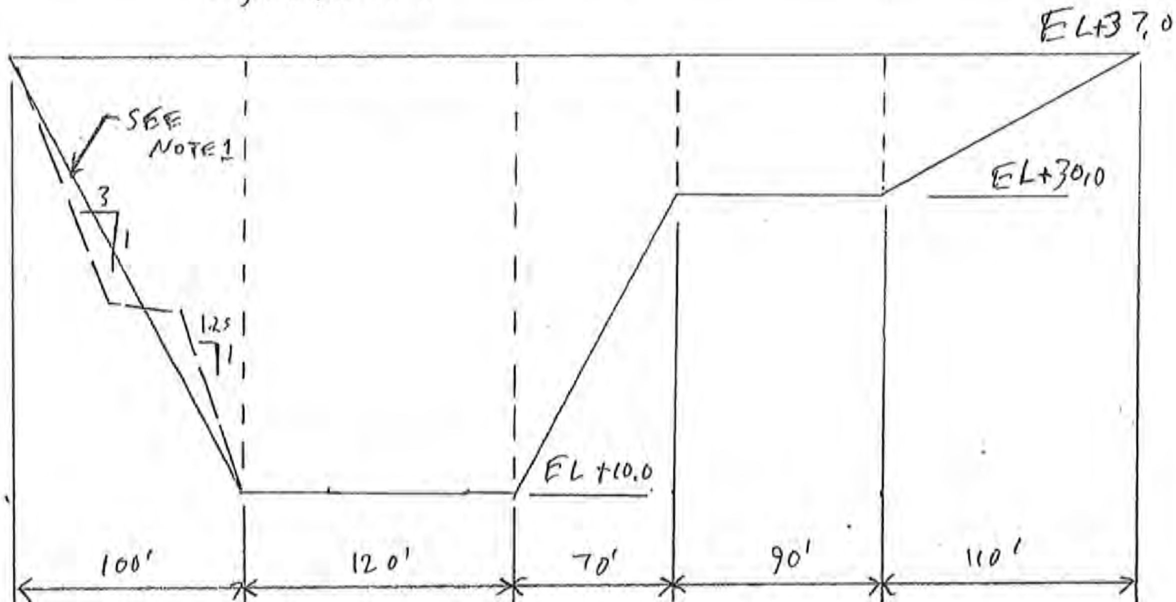
C.1 GEOTEXTILE
C.2 SOIL FILTER
C.3 ARTICULATED CONCRETE BLOCK MATRESS (ACBM)
C.4 ROCK RIPRAP
C.5 GATEWELL STRUCTURE 205

DESIGN NOTES AND COMPUTATIONS

SUBJECT: BROWNSVILLE IDEALIZED CHANNEL SECTIONS				
PREPARED BY CMA	DATE 12/2/2015	CHECKED BY SG 12/13/17	APPROVED BY	JOB NUMBER 4839-01
				SHEET NO. OF

SECTION 1 – UPSTREAM OF BRIGE
RUN A – 100 Year Flood Water Elevation +37

SEE NOTE 2



$A = AREA = (120')(27') + (70')(7') + (90')(7') + \frac{1}{2}\left[(100')(27') + (70')(20') + (110')(7')\right]$

$A = 6795$ SF

WETTED PERIMETER $= P = \sqrt{100^2 + 27^2} + 120' + \sqrt{70'^2 + 20^2} + 90' + \sqrt{110^2 + 7^2}$

$P = 497'$

NOTE:

1. SIDE SLOPE VARIES 1:3 for LEVEE + 1:1 1/2 for Armored Bank, Average Slope ≈ 27:100 ≈ 1:3
   Armored slope ≈ 12:20 ≈ 1:2 [SAY 1.25:1 Conservative]
2. RUN A0 increases velocity fill "No Good"
   RUN A00 increases velocity for 6" Block fill "No Good"

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3. The protrusion height, ΔZ, is a function of installation practice and block-to-block interface, and is often assumed to be 1/4 to 1/2 in. (6 to 13 mm). However, the designer must consider site-specific conditions and adjust ΔZ as required. The lift force, F'_L, resulting from the protrusion is conservatively assumed equal to the drag force, F'_D.

The factor of safety against loss of intimate contact is considered to be a function of design bed shear stress, critical shear stress, channel geometry and ACB unit geometry and weight. Figure 5 illustrates unit moment arms based on unit geometry.

The safety factor for a single ACB unit is determined from the ratio of restraining moments to overturning moments. Considering the submerged unit weight, W_S, unit moment arms and drag and lift forces, the safety factor, SF is defined as (ref. 3):

$$SF = \frac{\ell_2 W_S a_\theta}{\ell_1 W_S \sqrt{1-a_\theta^2}\cos\beta + \ell_3 F_D \cos\delta + \ell_4 F_L + \ell_3 F'_D \cos\delta + \ell_4 F'_L}$$

Dividing by $\ell_1 W_s$ and substituting terms, the equation for SF resolves to that presented in Table 4. Table 4 also outlines the calculations necessary for determining factor of safety.

DESIGN EXAMPLE

A trapezoidal channel section with 3H:1V side slopes ($Z = 3$, $\theta_1 = 18.4°$) and a base width b of 15 ft (4.6 m) requires stabilization. The 100-year design discharge is 450 ft³/s (12.7 m³/s), and the channel slope S_o is 0.03 ft/ft (0.03 m/m) ($\theta_0 = 1.72°$). The channel has a uniform bed and no flow obstructions (i.e. confluences, bends or changes in geometry). Manning's n is specified as 0.035. Based on design conditions, the energy grade line S_f is assumed equal to the channel slope S_o.

Step 1 Determine flow depth and cross-sectional averaged velocity:

$Q = 1.486/n\ A\ R^{2/3}\ S_f^{1/2}$

$A = by_o + Zy_o^2$, cross-sectional flow area

$P = b + 2(y_o^2 + (Zy_o)^2)^{1/2}$, wetted perimeter

$R = A/P$, hydraulic radius

By iteration, the flow depth y_o is determined to be 2.1 ft (0.6 m).

$V = Q/A = 450$ ft³/s / 44.73 ft² = 10.1 ft/s (3.1 m/s)

Step 2 Calculate design shear stress:

$\tau_{des} = \gamma\ R\ S_f =$ (62.4 lb/ft³)(1.582 ft)(0.03 ft/ft) = 2.96 psf (0.14 kPa)

Step 3 Select target factor of safety:

Assuming a base factor of safety SF_B equal to 1.3 for a channel bed, a low consequence of failure ($X_C = 1.2$), and an empirical hydraulic model ($X_M = 1.5$), the target factor of safety is:

$$SF_T = SF_B X_C X_M = (1.3)(1.2)(1.5) = 2.34$$

Step 4 Select potential ACB product and obtain geomorphic and critical shear stress data:

The proposed ACB manufacturer specifies a critical shear stress τ_c for the unit on a horizontal surface of 30 psf (1.4 kPa), submerged unit weight of 35 lb (16 kg) and dimensions of 15 (w) x 18 (l) x 5 (h) in. (381 x 457 x 127 mm).

Step 5 Calculate factor of safety against incipient unit movement:

Given;

W_s = 35 lb (16 kg)

b_u = 1.5 ft (460 mm)

τ_c = 30 psf (1.4 kPa)

η_o = 2.96/30 = 0.0987

and determining the following geometrically (see Figure 5);

ℓ_1 = 5/2/12 = 0.208 ft (64 mm)

$\ell_2 = \ell_4 = \sqrt{(18)^2 + (15)^2}$ /2/12 = 0.976 ft (297 mm)

ℓ_3 = 0.8(5)/12 = 0.333 ft (101 mm)

and assuming (see discussion);

ΔZ = 0.0417 ft (13 mm)

the following are calculated using the equations in Table 4:

$F'_L = F'_D$ = 6.14 lb (0.03 kN)

a_θ = 0.948

θ = 5.14°

β = 19.4°

η_1 = 0.0847

δ = 65.4°

SF = 2.72

Because the calculated factor of safety exceeds the target, the proposed ACB system is stable against loss of intimate contact.

NOTATIONS:

A = cross-sectional flow area, ft² (m²)

a_θ = projection of W_S into subgrade beneath block (Table 4)

b = width of channel base, ft (mm)

b_u = width of ACB unit in the direction of flow, ft (mm)

F_D = drag force, lb (kN)

F'_D = additional drag forces, lb (kN)

F_L = lift force, lb (kN)

F'_L = additional lift forces, lb (kN) (Table 4)

F_R = inter-block restraint, lb (kN)

ℓ_x = block moment arms, ft (mm)

n = Manning's roughness coefficient

Q = design discharge, ft³/s (m³/s)

R = hydraulic radius (A/wetted perimeter), ft (m)

S_C = specific gravity of concrete (assume 2.1)

S_f = energy grade line, ft/ft (m/m)

S_o = bed slope, ft/ft (m/m)

SF = calculated factor of safety (Table 4)

SF_B = base factor of safety (Table 1)

SF_T = target factor of safety

V = cross-sectional averaged flow velocity, ft/s (m/s)

W = weight of block, lb (kg)

W_s = submerged weight of block, lb (kg) (Table 4)

W_{s1} = gravity force parallel to slope, lb (kN)

W_{s2} = gravity force normal to slope, lb (kN)

X_C = multiplier based on consequence of failure (Table 2)

X_M = multiplier based on hydraulic model uncertainty (Table 3)

y_o = flow depth, ft (m)

Z = horizontal to vertical ratio of channel side slope

β = angle of block projection from downward direction, once in motion, degrees or radians

γ = unit weight of water, 62.4 pcf (1,000 kg/m³)

ΔZ = height of block protrusion above ACB matrix, ft (mm)

δ = angle between drag force and block motion, degrees or radians

η_o = stability number for a horizontal surface (Table 4)

η_1 = stability number for a sloped surface (Table 4)

θ = angle between side slope projection of W_s and the vertical, degrees or radians (Table 4)

θ_0 = channel bed slope, degrees or radians

θ_1 = channel side slope, degrees or radians

ρ = mass density of water, 1.94 slugs/ft³ (1,000 kg/m³)

τ_c = critical shear stress for block on a horizontal surface, lb/ft² (kPa)

τ_{des} = design shear stress, lb/ft² (kPa)

τ_o = cross-section averaged bed shear stress, lb/ft² (kPa)

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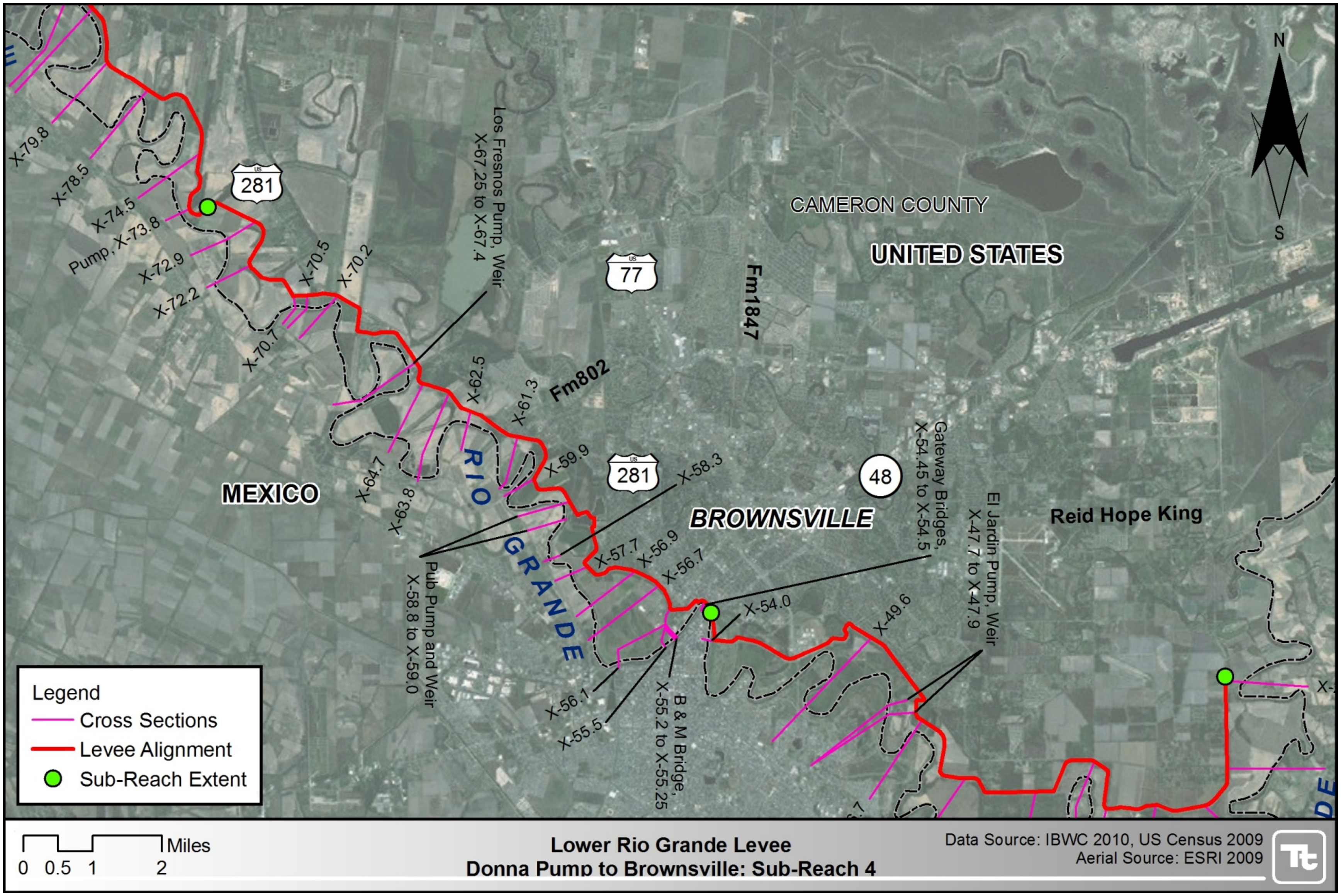
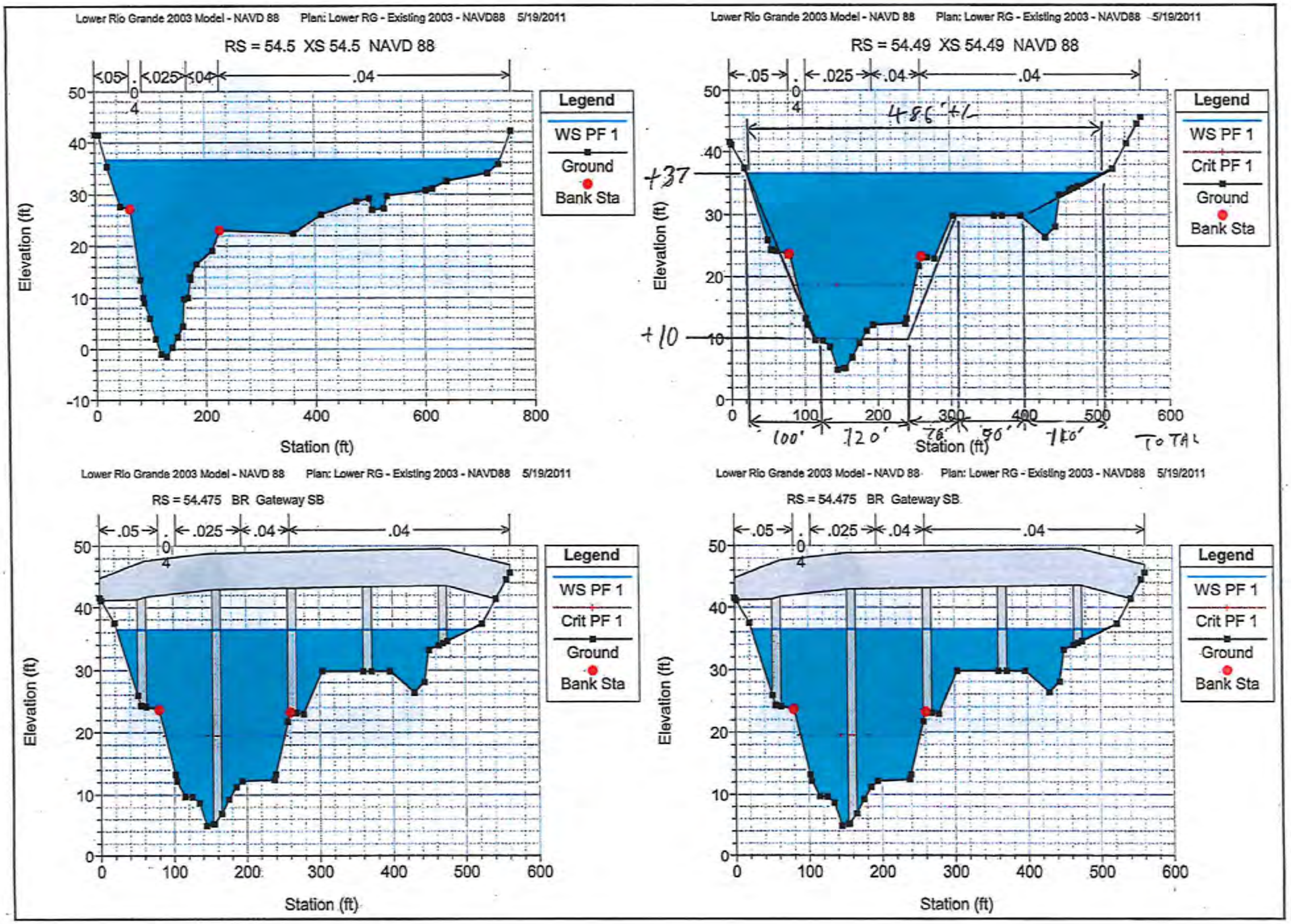
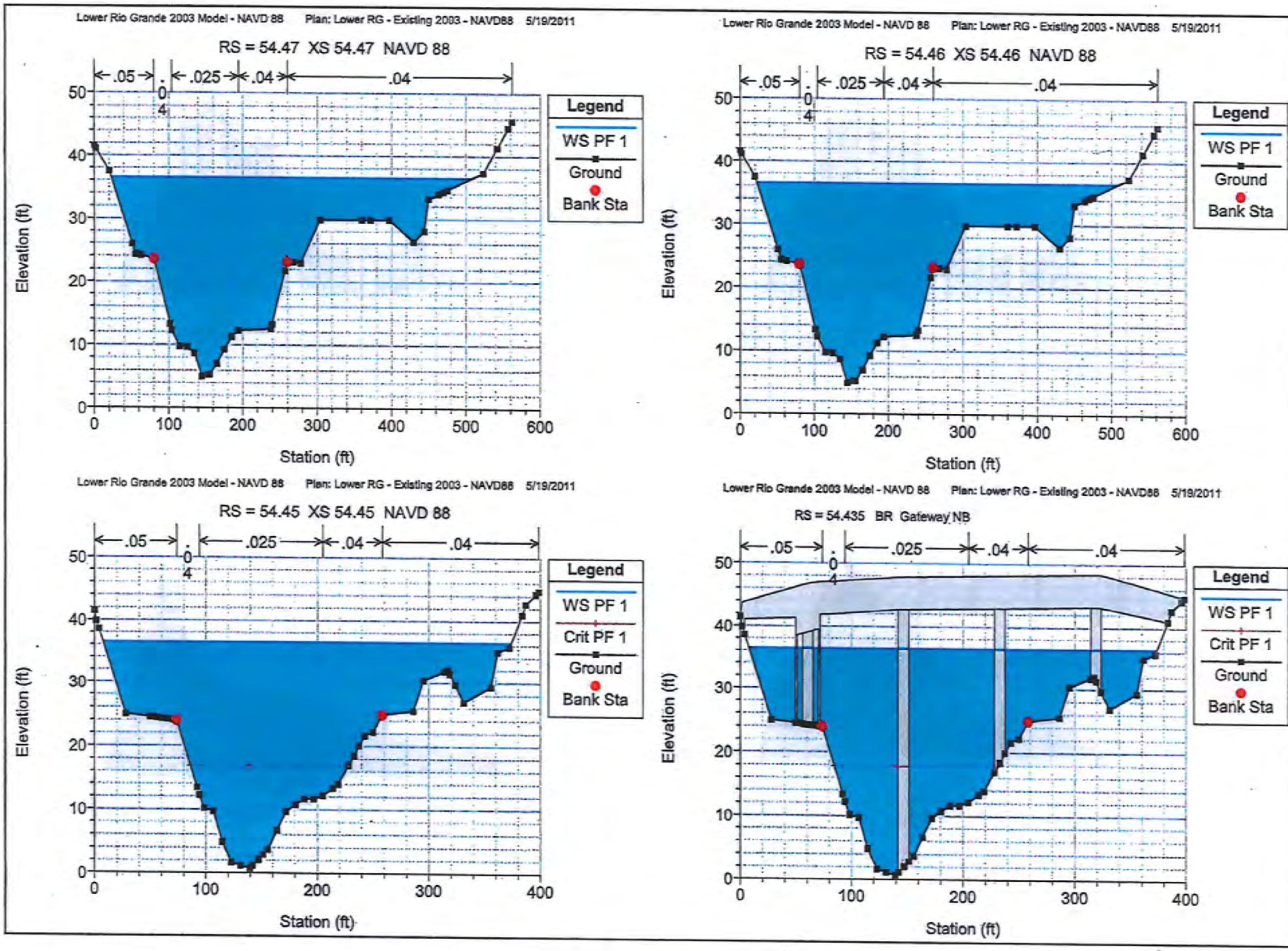


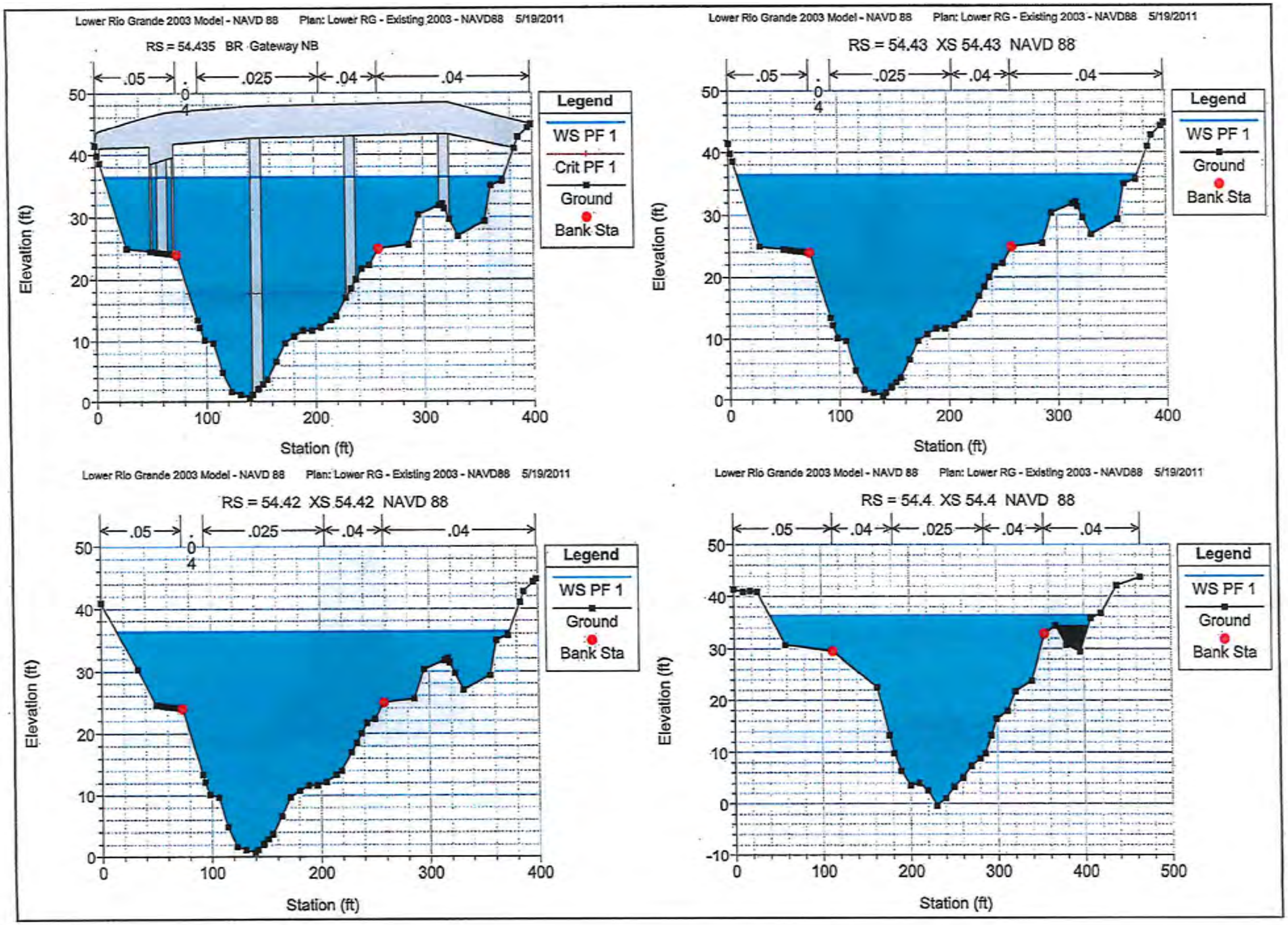
Figure 4.1 – Locations of HEC-RAS Cross Sections

SECTION 1

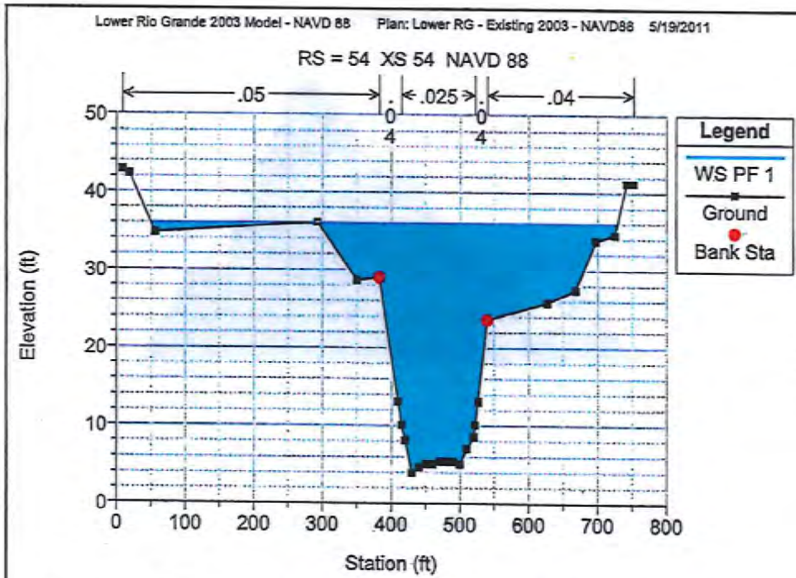


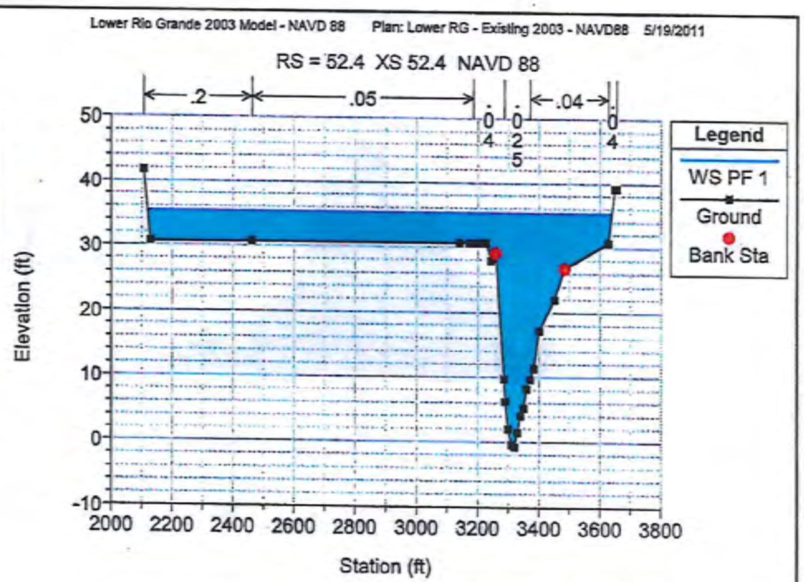
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.5 XS 54.5 NAVD 88
.05 .04 .025 .04 .04
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.49 XS 54.49 NAVD 88
.05 .04 .025 .04 .04
486'+/-
+37
+10
100' 120' 76' 50' 140' TOTAL
Legend
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.475 BR Gateway SB
.05 .04 .025 .04 .04
Legend
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.475 BR Gateway SB
.05 .04 .025 .04 .04
Legend
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)

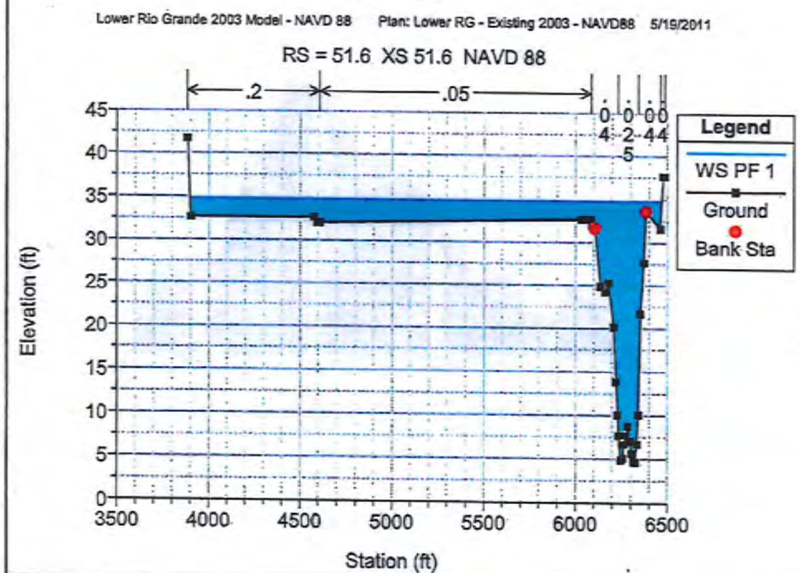

Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.47 XS 54.47 NAVD 88
.05
.04
.025
.04
.04
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.46 XS 54.46 NAVD 88
.05
.04
.025
.04
.04
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.45 XS 54.45 NAVD 88
.05
.04
.025
.04
.04
Legend
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.435 BR Gateway NB
.05
.04
.025
.04
.04
Legend
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)

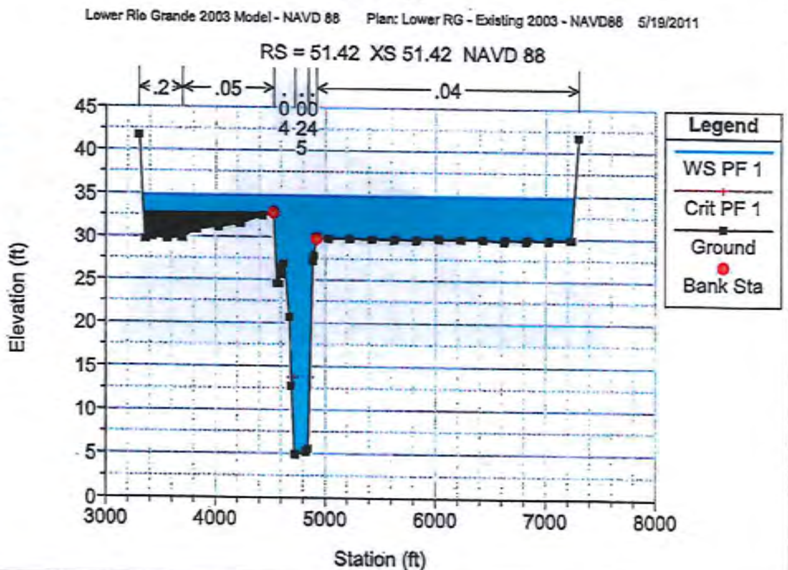


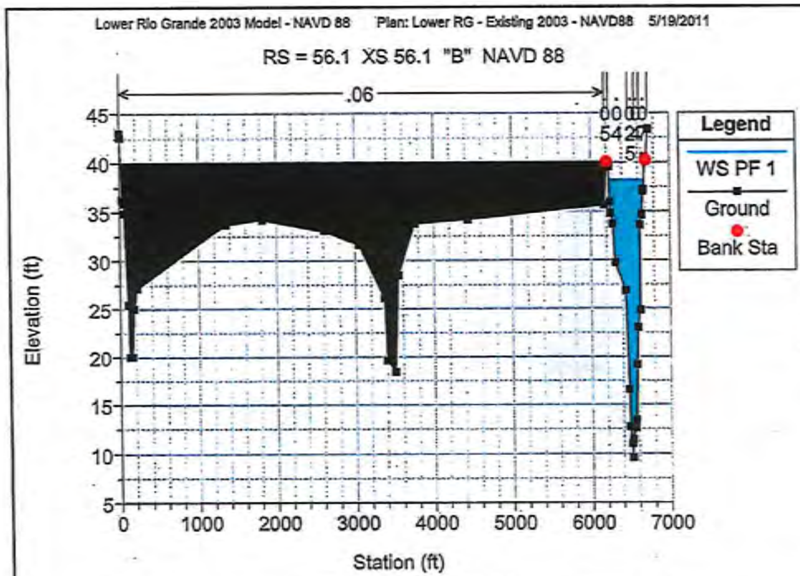
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.435 BR Gateway NB
.05
.04
.025
.04
.04
Legend
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
0
10
20
30
40
50
100
200
300
400
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.43 XS 54.43 NAVD 88
.05
.04
.025
.04
.04
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
0
10
20
30
40
50
100
200
300
400
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.42 XS 54.42 NAVD 88
.05
.04
.025
.04
.04
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
0
10
20
30
40
50
100
200
300
400
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54.4 XS 54.4 NAVD 88
.05
.04
.025
.04
.04
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
-10
0
10
20
30
40
50
100
200
300
400
500

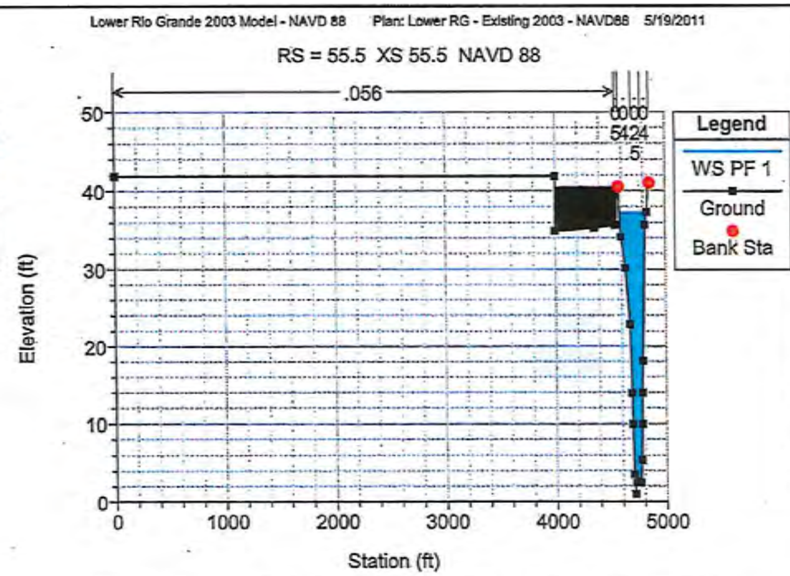

Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 54 XS 54 NAVD 88
.05
.025
.04
Elevation (ft)
Station (ft)
Legend
WS PF 1
Ground
Bank Sta

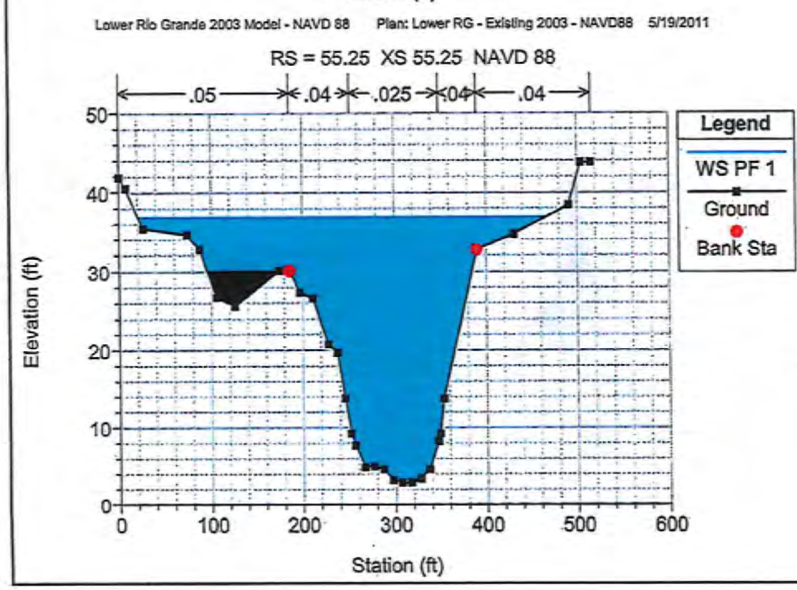

Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 52.4 XS 52.4 NAVD 88
.2
.05
.04
Elevation (ft)
Station (ft)
Legend
WS PF 1
Ground
Bank Sta

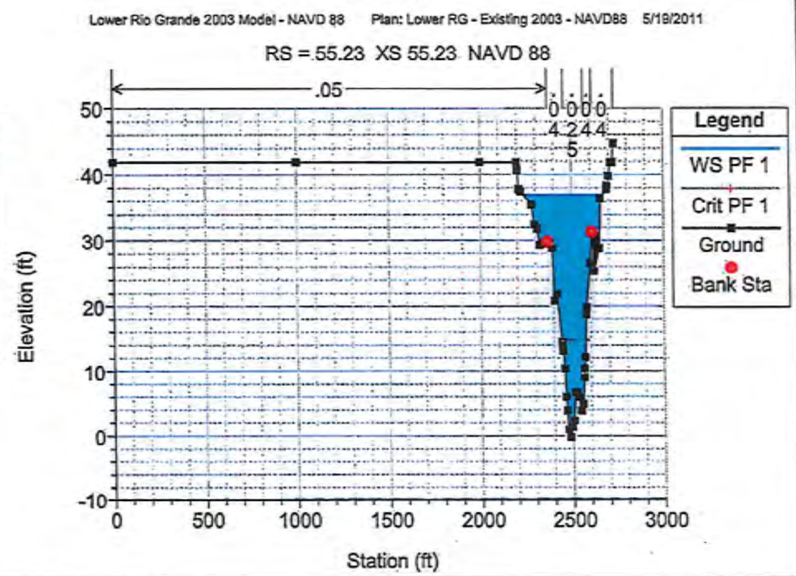

Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 51.6 XS 51.6 NAVD 88
.2
.05
Elevation (ft)
Station (ft)
Legend
WS PF 1
Ground
Bank Sta

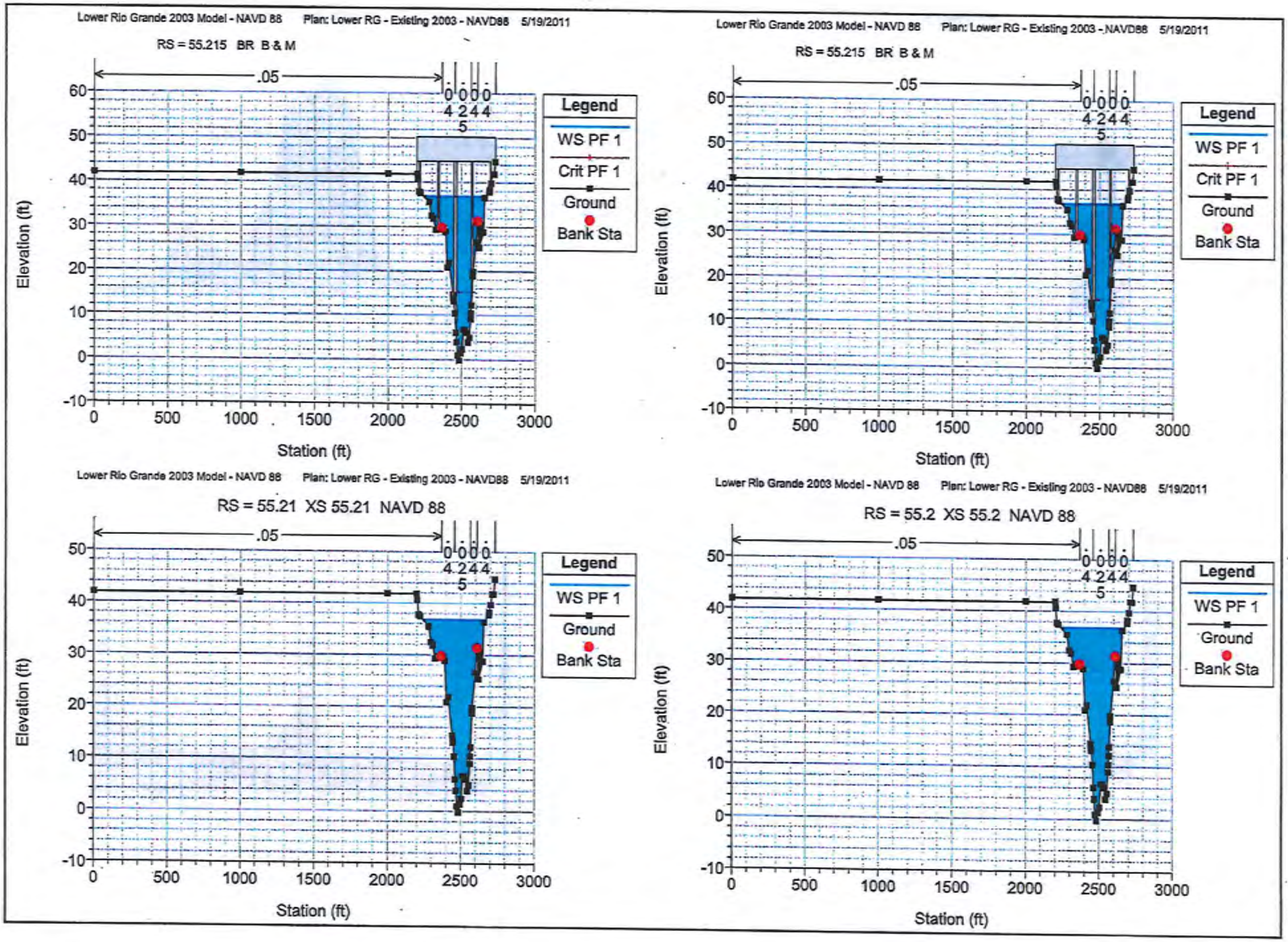

Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 51.42 XS 51.42 NAVD 88
.2
.05
.04
Elevation (ft)
Station (ft)
Legend
WS PF 1
Crit PF 1
Ground
Bank Sta


Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 56.1 XS 56.1 "B" NAVD 88
.06
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)


Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 55.5 XS 55.5 NAVD 88
.056
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)


Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 55.25 XS 55.25 NAVD 88
.05
.04
.025
.04
.04
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)


Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 55.23 XS 55.23 NAVD 88
.05
Legend
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)


Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 55.215 BR B & M
Legend
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 55.215 BR B & M
Legend
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 55.21 XS 55.21 NAVD 88
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Lower Rio Grande 2003 Model - NAVD 88 Plan: Lower RG - Existing 2003 - NAVD88 5/19/2011
RS = 55.2 XS 55.2 NAVD 88
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)

CHANNEL BED SLOPE AND ENERGY GRADE LINE

HEC-RAS Version 4.1.0 Jan 2010
U.S. Army Corps of Engineers
Hydrologic Engineering Center
609 Second Street
Davis, California

```
 X     X  XXXXXX   XXXX        XXXX     XX     XXXX
 X     X  X       X    X       X   X   X  X   X
 X     X  X       X            X   X  X    X  X
 XXXXXXX  XXXX    X      XXX   XXXX   XXXXXX   XXXX
 X     X  X       X            X X    X    X       X
 X     X  X       X    X       X  X   X    X       X
 X     X  XXXXXX   XXXX        X   X  X    X  XXXXX
```

PROJECT DATA
Project Title: Lower Rio Grande 2003 Model - NAVD 88
Project File : lrg2003NAVD88.prj
Run Date and Time: 5/19/2011 3:30:53 PM

Project in English units

Project Description:
CURRENT MAINTENANCE WITH CONTROLLED GROWTH

HEC-RAS BACKWATER MODEL OF THE RIO GRANDE , FROM BELOW BROWNSVILLE TO PEÑITAS.

RESULTS OF THIS MODEL ARE FROM HEC-RAS 3.0.1. OTHER VERSIONS MAY GIVE SLIGHTLY DIFFERENT RESULTS.

River sections generally surveyed during 1990. A few sections were surveyed earlier. This version contains 24 additional sections with respect to the 1992 model, that were surveyed in 2001.

Some of the geometry data at the dams, bridges, and floodway inlets were taken from construction drawings and other sources.

The flow of 235,000 cfs represents an attenuated flow from 250,000 cfs at Rio Grande City. An additional 5000 cfs attenuation in the flood peak is assumed to occur between Anzalduas Dam and Retamal Dam. Flows have been input to simulate this attenuation. Flow diversions at the Main Floodway in the United States and the Mexican Floodway are 105,000 cfs at each one, as agreed with Mexico in Minute No. 238.

Starting elevation at River Mile 28 does not include storm surge or tidal effects.

All elevations are referenced to Mean Sea Level Datum.

PLAN DATA

Plan Title: Lower RG - Existing 2003 - NAVD88
Plan File : p:\Watr\T27062 Donna to Brownsville\10 Hydraulics\Reduced XS Study\HEC-RAS\lrg2003NAVD88.p01

Geometry Title: LRGFCP 2003 NAVD 88
Geometry File : p:\Watr\T27062 Donna to Brownsville\10 Hydraulics\Reduced XS Study\HEC-RAS\lrg2003NAVD88.g01

Flow Title : LRGFCP Design Flood - Min. 238
Flow File : p:\Watr\T27062 Donna to Brownsville\10 Hydraulics\Reduced XS Study\HEC-RAS\lrg2003NAVD88.f01

Plan Summary Information:

Number of:				
Cross Sections	= 193	Multiple Openings	=	0
Culverts	= 0	Inline Structures	=	2
Bridges	= 9	Lateral Structures	=	0

Computational Information
Water surface calculation tolerance = 0.01
Critical depth calculation tolerance = 0.01
Maximum number of iterations = 20
Maximum difference tolerance = 0.3
Flow tolerance factor = 0.001

Computation Options
Critical depth computed only where necessary
Conveyance Calculation Method: At breaks in n values only
Friction Slope Method: Program Selects Appropriate method
Computational Flow Regime: Subcritical Flow

FLOW DATA

Flow Title: LRGFCP Design Flood - Min. 238
Flow File : p:\Watr\T27062 Donna to Brownsville\10 Hydraulics\Reduced XS Study\HEC-RAS\lrg2003NAVD88.f01

Flow Data (cfs)

River	Reach	RS	PF 1
RIVER-1	Reach-1	186	235000
RIVER-1	Reach-1	169.83	130000
RIVER-1	Reach-1	156.93	125000
RIVER-1	Reach-1	129.3	20000

Boundary Conditions

River	Reach	Profile	Upstream	Downstream
RIVER-1	Reach-1	PF 1		Normal S = 0.00015

Inline Structure Gate Openings
River = RIVER-1
Reach = Reach-1 RS = 169.13
Gate = Gate #1
# Open Open Ht
34.75 6
River = RIVER-1
Reach = Reach-1 RS = 129.215
Gate = Gate #1
# Open Open Ht
13.75 1

GEOMETRY DATA

Geometry Title: LRGFCP 2003 NAVD 88
Geometry File : p:\Watr\T27062 Donna to Brownsville\10 Hydraulics\Reduced XS Study\HEC-RAS\lrg2003NAVD88.g01

CROSS SECTION

RIVER: RIVER-1
REACH: Reach-1 RS: 186

INPUT
Description: XS 186 NAVD 88
Station Elevation Data num= 75

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	125.24	4	124.94	20	118.94	39	118.44	53	118.64
56	116.94	64	116.94	72	118.94	81	118.54	101	117.94
743	116.94	2095	117.64	2115	117.61	2134	119.86	2141	117.84
2147	113.94	2161	102.98	2169	96.64	2179	89.94	2189	87.94
2199	86.94	2209	85.64	2219	85.94	2229	86.94	2239	87.94
2249	89.64	2259	90.44	2269	90.94	2279	91.94	2289	92.74
2299	93.64	2309	93.64	2319	94.64	2329	95.64	2339	95.64
2349	95.64	2359	95.94	2369	96.64	2379	96.94	2389	96.74
2399	96.94	2409	97.64	2419	97.94	2429	98.94	2439	99.94
2449	99.94	2459	101.94	2462	102.98	2465	103.56	2497	109.74
2514	117.74	2526	118.11	2567	117.04	2576	117.08	2623	115.34
4197	113.74	4239	114.74	4255	119.24	4263	118.54	4270	119.24
4276	117.24	4288	116.74	4523	118.24	4528	120.24	4534	119.74
6507	116.44	6541	119.44	6563	118.74	6819	119.14	6847	119.74
9100	117.74	9111	121.14	9125	120.94	10226	120.54	10519	125.94

Manning's n Values num= 5

Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val
0	.114	2141	.066	2161	.015	2465	.066	2514	.062

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
2141	2514	3400	7800	4300	.1	.3

CROSS SECTION OUTPUT Profile #PF 1

		Element	Left OB	Channel	Right OB
E.G. Elev (ft)	134.27	Wt. n-Val.	0.114	0.029	0.062
Vel Head (ft)	0.09	Reach Len. (ft)	3400.00	7800.00	4300.00
W.S. Elev (ft)	134.18	Flow Area (sq ft)	35839.67	13937.11	128612.30
Crit W.S. (ft)		Area (sq ft)	35839.67	13937.11	128612.30
E.G. Slope (ft/ft)	0.000059	Flow (cfs)	23412.35	60884.38	150703.30
Q Total (cfs)	235000.00	Top Width (ft)	2141.00	373.00	8005.00
Top Width (ft)	10519.00	Avg. Vel. (ft/s)	0.65	4.37	1.17
Vel Total (ft/s)	1.32	Hydr. Depth (ft)	16.74	37.36	16.07
Max Chl Dpth (ft)	48.24	Conv. (cfs)	3046336.0	7922071.0	19609000.0
Conv. Total (cfs)	30577410.0	Wetted Per. (ft)	2152.18	385.94	8015.46
Length Wtd. (ft)	5222.43	Shear (lb/sq ft)	0.06	0.13	0.06
Min Ch El (ft)	85.94	Stream Power (lb/ft s)	10519.00	0.00	0.00
Alpha	3.38	Cum Volume (acre-ft)	336107.00	155279.80	353415.00
Frctn Loss (ft)	0.40	Cum SA (acres)	42281.89	6208.87	45824.43
C & E Loss (ft)	0.01				

Warning: The cross-section end points had to be extended vertically for the computed water surface.
Note: Manning's n values were composited to a single value in the main channel.

CROSS SECTION

RIVER: RIVER-1
REACH: Reach-1 RS: 184

Min El Prs (ft)	44.69	W.P. Total (ft)	514.98	514.68
Delta EG (ft)	0.05	Conv. Total (cfs)	1356297.0	1355823.0
Delta WS (ft)	0.05	Top Width (ft)	387.62	387.35
BR Open Area (sq ft)	9012.30	Frctn Loss (ft)	0.00	0.01
BR Open Vel (ft/s)	3.64	C & E Loss (ft)	0.00	0.03
Coef of Q		Shear Total (lb/sq ft)	0.14	0.14
Br Sel Method	Energy only	Power Total (lb/ft s)	0.00	0.00

Note: Manning's n values were composited to a single value in the main channel.
Warning: The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4.
This may indicate the need for additional cross sections.
Note: Manning's n values were composited to a single value in the main channel.

CROSS SECTION

RIVER: RIVER-1
REACH: Reach-1 RS: 55.21

INPUT
Description: XS 55.21 NAVD 88
Station Elevation Data num= 49

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	41.84	1000	41.84	2000	41.84	2200	41.84	2203.7	40.64
2212	37.84	2213.5	37.79	2221.5	37.52	2281	35.54	2297	32.44
2309	31.84	2325	29.34	2339	29.5	2348	29.61	2368	29.84
2392	28.84	2404	20.84	2415	21.84	2443	14.44	2445.2	13.76
2447.3	13.11	2456	10.36	2463	6.04	2467.3	3.87	2473	.94
2483	-.16	2493	1.34	2503	2.34	2513	6.64	2523	6.64
2533	6.04	2543	3.84	2553	4.94	2563	9.04	2564	10.36
2566.5	12.18	2575.5	18.75	2577	19.84	2599	26.54	2609	31.34
2619	25.34	2648	28.84	2655	36.44	2690	37.84	2693	38.39
2701	39.84	2712	41.84	2721	41.84	2729	44.69		

Manning's n Values num= 5

Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val
0	.05	2368	.04	2456	.025	2564	.04	2609	.04

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
2368	2609	15	15	15	.2	.5

Blocked Obstructions num= 1

Sta L	Sta R	Elev
2609	2729	31.34

CROSS SECTION OUTPUT Profile #PF 1

			Left OB	Channel	Right OB
E.G. Elev (ft)	37.09	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.18	Wt. n-Val.	0.050	0.034	0.040
W.S. Elev (ft)	36.90	Reach Len. (ft)	15.00	15.00	15.00
Crit W.S. (ft)		Flow Area (sq ft)	547.11	5523.25	246.59
E.G. Slope (ft/ft)	0.000105	Area (sq ft)	547.11	5523.25	246.59
Q Total (cfs)	20000.00	Flow (cfs)	437.70	19320.98	241.32
Top Width (ft)	426.52	Top Width (ft)	127.95	241.00	57.57
Vel Total (ft/s)	3.17	Avg. Vel. (ft/s)	0.80	3.50	0.98
Max Chl Dpth (ft)	37.06	Hydr. Depth (ft)	4.28	22.92	4.28
Conv. Total (cfs)	1951830.0	Conv. (cfs)	42715.4	1885563.0	23551.1
Length Wtd. (ft)	15.00	Wetted Per. (ft)	128.48	255.59	59.82
Min Ch El (ft)	-0.16	Shear (lb/sq ft)	0.03	0.14	0.03
Alpha	1.18	Stream Power (lb/ft s)	2729.00	0.00	0.00
Frctn Loss (ft)	0.00	Cum Volume (acre-ft)	4327.26	18104.83	3295.79
C & E Loss (ft)	0.00	Cum SA (acres)	1996.41	1004.12	1673.55

Note: Manning's n values were composited to a single value in the main channel.

CROSS SECTION

RIVER: RIVER-1
REACH: Reach-1 RS: 55.2

INPUT
Description: XS 55.2 NAVD 88
Station Elevation Data num= 50

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	41.84	1000	41.84	2000	41.84	2200	41.84	2203.7	40.64
2212	37.84	2213.5	37.79	2221.5	37.52	2281	35.54	2297	32.44
2309	31.84	2325	29.34	2339	29.5	2348	29.61	2368	29.84
2392	28.84	2404	20.84	2415	21.84	2443	14.44	2445.2	13.76
2447.3	13.11	2456	10.36	2463	6.04	2467.3	3.87	2473	.94
2483	-.16	2493	1.34	2503	2.34	2513	6.64	2523	6.64
2533	6.04	2543	3.84	2553	4.94	2563	9.04	2564	10.36
2566.5	12.18	2568.7	13.79	2575.5	18.75	2577	19.84	2599	26.54
2609	31.34	2619	25.34	2648	28.84	2655	36.44	2690	37.84
2693	38.39	2701	39.84	2712	41.84	2721	41.84	2729	44.69

Manning's n Values num= 5

Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val
0	.05	2368	.04	2456	.025	2564	.04	2609	.04

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
2368	2609	3875	3440	3875	.1	.3

Blocked Obstructions num= 1

Sta L	Sta R	Elev
2609	2729	31.34

CROSS SECTION OUTPUT Profile #PF 1

			Left OB	Channel	Right OB
E.G. Elev (ft)	37.09	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.18	Wt. n-Val.	0.050	0.034	0.040
W.S. Elev (ft)	36.90	Reach Len. (ft)	3875.00	3440.00	3875.00
Crit W.S. (ft)		Flow Area (sq ft)	546.91	5522.84	246.49
E.G. Slope (ft/ft)	0.000105	Area (sq ft)	546.91	5522.84	246.49
Q Total (cfs)	20000.00	Flow (cfs)	437.59	19321.10	241.31
Top Width (ft)	426.43	Top Width (ft)	127.90	241.00	57.53
Vel Total (ft/s)	3.17	Avg. Vel. (ft/s)	0.80	3.50	0.98
Max Chl Dpth (ft)	37.06	Hydr. Depth (ft)	4.28	22.92	4.28
Conv. Total (cfs)	1951579.0	Conv. (cfs)	42699.4	1885332.0	23546.9
Length Wtd. (ft)	3515.93	Wetted Per. (ft)	128.44	255.59	59.78
Min Ch El (ft)	-0.16	Shear (lb/sq ft)	0.03	0.14	0.03
Alpha	1.18	Stream Power (lb/ft s)	2729.00	0.00	0.00
Frctn Loss (ft)	0.31	Cum Volume (acre-ft)	4327.07	18102.93	3295.70
C & E Loss (ft)	0.02	Cum SA (acres)	1996.36	1004.03	1673.53

Note: Manning's n values were composited to a single value in the main channel.

CROSS SECTION

RIVER: RIVER-1
REACH: Reach-1 RS: 54.5

INPUT
Description: XS 54.5 NAVD 88
Station Elevation Data num= 35

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	41.54	7	41.44	22	35.34	45	27.74	63	27.24
81.3	13.55	86	10.03	88	9.04	98	6.04	108	2.04
118	-.96	128	-1.46	138	.34	148	2.44	158	4.54
160	9.74	168	10.03	172.3	13.56	173	14.14	184	16.54
213	19.14	227	23.14	361	22.44	412	26.14	477	28.64
500	29.24	505	27.04	526	27.34	532	29.74	603	30.74
615	31.14	642	32.54	716	34.04	736	35.74	759	42.14

Manning's n Values num= 5

Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val
0	.05	63	.04	86	.025	168	.04	227	.04

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
63	227	90	90	90	.1	.3

CROSS SECTION OUTPUT Profile #PF 1

			Left OB	Channel	Right OB
E.G. Elev (ft)	36.75	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.12	Wt. n-Val.	0.050	0.033	0.040
W.S. Elev (ft)	36.63	Reach Len. (ft)	90.00	90.00	90.00
Crit W.S. (ft)		Flow Area (sq ft)	283.80	4272.07	4487.14
E.G. Slope (ft/ft)	0.000073	Area (sq ft)	283.80	4272.07	4487.14
Q Total (cfs)	20000.00	Flow (cfs)	244.29	13696.70	6059.01
Top Width (ft)	720.39	Top Width (ft)	44.18	164.00	512.21
Vel Total (ft/s)	2.21	Avg. Vel. (ft/s)	0.86	3.21	1.35
Max Chl Dpth (ft)	38.09	Hydr. Depth (ft)	6.42	26.05	8.76
Conv. Total (cfs)	2334011.0	Conv. (cfs)	28509.3	1598412.0	707089.2
Length Wtd. (ft)	90.00	Wetted Per. (ft)	45.66	178.20	513.59
Min Ch El (ft)	-1.46	Shear (lb/sq ft)	0.03	0.11	0.04
Alpha	1.55	Stream Power (lb/ft s)	759.00	0.00	0.00
Frctn Loss (ft)	0.01	Cum Volume (acre-ft)	4290.12	17716.17	3085.16
C & E Loss (ft)	0.01	Cum SA (acres)	1988.71	988.04	1648.19

Note: Manning's n values were composited to a single value in the main channel.

CROSS SECTION

RIVER: RIVER-1
REACH: Reach-1 RS: 54.49

INPUT
Description: XS 54.49 NAVD 88
Station Elevation Data num= 41

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	41.54	2	41.13	20	37.44	50.8	25.91	55	24.34
56.5	24.3	62.3	24.14	80	23.64	101.9	13.23	104	12.23
115	9.74	125	9.64	135	8.64	145	4.94	153.5	5.2
155	5.24	165	6.94	165.1	6.94	175	9.24	185	11.24
194	12.23	238	12.44	239.6	13.23	256.9	21.72	260	23.24
268.4	23.1	278	22.94	304	29.84	360.3	29.84	371.8	29.84
397	29.84	430	26.34	444	28.04	450	33.14	463.7	33.93
469.4	34.26	475.2	34.59	523	37.34	542	41.36	557	44.54
562	45.64								

Manning's n Values num= 5

Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val
0	.05	80	.04	104	.025	194	.04	260	.04

Bank Sta: Left Right Lengths: Left Channel Right Coeff Contr. Expan.

80	260	.85	.85	.85	.2	.5

CROSS SECTION OUTPUT Profile #PF 1

			Left OB	Channel	Right OB
E.G. Elev (ft)	36.74	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.20	Wt. n-Val.	0.050	0.033	0.040
W.S. Elev (ft)	36.54	Reach Len. (ft)	10.00	10.00	10.00
Crit W.S. (ft)	18.62	Flow Area (sq ft)	512.28	4432.94	1673.78
E.G. Slope (ft/ft)	0.000108	Area (sq ft)	512.28	4432.94	1673.78
Q Total (cfs)	20000.00	Flow (cfs)	660.98	17061.74	2277.28
Top Width (ft)	486.59	Top Width (ft)	57.58	180.00	249.01
Vel Total (ft/s)	3.02	Avg. Vel. (ft/s)	1.29	3.85	1.36
Max Chl Dpth (ft)	31.60	Hydr. Depth (ft)	8.90	24.63	6.72
Conv. Total (cfs)	1928612.0	Conv. (cfs)	63738.9	1645274.0	219599.1
Length Wtd. (ft)	10.00	Wetted Per. (ft)	59.80	185.74	252.17
Min Ch El (ft)	4.94	Shear (lb/sq ft)	0.06	0.16	0.04
Alpha	1.41	Stream Power (lb/ft s)	562.00	0.00	0.00
Frctn Loss (ft)	0.00	Cum Volume (acre-ft)	4289.30	17707.18	3078.79
C & E Loss (ft)	0.01	Cum SA (acres)	1988.61	987.69	1647.40

Warning: The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4.
This may indicate the need for additional cross sections.
Note: Manning's n values were composited to a single value in the main channel.

BRIDGE

RIVER: RIVER-1
REACH: Reach-1 RS: 54.475

INPUT
Description: Gateway SB

Distance from Upstream XS = 10
Deck/Roadway Width = 36
Weir Coefficient = 2.6
Upstream Deck/Roadway Coordinates
num= 15

Sta	Hi Cord	Lo Cord	Sta	Hi Cord	Lo Cord	Sta	Hi Cord	Lo Cord
0	44.79	41.54	2	44.88	41.13	50.8	47.14	41.29
62.3	47.61	41.71	153.5	48.85	42.95	165	48.87	42.97
256.9	49.07	43.17	268.4	49.09	43.19	360.3	49.29	43.39
371.8	49.31	43.41	463.7	49.47	43.57	475.2	49.44	43.54
542	47.52	41.36	557	47.08	44.54	562	46.93	45.66

Upstream Bridge Cross Section Data
Station Elevation Data num= 41

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	41.54	2	41.13	20	37.44	50.8	25.91	55	24.34
56.5	24.3	62.3	24.14	80	23.64	101.9	13.23	104	12.23
115	9.74	125	9.64	135	8.64	145	4.94	153.5	5.2
155	5.24	165	6.94	165.1	6.94	175	9.24	185	11.24
194	12.23	238	12.44	239.6	13.23	256.9	21.72	260	23.24
268.4	23.1	278	22.94	304	29.84	360.3	29.84	371.8	29.84
397	29.84	430	26.34	444	28.04	450	33.14	463.7	33.93
469.4	34.26	475.2	34.59	523	37.34	542	41.36	557	44.54
562	45.64								

Manning's n Values num= 5

Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val
0	.05	80	.04	104	.025	194	.04	260	.04

Bank Sta: Left	Right	Coeff Contr.	Expan.
80	260	.2	.5

Downstream Deck/Roadway Coordinates
num= 15

Sta	Hi Cord	Lo Cord	Sta	Hi Cord	Lo Cord	Sta	Hi Cord	Lo Cord
0	44.79	41.54	2	44.88	41.13	50.8	47.14	41.29
62.3	47.61	41.71	153.5	48.85	42.95	165	48.87	42.97
256.9	49.07	43.17	268.4	49.09	43.19	360.3	49.29	43.39
371.8	49.31	43.41	463.7	49.47	43.57	475.2	49.44	43.54
542	47.52	41.36	557	47.08	44.54	562	46.93	45.66

Downstream Bridge Cross Section Data
Station Elevation Data num= 41

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	41.54	2	41.13	20	37.44	50.8	25.91	55	24.34
56.5	24.3	62.3	24.14	80	23.64	101.9	13.23	104	12.23
115	9.74	125	9.64	135	8.64	145	4.94	153.5	5.2
155	5.24	165	6.94	165.1	6.94	175	9.24	185	11.24
194	12.23	238	12.44	239.6	13.23	256.9	21.72	260	23.24
268.4	23.1	278	22.94	304	29.84	360.3	29.84	371.8	29.84
397	29.84	430	26.34	444	28.04	450	33.14	463.7	33.93
469.4	34.26	475.2	34.59	523	37.34	542	41.36	557	44.54
562	45.64								

Manning's n Values num= 5

Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val
0	.05	80	.04	104	.025	194	.04	260	.04

Bank Sta: Left	Right	Coeff Contr.	Expan.
80	260	.2	.5

Upstream Embankment side slope = 0 horiz. to 1.0 vertical
Downstream Embankment side slope = 0 horiz. to 1.0 vertical
Maximum allowable submergence for weir flow = .95
Elevation at which weir flow begins =
Energy head used in spillway design =
Spillway height used in design =
Weir crest shape = Broad Crested

Number of Piers = 5

Pier Data
Pier Station Upstream= 56.55 Downstream= 56.55
Upstream num= 2

Width	Elev	Width	Elev
11.5	0	11.5	42.07

Downstream num= 2

Width	Elev	Width	Elev
11.5	0	11.5	42.07

Pier Data
Pier Station Upstream= 159.25 Downstream= 159.25
Upstream num= 2

Width	Elev	Width	Elev
11.5	0	11.5	43.33

Downstream num= 2

Width	Elev	Width	Elev
11.5	0	11.5	43.33

Pier Data
Pier Station Upstream= 262.65 Downstream= 262.65
Upstream num= 2

Width	Elev	Width	Elev
11.5	0	11.5	43.55

Downstream num= 2

Width	Elev	Width	Elev
11.5	0	11.5	43.55

Pier Data
Pier Station Upstream= 366.05 Downstream= 366.05
Upstream num= 2

Width	Elev	Width	Elev
11.5	0	11.5	43.77

Downstream num= 2

Width	Elev	Width	Elev
11.5	0	11.5	43.77

Pier Data
Pier Station Upstream= 469.45 Downstream= 469.45
Upstream num= 2

Width	Elev	Width	Elev
11.5	0	11.5	43.93

Downstream num= 2

Width	Elev	Width	Elev
11.5	0	11.5	43.93

Number of Bridge Coefficient Sets = 1

Low Flow Methods and Data
Energy
Selected Low Flow Methods = Highest Energy Answer

High Flow Method
Pressure and Weir flow
Submerged Inlet Cd =
Submerged Inlet + Outlet Cd = .8
Max Low Cord =

Additional Bridge Parameters
Add Friction component to Momentum
Do not add Weight component to Momentum
Class B flow critical depth computations use critical depth
inside the bridge at the upstream end
Criteria to check for pressure flow = Upstream energy grade line

BRIDGE OUTPUT Profile #PF 1

			Inside BR US	Inside BR DS
E.G. US. (ft)	36.74	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	36.54	E.G. Elev (ft)	36.73	36.72
Q Total (cfs)	20000.00	W.S. Elev (ft)	36.50	36.49
Q Bridge (cfs)	20000.00	Crit W.S. (ft)	19.45	19.45
Q Weir (cfs)		Max Chl Dpth (ft)	31.56	31.55
Weir Sta Lft (ft)		Vel Total (ft/s)	3.42	3.42
Weir Sta Rgt (ft)		Flow Area (sq ft)	5853.79	5850.35
Weir Submerg		Froude # Chl	0.15	0.15
Weir Max Depth (ft)		Specif Force (cu ft)	60827.31	60781.69
Min El Weir Flow (ft)	44.80	Hydr Depth (ft)	13.67	13.66
Min El Prs (ft)	45.66	W.P. Total (ft)	569.34	569.10
Delta EG (ft)	0.04	Conv. Total (cfs)	1359573.0	1358769.0

Delta WS (ft)	0.04	Top Width (ft)	428.29	428.13
BR Open Area (sq ft)	8773.44	Frctn Loss (ft)	0.01	0.01
BR Open Vel (ft/s)	3.42	C & E Loss (ft)	0.00	0.02
Coef of Q		Shear Total (lb/sq ft)	0.14	0.14
Br Sel Method	Energy only	Power Total (lb/ft s)	0.00	0.00

Note: Manning's n values were composited to a single value in the main channel.
Note: Manning's n values were composited to a single value in the main channel.

CROSS SECTION

RIVER: RIVER-1
REACH: Reach-1 RS: 54.47

INPUT
Description: XS 54.47 NAVD 88
Station Elevation Data num= 41

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	41.54	2	41.13	20	37.44	50.8	25.91	55	24.34
56.5	24.3	62.3	24.14	80	23.64	101.9	13.23	104	12.23
115	9.74	125	9.64	135	8.64	145	4.94	153.5	5.2
155	5.24	165	6.94	165.1	6.94	175	9.24	185	11.24
194	12.23	238	12.44	239.6	13.23	256.9	21.72	260	23.24
268.4	23.1	278	22.94	304	29.84	360.3	29.84	371.8	29.84
397	29.84	430	26.34	444	28.04	450	33.14	463.7	33.93
469.4	34.26	475.2	34.59	523	37.34	542	41.36	557	44.54
562	45.64								

Manning's n Values num= 5

Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val
0	.05	80	.04	104	.025	194	.04	260	.04

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
80	260	5	5	5	.2	.5

CROSS SECTION OUTPUT Profile #PF 1

		Element	Left OB	Channel	Right OB
E.G. Elev (ft)	36.70	Wt. n-Val.	0.050	0.033	0.040
Vel Head (ft)	0.20	Reach Len. (ft)	5.00	5.00	5.00
W.S. Elev (ft)	36.50	Flow Area (sq ft)	510.05	4425.97	1664.15
Crit W.S. (ft)		Area (sq ft)	510.05	4425.97	1664.15
E.G. Slope (ft/ft)	0.000108	Flow (cfs)	659.20	17073.76	2267.04
Q Total (cfs)	20000.00	Top Width (ft)	57.48	180.00	248.34
Top Width (ft)	485.82	Avg. Vel. (ft/s)	1.29	3.86	1.36
Vel Total (ft/s)	3.03	Hydr. Depth (ft)	8.87	24.59	6.70
Max Chl Dpth (ft)	31.56	Conv. (cfs)	63355.8	1640966.0	217886.6
Conv. Total (cfs)	1922208.0	Wetted Per. (ft)	59.69	186.74	251.50
Length Wtd. (ft)	5.00	Shear (lb/sq ft)	0.06	0.16	0.04
Min Ch El (ft)	4.94	Stream Power (lb/ft s)	562.00	0.00	0.00
Alpha	1.41	Cum Volume (acre-ft)	4288.49	17699.09	3075.84
Frctn Loss (ft)	0.00	Cum SA (acres)	1988.51	987.35	1646.96
C & E Loss (ft)	0.00				

Note: Manning's n values were composited to a single value in the main channel.

CROSS SECTION

RIVER: RIVER-1
REACH: Reach-1 RS: 54.46

INPUT
Description: XS 54.46 NAVD 88
Station Elevation Data num= 40

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	41.54	2	41.13	20	37.44	50.8	25.91	55	24.34
56.5	24.3	62.3	24.14	80	23.64	101.8	13.28	104	12.23
115	9.74	125	9.64	135	8.64	145	4.94	153.5	5.2
155	5.24	165	6.94	175	9.24	185	11.24	194	12.23
238	12.44	239.7	13.27	256.9	21.72	260	23.24	268.4	23.1
278	22.94	304	29.84	360.3	29.84	371.8	29.84	397	29.84
430	26.34	444	28.04	450	33.14	463.7	33.93	469.4	34.26
475.2	34.59	523	37.34	542	41.36	557	44.54	562	45.64

Manning's n Values num= 5

Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val
0	.05	80	.04	104	.025	194	.04	260	.04

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
80	260	4	4	4	.1	.3

CROSS SECTION OUTPUT Profile #PF 1

		Element	Left OB	Channel	Right OB
E.G. Elev (ft)	36.70	Wt. n-Val.	0.050	0.033	0.040
Vel Head (ft)	0.20	Reach Len. (ft)	4.00	4.00	4.00
W.S. Elev (ft)	36.50	Flow Area (sq ft)	510.02	4425.81	1664.01
Crit W.S. (ft)		Area (sq ft)	510.02	4425.81	1664.01
E.G. Slope (ft/ft)	0.000108	Flow (cfs)	659.18	17073.89	2266.93
Q Total (cfs)	20000.00	Top Width (ft)	57.48	180.00	248.33
Top Width (ft)	485.81	Avg. Vel. (ft/s)	1.29	3.86	1.36
Vel Total (ft/s)	3.03	Hydr. Depth (ft)	8.87	24.59	6.70
Max Chl Dpth (ft)	31.56				
Conv. Total (cfs)	1922089.0	Conv. (cfs)	63350.2	1640877.0	217861.7
Length Wtd. (ft)	4.00	Wetted Per. (ft)	59.69	186.74	251.49
Min Ch El (ft)	4.94	Shear (lb/sq ft)	0.05	0.16	0.04
Alpha	1.41	Stream Power (lb/ft s)	562.00	0.00	0.00
Frctn Loss (ft)	0.00	Cum Volume (acre-ft)	4288.44	17698.58	3075.65
C & E Loss (ft)	0.00	Cum SA (acres)	1988.50	987.33	1646.93

Note: Manning's n values were composited to a single value in the main channel.

CROSS SECTION

RIVER: RIVER-1
REACH: Reach-1 RS: 54.45

INPUT
Description: XS 54.45 NAVD 88
Station Elevation Data num= 53

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	41.44	1.6	39.81	3.8	38.59	27.9	24.94	49.7	24.47
51.3	24.43	56	24.33	60.8	24.22	65.4	24.12	70.1	24.02
71.7	23.98	73.8	23.94	92.3	13.33	94.3	12.18	98.4	10.14
106.6	9.64	114.8	4.84	123	1.64	131.2	1.14	139.4	.64
142.1	1.14	146.9	2	147.6	2.14	151.5	2.86	155.8	3.64
164	6.64	172.2	9.64	180.4	10.74	188.6	11.64	196.8	11.64
205	12.18	214.1	13.33	218.9	13.94	228.2	16.95	233	18.48
237.6	20	242.7	21.64	250.1	22.24	258.3	24.94	286.2	25.54
295.2	30.34	314.3	31.81	317.3	32.04	319.1	31.42	323.7	29.69
331.3	26.94	355.9	29.34	361.6	34.94	372.3	35.74	383.4	40.99
387	42.74	395.9	44.28	398.5	44.74				

Manning's n Values num= 5

Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val	Sta	n Val
0	.05	73.8	.04	94.3	.025	205	.04	258.3	.04

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
73.8	258.3	80	80	80	.2	.5

CROSS SECTION OUTPUT Profile #PF 1

		Element	Left OB	Channel	Right OB
E.G. Elev (ft)	36.70	Wt. n-Val.	0.050	0.032	0.040
Vel Head (ft)	0.21	Reach Len. (ft)	5.00	5.00	5.00
W.S. Elev (ft)	36.49	Flow Area (sq ft)	670.52	4656.70	848.51
Crit W.S. (ft)	16.86	Area (sq ft)	670.52	4656.70	848.51
E.G. Slope (ft/ft)	0.000095	Flow (cfs)	888.37	17973.35	1138.28
Q Total (cfs)	20000.00	Top Width (ft)	66.28	184.50	115.58
Top Width (ft)	366.36	Avg. Vel. (ft/s)	1.32	3.86	1.34
Vel Total (ft/s)	3.24	Hydr. Depth (ft)	10.12	25.24	7.34
Max Chl Dpth (ft)	35.85	Conv. (cfs)	90449.1	1829952.0	115893.5
Conv. Total (cfs)	2036294.0	Wetted Per. (ft)	69.34	193.32	120.35
Length Wtd. (ft)	5.00	Shear (lb/sq ft)	0.06	0.15	0.04
Min Ch El (ft)	0.64	Stream Power (lb/ft s)	398.50	0.00	0.00
Alpha	1.29	Cum Volume (acre-ft)	4288.38	17698.16	3075.54
Frctn Loss (ft)	0.00	Cum SA (acres)	1988.50	987.32	1646.92
C & E Loss (ft)	0.01				

Note: Manning's n values were composited to a single value in the main channel.

BRIDGE

RIVER: RIVER-1
REACH: Reach-1 RS: 54.435

INPUT
Description: Gateway NB

Distance from Upstream XS = 5
Deck/Roadway Width = 36
Weir Coefficient = 2.6
Upstream Deck/Roadway Coordinates
num= 22

Sta	Hi Cord	Lo Cord	Sta	Hi Cord	Lo Cord	Sta	Hi Cord	Lo Cord
0	41.44	40.74	1.6	43.57	39.81	3.8	43.77	38.59
3.8	43.77	41.04	49.7	46.16	41.25	49.7	46.16	38.52
51.3	46.25	38.52	56	46.5	38.76	65.4	46.85	39.25
70.1	46.91	39.49	71.7	46.93	39.49	71.71	46.93	41.75
142.1	47.81	42.63	151.5	47.84	42.67	228.2	48.13	42.95
237.6	48.16	42.98	314.3	48.43	43.25	323.7	48.41	43.23
383.4	45.53	40.99	387	45.35	42.74	395.9	44.93	44.28
398.5	44.74	44.74						

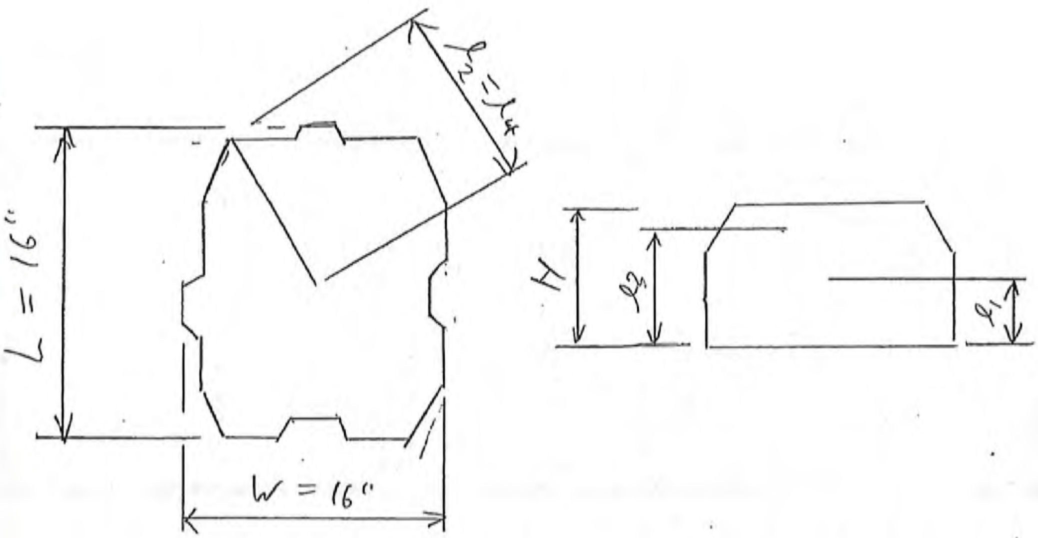
Upstream Bridge Cross Section Data
Station Elevation Data num= 53

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	41.44	1.6	39.81	3.8	38.59	27.9	24.94	49.7	24.47

DESIGN NOTES AND COMPUTATIONS

SUBJECT: BROWNSVILLE BLOCK MOMENT ARMS				
PREPARED BY CMA	DATE 12/2/2015	CHECKED BY	APPROVED BY	JOB NUMBER
				SHEET NO. OF

SHOREBLOCK BP SERIES



$l_2 = l_4 = 8.9''$ $l_1 = 1/2\,H$ $l_3 = 8/10\,(H)$

SEE SPREAD SHEET

$\frac{23.661}{16''} = 1.475$ $13.1/(1.475) = 8.88''$

17.2

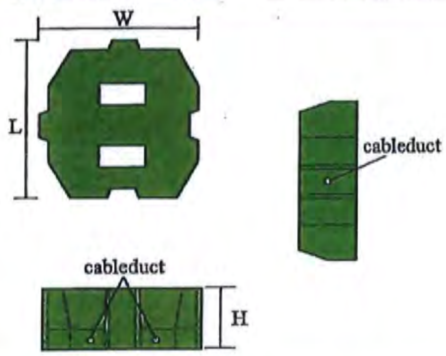
SHOREBLOCK™
BD Series

SPECIFICATIONS

OPEN CELL UNITS	Dimensions In.			Block		UNIT Coverage Sq. Ft.	Open Area %
	H	W	L	Unit Weight Lbs.	System Weight Lbs./Sq. Ft.		
BD-400 OC	4.00	15 7/8	15 7/8	56-62	32-35	1.78	20
BD-500 OC	5.00	15 7/8	15 7/8	71-76	40-44	1.78	20
BD-600 OC	6.00	15 7/8	15 7/8	86-93	50-54	1.78	20
BD-800 OC	8.00	15 7/8	15 7/8	114-130	66-72	1.78	20
BD-900 OC	9.00	15 7/8	15 7/8	127 - 146	73-82	1.78	20
BD-400S OC	4.00	15 7/8	11 7/8	43-46.	32-35	1.35	18
BD-500S OC	5.00	15 7/8	11 7/8	54-57	40-43	1.35	18
BD-600S OC	6.00	15 7/8	11 7/8	67-71	50-53	1.35	18

CLOSED CELL UNITS	Dimensions In.			Block		UNIT Coverage Sq. Ft.	Open Area %
	H	W	L	Unit Weight Lbs.	System Weight Lbs./Sq. Ft.		
BD-400 CC	4.00	15 7/8	15 7/8	66-72	37-40	1.78	7
BD-500 CC	5.00	15 7/8	15 7/8	82-88	46-51	1.78	7
BD-600 CC	6.00	15 7/8	15 7/8	101-108	58-62	1.78	7
BD-800 CC	8.00	15 7/8	15 7/8	130-150	74-83	1.78	7
BD-900 CC	9.00	15 7/8	15 7/8	148 - 170	84-97	1.78	7
BD-400S CC	4.00	15 7/8	11 7/8	50-54	37-40	1.35	7
BD-500S CC	5.00	15 7/8	11 7/8	61-65	45-48	1.35	7
BD-600S CC	6.00	15 7/8	11 7/8	75-81	56-60	1.35	7

***The BD Series denotes the Bi-Directional Cable System.**
Note: Additional block styles may be available in some areas.
Check with your local SHORETEC™, LLC representative for product availability.



Technical Specifications:

1) SHOREBLOCK™ Units are manufactured in accordance with ASTM C90 and C140 and the following criteria:
   - A) Concrete Unit Weight 130-150 lbs./CF
   - B) Minimum Compression Strength 4,000 PSI
   - C) Maximum Absorption 7%
   - D) Dimensional Tolerance +/- 1/8"
2) Galvanized or Polyester Cabling

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4. Hydrology and Hydraulics

4.1 Hydrology

No hydrologic analysis was performed by Tetra Tech for this project. Based on a study completed by USIBWC in 2003, the design flood for the LRGFCP is based on a peak flow of 250,000 cubic feet per second (cfs) at Rio Grande City, which attenuates to 235,000 cfs at Peñitas. During the design flood, Anzalduas Diversion Dam and Retamal Diversion Dam would each divert 105,000 cfs into the United States and Mexico. Flow diversion during the design flood would limit flood flows through the Brownsville-Matamoros area to 20,000 cfs (USIBWC 2007).

4.2 Hydraulic Model

The hydraulic analysis of the project reach along the Rio Grande was provided by USIBWC (2003). The hydraulic model was created using the Hydrologic Engineering Center-River Analysis System (HEC-RAS) program, version 3.0 (USACE 2001). The 2003 model was based on the geometrical cross-sectional data used in a 1992 HEC-2 model for the channel of the Rio Grande and the floodways in the United States. The Rio Grande data were modified for the established Restricted Use Zone (floodplains on both sides of the Rio Grande) and additional bridges and to include data for 24 additional new cross sections that were surveyed.

The 2003 HEC-RAS model contains 153 cross sections between Anzalduas Diversion Dam and the downstream extent of the U.S. levee embankment near Brownsville, and it extends beyond the limits of this study at the upstream end. A total of 113 HEC-RAS cross sections are located within the project limits for the Donna Pump to Brownsville levee rehabilitation; 33 HEC-RAS cross sections are located within Upper Brownsville Levee. The locations of HEC-RAS cross sections within Upper Brownsville Levee are shown on Figure 4.1.

The computed water surface elevations for the design discharge of 20,000 cfs were used in the hydraulic analysis for the levee rehabilitation (Table 4.1). The elevations in the model are referenced to the NAVD88. The outputs from the HEC-RAS model are included in Appendix C.

Table 4.1 – Hydraulic Summary along Donna Pump to Brownsville Upper Brownsville Levee (100-Year Flood Discharge)

RS feet
389664

HEC-RAS River Station	Invert Elevation (feet)	Water Surface Elevation (feet)	Flow Velocity (feet/second)	Top Width (feet)	Froude Number
73.8	14.64	49.97	3.5	1,933.27	0.13
72.9	14.14	49.31	1.83	5,789.94	0.08
72.2	10.74	48.72	3.34	3,727.23	0.12
70.7	15.64	47.76	2.74	1,962.03	0.1
70.5	12.34	47.61	3.22	2,319.55	0.12
70.2	15.44	47.45	2.84	4,389.66	0.1
67.4	15.84	44.96	3.11	6,658.7	0.13
67.25	13.14	44.71	3.95	6,939.18	0.14
64.7	11.64	43.15	3.11	5,891.24	0.11
63.8	7.84	42.81	2.4	4,195.46	0.09
62.5	10.24	42.23	3.57	3,009.07	0.13
61.3	3.04	41.65	2.39	4,190.06	0.1
59.9	9.14	40.55	3.58	2,741.37	0.13
59	4.14	40.15	3.38	3,862.51	0.12
58.8	6.64	40.11	2.65	3,542.62	0.1
58.3	5.64	39.83	3.3	405.77	0.13
57.7	2.74	39.59	3.1	1,516.32	0.11
56.9	7.54	39.26	2.84	1,542.04	0.12
56.7	5.84	38.95	3.15	430.51	0.14
56.1	9.64	38.22	3.53	440.92	0.17
55.5	0.94	37.2	4.52	243.6	0.19
55.25	2.94	36.91	4.09	445.75	0.15
55.23	-0.16	36.93	3.49	428.27	0.13
55.215	Railroad bridge				
55.21	-0.16	36.88	3.5	425.29	0.13
55.2	-0.16	36.88	3.5	425.2	0.13
54.5	-1.46	36.61	3.21	720.25	0.11
54.49	4.94	36.51	3.85	486.11	0.14
54.475	Southbound Gateway International Bridge				
54.47	4.94	36.47	3.86	485.33	0.14
54.46	4.94	36.47	3.86	485.32	0.14
54.45	0.64	36.46	3.86	366.27	0.14
54.435	Northbound Gateway International Bridge				
Note: All elevations are based on NAVD88.					

3,907'

211'

The minimum amount that the top of the levee must be raised was determined by USIBWC and provided as part of the design. The necessary increases in top of levee elevation are indicated in Table 4.2. It is noted that river station (RS) 73.8 of the existing levee needs to be raised to satisfy FEMA's freeboard requirement of 3 feet. The total length of the existing levee within Upper Brownsville Levee is approximately 12 miles, per USIBWC limits.

Table 4.2 – Top of Levee Increase along Donna Pump to Brownsville Upper Brownsville Levee

HEC-RAS River Station	Existing Top of Levee Elevation[1] (feet)	Water Surface Elevation[2] (feet)	Existing Freeboard (feet)	Minimum Top of Levee Elevation (feet)	Increased top of Levee[3] (feet)
73.8	51.70	49.97	1.73	52.97	1.27
72.9	52.69	49.31	3.37	52.31	0.00
72.2	51.97	48.72	3.25	51.72	0.00
70.7	51.38	47.76	3.62	50.76	0.00
70.5	51.06	47.61	3.45	50.61	0.00
70.2	51.30	47.45	3.85	50.45	0.00
67.4	49.15	44.96	4.19	47.96	0.00
67.25	-	44.71	-	-	-
64.7	47.82	43.15	4.67	46.15	0.00
63.8	-	42.81	-	-	-
62.5	48.15	42.23	5.92	45.23	0.00
61.3	47.15	41.65	5.50	44.65	0.00
59.9	43.55	40.55	3.00	43.55	0.00
59	43.58	40.15	3.43	43.15	0.00
58.8	44.66	40.11	4.55	43.11	0.00
58.3	43.92	39.83	4.09	42.83	0.00
57.7	43.31	39.59	3.72	42.59	0.00
56.9	42.79	39.26	3.53	42.26	0.00
56.7	43.10	38.95	4.15	41.95	0.00
56.1	42.48	38.22	4.26	41.22	0.00
55.5	-	37.20	-	-	-
55.25	-	36.91	-	-	-
55.23	42.73	36.93	5.80	39.93	0.00
55.215	B & M Bridge				
55.21	-	36.88	-	-	-
55.2	-	36.88	-	-	-
54.5	-	36.61	-	-	-
54.49	-	36.51	-	-	-
54.475	Southbound Gateway International Bridge				
54.47	-	36.47	-	-	-
54.46	-	36.47	-	-	-
54.45	-	36.46	-	-	-
54.435	Northbound Gateway International Bridge				

Note: All elevations are based on NAVD88.
[1] Top of levee elevations were provided by USIBWC.
[2] USIBWC assume 3 feet below minimum top of levee elevation.
[3] Increased top of levee done by USIBWC in 2007, not part of this project.
- = no information available

4.3 Gateway International Bridge

The Gateway International Bridge connects Brownsville, Texas, to Matamoros, Tamaulipas, Mexico. The bridge currently includes a southbound (upstream) span and a northbound span, which cross the Rio Grande at RS 54.475 and RS 54.435, respectively.

USIBWC is responsible for operation and maintenance of the levee along the left bank of the Rio Grande. Since the Rio Grande serves as the border between the United States and Mexico, 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 4.2). Per USIBWC, the length of levee in this location is not to be part of the Upper Brownsville Levee design. The fence obstructs access to the levee embankment, possibly limiting access by the USIBWC for flood fighting. 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 USIBWC is considering the construction of an erosion protection along the riverward slope of the levee embankment. Although improvements to this location are not part of the Upper Brownsville Levee design, Tetra Tech prepared an analysis of the existing hydraulic conditions and the risk of embankment erosion. The analysis was documented in a technical memorandum submitted to the USIBWC on March 30, 2011 (Tetra Tech 2011a).

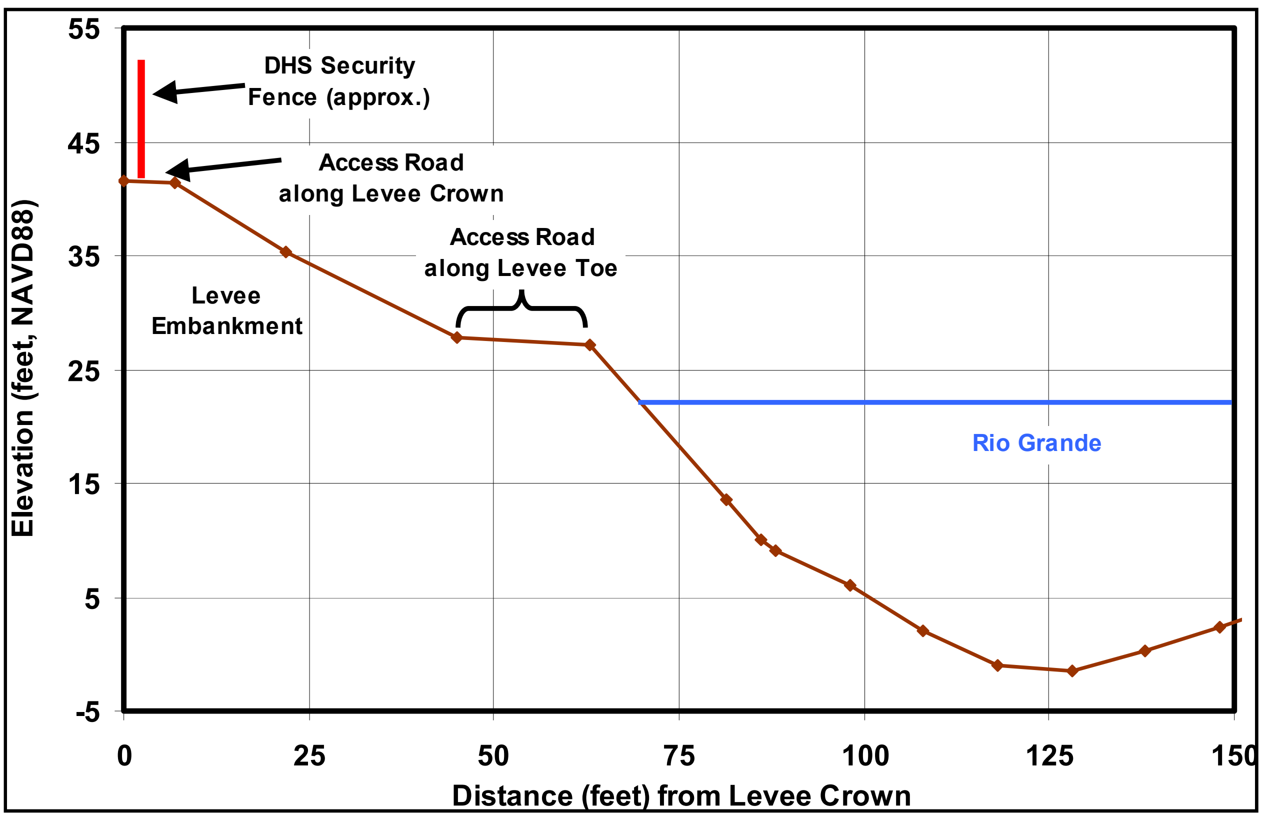


Figure 4.2 – U.S. Levee Embankment, Access Roads, and U.S. Department of Homeland Security Border Security Fence

An analysis was also 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. For several of the alternatives, loose rock revetment was assumed as the erosion protection. Future design phases should consider other options for sloped revetment such as concrete slope paving, Armorflex, and soil cement. The results of this analysis are also presented in the memorandum, which is included as Appendix D.

During the 90% Design Review/ Presentation, USIBWC requested that riprap be placed on the riverside banks from the top of levee to the existing riverside access road. Tetra Tech stated that this would be a temporary solution, as the riprap is not properly toed below the river scour potential. The river may scour and wash out the riverside road and the temporary riprap protection.

5. Compliance with Code of Federal Regulations, Title 44, Section 65.10

The Code of Federal Regulations (44 CFR 65.10) includes a description of the types of information FEMA needs in order to recognize (or "accredit"), on NFIP maps, that a levee system provides protection from the base flood. The following information addresses the design criteria identified in 44 CFR 65.10 and is intended to provide additional information for use by USIBWC in developing the FEMA levee certification package. Tetra Tech's scope of work does not include developing the information/analyses to support the FEMA levee certification package; however, the following information is provided to aid USIBWC in development of that information and analyses.

Specific design criteria are listed, as well as other criteria as specified on the MT-2 Forms. MT-2 Form 3 (Riverine Structures) includes a description of required sediment transport information.

5.1 Freeboard

According to 44 CFR 65.10, the freeboard requirements are as follows:

- A minimum of 3 feet above base flood (100-year) water surface level.
- An additional 1 foot above minimum within 100 feet of structures/bridges/constrictions.
- An additional 0.5 foot at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee.

A comparison of the 100-year water surface elevations from the USIBWC HEC-RAS model of the Rio Grande along Upper Brownsville Levee and the proposed top of levee elevations shown on the drawings of the proposed design (Tetra Tech 2011d) indicates one location where the 3-foot minimum freeboard is not provided. The deficient freeboard is located at the downstream edge of Los Fresnos Pump Canal, where the proposed top of levee elevation is a maximum of 0.88 feet lower than the required height based on the minimum 3 feet of freeboard for a horizontal distance of approximately 115 feet. A strategy for dealing with the deficient freeboard at the canal should be part of the future Los Fresnos Pump Canal closure design.

5.2 Closures

Upper Brownsville Levee includes 58 gatewell structure side drains. Of these gatewells, 33 are operational and 25 have been abandoned. The 33 operational gatewells include working existing sluice gates or will have new sluice gates installed, which meet FEMA's requirements for a closure device. During construction, the inlet and outlet pipes to the 25 abandoned gatewells will be excavated, and brick and mortar sealed to provide permanent closure. The soil within the existing gatewell structure will be excavated and the steel plates to be removed. The outlet pipe, inlet pipe and cavity of the gatewell structures would be filled with flowable fill. USACE requires that all discharge pipes be inspected using a television camera or other visual inspection at least every 5 years. This is not included specifically as part of FEMA's requirements for levee certification.

There is one opening along Upper Brownsville Levee where no closure device is provided. This opening is located at the Los Fresnos Pump Canal. The canal does not meet FEMA's requirement of providing a closure at openings along the levee system. The design of a closure device for the canal is not part of Tetra Tech's current scope of work, as it will be designed and constructed under a separate future contract.

5.3 Embankment Protection

According to 44 CFR 65.10, engineering analyses must be submitted that demonstrate that no appreciable erosion of the levee embankment can be expected during the base flood and that anticipated erosion will not result in failure of the levee embankment.

To demonstrate that no appreciable erosion of the levee embankment can be expected during the base flood, flow velocities were extracted from the HEC-RAS model of the Rio Grande along Upper Brownsville Levee provided by the USIBWC. No appreciable erosion of the levee embankment is expected during the base flood based on velocities identified in the HEC-RAS model along the left/north bank (maximum of 1.3 feet per second [fps]) or the maximum velocity expected through the bends based on the equation on Plate B-33 in Engineer Manual (EM) 1110-2-1601, a maximum of 1.7 fps (USACE 1994). Flood waters become erosive at velocities greater than 5 fps for grass-lined earthen channels per EM 1110-2-1601.

The one exception is at the Gateway International Bridge. At this location, the main channel is very close to the levee and the main channel velocity rather than left bank velocity was considered. The channel velocity at this location approaches 5 fps, which is the limit of permissible velocities for earthen embankments. Embankment protection should be considered along this reach as discussed in Section 4.3.

5.4 Embankment and Foundation Stability

According to 44 CFR 65.10, engineering analyses must be submitted that demonstrate that seepage into or through the levee foundation and embankment will not jeopardize embankment or levee stability. The geotechnical evaluation of this reach performed by Raba-Kistner Consultants, Inc. (R-K) indicated that construction of internal drainage was required to mitigate potential seepage conditions during flood stage (R-K 2011b). It was recommended that a toe drain with a minimum base width of 3 feet extending at least 18 inches below the landside toe of the levee be included in the proposed rehabilitation design. Seepage analysis indicated that implementation of the toe drain would control seepage pressures within the levee and foundation soils and would reduce hydraulic exit gradients to less than 0.5. The toe drain detail has been incorporated into the design drawings and specifications. With this condition, seepage associated with the base flood is not expected to jeopardize the embankment or foundation stability.

Slope stability analyses were performed on pertinent cross sections of the proposed rehabilitated levee. The results of the analyses relative to FEMA criteria are provided in Table 5.1.

Table 5.1 – Minimum Factors of Safety

Condition	Calculated Minimum Factor of Safety	USACE Required Factor of Safety
End of construction	1.7	1.3
Long-term (steady seepage)	2.1	1.4
Rapid drawdown	1.8	1.0 to 1.2

The results of the analyses indicate that the proposed levee design meets the FEMA levee criteria for embankment and foundation stability.

5.5 Settlement

According to 44 CFR 65.10, engineering analyses must be submitted that assess the potential and magnitude of future losses of freeboard as a result of levee settlement and demonstrate that freeboard will be maintained within the minimum freeboard standards. The geotechnical evaluation for this project indicated that placement of 3 feet of additional fill height could induce total settlement (compression and long-term consolidation) of approximately 3-$^{1}/_{4}$ inches. The proposed design would not significantly raise the current levee height but rather it would reconstruct portions of the existing levee with engineered fill. As a conservative measure to maintain the required freeboard, the proposed design height of the levee has been increased by 4 inches to mitigate potential settlement.

5.6 Interior Drainage

According to 44 CFR 65.10, joint probability analysis of the sources of flooding landward of the levee and the river itself must be completed and the extent of the flooded area provided. It is expected that this will be a significant effort because of the number of side drains (approximately 58 over Upper Brownsville Levee) and the ownership of these systems outside of USIBWC right-of-way. USIBWC should note that this analysis will be required as part of a levee certification submittal to FEMA. Tetra Tech's current contract does not include providing any support with regard to the interior drainage analysis.

5.7 Other Design Criteria – Sediment Transport

A sediment transport analysis is required if there is any indication from historical records that scour and deposition can affect the base flood elevation (BFE), or if based on stream morphology, vegetative cover, development of the watershed, and bank conditions, there is a potential for debris and sediment transport to affect the BFE or a structure. If sediment transport will not affect the BFE or any structures, an explanation is required as to why a sediment transport analysis was not performed.

Related to the U.S. (north) levee embankment structure along the Rio Grande between Retamal Dam (south of Donna, Texas) and Brownsville, Texas, the following initial investigations were conducted to identify whether there is potential for sediment transport to affect the BFE or the levee embankment. These investigations focused on the Rio Grande along Upper Brownsville Levee levee but extended upstream as required by the available information.

5.7.1 Analysis of Long-Term Aggradation/Degradation

A trend of long-term aggradation could reduce channel capacity and impact the freeboard of the levee; a trend of degradation could impact channel stability, which could in turn influence the stability of the levee. A review of hydrologic conditions was conducted to characterize the influence of floods on the potential to mobilize, transport, and deposit sediment within the Rio Grande along the Upper Brownsville Levee levee.

USIBWC operates a flow gage on the Rio Grande upstream of the north levee embankment (Gage No. 08-4692.00 below Anzalduas Dam near Reynosa, Tamaulipas, Mexico). Construction of Anzalduas Dam was completed in April 1960 so that floodwaters could be diverted from the Rio Grande into the interior U.S. floodway. Construction of upgrades to Retamal Dam, located approximately 39 miles downstream of Anzalduas Dam, were completed in May 1975 to divert floodwaters to the interior Mexican floodway and to limit floods at Brownsville-Matamoros to the safe capacity of the Rio Grande. According to USIBWC Minute No. 238, the design flood flow below Anzalduas Dam is 20,000 cfs (USIBWC 2003). A review of the average daily discharges recorded between 1960 (when Anzalduas Dam was constructed) and 2010 indicates that floods in excess of the design flow occurred in 9 years. Other than the flooding associated with Hurricane Beulah in September 1967 (maximum average daily discharge of 121,000 cfs), the maximum average daily flow rate was 41,300 cfs in water year 1972 (Figure 5.1). However, considering only the period after 1975 when the upgrades to Retamal Dam were completed, four floods with maximum average daily discharges exceeding 20,000 cfs occurred; the maximum average daily flow of 27,300 cfs occurred in water year 1976. Further investigation of the magnitude of the design flood is warranted to confirm that 20,000 cfs is consistent with the peak flow during the 100-year flood (i.e., the base flood). The current effective FEMA Flood Insurance Study (FIS) indicates that a discharge of 20,000 cfs has a 500-year recurrence level.

DESIGN FLOOD 20,000 cfs

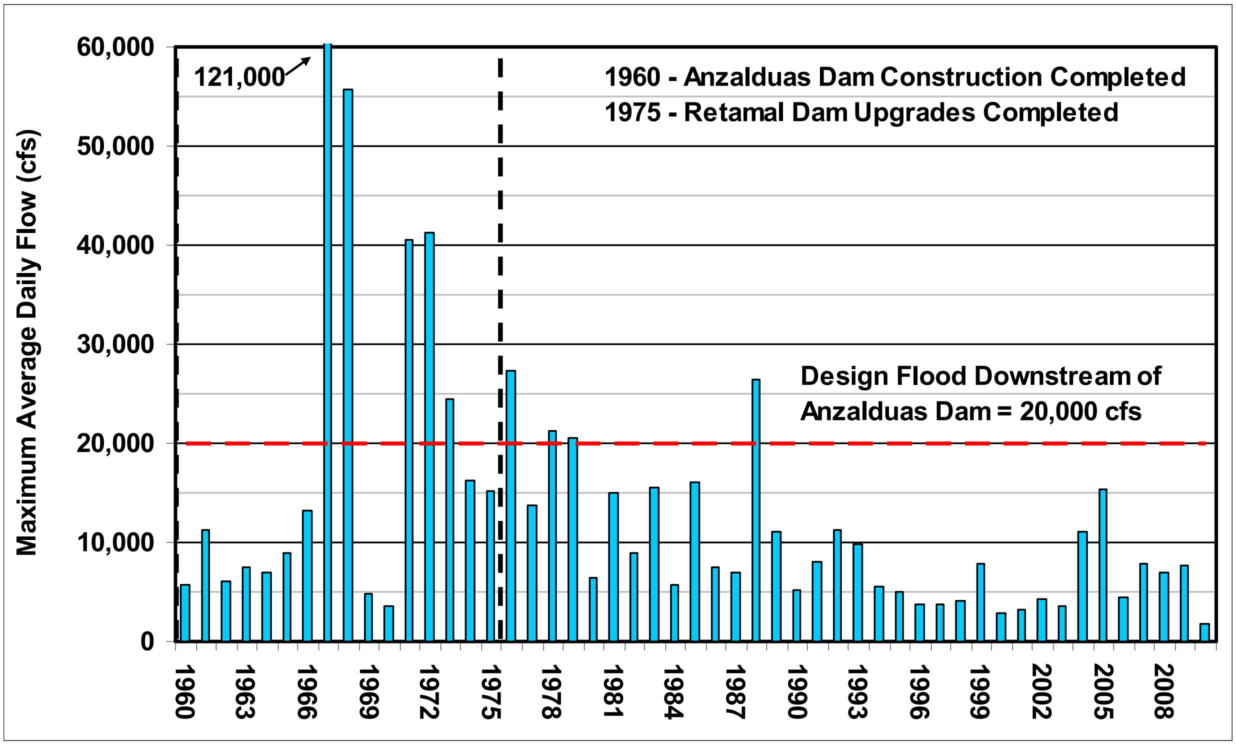


Figure 5.1 – Maximum Average Daily Flow by Water Year Recorded at USIBWC Gage No. 04-4692.00 below Anzalduas Dam

On a long-term basis, most bed material sediment is mobilized, transported, and deposited during flood flows. Since the design flood has been exceeded four times since the upgrades to Retamal Dam were completed in 1975, the system has been tested by major flooding. Assuming the flood regime since 1975 is representative of the future flood regime (due to the regulation of floods by Anzalduas Dam and Retamal Dam), it is expected that future opportunities for floods to transport sediment will be similar. Therefore, any existing trends in long-term aggradation or degradation that are in response to flood hydrology may continue into the future.

Little information was available to evaluate whether long-term aggradation/degradation has the potential to affect the BFE or the Upper Brownsville Levee. Due to the set-back of the levee embankment from the Rio Grande channel, the risk of degradation of the channel and corresponding bank failures appears low; however, this expectation should be confirmed by USIBWC maintenance staff. Confirmation would include a statement that USIBWC does not need to conduct regular dredging of excessive sediment deposits that would indicate the potential for aggradation. The sediment trapping of the impoundments upstream of Anzalduas Dam and Retamal Dam likely limit aggradation potential, but this should be confirmed through USIBWC maintenance records.

5.7.2 Analysis of Bend Scour

In meandering alluvial channels, such as the Rio Grande along Upper Brownsville Levee of the levee, the bed profile and the planform of the river are related. Pools form due to scour in the bends, and riffles or crossings form between bends. The increase in depth through a pool corresponds to an increase in the height of the bank, which increases the potential for the bend scour to lead to bank instabilities. The planform of the Rio Grande channel was reviewed to identify bends that were located within 500 feet of the levee embankment. An equation developed by Thorne (1988) was used to calculate maximum bend scour as a function of bend geometry. An assumed failure plane with a 2H:1V slope was extended up from the calculated maximum scour depth to evaluate whether bank failure could impact the levee embankment.

Four existing bends in the Rio Grande channel were evaluated along the alignment of the Upper Brownsville Levee. Channel cross-sectional geometry was extracted from the HEC-RAS model provided by USIBWC. Depths during the recommended "formative flow" (Thorne 1988) were based on the design flood flow of 20,000 cfs. Under this flow, the calculated maximum scour depths ranged from 0 to 24.8 feet below the existing channel bottom. At Station 1893+00 near the Gateway International Bridge (cross section 54.49 in the HEC-RAS model), bank failure from the calculated maximum scour at the assumed 2H:1V failure plane would intercept the riverward toe of the levee embankment. Embankment protection along this bend may be warranted to protect the levee (Note: Design recommendations for embankment protection were submitted to USIBWC as a separate memorandum [Tetra Tech 2011a].) For the other three bends evaluated along the Upper Brownsville Levee levee, bank failure associated with calculated maximum bend scour would not impact the levee, and embankment protection may not be required. At each of the four bends, the calculated bend scour is not expected to increase the BFE. Scour protection measures constructed at the bend upstream of the Gateway International Bridge should be evaluated in the future to confirm that they do not increase the BFE.

5.7.3 Conclusion

The initial review of available information related to sediment transport in the Rio Grande along the Upper Brownsville Levee indicates limited potential for increases in the BFE or threats to the integrity and performance of the levee embankment due to local scour or general aggradation/degradation processes. A noted exception is the bend upstream of the Gateway International Bridge, where bend scour may threaten the levee embankment, and embankment protection has been recommended. The combined influence of Anzalduas Dam and Retamal Dam have limited the frequency of major floods capable of transporting substantial amounts of bed material sediment between Retamal Dam and Brownsville. Where existing bends are within 500 feet of the levee embankment, calculations of expected maximum bend scour indicate that except as previously noted, expected bank failure will not likely threaten the levee embankment or foundation.

The potential for long-term aggradation/degradation to affect the BFE or the levee embankment is expected to be minimal due to (1) the controlling influence of the diversion dams on the flood regime, (2) the capacity of the dams to trap sediment that could otherwise aggrade in the channel along Upper Brownsville Levee, and (3) the setback between the Rio Grande channel and the levee embankment, allowing for bank failures without threatening the levee embankment or foundation, but further analyses of operation and maintenance records should be made to confirm this expectation.

It should be noted that the analyses to determine the impact of sediment transport on the water surface elevation and freeboard is not part of Tetra Tech's scope of work. However, this initial information has been developed to aid USIBWC in the development of the necessary information to support a FEMA levee certification package. Details of additional investigations that could be undertaken by USIBWC to further develop the information related to sediment transport and levee certification are identified in the following section.

5.7.4 Additional Investigations

The following additional investigations could be performed to confirm that sediment transport is not expected to increase the BFE or threaten riverine structures:

Specific Gage – Using long-term stage and flow measurements collected at the USIBWC gauging station downstream of Retamal Dam (or the gage near Brownsville, Texas, and Matamoros, Tamaulipas, Mexico, Gage No. 08-4750.00) changes in gage height over time for a specified flow rate can be determined. The specific gage analysis is useful in indicating whether there is a trend of aggradation or degradation. Assuming the USIBWC gage is installed in an alluvial reach of the Rio Grande channel, the history of shifts in the rating curve or a specific gage analysis would indicate changes in channel morphology over time. While this gage is located upstream of Upper Brownsville Levee, the analysis would still provide useful information related to Upper Brownsville Levee.

As-Built Plans –The as-built plans for crossings of the Rio Grande (i.e., bridges, diversion structures, dams, and utilities) should show the local elevation of the Rio Grande channel, possibly even the geometry of the channel. The channel geometry on the plans can be compared to the channel geometry represented in the HEC-RAS model to identify an average rate of change over the period of comparison. Any historical surveys that include the channel geometry could be used for these comparisons.

Comparison of Historical Aerial Photographs – The planform of the Rio Grande can be compared at different points in time (e.g., preceding and following flood events) to identify whether sediment transport is sufficient to alter the alignment of the channel. Initially a comparison could be made using recent imagery and imagery closely following the 1975 completion of the upgrades to Retamal Dam. Comparisons to earlier imagery could be made to determine whether there was an ongoing trend before the construction/upgrades to the diversion dams, but comparisons before and after 1975 need to consider the influence of the dam operations on the hydrologic regime. If substantial changes in planform are apparent between 1975 and 2010, intermediate dates could be compared, preferably close in time to major flooding.

Of specific interest in the comparison of historical aerial photographs would be the location of bends in the Rio Grande channel. It is clear in recent aerial imagery that bends form and cut off over time. If bends have historically migrated across the floodplain and encroached upon the levee embankment, IBWC operation and maintenance procedures may show how USIBWC has historically protected the levee embankment.

Review of Operation and Maintenance Records – The records maintained by USIBWC could be reviewed to identify whether natural responses of unbalanced sediment transport have historically resulted in the need for intervention to maintain channel conveyance. Intervention may include the following:

- Construction of grade control structures (e.g., diversion structures, drop structures, weirs, or sills) to prevent down cutting of the bed
- Installation of revetments to arrest bank erosion
- Excavation/dredging of deposited sediment
- Repairs such as unclogging or reinforcing outlets of drainage structures that enter the channel

5.8 Operation Plans

According to 44 CFR 65.10(c), an officially adopted operation manual will need to be provided. The current IBWC manual may need to be updated to meet FEMA requirements. Because the system includes closures (such as sluice gates at side drains), the following information must be included in the manual:

- Documentation of the flood warning system that will be used to trigger emergency operation activities and demonstration that sufficient flood warning time exists to permit activation of the mechanized portions of the drainage system
- Specific actions and assignments of responsibility by individual name or title
- Provisions for periodic operation of the closure at not less than 1-year intervals

FEMA will not accept a general operation plan that is not specific to the levee system, i.e., one that covers an entire county or state.

5.9 Maintenance Plans

According to 44 CFR 65.10(d), an officially adopted maintenance manual will need to be provided. Upon submittal of a levee certification request, FEMA may review all available inspection information to ensure that it is in compliance with the maintenance manual. In order for FEMA to recognize the certification and accredit the levee on the NFIP maps, any issues of noncompliance identified during inspections will require documentation of the correction necessary to meet the standards in the maintenance plan.

5.10 Ongoing FEMA Study Efforts

FEMA is currently working with the local communities to update the flood information for the Rio Grande in the project area. USIBWC should consider coordinating with that effort to ensure that all base information (such as the 100-year discharge) is consistent and that no conflicts arise (such as designation of a floodway in an area where levee improvements are planned).

Summary Table of Articulated Concrete Block Mattress (ACBM) Revetment Models

Model	Velocity (ft/sec)	Critical Bed Shear (psf)*	ACB Height (ft)	Condition	Factor of Safety**
Type A are at River Elevation 37 at Upstream of Bridge					
A	6.4	13.9	0.333		3.7
A0	10	13.9	0.333	Velocity increased until "No Good" condition.	2.4
A00	6.4	39.96	0.5		3.0
A000	10	71.84	0.75	Velocity increased until "No Good" condition.	1.8
Type B are at River Elevation 24 at Upstream of Bridge					
B	6.4	13.9	0.333		3.7
B0	10	13.9	0.333	Velocity increased until "No Good" condition.	2.4
Type AB are at River Elevation 24 at Bridge					
2AB	6.4	13.9	0.333		3.7

* Critical bed shear differs per block type and size.

** Required factor of safety is 2.5.

DANNENBAUM

CLIENT: IBWC
PROJECT: Brownsville Levee Slope Repair
SUBJECT: ACB Bed Shear Design

JOB NO.: 4839-01
DESIGNER: CMA DATE: Dec-15
CHECKER: SG DATE: Dec, 15

Bed Shear Calculation

Section 1 - Upstream of Bridge
Run A - 100 Year Flood Water Elevation +37
6.4 ft/s Velocity **1A**

UNITS	Feet, sec, radians, and lbs	
KEY	Input	Output

Channel Slopes				
Channel Bed Slope				Comments and Definitions
So	0.000108	ft/ft		Riverbed Slope, Direction of Flow
Sf	0.000108	ft/ft		Energy Grade Line, Tetratech HecRas Ouput
Bank Slope				
H	V			
1.25	1	feet		Average Value sith flood stage, See HecRas
Design Shear Stress, (See Tetratech HecRas Ouput Sections)				
Channel Geometry				Comments and Definitions
b	N/A	feet		Width of Channel Base, See attached Channel
yo	37	feet		Depth of flow
Z	N/A	ratio		Side slope ratio
A	6795	ft^2		Flow Area
P	497	feet		Wetted Perimeter
R	13.672	feet		Hydraulic Radius
Gamma	62.4	ft^3		Weight of water
Bed Shear				
τdes	0.092	psf	Design Bed Shear	
τc	13.900	psf	Critical Bed Shear (Per block type and size)	

DANNENBAUM

CLIENT:	IBWC	JOB NO.:	4839-01		
PROJECT:	Brownsville Levee Slope Repair	DESIGNER:	CMA	DATE:	Dec-15
SUBJECT:	ACB Bed Shear Design	CHECKER:	SG	DATE:	Dec 15

Bed Shear Calculation

Section 1 - Upstream of Bridge
Run A - 100 Year Flood Water Elevation +37
6.4 ft/s Velocity **1A**

UNITS Feet, sec, radians, and lbs
KEY Input Output

Safety Factor Design				
			Degrees	Comments and Definitions
H	0.333	feet		Block Height
θ_0	0.000	radians	0.01	Channel Bed Slope
θ_1	0.675	radians	38.66	Channel Side Slope
L_1	0.167	feet		1/2 Block height
L_2	0.788	feet		Pivot Point Moment Arm
L_3	0.266	feet		8/10 Block height
L_4	0.788	feet		Pivot Point Moment Arm
δ	1.558	radians	89.27	Block Projection Angle
θ	0.000	radians	0.01	Block Projection Angle
β	0.013	radians	0.73	Block Projection Angle
Total	1.571	radians	90.00	Total equals 90 degrees
V_{des}	6.400	ft/sec		Design Velocity
ρ	1.940	slug/ft^3		mass Density of Water
a_θ	0.781			projection of Ws into Subgrade
b_u	1.333	feet		Block Width in flow direction
FL'	2.209	lb		Additional Uplift Force
FD'	2.209	lb		Additional Drag Force
Ws	30.100	lb		Submerged Weight of Block
η_0	0.007	ratio		Stability number for horizontatl surface
η_1	0.005	ratio		stabilit number for sloped surface
ΔZ	0.042	feet		height of block protrusion
Target SF				Comments and Definitions
SF_B	1.3			Channel Bank, See Table 1 NCMA TEK
X_C	1.3			Medium, See Table 2 NCMA TEK
X_M	1.5			Say 1.5, HecRas model performed, Table 3 NCMA TEK
		Actual SF		**Status**
SF_T	2.535	SF	3.707	GOOD

DANNENBAUM

CLIENT:	IBWC	JOB NO.:	4839-01		
PROJECT:	Brownsville Levee Slope Repair	DESIGNER:	CMA	DATE:	Dec-15
SUBJECT:	ACB Bed Shear Design	CHECKER:	SG	DATE:	Dec-15

Bed Shear Calculation

Section 1 - Upstream of Bridge
Run Ao - 100 Year Flood Water Elevation +37
10.0 ft/s Velocity **1Ao**

UNITS Feet, sec, radians, and lbs
KEY Input Output

Channel Slopes			
Channel Bed Slope			Comments and Definitions
S_o	0.000108	ft/ft	Riverbed Slope, Direction of Flow
S_f	0.000108	ft/ft	Energy Grade Line, Tetratech HecRas Ouput
Bank Slope			
H	V		
1.25	1	feet	Average Value sith flood stage, See HecRas
Design Shear Stress, (See Tetratech HecRas Ouput Sections)			
Channel Geometry			Comments and Definitions
b	N/A	feet	Width of Channel Base, See attached Channel
y_o	37	feet	Depth of flow
Z	N/A	ratio	Side slope ratio
A	6795	ft^2	Flow Area
P	497	feet	Wetted Perimeter
R	13.672	feet	Hydraulic Radius
Gamma	62.4	ft^3	Weight of water
Bed Shear			
τ_{des}	0.092	psf	Design Bed Shear
τ_c	13.900	psf	Critical Bed Shear (Per block type and size)

DANNENBAUM

CLIENT: IBWC | JOB NO.: 4839-01
PROJECT: Brownsville Levee Slope Repair | DESIGNER: CMA | DATE: Dec-15
SUBJECT: ACB Bed Shear Design | CHECKER: SG | DATE: Dec-15

Bed Shear Calculation

Section 1 - Upstream of Bridge
Run Ao - 100 Year Flood Water Elevation +37
10.0 ft/s Velocity **1Ao**

UNITS Feet, sec, radians, and lbs
KEY Input Output

Safety Factor Design				
			Degrees	Comments and Definitions
H	0.333	feet		Block Height
θ_0	0.000	radians	0.01	Channel Bed Slope
θ_1	0.675	radians	38.66	Channel Side Slope
L_1	0.167	feet		1/2 Block height
L_2	0.788	feet		Pivot Point Moment Arm
L_3	0.266	feet		8/10 Block height
L_4	0.788	feet		Pivot Point Moment Arm
δ	1.558	radians	89.27	Block Projection Angle
θ	0.000	radians	0.01	Block Projection Angle
β	0.013	radians	0.73	Block Projection Angle
Total	1.571	radians	90.00	Total equals 90 degrees
V_{des}	10.000	ft/sec		Design Velocity
ρ	1.940	slug/ft^3		mass Density of Water
a_θ	0.781			projection of Ws into Subgrade
b_u	1.333	feet		Block Width in flow direction
FL'	5.392	lb		Additional Uplift Force
FD'	5.392	lb		Additional Drag Force
Ws	30.100	lb		Submerged Weight of Block
η_0	0.007	ratio		Stability number for horizontatl surface
η_1	0.005	ratio		stabilit number for sloped surface
ΔZ	0.042	feet		height of block protrusion
Target SF				Comments and Definitions
SF_B	1.3			Channel Bank, See Table 1 NCMA TEK
X_C	1.3			Medium, See Table 2 NCMA TEK
X_M	1.5			Say 1.5, HecRas model performed, Table 3 NCMA TEK
		Actual SF		**Status**
SF_T	2.535	SF	2.464	no good

DANNENBAUM

CLIENT: IBWC
PROJECT: Brownsville Levee Slope Repair
SUBJECT: ACB Bed Shear Design

JOB NO.: 4839-01
DESIGNER: CMA DATE: Dec-15
CHECKER: SG DATE: Dec-15

Bed Shear Calculation

Section 1 - Upstream of Bridge
Run A - 100 Year Flood Water Elevation +37
1A00

UNITS	Feet, sec, radians, and lbs	
KEY	Input	Output

Channel Slopes			
Channel Bed Slope			Comments and Definitions
S_o	0.000108	ft/ft	Riverbed Slope, Direction of Flow
S_f	0.000108	ft/ft	Energy Grade Line, Tetratech HecRas Ouput
Bank Slope			
H	V		
1.25	1	feet	Average Value sith flood stage, See HecRas
Design Shear Stress, (See Tetratech HecRas Ouput Sections)			
Channel Geometry			Comments and Definitions
b	N/A	feet	Width of Channel Base, See attached Channel
y_o	37	feet	Depth of flow
Z	N/A	ratio	Side slope ratio
A	6795	ft^2	Flow Area
P	497	feet	Wetted Perimeter
R	13.672	feet	Hydraulic Radius
Gamma	62.4	ft^3	Weight of water
Bed Shear			
τ_{des}	0.092	psf	Design Bed Shear
τ_c	39.959	psf	Critical Bed Shear (Per block type and size)

DANNENBAUM

CLIENT:	IBWC	JOB NO.:	4839-01		
PROJECT:	Brownsville Levee Slope Repair	DESIGNER:	CMA	DATE:	Dec-15
SUBJECT:	ACB Bed Shear Design	CHECKER:	SG	DATE:	Dec-15

Bed Shear Calculation

Section 1 - Upstream of Bridge

Run A - 100 Year Flood Water Elevation +37

UNITS Feet, sec, radians, and lbs **1A00**

KEY Input Output

Safety Factor Design				
			Degrees	Comments and Definitions
H	0.5	feet		Block Height
θ_0	0.000	radians	0.0062	Channel Bed Slope
θ_1	0.675	radians	38.66	Channel Side Slope
L_1	0.250	feet		1/2 Block height
L_2	0.750	feet		Pivot Point Moment Arm
L_3	0.400	feet		8/10 Block height
L_4	0.750	feet		Pivot Point Moment Arm
δ	1.567	radians	89.772	Block Projection Angle
θ	0.000	radians	0.008	Block Projection Angle
β	0.004	radians	0.221	Block Projection Angle
Total	1.571	radians	90.00	Total equals 90 degrees
V_{des}	6.400	ft/sec		Design Velocity
ρ	1.940	slug/ft^3		mass Density of Water
a_θ	0.781			projection of Ws into Subgrade
b_u	1.333	feet		Block Width in flow direction
FL'	2.209	lb		Additional Uplift Force
FD'	2.209	lb		Additional Drag Force
Ws	46.619	lb		Submerged Weight of Block
η_0	0.0023	ratio		Stability number for horizontatl surface
η_1	0.0015	ratio		stabilit number for sloped surface
ΔZ	0.042	feet		height of block protrusion

Target SF				Comments and Definitions
SF_B	1.3			Channel Bank, See Table 1 NCMA TEK
X_C	1.3			Medium, See Table 2 NCMA TEK
X_M	1.5			Say 1.5, HecRas model performed, Table 3 NCMA TEK
		Actual SF		**Status**
SF_T	2.535	SF	3.04	GOOD

DANNENBAUM

CLIENT: IBWC
PROJECT: Brownsville Levee Slope Repair
SUBJECT: ACB Bed Shear Design

JOB NO.: 4839-01
DESIGNER: CMA DATE: Dec-15
CHECKER: SG DATE: Dec-15

Bed Shear Calculation

Section 1 - Upstream of Bridge
Run A - 100 Year Flood Water Elevation +37
1A000

UNITS Feet, sec, radians, and lbs
KEY Input Output

Channel Slopes				
Channel Bed Slope				Comments and Definitions
S_o	0.000108	ft/ft		Riverbed Slope, Direction of Flow
S_f	0.000108	ft/ft		Energy Grade Line, Tetratech HecRas Ouput
Bank Slope				
H	V			
1.25	1	feet		Average Value sith flood stage, See HecRas
Design Shear Stress, (See Tetratech HecRas Ouput Sections)				
Channel Geometry				Comments and Definitions
b	N/A	feet		Width of Channel Base, See attached Channel
y_o	37	feet		Depth of flow
Z	N/A	ratio		Side slope ratio
A	6795	ft^2		Flow Area
P	497	feet		Wetted Perimeter
R	13.672	feet		Hydraulic Radius
Gamma	62.4	ft^3		Weight of water
Bed Shear				
τ_{des}	0.092	psf	Design Bed Shear	
τ_C	71.837	psf	Critical Bed Shear (Per block type and size)	

DANNENBAUM

CLIENT:	IBWC	JOB NO.:	4839-01		
PROJECT:	Brownsville Levee Slope Repair	DESIGNER:	CMA	DATE:	Dec-15
SUBJECT:	ACB Bed Shear Design	CHECKER:	SG	DATE:	Dec-15

Bed Shear Calculation

Section 1 - Upstream of Bridge

Run A - 100 Year Flood Water Elevation +37

UNITS Feet, sec, radians, and lbs **1A000**

KEY Input Output

Safety Factor Design				
			Degrees	Comments and Definitions
H	0.75	feet		Block Height
θ_0	0.000	radians	0.0062	Channel Bed Slope
θ_1	0.675	radians	38.66	Channel Side Slope
L_1	0.375	feet		1/2 Block height
L_2	0.750	feet		Pivot Point Moment Arm
L_3	0.600	feet		8/10 Block height
L_4	0.750	feet		Pivot Point Moment Arm
δ	1.569	radians	89.888	Block Projection Angle
θ	0.000	radians	0.008	Block Projection Angle
β	0.002	radians	0.105	Block Projection Angle
Total	1.571	radians	90.00	Total equals 90 degrees
V_{des}	10.000	ft/sec		Design Velocity
ρ	1.940	slug/ft^3		mass Density of Water
a_θ	0.781			projection of Ws into Subgrade
b_u	1.333	feet		Block Width in flow direction
FL'	5.392	lb		Additional Uplift Force
FD'	5.392	lb		Additional Drag Force
Ws	46.619	lb		Submerged Weight of Block
η_0	0.0013	ratio		Stability number for horizontatl surface
η_1	0.0007	ratio		stabilit number for sloped surface
ΔZ	0.042	feet		height of block protrusion

Target SF				Comments and Definitions
SF_B	1.3			Channel Bank, See Table 1 NCMA TEK
X_C	1.3			Medium, See Table 2 NCMA TEK
X_M	1.5			Say 1.5, HecRas model performed, Table 3 NCMA TEK
		Actual SF		**Status**
SF_T	2.535	SF	1.82	no good

DANNENBAUM

CLIENT:	IBWC	JOB NO.:	4839-01		
PROJECT:	Brownsville Levee Slope Repair	DESIGNER:	CMA	DATE:	Dec-15
SUBJECT:	ACB Bed Shear Design	CHECKER:	SG	DATE:	Dec 15

Bed Shear Calculation

Section 1 - Upstream of Bridge
Run B - Typical Highwater Water Elevation +24
6.4 ft/s Velocity **1B**

UNITS Feet, sec, radians, and lbs
KEY Input Output

Channel Slopes			
Channel Bed Slope			Comments and Definitions
S_o	0.000108	ft/ft	Riverbed Slope, Direction of Flow
S_f	0.000108	ft/ft	Energy Grade Line, Tetratech HecRas Ouput
Bank Slope			
H	V		
1.25	1	feet	Average Value sith flood stage, See HecRas
Design Shear Stress, (See Tetratech HecRas Ouput Sections)			
Channel Geometry			Comments and Definitions
b	N/A	feet	Width of Channel Base, See attached Channel
y_o	14	feet	Depth of flow
Z	N/A	ratio	Side slope ratio
A	2044	ft^2	Flow Area
P	179	feet	Wetted Perimeter
R	11.419	feet	Hydraulic Radius
Gamma	62.4	ft^3	Weight of water
Bed Shear			
T_{des}	0.077	psf	Design Bed Shear
T_C	13.900	psf	Critical Bed Shear (Per block type and size)

DANNENBAUM

CLIENT:	IBWC	JOB NO.:	4839-01		
PROJECT:	Brownsville Levee Slope Repair	DESIGNER:	CMA	DATE:	Dec-15
SUBJECT:	ACB Bed Shear Design	CHECKER:	SG	DATE:	Dec-15

Bed Shear Calculation

Section 1 - Upstream of Bridge

Run B - Typical Highwater Water Elevation +24

6.4 ft/s Velocity **1B**

UNITS Feet, sec, radians, and lbs

KEY Input Output

Safety Factor Design				
			Degrees	Comments and Definitions
H	0.333	feet		Block Height
θ_0	0.000	radians	0.01	Channel Bed Slope
θ_1	0.675	radians	38.66	Channel Side Slope
L_1	0.167	feet		1/2 Block height
L_2	0.788	feet		Pivot Point Moment Arm
L_3	0.266	feet		8/10 Block height
L_4	0.788	feet		Pivot Point Moment Arm
δ	1.560	radians	89.39	Block Projection Angle
θ	0.000	radians	0.01	Block Projection Angle
β	0.011	radians	0.61	Block Projection Angle
Total	1.571	radians	90.00	Total equals 90 degrees
V_{des}	6.400	ft/sec		Design Velocity
ρ	1.940	slug/ft^3		mass Density of Water
a_θ	0.781			projection of Ws into Subgrade
b_u	1.333	feet		Block Width in flow direction
FL'	2.209	lb		Additional Uplift Force
FD'	2.209	lb		Additional Drag Force
Ws	30.100	lb		Submerged Weight of Block
η_0	0.006	ratio		Stability number for horizontatl surface
η_1	0.004	ratio		stabilit number for sloped surface
ΔZ	0.042	feet		height of block protrusion

Target SF				
				Comments and Definitions
SF_B	1.3			Channel Bank, See Table 1 NCMA TEK
X_C	1.3			Medium, See Table 2 NCMA TEK
X_M	1.5			Say 1.5, HecRas model performed, Table 3 NCMA TEK
		Actual SF		Status
SF_T	2.535	SF	3.722	GOOD

DANNENBAUM

CLIENT:	IBWC	JOB NO.:	4839-01		
PROJECT:	Brownsville Levee Slope Repair	DESIGNER:	CMA	DATE:	Dec-15
SUBJECT:	ACB Bed Shear Design	CHECKER:	SG	DATE:	Dec-15

Bed Shear Calculation

Section 1 - Upstream of Bridge
Run Bo - Typical Highwater Water Elevation +24
10.0 ft/s Velocity **1Bo**

UNITS Feet, sec, radians, and lbs
KEY Input Output

Channel Slopes				
Channel Bed Slope				Comments and Definitions
S_o	0.000108	ft/ft		Riverbed Slope, Direction of Flow
S_f	0.000108	ft/ft		Energy Grade Line, Tetratech HecRas Ouput
Bank Slope				
H	V			
1.25	1	feet		Average Value sith flood stage, See HecRas
Design Shear Stress, (See Tetratech HecRas Ouput Sections)				
Channel Geometry				Comments and Definitions
b	N/A	feet		Width of Channel Base, See attached Channel
y_o	14	feet		Depth of flow
Z	N/A	ratio		Side slope ratio
A	2044	ft^2		Flow Area
P	179	feet		Wetted Perimeter
R	11.419	feet		Hydraulic Radius
Gamma	62.4	ft^3		Weight of water
Bed Shear				
τ_{des}	0.077	psf	Design Bed Shear	
τ_C	13.900	psf	Critical Bed Shear (Per block type and size)	

DANNENBAUM

CLIENT:	IBWC	JOB NO.:	4839-01		
PROJECT:	Brownsville Levee Slope Repair	DESIGNER:	CMA	DATE:	Dec-15
SUBJECT:	ACB Bed Shear Design	CHECKER:	SG	DATE:	Dec-15

Bed Shear Calculation

Section 1 - Upstream of Bridge
Run Bo - Typical Highwater Water Elevation +24
10.0 ft/s Velocity **1Bo**

UNITS Feet, sec, radians, and lbs
KEY Input Output

Safety Factor Design

			Degrees	Comments and Definitions
H	0.333	feet		Block Height
θ_0	0.000	radians	0.01	Channel Bed Slope
θ_1	0.675	radians	38.66	Channel Side Slope
L_1	0.167	feet		1/2 Block height
L_2	0.788	feet		Pivot Point Moment Arm
L_3	0.266	feet		8/10 Block height
L_4	0.788	feet		Pivot Point Moment Arm
δ	1.560	radians	89.39	Block Projection Angle
θ	0.000	radians	0.01	Block Projection Angle
β	0.011	radians	0.61	Block Projection Angle
Total	1.571	radians	90.00	Total equals 90 degrees
V_{des}	10.000	ft/sec		Design Velocity
ρ	1.940	slug/ft^3		mass Density of Water
a_θ	0.781			projection of Ws into Subgrade
b_u	1.333	feet		Block Width in flow direction
FL'	5.392	lb		Additional Uplift Force
FD'	5.392	lb		Additional Drag Force
Ws	30.100	lb		Submerged Weight of Block
η_0	0.006	ratio		Stability number for horizontatl surface
η_1	0.004	ratio		stabilit number for sloped surface
ΔZ	0.042	feet		height of block protrusion

Target SF				Comments and Definitions
SF_B	1.3			Channel Bank, See Table 1 NCMA TEK
X_C	1.3			Medium, See Table 2 NCMA TEK
X_M	1.5			Say 1.5, HecRas model performed, Table 3 NCMA TEK
		Actual SF		**Status**
SF_T	2.535	SF	2.472	no good

DANNENBAUM

CLIENT:	IBWC	JOB NO.:	4839-01		
PROJECT:	Brownsville Levee Slope Repair	DESIGNER:	CMA	DATE:	Dec-15
SUBJECT:	ACB Design	CHECKER:	SG	DATE:	Dec-15

Bed Shear Calculation

Section 2 - At Bridge
Run B - Typical Highwater Water Elevation +24
6.4 ft/s Velocity **2AB**

UNITS Feet, sec, radians, and lbs
KEY Input Output

Channel Slopes			
Channel Bed Slope			Comments and Definitions
S_o	0.000108	ft/ft	Riverbed Slope, Direction of Flow
S_f	0.000108	ft/ft	Energy Grade Line, Tetratech HecRas Ouput
Bank Slope			
H	V		
1.25	1	feet	Average Value sith flood stage, See HecRas
Design Shear Stress, (See Tetratech HecRas Ouput Sections)			
Channel Geometry			Comments and Definitions
b	N/A	feet	Width of Channel Base, See attached Channel
y_o	14	feet	Depth of flow
Z	N/A	ratio	Side slope ratio
A	2044	ft^2	Flow Area
P	179	feet	Wetted Perimeter
R	11.419	feet	Hydraulic Radius
Gamma	62.4	ft^3	Weight of water
Bed Shear			
τ_{des}	0.077	psf	Design Bed Shear
τ_C	13.900	psf	Critical Bed Shear (Per block type and size)

DANNENBAUM

CLIENT: IBWC	JOB NO.: 4839-01		
PROJECT: Brownsville Levee Slope Repair	DESIGNER: CMA	DATE: Dec-15	
SUBJECT: ACB Design	CHECKER: SG	DATE: Dec-15	

Bed Shear Calculation

Section 2 - At Bridge

Run B - Typical Highwater Water Elevation +24

UNITS Feet, sec, radians, and lbs — **6.4 ft/s Velocity** **2AB**

KEY Input Output

Safety Factor Design				
			Degrees	Comments and Definitions
H	0.333	feet		Block Height
θ_0	0.000	radians	0.01	Channel Bed Slope
θ_1	0.675	radians	38.66	Channel Side Slope
L_1	0.167	feet		1/2 Block height
L_2	0.788	feet		Pivot Point Moment Arm
L_3	0.266	feet		8/10 Block height
L_4	0.788	feet		Pivot Point Moment Arm
δ	1.560	radians	89.39	Block Projection Angle
θ	0.000	radians	0.01	Block Projection Angle
β	0.011	radians	0.61	Block Projection Angle
Total	1.571	radians	90.00	Total equals 90 degrees
V_{des}	6.400	ft/sec		Design Velocity
ρ	1.940	slug/ft^3		mass Density of Water
a_θ	0.781			projection of Ws into Subgrade
b_u	1.333	feet		Block Width in flow direction
FL'	2.209	lb		Additional Uplift Force
FD'	2.209	lb		Additional Drag Force
Ws	30.100	lb		Submerged Weight of Block
η_0	0.006	ratio		Stability number for horizontatl surface
η_1	0.004	ratio		stabilit number for sloped surface
ΔZ	0.042	feet		height of block protrusion
Target SF				Comments and Definitions
SF_B	1.3			Channel Bank, See Table 1 NCMA TEK
X_C	1.3			Medium, See Table 2 NCMA TEK
X_M	1.5			Say 1.5, HecRas model performed, Table 3 NCMA TEK
		Actual SF		**Status**
SF_T	2.535	SF	3.722	GOOD

ATTACHMENT C

DESIGN CALCULATIONS

C.1 GEOTEXTILE

C.2 SOIL FILTER

C.3 ARTICULATED CONCRETE BLOCK MATRESS (ACBM)

C.4 ROCK RIPRAP

C.5 GATEWELL STRUCTURE 205

SUBJECT: Rip Rap Design
PREPARED BY CMA DATE 12-7-2015 CHECKED BY APPROVED BY SG 12/13/17 JOB NUMBER
SHEET NO. OF

STONE SIZE CONT.

MIN => #18 Rip Rap (RUN A) SF=2.0
MAX => #54 Rip Rap (RUN A_0, B_0, B_{00}) SF=2.0
MAX' => #33 Rip Rap (RUN B_0) SF=1.1

RESULTS INTERPRETATION

- USE 18" Rip Rap AT UPSTREAM + DOWNSTREAM TOE INS SF=2.0

$\frac{54+33}{2} = 43.5"$ SF ≈ 1.55

- USE 42" STONE AT BRIDGE PIERS
- TAPER TO 18" UPSTREAM TOE IN, LEEWARD SIDE OF PIER, + @ LAUNCHABLE TOE
- USE 18" BEDDING STONE @ PIER GOUGE

DESIGN NOTES AND COMPUTATIONS

SUBJECT: Rip Rap Design
PREPARED BY: CMA DATE: 12-7-2015 CHECKED BY: SG 12/13/17 APPROVED BY JOB NUMBER
SHEET NO. OF

Stone Size

[Reference EM 1110-2-1601 Attached]

$$D_{30} = S_f C_s C_v C_T d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{1/2} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5}$$

D_{30} = 30 finer by wt.

✓ S_f = SAFETY FACTOR Minimum 1.1
TRY 2.0 SEE c, 3-6 EM 1110-2-1601

✓ C_s = STABILITY FACTOR, D_{85}/D_{15} = 1.7 to 5.2
= 0.30 Assume Angular Rock
per Tetra tech Design

✓ C_v = 1.283 − 0.2 log (R/W) outside bends (1 for R/W) ≥ 26

R = 550 feet TETRA TECH MEMO, page 4
W = 165.7 feet " " page 3

✓ C_v = 1.179 TABLE 1, SECTION 54,435

✓ C_T = 1.0 (1 D_{100}) or 1.5 D_{50}

d = Depth FLOW $g = 32.2\ ft/s^2$
RUN A +37 − 10 = 27'
RUN B +24 − 10 = 14'

γ_w = 62.4 pcf γ_s = 150 pcf (Conserv.)
V = 6.4 ft/s RUN A + B
10.0 ft/s RUN Ao + Bo

K_1 = SIDE SLOPE CORRECTION FACTOR

✓ $K_1 = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}$ $\theta_2 = ATAN\left(\frac{1}{1.25}\right) = 38.66°$
$\phi = 40°$ Rip Rap Repose

K_{11} = 0.505
K_{12} = 0.235 $\theta_1 = ATAN\left(\frac{1}{1.5}\right) = 33.69°$

DANNENBAUM
DANNENBAUM ENGINEERING CORPORATION

DESIGN NOTES AND COMPUTATIONS

SUBJECT: RIP RAP DESIGN

PREPARED BY: CMA DATE: 12-7-2015 CHECKED BY: APPROVED BY: SG 12/13/17 JOB NUMBER

SHEET NO. OF

STONE SIZE [SEE TABLE 3-1 EM 1110-2-1601]

RUN A $d = 27'$ $V = 6.4$ ft/s $\theta = 33.69°$

$$D_{30} = 2.0(0.30)(1.179)(C_T = 1.0)(27)\left[\left(\frac{62.4}{150-62.4}\right)^{1/2}\frac{6.4}{\sqrt{0.505(32.2)(27')}}\right]^{2.5}$$

$D_{30} = 0.64 \Rightarrow$ #18 Rip Rap

0.707 RUN A_o $d = 27'$ $V = 10$ ft/s $\theta_1 = 33.69$

.844 $D_{30} = 1.97 \Rightarrow$ #54 Rip Rap

w/ SF = 1.1 $\Rightarrow$ 1.08 $\Rightarrow$ #27 Rip Rap

RUN A_{oo} $d = 27'$ $V = 6.4$ ft/s $\theta_2 = 38.66°$

$D_{30} = 1.677 \Rightarrow$ #42 Rip Rap

w/ SF = 1.1 $\Rightarrow$ 0.922 $\Rightarrow$ #24 Rip Rap

RUN B $d = 14'$ $V = 6.4$ ft/s $\theta_1 = 33.69°$

$$D_{30} = 2.0(0.30)(1.179)(1.0)(14')\left[\left(\frac{62.4}{150-62.4}\right)^{1/2}\frac{6.4}{\sqrt{0.505(32.2)(14')}}\right]^{2.5}$$

$D_{30} = 0.76 \Rightarrow$ #21 Rip Rap

w/ SF = 1.1 $\Rightarrow$ 0.418 $\Rightarrow$ #12 (Don't use)

RUN B_o $\boxed{SF = 1.1}$ $d = 14'$ $V = 10$ ft/s $\theta_1 = 33.69°$

$D_{30} = 2.318$ No good $>$ #54

w/ SF = 1.1 $\Rightarrow$ 1.27 $\Rightarrow$ #33

RUN B_{oo} $d = 14'$ $V = 6.4$ ft/s $\theta_2 = 38.66°$

$D_{30} = 1.98 \Rightarrow$ #54 Rip Rap

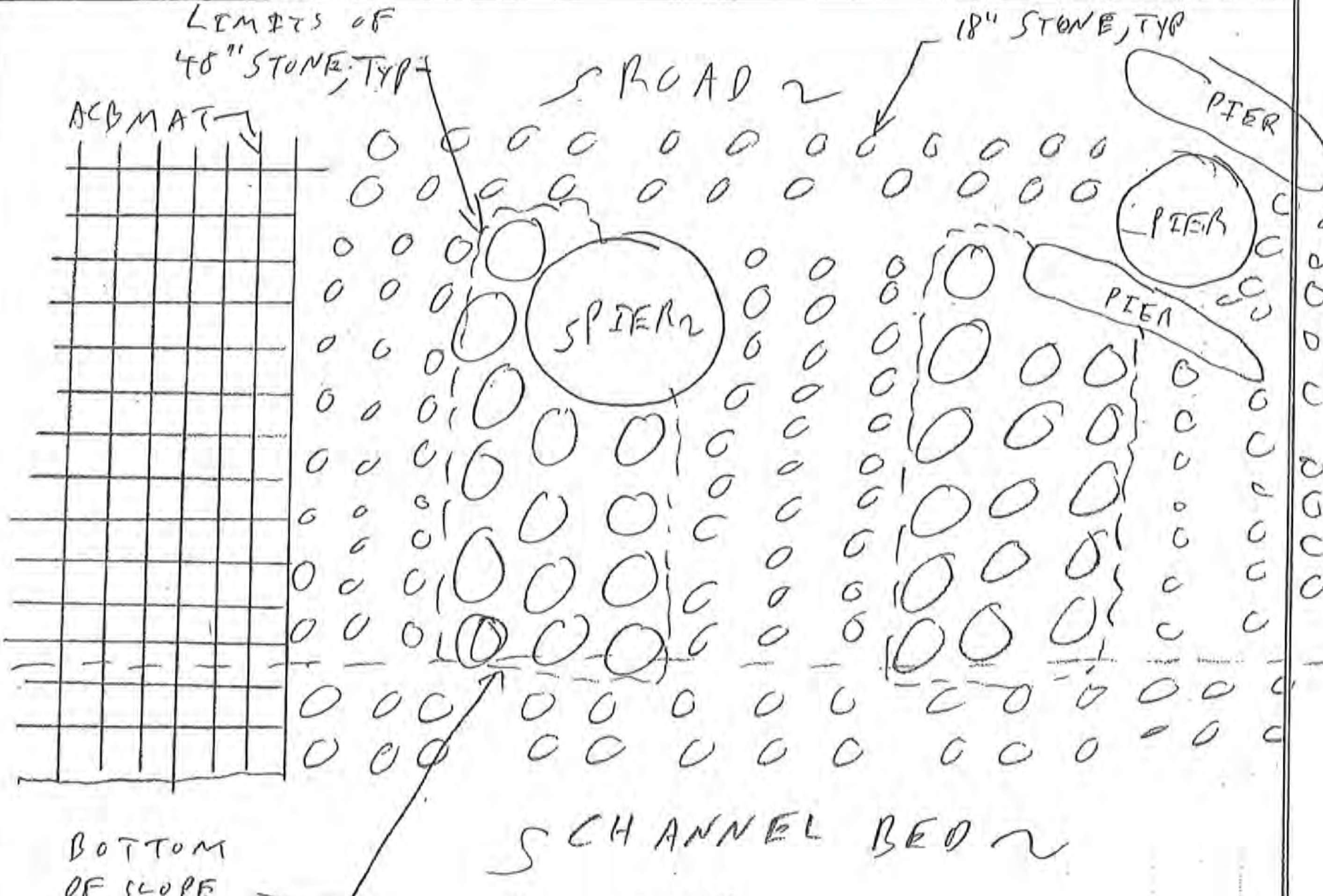
w/ SF = 1.1 $\Rightarrow$ 1.089 $\Rightarrow$ #27 Rip Rap

DESIGN NOTES AND COMPUTATIONS

SUBJECT: RIP RAP DESIGN.

PREPARED BY: CMA | DATE: 12-7-2015 | CHECKED BY: | APPROVED BY: SG 12/13/17 | JOB NUMBER:

SHEET NO. OF



PLAN

Chapter 3
Riprap Protection

Section I
Introduction

3-1. General

* The guidance presented herein applies to riprap design for open channels not immediately downstream of stilling basins or other highly turbulent areas (for stilling basin riprap, use HDC 712-1, Plates 29 and 30). The ability of riprap slope protection to resist the erosive forces of channel flow depends on the interrelation of the following factors: stone shape, size, weight, and durability; riprap gradation and layer thickness; and channel alignment, cross-section, gradient, and velocity distribution. The bed material and local scour characteristics determine the design of toe protection which is essential for riprap revetment stability. The bank material and groundwater conditions affect the need for filters between the riprap and underlying material. Construction quality control of both stone production and riprap placement is essential for successful bank protection. Riprap protection for flood control channels and appurtenant structures should be designed so that any flood that could reasonably be expected to occur during the service life of the channel or structure would not cause damage exceeding nominal maintenance or replacement (see ER 1110-2-1150). While the procedures presented herein yield definite stone sizes, results should be used for guidance purposes and revised as deemed necessary to provide a practical protection design for the specific project conditions.

3-2. Riprap Characteristics

The following provides guidance on stone shape, size/weight relationship, unit weight, gradation, and layer thickness. Reference EM 1110-2-2302 for additional guidance on riprap material characteristics and construction.

a. Stone shape. Riprap should be blocky in shape rather than elongated, as more nearly cubical stones "nest" together best and are more resistant to movement. The stone should have sharp, angular, clean edges at the intersections of relatively flat faces. Stream rounded stone is less resistant to movement, although the drag force on a rounded stone is less than on angular, cubical stones. As rounded stone interlock is less than that of equal-sized angular stones, the rounded stone mass is more likely to be eroded by channel flow. If used, the rounded stone should be placed on flatter side slopes than angular stone and should be about 25 percent larger in diameter. The following shape limitations should be specified for riprap obtained from quarry operations:

(1) The stone shall be predominantly angular in shape.

(2) Not more than 30 percent of the stones distributed throughout the gradation should have a ratio of a/c greater than 2.5. *

(3) Not more than 15 percent of the stones distributed throughout the gradation should have a ratio of a/c greater than 3.0.

(4) No stone should have a ratio of a/c greater than 3.5.

To determine stone dimensions a and c, consider that the stone has a long axis, an intermediate axis, and a short axis, each being perpendicular to the other. Dimension a is the maximum length of the stone, which defines the long axis of the stone. The intermediate axis is defined by the maximum width of the stone. The remaining axis is the short axis. Dimension c is the maximum dimension parallel to the short axis. These limitations apply only to the stone within the required riprap gradation and not to quarry spalls and waste that may be allowed.

b. Relation between stone size and weight. The ability of riprap revetment to resist erosion is related to the size and weight of stones. Design guidance is often expressed in terms of the stone size $D_{\%}$, where % denotes the percentage of the total weight of the graded material (total weight including quarry wastes and spalls) that contains stones of less weight. The relation between size and weight of stone is described herein using a spherical shape by the equation

$$D_{\%} = \left(\frac{6W_{\%}}{\pi\gamma_s}\right)^{1/3} \tag{3-1}$$

where

$D_{\%}$ = equivalent-volume spherical stone diameter, ft

$W_{\%}$ = weight of individual stone having diameter of $D_{\%}$

γ_s = saturated surface dry specific or unit weight of stone, pcf

Plate 31 presents relations between spherical diameter and weight for several values of specific or unit weight. Design procedures for determining the stone size required to resist the erosive forces of channel flow are presented in paragraph 3-5 below.

c. Unit weight. Unit weight of stone γ_s generally varies from 150 to 175 pcf. Riprap sizing relations are relatively sensitive to unit weight of stone, and γ_s should be determined as accurately as possible. In many cases, the unit weight of stone is not known because the quarry is selected from a list of approved riprap sources after the construction contract is awarded. Riprap coming from the various quarries will not be of the same unit weight. Under these circumstances, a unit weight of stone close to the minimum of the available riprap sources can be used in design. Contract options covering specific weight ranges of 5 or 10 pcf should be offered when sufficient savings warrant.

d. Gradation.

(1) The gradation of stones in riprap revetment affects the riprap's resistance to erosion. Stone should be reasonably well graded throughout the in-place layer thickness. Specifications should provide for two limiting gradation curves, and any stone gradation as determined from quarry process, stockpile, <u>and</u> in-place field test samples that lies within these limits should be acceptable. Riprap sizes and weights are frequently used such as D_{30}(min), D_{100}(max), W_{50}(min), etc. The D or W refers to size or weight, respectively. The number is the percent finer by weight as discussed in b above. The (max) or (min) refers to the upper or lower limit gradation curves, respectively. Engineer Form 4794-R is a standard form for plotting riprap gradation curves (Plate 32). The gradation limits should not be so restrictive that production costs would be excessive. The choice of limits also depends on the underlying bank soils and filter requirements if a graded stone filter is used. Filters may be required under riprap revetments. Guidance for filter requirements is given in EM 1110-2-1901. Filter design is the responsibility of the Geotechnical Branch in each District.

(2) Standardized gradations having a relatively narrow range in sizes (D_{85}/D_{15} of 1.4-2.2) are shown in Table 3-1. Other gradations can be used and often have a wider range of allowable sizes than those given in Table 3-1. One example is the Lower Mississippi Valley Division (LMVD) Standardized Gradations presented in Appendix F. The LMVD gradations are similar to the gradations listed in Table 3-1 except the LMVD W_{50}(max) and W_{15}(max) weights are larger, which can make the LMVD gradations easier to produce. Most graded ripraps have ratios of D_{85}/D_{15} less than 3. Uniform riprap ($D_{85}/D_{15} < 1.4$) has been used at sites in the US Army Engineer Division, Missouri River, for reasons of economy and quality control of sizes and placement.

(3) Rather than a relatively expensive graded riprap, a greater thickness of a quarry-run stone may be considered. Some designers consider the quarry-run stone to have another advantage: its gravel- and sand-size components serve as a filter. The gravel and sand sizes should be less by volume than the voids among the larger stone. This concept has resulted in considerable cost savings on large projects such as the Arkansas and Red River Navigation Projects. Not all quarry-run stone can be used as riprap; stone that is gap graded or has a large range in maximum to minimum size is probably unsuitable. Quarry-run stone for riprap should be limited to $D_{85}/D_{15} \leq 7$.

(4) Determining optimum gradations is also an economics problem that includes the following factors:

(a) Rock quality (durability under service conditions)

(b) Cost per ton at the quarry (including capability of quarry to produce a particular size)

(c) Number of tons required

(d) Miles transported

(e) Cost of transportation per ton-mile

(f) Cost per ton for placement

(g) Need for and cost of filter

(h) Quality control during construction (it is easier to ensure even coverage with a narrow gradation than with a wide gradation)

(i) Number of different gradations required. Sometimes cost savings can be realized by using fewer gradations.

See EM 1110-2-2302 for further discussion of these factors.

Table 3-1
Gradations for Riprap Placement in the Dry, Low-Turbulence Zones

Limits of Stone Weight, lb[1], for Percent Lighter by Weight

D_{100}(max) in.	100 Max	100 Min	50 Max[2]	50 Min	15 Max[2]	15 Min	D_{30}(min) ft	D_{90}(min) ft
Specific Weight = 155 pcf								
* 9	34	14	10	7	5	2	0.37	0.53 *
12	81	32	24	16	12	5	0.48	0.70
15	159	63	47	32	23	10	0.61	0.88
18	274	110	81	55	41	17	0.73	1.06
21	435	174	129	87	64	27	0.85	1.23
24	649	260	192	130	96	41	0.97	1.40
27	924	370	274	185	137	58	1.10	1.59
30	1,268	507	376	254	188	79	1.22	1.77
33	1,688	675	500	338	250	105	1.34	1.94
36	2,191	877	649	438	325	137	1.46	2.11
42	3,480	1,392	1,031	696	516	217	1.70	2.47
48	5,194	2,078	1,539	1,039	769	325	1.95	2.82
54	7,396	2,958	2,191	1,479	1,096	462	2.19	3.17
Specific Weight = 165 pcf								
* 9	36	15	11	7	5	2	0.37	0.53 *
12	86	35	26	17	13	5	0.48	0.70
15	169	67	50	34	25	11	0.61	0.88
18	292	117	86	58	43	18	0.73	1.06
21	463	185	137	93	69	29	0.85	1.23
24	691	276	205	138	102	43	0.97	1.40
27	984	394	292	197	146	62	1.10	1.59
30	1,350	540	400	270	200	84	1.22	1.77
33	1,797	719	532	359	266	112	1.34	1.96
36	2,331	933	691	467	346	146	1.46	2.11
42	3,704	1,482	1,098	741	549	232	1.70	2.47
48	5,529	2,212	1,638	1,106	819	346	1.95	2.82
54	7,873	3,149	2,335	1,575	1,168	492	2.19	3.17
Specific Weight = 175 pcf								
* 9	39	15	11	8	6	2	0.37	0.53 *
12	92	37	27	18	14	5	0.48	0.70
15	179	72	53	36	27	11	0.61	0.88
18	309	124	92	62	46	19	0.73	1.06
21	491	196	146	98	73	31	0.85	1.23
24	733	293	217	147	109	46	0.97	1.40
27	1,044	417	309	209	155	65	1.10	1.59
30	1,432	573	424	286	212	89	1.22	1.77
33	1,906	762	565	381	282	119	1.34	1.94
36	2,474	990	733	495	367	155	1.46	2.11
42	3,929	1,571	1,164	786	582	246	1.70	2.47
48	5,864	2,346	1,738	1,173	869	367	1.95	2.82
54	8,350	3,340	2,474	1,670	1,237	522	2.19	3.17

Notes:
1. Stone weight limit data from ETL 1110-2-120 (HQUSACE, 1971 (14 May), "Additional Guidance for Riprap Channel Protection, Ch 1," US Government Printing Office, Washington, DC). Relationship between diameter and weight is based on the shape of a sphere.
2. The maximum limits at the W_{50} and W_{15} sizes can be increased as in the Lower Mississippi Valley Division Standardized Gradations shown in Appendix F.

e. Layer thickness. All stones should be contained within the riprap layer thickness to provide maximum resistance against erosive forces. Oversize stones, even in isolated spots, may result in riprap failure by precluding mutual support and interlock between individual stones, causing large voids that expose filter and bedding materials, and creating excessive local turbulence that removes smaller size stone. Small amounts of oversize stone should be removed individually and replaced with proper size stones. The following criteria apply to the riprap layer thickness:

(1) It should not be less than the spherical diameter of the upper limit W_{100} stone or less than 1.5 times the spherical diameter of the upper limit W_{50} stone, whichever results in the greater thickness.

(2) The thickness determined by (1) above should be increased by 50 percent when the riprap is placed underwater to provide for uncertainties associated with this type of placement. At one location in the US Army Engineer Division, Missouri River, divers and sonic sounders were used to reduce the underwater thickness to 1.25 times the dry placement thickness.

Section II
Channel Characteristics

3-3. Side Slope Inclination

The stability of riprap slope protection is affected by the steepness of channel side slopes. Side slopes should ordinarily not be steeper than 1V on 1.5H, except in special cases where it may be economical to use larger hand-placed stone keyed well into the bank. Embankment stability analysis should properly address soils characteristics, groundwater and river conditions, and probable failure mechanisms. The size of stone required to resist the erosive forces of channel flow increases when the side slope angle approaches the angle of repose of a riprap slope protection. Rapid water-level recession and piping-initiated failures are other factors capable of affecting channel side slope inclination and needing consideration in design.

3-4. Channel Roughness, Shape, Alignment, and Gradient

As boundary shear forces and velocities depend on channel roughness, shape, alignment, and invert gradient, these factors must be considered in determining the size of stone required for riprap revetment. Comparative cost estimates should be made for several alternative channel plans to determine the most economical and practical combination of channel factors and stone size. Resistance coefficients (Manning's n) for riprap placed in the dry should be estimated using the following form of Strickler's equation:

$$n = K \left[D_{90}(\text{min})\right]^{1/6} \qquad (3\text{-}2)$$

where

K = 0.036, average of all flume data

= 0.034 for velocity and stone size calculation

= 0.038 for capacity and freeboard calculation

$D_{90}(\text{min})$ = size of which 90 percent of sample is finer, from minimum or lower limit curve of gradation specification, ft

The K values represent the upper and lower bounds of laboratory data determined for bottom riprap. Resistance data from a laboratory channel which had an irregular surface similar to riprap placed underwater show a Manning's n about 15 percent greater than for riprap
* placed in the dry. Equation 3-2 provides resistance losses due to the surface roughness of the riprap and does not include form losses such as those caused by bends. Equation 3-2 should be limited to slopes less than 2 percent. *

Section III
Design Guidance for Stone Size

3-5. General

Riprap protection for open channels is subjected to hydrodynamic drag and lift forces that tend to erode the revetment and reduce its stability. Undermining by scour beyond the limits of protection is also a common cause of failure. The drag and lift forces are created by flow velocities adjacent to the stone. Forces resisting motion are the submerged weight of the stone and any downward and lateral force components caused by contact with other stones in the revetment. Stone availability and experience play a large part in determining size of riprap. This is particularly true on small projects where hydraulic parameters are ill-defined and the total amount of riprap required is small.

3-6. Design Conditions

Stone size computations should be conducted for flow conditions that produce the maximum velocities at the riprapped boundary. In many cases, velocities continue to increase beyond bank-full discharge; but sometimes back-water effects or loss of flow into the overbanks results in velocities that are less than those at bank-full. Riprap at channel bends is designed conservatively for the point having the maximum force or velocity. For braided channels, bank-full discharges may not be the most severe condition. At lesser flows, flow is often divided into multiple channels. Flow in these channels often impinges
* abruptly on banks or levees at sharp angles.

3-7. Stone Size

This method for determining stone size uses depth-averaged local velocity. The method is based on the idea that a designer will be able to estimate local velocity
* better than local boundary shear. Local velocity and local flow depth are used in this procedure to quantify the imposed forces. Riprap size and unit weight quantify the resisting force of the riprap. This method is based on a large body of laboratory data and has been compared to available prototype data (Maynord 1988). It defines the stability of a wide range of gradations if placed to a thick-
* ness of $1D_{100}$(max). Guidance is also provided for thickness greater than $1D_{100}$(max). This method is applicable to side slopes of 1V on 1.5H or flatter.

a. Velocity estimation. The characteristic velocity for side slopes V_{SS} is the depth-averaged local velocity over the slope at a point 20 percent of the slope length from
* the toe of slope. Plate 33 presents the ratio V_{SS}/V_{AVG}, where V_{AVG} is the average channel velocity at the upstream end of the bend, as a function of the channel geometry, which is described by R/W, where R is the center-line radius of bend and W is the water-surface width. V_{AVG}, R, and W should be based on flow in the main channel only and should not include overbank areas. The trapezoidal curve for V_{SS}/V_{AVG} shown in Plate 33 is based on the STREMR numerical model described in Bernard (1993). The primary factors affecting velocity distribution in riprap-lined, trapezoidal channel bendways are R/W, bend angle, and aspect ratio (bottom width/depth). Data in Maynord (1992) show a trapezoidal channel having the same bottom width but side slopes ranging from 1V:1.5H to 1V:3H to have the same maximum V_{SS}/V_{AVG} at the downstream end of the bend. Plate 33 should be used for side slopes from 1V:3H to 1V:1.5H. For straight channels sufficiently far (>5W) from upstream bends, large values of R/W should be used, resulting in constant values of V_{SS}/V_{AVG}. Very few channels are straight enough to justify using $V_{SS}/V_{AVG} < 1$. A minimum ratio of $V_{SS}/V_{AVG} = 1$ is recommended for side slopes in straight channels. Rock stability should be checked for both side slopes and the channel bottom. In bendways, the outer bank side slope will generally require the largest rock size. In straight reaches, the channel bottom will often require the largest stone size. Velocities in the center of a straight channel having equal bottom and side slope roughness range from 10 to 20 percent greater than V_{AVG}. Plate 34 describes V_{SS} and Plate 35 shows the location in a trapezoidal channel bend of the maximum V_{SS}. Velocity downstream of bends decays at approximately the following rate: No decay in first channel width downstream of bend exit; decay of $V_{SS}/V_{AVG} = 0.1$ per channel width until $V_{SS}/V_{AVG} = 1.0$. Plate 36 shows the variation in velocity over the side slope in a channel. The straight channel curve in Plate 36 was found applicable to both 1V:2H and 1V:3H side slopes. The bend curve for $R/W = 2.6$ was taken from a channel having strong secondary currents and represents a severe concentration of high velocity upon the channel side slope. These two curves represent the extremes in velocity distribution to be expected along the outer bank of a channel bend having a riprap side slope from toe of bank to top of bank. Knowing V_{SS} from Plate 33, the side slope
velocity distribution can be determined at the location of *
V_{SS}. An alternate means of velocity estimation based on field observation is discussed in Appendix G. The alpha method (Appendix C), or velocities resulting from subsections of a water-surface profile computation, should be used only in straight reaches. When the alpha method is used, velocity from the subsection adjacent to the bank subsection should be used as V_{SS} in design of bank riprap.

b. Stone size relations. The basic equation for the representative stone size in straight or curved channels is

$$D_{30} = S_f C_s C_V C_T d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{1/2} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5} \qquad (3\text{-}3)$$

where

D_{30} = riprap size of which 30 percent is finer by weight, length

S_f = safety factor (see *c* below)

* C_s = stability coefficient for incipient failure, D_{85}/D_{15} = 1.7 to 5.2

= 0.30 for angular rock

* = 0.375 for rounded rock

C_V = vertical velocity distribution coefficient

= 1.0 for straight channels, inside of bends

= 1.283 - 0.2 log (R/W), outside of bends (1 for (R/W) > 26)

= 1.25, downstream of concrete channels

= 1.25, ends of dikes

C_T = thickness coefficient (see d(1) below)

* = 1.0 for thickness = $1D_{100}$(max) or 1.5 D_{50}(max), whichever is greater

* d = local depth of flow, length (same location as V)

γ_w = unit weight of water, weight/volume

* V = local depth-averaged velocity, V_{SS} for side slope riprap, length/time

K_1 = side slope correction factor (see *d*(1) below)

g = gravitational constant, length/time^2

* Some designers prefer to use the traditional D_{50} in riprap design. The approximate relationship between D_{50} and D_{30} is $D_{50} = D_{30}\ (D_{85}/D_{15})^{1/3}$. Equation 3-3 can be used with either SI (metric) or non-SI units and should be limited to slopes less than 2 percent.

c. Safety factor. Equation 3-3 gives a rock size that should be increased to resist hydrodynamic and a variety of nonhydrodynamic-imposed forces and/or uncontrollable physical conditions. The size increase can best be accomplished by including the safety factor, which will be a
* value greater than unity. The minimum safety factor is
* S_f = 1.1 . The minimum safety factor may have to be increased in consideration for the following conditions:

(1) Imposed impact forces resulting from logs, uprooted trees, loose vessels, ice, and other types of large floating debris. Impact will produce more damage to alighter weight riprap section than to a heavier section. For moderate debris impact, it is unlikely that an added
* safety factor should be used when the blanket thickness exceeds 15 in.

(2) The basic stone sizing parameters of velocity,
* unit weight of rock, and depth need to be determined as accurately as possible. A safety factor should be included to compensate for small inaccuracies in these parameters. If conservative estimates of these parameters are used in the analysis, the added safety factor should not be used. The safety factor should be based on the anticipated error in the values used. The following discussion shows the importance of obtaining nearly correct values rather than relying on a safety factor to correct inaccurate or assumed stone sizing parameters. The average velocity over the toe of the riprap is an estimate at best and is the parameter to which the rock size is the most sensitive. A check of the sensitivity will show that a 10 percent change in velocity will result in a nearly 100 percent change in the weight limits of the riprap gradation (based on a sphere) and about a 30 percent change in the riprap
* thickness. The riprap size is also quite sensitive to the unit weight of the rock to be used: a 10 percent change in the
* unit weight will result in a 70 percent change in the weight limits of the riprap gradation (based on a sphere) and about a 20 percent change in the riprap thickness. The natural variability of unit weight of stone from a stone source adds to the uncertainty (EM 1110-2-2302).
* The rock size is not nearly as sensitive to the depth parameter.

(3) Vandalism and/or theft of the stones is a serious problem in urban areas where small riprap has been placed. A W_{50}(min) of 80 lb should help prevent theft and vandalism. Sometimes grouted stone is used around vandalism-prone areas.

(4) The completed revetment will contain some
* pockets of undersized rocks, no matter how much effort is devoted to obtaining a well-mixed gradation throughout the revetment. This placement problem can be assumed to occur on any riprap job to some degree but probably more frequently on jobs that require stockpiling or additional handling. A larger safety factor should be considered with stockpiling or additional hauling and where placement will be difficult if quality control cannot be expected to address these problems.

(5) The safety factor should be increased where severe freeze-thaw is anticipated.

The safety factor based on each of these considerations should be considered separately and then the largest of these values should be used in Equation 3-3.

d. Applications.

(1) The outer bank of straight channels downstream of bends should be designed using velocities computed for the bend. In projects where the cost of riprap is high, a channel model to indicate locations of high velocity might be justified. Equation 3-3 has been developed into Plate 37, which is applicable to thicknesses equal to $1D_{100}(max)$, γ_s of 165 pcf, and the S_f of 1.1. Plate 38 is used to correct for values of other than γ_s of 165 pcf (when D_{30} is determined from Plate 37). The K_1 side slope factor is normally defined by the relationship of Carter, Carlson, and Lane (1953)

$$K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} \tag{3-4}$$

where

θ = angle of side slope with horizontal

ϕ = angle of repose of riprap material (normally 40 deg)

Results given in Maynord (1988) show Equation 3-4 to be conservative and that the repose angle is not a constant 40 deg but varies with several factors. The recommended relationship for K_1 as a function of θ is given in Plate 39 along with Equation 3-4 using ϕ = 40 deg. Using the recommended curve for side slope effects, the least volume of rock per unit length of bank line occurs on a 1V:1.5H to 1V:2H side slope. Also shown on Plate 39 is the correction for side slope when D_{30} is determined from Plate 37. Correction for the vertical velocity distribution in bends is shown in Plate 40. Testing has been conducted to determine the effects of blanket thickness greater than $1D_{100}(max)$ on the stability of riprap. Results are shown in Plate 40. The thickness coefficient C_T accounts for the increase in stability that occurs when riprap is placed thicker than the minimum thickness of $1D_{100}(max)$ or 1.5 $D_{50}(max)$, whichever is greater.

(2) The basic procedure to determine riprap size using the graphical solution of this method is as follows:

(a) Determine average channel velocity (HEC-2 or other uniform flow computational methods, or measurement).

(b) Find V_{SS} using Plate 33.

(c) Find D_{30} using Plate 37.

(d) Correct for other unit weights, side slopes, vertical velocity distribution, or thicknesses using Plates 38 through 40.

(e) Find gradation having $D_{30}(min) \geq$ computed D_{30}. Alternately Equation 3-3 is used with Plates 39 and 40 to replace steps (c) and (d).

(3) This procedure can be used in both natural channels with bank protection only and prismatic channels having riprap on bed and banks. Most bank protection sections can be designed by direct solution. In these cases, the extent of the bank compared to the total perimeter of the channel means that the average channel velocity is not significantly affected by the riprap. The first example in Appendix H demonstrates this type of bank protection.

(4) In some cases, a large part of the channel perimeter is covered with riprap; the average channel velocity, depth, and riprap size are dependent upon one another; and the solution becomes iterative. A trial riprap gradation is first assumed and resistance coefficients are computed using Equation 3-2. Then the five steps described in (2) above are conducted. If the gradation found in paragraph (e) above is equal to the assumed trial gradation, the solution is complete. If not, a new trial gradation is assumed and the procedure is repeated. The second example in Appendix H demonstrates this type of channel riprap.

(5) In braided streams and some meandering streams, flow is often directed into the bank line at sharp angles (angled flow impingement). For braided streams having impinged flow, the above stone sizing procedures require modification in two areas: the method of velocity estimation and the velocity distribution coefficient C_V. All other factors and coefficients presented are applicable.

(a) The major challenge in riprap design for braided streams is estimating the imposed force at the impingement point. Although unproven, the most severe bank

* attack in braided streams is thought to occur when the water surface is at or slightly above the tops of the mid-channel bars. At this stage, flow is confined to the multiple channels that often flow into or "impinge" against bank lines or levees. At lesser flows, the depths and velocities in the multiple channels are decreased. At higher flows, the channel area increases drastically and streamlines are in a more downstream direction rather than into bank lines or levees.

(b) The discharge that produces a stage near the tops of the midchannel bars is Q_{tmcb} . Q_{tmcb} is probably highly correlated with the channel-forming discharge concept. In the case of the Snake River near Jackson, Wyoming, Q_{tmcb} is 15,000-18,000 cfs, which has an average recurrence interval of about 2-5 years. Using cross-section data to determine the channel area below the tops of the midchannel bars and Q_{tmcb} allows determination of the average channel velocity at the top of the midchannel bars, V_{tmcb} .

(c) Field measurements at impingement sites were taken in 1991 on the Snake River near Jackson, Wyoming, and reported in Maynord (1993). The maximum observed ratio $V_{SS}/V_{tmcb} = 1.6$, which is almost identical to the ratio shown in Plate 33 for sharp bendways having $R/W = 2$ in natural channels, and this ratio is recommended for determining V_{SS} for impinged flow. The second area of the design procedure requiring modification for impinged flow is the velocity distribution coefficient C_V , which varies with R/W in bendways as shown in Plate 40. Impinged flow areas are poorly aligned bends having low R/W , and $C_V = 1.25$ is recommended for design.

(6) Transitions in size or shape may also require riprap protection. The procedures in this paragraph are applicable to gradual transitions where flow remains tranquil. In areas where flow changes from tranquil to rapid and then back to tranquil, riprap sizing methods applicable to hydraulic structures (HDC 712-1) should be used. In converging transitions, the procedures based on Equation 3-3 can be used unaltered. In expanding transitions, flow can concentrate on one side of the expansion and design velocities should be increased. For installations immediately downstream of concrete channels, a vertical velocity distribution coefficient of 1.25 should be used due to the difference in velocity profile over the two surfaces.

* *e. Steep slope riprap design.*

In cases where unit discharge is low, riprap can be used on steep slopes ranging from 2 to 20 percent. A typical application is a rock-lined chute. The stone size equation is

$$D_{30} = \frac{1.95\ S^{0.555}\ q^{2/3}}{g^{1/3}} \tag{3-5}$$

where

S = slope of bed

q = unit discharge

Equation 3-5 is applicable to thickness = 1.5 D_{100}, angular rock, unit weight of 167 pcf, D_{85}/D_{15} from 1.7 to 2.7, slopes from 2 to 20 percent, and uniform flow on a downslope with no tailwater. The following steps should be used in application of Equation 3-5:

(1) Estimate $q = Q/b$ where b = bottom width of chute.

(2) Multiply q by flow concentration factor of 1.25. Use greater factor if approach flow is skewed.

(3) Compute D_{30} using Equation 3-5.

(4) Use uniform gradation having $D_{85}/D_{15} \le 2$ such as Table 3-1.

* (5) Restrict application to straight channels with side slope of 1V:2.5H or flatter.

(6) Use filter fabric beneath rock.

The guidance for steep slope riprap generally results in large riprap sizes. Grouted riprap is often used instead of loose riprap in steep slope applications. *

3-8. Revetment Top and End Protection

Revetment top and end protection requirements, as with all channel protective measures, are to assure the project benefits, to perform satisfactorily throughout the project economic life, and not to exceed reasonable maintenance costs. Reference is made to ER 1110-2-1405, with emphasis on paragraph 6*c*.

a. Revetment top. When the full height of a levee is to be protected, the revetment will cover the freeboard, i.e., extend to the top of the levee. This provides protection against waves, floating debris, and water-surface irregularities. Similar provisions apply to incised channel banks. A horizontal collar, at the top of the bank, is provided to protect against escaping and returning flows as necessary. The end protection methods illustrated in Plate 41 can be adapted for horizontal collars. Plate 36 provides general guidance for velocity variation over channel side slopes that can assist in evaluating the economics of reducing or omitting revetment for upper bank areas. Revetment size changes should not be made unless a sufficient quantity is involved to be cost effective. Many successful revetments have been constructed where the top of the revetment was terminated below the design flow line. See USACE (1981) for examples.

b. Revetment end protection. The upstream and downstream ends of riprap revetment should be protected against erosion by increasing the revetment thickness T or extending the revetment to areas of noneroding veloci-
* ties and relatively stable banks. A smooth transition should be provided from where the end protection begins to the design riprap section. The keyed-in section should satisfy filter requirements. The following guidance applies to the alternative methods of end protection illustrated in Plate 41.

(1) Method A. For riprap revetments 12 in. thick or less, the normal riprap layer should be extended to areas where velocities will not erode the natural channel banks.

(2) Method B. For riprap revetments exceeding 12 in. in thickness, one or more reductions in riprap thick-
* ness and stone size may be required (Plate 41) until velocities decrease to a noneroding natural channel velocity.

(3) Method C. For all riprap revetments that do not terminate in noneroding natural channel velocities, the ends of the revetment should be enlarged, as shown in
* Plate 41. The decision to terminate the revetment in erosive velocities should be made with caution since severe erosion can cause the revetment to fail by progressive flanking.

c. Length. Riprap revetment is frequently carried too far upstream and not far enough downstream of a channel bend. In a trapezoidal channel, the maximum velocities along the outer bank are often located in the straight reach immediately downstream of the bend for relatively large distances downstream. In a natural channel, the limit of protection on the downstream end should depend on where the flow crosses to the opposite bank, and should consider future bar building on the opposite bank, resulting in channel constriction and increased velocities. Guidance is generally lacking in this area, but review of aerial photographs of the subject location can provide some insight on where the crossover flow occurs. Model tests in a sand bed and bank flume (USACE 1981) were conducted to determine the limits of protection required to prevent scour that would lead to destruction of the revetment. These tests were conducted in a 110-deg bend having a constant discharge. The downstream end of the revetment had to be 1.5 channel widths downstream of the end of the bend. Geomorphic studies to determine revetment ends should be considered.

Section IV
Revetment Toe Scour Estimation and Protection

3-9. General

Toe scour is probably the most frequent cause of failure of riprap revetments. This is true not only for riprap, but also for a wide variety of protection techniques. Toe
* scour is the result of several factors, including these three:

a. Meandering channels, change in cross section that occurs after a bank is protected. In meandering channels the thalweg often moves toward the outer bank after the bank is protected. The amount of change in cross section that occurs after protection is added is related to the erodibility of the natural channel bed and original bank material. Channels with highly erodible bed and banks can experience significant scour along the toe of the new revetment.

b. Meandering channels, scour at high flows. Bed profile measurements have shown that the bed observed at low flows is not the same bed that exists at high flows. At high flows the bed scours in channel bends and builds up in the crossings between bends. On the recession side of the flood, the process is reversed. Sediment is eroded from the crossings and deposited in the bends, thus obscuring the maximum scour that had occurred.

c. Braided channels. Scour in braided channels can reach a maximum at intermediate discharges where flow in the channel braids attacks banks at sharp angles.

Note that local scour is the mechanism being addressed herein. When general bed degradation or headcutting is expected, it must be added to the local scour. When scour mechanisms are not considered in the design of protection works, undermining and failure may result. * Plate 42 may be used for depth of scour estimates. The design curve in Plate 42 represents an upper limit for scour in channels having irregular alignments. For bendways having a relatively smooth alignment, a 10 percent reduction from the design curve is recommended. Neill (1973) provides additional information on scour depth estimation.

3-10. Revetment Toe Protection Methods

Toe protection may be provided by two methods:

a. Extend to maximum scour depth. Place the lower extremity below the expected scour depth or found it on nonerodible material. These are the preferred methods, but they can be difficult and expensive when underwater excavation is required.

b. Place launchable stone. Place sufficient launchable stone to stabilize erosion. Launchable stone is defined as stone that is placed along expected erosion areas at an elevation above the zone of attack. As the attack and resulting erosion occur below the stone, the stone is undermined and rolls/slides down the slope, stopping the erosion. This method has been widely used on sand bed streams. Successful applications include:

(1) Windrow revetments: riprap placed at top of bank.

(2) Trench-fill revetments: riprap placed at low water level.

(3) Weighted riprap toes: riprap placed at intersection of channel bottom and side slope.

Trench-fill revetments on the Mississippi River have successfully launched to protect for a vertical scour depth of up to 50 ft. On gravel bed streams, the use of launchable stone is not as widely accepted as in sand bed streams. Problems with using launchable stone in some gravel bed rivers may be the result of underestimating stone size, scour depth, or launchable stone volume because the concept of launchable stone has been successful on several gravel bed rivers.

3-11. Revetment Toe Protection Design

The following guidance applies to several alternative methods of toe protection illustrated in Plate 43.

a. Method A. When toe excavation can be made in the dry, the riprap layer may be extended below the existing groundline a distance exceeding the anticipated depth of scour. If excavation quantities are prohibitive, the * concept of Method D can be adapted to reduce excavation.

b. Method B. When the bottom of the channel is nonerodible material, the normal riprap should be keyed in at streambed level.

c. Method C. When the riprap is to be placed underwater and little toe scour is expected (such as in straight reaches that are not downstream of bends, unless stream is braided), the toe may be placed on the existing bottom with height a and width c equal to 1.5T and 5T , respectively. This compensates for uncertainties of underwater placement.

d. Method D. An extremely useful technique where water levels prohibit excavation for a toe section is to place a launchable section at the toe of the bank. Even if excavation is practicable, this method may be preferred for cost savings if the cost of extra stone required to produce a launched thickness equal to or greater than * T plus the increase shown in Table 3-2 is exceeded by the cost of excavation required to carry the design thickness T down the slope. This concept simply uses toe scour as a substitute for mechanical excavation. This method also has the advantage of providing a "built-in" scour gage, allowing easy monitoring of high-flow scour and the need for additional stone reinforcement by visual inspection of the remaining toe stone after the high flow subsides or by surveyed cross sections if the toe stone is underwater. It is readily adaptable to emergency protection, where high flow and the requirement for quick action make excava- * tion impractical. Shape of the stone section before launching is not critical, but thickness of the section is important because thickness controls the rate at which rock is released in the launching process. For gradual scour in regular bendways, the height of the stone section before launching should be from 2.5 to 4.0 times the bank protection thickness (T). For rapid scour in impinged flow environments or in gravel bed streams, the stone section height before launching should be 2.5 to 3.0 T. In

*

Table 3-2
Increase in Stone Volume for Riprap Launching Sections

Vertical Launch Distance, ft[1]	Volume Increase, Percent Dry Placement	Volume Increase, Percent Underwater Placement
≤ 15	25	50
> 15	50	75

Note:
[1] From bottom of launch section to maximum scour.

any case, the thinner and wider rock sections represented by the lower values of thickness have an apparent advantage in that the rock in the stream end of the before-launch section has a lesser distance to travel in the launching process. Providing an adequate volume of stone is critical. Stone is lost downstream in the launching process; and the larger the scour depth, the greater the percentage of stone lost in the launching process. To compute the required launchable stone volume for Method D, the following assumptions should be used:

(1) Launch slope = 1V on 2H. This is the slope resulting from rock launched on noncohesive material in both model and prototype surveys. Launch slope is less predictable if cohesive material is present, since cohesive material may fail in large blocks.

(2) Scour depth = existing elevation - maximum scour elevation.

* (3) Thickness after launching = thickness of the bank revetment T .

* To account for the stone lost during launching and for placement underwater, the increases in stone volume listed in Table 3-2 are recommended. Using these assumptions, the required stone volume for underwater placement for vertical launch distance less than 15 ft = 1.5T times launch slope length

$= 1.5T$ times scour depth times $\sqrt{5}$

$= 3.35T$ (scour depth)

Add a safety factor if data to compute scour depth are unreliable, if cohesive bank material is present, or if monitoring and maintenance after construction cannot be guaranteed. Guidance for a safety factor is lacking, so to some extent it must be determined by considering conse-
* quences of failure. Widely graded ripraps are recommended because of reduced rock voids that tend to prevent leaching of lower bank material through the launched riprap. Launchable stone should have $D_{85}/D_{15} \geq 2$. *

3-12. Delivery and Placement

Delivery and placement can affect riprap design. See EM 1110-2-2302 for detailed guidance. The common methods of riprap placement are hand placing; machine placing, such as from a skip, dragline, or some form of bucket; and dumping from trucks and spreading by bulldozer. Hand placement produces the most stable riprap revetment because the long axes of the riprap particles are oriented perpendicular to the bank. It is the most expensive method except when stone is unusually costly and/or labor unusually cheap. Steeper side slopes can be used with hand-placed riprap than with other placing methods. This reduces the required volume of rock. However, the greater cost of hand placement usually makes machine or dumped placement methods and flatter slopes more economical. Hand placement on steep slopes should be con-
* sidered when channel widths are constricted by existing bridge openings or other structures when rights-of-way are costly. In the machine placement method, sufficiently small increments of stone should be released as close to their final positions as practical. Rehandling or dragging operations to smooth the revetment surface tend to result in segregation and breakage of stone. Stone should not be dropped from an excessive height or dumped and spread as this may result in the same undesirable conditions. However, in some cases, it may be economical to increase the layer thickness and stone size somewhat to offset the shortcomings of this placement method. Smooth, compact riprap sections have resulted from compacting the placed stone sections with a broad-tracked bulldozer. This stone must be quite resistant to abrasion. Thickness for underwater placement should be increased by 50 percent to provide for the uncertainties associated with this type of placement. Underwater placement is usually specified in terms of weight of stone per unit area, to be distributed uniformly and controlled by a "grid" established by
* shoreline survey points.

Section V
Ice, Debris, and Vegetation

3-13. Ice and Debris

Ice and debris create greater stresses on riprap revetment by impact and flow concentration effects. Ice attachment to the riprap also causes a decrease in stability. The Cold Regions Research Engineering Laboratory, Hanover, NH, should be contacted for detailed guidance relative to ice effects on riprap. One rule of thumb is that thickness should be increased by 6-12 in., accompanied by appropriate increase in stone size, for riprap subject to attack by large floating debris. Riprap deterioration from debris impacts is usually more extensive on bank lines with steep slopes. Therefore, riprapped slopes on streams with heavy debris loads should be no steeper than 1V on 2.5H.

3-14. Vegetation

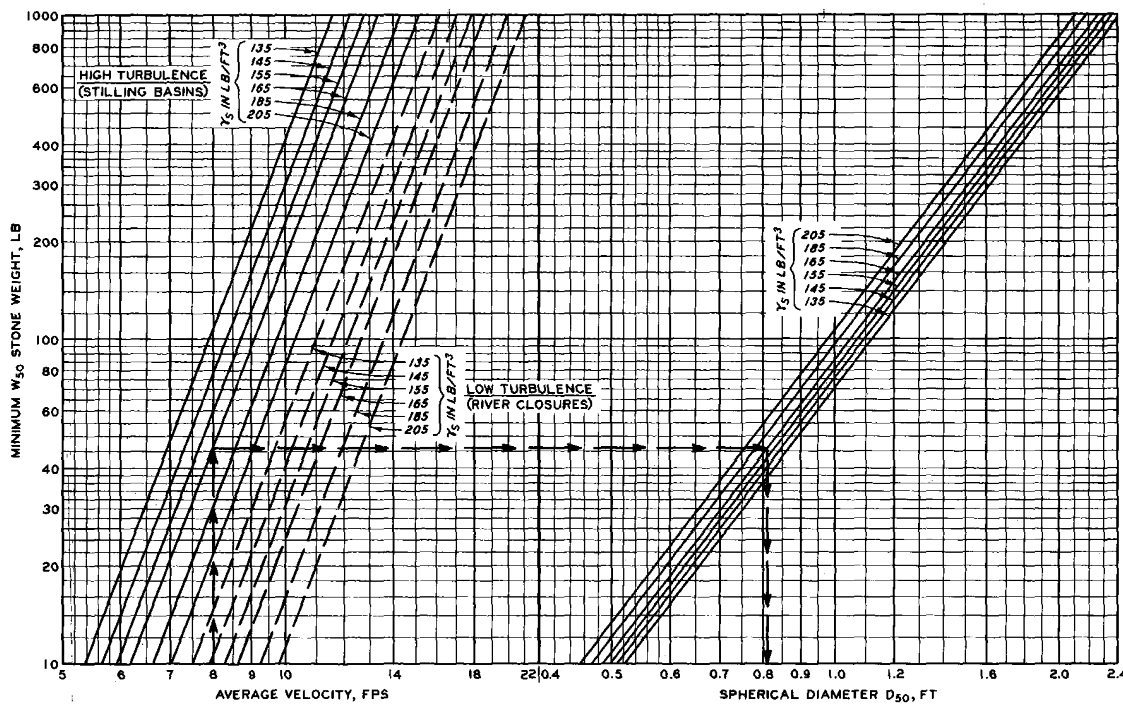
The guidance in this chapter is based on maintaining the riprap free of vegetation. When sediment deposits form lowflow berms on riprap installations, vegetation may be allowed on these berms under the following conditions: roots do not penetrate the riprap; failure of the riprap would not jeopardize project purposes prior to repairs; and the presence of the berm and vegetation does not significantly reduce the discharge capacity of the project. For riprap areas above the 4 or 5 percent exceedence flow line, consideration may be given to overlaying the riprap with soil and sod to facilitate maintenance by mowing rather than by hand or defoliants. This may be particularly appropriate for riprap protecting against eddy action around structures such as gate wells and outlet works in levees that are otherwise maintained by mowing. Recognizing that vegetation is, in most instances, inimical to riprap installations, planned use of vegetation with riprap should serve some justifiable purpose, be accounted for in capacity computations, be controllable throughout the project life, have a strengthened riprap design that will withstand the additional exigencies, and account for increased difficulty of inspection.

Section VI
Quality Control

3-15. Quality Control

Provisions should be made in the specifications for sampling and testing in-place riprap as representative sections of revetment are completed. Additional sample testing of in-place and in-transit riprap material at the option of the Contracting Officer should be specified. The primary concern of riprap users is that the in-place riprap meets specifications. Loading, transporting, stockpiling, and placing can result in deterioration of the riprap. Coordination of inspection efforts by experienced staff is necessary. Reference EM 1110-2-2302 for detailed sampling guidance and required sample volumes for in-place riprap.

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI



BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}(D_{50})^{1/2}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi\gamma_s}\right)^{1/3}$$

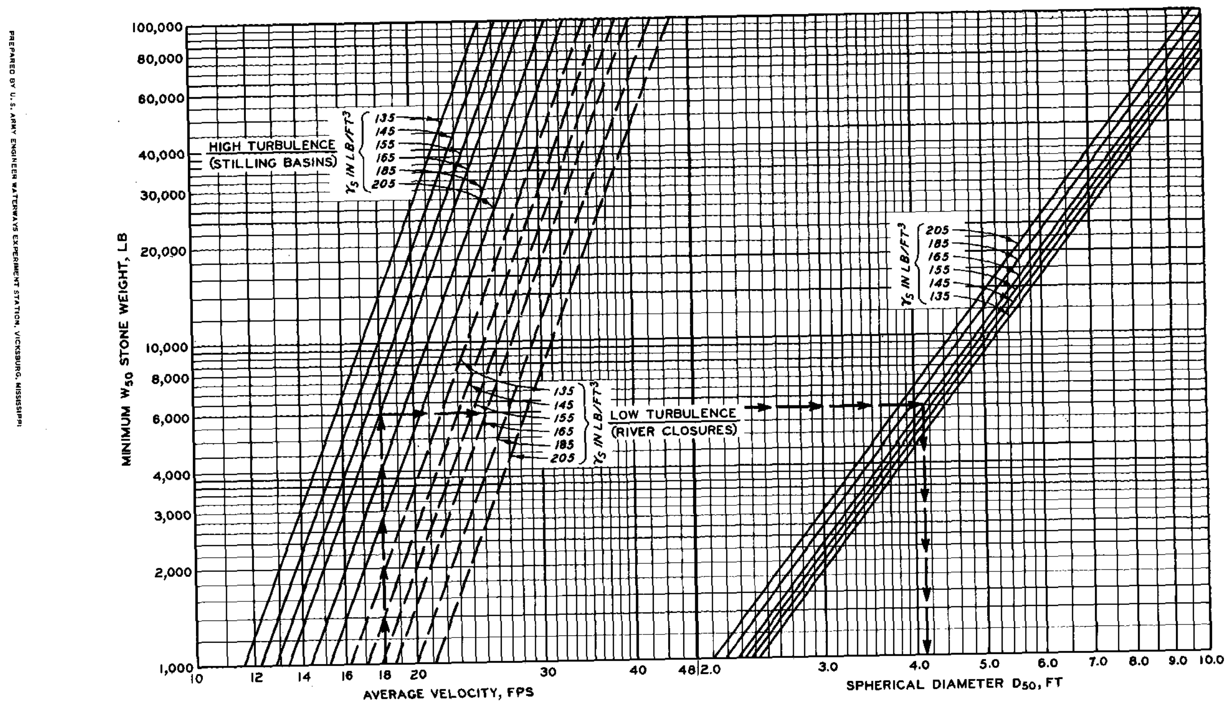
WHERE:
- V = VELOCITY, FPS
- γ_s = SPECIFIC STONE WEIGHT, LB/FT^3
- γ_w = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT^3
- W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
- D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}
- C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
- g = ACCELERATION OF GRAVITY, FT/SEC^2

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 1 OF 2)

REV 8-58, 9-70 WES 6-57

PLATE B-29



BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}\left(D_{50}\right)^{1/2}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi \gamma_s}\right)^{1/3}$$

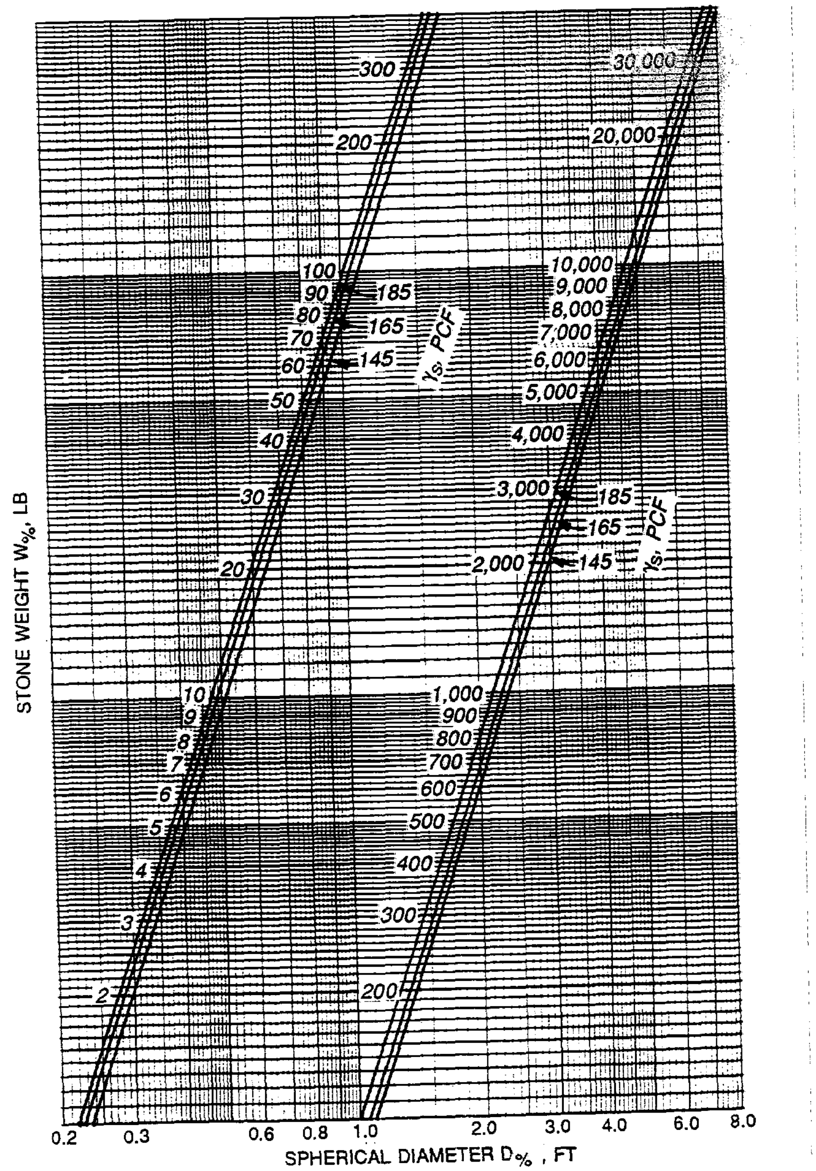
WHERE:
- V = VELOCITY, FPS
- γ_s = SPECIFIC STONE WEIGHT, LB/FT^3
- γ_w = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT^3
- W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
- D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}
- C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
- g = ACCELERATION OF GRAVITY, FT/SEC^2

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 2 OF 2)

REV 8-58, 9-70 WES 6-57

PLATE B-30

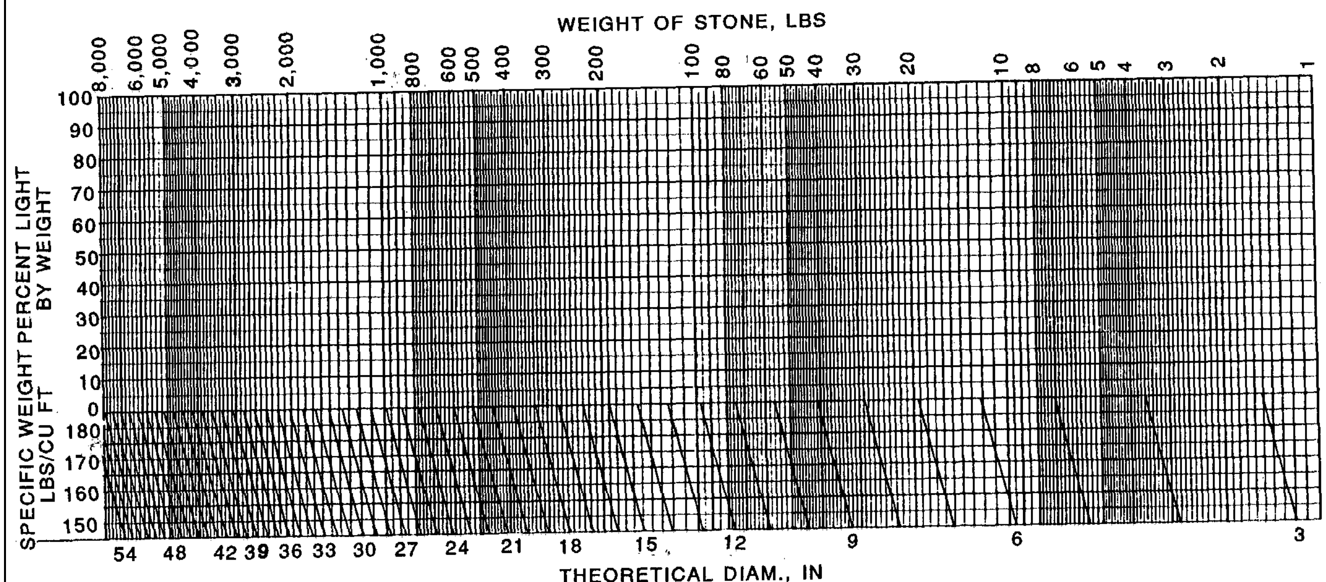


EQUATION:

$$W_{\%} = \frac{\pi \gamma_S D_{\%}^3}{6} \quad \text{OR} \quad D_{\%} = \left(\frac{6W_{\%}}{\pi \gamma_S}\right)^{1/3}$$

STONE WEIGHT VS SPHERICAL DIAMETER

PLATE B-31



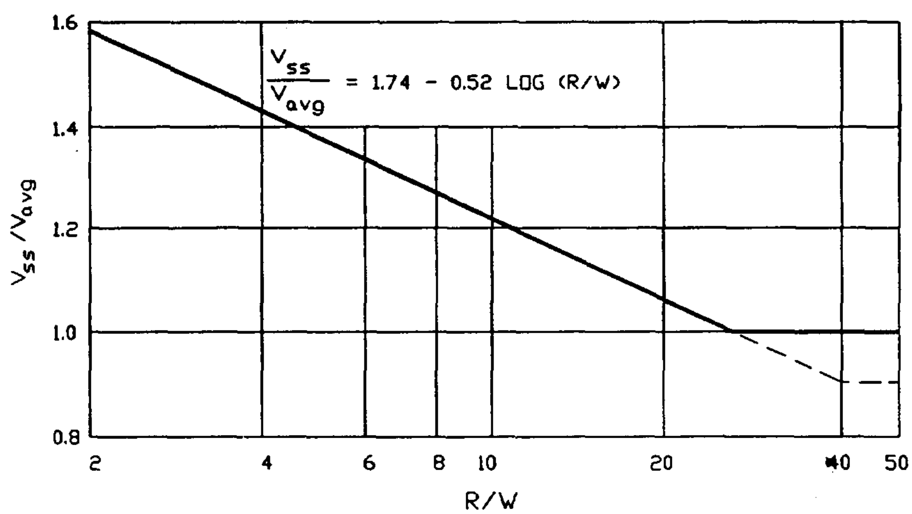
SPECIFIC WEIGHT OF STONE______LBS/CU FT

PROJECT______________
AREA________________
DATE______BY_________

RIPRAP GRADATION CURVES

PLATE B-32

*



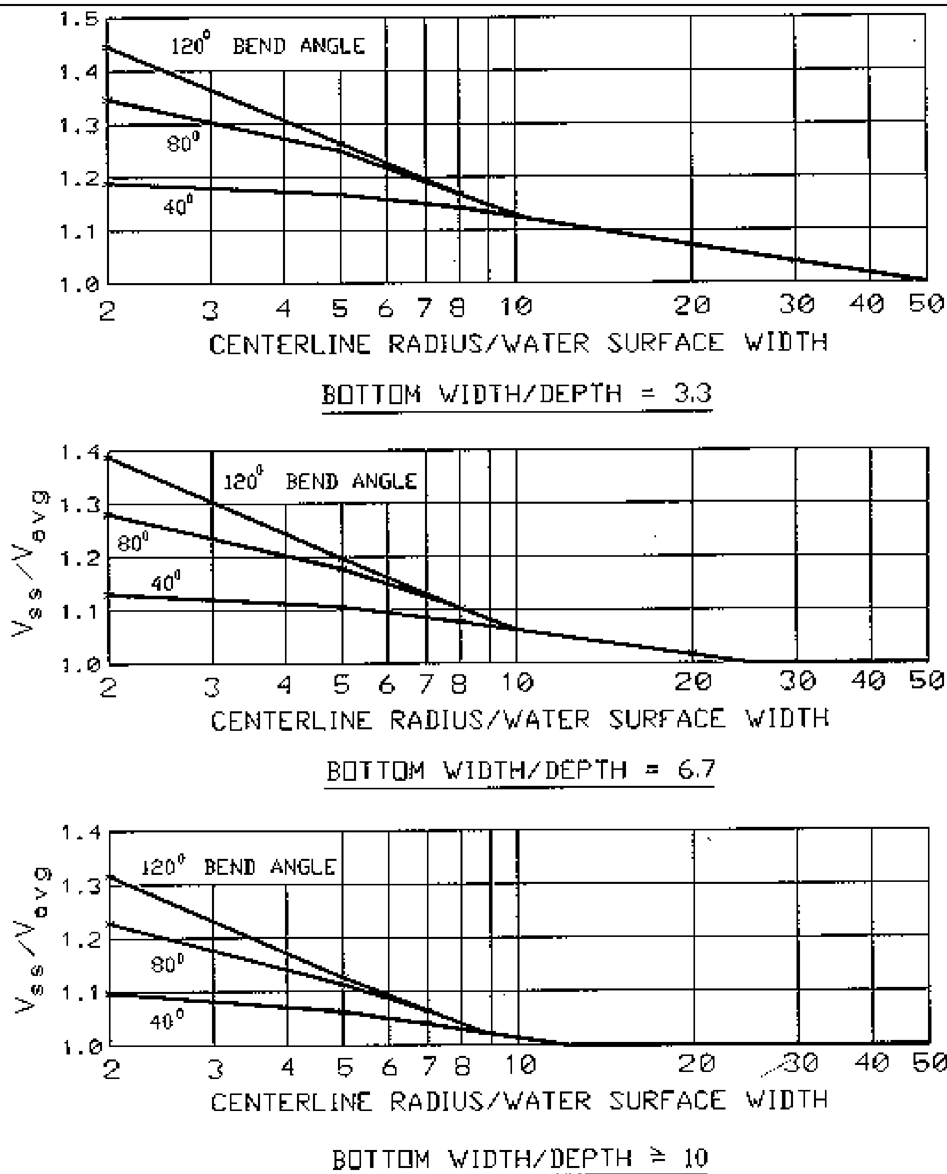
NOTE: V_{ss} IS DEPTH-AVERAGED VELOCITY AT 20 PERCENT OF SLOPE LENGTH UP FROM TOE

RIPRAP DESIGN VELOCITIES
NATURAL CHANNEL

Plate B-33
(Sheet 1 of 2)

*

*



NOTE: V_{ss} IS DEPTH-AVERAGED VELOCITY AT 20 PERCENT OF SLOPE LENGTH UP FROM TOE, MAXIMUM VALUE IN BEND

CURVES BASED ON STREMR NUMERICAL MODEL (BERNARD AND SCHNEIDER 1992)

APPLICABLE TO 1V:1.5H TO 1V:3H SIDE SLOPES

$n/(\text{MAXIMUM DEPTH})^{1/6} \leq 0.026$

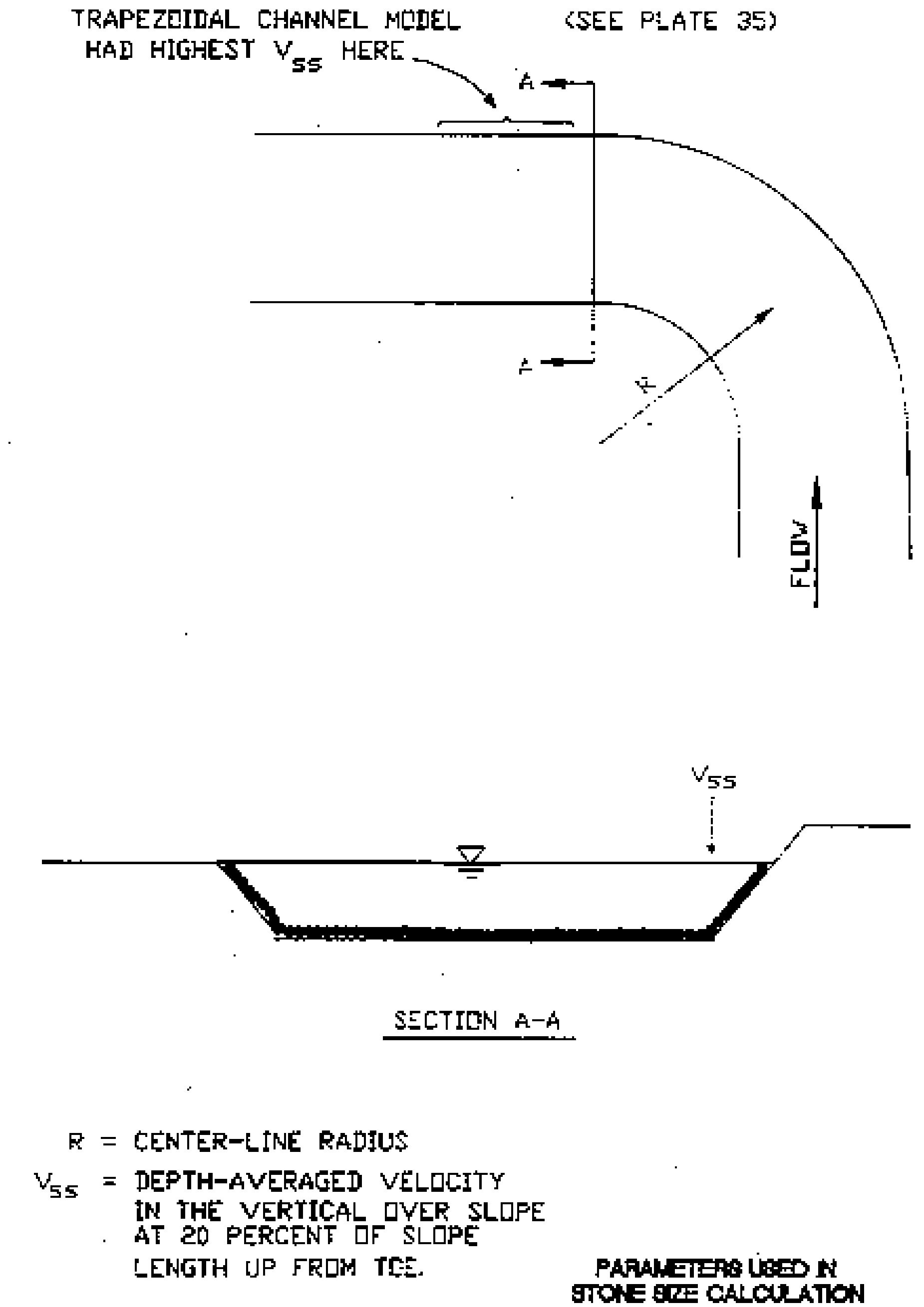
RIPRAP DESIGN VELOCITIES

TRAPEZOIDAL CHANNEL

Plate B-33
(Sheet 2 of 2)

*

*

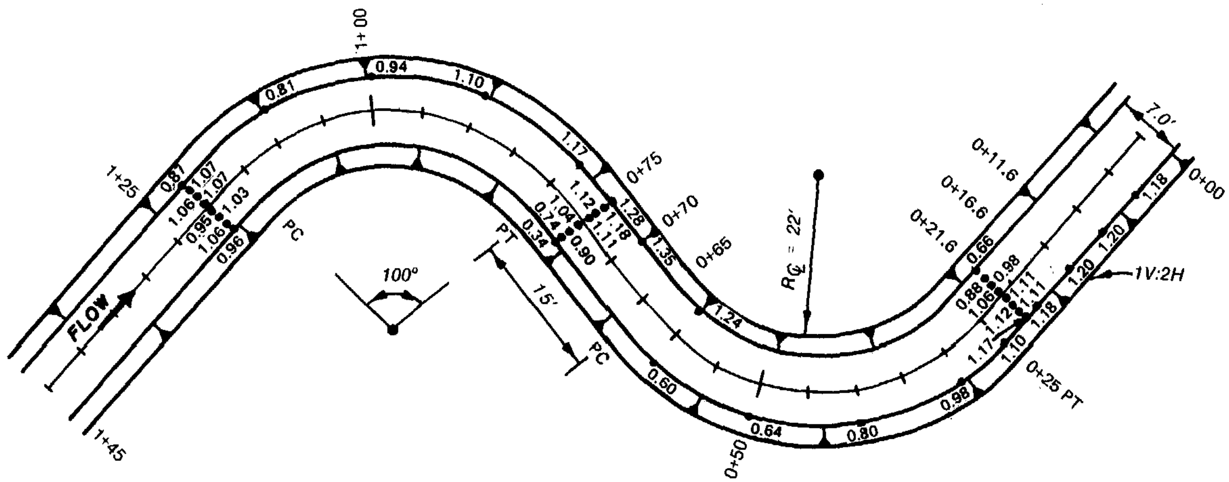


PARAMETERS USED IN STONE SIZE CALCULATION

Plate B-34

*

Plate B-35



NOTE: • 1.04 REPRESENTS $\frac{\text{AVERAGE VELOCITY IN VERTICAL}}{\text{AVERAGE CHANNEL VELOCITY}}$

BOTTOM SLOPE ≈ WATER-SURFACE SLOPE = 0.0025 FT/FT

RIPRAP: 50% #4 – 3/8, 50% 3/8 – 1/2

AVERAGE CHANNEL VELOCITY = 1.87 FPS

FROUDE NO. = 0.52

STATIONARY IN FEET

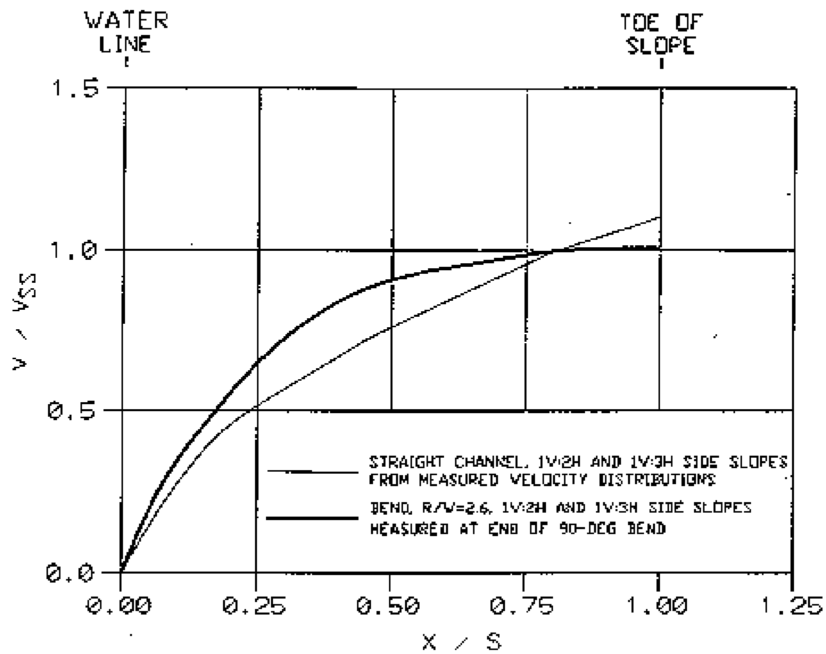
SEE PLATE 36 FOR VELOCITY X-SECTIONS

VELOCITY DISTRIBUTION IN
TRAPEZOIDAL CHANNEL

DISCHARGE 6.75 CFS DEPTH 0.455 FT

IV: 2H SIDE SLOPES

*



V = DEPTH-AVERAGED VELOCITY AT X

V_{SS} = DEPTH-AVERAGED VELOCITY AT 20 PERCENT UP SLOPE FROM TOE

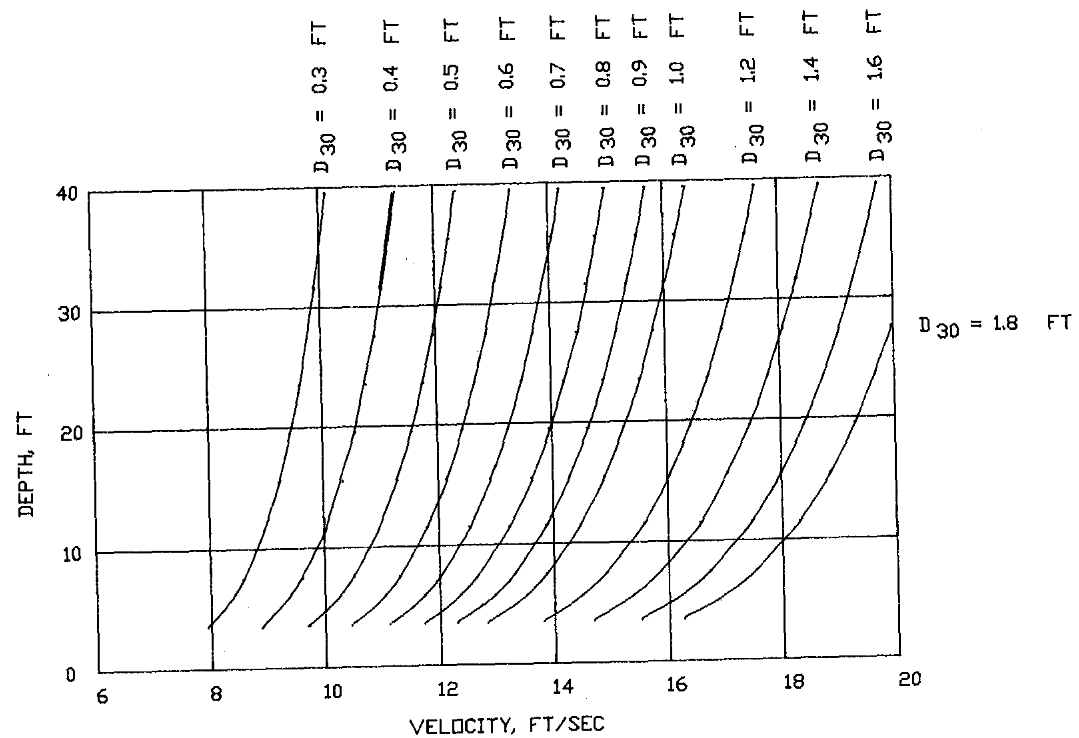
X = HORIZONTAL DISTANCE FROM WATER LINE

S = HORIZONTAL DISTANCE FROM WATER LINE TO TOE OF SLOPE

SIDE SLOPE VELOCITY DISTRIBUTION

Plate B-36

*

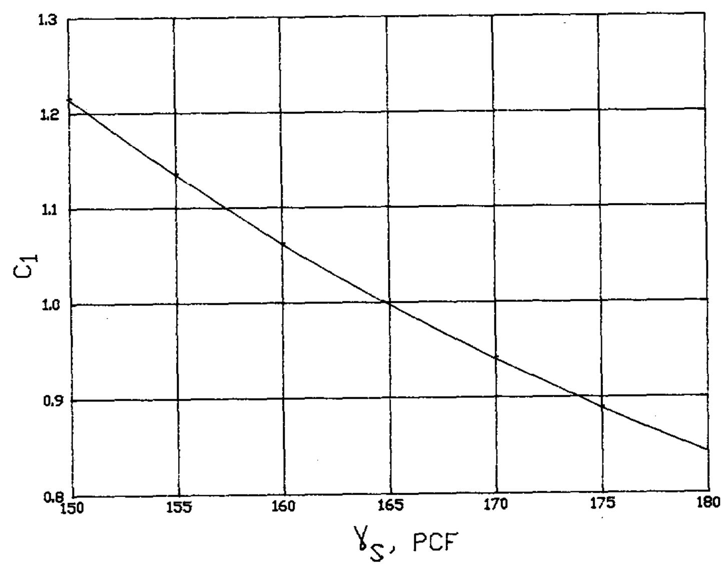


NOTE: APPLICABLE TO THICKNESS $1D_{100}$ (max)
AND CHANNEL BOTTOMS OR SIDE SLOPES
FLATTER THAN OR EQUAL TO 1V ON 4H.
STONE WEIGHT 165 pcf, $C_S = 0.30$, $C_V = C_T = 1.0$
$S_f = 1.1$ BASED ON EQUATION 3-3.

**DEPTH-AVERAGED VELOCITY
VS D30
AND DEPTH**

Plate B-37

*



$D_{30} = C_1 * (D_{30}$ FROM PLATE 37)

WHERE C_1 = CORRECTION FOR UNIT STONE WEIGHT

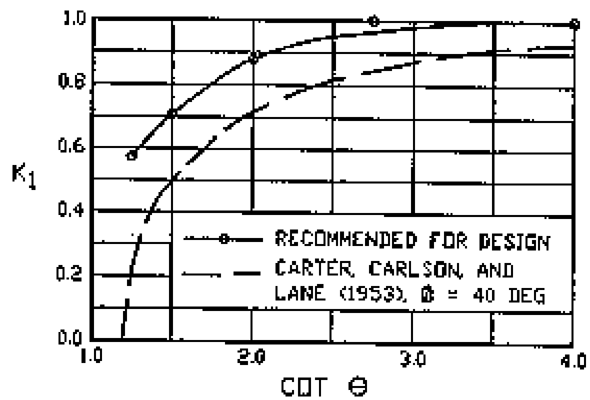
NOTE: DO NOT MAKE THIS CORRECTION IF
D_{30} COMPUTED FROM EQUATION 3-3

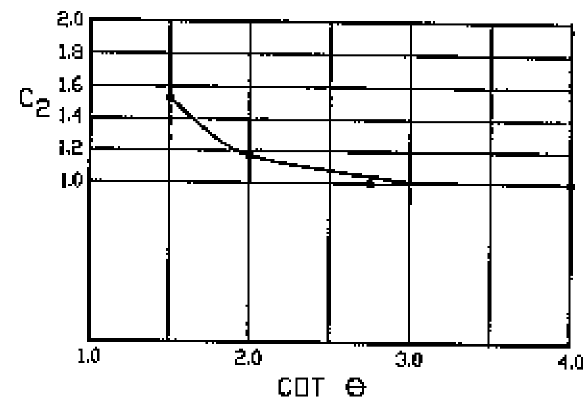
CORRECTION FOR UNIT STONE WEIGHT

Plate B-38

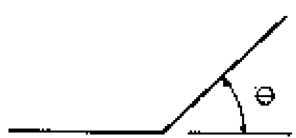
*

*





K_1 = SIDE SLOPE CORRECTION COEFFICIENT FOR USE IN EQUATION 3-3 ONLY.



D_{30} = C_2 * (D_{30} FROM PLATE 37)

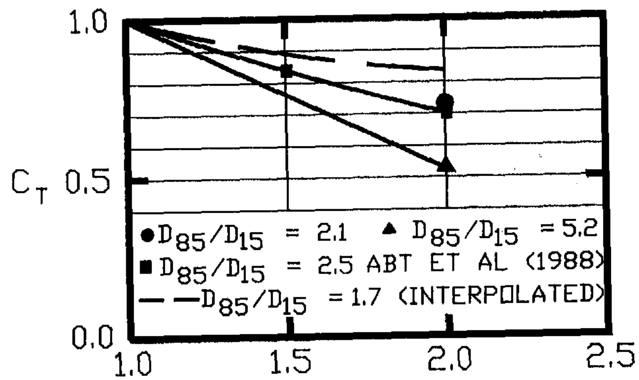
WHERE C_2 = CORRECTION FOR SIDE SLOPE ANGLE

NOTE : DO NOT MAKE THIS CORRECTION IF D_{30} COMPUTED FROM EQUATION 3-3, EQUIVALENT TO "RECOMMENDED FOR DESIGN" CURVE IN THE PLOT OF K_1 VERSUS COT Θ.

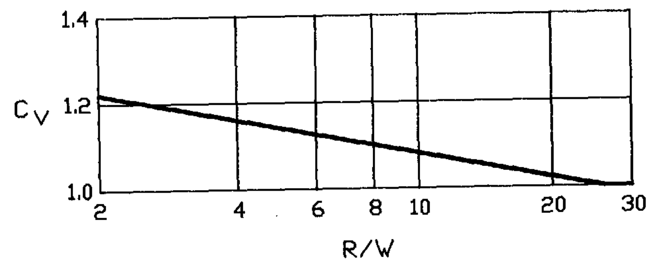
CORRECTION FOR SIDE SLOPE ANGLE

Plate B-39

*



$$N = \frac{\text{THICKNESS}}{T*}$$



WHERE C_T = CORRECTION FOR THICKNESS

$$= \frac{D_{30} \text{ FOR THICKNESS OF } NT*}{D_{30} \text{ FOR THICKNESS OF } T*}$$

$T*$ = $1D_{100}$ OR $1.5D_{50}$, WHICHEVER IS GREATER

D_{30} = C_V*(D_{30} FROM PLATE 37)

WHERE C_V = CORRECTION FOR VERTICAL VELOCITY DISTRIBUTION

CORRECTION FOR VERTICAL VELOCITY DISTRIBUTION IN BEND AND RIPRAP THICKNESS

*

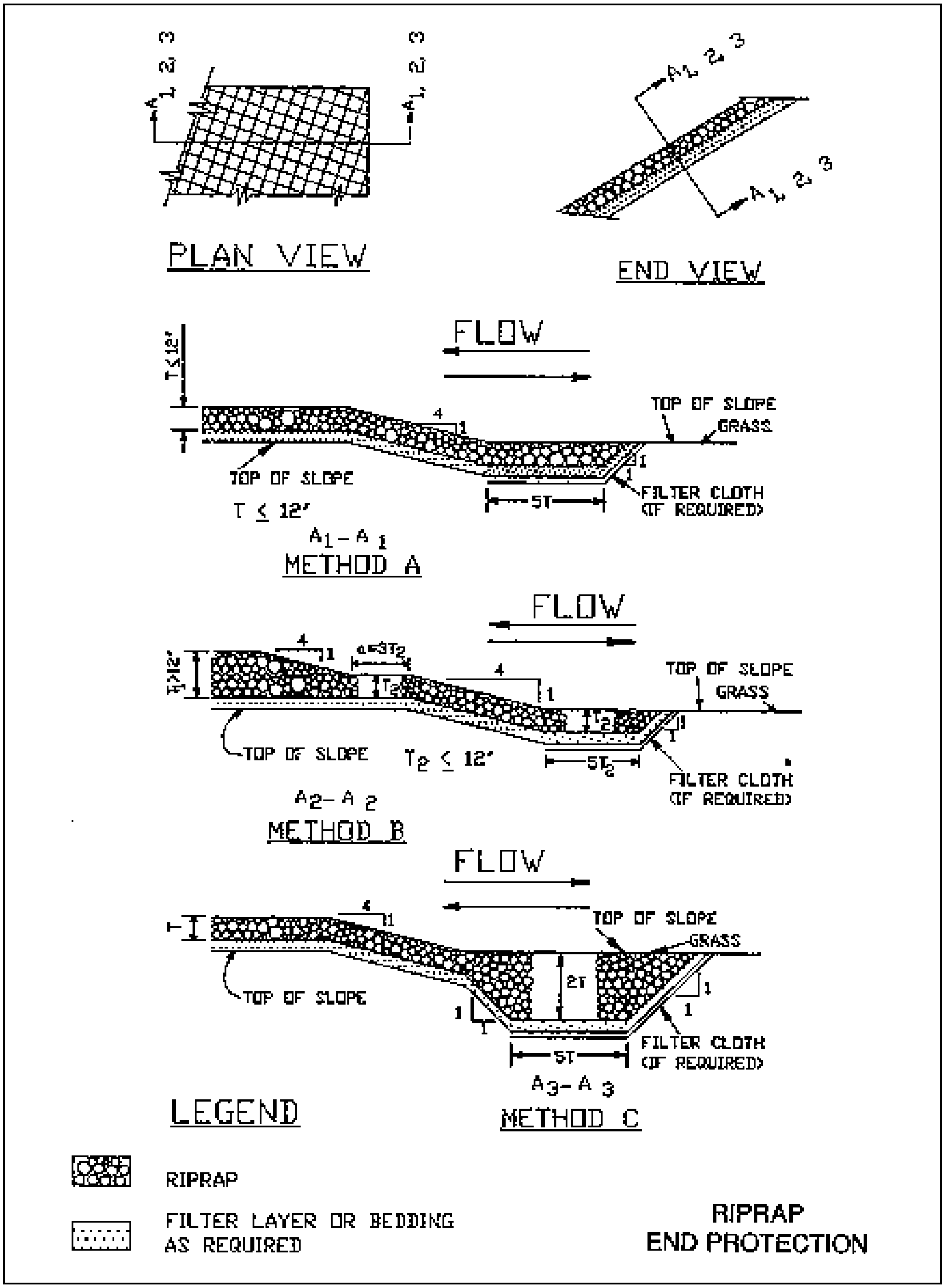
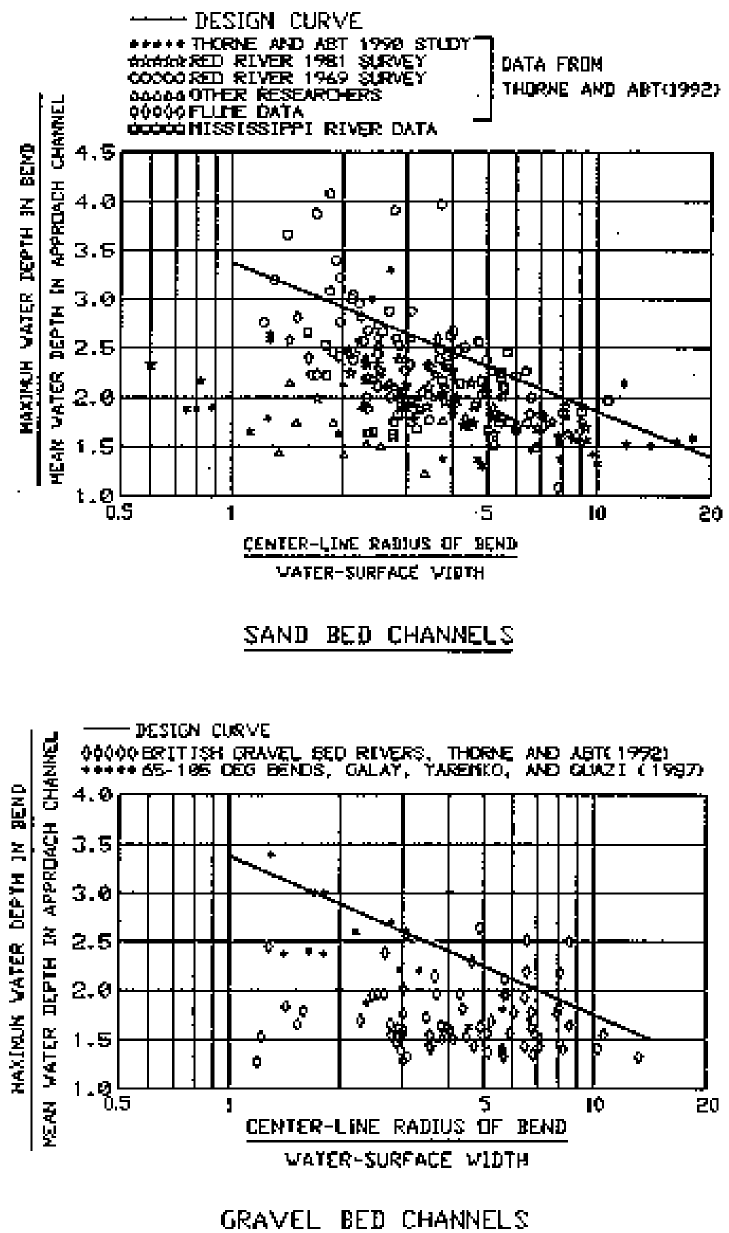

PLAN VIEW
END VIEW
FLOW
TOP OF SLOPE
GRASS
TOP OF SLOPE
T ≤ 12"
FILTER CLOTH (IF REQUIRED)
A1 - A1
METHOD A
FLOW
TOP OF SLOPE
GRASS
TOP OF SLOPE
T2 ≤ 12"
FILTER CLOTH (IF REQUIRED)
A2 - A2
METHOD B
FLOW
TOP OF SLOPE
GRASS
TOP OF SLOPE
FILTER CLOTH (IF REQUIRED)
A3 - A3
METHOD C
LEGEND
RIPRAP
FILTER LAYER OR BEDDING AS REQUIRED
RIPRAP
END PROTECTION

Plate B-41

*

*



SCOUR DEPTH IN BENDS

Plate B-41

*

*

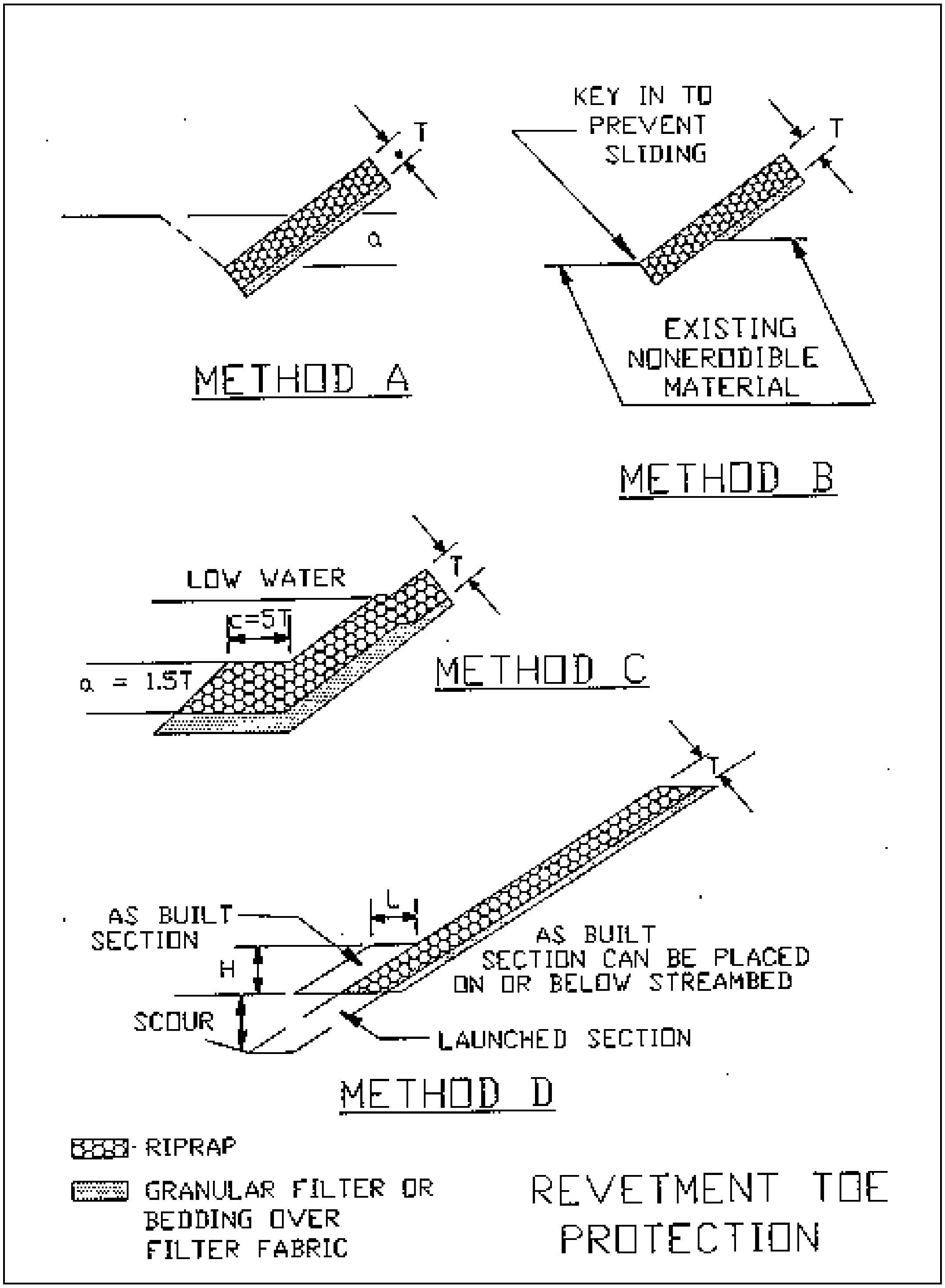
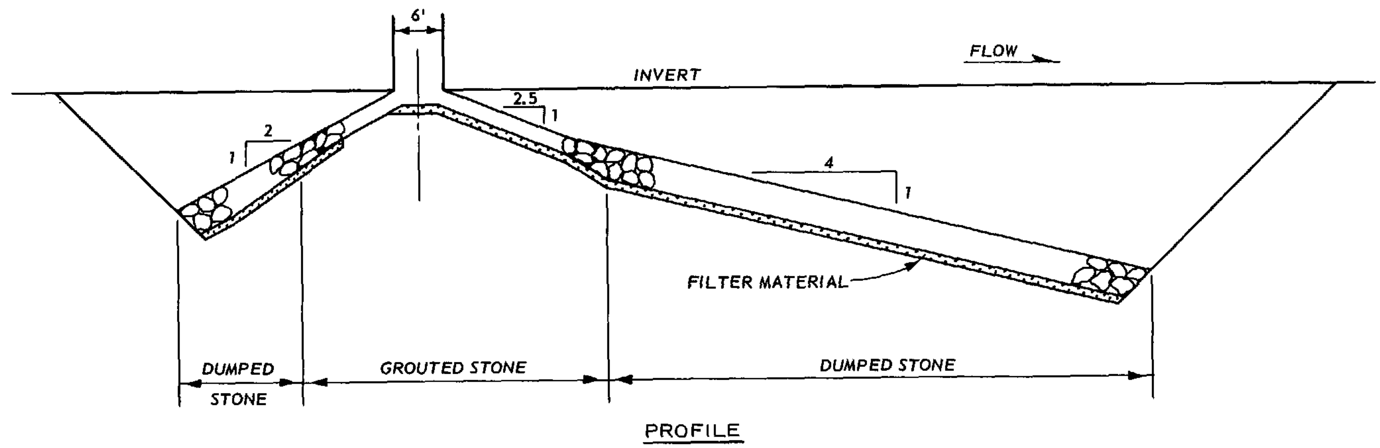

T
a
METHOD A
KEY IN TO
PREVENT
SLIDING
T
EXISTING
NONERODIBLE
MATERIAL
METHOD B
LOW WATER
T
c=5T
a = 1.5T
METHOD C
T
AS BUILT
SECTION
L
H
SCOUR
AS BUILT
SECTION CAN BE PLACED
ON OR BELOW STREAMBED
LAUNCHED SECTION
METHOD D
- RIPRAP
GRANULAR FILTER OR
BEDDING OVER
FILTER FABRIC
REVETMENT TOE
PROTECTION

Plate B-43

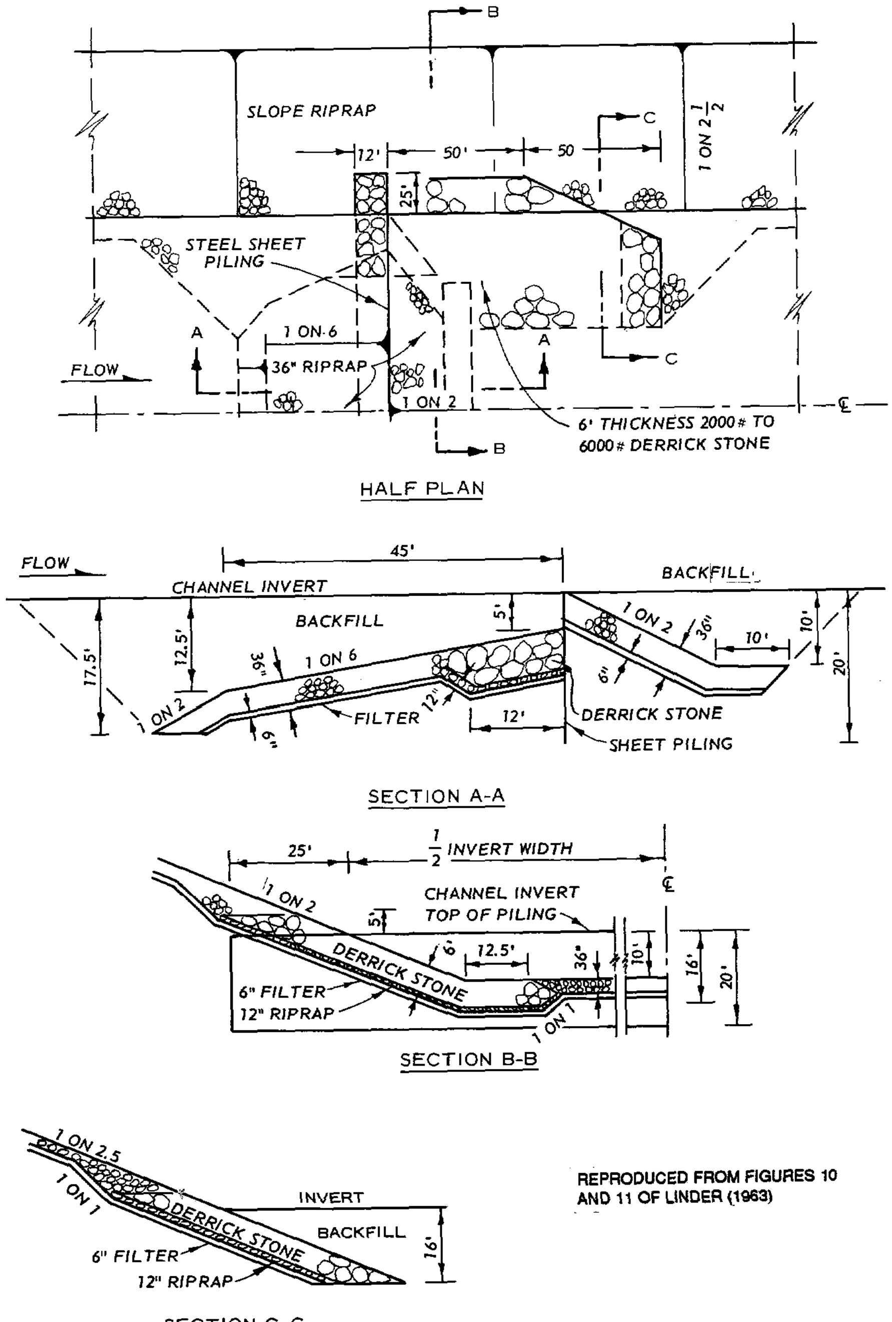
*



ROCK STABILIZER

Plate B-44

*

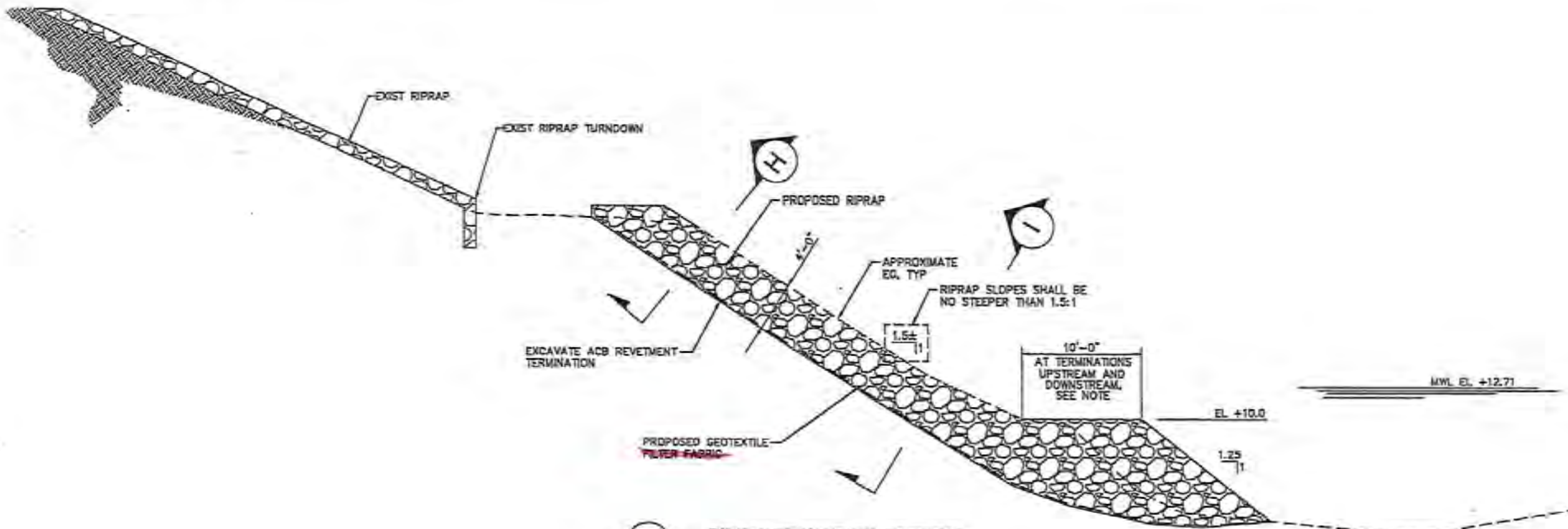


REPRODUCED FROM FIGURES 10 AND 11 OF LINDER (1963)

SHEET PILING STABILIZER

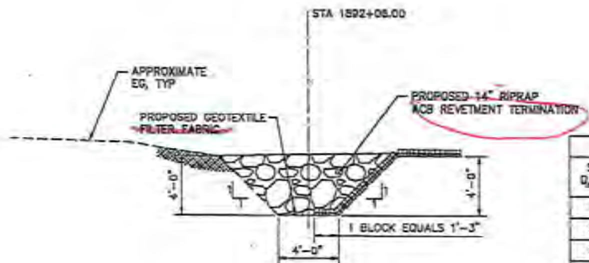
Plate B-45

*



C RIPRAP TRANSITION SECTION
SCALE: 1"=5'-0"

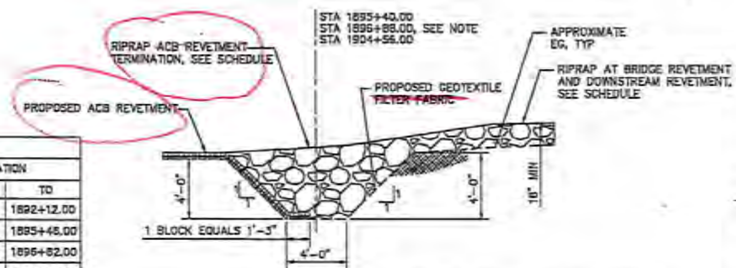
NOTE:
TYPICAL BENCH WIDTH OF 16'-0" SHALL BE MAINTAINED OVER TRANSITIONS AT BRIDGE.



H TYPICAL SECTION
SCALE: 1"=4'-0"

RIPRAP SCHEDULE			
SIZE D_{50}(IN.)	THICKNESS (IN.)	STATION	
		FROM	TO
14	48	1892+00.00	1892+12.00
20	48	1895+34.00	1895+46.00
20	18	1895+46.00	1896+82.00
20	48	1896+82.00	1896+94.00
14	48	1904+50.00	1904+62.00
14	12	1904+62.00	1904+85.00

Check w/ Design Report Text



NOTE: STA 1896+88.00 IS MIRROR IMAGE

I TYPICAL SECTION
SCALE: 1"=4'-0"

PLANS PREPARED BY:

ARCADIS

60% DESIGN DRAWINGS
CONTRACT No.: IBM15D0001
DATE: 12/14/2017

INTERNATIONAL BOUNDARY AND WATER COMMISSION
UNITED STATES SECTION
UPPER BROWNSVILLE LEVEE AND FLOODPLAIN REHABILITATION
CAMERON COUNTY, TEXAS

TYPICAL SECTIONS

DESIGNED BY: CMA
DRAWN BY: DR
CHECKED BY: LM
FILE:
RECOMMENDED:
APPROVED:

SHEET 20 OF 33

DANNENBAUM

CLIENT:	IBWC	JOB NO. 4839-01		
PROJECT:	Brownsville Levee Slope Repai	DESIGNE CMA	DATE: Dec-15	
SUBJECT:	Rip Rap Design	CHECKER: SG	DATE: Dec-17	

Rip Rap Size Calculation

Section 2 - At Bridge

(STA 1895+34 to 1896+94)

	Run A - High Water				Run B - Lower High Water			
V = depth average velocity	6.4	try	6.4	try	6.4	try	6.4	try
d = depth of flow	27	try	27	try	14	try	14	try
Sf = safety factor	2	Desired	1.1	min	2	desired	1.1	desired
R = radius of bend	150		150		150		150	
W = water surface width	165.7		165.7		140		140	
Cv = vert. velocity dist. coefficient	1.2916		1.2916		1.2770		1.2770	
Cs = stability coefficient	0.3		0.3		0.3		0.3	
Ct = thickness coefficient	1		1		1		1	
γw = unit weight of water	62.4		62.4		62.4		62.4	
γs = specific weight	155		155		155		155	
g = gravity	32.2		32.2		32.2		32.2	
K_1 = side slope factor 1:1.5	0.5052		0.5052		0.5052		0.5052	
$Cv = 1.283 - 0.2\log(R/W)$	1.2916		1.2916		1.2770		1.2770	
$K_1 = (1 - \sin^2\theta/\sin^2\phi)^{1/2}$	0.5052		0.5052		0.5052		0.5052	
$(\gamma w/\gamma s - \gamma w)^{1/2}$	0.8440		0.8440		0.8440		0.8440	
$V/(K_1 gd)^{1/2}$	0.3054		0.3054		0.4241		0.4241	
$D30(min) = SfCsCvCtd[(\gamma w/\gamma s - \gamma w)^{1/2} * (V/(K_1 gd))^{1/2}]^{2.5}$	0.7056		0.3881		0.8221		0.4522	
D100(max) per USACE =	18		12		21		12	

Per USIBWC Specs SAY D100 = 20 inches

DANNENBAUM

CLIENT:	IBWC	**JOB NO.** 4839-01	
PROJECT:	Brownsville Levee Slope Repai	**DESIGNE** CMA	DATE: Dec-15
SUBJECT:	**Rip Rap Design**	**CHECKER:** SG	DATE: Dec_17

Rip Rap Size Calculation

Section 3 - Termination Points

(STA 1892+00 and STA 1904+85)

	Run A - High Water				Run B - Lower High Water			
V = depth average velocity	5	try	5	try	5	try	5	try
d = depth of flow	27	try	27	try	14	try	14	try
Sf = safety factor	2	Desired	1.1	min	2	desired	1.1	desired
R = radius of bend	150		150		150		150	
W = water surface width	165.7		165.7		140		140	
Cv = vert. velocity dist. coefficient	1.2916		1.2916		1.2770		1.2770	
Cs = stability coefficient	0.3		0.3		0.3		0.3	
Ct = thickness coefficient	1		1		1		1	
γw = unit weight of water	62.4		62.4		62.4		62.4	
γs = specific weight	155		155		155		155	
g = gravity	32.2		32.2		32.2		32.2	
K_1 = side slope factor 1:1.5	0.5052		0.5052		0.5052		0.5052	
$Cv = 1.283 - 0.2\log(R/W)$	1.2916		1.2916		1.2770		1.2770	
$K_1 = (1 - \sin^2\theta/\sin^2\phi)^{½}$	0.5052		0.5052		0.5052		0.5052	
$(\gamma w/\gamma s - \gamma w)^{½}$	0.8440		0.8440		0.8440		0.8440	
$V/(K_1gd)^{½}$	0.2386		0.2386		0.3313		0.3313	
D30(min) = $SfCsCvCtd[(\gamma w/\gamma s - \gamma w)^{½} * (V/(K_1gd))^{½}]^{2.5}$	0.3807		0.2094		0.4435		0.2439	
D100(max) per USACE =	12 in		9 in		12 in		9 in	

Per USIBWC Specs SAY D100 = 14 inches

ATTACHMENT C

DESIGN CALCULATIONS

C.1 GEOTEXTILE
C.2 SOIL FILTER
C.3 ARTICULATED CONCRETE BLOCK MATRESS (ACBM)
C.4 ROCK RIPRAP
C.5 GATEWELL STRUCTURE 205

DESIGN NOTES AND COMPUTATIONS

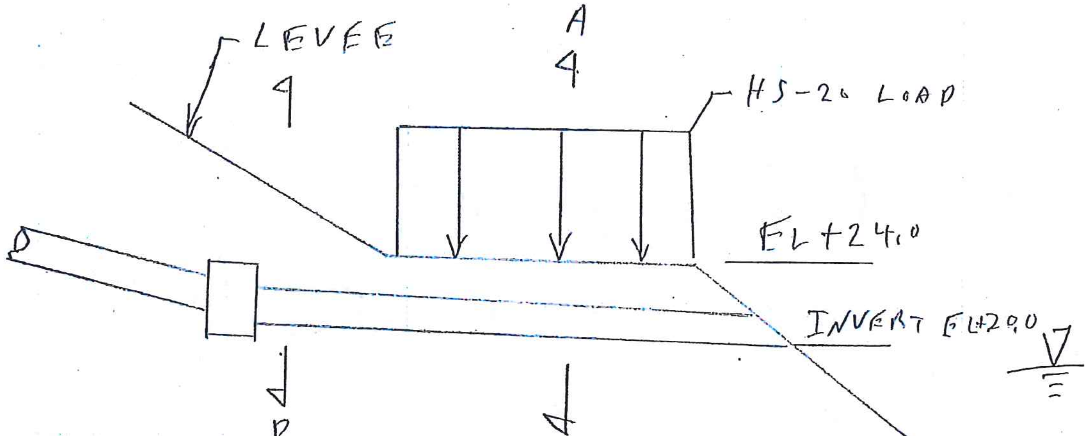
SUBJECT: BROWNSVILLE LEVEE REPAIR				JOB NUMBER 4839-01
PREPARED BY CMA	DATE 1/21/2015	CHECKED BY	APPROVED BY	
				SHEET NO. OF

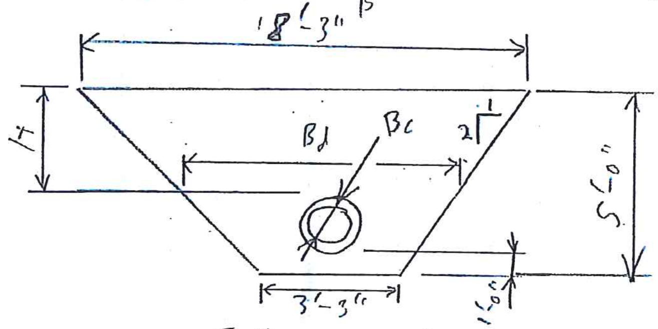
REINFORCED CONCRETE PIPE DESIGN

2 1/4 CL IV
2 CL III

SUBJECT: BROWNSVILLE LEVEE — JOB NUMBER: 4839-01
PREPARED BY: CMA — DATE: 1/21/2015 — CHECKED BY: — APPROVED BY:

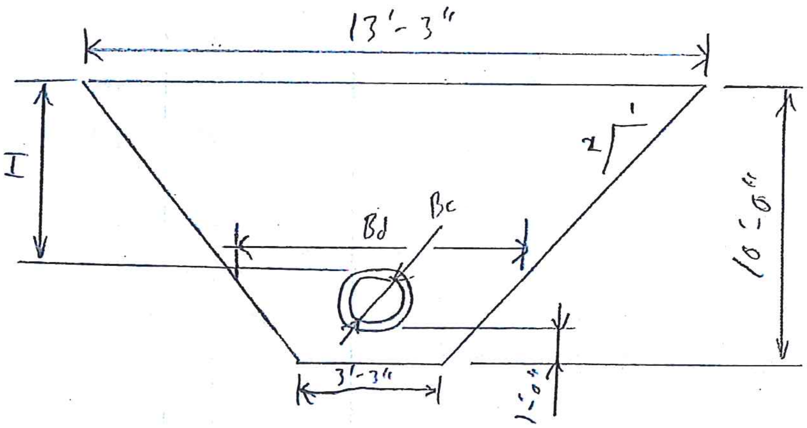
RCP PIPE DESIGN @ GATEWELL





SECTION A

$B_c = 1.625'$
TRANSITION WIDTH @ 5'
TABLE 17 A 3'-1"
∴ $W_d = (1.2)\ 1203' = 1443.6$ lb/ft
FS = 2.0
USE CLASS C BED
Figure 227 $B_f = 1.5$ 2.8



SECTION B

$B_c = 1.625'$ 10'-2½ 8'
TRANSITION WIDTH @ 8'
TABLE 17 C 3'-3"
∴ $W_d = (1.2)(1938) = 2325.6$
FS = 2.0
USE CLASS C BED
FIGURE 227 $B_f = 1.5$ 2.8

DANNENBAUM
DANNENBAUM ENGINEERING CORPORATION

SUBJECT: BROWNSVILLE LEVEE — JOB NUMBER 4839-01

PREPARED BY CMA — DATE 1/21/2018 — CHECKED BY KK — APPROVED BY

RCP PIPE DESIGN @ GATEWELL

SECTION A

HS-20 TRUCK LL

∴ TABLE 4B @ 4' − 1.625' = 2.375'

$W_L = 1080$

$D = 1.25' = \phi$ Pipe

FOR TRENCH SHALLOW

ALT $W_d = (120)(3')(1.625')$

$W_d = 463$ lb/ft

DEEP OPEN CUT CONTROLS

$$D_{0.01} = \frac{W_d + W_L}{B_f\,(D)} \times FS = 2691.8 \text{ lb/ft}$$

1442.1 lb/ft

B_f 2.8

SECTION B

No LL

FOR TRENCH SHALLOW

ALT $W_d = (120')(7')(1.625')$

$W_d = 1365$ lb/ft

DEEP OPEN CUT CONTROLS

$$D_{0.01} = \frac{W_d}{B_f\ D} FS = 2480.64$$

1328.9 lb/ft

B_f 2.8

SECTION A CONTROLS

FROM TABLE 1 ASTM C76 & C655

D-LOAD = 3000 FOR CLASS V ok

> 2691.8 lb/ft

IV OK

= 2000 > 1442.1

BLE 17

15" BACKFILL LOADS ON CIRCULAR PIPE IN TRENCH INSTALLATION 15"

*100 POUNDS PER CUBIC FOOT BACKFILL MATERIAL. LOADS IN POUNDS PER LINEAR FOOT

A

A SAND AND GRAVEL Kμ'—0.165

HEIGHT OF BACKFILL H ABOVE TOP OF PIPE, FEET	TRENCH WIDTH AT TOP OF PIPE 2'-3"	2'-6"	2'-9"	3'-0"	3'-3"	3'-6"	4'-0"	4'-6"	5'-0"	6'-0"	▲TRANSITION WIDTH
5	797	915	1033	1153	1203						3'- 1"
6	897	1036	1176	1317	1448						3'- 3"
7	984	1142	1302	1464	1628	1692					3'- 4"
8	1059	1235	1414	1596	1780	1938					3'- 5"
9	1124	1316	1513	1713	1917	2123	2183				3'- 7"
10	1180	1388	1601	1819	2041	2266	2429				3'- 8"
11	1228	1450	1679	1914	2153	2396	2672				3'- 9"
12	1270	1505	1748	1998	2254	2514	2920				3'-11"
13	1306	1553	1810	2074	2345	2622	3162				4'- 0"
14	1337	1595	1864	2142	2428	2720	3320	3408			4'- 1"
15	1364	1632	1912	2203	2502	2809	3441	3647			4'- 2"
16	1387	1664	1955	2258	2570	2890	3553	3900			4'- 3"
17	1407	1693	1993	2306	2631	2964	3655	4142			4'- 4"
18	1424	1717	2027	2350	2686	3032	3750	4382			4'- 5"
19	1439	1739	2057	2389	2735	3093	3837	4624			4'- 6"
20	1452	1758	2083	2425	2780	3148	3917	4720	4870		4'- 7"
21	1463	1775	2107	2456	2821	3199	3991	4820	5123		4'- 8"
22	1473	1790	2128	2484	2857	3245	4058	4913	5368		4'- 9"
23	1481	1802	2146	2510	2891	3287	4121	5000	5603		4'-10"
24	1488	1814	2163	2532	2920	3325	4179	5080	5851		4'-11"
25	1494	1824	2177	2552	2947	3360	4232	5155	6091		5'- 0"
26	1500	1832	2190	2571	2972	3392	4280	5224	6213	6350	5'- 1"
27	1504	1840	2201	2587	2994	3421	4325	5289	6300	6588	5'- 2"
28	1508	1846	2212	2601	3014	3447	4367	5349	6382	6833	5'- 3"
29	1512	1852	2221	2614	3032	3471	4405	5404	6458	7070	5'- 3"
30	1515	1857	2229	2626	3048	3492	4440	5456	6529	7319	5'- 4"
31	1517	1862	2236	2637	3063	3512	4472	5504	6596	7561	5'- 5"
32	1520	1866	2242	2646	3076	3530	4502	5549	6659	7818	5'- 6"
33	1521	1869	2247	2654	3088	3546	4529	5590	6717	8046	5'- 7"
34	1523	1872	2252	2662	3099	3561	4555	5629	6772	8290	5'- 8"
35	1525	1875	2257	2669	3109	3575	4578	5665	6823	8556	5'- 8"
36	1526	1877	2261	2675	3118	3587	4599	5698	6871	8789	5'- 9"
37	1527	1879	2264	2680	3126	3598	4619	5729	6916	9045	5'-10"
38	1528	1881	2267	2685	3133	3608	4637	5758	6958	9265	5'-11"
39	1529	1882	2270	2689	3139	3618	4654	5784	6998	9510	6'- 0"
40	1529	1884	2272	2693	3145	3626	4669	5809	7035	9785	6'- 0"

B SATURATED TOP SOIL Kμ'—0.150

HEIGHT OF BACKFILL H ABOVE TOP OF PIPE, FEET	TRENCH WIDTH AT TOP OF PIPE 2'-3"	2'-6"	2'-9"	3'-0"	3'-3"	3'-6"	4'-0"	4'-6"	5'-0"	6'-0"	▲TRANSITION WIDTH
5	821	939	1059	1180	1203						3'- 1"
6	929	1069	1210	1353	1448						3'- 2"
7	1023	1183	1346	1510	1692						3'- 3"
8	1106	1285	1467	1652	1838	1938					3'- 5"
9	1179	1375	1576	1780	1986	2183					3'- 6"
10	1242	1455	1674	1896	2122	2350	2429				3'- 7"
11	1298	1526	1761	2001	2245	2492	2672				3'- 8"
12	1346	1589	1840	2096	2357	2623	2920				3'- 9"
13	1389	1645	1910	2182	2460	2743	3162				3'-10"
14	1426	1695	1973	2260	2553	2853	3408				3'-11"
15	1459	1738	2030	2330	2639	2954	3647				4'- 0"
16	1487	1777	2080	2394	2716	3047	3726	3900			4'- 2"
17	1512	1812	2126	2451	2787	3132	3843	4142			4'- 2"
18	1534	1843	2167	2504	2852	3210	3950	4382			4'- 3"
19	1553	1870	2203	2551	2911	3282	4050	4624			4'- 4"
20	1570	1894	2236	2593	2965	3347	4143	4870			4'- 5"
21	1584	1915	2265	2632	3014	3408	4229	5123			4'- 6"
22	1597	1934	2292	2667	3058	3463	4309	5192	5368		4'- 7"
23	1608	1951	2315	2699	3099	3514	4383	5293	5603		4'- 8"
24	1618	1966	2336	2727	3136	3561	4451	5387	5851		4'- 9"
25	1627	1979	2355	2753	3170	3604	4515	5475	6091		4'-10"
26	1634	1991	2373	2777	3201	3643	4574	5557	6350		4'-11"
27	1641	2001	2388	2798	3229	3679	4629	5634	6588		4'-11"
28	1647	2010	2401	2817	3255	3712	4680	5706	6833		5'- 0"
29	1652	2019	2414	2834	3278	3743	4727	5773	6870	7070	5'- 1"
30	1656	2026	2425	2850	3300	3771	4771	5836	6955	7319	5'- 2"
31	1660	2032	2435	2864	3319	3796	4811	5895	7036	7561	5'- 3"
32	1663	2038	2444	2877	3337	3820	4849	5950	7111	7818	5'- 3"
33	1666	2043	2451	2889	3353	3842	4884	6002	7182	8046	5'- 4"
34	1669	2048	2459	2899	3368	3861	4916	6050	7249	8290	5'- 5"
35	1671	2052	2465	2909	3381	3880	4946	6095	7312	8556	5'- 6"
36	1673	2055	2471	2918	3393	3896	4974	6137	7372	8789	5'- 7"
37	1675	2058	2476	2925	3405	3912	5000	6177	7428	9045	5'- 7"
38	1676	2061	2480	2932	3415	3926	5024	6214	7480	9265	5'- 8"
39	1678	2064	2485	2939	3424	3939	5047	6248	7530	9510	5'- 9"
40	1679	2066	2488	2945	3433	3950	5067	6280	7577	9785	5'-10"

C

C ORDINARY CLAY Kμ'—0.130

HEIGHT OF BACKFILL H ABOVE TOP OF PIPE, FEET	TRENCH WIDTH AT TOP OF PIPE 2'-3"	2'-6"	2'-9"	3'-0"	3'-3"	3'-6"	4'-0"	4'-6"	5'-0"	6'-0"	▲TRANSITION WIDTH
5	854	974	1095	1203							3'- 0"
6	973	1115	1259	1403	1448						3'- 1"
7	1079	1243	1408	1574	1692						3'- 2"
8	1174	1357	1543	1731	1938						3'- 3"
9	1258	1461	1666	1874	2085	2183					3'- 4"
10	1334	1554	1778	2006	2237	2429					3'- 5"
11	1400	1638	1880	2127	2377	2672					3'- 6"
12	1460	1713	1973	2238	2506	2779	2920				3'- 8"
13	1513	1781	2057	2339	2626	2917	3162				3'- 8"
14	1560	1843	2134	2432	2736	3046	3408				3'- 9"
15	1603	1898	2204	2518	2838	3165	3647				3'-10"
16	1640	1948	2267	2596	2932	3276	3900				3'-11"
17	1674	1993	2325	2668	3019	3378	4142				4'- 0"
18	1703	2034	2378	2734	3099	3474	4243	4382			4'- 1"
19	1730	2070	2426	2794	3173	3562	4364	4624			4'- 2"
20	1754	2103	2469	2849	3242	3645	4476	4870			4'- 3"
21	1775	2133	2509	2900	3305	3721	4582	5123			4'- 4"
22	1793	2159	2545	2947	3363	3792	4681	5368			4'- 5"
23	1810	2184	2578	2989	3417	3858	4773	5603			4'- 5"
24	1825	2205	2607	3029	3466	3919	4860	5851			4'- 6"
25	1838	2225	2635	3064	3512	3975	4942	5951	6091		4'- 7"
26	1850	2242	2659	3097	3554	4028	5018	6054	6350		4'- 8"
27	1861	2258	2682	3128	3593	4077	5089	6151	6588		4'- 8"
28	1870	2273	2702	3155	3630	4122	5156	6243	6833		4'- 9"
29	1878	2286	2721	3181	3663	4165	5219	6330	7070		4'-10"
30	1886	2297	2738	3204	3693	4204	5278	6412	7319		4'-11"
31	1892	2308	2753	3225	3722	4240	5333	6489	7561		4'-11"
32	1898	2317	2767	3245	3748	4274	5385	6562	7818		5'- 0"
33	1904	2326	2780	3263	3772	4305	5433	6631	7886	8046	5'- 1"
34	1908	2333	2791	3279	3794	4334	5478	6696	7974	8290	5'- 1"
35	1913	2340	2802	3294	3815	4361	5521	6757	8057	8556	5'- 2"
36	1916	2346	2811	3308	3834	4386	5561	6815	8136	8789	5'- 3"
37	1920	2352	2820	3321	3851	4409	5598	6870	8211	9045	5'- 4"
38	1922	2357	2828	3333	3868	4431	5633	6921	8282	9265	5'- 4"
39	1925	2362	2835	3343	3883	4451	5666	6970	8350	9510	5'- 5"
40	1927	2366	2842	3353	3896	4470	5696	7016	8414	9785	5'- 6"

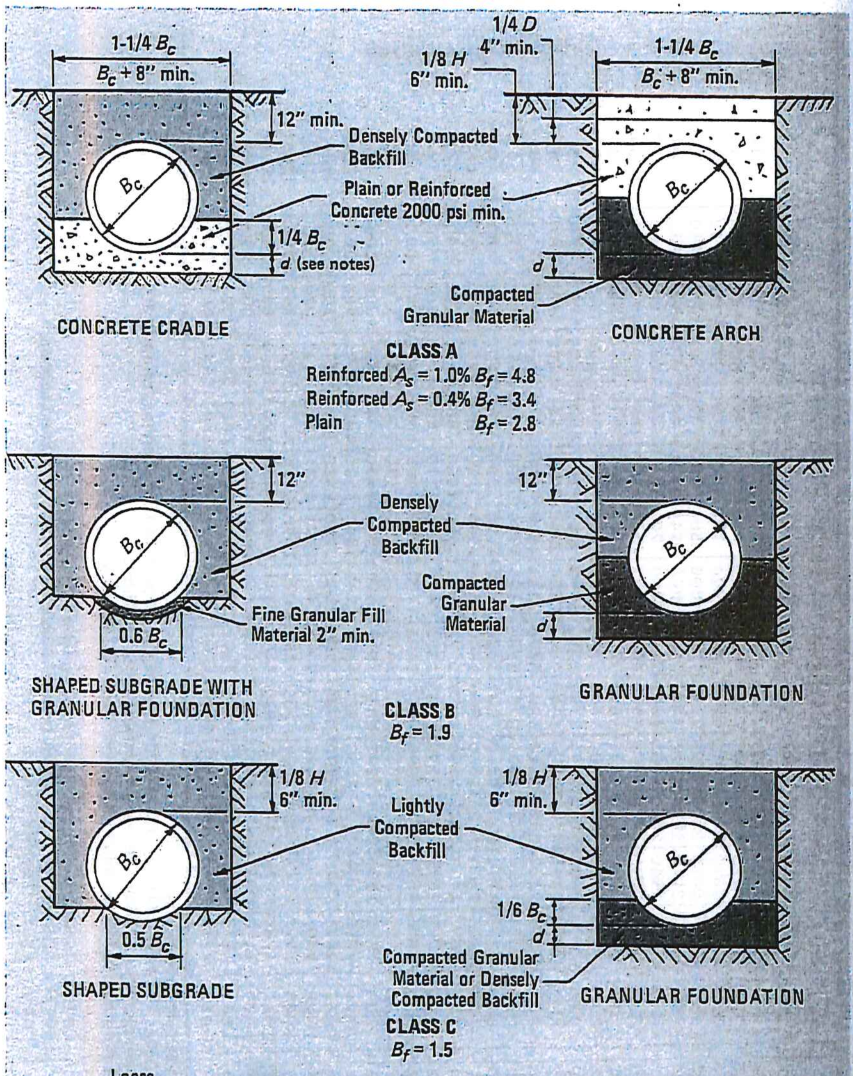
D SATURATED CLAY Kμ'—0.110

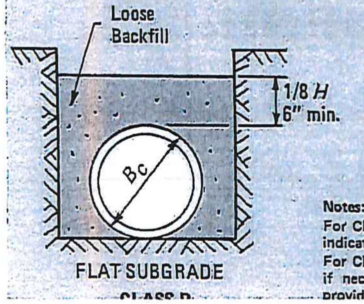
HEIGHT OF BACKFILL H ABOVE TOP OF PIPE, FEET	TRENCH WIDTH AT TOP OF PIPE 2'-3"	2'-6"	2'-9"	3'-0"	3'-3"	3'-6"	4'-0"	4'-6"	5'-0"	6'-0"	▲TRANSITION WIDTH
5	889	1011	1133	1203							2'-11"
6	1021	1165	1310	1448							3'- 0"
7	1140	1306	1473	1642	1692						3'- 1"
8	1248	1435	1624	1815	1938						3'- 2"
9	1346	1554	1764	1976	2183						3'- 3"
10	1435	1662	1892	2126	2361	2429					3'- 4"
11	1516	1761	2011	2264	2520	2672					3'- 5"
12	1589	1852	2121	2394	2670	2920					3'- 6"
13	1655	1935	2222	2514	2809	3162					3'- 6"
14	1715	2012	2315	2625	2940	3258	3408				3'- 7"
15	1770	2082	2402	2729	3061	3399	3647				3'- 8"
16	1819	2145	2481	2825	3175	3531	3900				3'- 9"
17	1864	2204	2555	2914	3282	3655	4142				3'-10"
18	1905	2258	2623	2998	3381	3772	4382				3'-11"
19	1942	2307	2685	3075	3474	3881	4624				3'-11"
20	1975	2352	2743	3147	3561	3984	4870				4'- 0"
21	2005	2393	2796	3213	3642	4080	4981	5123			4'- 1"
22	2033	2431	2846	3275	3718	4171	5104	5368			4'- 2"
23	2058	2465	2891	3333	3789	4256	5220	5603			4'- 2"
24	2080	2497	2933	3387	3855	4336	5329	5851			4'- 3"
25	2101	2526	2972	3436	3917	4411	5433	6091			4'- 4"
26	2120	2552	3008	3483	3975	4481	5532	6350			4'- 5"
27	2136	2576	3041	3526	4029	4548	5625	6588			4'- 5"
28	2152	2599	3071	3566	4079	4610	5713	6833			4'- 6"
29	2166	2619	3099	3603	4126	4668	5797	7070			4'- 6"
30	2178	2638	3125	3637	4171	4723	5876	7081	7319		4'- 7"
31	2190	2655	3149	3669	4212	4774	5950	7182	7561		4'- 8"
32	2200	2670	3171	3699	4250	4823	6021	7278	7818		4'- 8"
33	2209	2685	3192	3727	4286	4868	6088	7370	8046		4'- 9"
34	2218	2698	3211	3752	4320	4911	6151	7458	8290		4'-10"
35	2226	2710	3228	3776	4351	4951	6211	7541	8556		4'-10"
36	2233	2721	3244	3798	4381	4988	6268	7620	8789		4'-11"
37	2239	2731	3259	3819	4408	5024	6322	7696	9045		5'- 0"
38	2245	2740	3273	3838	4434	5057	6373	7768	9265		5'- 0"
39	2250	2749	3285	3856	4458	5088	6421	7836	9320	9510	5'- 1"
40	2255	2756	3297	3873	4480	5117	6466	7902	9408	9785	5'- 2"

...ackfill weighing 110 pounds per cubic foot, increase loads 10%; for 120 pounds per cubic foot, increase 20%; etc.

FIGURE 227

TRENCH BEDDINGS CIRCULAR PIPE





Depth of Bedding Material Below Pipe

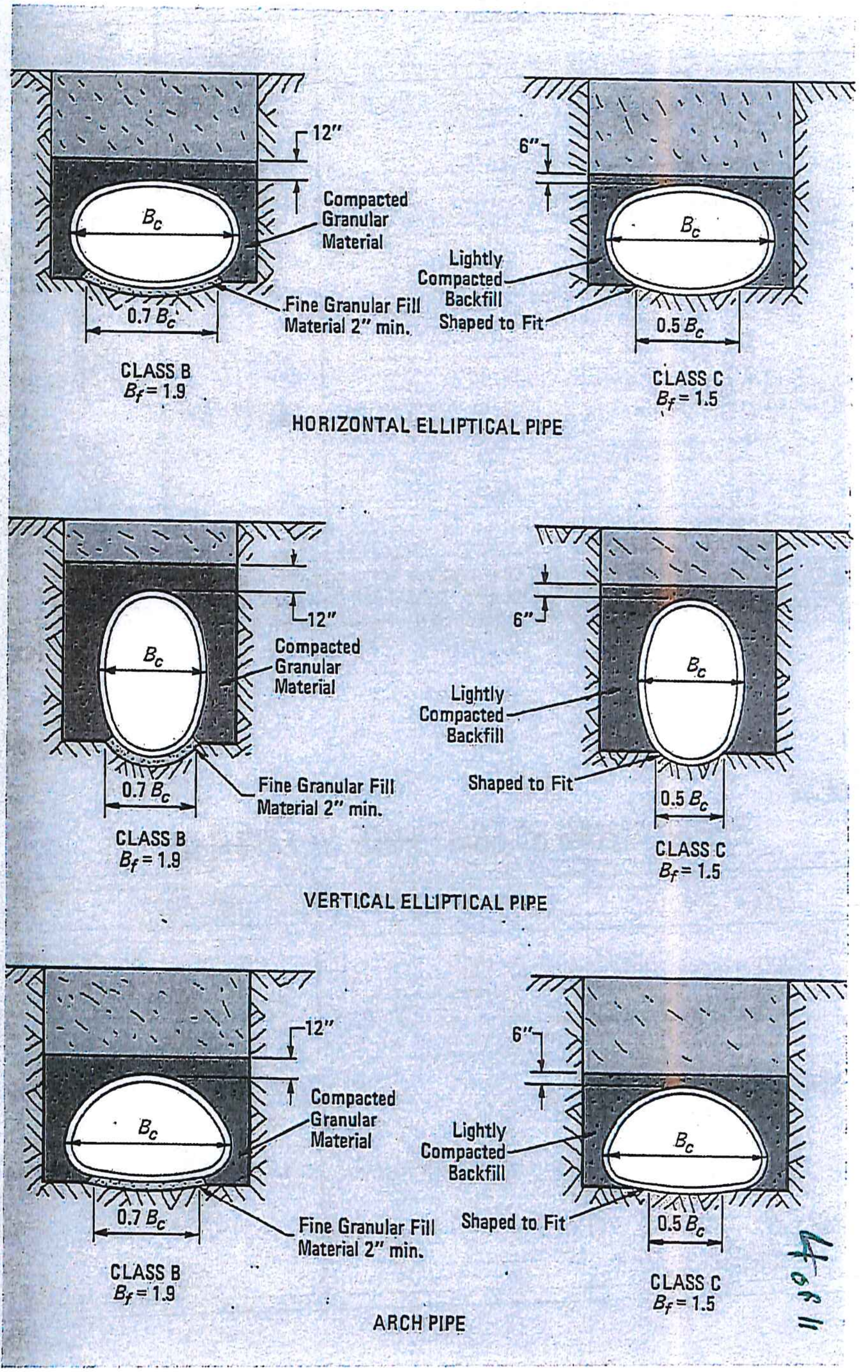
D	d (min.)
27" & smaller	3"
30" to 60"	4"
66" & larger	6"

Legend
B_c = outside diameter
H = backfill cover above top of pipe
D = inside diameter
d = depth of bedding material below pipe
A_s = area of transverse steel in the cradle of arch expressed as a percentage of area of concrete at invert or crown.

Notes:
For Class A beddings, use d as depth of concrete below pipe unless otherwise indicated by soil or design conditions.
For Class B and C beddings, subgrades should be excavated or over excavated, if necessary, so a uniform foundation free of protruding rocks may be provided.

FIGURE 228

TRENCH BEDDINGS



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ABLE 43

DESIGN VALUES OF SETTLEMENT RATIO

5F11

Installation and Foundation Condition	Settlement Ratio r_{sd} Usual Range	Settlement Ratio r_{sd} Design Value
Positive Projecting	0.0 to +1.0	
Rock or Unyielding Soil	+1.0	+1.0
*Ordinary Soil	+0.5 to +0.8	+0.7
Yielding Soil	0.0 to +0.5	+0.3
Zero Projecting		0.0
Negative Projecting	−1.0 to 0.0	
p′ = 0.5		−0.1
p′ = 1.0		−0.3
p′ = 1.5		−0.5
p′ = 2.0		−1.0
Induced Trench	−2.0 to 0.0	
p′ = 0.5		−0.5
p′ = 1.0		−0.7
p′ = 1.5		−1.0
p′ = 2.0		−2.0

value of the settlement ratio depends on the degree of compaction of the fill material adjacent to sides of the pipe. With good construction methods resulting in proper compaction of bedding and efill materials, a settlement ratio design value of +0.5 is recommended.

BLE 44

DESIGN VALUES OF COEFFICIENT OF COHESION

Type of Soil	Values of c
Clay	
Soft	40
Medium	250
Hard	1000
Sand	
Loose Dry	0
Silty	100
Dense	300
Top Soil	
Saturated	100

TABLE 45

HIGHWAY LOADS ON CIRCULAR PIPE

POUNDS PER LINEAR FOOT

PIPE SIZE D IN INCHES	B_c (ft.)	H = 0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0	8.0	9.0	PIPE SIZE D IN INCHES
		HEIGHT OF FILL H ABOVE TOP OF PIPE IN FEET													
12	1.33	3780	2080	1470	1080	760	550	450	380	290	230	190	160	130	12
15	1.63	4240	2360	1740	1280	900	660	540	450	350	280	230	190	160	15
18	1.92	4110	2610	1970	1460	1030	750	620	520	400	320	260	220	190	18
21	2.21	3920	2820	2190	1620	1150	840	690	580	450	360	300	250	210	21
24	2.50	4100	3010	2400	1780	1270	930	760	640	500	400	330	280	240	24
27	2.79	3880	2940	2590	1930	1380	1010	830	700	560	440	360	300	260	27
30	3.08	3620	2830	2770	2070	1480	1080	890	750	590	480	390	330	280	30
33	3.38	3390	2930	2950	2200	1580	1160	960	810	630	510	420	360	300	33
36	3.67	3190	2810	2930	2330	1670	1230	1020	860	670	550	450	380	330	36
39	3.96	3010	2670	2850	2440	1760	1290	1070	910	710	580	480	410	350	39
42	4.25	2860	2550	2770	2560	1840	1360	1130	950	750	610	510	430	370	42
48	4.83	2590	2330	2620	2480	1990	1470	1230	1040	820	670	560	470	410	48
54	5.42	2360	2150	2490	2360	2050	1580	1320	1120	890	730	610	520	440	54
60	6.00	2170	1990	2450	2250	1960	1680	1400	1190	950	780	650	560	480	60
66	6.58	2010	1850	2520	2160	1880	1640	1480	1260	1010	830	700	590	510	66
72	7.17	1870	1730	2580	2190	1810	1570	1510	1330	1060	880	740	630	540	72
78	7.75	1750	1630	2630	2240	1770	1520	1460	1390	1110	920	780	660	570	78
84	8.33	1650	1540	2730	2290	1810	1460	1410	1360	1160	960	810	690	600	84
90	8.92	1550	1460	2530	2330	1850	1470	1360	1310	1210	1000	850	720	630	90
96	9.50	1470	1380	2410	2290	1880	1500	1330	1270	1250	1040	880	750	650	96
102	10.08	1390	1320	2300	2190	1910	1530	1350	1240	1290	1070	910	780	680	102
108	10.67	1320	1260	2200	2090	1830	1560	1380	1230	1330	1110	940	810	700	108
114	11.25	1260	1200	2110	2010	1760	1540	1410	1260	1362	1140	970	830	730	114
120	11.83	1210	1150	2020	1930	1700	1480	1420	1280	1400	1170	990	860	750	120
126	12.42	1160	1100	1940	1860	1640	1430	1380	1300	1430	1200	1020	880	770	126
132	13.00	1110	1060	1870	1800	1580	1380	1330	1290	1460	1220	1040	900	790	132
138	13.58	1070	1020	1800	1730	1530	1340	1290	1250	1490	1250	1070	920	810	138
144	14.17	1020	980	1740	1670	1480	1300	1250	1210	1470	1280	1090	940	830	144

DATA:
1. Unsurfaced roadway.
2. Loads — AASHTO HS 20, two 16,000 lb dual-tired wheels, 4 ft. on centers, or alternate loading, four 12,000 lb. dual-tired wheels, 4 ft. on centers.

NOTES:
1. Interpolate for intermediate pipe sizes and/or fill heights.
2. Critical loads:
   a. For H = 0.5 and 1.0 ft., a single 16,000 lb. dual-tired wheel.
   b. For H = 1.5 through 4.0 ft., two 16,000 lb. dual-tired wheels, 4 ft. on centers.
   c. For H > 4.0 ft. alternate loading.
3. Truck live loads for H = 10.0 ft. or more are insignificant.

TEX DOT

ITEM 464

REINFORCED CONCRETE PIPE

464.1. Description. Furnish and install reinforced concrete pipe, materials for precast concrete pipe culverts, or precast concrete storm drain mains, laterals, stubs, and inlet leads.

464.2. Materials.

A. Fabrication. Provide precast reinforced concrete pipe that conforms to the design shown on the plans and to the following:

- ASTM C 76 or ASTM C 655 unless otherwise shown on the plans for circular pipe, or
- ASTM C 506 for arch pipe, or
- ASTM C 507 for horizontal elliptical pipe.

Provide precast concrete pipe that is machine-made or cast by a process that will provide for uniform placement of the concrete in the form and compaction by mechanical devices that will assure a dense concrete. Mix concrete in a central batch plant or other approved batching facility where the quality and uniformity of the concrete is assured. Do not use transit-mixed concrete for precast concrete pipe. When sulfate-resistant concrete is required, do not use Class C fly ash.

Do not place more than 2 holes for lifting and placing in the top section of precast pipe. Cast, cut, or drill the lifting holes in the wall of the pipe. The maximum hole diameter is 3 in. at the inside surface of the pipe wall and 4 in. at the outside surface. Do not cut more than 1 longitudinal wire or 2 circumferential wires per layer of reinforcing steel when locating lift holes.

B. Design.

1. **General.** The class and D-load equivalents are shown in Table 1. Furnish arch pipe in accordance with ASTM C 506 and the dimensions shown in Table 2. Furnish horizontal elliptical pipe in accordance with ASTM C 507 and the dimensions shown in Table 3. For arch pipe and horizontal elliptical pipe the minimum height of cover required is 1 ft.

Table 1
Circular Pipe
ASTM C 76 & ASTM C 655

Class	D-Load
I	800
II	1,000
III	1,350
IV	2,000
V	3,000

Table 2
Arch Pipe

Design Size	Equivalent Diameter (in.)	Rise (in.)	Span (in.)
1	18	13-1/2	22
2	21	15-1/2	26
3	24	18	28-1/2
4	60	22-1/2	36-1/4
5	36	26-5/8	43-3/4
6	42	31-5/16	51-1/8
7	48	36	58-1/2
8	54	40	65
9	60	45	73
10	72	54	88

Table 3
Horizontal Elliptical Pipe

Design Size	Equivalent Diameter (in.)	Rise (in.)	Span (in.)
1	18	14	23
2	24	19	30
3	27	22	34
4	30	24	38
5	33	27	42
6	36	29	45
7	39	32	49
8	42	34	53
9	48	38	60
10	54	43	68

2. **Jacking, Boring, or Tunneling.** Design pipe for jacking, boring, or tunneling considering the specific installation conditions such as the soil conditions, installation methods, anticipated deflection angles, and jacking stresses. When requested, provide design notes and drawings signed and sealed by a Texas licensed professional engineer.

C. **Physical Test Requirements.** Acceptance of the pipe will be determined by the results of the following tests:

- material tests required in ASTM C 76, C 655, C 506, or C 507,
- absorption tests in accordance with ASTM C 497,
- three-edge bearing tests in accordance with ASTM C 497 (Perform 3-edge bearing tests on 1 pipe for each 300 pipes or fraction thereof for each design or shape, size, class, or D-load produced within 30 calendar days. Test for the load to produce a 0.01-in. crack or 15% in excess of the required D-load, whichever is less. Test the pipe to ultimate load if so directed. Tested pipe that satisfies the requirements of Section 464.2.F., "Causes for Rejection," may be used for construction. As an alternate to the 3-edge bearing test, concrete pipe 54 in. in diameter and larger may be accepted on the basis of compressive strength of cores cut from the wall of the pipe. The

Find: The required pipe strength in terms of 0.01-inch crack *D*-load.

Solution: 1. *Determination of Earth Load* (W_E)

A settlement ratio must first be assumed. In Table 43 values of settlement ratio from +0.5 to +0.8 are given for positive projecting installations on a foundation of ordinary soil. A value of 0.5 will be used. The product of the settlement ratio and the projection ratio will be 0.35 ($r_{sd}\, p \approx 0.3$).

Enter Figure 170 on the horizontal scale at $H = 50$ feet. Proceed vertically until the line representing $R \times S = 76'' \times 48''$ is intersected. At this point the vertical scale shows the fill load to be 37,100 pounds per linear foot for 100 pounds per cubic foot fill material. Increase the load 20 percent for 120 pound material.

$$W_c = 1.20 \times 37{,}100$$

$$W_c = 44{,}520 \text{ pounds per linear foot}$$

2. *Determination of Live Load* (W_L)

From Table 47, live load is negligible at a depth of 50 feet.

3. *Selection of Bedding*

A Class B bedding will be assumed for this example.

4. *Determination of Bedding Factor* (B_f)

From Table 63, for an H/B_c ratio of 9.84 $r_{sd}p$ value of 0.3, p value of 0.7 and a Class B bedding, a bedding factor of 2.80 is obtained.

5. *Application of Factor of Safety (F.S.)*

A factor of safety of 1.0 based on the 0.01-inch crack will be applied.

6. *Selection of Pipe Strength*

The *D*-load is given by equation 34:

$$D_{0.01} = \frac{W_L + W_E}{B_f \times S} \times F.S.$$

$$W_L + W_E = W_c = 44{,}520 \text{ pounds per linear foot}$$

$$D_{0.01} = \frac{44{,}520}{2.80 \times 4.0} \times 1.0$$

$$D_{0.01} = 3{,}975 \text{ pounds per linear foot per foot of inside horizontal span}$$

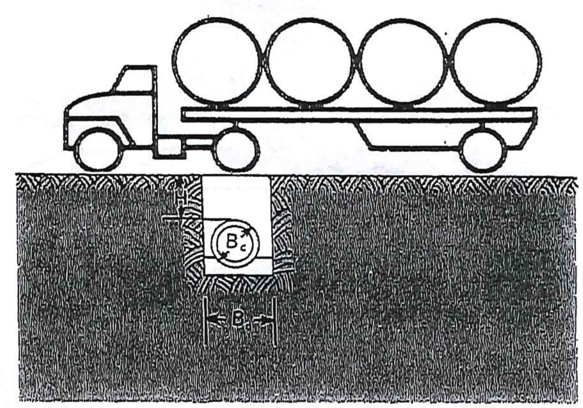
Answer: A pipe which would withstand a minimum three-edge bearing test load for the 0.01-inch crack of 3,975 pounds per linear foot per foot of inside horizontal span would be required.

EXAMPLE 4 - 8

HIGHWAY LIVE LOAD

Given: A 12-inch circular pipe is to be installed in a trench under an unsurfaced roadway and covered with 1.0 foot of 120 pounds per cubic foot backfill material.

Example 4 - 8



Find: The required pipe strength in terms of 0.01-inch crack *D*-load.

Solution: 1. *Determination of Earth Load* (W_E)

For pipe installed with less than 3 feet of cover, it is sufficiently accurate to calculate the backfill or fill load as being equal to the weight of the prism of earth on top of the pipe.

$$W_d = wHB_c$$

$$W_d = 120 \times 1.0 \times 1.33$$

$$W_d = 160 \text{ pounds per linear foot}$$

2. *Determination of Live Load* (W_L)

Since the pipe is being installed under an unsurfaced roadway with shallow cover, a truck loading based on legal load limitations should be evaluated. From Table 45, for $D = 12$ inches, $H = 1.0$ foot and AASHTO loading a live load of 2,080 pounds per linear foot is obtained. This live load value includes impact.

3. *Selection of Bedding*

A Class C bedding will be assumed for this example.

4. *Determination of Bedding Factor* (B_f)

From Figure 227, for circular pipe installed on a Class C bedding, a bedding factor of 1.5 is obtained.

EXAMPLE

No. 8

5. *Application of Factor of Safety (F.S.)*

A factor of safety of 1.0 based on the 0.01-inch crack will be applied.

6. *Selection of Pipe Strength*

The *D*-load is given by equation 33:

$$D_{0.01} = \frac{W_L + W_E}{B_f \times D} \times F.S.$$

$$D_{0.01} = \frac{2{,}080 + 160}{1.5 \times 1.0} \times 1.0$$

$D_{0.01} = 1{,}443$ pounds per linear foot
per foot of inside diameter

nswer: A pipe which would withstand a minimum three-edge bearing test load for the 0.01-inch crack of 1,443 pounds per linear foot per foot of inside diameter would be required.

---

iven: All data will remain the same as above except that the pipe size will be increased from 12-inch circular pipe to 48-inch circular pipe.

nd: The required pipe strength in terms of the 0.01-inch crack *D*-load.

lution: 1. *Determination of Earth Load* (W_E)

$$W_d = wHB_c$$

$$W_d = 120 \times 1.0 \times 4.83$$

$W_d = 580$ pounds per linear foot

2. *Determination of Live Load* (W_L)

From Table 45, for $D = 48$ inches, $H = 1.0$ foot and AASHTO loading a live load of 2,330 pounds per linear foot is obtained. This live load value includes impact.

3. *Selection of Bedding*

A Class C bedding will be assumed for this example.

4. *Determination of Bedding Factor* (B_f)

From Figure 227, for circular pipe installed on a Class C bedding, a bedding factor of 1.5 is obtained.

5. *Application of Factor of Safety (F.S.)*

A factor of safety of 1.0 based on the 0.01-inch crack will be applied.

6. *Selection of Pipe Strength*

The *D*-load is given by equation 33:

$$D_{0.01} = \frac{W_L + W_E}{B_f \times D} \times F.S.$$

$$D_{0.01} = \frac{2{,}330 + 580}{1.5 \times 4} \times 1.0$$

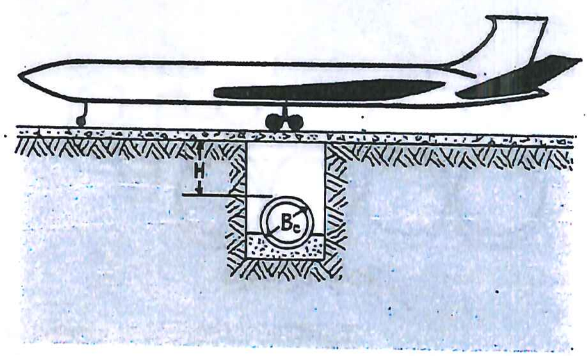
$D_{0.01} = 485$ pounds per linear foot
per foot of inside diameter

Answer: A pipe which would withstand a minimum three-edge bearing test load for the 0.01-inch crack of 485 pounds per linear foot per foot of inside diameter would be required.

Example 4 - 9
Aircraft Live Load — Rigid Pavement

Given: A 12-inch circular pipe is to be installed in a trench condition under a 12-inch thick concrete airfield pavement and subjected to heavy commercial aircraft loadings. The pipe will be covered with 1.0 foot (measured from top of pipe to bottom of pavement slab) of sand and gravel material weighing 120 pounds per cubic foot.

Example 4 - 9



Find: The required pipe strength in terms of 0.01-inch crack *D*-load.

EXAMPLE

10 of 11

Concrete Pipe Backfill Heights[i]

Round Pipe

	Pipe Class						
Pipe Size	Class I	Class II	Class III	Class IV	Class IV Alternate	Class V	Class V Alternate
(inches)	Round Reinforced Concrete Pipe Backfill Heights (feet)						
12			1-16	16-24		24-35	
15			1-16	16-24		24-35	
18			1-16	16-24		24-35	
21			1-16	16-24		24-35	
24			1-16	16-24		24-35	
27		3-11	1-3, 11-16	16-24		24-35	
30		3-11	1-3, 11-16	16-24		24-35	
33		3-11	1-3, 11-16	16-24		24-35	
36		3-11	1-3, 11-16	16-24		24-35	
42		3-11	1-3, 11-16	16-24		24-35	
48		3-11	1-3, 11-16	16-24		24-35	24-35
54		3-11	1-3, 11-16	16-24		24-35	24-35
60	6-9	3-11	1-3, 11-16	16-24	16-24	24-35	24-35
66	6-9	3-11	1-3, 11-16	16-24	16-24	24-35	24-35
72	6-9	3-11	1-3, 11-16	16-24	16-24	24-35	24-35
78	6-9	3-11	1-3, 11-16	16-24	16-24		24-35
84	6-9	3-11	1-3, 11-16	16-24	16-24		24-35
90	6-9	3-11	1-3, 11-16		16-24		24-35
96	6-9	3-11	1-3, 11-16		16-24		24-35
102	6-9	3-11	1-3, 11-16		16-24		24-35
108	6-9	3-11	1-3, 11-16		16-24		24-35

Backfill heights for Sewer Trench Conditions other than Class I				
Pipe Size	Pipe class			
	Class II	Class III	Class IV	Class V
	FT			
2" thru 54"	3-9	1-3, 9-13	13-23	23+
60" thru 108"	1-9	9-13	13-23	23+

[i] The Table is based on the following criteria:

1. Minimum cover shall be 12"
2. Minimum cover for unpaved roadways is from the top of gravel surfacing.
3. Minimum cover for paved roadways is:
   a) To the top of the base for asphalt surfaces
   b) To the top of the pavement for concrete surfaces