www.sjra.net

ADDENDUM NO. 1
RFCSP # 18-0036

Date: June 14, 2018

To: All Interested Parties

From: Grady Garrow, CPPB
Buyer

Re: **RFCSP No. 18-0036 Siphon 7 Improvements – Phase 1**

The following additions, deletions, changes or clarifications to RFCSP No. 18-0036 are hereby made a part of the originally issued documents for the above referenced project as fully and as completely as though the same were included therein.

RFCSP #18-0036

General

1. **Reference documents are attached to this addendum as Exhibit A for the purpose of information only per Technical Specification Section 00 31 19 – EXISTING CONDITION INFORMATION. Such reports and supplemental information are not part of the Contract Documents.**

2. **Proposers to submit an electronic copy of their proposal using the Excel template file titled: *CSP 18-0036 Siphon 7 Proposal Form.xls* that will be available for download on the SJRA website and Brazos Valley bidding site. See modifications regarding the submission of the file under the Technical Specifications and Drawings section of this addendum.**

Questions

1. **Is there an estimated budget for the project?**
   Answer: The Engineer's Opinion of Probable Construction Cost is $1,667,050.00 per Specification Section 00 21 13.02 – INSTRUCTIONS TO OFFERORS, Section 8.2. The estimate is based on Proposal Option 1 only.

2. **I would like to formally request that Quadex GeoKrete Geoploymer be an approved product for the geopolymer pipe lining scope for this project.**
   Answer: Product substitutes will be considered and all substitutes and "Approved Equal" items should follow the submittal process as stated in Specification 00 72 00 - GENERAL CONDITIONS, Section 6.02.5.

Technical Specifications and Drawings

1. **Section 00 21 13.03 – INSTRUCTIONS TO OFFERORS- Add the following to the last sentence in Section 21.1 "and completed Excel template file titled *CSP 18-0036 Siphon 7 Proposal Form.xls*."**

2. **Section 00 21 13.03 – INSTRUCTIONS TO OFFERORS- Add the following to the last sentence in Section 21.4.10 "and completed Excel template file titled *CSP 18-0036 Siphon 7 Proposal Form.xls*."**

3. **Section 00 41 00.02 – PROPOSAL FORM - Add the following to the end of the last bullet item in Section F. Proposal Supplements: "and completed Excel template file titled *CSP 18-0036 Siphon 7 Proposal Form.xls*."**

All provisions which are not so amended or supplemented remain in full force and effect.

Please acknowledge receipt of this addendum with signature and date and return with completed Proposal/Quotation. Failure to do so may cause your Proposal to be considered non-responsive.

Receipt of this Addendum No. 1 is hereby acknowledged

____

Authorized Signature ***Date***

____

Company Name

EXHIBIT A

SIPHON 7 IMPROVEMENTS – PHASE 1

SUPPLEMENTAL INFORMATION PACKET

REFERENCE DOCUMENTS INCLUDED:

- **TEMPORARY CONSTRUCTION EASEMENT & SIDE LETTER AGREEMENT**
- **2015 GEOTECHNICAL INVESTIGATION REPORT BY AVILES ENGINEERING CORP.**
- **2018 PIPE INSPECTION TECHNICAL MEMORANDUM BY V&A CONSULTING ENGINEERS**
- **2018 FLOW TEST TECHNICAL MEMORANDUM BY TEXAS WATER ENGINEERING, PLLC**
- **EXCERPT FROM 2008 DIVE INSPECTION REPORT BY INTERNATIONAL DIVING SERVICES, LLC**

TEMPORARY CONSTRUCTION EASMEENT & SIDE LETTER AGREEMENT

TEMPORARY CONSTRUCTION EASEMENT AGREEMENT

NOTICE OF CONFIDENTIALITY RIGHTS: IF YOU ARE A NATURAL PERSON, YOU MAY REMOVE OR STRIKE ANY OR ALL OF THE FOLLOWING INFORMATION FROM ANY INSTRUMENT THAT TRANSFERS AN INTEREST IN REAL PROPERTY BEFORE IT IS FILED FOR RECORD IN THE PUBLIC RECORDS: YOUR SOCIAL SECURITY NUMBER OR YOUR DRIVER'S LICENSE NUMBER.

THE STATE OF TEXAS §
§ KNOW ALL PERSONS BY THESE PRESENTS:
COUNTY OF HARRIS §

THAT FOR AND IN CONSIDERATION of the sum of Ten and No/100 Dollars ($10.00) and other good and valuable consideration, the receipt and sufficiency of which are hereby acknowledged, **MWV INVESTMENTS, LLC,** a Texas limited liability company (being referred to herein as "Grantor," whether one or more) has GRANTED, SOLD, and CONVEYED, and by these presents does GRANT, SELL, and CONVEY, unto the **SAN JACINTO RIVER AUTHORITY**, a governmental agency and body politic and corporate operating in Harris and Montgomery Counties, Texas (the "Authority"), and having a mailing address of P.O. Box 329, Conroe, Texas 77305, Attention: General Manager, its successors and assigns, a temporary construction easement over, on, across, and under the parcel of land described in Exhibit A (the "Work Easement"), for the purpose of providing a work area for the Authority, its agents, contractors, subcontractors, and its and their employees for the construction, installation, inspection, maintenance, and repair of the Authority's canal facilities and related appurtenances, improvements, and equipment, including, without limitation, (a) canals, siphon structures, and control gates, (b) levees and associated lateral and subjacent support for existing and future levees, (c) access improvements and road(s) for vehicles and equipment, (d) metering, measurement, testing, communication, telecommunication, and telemetry structures and facilities, (e) ditches, culverts, and related works for the control and diversion of drainage, and (f) fencing for the control of access to and along the Work Easement (collectively, the "Canal Facilities").

TO HAVE AND TO HOLD the rights, privileges and authority hereby granted unto the Authority, its successors and assigns, forever, and Grantor does hereby agree to warrant and defend said Work Easement unto the Authority, its successors and assigns, against every person whomsoever lawfully claiming or to claim the same or any part thereof by, through or under Grantor, but not otherwise. This Temporary Construction Easement Agreement (this "Agreement") and all of its terms, provisions and obligations shall be covenants running with the land affected thereby and shall inure to the benefit of and be binding upon Grantor and the Authority and their respective successors and assigns.

The Work Easement is subject to the following terms and conditions:

1. Grantor shall not do, or permit to be done, by act or omission, anything that interferes with the Authority's use of the Work Easement for the purposes described above.

2. The Authority hereby agrees to restore the Work Easement to as near the original condition as is reasonably practicable.

3. The Authority's rights in and to the Work Easement, and its right to use the same, shall expire upon completion of construction or twenty-four (24) months from the date on of execution hereof, whichever occurs first.

The foregoing terms, conditions, and provisions shall extend to and be binding upon the heirs, executors, administrators, successors, and assigns, as applicable, of Grantor and the Authority. The rights granted to the Authority (and the obligations of the Authority hereunder) may be assigned in whole or in part by the Authority only to a state agency or another political subdivision of the State of Texas that will operate and maintain the Canal Facilities for the transportation of water.

Grantor warrants that Grantor owns the land subject to the Work Easement in fee simple, that Grantor has the right, title, and power to convey the rights granted in this Agreement, and that Grantor shall execute any further assurance of title reasonably requested by the Authority, its successors or assigns.

This Agreement shall be governed and construed in accordance with the laws of the State of Texas. In case any one or more of the provisions contained in this Agreement shall for any reason be held to be invalid, illegal or unenforceable in any respect, such invalidity, illegality or unenforceability shall not affect any other provision hereof, and this Agreement shall be construed as if such invalid, illegal or unenforceable provision had never been contained herein.

This Agreement contains the entire agreement between the parties relating to the rights herein granted and the obligations herein assumed. Any or all amendments or modifications concerning this Agreement shall be of no force and effect, unless such subsequent amendment or modification is in writing and signed by all of the parties hereto or their successors and assigns.

It shall be conclusively presumed that persons signing on behalf of Grantor and the Authority have all requisite power and authority to enter into this Agreement. The execution and delivery of this Agreement by Grantor has been duly authorized by all necessary parties.

This Agreement may be executed in multiple counterpart originals which, when taken together, shall constitute one and the same instrument.

[Signature Pages Follow]

IN WITNESS WHEREOF, Grantor has hereunto set his hand this ___ day of _____________, 2018.

MWV INVESTMENTS, LLC, a
Texas limited liability company

By: ________________________________
Chad D. Vincent, Managing Member

ACKNOWLEDGEMENT

THE STATE OF TEXAS §
§
COUNTY OF _________ §

This instrument was acknowledged before me on this _____ day of ____________, 2018, by Chad D. Vincent, Managing Member of **MWV INVESTMENTS, LLC**, a Texas limited liability company, on behalf of said limited liability company.

____
Notary Public in and for the
State of Texas

(SEAL)

SAN JACINTO RIVER AUTHORITY

By: ______________________________
Jace A. Houston, General Manager

ACKNOWLEDGEMENT

THE STATE OF TEXAS §
§
COUNTY OF MONTGOMERY §

This instrument was acknowledged before me on this _____ day of ____________, 2018, by Jace A. Houston, General Manager of **SAN JACINTO RIVER AUTHORITY**, a governmental agency and body politic and corporate, on behalf of said governmental agency.

____
Notary Public in and for the
State of Texas

(SEAL)

METES AND BOUNDS DESCRIPTION
OF 0.0793 ACRES
(SJRA Parcel No. MC16-02)

Being a 0.0793 of an acre (3,456 square feet) tract of land in the L.A. Levy Survey, Abstract Number 517, situated in Harris County, Texas and being a portion of that certain called 2.0490 acre tract of land conveyed from Michael Machala and Thomas Ray Machala to MWV Investments, LLC by deed dated August 22, 2016 and recorded under Harris County Clerks' (H.C.C.F.) No. 2016-380295 of the Official Public Records of Real Property Harris County, Texas (O.P.R.O.R.P.H.C.T.); said 0.0793 of an acre more particularly described by metes and bounds as follows:

COMMENCING at a found 5/8-inch iron rod in the southerly line of that certain called 7.03 acre tract (designated as Parcel E-6) of land conveyed from the Lars Nelson Estate to the United States of America by deed dated September 13, 1943 and recorded under Volume 1298, Page 49 of the Deed Records of Harris County, Texas, said 7.06 acre tract quitclaimed to the San Jacinto River Conservation and Reclamation District of Texas by deed dated April 17, 1945 and recorded under Volume 1163, Page 576 of said Deed Records, being the northwesterly corner of that certain called 2.872 acre tract of land conveyed from Stanley Machala and wife, Clara Machala to the County of Harris by deed dated May 20, 1997 and recorded under H.C.C.F. No. S623571, Film Code No. 514-65-3604 of said Official Public Records, same being the northeasterly corner of said 2.0490 acre tract;

THENCE, South 78°08'14" West, 382.87 feet along the southerly line of said 7.03 acre tract, being the northerly line of said 2.0490 acre tract to a set 5/8-inch iron rod with "GeoSolutions" cap for the **POINT OF BEGINNING** and the northeasterly corner of the herein described parcel;

THENCE, South 11°43'14" East, 32.00 feet leaving the southerly line of said 7.06 acre tract, over and across said 2.0490 acre tract to a set 5/8-inch iron rod with "GeoSolutions" cap for the southeasterly corner of the herein described parcel;

THENCE, South 78°08'14" West, 108.00 feet continuing over and across said 2.0490 acre tract to a set 5/8-inch iron rod with "GeoSolutions" cap in the existing easterly right-of-way line of Farm-to-Market Road 2100 (F.M. 2100) (100' R.O.W.) of record under Volume 3117, Page 572 and Volume 3114, Page 531 both of said Deed Records, being the westerly line of said 2.0490 acre tract, from which a found concrete monument for the point of curvature of a curve to the right in the existing easterly right-of-way line of F.M. 2100 bears South 11°43'14" East, 848.18 feet;

THENCE, North 11°43'14" West, 32.00 feet along the existing easterly right-of-way line of F.M. 2100, being the westerly line of said 2.0490 acre tract to the southwesterly corner of said 7.03 acre tract, being the northwesterly corner of said 2.0490 acre tract;

THENCE, North 78°08'14" East, 108.00 feet along the southerly line of said 7.03 acre tract, being the northerly line of said 2.0490 acre tract to the **POINT OF BEGINNING** and containing 0.0793 of an acre (3,456 square feet) of land.

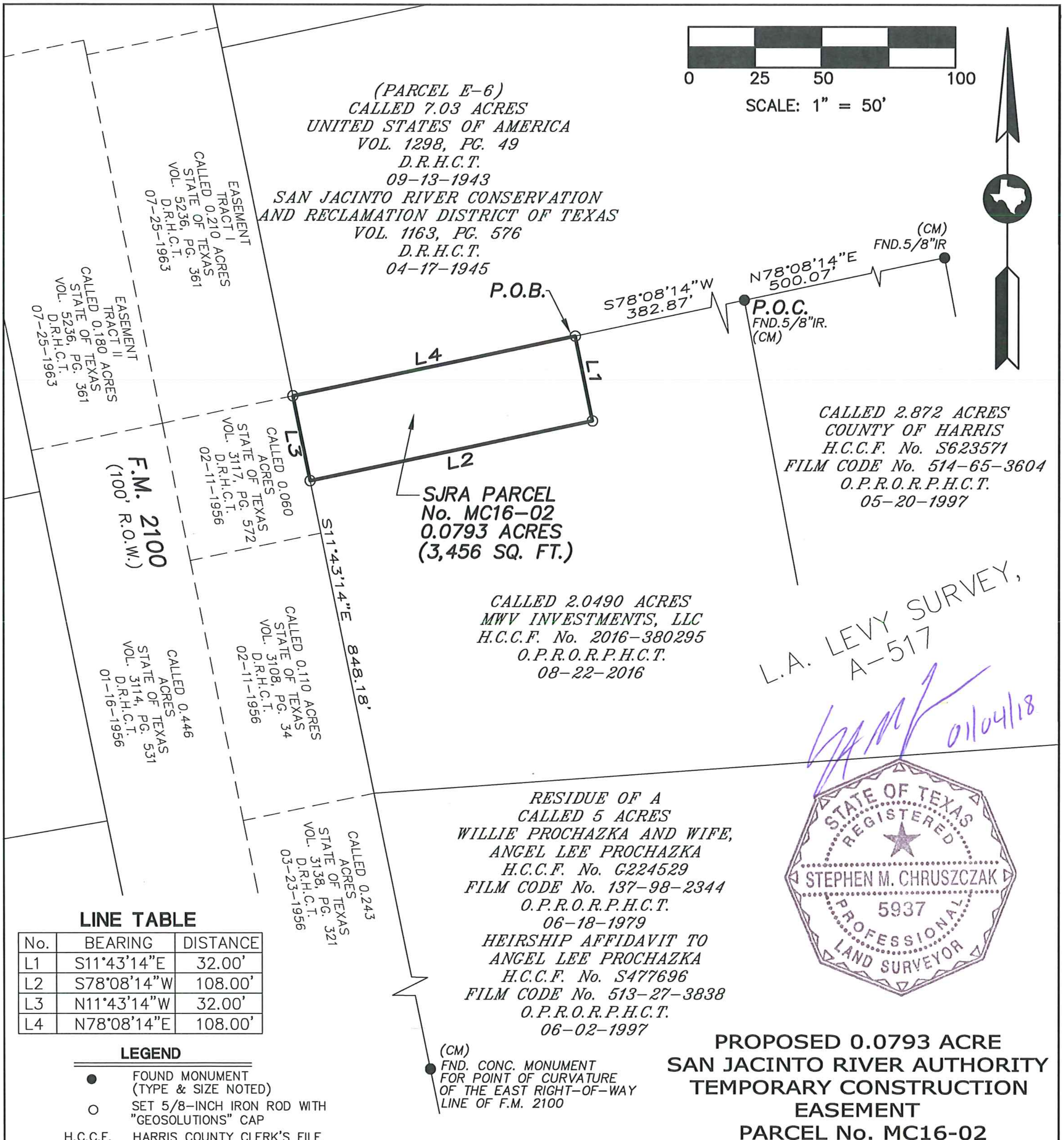
Bearings are referenced to the Texas Coordinate System, NAD 83, South Central Zone. Distances may be converted to GRID by dividing by the following combined scale factor of 1.0000979582. Control was prepared by others and provided by client.

A separate survey map was prepared in conjunction with this metes and bounds description.

Prepared By:
GeoSolutions, LLC
Firm No. 100159-00
25211 Grogan's Mill Road, Suite 375
The Woodlands, Texas 77380

STATE OF TEXAS
REGISTERED
STEPHEN M. CHRUSZCZAK
5937
PROFESSIONAL
LAND SURVEYOR

01/04/18



01/04/18

STEPHEN M. CHRUSZCZAK
5937

LINE TABLE

No.	BEARING	DISTANCE
L1	S11°43'14"E	32.00'
L2	S78°08'14"W	108.00'
L3	N11°43'14"W	32.00'
L4	N78°08'14"E	108.00'

LEGEND

●	FOUND MONUMENT (TYPE & SIZE NOTED)
○	SET 5/8-INCH IRON ROD WITH "GEOSOLUTIONS" CAP
H.C.C.F.	HARRIS COUNTY CLERK'S FILE
O.P.R.R.P.H.C.T.	OFFICIAL PUBLIC RECORDS OF REAL PROPERTY HARRIS COUNTY, TEXAS
H.C.M.R.	HARRIS COUNTY MAP RECORDS
D.R.H.C.T.	DEED RECORDS HARRIS COUNTY, TEXAS
(CM)	CONTROLLING MONUMENT

NOTES:

1. Bearings are referenced to the Texas Coordinate System, NAD83, South Central Zone. Distances may be converted to GRID by dividing by the following combined scale factor of 1.0000979582. Control was prepared by others and provided by client.

2. A metes and bounds description of even date accompanies this survey plat.

PROPOSED 0.0793 ACRE SAN JACINTO RIVER AUTHORITY TEMPORARY CONSTRUCTION EASEMENT PARCEL No. MC16-02 OUT OF THE L.A. LEVY SURVEY, ABSTRACT No. 517 SITUATED IN HARRIS COUNTY, TEXAS

GeoSolutions, LLC

25211 Grogan's Mill Road, Suite 375
The Woodlands, Texas 77380
Phone: 281-681-9766 Fax: 281-681-9779

Drawing By: SMC	Date: 12/06/17	Scale: 1"=50'
Checked By: MEC	FIRM No. 100159-00	
Project No. 17-317-034	Drawing Name 17317034TI.dwg	SJRA PARCEL: MC16-02

May 3, 2018

MWV Investments, LLC
Attn: Mark Walker
15220 Bohemian Hall Rd.
Crosby, TX 77532

Re: Letter Agreement between MWV Investments, LLC (the "Owner") and the San Jacinto River Authority (the "Authority")

Dear Mr. Walker:

Pursuant to that certain Temporary Construction Easement Agreement, recorded in Volume ______, Page ____, of the Deed Records of Harris County, Texas, the Authority holds a temporary construction easement (the "Temporary Construction Easement") over, under and across certain real property owned by Owner for the purpose of providing a work area for the Authority, its agents, contractors, subcontractors, and its and their employees for the construction, installation, inspection, maintenance, and repair of the Authority's canal facilities and related appurtenances, improvements, and equipment. This Letter Agreement memorializes the following supplemental agreements concerning the Temporary Construction Easement by and between the Authority and Owner:

1. The Owner agrees to allow access to and from the "Temporary Construction Easement", in, along, upon and across the "Access Area", all as depicted on Exhibit “A” attached hereto.

2. The Authority will take all reasonable measures to protect the existing driveway and culvert located within the Access Area, including but not limited to, the installation of construction matting. If the driveway or culvert is damaged, the Authority shall at its expense restore the driveway to substantially the same condition as existed prior to the Authority’s entry upon the Access Area.

3. The Authority will take all reasonable measures to protect the existing trees within and adjacent to the Temporary Construction Easement, including the Access Area. The Owner agrees not to hold the Authority responsible for the loss of trees within or adjacent to the Temporary Construction Easement, including the Access Area.

4. Although the Temporary Construction Easement Agreement states that the Authority agrees to restore the Work Area to as near the original condition as is reasonably practicable, this will not include the replacement of sod within the Temporary Construction Easement or the Access Area, which the Owner has agreed to do at their expense.

Sincerely,

SAN JACINTO RIVER AUTHORITY

____
Jace Houston
General Manager

AGREED on the date(s) indicated below.

MWV INVESTMENTS, LLC

By: ____________________________________

Name: __________________________________

Title: ___________________________________

Date: ___________________________________

Attachments

2015 GEOTECHNICAL INVESTIGATION REPORT BY AVILES ENGINEERING CORPORATION

GEOTECHNICAL INVESTIGATION

SAN JACINTO RIVER AUTHORITY
HIGHLANDS CANAL SYSTEM IMPROVEMENTS PROGRAM
SIPHON NO. 7 IMPROVEMENTS
HARRIS COUNTY, TEXAS

Reported to

Texas Water Engineering, PLLC
c/o SJRA
Sugar Land, Texas

by

Aviles Engineering Corporation
5790 Windfern
Houston, Texas 77041
713-895-7645

REPORT NO. G134-15

September 2015

AVILES
ENGINEERING CORP.

5790 Windfern Road
Houston, Texas 77041
Tel: (713)-895-7645
Fax: (713)-895-7943

September 11, 2015

Ms. Victoria A. Foss, P.E.
Texas Water Engineering, PLLC
19901 Southwest Freeway, Suite 134
Houston, Texas 77479

Reference: **Geotechnical Investigation**
San Jacinto River Authority
Highlands Canal System Improvement Program
Siphon No. 7 Improvements
Harris County, Texas
AEC Report No. G134-15

Dear Ms. Foss,

Aviles Engineering Corporation (AEC) is pleased to present this final report of the results of our geotechnical investigation for the above referenced project. This investigation was authorized to proceed on June 1, 2015 by Mr. Steven P. Fenney, CCM, Senior Technical Services Manager of SJRA, under a Purchase Order No. 15-0939, based upon AEC Proposal No. G2015-07-02R, dated May 13, 2015.

AEC appreciates the opportunity to be of service to you. Please call us if you have any questions or comments concerning this report or when we can be of further assistance.

Respectfully submitted,
Aviles Engineering Corporation
(TBPE Firm Registration No. F-42)

SHOU TING HU
84686

9/11/2015

Chun Ho Lee, M.S.C.E., E.I.T.
Staff Engineer

Shou Ting Hu, M.S.C.E., P.E.
Principal Engineer

Final Reports Submitted: 3 SJRA
1 File (electronic)

Z:\ENGINEERING\REPORTS\2015\134-15 SIPHON NO. 7, SJRA MAIN CANAL - SJRA (CHUN HO)\G134-15 FINAL.DOCX

TABLE OF CONTENTS

EXECUTIVE SUMMARY i
1.0 INTRODUCTION 1
  1.1 Project Description 1
  1.2 Purpose and Scope 1
2.0 SUBSURFACE EXPLORATION 2
3.0 LABORATORY TESTING 2
4.0 SITE CONDITIONS 3
  4.1 Subsurface Conditions 4
  4.2 Subsurface Variations 4
5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS 5
  5.1 Backfill Existing Siphon Pipes 5
  5.2 Tunneling and Its Influence on Adjacent Structures 5
    5.2.1 Loadings on Pipes 6
    5.2.2 Tunnel Access Shafts 7
      5.2.2.1 Tunnel Access Shaft Stability 8
      5.2.2.2 Interlocking Steel Sheet Piles 11
      5.2.2.3 Trench Backfill 12
      5.2.2.4 Reaction Wall 12
    5.2.3 Tunnel Face Stability during Construction 12
      5.2.3.1 General 12
      5.2.3.2 Anticipated Ground Behavior 13
      5.2.3.3 Influence of Tunneling on Existing Structures 14
    5.2.4 Measures to Reduce Distress from Tunneling 15
    5.2.5 Monitoring Existing Structures 16
  5.3 Siphon Inlet/Outlet Structures 16
    5.3.1 Allowable Bearing Capacity 17
    5.3.2 Hydrostatic Uplift 17
    5.3.3 Lateral Earth Pressures 18
  5.4 Slope Stability Analysis 18
    5.4.1 Conditions Analyzed for Slope Stability 19
  5.5 Select Fill 21
    5.5.1 Backfill for Headwall Structures or Retaining Walls 21
    5.5.2 Backfill for Embankment 22
6.0 CONSTRUCTION CONSIDERATIONS 22
  6.1 Site Preparation 22
  6.2 Groundwater Control 23
  6.3 Construction Monitoring 24
  6.4 Monitoring of Existing Structures 24
7.0 GENERAL 24
8.0 LIMITATIONS 25
REFERENCES 25

APPENDICIES

APPENDIX A

Plate A-1	Vicinity Map
Plate A-2	Boring Location Plan
Plates A-3 and A-4	Boring Logs
Plate A-5	Key to Symbols
Plate A-6	Classification of Soils for Engineering Purposes
Plate A-7	Terms Used on Boring Logs
Plate A-8	ASTM & TXDOT Designation for Soil Laboratory Tests
Plate A-9	Sieve and Hydrometer Analysis Test Results
Plate A-10	Crumb Test Results
Plates A-11 thru A-14	CU Test Results

APPENDIX B

Plate B-1	Canal Levee Topo Provided by SJRA
Plate B-2	Canal Levee Cross Sections (A-A thru D-D) Provided by SJRA

APPENDIX C

Plate C-1	Recommended Geotechnical Design Parameters
Plate C-2	Load Coefficients for Pipe Loading
Plate C-3	Live Loads on Pipe Crossing Under Roadway

APPENDIX D

Plate D-1	Critical Heights of Cuts in Nonfissured Clays
Plate D-2	Maximum Allowable Slopes
Plate D-3	A Combination of Bracing and Open Cuts
Plate D-4	Lateral Pressure Diagrams for Open Cuts in Cohesive Soil-Long Term Conditions
Plate D-5	Lateral Pressure Diagrams for Open Cuts in Cohesive Soil-Short Term Conditions
Plate D-6	Lateral Pressure Diagrams for Open Cuts in Sand
Plate D-7	Bottom Stability for Braced Excavation in Clay
Plate D-8	Relation between the Width of Surface Depression and Depth of Cavity for Tunnels
Plate D-9	Tunnel Behavior and TBM Selection
Plate D-10	Methods of Controlling Ground Water in Tunnel and Grouting Material Selection
Plate D-11	Buoyant Uplift Resistance

APPENDICIES (CONT…)

APPENDIX E

Plate E-1	Design Soil Parameters for Slope Stability Analysis
Plate E-2a	Slope Stability Analysis on CS-AA Existing Waterside Slope (Global Slide for North Embankment), H:V=3.9:1, Short Term Condition, Based on Boring B-2
Plate E-2b	Slope Stability Analysis on CS-AA Existing Waterside Slope (Local Slide for North Embankment), H:V=3.9:1, Short Term Condition, Based on Boring B-2
Plate E-3a	Slope Stability Analysis on CS-AA Existing Waterside Slope (Global Slide for North Embankment), H:V=3.9:1, Long Term Condition, Based on Boring B-2
Plate E-3b	Slope Stability Analysis on CS-AA Existing Waterside Slope (Local Slide for North Embankment), H:V=3.9:1, Long Term Condition, Based on Boring B-2
Plate E-4a	Slope Stability Analysis on CS-AA Existing Waterside Slope (Global Slide for North Embankment), H:V=3.9:1, Rapid Drawdown Condition, Based on Boring B-2
Plate E-4b	Slope Stability Analysis on CS-AA Existing Waterside Slope (Local Slide for North Embankment), H:V=3.9:1, Rapid Drawdown Condition, Based on Boring B-2
Plate E-5a	Slope Stability Analysis on CS-BB Existing Waterside Slope (Global Slide for North Embankment), H:V=3:1, Short Term Condition, Based on Boring B-2
Plate E-5b	Slope Stability Analysis on CS-BB Existing Waterside Slope (Local Slide for North Embankment), H:V=3:1, Short Term Condition, Based on Boring B-2
Plate E-6a	Slope Stability Analysis on CS-BB Existing Waterside Slope (Global Slide for North Embankment), H:V=3:1, Long Term Condition, Based on Boring B-2
Plate E-6b	Slope Stability Analysis on CS-BB Existing Waterside Slope (Local Slide for North Embankment), H:V=3:1, Long Term Condition, Based on Boring B-2
Plate E-7a	Slope Stability Analysis on CS-BB Existing Waterside Slope (Global Slide for North Embankment), H:V=3:1, Rapid Drawdown Condition, Based on Boring B-2
Plate E-7b	Slope Stability Analysis on CS-BB Existing Waterside Slope (Local Slide for North Embankment), H:V=3:1, Rapid Drawdown Condition, Based on Boring B-2
Plate E-8a	Slope Stability Analysis on CS-CC Existing Waterside Slope (Global Slide for North Embankment), Upper H:V=1.9:1, Short Term Condition, Based on Boring B-1
Plate E-8b	Slope Stability Analysis on CS-CC Existing Waterside Slope (Local Slide for North Embankment), Upper H:V=1.9:1, Short Term Condition, Based on Boring B-1
Plate E-9a	Slope Stability Analysis on CS-CC Existing Waterside Slope (Global Slide for North Embankment), Upper H:V=1.9:1, Long Term Condition, Based on Boring B-1
Plate E-9b	Slope Stability Analysis on CS-CC Existing Waterside Slope (Local Slide for North Embankment), Upper H:V=1.9:1, Long Term Condition, Based on Boring B-1
Plate E-10a	Slope Stability Analysis on CS-CC Existing Waterside Slope (Global Slide for North Embankment), Upper H:V=1.9:1, Rapid Drawdown Condition, Based on Boring B-1
Plate E-10b	Slope Stability Analysis on CS-CC Existing Waterside Slope (Local Slide for North Embankment), Upper H:V=1.9:1, Rapid Drawdown Condition, Based on Boring B-1
Plate E-11a	Slope Stability Analysis on CS-CC Existing Landside Slope (Global Slide for North Embankment), H:V=3.1:1, Short Term Condition, Based on Boring B-1
Plate E-11b	Slope Stability Analysis on CS-CC Existing Landside Slope (Local Slide for North Embankment), H:V=3.1:1, Short Term Condition, Based on Boring B-1
Plate E-12a	Slope Stability Analysis on CS-CC Existing Landside Slope (Global Slide for North Embankment), H:V=3.1:1, Long Term Condition, Based on Boring B-1
Plate E-12b	Slope Stability Analysis on CS-CC Existing Landside Slope (Local Slide for North Embankment), H:V=3.1:1, Long Term Condition, Based on Boring B-1
Plate E-13a	Slope Stability Analysis on CS-CC Existing Landside Slope (Global Slide for North Embankment), H:V=3.1:1, Rapid Drawdown Condition, Based on Boring B-1
Plate E-13b	Slope Stability Analysis on CS-CC Existing Landside Slope (Local Slide for North Embankment), H:V=3.1:1, Rapid Drawdown Condition, Based on Boring B-1

EXECUTIVE SUMMARY

This report presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for the proposed improvements to the San Jacinto River Authority's (SJRA) Siphon No. 7 along the SJRA Highlands Division Main Canal at FM 2100 in northeast Harris County, Texas.

According to latest information provided by SJRA to AEC, the existing Siphon No. 7 consists of three in-service underground conduits (one 60 inch diameter bypass concrete siphon and dual 48 inch diameter concrete pipes) running beneath FM 2100 in west-to-east direction and connecting two sections of existing canal. The invert depth of the existing siphons is approximately 11 to 14 feet below the top of the levee. Based on the updated Plan and Profile drawings provided by SJRA to AEC on August 5, 2015, the proposed improvements to the Siphon No. 7 include: (i) abandoning the three existing siphons in-place and replacing with dual 60-inch diameter siphons; (ii) abandoning the existing inlet and outlet headwall structures and replacing with new headwall structures; and (iii) widening the existing roadway right of way (ROW) from 100 feet to 120 feet, which is being done by TxDOT. The proposed siphon pipes will have an invert depth of approximately 18 feet deep below the top of levee (i.e. Flow Line Elevation = 34.5 feet) and will be installed using tunneling method. Geotechnical recommendation for widening the roadway is beyond AEC's scope of this service.

1. Subsurface Soil Conditions: Based on Borings B-1 and B-2, subsurface soil conditions at the site generally consist of approximately 8 to 10 feet of fill [soft to very stiff very high plasticity fat clay (CH)] below the top of the existing levee, underlain by approximately 17 to 24 feet of firm to very stiff very high plasticity fat clay (CH), then by 8 to 13 feet of firm to very stiff lean clay w/sand (CL) to the boring termination depth of 40 feet below grade.
2. Subsurface Soil Properties: The clayey soils encountered at the top 15 feet of the levee predominantly consist of very high plasticity clay, with liquid limits (LL) ranging from 67 to 96, and plasticity indices (PI) ranging from 45 to 69. The cohesive soils encountered at the site are classified as "CL" and "CH" type soils in accordance with ASTM D 2487.
3. Groundwater Conditions: Groundwater was initially encountered in Borings B-1 and B-2 at a depth of 39 feet below the top of the levee during drilling and rose to 21 and 23.1 feet, respectively, approximately 24 hours after completion of the drilling. This indicates that the groundwater below the levee is pressurized.
4. Recommendations for siphon pipe installation using tunneling method are presented in Section 5.2 of this report.
5. Recommendations for head wall structure foundations are presented in Section 5.3 of this report.
6. AEC performed slope stability analysis on the 4 most critical cross sections in accordance with Harris County Flood Control District (HCFCD) requirements. The resultants factor of safety (FS) for the selected cross sections meet HCFCD requirements under short-term, long-term, and rapid drawdown conditions. Results of slope stability analysis for the existing levee slopes are presented in Table 3 in Section 5.4 of this report

This Executive Summary is intended as a summary of the investigation and should not be used without the full text of this report.

AVILES
ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

SAN JACINTO RIVER AUTHORITY
HIGHLANDS CANAL SYSTEM IMPROVEMENTS PROGRAM
SIPHON NO. 7 IMPROVEMENTS
HARRIS COUNTY, TEXAS

1.0 INTRODUCTION

1.1 Project Description

This report presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for the proposed improvements to the San Jacinto River Authority's (SJRA) Siphon No. 7 along the SJRA Highlands Division Main Canal at FM 2100 in northeast Harris County, Texas (Houston/Harris County Key Map: 419C). A vicinity map is presented on Plate A-1, in Appendix A.

According to latest information provided by SJRA to AEC, the existing Siphon No. 7 consists of three in-service underground conduits (one 60 inch diameter bypass concrete siphon and dual 48 inch diameter concrete pipes) running beneath FM 2100 in west-to-east direction and connecting two sections of existing canal. The invert depth of the existing siphons is approximately 11 to 14 feet below the top of the levee. Based on the updated Plan and Profile drawings provided by SJRA to AEC on August 5, 2015, the proposed improvements to the Siphon No. 7 include: (i) abandoning the three existing siphons in-place and replacing with dual 60-inch diameter siphons; (ii) abandoning the existing inlet and outlet headwall structures and replacing with new headwall structures; and (iii) widening the existing roadway right of way (ROW) from 100 feet to 120 feet, which is being done by TxDOT. The proposed siphon pipes will have an invert depth of approximately 18 feet deep below the top of levee (i.e. Flow Line Elevation = 34.5 feet) and will be installed using tunneling method. Geotechnical recommendation for widening the roadway is beyond AEC's scope of this service.

1.2 Purpose and Scope

The purpose of this geotechnical investigation is to evaluate the subsurface soil and ground water conditions at the project site and develop geotechnical engineering recommendations for design and construction of underground siphon pipes as well as the headwall structures. The scope of this geotechnical investigation is summarized as below:

1. Soil drilling and sampling 2 geotechnical borings to a depth of 40 feet below the top of the existing levee;
2. Soil laboratory testing on selected soil samples;
3. Engineering analyses and recommendations for the installation of siphon pipes using tunneling method, including loadings on pipes, bedding, lateral earth pressure parameters, tunnel pit stability and backfill requirement, tunnel access shaft, reaction wall, and tunnel stability;
4. Engineering analyses and recommendations for the headwall structures, including allowable bearing capacities for the wall footings and design soil parameters for lateral earth pressures on the headwalls;
5. Construction recommendations and ground water control guidelines for the proposed siphon pipes; and
6. Slope stability analysis on the selected cross-section(s) of the canal levees and, if necessary, providing recommendations for regrading slope.

2.0 SUBSURFACE EXPLORATION

Subsurface conditions at the site were investigated by drilling two soil borings to a depth of 40 feet below the top of the existing levee. Both borings were drilled adjacent to the existing wingwalls at each ends of the existing inlet/outlet structure. The approximate boring locations are shown on the attached Boring Location Plan on Plate A-2, in Appendix A. Boring locations were surveyed by others but the completed survey data was not available to AEC when this report was prepared.

The field drilling was performed with a buggy-mounted drilling rig using dry auger method. Undisturbed samples of cohesive soils were obtained from the borings by pushing 3-inch diameter thin-wall, seamless steel Shelby tube samplers in accordance with ASTM D 1587. Strength of the cohesive soils was estimated in the field using a hand penetrometer. Undisturbed samples of cohesive soils were extruded mechanically from the core barrels in the field and wrapped in aluminum foil; all samples were sealed in plastic bags to reduce moisture loss and disturbance. The samples were then placed in core boxes and transported to the AEC laboratory for testing and further study. After completion of drilling, the borings were left open and covered at top so that 24 hour groundwater readings could be obtained. After the groundwater readings were measured, the borings were backfilled with bentonite chips. Details of the soils encountered in the borings are presented on Plates A-3 and A-4, in Appendix A.

3.0 LABORATORY TESTING

Soil laboratory testing was performed by AEC personnel. Samples from the borings were examined and classified in the laboratory by a technician under supervision of a geotechnical engineer. Laboratory tests were performed on selected soil samples in order to evaluate the engineering properties of the subsurface soils in accordance with applicable ASTM Standards. Atterberg limits, moisture contents, percent passing a No. 200 sieve, sieve and hydrometer, and dry unit weight tests were performed on typical samples to establish the index properties and confirm field classification of the subsurface soils. Strength properties of cohesive soils were determined by means of unconfined compression (UC) and unconsolidated undrained (UU) triaxial tests performed on undisturbed samples. The test results are presented on the representative boring logs. A key to the boring logs, classification of soils for engineering purposes, terms used on boring logs, and reference ASTM Standards for laboratory testing are presented on Plates A-5 through A-8, in Appendix A. Sieve and hydrometer analysis results are presented on Plate A-9, in Appendix A.

Crumb Tests: Two crumb tests were performed on the selected samples to evaluate the dispersive potential of clayey soils at the levee slopes. The crumb test results indicate that the soil samples from Boring B-1, 4 to 6 feet, and Boring B-2, 8 to 10 feet, had a “no sign of cloudy water caused by colloidal suspension”, and can be considered non dispersive. The crumb test results are presented on Plate A-10, in Appendix A.

Consolidated-Undrained Triaxial Tests: Two consolidated-undrained (CU) triaxial tests were performed to determine design soil parameters for slope stability analysis of levee slopes. The results of the CU triaxial tests are included on Plates A-11 through A-14, in Appendix A. The shear strength parameters obtained from the CU triaxial test are summarized in Table 1.

Table 1. Summary of Shear Strength Parameters from CU Triaxial Tests

Sample ID and Description	**C_u (psf)**	**EFFECTIVE STRESS**		**TOTAL STRESS**	
		c' (psf)	**φ' (deg)**	**c_{cu} (psf)**	**φ_{cu} (deg)**
B-1, 6’-8’, Fat Clay (CH)	400	210	19.9	230	14.5
B-1, 12’-14’, Fat Clay (CH)	1,400	540	18.0	540	13.4

Notes: (1) C_u = cohesion, obtained from UC or UU tests;
(2) c' = effective cohesion, φ' =effective friction angle, obtained from CU tests with pore pressure measurements;
(3) c_{cu} = cohesion in total stress, φ_{cu} = friction angle in total stress, obtained from CU tests.

4.0 SITE CONDITIONS

The site currently is covered with mowed grass on the levee. The existing siphons run in a west-to-east direction beneath the existing FM 2100 roadway and begin and end at concrete headwall structures on the canal levee. A hole is observed beneath the existing headwall due to erosion. At the time of our site visit on June 8, 2015, water in the canal (i.e. freeboard) was measured approximately 2 feet below the top of the levee. Desiccation cracks are observed on the top of the existing levee. FM 2100 at the siphon location is a medium to heavy traffic roadway.

4.1 Subsurface Conditions

Soil strata encountered in our borings are summarized below.

Boring	Depth	Description of Stratum
B-1	0’ - 10’	Fill: soft to very stiff, Fat Clay (CH), with slickensides
	10’ - 27’	Stiff to very stiff, Fat Clay (CH), with slickensides
	27’ - 40’	Stiff to very stiff, Lean Clay w/Sand (CL), with silt partings
B-2	0’ - 8’	Fill: firm to very stiff, Fat Clay (CH), with slickensides
	8’ - 32’	Firm to very stiff, Fat Clay (CH), with slickensides
	32’ - 40’	Firm to very stiff, Lean Clay w/Sand (CL), with abundant sand seams and partings

The cohesive soils encountered at the top 15 feet of the levee have Liquid Limits (LL) that varied from 67 to 96 and Plasticity Indices (PI) that varied from 45 to 69. This indicates that the cohesive soils at the site have very high plasticity. The cohesive soils encountered are classified as “CH” and “CL” type soils in accordance with the ASTM D 2478. “CH” soils undergo significant volume changes due to seasonal changes in soil moisture contents. “CL” type soils with lower LL (less than 40) and PI (less than 20) generally do not undergo significant volume changes with changes in moisture content. However, “CL” soils with LL approaching 50 and PI greater than 20 essentially behave as “CH” soils and could undergo significant volume changes. Slickensides were encountered in fat clay soils.

Ground water was initially encountered in Borings B-1 and B-2 at a depth of 39 feet below the top of the levee during drilling and rose to depths of 21 and 23.1 feet, respectively, approximately 24 hours after completion of drilling. This indicates that the ground water below the levee may be pressurized. The information in this report summarizes conditions found on the date the borings were drilled. It should be noted that our ground water observations are short term; ground water depths and subsurface soil moisture contents will vary with environmental variations such as frequency and magnitude of rainfall and the time of year when construction is in progress.

4.2 Subsurface Variations

It should be emphasized that: (i) at any given time, ground water depths can vary from location to location, and (ii) at any given location, ground water depths can change with time. Ground water depths will vary with seasonal rainfall and other climatic/environmental events.

Clay soils in the Harris County area typically have secondary features such as slickensides and contain sand/silt seams/lenses/layers/pockets. It should be noted that the information in the boring logs is based on

3-inch diameter soil samples which were generally obtained at intervals of 2 feet in the top 20 feet of the borings and at 5 foot intervals thereafter to the boring termination depth of 40 feet below the top of existing levee. A detailed description of the soil secondary features may not have been obtained due to the small sample size and sampling interval between the samples. Therefore, while AEC's logs may or may not show the soil secondary features, it should not be assumed that the features are absent from the site.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

According to latest information provided by SJRA to AEC, the existing Siphon No. 7 consists of three in-service underground conduits (one 60 inch diameter bypass concrete siphon and dual 48 inch diameter concrete pipes) running beneath FM 2100 in west-to-east direction and connecting two sections of existing canal. The invert depth of the existing siphons is approximately 11 to 14 feet below the top of the levee. Based on the updated Plan and Profile drawings provided by SJRA to AEC on August 5, 2015, the proposed improvements to the Siphon No. 7 include: (i) abandoning the three existing siphons in-place and replacing with dual 60-inch diameter siphons; (ii) abandoning the existing inlet and outlet headwall structures and replacing with new headwall structures; and (iii) widening the existing roadway right of way (ROW) from 100 feet to 120 feet, which is being done by TxDOT. The proposed siphon pipes will have an invert depth of approximately 18 feet deep below the top of levee (i.e. Flow Line Elevation = 34.5 feet) and will be installed using tunneling method. Geotechnical recommendation for widening the roadway is beyond AEC's scope of this service.

5.1 Backfill Existing Siphon Pipes

AEC understands that the 3 existing siphon pipes will be abandoned in place. AEC recommends that flowable fill be used to backfill the existing siphon pipes in general accordance with Section 02322 of the 2012 City of Houston Standard Construction Specifications (COHSCS), or the corresponding SJRA Specification, whichever is more stringent.

5.2 Tunneling and Its Influence on Adjacent Structures

AEC understands that the proposed siphon that crosses beneath FM 2100 will be installed by tunnel method. The Contractor is responsible for designing, constructing, implementing, and monitoring safe tunneling excavation and protecting existing structures in the vicinity from adverse effects resulting from construction.

Based on the Plan and Profile Drawings of the existing siphon pipes provided by SJRA on June 24, 2015, the invert depth of the existing dual 48-inch siphon pipes is approximately 11 to 12 feet below the top of levee, while the invert depth of the existing 60-inch by-pass siphon pipe is approximately 14 feet below the top of the levee. Based on the most updated Plan and Profile Drawings for the proposed siphon pipes, the invert depth of the proposed twin 60-inch siphon pipes is approximately 18 feet below the top of levee [i.e. Elevation = 34.5 feet based on North American Datum of 1983 (NAD83)]. AEC should be notified if the proposed siphon pipe depth is different so that the recommendations herein can be updated accordingly. The proposed tunnel invert depths and possible subsurface conditions at the tunnel crossing are summarized in Table 1 below.

Table 1. Subsurface Conditions in Borings within Tunnel Zone at FM 2100

Soil Boring	Pipe Invert Depth [(a)] (ft)	Elevation (ft)	Soil Types Encountered in Borings [(a)]	Ground Water Depth below Existing Ground Surface (ft)		
				During Drilling	¼ Hour After First Encounter	24 Hour After Drilling
B-1	18	34	7’-28’: soft to very stiff Fat Clay (CH)	39	28	21[(b)] (6/8/15)
B-2	18	34	7’-28’ Firm to very stiff Fat Clay (CH)	39	39.5	23.1[(b)] (6/8/15)

Note: (a) Pipe invert depth and the depth of soil type encountered in boring are measured from the top of the existing levee (Elev.=52 feet).
(b) AEC conservatively assumes that the cave-in depth is equal to the water level.

Tunneling operations and placement of pipe inside tunnel constructed with primary liner should comply with Section 31 71 00.01 of the SJRA Specification, or Sections 02425 and 02517 of the 2012 COHSCS, whichever is more stringent.

5.2.1 Loadings on Pipes

Underground utilities support the weight of the soil and water above the crown (i.e. the crown of the siphon pipe), as well as roadway traffic and any structures that exist above the utilities.

Earth Loads: The vertical soil load W_e can be calculated as the larger of the two values from Equations (1) and (3):

$$W_e = C_d \gamma B_d^2 \qquad \text{...........Equation (1)}$$

$$C_d = [1- e^{-2K\mu'(H/B_d)}]/(2K\mu') \qquad \text{...........Equation (2)}$$

$$W_e = \gamma B_c H \qquad \text{...........Equation (3)}$$

where: W_e = soil load, in pounds per linear foot (lb/ft);
C_d = soil load coefficient, see Plate C-2, in Appendix C;
γ = effective unit weight of soil over the conduit, in pounds per cubic foot (pcf);
B_d = tunnel width at top of the conduit $< 1.5\ B_c$ (ft);
B_c = outside diameter of the conduit (ft);
H = variable height of soil (ft);
when the height of soil above the top of the conduit $H_c > 2\ B_d$, $H = H_h$ (height of soil above the middle of the conduit). When $H_c < 2\ B_d$, H varies over the height of the conduit; and
$K\mu'$ = 0.1650 maximum for sand and gravel,
0.1500 maximum for saturated top soil,
0.1300 maximum for ordinary clay,
0.1100 maximum for saturated clay.

When underground conduits are located below groundwater, the total vertical dead loads should include the weight of the projected volume of water above the conduits.

Traffic Loads: The vertical stress on top of an underground conduit, p_L (psf), resulting from traffic loads (from a H-20 or HS-20 truck) can be obtained from Plate C-3, in Appendix C. The live load on top of the underground conduit can be calculated from Equation (4):

$$W_L = p_L\ B_c \qquad \text{...........Equation (4)}$$

where: W_L = live load on the top of the conduit (lb/ft);
p_L = vertical stress (on the top of the conduit) resulting from traffic loads (psf);
B_c = outside diameter of the conduit, (ft);

Lateral Loads: The lateral soil pressure p_l can be calculated from Equation (5); hydrostatic pressure should be added, if applicable.

$$p_l = 0.5\ (\gamma H_h + p_s) \qquad \text{...........Equation (5)}$$

where: H_h = height of fill above the center of the conduit (ft);
γ = effective unit weight of soil over the conduit (pcf);
p_s = vertical pressure on conduit resulting from traffic and/or construction equipment (psf).

5.2.2 Tunnel Access Shafts

Tunnel access shafts should be constructed in accordance with Section 31 75 00 of the SJRA Specification, or Section 02400 of the 2012 COHSCS, whichever is more stringent. Since the tunnel access shafts for the siphon construction will be likely extend into soft to firm fat clay (See Table 1 in Section 5.2), the access shaft walls can be supported by internally-braced, water-tight steel sheet piles. Sheet pile recommendations are presented in Section 5.2.2.2 of this report.

Although groundwater was encountered below the anticipated tunnel construction zone during drilling, groundwater can be higher during or prior to the construction period after prolonged rainfall. Ground water control may be required for the tunnel shafts. In cohesive soils seepage rates are lower than in granular soils and groundwater is usually collected in sumps and channeled by gravity flow to storm sewers. If cohesive soils contain significant secondary features, seepage rates will be higher. This may require larger sumps and drainage channels, or if significant granular layers are interbedded within the cohesive soils, methods used for granular soils may be required. Where it is present, pressurized groundwater will also yield higher seepage rates. General groundwater control recommendations are presented in Section 6.2 of this report. The options for dewatering presented here are for reference purposes only; it is the Contractor's responsibility to take the necessary precautions to minimize the effect on existing structures in the vicinity of the dewatering operation.

5.2.2.1 Tunnel Access Shaft Stability

Cohesive soils in the Harris County area contain many secondary features which affect tunnel access shaft stability, including sand seams and slickensides. Slickensides are shiny weak failure planes which are commonly present in fat clays; such clays often fail along these weak planes when they are not laterally supported, such as in an open excavation. The Contractor should not assume that slickensides and sand seams/layers/pockets are absent where not indicated on the logs.

The Contractor should be responsible for designing, constructing and maintaining safe excavations. The excavations should not cause any distress to existing structures.

Trenches 20 Feet and Deeper: OSHA requires that shoring or bracing for trenches 20 feet and deeper be specifically designed by a licensed professional engineer.

Trenches Less than 20 Feet Deep: Trench excavations that are less than 20 feet deep may be shored, sheeted and braced, or laid back to a stable slope for the safety of workers, the general public, and adjacent structures, except for excavations which are less than 5 feet deep and verified by a competent person to have no cave-in potential. The excavation and trenching should be in accordance with Occupational Safety and Health Administration (OSHA), Safety and Health Regulations, 29 CFR, Part 1926. Recommended OSHA soil types for trench design for existing soils can be found on Plate C-1, in Appendix C. Fill soils are considered OSHA Class 'C'; submerged cohesive soils should also be considered OSHA Class 'C', unless they are dewatered first.

Critical Height is defined as the height a slope will stand unsupported for a short time; in cohesive soils, it is used to estimate the maximum depth of open-cuts at given side slopes. Critical Height may be calculated based on the soil cohesion. Values for various slopes and cohesion are shown on Plate D-1, in Appendix D. Cautions listed below should be exercised in use of Critical Height applications:

1. No more than 50 percent of the Critical Height computed should be used for vertical slopes. Unsupported vertical slopes are not recommended where granular soils or soils that will slough when not laterally supported are encountered within the excavation depth.
2. If the soil at the surface is dry to the point where tension cracks occur, any water in the crack will increase the lateral pressure considerably. In addition, if tension cracks occur, no cohesion should be assumed for the soils within the depth of the crack. The depth of the first waler should not exceed the depth of the potential tension crack. Struts should be installed before lateral displacement occurs.
3. Shoring should be provided for excavations where limited space precludes adequate side slopes, e.g., where granular soils will not stand on stable slopes and/or for deep open cuts.
4. All excavation, trenching and shoring should be designed and constructed by qualified professionals in accordance with OSHA requirements.

The maximum (steepest) allowable slopes for OSHA Soil Types for excavations less than 20 feet are presented on Plate D-2, in Appendix D.

If limited space is available for the required open trench side slopes, the space required for the slope can be reduced by using a combination of bracing and open cut as illustrated on Plate D-3, in Appendix D. Guidelines for bracing and calculating bracing stress are presented below.

<u>Computation of Bracing Pressures:</u> The following method can be used for calculating earth pressure against bracing for open cuts. Lateral pressure resulting from construction equipment, traffic loads, or other surcharge should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. Hydrostatic pressure, if any, should also be considered. The active earth pressure at depth z can be determined by Equation (6). The design soil parameters for trench bracing design are presented on Plate C-1, in Appendix C.

$$p_a = (q_s + \gamma h_1 + \gamma' h_2)Ka - 2c\sqrt{K_a} + \gamma_w h_2 \qquad \text{...........Equation (6)}$$

where: p_a = active earth pressure (psf);
q_s = uniform surcharge pressure (psf);
γ, γ' = wet unit weight and buoyant unit weight of soil (pcf);
h_1 = depth from ground surface to groundwater table (ft);
h_2 = $z-h_1$, depth from groundwater table to the point under consideration (ft);
z = depth below ground surface for the point under consideration (ft);
K_a = coefficient of active earth pressure;
c = cohesion of clayey soils (psf); c can be omitted conservatively;
γ_w = unit weight of water, 62.4 pcf.

Pressure distribution for the practical design of struts in open cuts for clays and sands are illustrated on Plates D-4 through D-6, in Appendix D.

Bottom Stability: In open-cuts, it is necessary to consider the possibility of the bottom failing by heaving, due to the removal of the weight of excavated soil. Heaving typically occurs in soft plastic clays (such as at the depth of 8 to 12 feet in the vicinity of Borings B-1 and B-2) when the excavation depth is sufficiently deep enough to cause the surrounding soil to displace vertically due to bearing capacity failure of the soil beneath the excavation bottom, with a corresponding upward movement of the soils in the bottom of the excavation. In fat and lean clays, heave normally does not occur unless the ratio of Critical Height to Depth of Cut approaches one. In very sandy and silty lean clays and granular soils, heave can occur if an artificially large head of water is created due to installation of impervious sheeting while bracing the cut. This can be mitigated if groundwater is lowered below the excavation by dewatering the area. Guidelines for evaluating bottom stability in clay soils are presented on Plate D-7, in Appendix D.

If the excavation extends below groundwater and the soils at or near the bottom of the excavation are mainly sands or silts, the bottom can fail by blow-out (boiling) when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized groundwater, the groundwater table should be lowered at least 5 feet below the excavation.

Calcareous nodules, silt/sand seams, and fat clays with slickensides were encountered in our borings. These secondary structures may become sources of localized instability when they are exposed during excavation, especially when they become saturated. Such soils have a tendency to slough or cave in when not laterally confined, such as in trench excavations. The Contractor should be aware of the potential for cave-in of the soils. Low plasticity soils (silts and clayey silts) will lose strength and may behave like granular soils when saturated.

5.2.2.2 Interlocking Steel Sheet Piles

Since weak and very high plasticity fat clays were encountered in the top 10 feet of the existing levee and pressurized ground water was encountered in our borings, AEC recommends the use of interlocking steel sheet piles for excavation shoring and potential seepage cut off. In general, a cantilever sheet pile bulkhead is only suitable for exposed heights up to about 15 feet. Design soil parameters for sheet pile design are presented on Plates C-1, in Appendix C. AEC recommends that the sheet pile design consider both short-term and long-term parameters; whichever is critical should be used for design. The determination of the pressures exerted on the sheet piles by the retained soils shall consider active earth pressure, hydrostatic pressure, and uniform surcharge (including construction equipment, soil stockpiles, and traffic load, whichever surcharge is more critical).

Sheet pile design should be based on the following considerations:

(1) Ground water elevation at the top of the ground surface on the retained side;
(2) Ground water elevation should be maintained 5 feet below the bottom of the access shaft excavation (assuming dewatering operations using deep wells);
(3) Neglect cohesion for active pressure determination, Equation (6) in Section 5.2.2.1;
(4) The design retained height should extend from the ground surface to the water line tunnel invert depth;
(5) A 300 psf uniform surcharge pressure from construction equipment or soil stockpiles should be considered at the top of the sheet piles; loose soil stockpiles during access shaft construction should be limited to 3 foot high or less;
(6) Use a Factor of Safety of 2.0 for passive earth pressure in front of (i.e. the shaft side) the sheet piles.

Design, construction, and monitoring of sheet piles should be performed by qualified personnel who are experienced in this operation. Sheet piles should be driven in pairs, and proper construction controls provided to maintain alignment along the wall and prevent outward leaning of the sheet piles. Construction of sheet piles should be in accordance with Item 408 of the 2014 Harris County Public Infrastructure Department Standard Engineering Construction Specifications (HCPID-SECS), or corresponding SJRA specification, whichever is more stringent.

We recommend that the steel sheet piles be driven in pairs. It is important that the sheet pile with the ball end be driven first. If the sheet pile with the socket end is driven first, it may clog with soil and make it difficult to drive the adjacent pile. Regular inspection of sheet pile tops should be performed to assess damage resulting from driving through relatively hard soils.

5.2.2.3 Trench Backfill

Trench excavation, pipe embedment material, and backfill for the proposed siphon should be in general accordance with Section 02320 of the 2012 COHSCS. Backfill should be placed in loose lifts not exceeding 8 inches and compacted to 95 percent of its ASTM D-698 (Standard Proctor) maximum dry density at a moisture content ranging between optimum and 3 percent above optimum.

5.2.2.4 Reaction Wall

Reaction walls (if used) will be part of the tunnel shaft walls; they will be rigid structures and support tunneling operations by mobilizing passive pressures of the soils behind the walls. Passive earth pressure can be calculated using Equation (7); we recommend that a factor safety of 2.0 be used when using passive earth pressure for design of reaction wall. The design soil parameters for reaction wall design are presented on Plate C-1, in Appendix C.

$$p_p = \gamma z K_p + 2c(K_p)^{1/2} \qquad \text{...........Equation (7)}$$

where, p_p = passive earth pressure (psf);
γ = wet unit weight of soil (pcf);
z = depth below ground surface for the point under consideration (ft);
K_p = coefficient of passive earth pressure;
c = cohesion of clayey soils (psf).

Due to subsurface variations, soils with different strengths and characteristics will likely be encountered at a given location. The soil resulting in the lowest passive pressure should be used for design of the walls. The soil conditions should be checked by geotechnical personnel to confirm the recommended soil parameters.

5.2.3 Tunnel Face Stability during Construction

5.2.3.1 General

The stability of a tunnel face is governed primarily by ground water and subsurface soil conditions, type of tunnel machine used, and workmanship. Based on the subsurface conditions encountered in our borings and the anticipated invert depth, we anticipate that soft to very stiff fat clay (CH) will generally be encountered at the tunneling zone. Secondary features such as slickensides, sand or silt partings/seams/pockets/layers may be also encountered within the cohesive soils, and could be significant at some locations. In addition, the type and property of subsurface soils are subject to change between borings, and can be different at locations away from and in between our borings.

When granular soils, if any, are encountered during construction the tunnel face can become unstable. Granular soils below ground water will tend to flow into the excavation hole; granular soils above the ground water level will generally not stand unsupported but will tend to ravel until a stable slope is formed at the face with a slope equal to the angle of repose of the material in a loose state. Thus, granular soils are generally considered unstable in an unsupported excavation face; uncontrolled flowing soil can result in large loss of ground.

5.2.3.2 Anticipated Ground Behavior

A Stability Factor, $N_t = (P_z - P_a)/C_u$ may be used to evaluate the stability of an unsupported bore face in cohesive soils, where P_z is the overburden pressure to the bore centerline; P_a is the equivalent uniform interior pressure applied to the face; and C_u is the soil undrained shear strength. For tunneling operations, no interior pressure is applied. Generally, N_t values of 4 or less are desirable as it represents a practical limit below which augering may be accomplished without significant difficulty. Higher N_t values usually lead to large deformations of the soil around the bore and problems associated with increased subsidence. It should be noted that the exposure time of the face is most important; with time, creep of the soil will occur, resulting in a reduction of shear strength. The N_t values will therefore increase when construction is slow.

Note that the cohesive soils have secondary structures such as fissures, sand seams, and sand lenses which can cause the bore face to become unstable. As indicated on Table 1, soft clayey soils were encountered in Boring B-1 within the anticipated tunnel invert depths. Where granular or soft cohesive soils, if any, are encountered, the Contractor should make provisions for casing to stabilize the tunnel. The Contractor should not base their bid on the above information alone, since granular or soft cohesive soils may be encountered between boring locations; the Contractor should verify the subsurface conditions between boring locations or add a contingency.

The N_t values estimated for the cohesive soils encountered in the tunnel zone are presented in Table 2. N_t cannot be determined for granular soils. We also estimated the maximum settlements [caused by volume loss if a slurry face machine (SFM) or earth pressure balance tunnel boring machine (EPB) is NOT used] at the proposed tunnel location and the results are included in Table 2.

Table 2. Tunnel Face Stability Factor and Estimated Settlements along Tunnel Alignment

Soil Boring	Tunnel Invert Depth (ft) (1)	Elevation (ft)	Anticipated Soil Types in Tunnel Zone	Stability Factor N_t	S_{max} (in)	Note/Suggestion
B-1	18	34	7'-28': soft to very stiff very high plasticity Fat Clay (CH)	1.8	0.15	Small creep and swelling ground, suggest using earth pressure balance tunnel boring machine (EPB)
B-2	18	34	7'-28': firm to very stiff very high plasticity Fat Clay (CH)	2.6	0.14	Small creep and swelling ground, suggest using earth pressure balance tunnel boring machine (EPB)

Note: (1) Tunnel invert depth is measured from the top of levee based on the updated Plan and Profile Drawings provided by SJRA; AEC should be notified if the pipe invert depth is different so that our estimated can be updated accordingly.

(2) S_{max} = Estimated settlement along the tunnel alignment due to volume loss if slurry face machine (SFM) or EPB are not used; not including consolidation settlement.

It should be noted that the estimated settlement at the locations of Borings B-1 and B-2 is 0.14 to 0.15 inches (which does not include consolidation settlement) or more, as indicated on Table 2, and dewatering in the vicinity of Borings B-1 and B-2 will cause additional settlement due to increases in effective stress of the soil strata. The information in this report should be reviewed so that appropriate tunneling equipment and operation can be planned and factored into the construction plan and cost estimate. Plate D-9 in Appendix D provides a general guideline for Tunnel Boring Machine (TBM) selection. AEC recommends that the siphon designer verify the tolerable settlement for the ground surface at FM 2100. We suggest that the tunnel construction consider the use of: (i) a close face EPB; (ii) jet grout to stabilize the saturated granular soils, if any; or (iii) micro-tunneling. However, the choice of tunneling machine will be selected by the Contractor. Tunnel construction should be in accordance with Section 31 71 00.01 of the SJRA Specification, or Section 02425 of the 2012 COHSCS, whichever is more stringent.

5.2.3.3 Influence of Tunneling on Existing Structures

Based on Borings B-1 and B-2 and the assumed invert depth of the tunnel, we estimated the resulting influence zones (extending from the centerline of the tunnel) to be approximately 16.3 to 17.5 feet; although the values of tunnel influence zone presented are rough estimates. The estimated maximum settlements [caused by volume loss if a SFM or EPB is not used] along the tunnel alignment at the proposed tunnel locations are included in Table 2. The tunnel influence zone is assumed to extend a distance of about 2.5i (where i is the trough width) from the center of the auger tunnel, as shown on Plate D-8, in Appendix D. AEC emphasizes that the size of the influence zone of a tunnel is difficult to determine because several factors influence the response of the soil to tunneling operations including type of soil, ground water level and control method, type of tunneling equipment, tunneling operations, experience of operator, and other construction in the vicinity. Methods to prevent movement and/or distress to existing structures will require the services of a specialty contractor.

5.2.4 Measures to Reduce Distress from Tunneling

To control tunneling face loss and reduce potential impact on existing foundations and structures, AEC recommends the use of a steel casing (or equivalent methods) to support the tunnel excavation during tunnel construction. Considering the ground conditions discussed in Table 2, AEC recommends that the following tunneling operations be considered: (i) use a close face EPB and keep the pressure at least equal to if not greater than the combined soil and groundwater pressure in the ground at the tunnel level; (ii) if the contractor selects bore and jack operation, boring and jacking steel casing should be performed simultaneously to minimize the soil loss outside the steel casing; ground movement along the tunnel zone should be monitored during tunneling operation; and (iii) if excessive voids occur during tunneling, the contractor should immediately and completely grout the annular space between the steel casing and the ground at the tail of the machine, in accordance with Section 31 71 02.02 of the SJRA Specification, or Section 02431 of the 2012 COHSCS, whichever is more stringent. It should be noted that grouting may increase friction resistance while advancing the casing and the contractor will need to address this condition as part of his tunnel work plan. Plate D-10, in Appendix D, provides a general guideline for selection of grouting material. The tunneling machine selection, tunneling operation, and grouting (as necessary) will be the full responsibility of the Contractor.

To reduce the potential for the tunneling to influence existing foundations or structures, we recommend that the outer edge of the influence zone of the tunnel be a minimum of 5 feet from the outer edge of the bearing (stress) zone of existing foundations. The bearing (stress) zone is defined by a line drawn downward from the outer edge of an existing foundation and inclined at an angle of 45 degrees to the vertical.

We recommend that the following situations be evaluated on a case by case basis, where:

1. tunneling cannot be located farther than the minimum distance recommended above;
2. tunneling cannot be located outside the stress zone of the foundations for existing structures;
3. unstable soils are encountered near existing structures;
4. heavily loaded or critical structures are located close to the influence zone of the tunnels;

As an option, existing structure foundations should be protected by adequate shoring or strengthened by underpinning or other techniques, provided that tunneling cannot be located outside the stress zone of the existing foundations.

Disturbance and loss of ground from the tunneling operation may create surface soil disturbance and subsidence which in turn may cause distress to existing structures (including underground utilities and pavements) located in the zone of soil disturbance. Any open-cut excavation in the proposed tunneling areas should be adequately shored.

5.2.5 Monitoring Existing Structures

The Contractor should be responsible for monitoring existing structures nearby and taking necessary action to mitigate impact to adjacent structures. Existing structures located close to the proposed construction excavations should be surveyed prior to construction and pre-existing conditions of such structures and their vicinity be adequately recorded. This can be accomplished by conducting a pre-construction survey, taking photographs and/or video, and documenting existing elevations, cracks, settlements, and other existing distress in the structures. The monitoring should include establishment of elevation monitor stations, crack gauges, and inclinometers, as required. The monitoring should be performed before, periodically during, and after construction. The data should be reviewed by qualified engineers in a timely manner to evaluate the impact on existing structures and develop plans to mitigate the impact, should it be necessary.

5.3 Siphon Inlet/Outlet Structures

Based on the updated Plan and Profile Drawings provided by SJRA to AEC on August 5, 2015, the existing inlet/outlet structures will be abandoned in-place and replaced with new inlet/outlet structures. To accommodate TxDOT’s plans to widen the existing FM 2100 ROW from 100 feet to 120 feet, the new inlet headwall will be located approximately 40 feet east of the existing ROW boundary while the new outlet headwall will be located approximately 20 feet west of the ROW boundary. The inlet/outlet structures will be reinforced concrete cantilever walls and supported by wall footings founded at a depth of approximately 21 feet (i.e. Elevation = 32.0 feet) below the top of head wall. Design of the siphon structure headwalls and wingwalls should consider the allowable bearing capacity of the foundation soils, sliding, and overturning stability. We recommend using a FS of 2 for passive earth pressure, a FS of 1.5 for sliding, and a FS of 2 for overturning stability of the walls.

5.3.1 Allowable Bearing Capacity

Based on Borings B-1 and B-2, a wall footing at a depth of at 21 feet (i.e. Elevation = 32 feet) below the top of headwall/levee can be designed for an allowable net bearing capacity of 2,000 psf for sustained loads and 3,000 psf for total loads. These allowable bearing pressures include a minimum FS of 3 for sustained loads and 2 for total loads, whichever is more critical should be used for design.

Based on our borings, the top 10 feet of the subsurface soil conditions at the inlet and outlet structures consist of soft to firm fat clay fill. Foundation construction and excavation should be protected by adequate shoring. Recommendations for excavation stability and interlocked sheet pile are presented in Sections 5.2.2.1 and 5.2.2.2 of this report, respectively.

Based on the groundwater encountered in Borings B-1 and B-2, groundwater control may be required during the headwall construction. Dewatering guidelines are presented in Section 6.2 of this report. AEC also recommends that scour protection (such as riprap) be provided for the footings of headwalls and wingwalls.

Foundation Settlement: A detailed settlement analysis is beyond the scope of this investigation. Based on the soil conditions encountered, we estimate that the retaining wall footings, designed and constructed as recommended in this report, will experience total settlements on the order of 1 inch.

5.3.2 Hydrostatic Uplift

Since the siphon inlet/outlet structures will typically remain submerged, AEC recommends that the siphon structure be assumed to be continually submerged and subjected to hydrostatic uplift forces. We recommend that the design water level be assumed to be at the top of wall or 100-year flood elevation, whichever is more critical. If the dead weight of the structure (including the backfill on top of the pipe) and the frictional resistance between the wall and backfill are inadequate to resist uplift forces, the width of the wall footing and the wall thickness can be increased to provide additional uplift resistance. The buoyant unit weight of concrete can be taken as 90 pcf. The minimum recommended factors of safety against uplift should be 1.1 for concrete weight, 1.5 for soil weight and 3.0 for soil friction. Wall design should consider short-term and long-term conditions, whichever is critical. Design soil parameters are presented on Plate C-1, in Appendix C. Recommended design criteria for uplift resistance are shown on Plate D-11, in Appendix D.

5.3.3 Lateral Earth Pressures

The head walls and wing walls will be subjected to lateral earth pressures. The magnitudes of the lateral earth pressures will depend on the type and density of the backfill, surcharge on the backfill and hydrostatic pressure. If the backfill is over-compacted or if highly plastic clays are placed behind the walls, the lateral earth pressure could exceed the vertical pressure. Lateral pressure resulting from construction equipment, and traffic, or other surcharge on the top of the walls should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. Hydrostatic pressure should also be included, unless adequate drainage is provided behind the walls.

Wall design should consider short-term and long-term conditions, whichever is critical. AEC assumes that the concrete headwalls and wingwalls of the inlet/outlet structures will not be allowed to move, and should be designed based on at-rest earth pressures. The at-rest earth pressure at depth z can be determined by Equation (8). Design soil parameters are presented on Plate C-1, in Appendix C.

$$p_0 = (q_s + \gamma\, h_1 + \gamma'\, h_2)\, K_0 + \gamma_w h_2 \qquad \text{...........Equation (8)}$$

where, p_0 = at-rest earth pressure, psf.
q_s = uniform surcharge pressure, psf.
γ, γ' = wet and buoyant unit weights of soil, pcf.
h_1 = depth from ground surface to ground water table, feet.
h_2 = $z-h_1$, depth from ground water table to point under consideration, feet.
z = depth below ground surface, feet.
K_0 = coefficient of at-rest earth pressure.
γ_w = unit weight of water, 62.4 pcf.

Backfill Material: AEC anticipates that the inlet/outlet structures will be installed by open cut method. AEC recommends select fill or compacted cement stabilized sand (CSS) be used for the backfill behind the structures. Select fill criteria are presented in Section 5.5 of this report. CSS should be in accordance with Section of 31 32 13.16 of the SJRA Specification, or Section 02321 of the 2012 COHSCS, whichever is more stringent.

5.4 Slope Stability Analysis

AEC performed stability analysis on the existing levee slopes based on the four cross sections (CS-AA through DD) provided by SJRA. The CS locations and cross sections are presented on Plates B-1 and B-2, in Appendix B, respectively.

According to SJRA, water surface inside the canal is normally maintained at 1 foot freeboard (i.e. 1 foot below the top of levee). During the maintenance periods (when Lake Houston Pump Station is shut down), water inside the canal will be drained to the bottom of canal within 8 to 10 hours. Canal will also be dewatered during the construction of the new siphon pipes and headwall structures. Based on the Harris County Flood Insurance Rate Map (FIRM) (dated June 18, 2007) provided by SJRA to AEC, Siphon No.7 is within 100-year flood area zone; however, the flood water surface elevation (WSE) was not available at this time. AEC assumes that the 100 year flood water surface elevation be the same as the top of levee.

5.4.1 Conditions Analyzed for Slope Stability

Based on our borings, levee heights, and slope inclinations, AEC performed slope stability analysis on 4 of selected CSs. The slope stability analyses consider three different conditions: short-term, long-term, and rapid drawdown. AEC performed the stability analyses in accordance with Harris County Flood Control District (HCFCD) requirements.

Design Soil Parameters and Profiles: Soil parameters used in the analyses include wet unit weights, unconsolidated-undrained (UU) shear strengths, consolidated-drained (CD) shear strengths, and consolidated-undrained (CU) shear strengths. Predominately very high plasticity fat clay levee fill was encountered in our borings. Exposing these fat clays to the atmosphere and cycles of wetting-drying from seasonal moisture changes will result in desiccation, cracking, and progressive movement of these clays, and a reduction in their shear strengths. We considered the desiccation zone for fat clay from the top of levee or levee slope surface to the assumed seepage line in the existing levee. For fat clay within the desiccation zone, we used effective residual shear strengths of $c'_r = 65$ psf and $\phi'_r = 21$ degrees to evaluate slope stability for short-term, long-term, and rapid drawdown conditions. We also reduced the c' and c_{cu} of clay soils (with a PI greater than 20) within the non-desiccated zone based on our experience with similar levee projects in the Houston area (Reference 1).

Conditions Analyzed for Slope Stability: AEC used the Simplified Bishop Method of Slices option in the SLOPE/W computer program (Reference 2) to analyze slope stability for 2-dimensional limiting equilibrium. The program has the capability to compute pore water pressures based on a defined piezometric surface. For rapid drawdown conditions for the canal side of levee slopes, we considered that the water level drops from the maximum WSE, which is one foot below the top of levee, to the bottom of the levee; this models the condition of dewatering during maintenance or construction period at Siphon 7. For the rapid drawdown condition for the landside of levee slope, we considered that the water level drops from the top of levee to the bottom of landside levee; this models the condition where a 100-year flood event occurs and then the water level drops down quickly. For short term, long term, and rapid drawdown conditions, we considered that the seepage from the canal water has saturated the levee soil below the defined piezometric surface.

HCFCD requires a minimum safety factor (SF) of 1.3 for short-term conditions, 1.5 for long-term conditions, and 1.25 for rapid drawdown conditions. Stability analyses for the selected slopes were conducted for the short-term, long-term, and rapid drawdown conditions. A brief description of these conditions is presented below:

1. End-of-Construction Condition - This condition models rapid construction loading taking place, so that there is no time for the induced excess pore water pressure to dissipate or for consolidation to occur during the loading period. Unconsolidated-undrained shear strength parameters were used for this analysis.

2. Long-Term Condition - This condition models long-term steady seepage through embankments and the long-term stability of slopes in clays. Consolidated-drained effective stress shear strength parameters (obtained from CU triaxial tests with pore water pressure measurements) were used for this analysis.

3. Rapid Drawdown Condition - The majority of slope failures in the Harris County/Houston area occur under rapid drawdown conditions. This condition models when the slope becomes fully saturated and consolidated and is at equilibrium with the existing stress system, then encounters rapid drawdown and simultaneously allows no drainage to occur. Consolidated-undrained total stress shear strength with pore pressures parameters modeling rapid drawdown conditions were used for this analysis.

Slope Stability Analysis: We performed slope stability analyses on the 4 most critical CSs, which are the north embankment of CS-AA waterside slope, CS-BB waterside slope, CS-CC waterside slope, and CS-CC landside slope. Design soil parameters for the cross sections are presented on Plate E-1, in Appendix E. A 300 psf construction surcharge was added to the top of the slope for the short term condition and rapid drawdown condition for waterside slope only. The results of the slope stability analyses for the levee slopes under the short-term, long-term, and rapid drawdown conditions are presented in Appendix E (see Table 3). The SF for the levee slopes under short term, long term, and rapid drawdown conditions are summarized in Table 3.

Table 3. Existing Levee Slope Stability Analysis Results

CS, Inclination (H:V), and Boring	Minimum Factor of Safety		
	Short-Term	Long-Term	Rapid Drawdown
CS-AA (NB Waterside), H:V =3:1, Boring B-2	6.22 (GS, Plate E-2a) 2.56 (LS, Plate E-2b)	3.27 (GS, Plate E-3a) 3.84 (LS, Plate E-3b)	1.28 (GS, Plate E-4a) 1.56 (LS, Plate E-4b)
CS-BB (NB Waterside), H:V =3:1, Boring B-2	6.42 (GS, Plate E-5a) 2.29 (LS, Plate E-5b)	3.55 (GS, Plate E-6a) 4.13 (LS, Plate E-6b)	1.39 (GS, Plate E-7a) 1.68 (LS, Plate E-7b)
CS-CC (NB Waterside), Upper Slope H:V = 1.9:1, Boring B-1	4.95 (GS, Plate E-8a) 2.18 (LS, Plate E-8b)	3.26 (GS, Plate E-9a) 3.42 (LS, Plate E-9b)	1.31 (GS, Plate E-10a) 1.46 (LS, Plate E-10b)
CS-CC (NB Landside), H:V = 3.1:1, Boring B-1	3.65 (GS, Plate E-11a) 1.93 (LS, Plate E-11b)	1.96 (GS, Plate E-12a) 2.41 (LS, Plate E-12b)	1.61 (GS, Plate E-13a) 2.37 (LS, Plate E-13b)

Notes: (1) NB = North Bank Levee; GS = Global Slide; LS = Local Slide.
(2) CSs are presented on Plates B-1 and B-2, in Appendix B.

Based on the summary in Table 3, the resultant SFs for the existing slope of the selected critical sections meet HCFCD guidelines under short-term, long-term, and rapid drawdown conditions.

5.5 Select Fill

5.5.1 Backfill for Headwall Structures or Retaining Walls

Select fill should consist of uniform, non-dispersive, non-active inorganic lean clays with a PI between 10 and 20 percent, and more than 50 percent passing a No. 200 sieve. Excavated or imported material delivered to the site for use as select fill shall not have clay clods with PI greater than 20, clay clods greater than 2 inches in diameter, or contain sands/silts with PI less than 10. Prior to construction, the Contractor should determine if he or she can obtain qualified select fill meeting the above select fill criteria.

As an alternative to imported fill, on-site soils excavated during construction can be stabilized with hydrated lime. Excavated clay soils should be stabilized with at least 7 percent hydrated lime by dry soil weight. Lime stabilization shall be performed in accordance with Section 32 11 13.13 of the SJRA Specification, or Section 02336 of the 2012 COHSCS, whichever is more stringent. The percentage of lime required for stabilization is a preliminary estimate for planning purposes only; laboratory testing should be performed to determine optimum contents for stabilization prior to construction. AEC prefers using stabilized on-site clay as select fill since compacted lime-stabilized clay generally has high shear strength, low compressibility, and relatively low permeability. Blended or mixed soils (sand and clay) should not be used as select fill.

All material intended for use as select fill should be tested prior to use to confirm that it meets select fill criteria. The fill should be placed in loose lifts not exceeding 8 inches in thickness. Backfill within 3 feet of walls or columns should be placed in loose lifts no more than 4-inches thick and compacted using hand tampers, or small self-propelled compactors. The lime-stabilized onsite soils or select fill should be compacted to a minimum of 95 percent of the ASTM D 698 (Standard Proctor) maximum dry unit weight at a moisture content ranging between optimum and 3 percent above optimum.

5.5.2 Backfill for Embankment

If necessary, select fill for the re-graded slopes should consist of sandy lean clay or lean clay meeting the requirements of HCFCD's Standard Specifications Book (2005), Section 02314, Item 2.1 "Imported Select Fill Material". The select fill should be free from roots, trash, organic matter and other objectionable materials. The select fill should be non-dispersive sandy lean clay or lean clay, with a maximum LL of 49, a PI between 15 and 30, and between 60 and 85 percent of the material passing a No. 200 sieve. The select fill should be placed in loose lifts not exceeding 8 inches in thickness. Heavy compaction equipment and excessive equipment passes should be avoided within 3 feet of existing structures or wall. Backfill within 3 feet of structures/walls should be placed in loose lifts no more than 4 inches thick and compacted using portable compaction equipment such as hand tampers, etc. The fill should be compacted to a minimum 95 percent of the ASTM D 698 maximum dry unit weight and at moisture content within optimum and 3 percent wet of optimum.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation

To mitigate site problems that may develop following prolonged periods of rainfall, it is essential to have adequate drainage to maintain a relatively dry and firm surface prior to starting any work at the site. Adequate drainage should be maintained throughout the construction period. Methods for controlling surface runoff and ponding include proper site grading, berm construction around exposed areas, and installation of sump pits with pumps.

6.2 Groundwater Control

The need for groundwater control will depend on the depth of excavation relative to the groundwater depth at the time of construction. In the event that there is heavy rain prior to or during construction, the groundwater table may be higher than indicated in this report; higher seepage is also likely and may require a more extensive groundwater control program. In addition, groundwater may be pressurized in certain areas of the alignment, requiring further evaluation and consideration of the excess hydrostatic pressures.

The Contractor should be responsible for selecting, designing, constructing, maintaining and monitoring a groundwater control system and adapt his operations to ensure the stability of the excavations. Groundwater information presented in Section 4.1 and elsewhere in this report, along with consideration for potential environmental and site variation between the time of our field exploration and construction, should be incorporated in evaluating groundwater depths. The following recommendations are intended to guide the Contractor during design and construction of the dewatering system.

In cohesive soils seepage rates are lower than in granular soils and groundwater is usually collected in sumps and channeled by gravity flow to storm sewers. If cohesive soils contain significant secondary features, seepage rates will be higher. This may require larger sumps and drainage channels, or if significant granular layers are interbedded within the cohesive soils, methods used for granular soils may be required. Where it is present, pressurized groundwater will also yield higher seepage rates.

Groundwater for excavations within saturated sands can be controlled by the installation of wellpoints. The practical maximum dewatering depth for well points is about 15 feet. When groundwater control is required below 15 feet, multiple staged wellpoint or educator and ejector-type system have generally proved successful. Generally, the groundwater depth should be lowered at least 5 feet below the excavation bottom to be able to work on a firm surface when water-bearing granular soils are encountered. Another groundwater control option is to use water-tight sheet pile cutoff walls to seal off water bearing sand/silt layers (see Section 5.2.2.2 of this report).

Extended and/or excessive dewatering can result in settlement of existing structures in the vicinity; the Contractor should take the necessary precautions to minimize the effect on existing structures in the vicinity of the dewatering operation. We recommend that the Contractor verify the groundwater depths and seepage rates prior to and during construction and retain the services of a dewatering expert (if necessary) to assist him in identifying, implementing, and monitoring the most suitable and cost-effective method of controlling groundwater.

For open cut construction in cohesive soils, the possibility of bottom heave must be considered due to the removal of the weight of excavated soil. In lean and fat clays, heave normally does not occur unless the ratio of Critical Height to Depth of Cut approaches one. In silty clays, heave does not typically occur unless an artificially large head of water is created through the use of impervious sheeting in bracing the cut. Guidelines for evaluating bottom stability are presented in Section 5.2.2.1 of this report.

6.3 Construction Monitoring

Excavation, bedding, and backfilling of underground utilities should be monitored by qualified geotechnical professionals to check for compliance with project documents and changed conditions, if encountered. AEC should be allowed to review the design and construction plans and specifications prior to release to check that the geotechnical recommendations and design criteria presented herein are properly interpreted.

6.4 Monitoring of Existing Structures

Existing structures in the vicinity of the proposed alignment should be closely monitored prior to, during, and for a period after excavation. Several factors (including soil type and stratification, construction methods, weather conditions, other construction in the vicinity, construction personnel experience and supervision) may impact ground movement in the vicinity of the alignment. We therefore recommend that the Contractor be required to survey and adequately document the condition of existing structures in the vicinity of the proposed alignments.

7.0 GENERAL

AEC should be allowed to review the design and construction plans and specifications prior to release to check that the geotechnical recommendations and design criteria presented herein are properly interpreted.

The information contained in this report summarizes conditions found on the date the borings were drilled. The attached boring logs are true representation of the soils encountered at the specific boring locations on the date of drilling. Due to variations encountered in the subsurface conditions across the site, changes in soil conditions from those presented in this report should be anticipated. AEC should be notified immediately when conditions encountered during construction are significantly different from those presented in this report.

8.0 LIMITATIONS

The investigation was performed using the standard level of care and diligence normally practiced by recognized geotechnical engineering firms in this area, presently performing similar services under similar circumstances. The report has been prepared exclusively for the project and location described in this report, and is intended to be used in its entirety. If pertinent project details change or otherwise differ from those described herein, AEC should be notified immediately and retained to evaluate the effect of the changes on the recommendations presented in this report, and revise the recommendations if necessary. The recommendations presented in this report should not be used for other structures located at this site or similar structures located at other sites, without additional evaluation and/or investigation.

REFERENCES

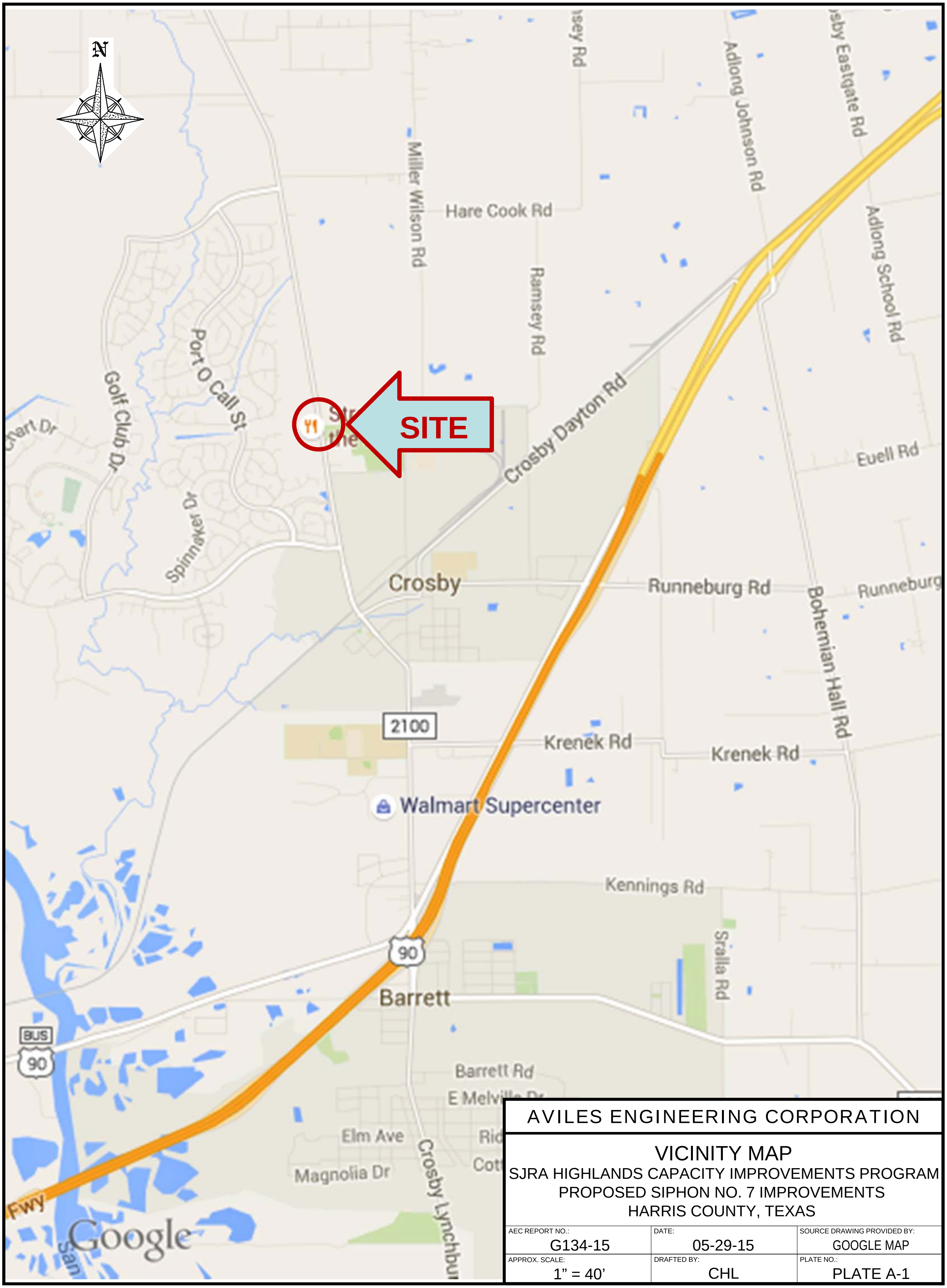
1. Mesri, G., and Abdel-Ghaffar, M.E.M., “Cohesion Intercept in Effective Stress-Stability Analysis”, *Journal of Geotechnical Engineering*, Vol. 119, No. 8, August 1993, pg 1229-1249.

2. GEO-SLOPE International Ltd., 2004, “SLOPE/W for Slope Stability Analysis”, Version 6, Calgary, Alberta, Canada.

3. Holtz, R.D., and W.D. Kovacs, 1981, “An Introduction to Geotechnical Engineering”, Prentice-Hall, Englewood Cliff, New Jersey.

APPENDIX A

Plate A-1	Vicinity Map
Plate A-2	Boring Location Plan
Plates A-3 and A-4	Boring Logs
Plate A-5	Key to Symbols
Plate A-6	Classification of Soils for Engineering Purposes
Plate A-7	Terms Used on Boring Logs
Plate A-8	ASTM & TXDOT Designation for Soil Laboratory Tests
Plate A-9	Sieve and Hydrometer Analysis Test Results
Plate A-10	Crumb Test Results
Plates A-11 thru A-14	CU Test Results



AVILES ENGINEERING CORPORATION

VICINITY MAP

SJRA HIGHLANDS CAPACITY IMPROVEMENTS PROGRAM
PROPOSED SIPHON NO. 7 IMPROVEMENTS
HARRIS COUNTY, TEXAS

AEC REPORT NO.: G134-15	DATE: 05-29-15	SOURCE DRAWING PROVIDED BY: GOOGLE MAP
APPROX. SCALE: 1” = 40’	DRAFTED BY: CHL	PLATE NO.: PLATE A-1

NOTE: BORING LOCATIONS ARE APPROXIMATE

AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN

SJRA HIGHLANDS CAPACITY IMPROVEMENTS PROGRAM
PROPOSED SIPHON NO. 7 IMPROVEMENTS
HARRIS COUNTY, TEXAS

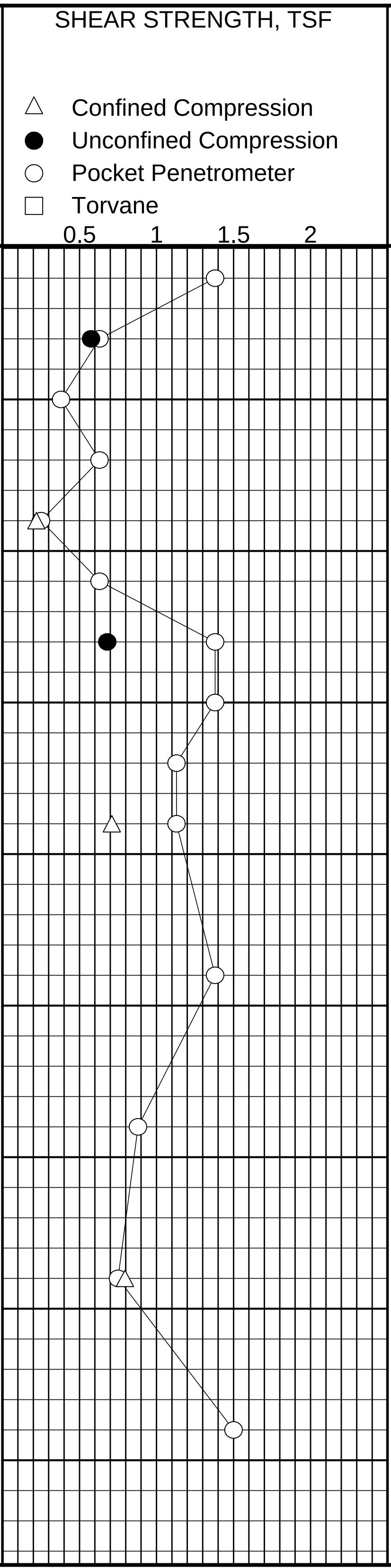
AEC REPORT NO.: G134-15	DATE: 06-09-15	SOURCE DRAWING PROVIDED BY: GOOGLE EARTH PRO.
APPROX. SCALE: 1" = 40'	DRAFTED BY: CHL	PLATE NO.: PLATE A-2

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: SJRA Main Canal Siphon No. 7 Improvements
BORING **B-1**

DATE **6/18/15**
TYPE **4" Dry Auger**
LOCATION **See Boring Location Plan**

DESCRIPTION Surface Elevation (feet): 52.30	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Fill: soft to very stiff, dark gray Fat Clay (CH), with slickensides		35					
-with gravel 0'-6', and roots 0'-2'		41	81	97	96	27	69
-with ferrous nodules 4'-6'		47					
		42			90	29	61
-light gray and dark gray 8'-10'		38	83				
Stiff to very stiff, tan and gray Fat Clay (CH), with slickensides		27					
		28	96	100	75	25	50
-with ferrous nodules 14'-16'		29					
		29					
		30	96		79	23	56
-boring cave in at 21' after 24 hours							
-with ferrous nodules 23'-25'		26					
Stiff to very stiff, tan and gray Lean Clay w/ Sand (CL), with silt partings and ferrous nodules		18					
-with silty sand pockets 28'-30'							
		18	114	81	31	13	18
		17					
Termination depth = 40 feet.							



DEPTH IN FEET: 0, 6, 12, 18, 24, 30, 36, 42

BORING DRILLED TO 40 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 39 FEET WHILE DRILLING ▽
WATER LEVEL AT 21 FEET AFTER 24 HRS ▼
DRILLED BY V&S DRAFTED BY CHL LOGGED BY CHL

PROJECT NO. G134-15

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: SJRA Main Canal Siphon No. 7 Improvements

BORING **B-2**

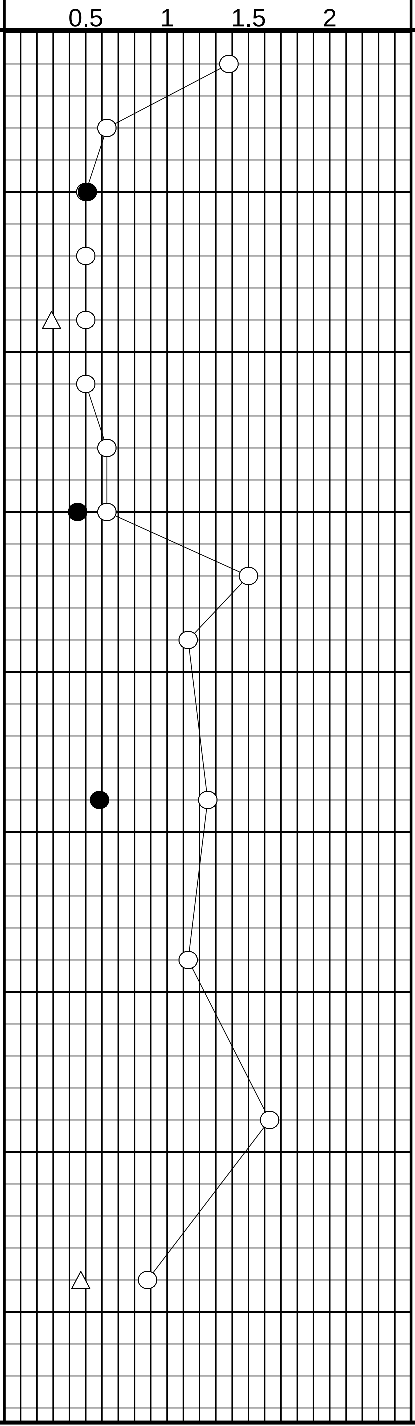
DATE **6/18/15** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

Surface Elevation (feet): 50.40

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
☐ Torvane

DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Fill: firm to very stiff, dark gray Fat Clay (CH), with slickensides -with roots 0'-2' -with ferrous nodules 2'-4'		35					
		35					
		38	83	98	88	27	61
		45					
Firm to very stiff, tan and gray Fat Clay (CH), with slickensides -with calcareous nodules 10'-12', and ferrous nodules 10'-16'		41	81		96	29	67
		41					
		35		98	90	28	62
					67	22	45
		32	92				
		28					
		28					
-boring cave in at 23.1' after 24 hours							
		30	93		74	25	49
		32					
Firm to very stiff, gray and tan Lean Clay w/ Sand (CL), with abundant sand seams and partings -with fat clay seams 33'-35'		23					
				77	26	17	9
		20	111				
Termination depth = 40 feet.							



BORING DRILLED TO **40** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **39** FEET WHILE DRILLING
WATER LEVEL AT **23.1** FEET AFTER **24 HRS**
DRILLED BY **V&S** DRAFTED BY **CHL** LOGGED BY **CHL**

PROJECT NO. G134-15

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
▩	Fill
▨	High plasticity clay
▨	Low plasticity clay
Misc. Symbols	
∇	Water table depth during drilling
▼	Subsequent water table depth
○	Pocket Penetrometer
●	Unconfined Compression
△	Confined Compression
Soil Samplers	
■	Undisturbed thin wall Shelby tube

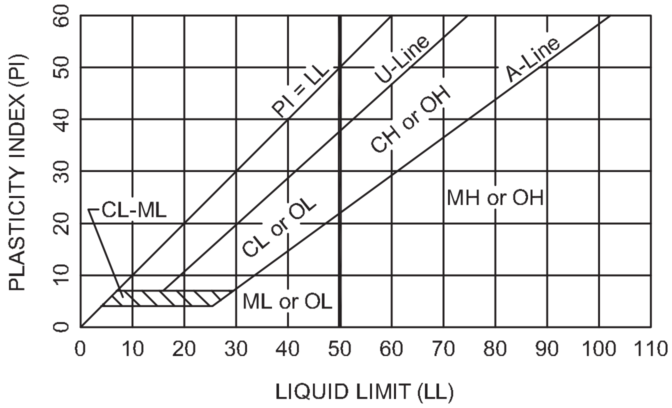
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation D-2487

MAJOR DIVISIONS				GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (Less than 50% of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)		GW	Well-graded gravel, well-graded gravel with sand
				GP	Poorly-graded gravel, poorly-graded gravel with sand
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	GM	Silty gravel, silty gravel with sand
			Limits plot above "A" line & hatched zone on plasticity chart	GC	Clayey gravel, clayey gravel with sand
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)		SW	Well-graded sand, well-graded sand with gravel
				SP	Poorly-graded sand, poorly-graded sand with gravel
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	SM	Silty sand, silty sand with gravel
			Limits plot above "A" line & hatched zone on plasticity chart	SC	Clayey sand, clayey sand with gravel
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS AND CLAYS (Liquid Limit Less Than 50%)			ML	Silt, silt with sand, silt with gravel, sandy silt, gravelly silt
				CL	Lean clay, lean clay with sand, lean clay with gravel, sandy lean clay, gravelly lean clay
				OL	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt
	SILTS AND CLAYS (Liquid Limit 50% or More)			MH	Elastic silt, elastic silt with sand, sandy elastic silt, gravelly elastic silt
				CH	Fat clay, fat clay with sand, fat clay with gravel, sandy fat clay, gravelly fat clay
				OH	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt

NOTE: Coarse soils between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone of the plasticity chart are to have dual symbols.

PLASTICITY CHART



Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

Equation of U-Line: Vertical at LL=16 to PI=7, then PI=0.9(LL-8)

DEGREE OF PLASTICITY OF COHESIVE SOILS

Degree of Plasticity	Plasticity Index
None	0 - 4
Slight	5 - 10
Medium	11 - 20
High	21 - 40
Very High	>40

SOIL SYMBOLS

▦	Fill	░	Sand
▨	Clay (CH)	▥	Silt
▧	Clay (CL)		

TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

6" 3" 3/4" #4 #10 #40 #200

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

152 76.2 19.1 4.76 2.00 0.420 0.074 0.002

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength, Kips per Sq. ft.
Very Soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	greater than 4.00

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

Very Loose	<4 bpf
Loose	5-10 bpf
Medium Dense	11-30 bpf
Dense	31-50 bpf
Very Dense	>50 bpf

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows per Foot	Description
25	25 blows driving sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows driving sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows driving sampler 3 inches, during initial 6-inches seating interval.

NOTE: To avoid change to sampling tools, driving is limited to 50 blows during or after seating interval.

DRY STRENGTH ASTM D2488

None	Dry specimen crumbles into powder with mere pressure of handling
Low	Dry specimen crumbles into powder with some finger pressure
Medium	Dry specimen breaks into pieces or crumbles with considerable pressure
High	Dry specimen cannot be broken with finger pressure, it can be broken between thumb and hard surface
Very High	Dry specimen cannot be broken between thumb and hard surface

MOISTURE CONDITION ASTM D2488

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy. The degree of slickensidedness depends upon the spacing of slickensides and the easiness of breaking along these planes.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil types.
Interlayered	Soil sample composed of alternating layers of different soil types.
Intermixed	Soil sample composed of pockets of different soil types and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of calcium material.

ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

NAME OF TEST	ASTM TEST DESIGNATION	TXDOT TEST DESIGNATION
Moisture Content	D 2216	Tex-103-E
Specific Gravity	D 854	Tex-108-E
Sieve Analysis	D 421 D 422	Tex-110-E (Part 1)
Hydrometer Analysis	D 422	Tex-110-E (Part 2)
Minus No. 200 Sieve	D 1140	Tex-111-E
Liquid Limit	D 4318	Tex-104-E
Plastic Limit	D 4318	Tex-105-E
Shrinkage Limit	D 427	Tex-107-E
Standard Proctor Compaction	D 698	Tex-114-E
Modified Proctor Compaction	D 1557	Tex-113-E
Permeability (constant head)	D 2434	-
Consolidation	D 2435	-
Direct Shear	D 3080	-
Unconfined Compression	D 2166	-
Unconsolidated-Undrained Triaxial	D 2850	Tex-118-E
Consolidated-Undrained Triaxial	D 4767	Tex-131-E
Pinhole Test	D 4647	-
California Bearing Ratio	D 1883	-
Unified Soil Classification System	D 2487	Tex-142-E

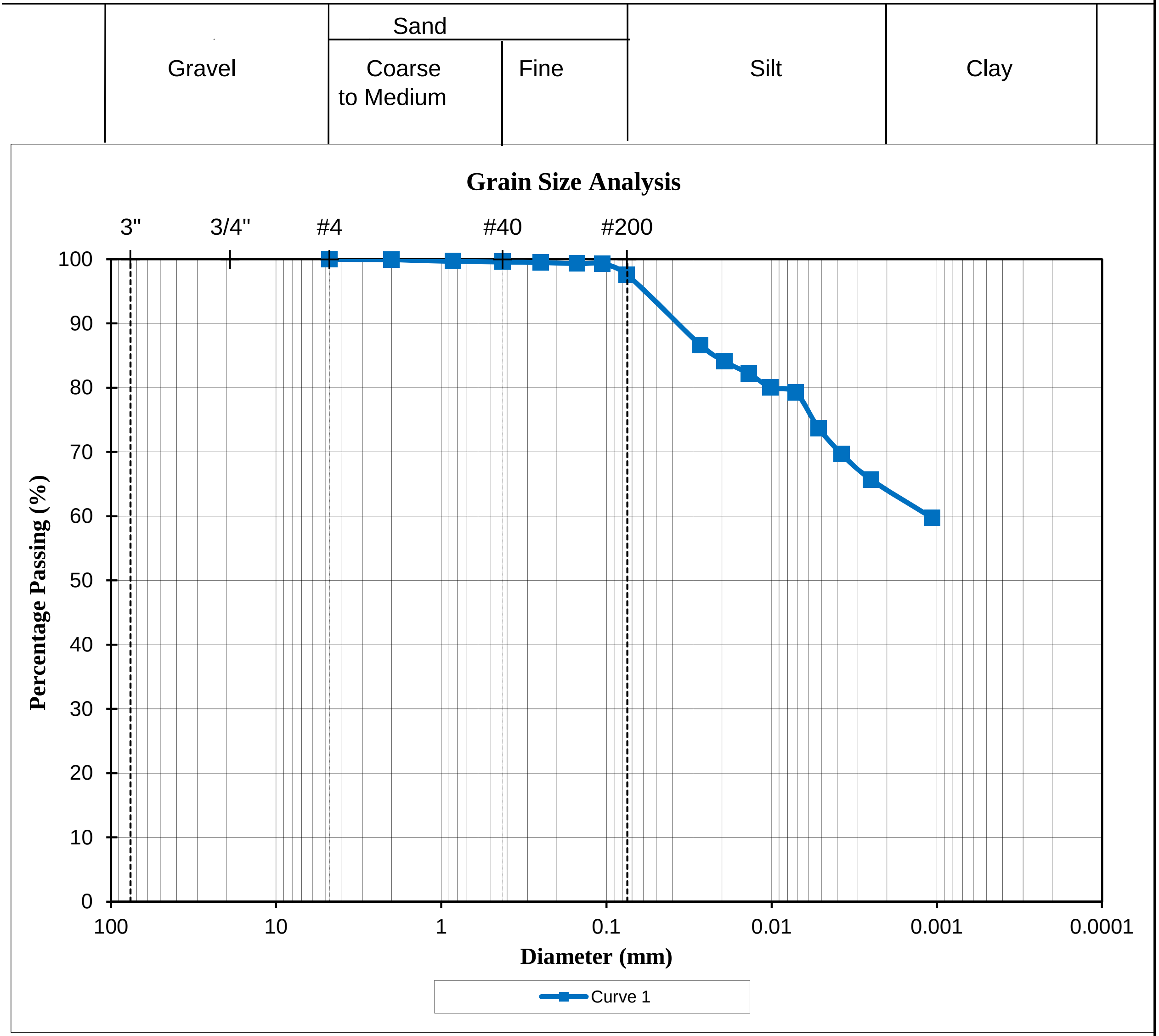
AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE & HYDROMETER

Project : SJRA Siphon No. 7 Improvements
Location of Project: Harris County, Texas

Job No.: G134-15
Date of Testing: 6/11/2015



Curve	**Boring**	**Depth (ft)**	**Soil Description**	**Cu**	**Cc**	**D_{50} (mm)**
1	B-2	4-6	Fill: Fat Clay (CH)	N/A	N/A	N/A

PLATE A-9

AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

RESULTS OF CRUMB TESTS (ASTM D 6572)

Project Name: SJRA Siphon No. 7 Improvements
Project No.: G134-15 Test Date: 6/10/2015

Boring Number	Depth, feet	Crumbling Characteristics				Crumb Test Classification	
		Crumbles? (Y/N)	If soil crumbles, rate of crumbling			@15 minutes	@30 minutes
			Minor	Moderate	Immediate		
B-1	4-6	N				1	1
B-2	8-10	N				1	1

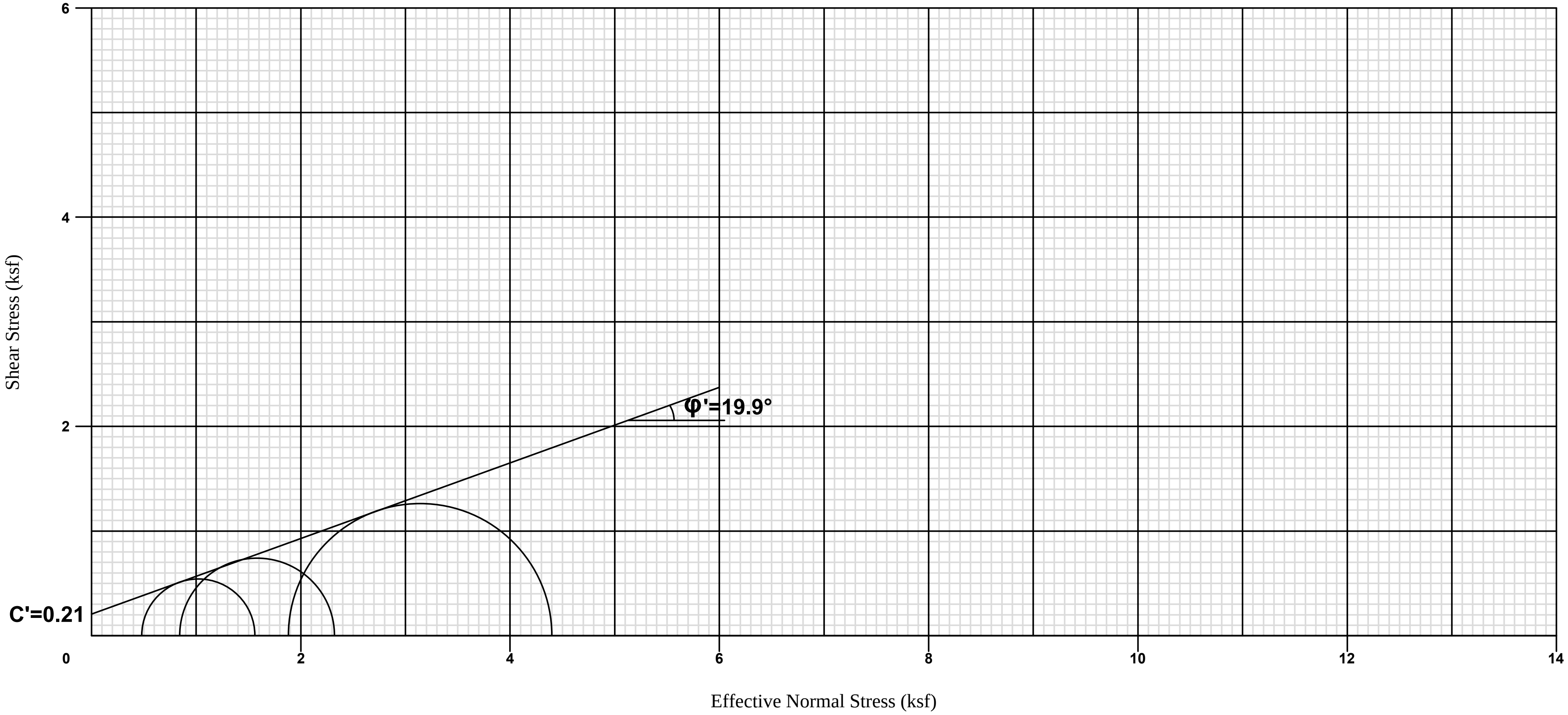
Results interpretation:

1	No sign of cloudy water caused by colloidal suspension
2	Bare hint of colloidal cloud formation at surface or soil crumb
3	Easily recognized colloidal cloud covering at least 1/4 to 1/2 of the bottom of the glass container
4	Strong reaction with colloidal cloud covering most of the bottom of the glass container

Olive-gray and gray FAT CLAY (CH)

LL = 90, PL = 29, PI = 61

W_n = 41.53%, r_d = 80.6 pcf, e_o = 1.129

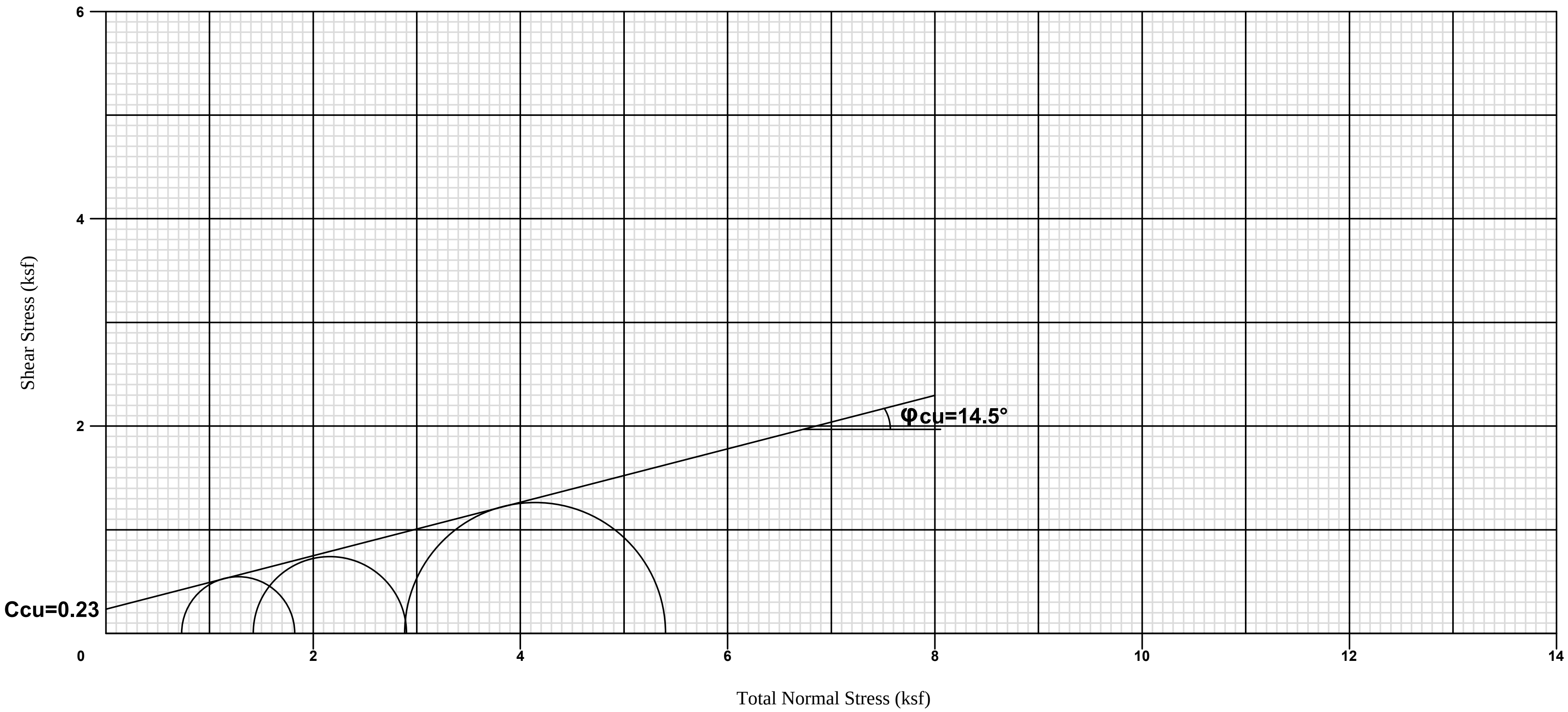


TRIAXIAL CU TESTS
MOHR'S CIRCLES
Effective Stress
G134-15 Boring B-1, 6'-8'

Olive-gray and gray FAT CLAY (CH)

LL = 90, PL = 29, PI = 61

W_n = 41.53%, r_d = 80.6 pcf, e_o = 1.129



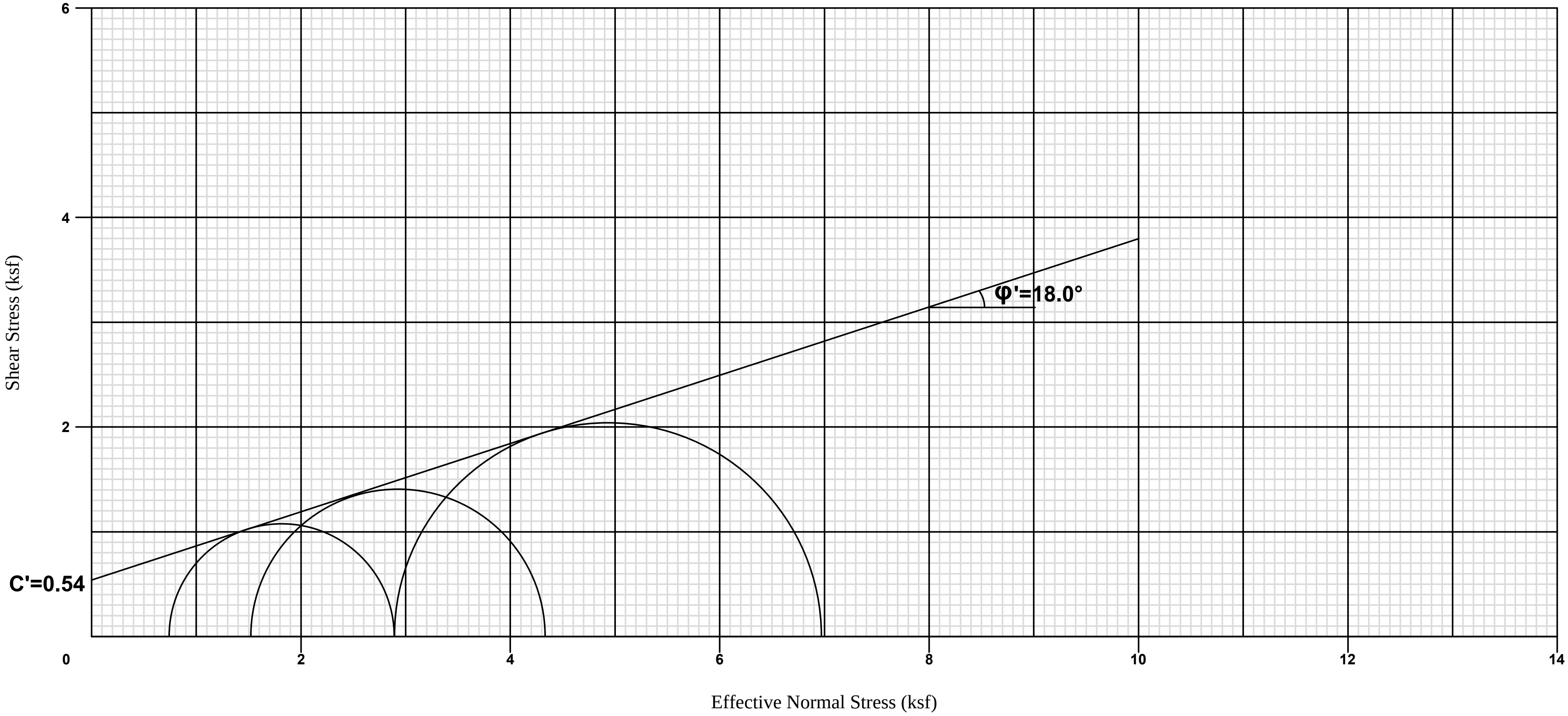
TRIAXIAL CU TESTS
MOHR'S CIRCLES
Total Stress
G134-15 Boring B-1, 6'-8'

PLATE A-12

Tan and light gray FAT CLAY (CH)

LL = 74, PL = 25, PI = 49

W_n = 31.55%, r_d = 93.8 pcf, e_o = 0.829

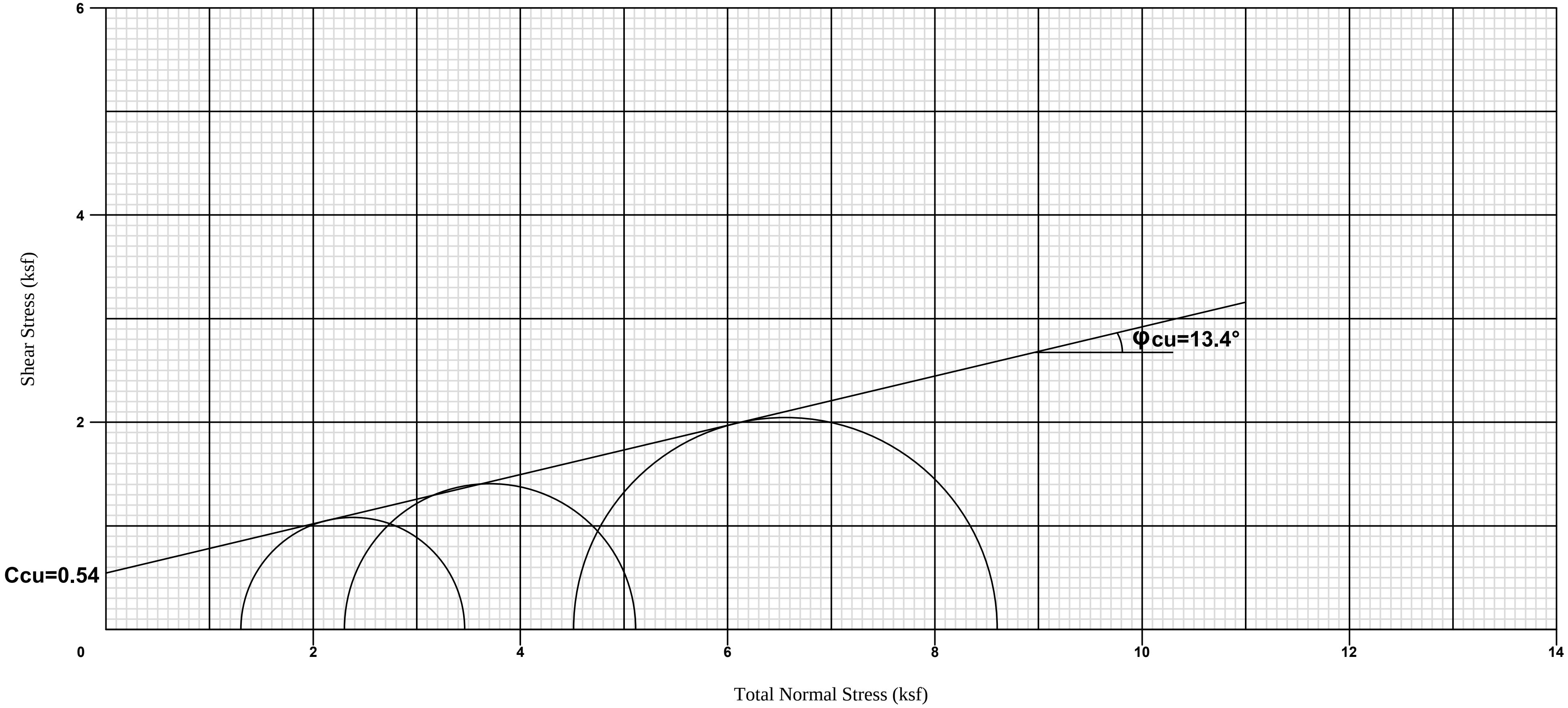


TRIAXIAL CU TESTS
MOHR'S CIRCLES
Effective Stress
G134-15 Boring B-1, 12'-14'

Tan and light gray FAT CLAY (CH)

LL = 74, PL = 25, PI = 49

Wn = 31.55%, rd = 93.8 pcf, eo = 0.829

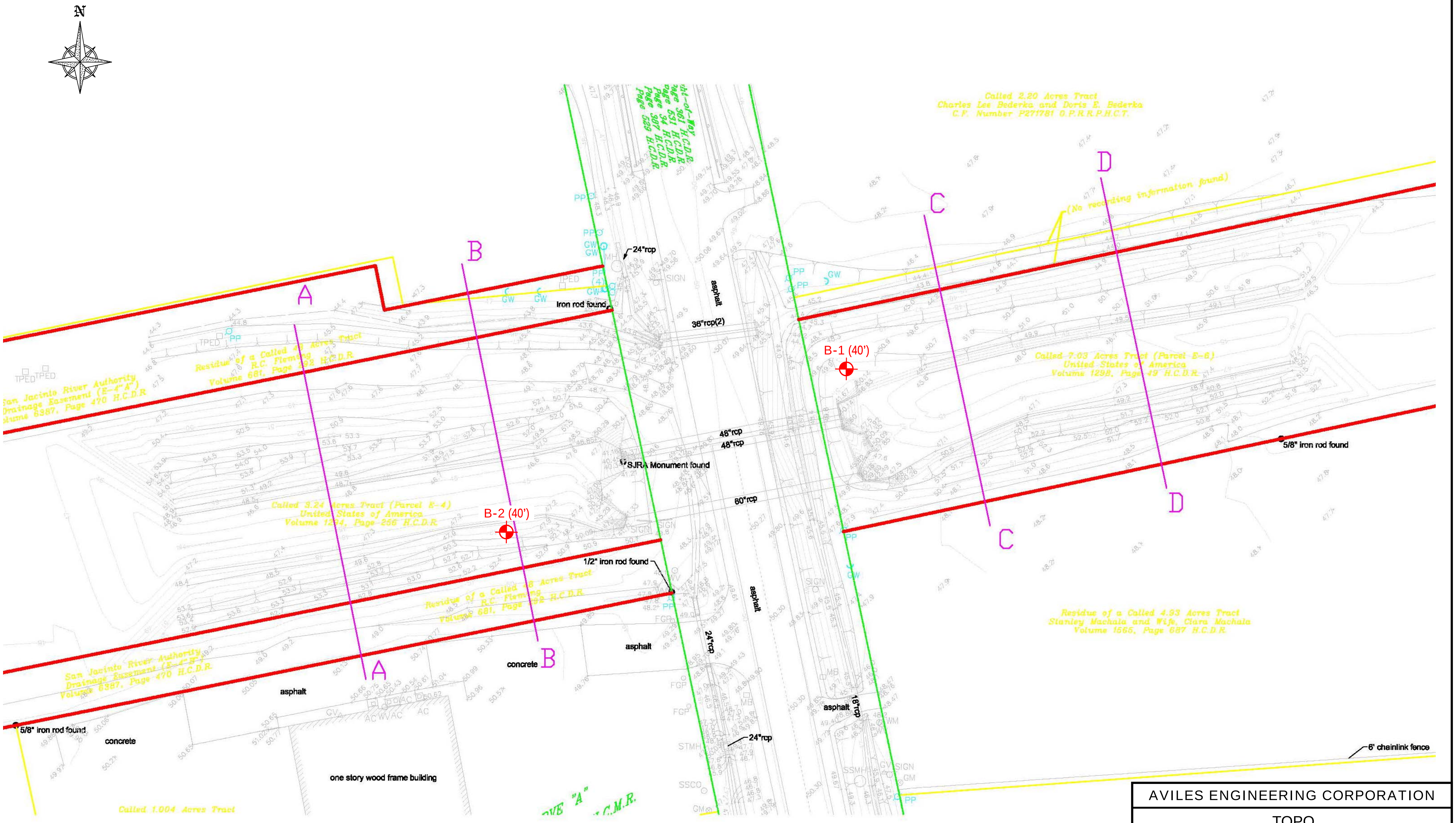


TRIAXIAL CU TESTS
MOHR'S CIRCLES
Total Stress
G134-15 Boring B-1, 12'-14'

PLATE A-14

APPENDIX B

Plate B-1	Canal Levee Topo Provided by SJRA
Plate B-2	Canal Levee Cross Sections (A-A thru D-D) Provided by SJRA



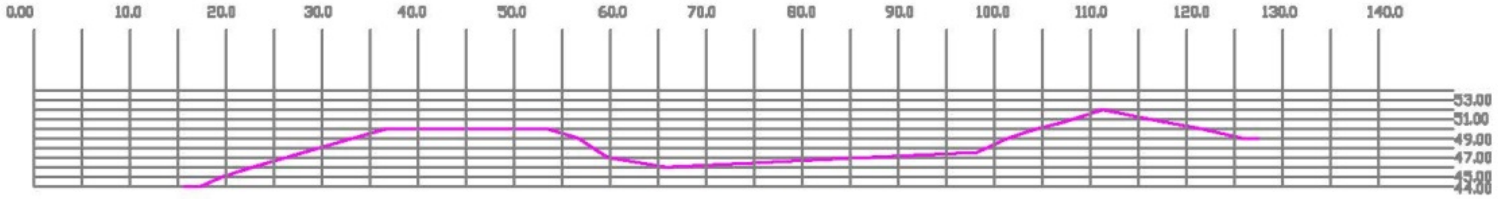
NOTE: BORING LOCATIONS ARE APPROXIMATE

AVILES ENGINEERING CORPORATION

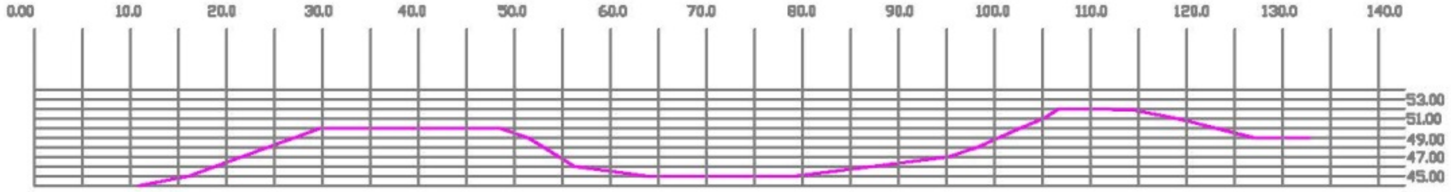
TOPO

SJRA MAIN CANAL SIPHON NO. 7
HARRIS COUNTY, TEXAS

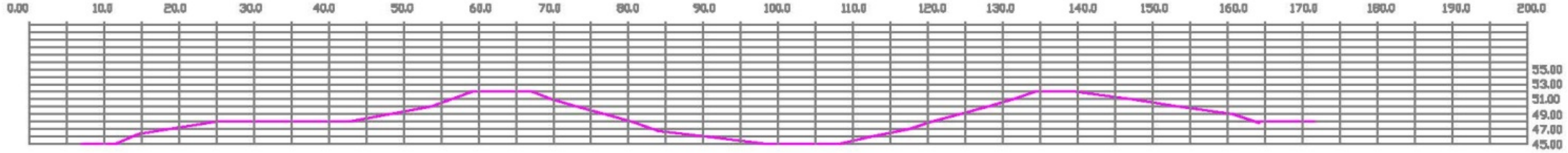
AEC REPORT NO.: G134-15	DATE: 07-28-15	SOURCE DRAWING PROVIDED BY: SJRA
APPROX. SCALE: 1" = 50'	DRAFTED BY: BpJ	PLATE NO.: PLATE B-1



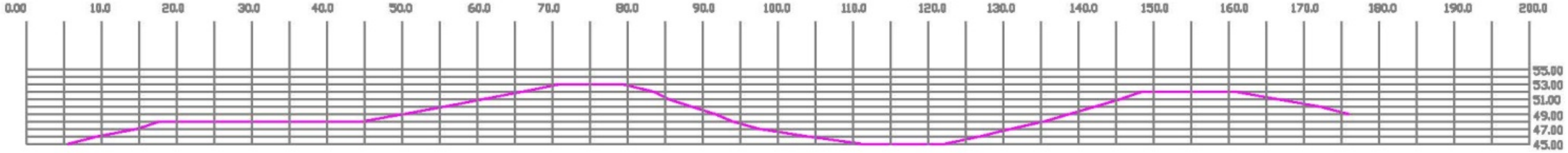
SECTION DD



SECTION CC



SECTION BB



SECTION AA

AVILES ENGINEERING CORPORATION

CROSSING SECTION PROFILES

SJRA MAIN CANAL SIPHON NO. 7
HARRIS COUNTY, TEXAS

AEC REPORT NO.: G134-15	DATE: 07-28-15	SOURCE DRAWING PROVIDED BY: SJRA
APPROX. SCALE: 1" = 20'	DRAFTED BY: BpJ	PLATE NO.: PLATE B-2

APPENDIX C

Plate C-1	Recommended Geotechnical Design Parameters
Plate C-2	Load Coefficients for Pipe Loading
Plate C-3	Live Loads on Pipe Crossing Under Roadway

G134-15 SJRA SIPHON NO. 7 IMPROVEMENTS
DESIGN SOIL PARAMETERS FOR UNDERGROUND UTILITIES AND HEADWALLS

Location	Depth (ft)	Soil Type	γ (pcf)	γ' (pcf)	OSHA Type	Short-Term						Long-Term					
						C (psf)	C_α (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	C_α' (psf)	φ' (deg)	K_a	K_0	K_p
	N/A	Select Fill	123	61	C	1600	1100	0	1.00	1.00	1.00	150	100	22	0.45	0.63	2.20
B-1	0-10	Fill: soft to stiff CH	115	53	C	600	400	0	1.00	1.00	1.00	100	50	20	0.49	0.66	2.04
	10-20	Stiff to very stiff CH	125	63	C*	1400	1000	0	1.00	1.00	1.00	270	100	18	0.53	0.69	1.89
	20-30	Stiff to very stiff CL/CH	135	73	N/A	1600	1100	0	1.00	1.00	1.00	150	100	18	0.53	0.69	1.89
B-2	0-8	Fill: firm to very stiff CH	115	53	C	1000	700	0	1.00	1.00	1.00	100	50	20	0.49	0.66	2.04
	8-12	Firm to stiff CH	114	52	B	600	400	0	1.00	1.00	1.00	100	50	20	0.49	0.66	2.04
	12-18	Firm to stiff CH	121	59	B	900	630	0	1.00	1.00	1.00	100	50	18	0.53	0.69	1.89
	18-30	Stiff to very stiff CH	121	59	C* (18-20)	1200	840	0	1.00	1.00	1.00	250	100	18	0.53	0.69	1.89

Notes: (1) γ = Unit weight for soil above water level, γ' = Buoyant unit weight for soil below water level.

(2) C = Soil ultimate cohesion, C_α = Soil ahesion between soil and wall base, and φ = Soil friction angle for short term.

(3) C' = Soil ultimate cohesion, C_α' = Soil ahesion between soil and wall base, and φ' = Soil friction angle for long term.

(4) Friction angle between soil and retaining wall for short term and long term, δ or δ' = 2/3 φ or φ'.

(5) K_a = Coefficient of active earth pressure, K_0 = Coefficient of at-rest earth pressure, K_p = Coefficient of passive earth pressure, for level backfill.

(6) CL = Lean Clay, CH = Fat Clay

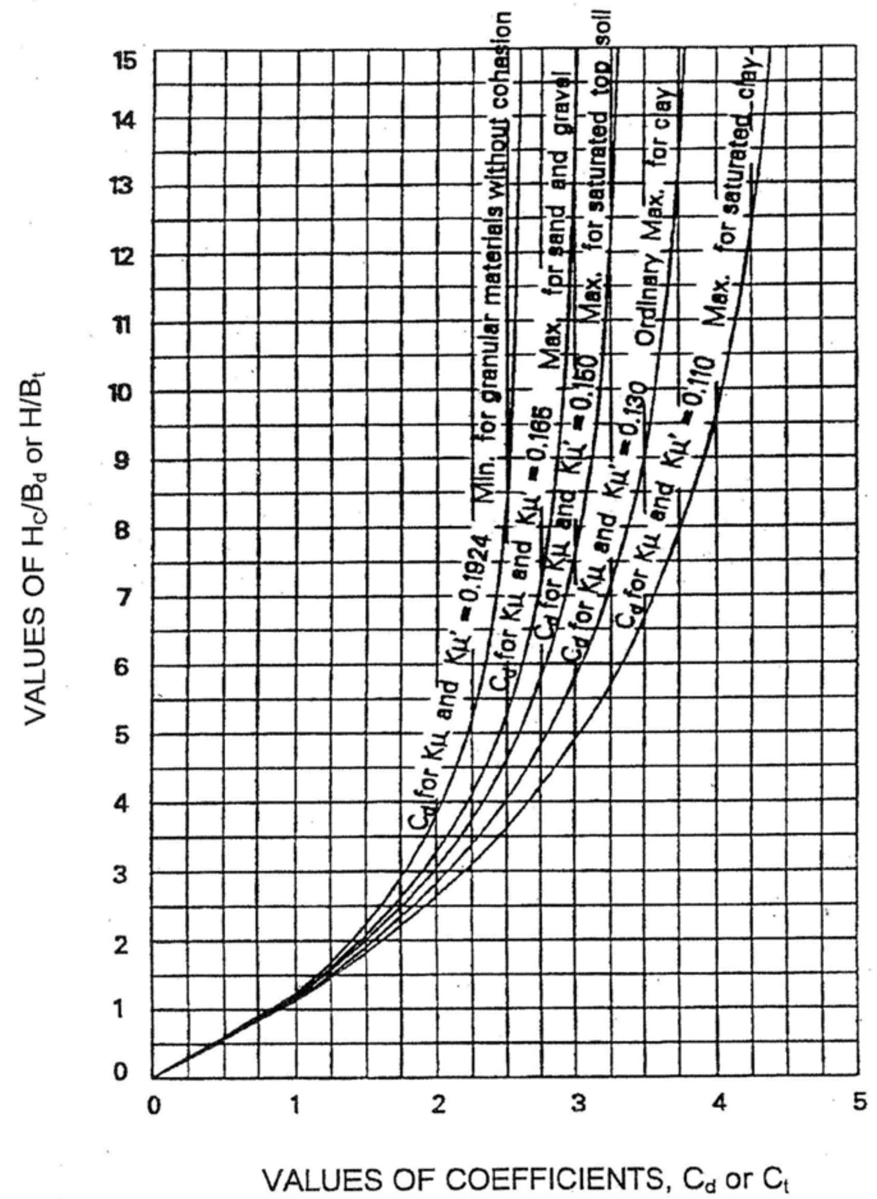
(7) OSHA Soil Types for soils in the top 20 feet below grade:

A: cohesive soils with qu = 1.5 tsf or greater (qu = Unconfined Compressive Strength of the Soil)

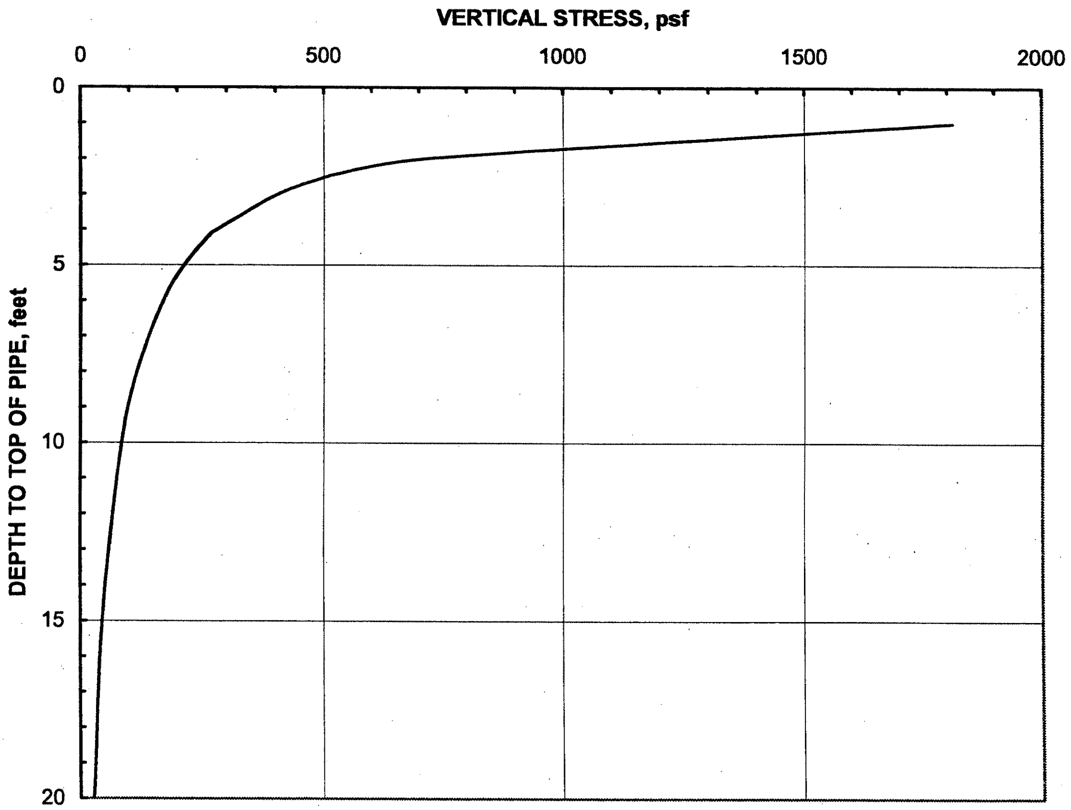
B: cohesive soils with qu = 0.5 tsf or greater

C: cohesive soils with qu = less than 0.5 tsf, fill materials, or granular soil

(8) The above OSHA Soil Types were recommended on assumption that the excavations are dewatered; if the site is not dewatered, all submerged soils should be classified as OSHA Type C.



Reference: US Army Corps of Engineers Engineering Manual, EM 1110-2-2909, Oct. 31, 1997, Figure 2-5.



LIVE LOADS ON PIPE CROSSING UNDER ROADWAY

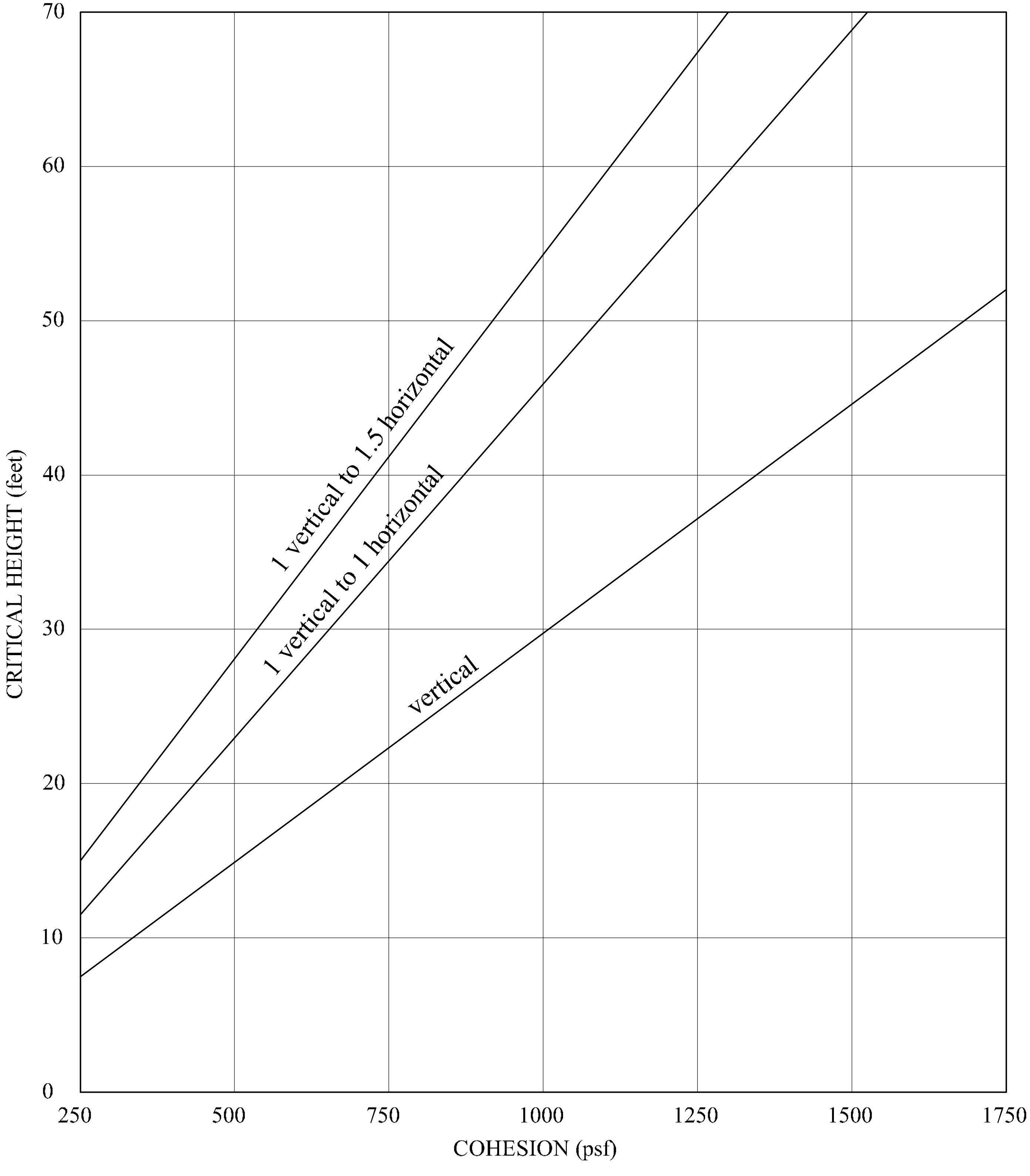
Note: 1. The vertical stress was estimated using AASHTO HS20 truck axle loadings on paved surfaces (Reference: ASCE 15-98, "Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standandard Installations").

2. Single truck passing.

APPENDIX D

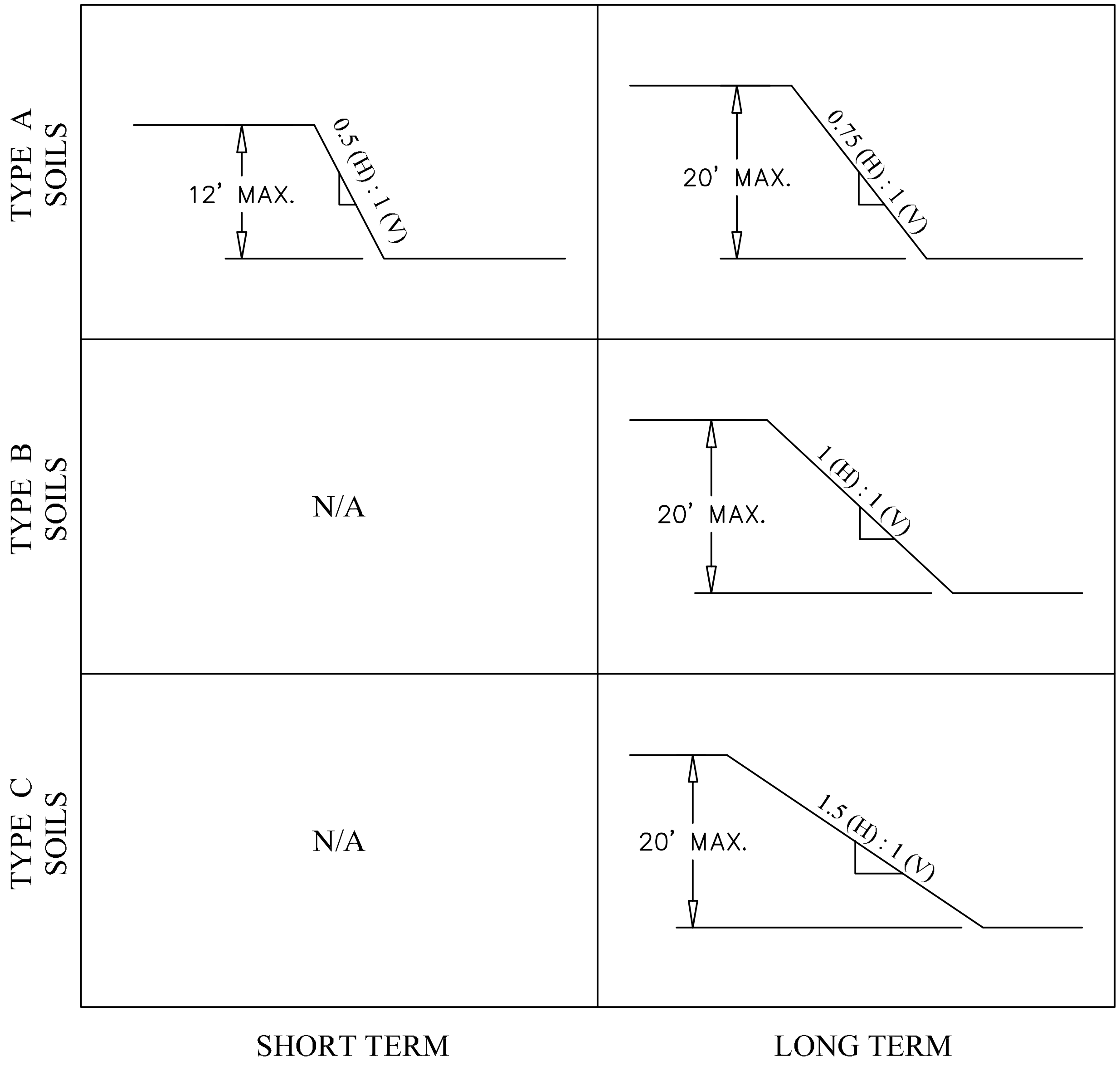
Plate D-1	Critical Heights of Cuts in Nonfissured Clays
Plate D-2	Maximum Allowable Slopes
Plate D-3	A Combination of Bracing and Open Cuts
Plate D-4	Lateral Pressure Diagrams for Open Cuts in Cohesive Soil-Long Term Conditions
Plate D-5	Lateral Pressure Diagrams for Open Cuts in Cohesive Soil-Short Term Conditions
Plate D-6	Lateral Pressure Diagrams for Open Cuts in Sand
Plate D-7	Bottom Stability for Braced Excavation in Clay
Plate D-8	Relation between the Width of Surface Depression and Depth of Cavity for Tunnels
Plate D-9	Tunnel Behavior and TBM Selection
Plate D-10	Methods of Controlling Ground Water in Tunnel and Grouting Material Selection
Plate D-11	Buoyant Uplift Resistance

Critical Heights of Cut Slopes in Nonfissured Clays



Note: The charts are calculated based on NAVFAC DM7.1, Page 7.1-319, assuming the critical circles are toe circles, and wet unit weight of soils = 125pcf.

MAXIMUM ALLOWABLE SLOPES



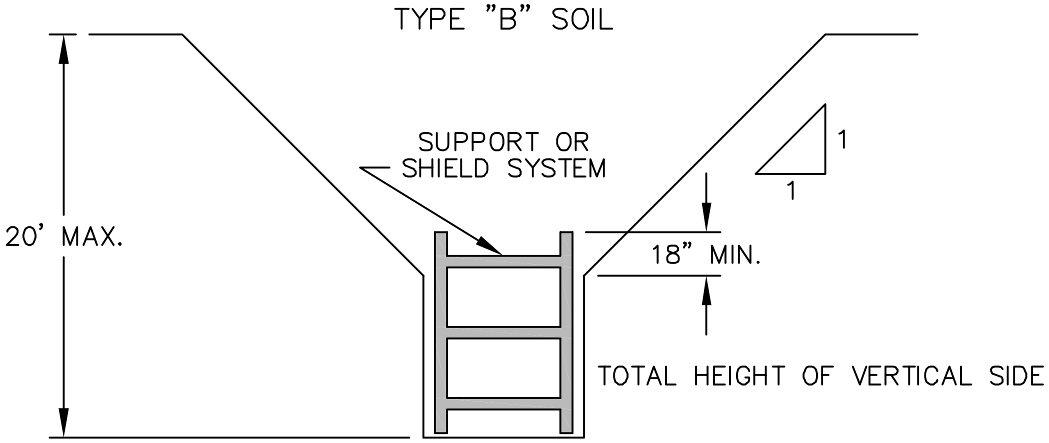
NOTES:

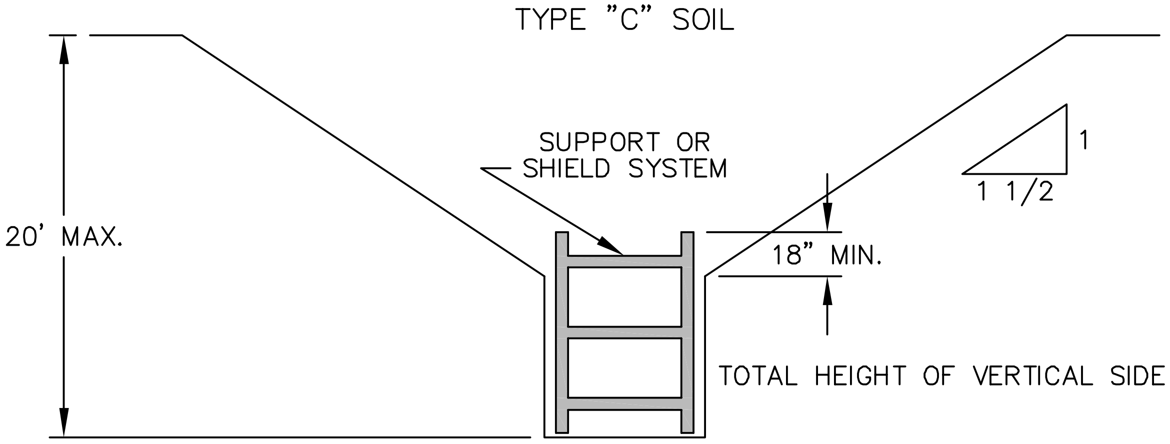
(1) For Type A soils, a short term maximum allowable slope of 0.5 (H) : 1 (V) is allowed in excavations that are 12 feet or less in depth; short term (24 hours or less) maximum allowable slopes for excavations greater than 12 feet in depth shall be 0.75 (H) : 1 (V).

(2) Maximum depth for above slopes is 20 feet. For slopes deeper than 20 feet, trench protection should be designed by the Contractor's professional engineer.

Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.

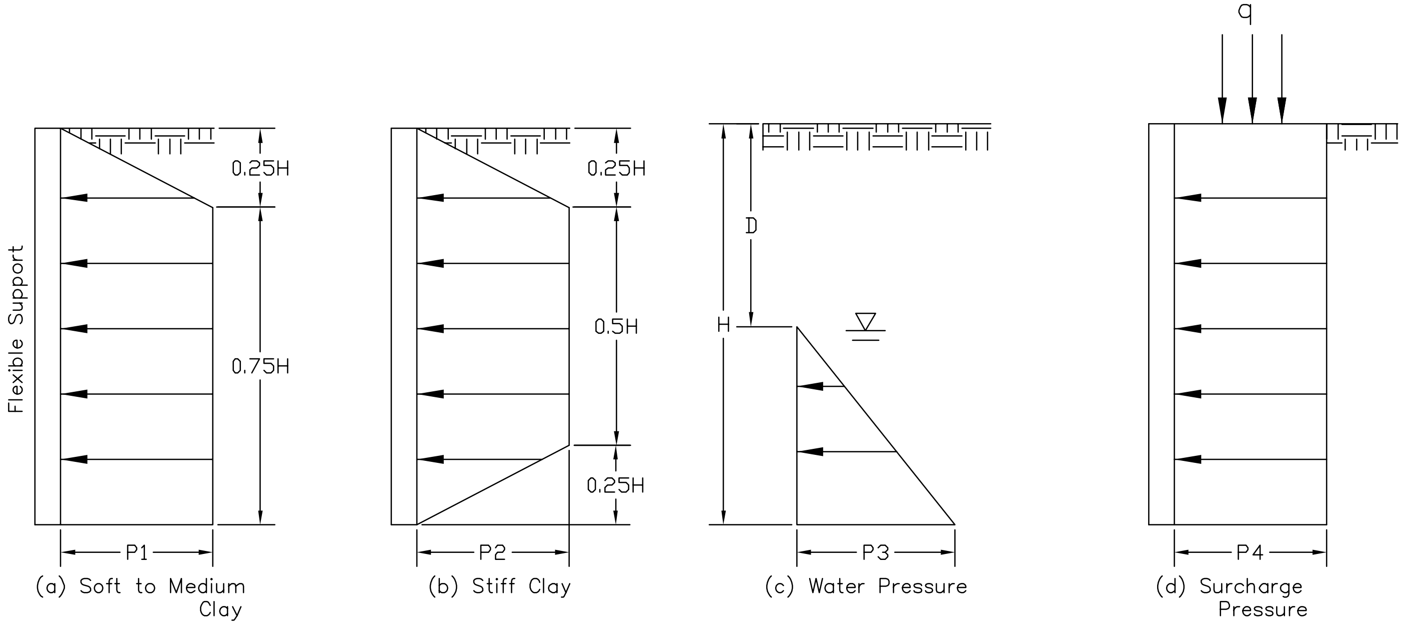
A COMBINATION OF BRACING AND OPEN CUTS





Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.

LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN COHESIVE SOIL - LONG TERM CONDITIONS



Empirical Pressure Distributions

Where:

H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = γH-4C, psf
P2 = Lateral earth pressure = 0.4γH, psf
P3 = Water pressure = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
C = Drained shear strength or cohesion, psf
K_a = Coefficient of active earth pressure

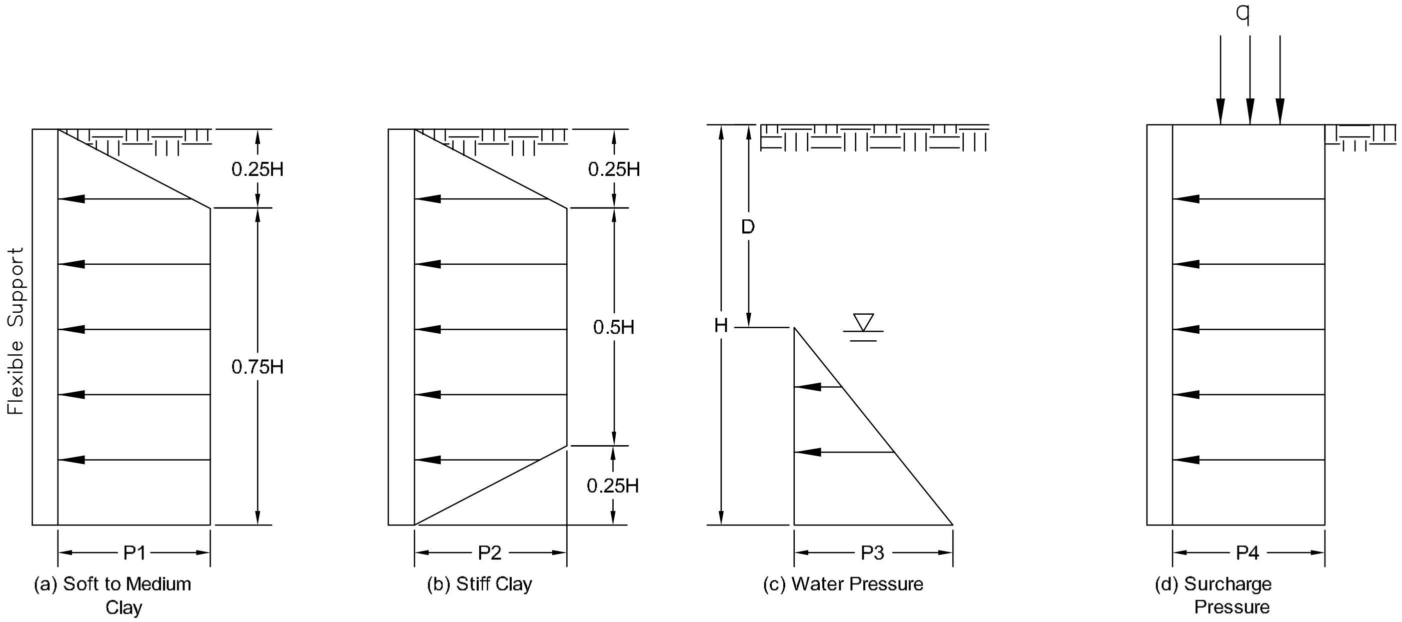
Notes:

1. All pressures are additive.
2. No safety factors are included.
3. For use only during long term construction.
4. If γH/C < 4, use section (b),
   If 4 < γH/C < 6, use larger of section (a) or (b),
   If γH/C > 6, use section (a).

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

LATERAL PRESSURE DIAGRAMS

FOR OPEN CUTS IN COHESIVE SOIL - SHORT TERM CONDITIONS



(a) Soft to Medium Clay

(b) Stiff Clay

(c) Water Pressure

(d) Surcharge Pressure

Empirical Pressure Distributions

Where:

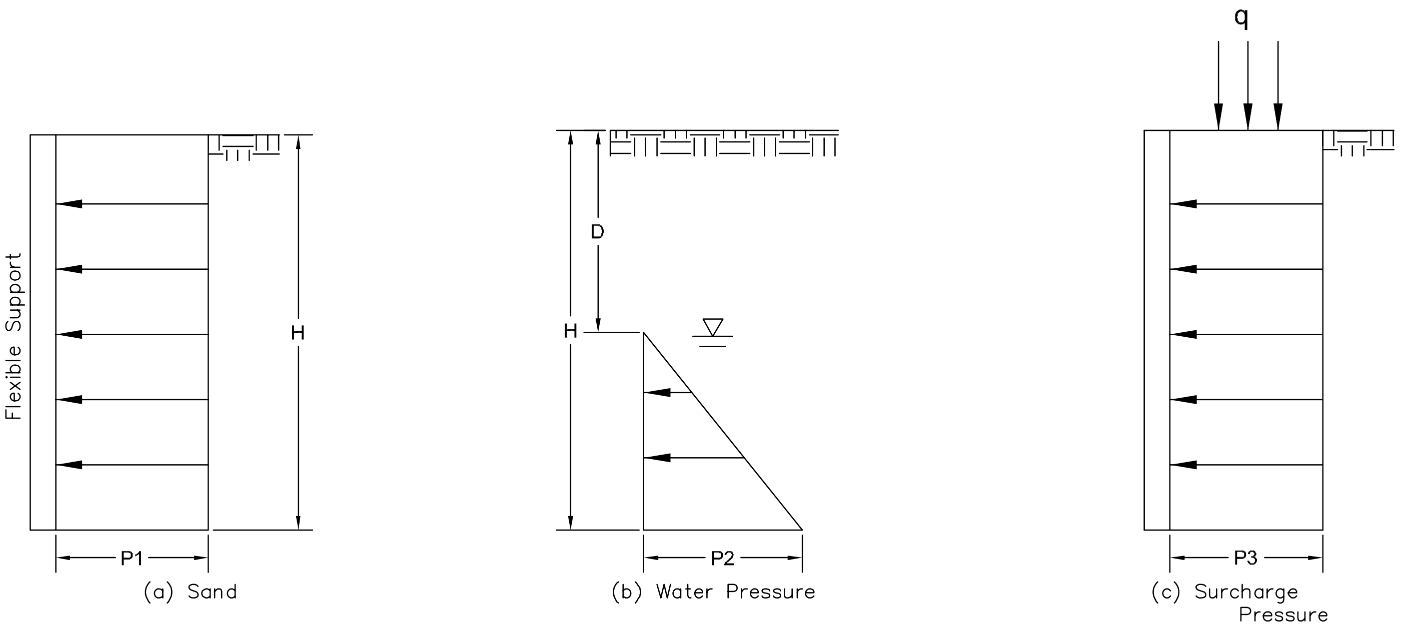
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $\gamma H\text{-}4S_u$, psf
P2 = Lateral earth pressure = $0.2\gamma H$, psf
P3 = Water pressure = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
S_u = Undrained shear strength = $q_u/2$, psf
q_u = Unconfined compressive strength, psf
K_a = Coefficient of active earth pressure

Notes:

1. All pressures are additive.
2. No safety factors are included.
3. For use only during short term construction.
4. If $\gamma H/S_u < 4$, use section (b),
   If $4 < \gamma H/S_u < 6$, use larger of section (a) or (b),
   If $\gamma H/S_u > 6$, use section (a).

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND



(a) Sand

(b) Water Pressure

(c) Surcharge Pressure

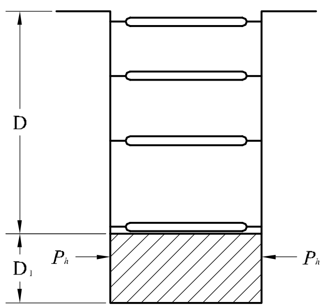
Empirical Pressure Distributions

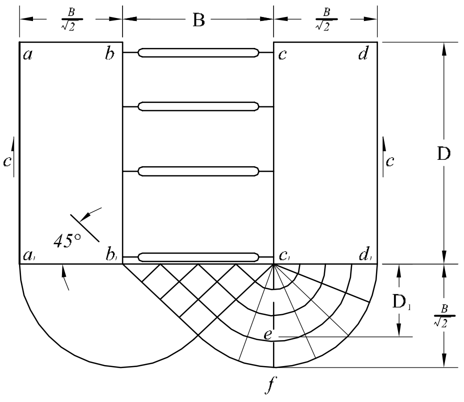
Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = 0.65*$\gamma H K_a$, psf
P2 = Water pressure = γ_w (H-D), psf
P3 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
K_a = Coefficient of active earth pressure = (1-sinφ)/(1+sinφ)
φ = Drained friction angle

Notes:
1. All pressures are additive.
2. No safety factors are included.

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

BOTTOM STABILITY FOR BRACED EXCAVATION IN CLAY





Factor of Safety against bottom of heave,

$$F.S = \frac{NcC}{(\gamma D+q)}$$

where, Nc = Coefficient depending on the dimension of the excavation (see Figure at the bottom)
C = Undrained shear strength of soil in zone immediately around the bottom of the excavation,
γ = Unit weight of soil,
D = Depth of excavation,
q = Surface surcharge.

If F.S < 1.5, sheeting should be extended further down to achieve stability

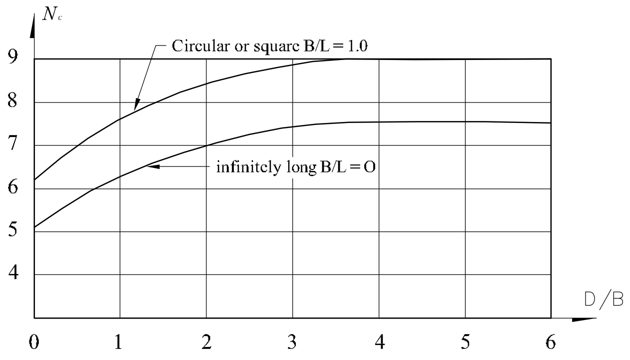
$$\text{Depth of Buried Length, } (D_1) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma}\ ;\ D_1 \geq 5\ ft.$$

Pressure on buried length, P_h:

For $D_1 < 0.47B$; $P_h = 1.5\ D_1(\gamma D - 1.4\ CD/B - 3.14C)$

For $D_1 > 0.47B$; $P_h = 0.7\ (\gamma DB - 1.4\ CD - 3.14CB)$

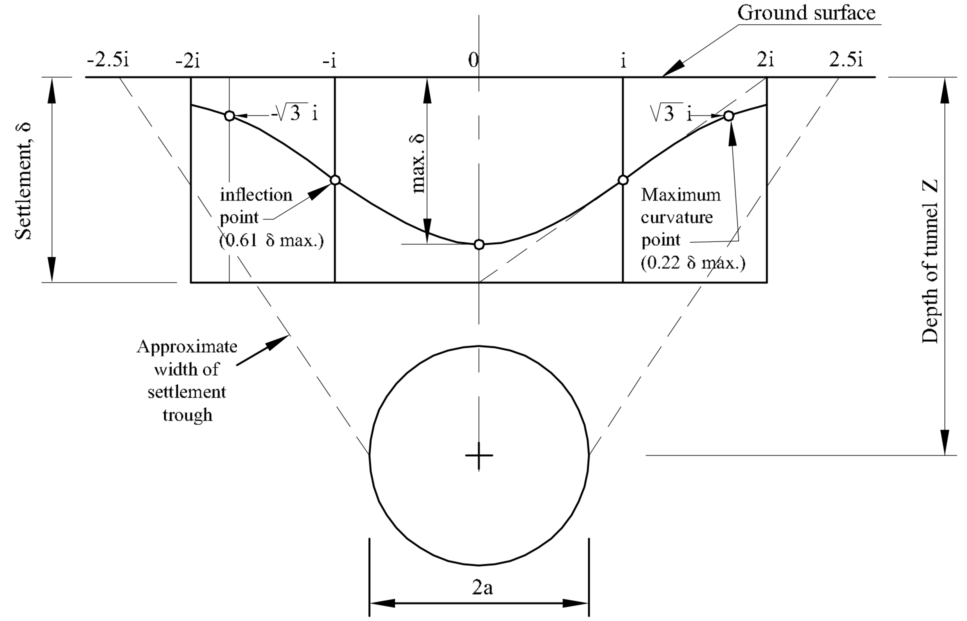
where; B = width of excavation



N_c rectangular = $(0.84 + 0.16B/L)N_c$ square

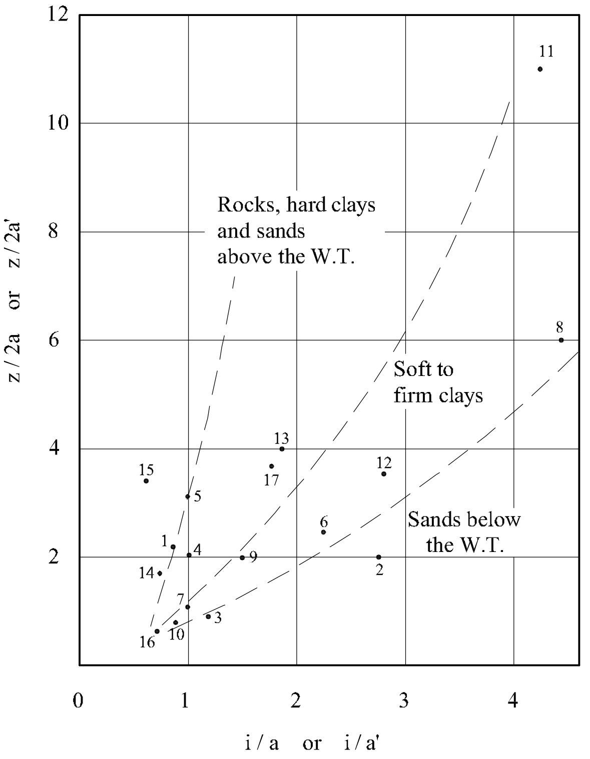
Reference: *Bjerrum, L. and Eide, O., Stability of Strutted Excavations in Clay, Geotechnique, 6, 32-47 (1956).*

Relation between the Width of the Surface Depression (i/a) and the Depth of the Cavity (z/a) for Tunnels



Volume of depression = 2.5i δ max.

(a)



(b)

Reference: Peck, R. B. (1969) "Deep Excavations and Tunneling in Soft Ground," Proceedings, Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico City, State of the Art Volume, pp. 225-290.

Tunnel Behavior: Sands and Gravels

(Terzaghi, 1977)

Designation	Degree of Compactness	Tunnel Behavior	
		Above Water Table	Below Water Table
Very Fine Clean Sand	Loose, $N \leq 10$	Cohesive Running	Flowing
	Dense, $N > 30$	Fast Raveling	Flowing
Fine Sand with Clay Binder	Loose, $N \leq 10$	Rapid Raveling	Flowing
	Dense, $N > 30$	Firm or Slowly Raveling	Slowly Raveling
Sand or Sandy Gravel with Clay Binder	Loose, $N < 10$	Rapid Raveling	Rapidly Raveling or Flowing
	Dense, $N > 30$	Firm	Firm/slow Raveling
Sandy Gravel and Medium to Coarse Sand		Running Ground. Uniform ($C_u < 3$) and loose ($N < 10$) materials with round grains run much more freely than well graded ($C_u > 6$) and dense ($N > 30$) ones with angular grains.	Flowing Conditions combined with extremely heavy discharge of water.

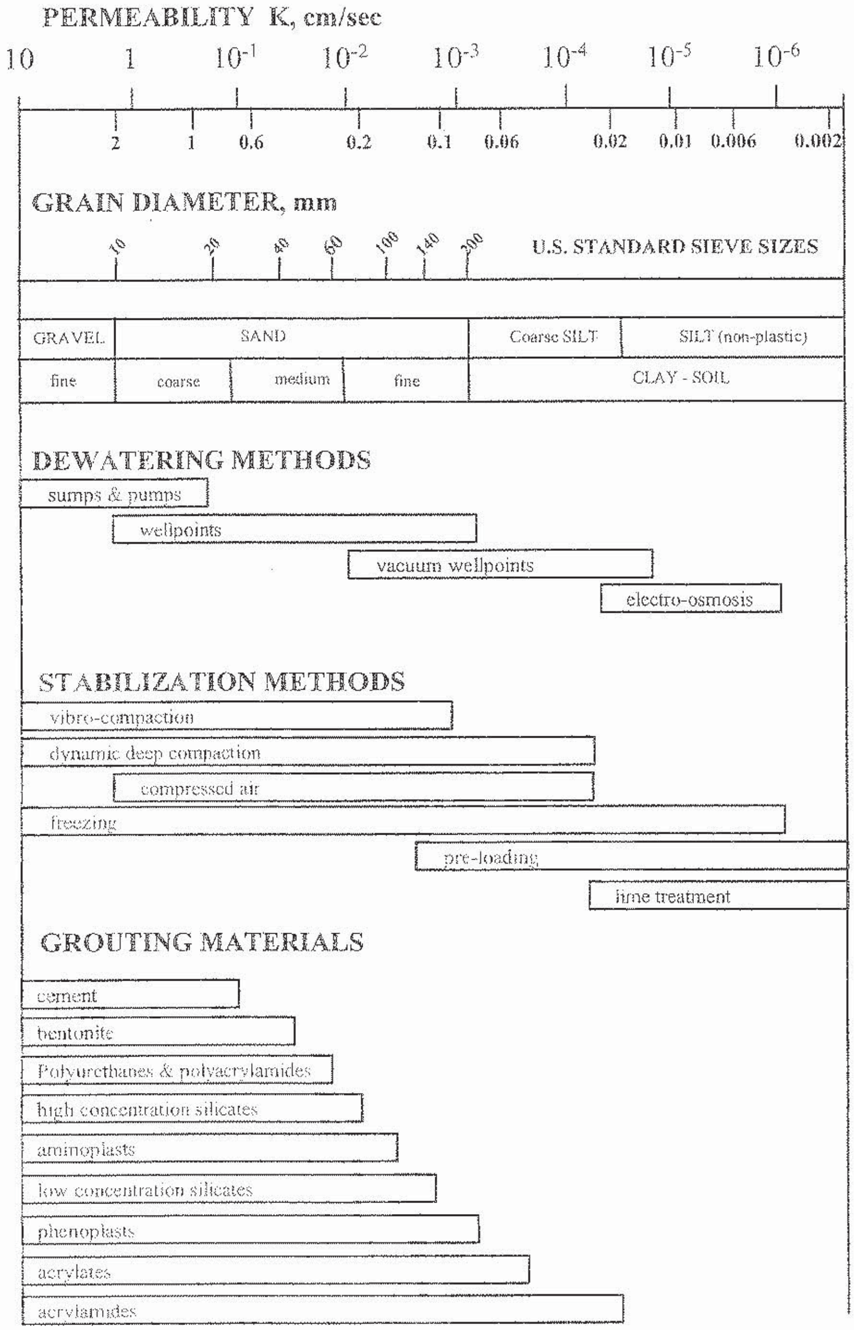
TBM FAMILY OF MACHINES

(From Kessler & Moore,)

Machine Type	Typical Machine Diameters	Ground Condition TBM is Best Suited For
Pipe Jacking Machines	Up to approx. 10 – 13 ft (3 - 4m)	Any ground
Small Bore Unit (SBU)	Up to 6.6 ft (2m)	Any ground
Shielded TBMs	6.6 – 46 ft (2 to 14m) plus	Soft ground above the water table
Mix Face TBMs	6.6 – 46 ft (2 to 14m) plus	Mixed ground above the water table
Slurry TBMs	6.6 – 46 ft (2 to 14m) plus	Coarse-grained soft ground below the water table
EPB TBMs	6.6 – 46 ft (2 to 14m) plus	Fine-grained soft ground below the water table
Hard Rock TBMs	6.6 – 46 ft (2 to 14m) plus	Hard rock
Reamer TBMs	Various	Hard rock
Multi-head TBMs	Various	Various

Reference: Dots Oyenuga (2004), "FHWA Road Tunnel Design Guidelines", Pages 8 and 10, published by U.S. Department of Transportation Federal Highway Administration, Report No. FHWA-IF-05-023, Washington DC.

Methods of Controlling Groundwater
(after Karol, 1990)



Note: 1 cm/sec = 0.4 in/sec; 1 mm = 0.04 in.

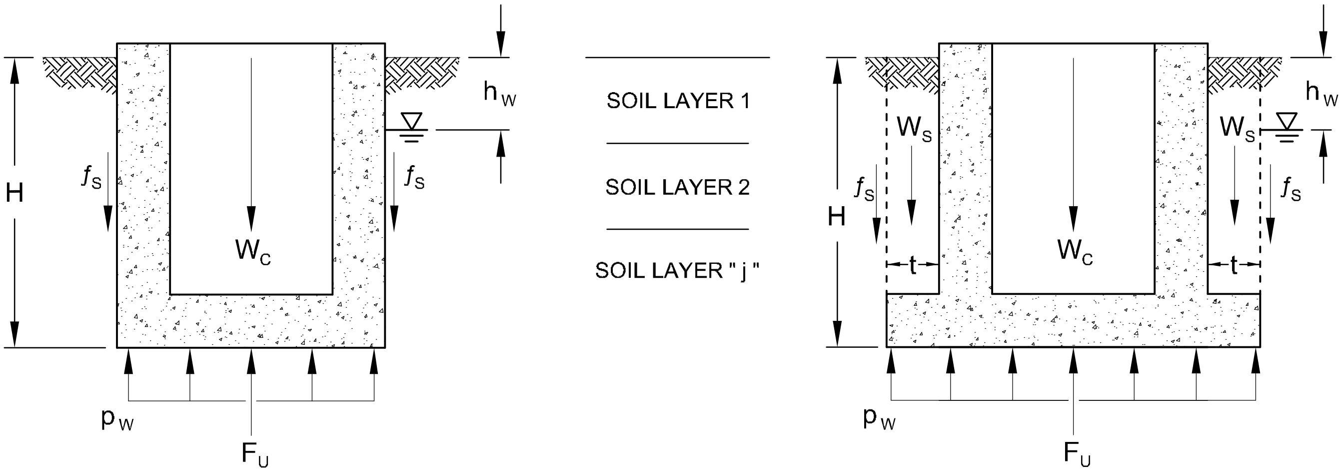
Reference: Dots Oyenuga (2004), "FHWA Road Tunnel Design Guidelines", Page 9, published by U.S. Department of Transportation Federal Highway Administration, Report No. FHWA-IF-05-023, Washington DC.

PLATE D-10

BUOYANT UPLIFT RESISTANCE FOR BURIED STRUCTURES

(a) WALL / SOIL FRICTION PLUS STRUCTURAL WEIGHT

(b) SOIL WEIGHT ABOVE BASE EXTENSION



(a)

cohesive soils: $f_{S_j} = \alpha \; c_j \leq$ 3,000 psf

cohesionless soils: $f_{S_j} = 0.75 \; K_S \; \sigma_{V_j} \; \tan\delta_j$

$$Q_S = P_S \sum f_{S_j} h_j$$

$$\frac{W_C}{S_{f_a}} + \frac{Q_S}{S_{f_b}} \geqq F_U$$

(b)

cohesive soils: $f_{S_j} = c_j \leq$ 3,000 psf

cohesionless soils: $f_{S_j} = 0.75 \; K_S \; \sigma_{V_j} \; \tan\Phi_j$

$$Q_S = P_S \sum f_{S_j} h_j$$

$$\frac{W_C}{S_{f_a}} + \frac{Q_S}{S_{f_b}} + \frac{W_S}{S_{f_c}} \geqq F_U$$

Where:

A_B = area of base, sq. ft.
H = buried height of structure, ft.
h_W = depth to water table, ft.
$p_W = \gamma_W (H - h_W)$, unit hydrostatic uplift, psf.
γ_W = 62.4 pcf, unit weight of water
$F_U = p_W A_B$, hydrostatic uplift force, lbs.
f_{S_j} = unit frictional resistance of soil layer " j ", psf.
c_j = undrained cohesion of soil layer " j ", psf.
α = 0.55, cohesion factor between soil and structure wall
σ_{V_j} = effective overburden pressure at midpoint of soil layer " j ", psf.
$\delta_j = 0.75 \; \Phi_j$, friction angle between soil layer " j " and concrete wall, degrees

Φ_j = internal angle of friction of soil layer " j ", degrees
K_S = 0.4, coefficient of lateral pressure
h_j = thickness of soil layer " j ", ft.
j = 1, 2, ...
P_S = perimeter of structure base, ft.
Q_S = ultimate skin friction, lbs.
W_C = weight of structure, lbs.
W_S = weight of backfill above base extension, lbs.
S_{f_a} = 1.1, factor of safety for dead weight of structure
S_{f_b} = 3.0, factor of safety for soil / structure friction
S_{f_c} = 1.5, factor of safety for soil weight above base extension
t = width of base extension, ft.

NOTE: neglect f_S in upper 5 feet for expansive clay with a plasticity index > 20.

Reference:

1) American Concrete Pipe Association, (1996), Manhole Floatation

2) O'Neill, M.W., and Reese, L.C., (1999), "Drilled Shafts: Construction Procedures and Design Methods", FHWA-IF-99-025

APPENDIX E

Plate E-1 Design Soil Parameters for Slope Stability Analysis
Plate E-2a Slope Stability Analysis on CS-AA Existing Waterside Slope (Global Slide for North Embankment), H:V=3.9:1, Short Term Condition, Based on Boring B-2
Plate E-2b Slope Stability Analysis on CS-AA Existing Waterside Slope (Local Slide for North Embankment), H:V=3.9:1, Short Term Condition, Based on Boring B-2
Plate E-3a Slope Stability Analysis on CS-AA Existing Waterside Slope (Global Slide for North Embankment), H:V=3.9:1, Long Term Condition, Based on Boring B-2
Plate E-3b Slope Stability Analysis on CS-AA Existing Waterside Slope (Local Slide for North Embankment), H:V=3.9:1, Long Term Condition, Based on Boring B-2
Plate E-4a Slope Stability Analysis on CS-AA Existing Waterside Slope (Global Slide for North Embankment), H:V=3.9:1, Rapid Drawdown Condition, Based on Boring B-2
Plate E-4b Slope Stability Analysis on CS-AA Existing Waterside Slope (Local Slide for North Embankment), H:V=3.9:1, Rapid Drawdown Condition, Based on Boring B-2
Plate E-5a Slope Stability Analysis on CS-BB Existing Waterside Slope (Global Slide for North Embankment), H:V=3:1, Short Term Condition, Based on Boring B-2
Plate E-5b Slope Stability Analysis on CS-BB Existing Waterside Slope (Local Slide for North Embankment), H:V=3:1, Short Term Condition, Based on Boring B-2
Plate E-6a Slope Stability Analysis on CS-BB Existing Waterside Slope (Global Slide for North Embankment), H:V=3:1, Long Term Condition, Based on Boring B-2
Plate E-6b Slope Stability Analysis on CS-BB Existing Waterside Slope (Local Slide for North Embankment), H:V=3:1, Long Term Condition, Based on Boring B-2
Plate E-7a Slope Stability Analysis on CS-BB Existing Waterside Slope (Global Slide for North Embankment), H:V=3:1, Rapid Drawdown Condition, Based on Boring B-2
Plate E-7b Slope Stability Analysis on CS-BB Existing Waterside Slope (Local Slide for North Embankment), H:V=3:1, Rapid Drawdown Condition, Based on Boring B-2
Plate E-8a Slope Stability Analysis on CS-CC Existing Waterside Slope (Global Slide for North Embankment), Upper H:V=1.9:1, Short Term Condition, Based on Boring B-1
Plate E-8b Slope Stability Analysis on CS-CC Existing Waterside Slope (Local Slide for North Embankment), Upper H:V=1.9:1, Short Term Condition, Based on Boring B-1
Plate E-9a Slope Stability Analysis on CS-CC Existing Waterside Slope (Global Slide for North Embankment), Upper H:V=1.9:1, Long Term Condition, Based on Boring B-1
Plate E-9b Slope Stability Analysis on CS-CC Existing Waterside Slope (Local Slide for North Embankment), Upper H:V=1.9:1, Long Term Condition, Based on Boring B-1
Plate E-10a Slope Stability Analysis on CS-CC Existing Waterside Slope (Global Slide for North Embankment), Upper H:V=1.9:1, Rapid Drawdown Condition, Based on Boring B-1
Plate E-10b Slope Stability Analysis on CS-CC Existing Waterside Slope (Local Slide for North Embankment), Upper H:V=1.9:1, Rapid Drawdown Condition, Based on Boring B-1
Plate E-11a Slope Stability Analysis on CS-CC Existing Landside Slope (Global Slide for North Embankment), H:V=3.1:1, Short Term Condition, Based on Boring B-1
Plate E-11b Slope Stability Analysis on CS-CC Existing Landside Slope (Local Slide for North Embankment), H:V=3.1:1, Short Term Condition, Based on Boring B-1
Plate E-12a Slope Stability Analysis on CS-CC Existing Landside Slope (Global Slide for North Embankment), H:V=3.1:1, Long Term Condition, Based on Boring B-1
Plate E-12b Slope Stability Analysis on CS-CC Existing Landside Slope (Local Slide for North Embankment), H:V=3.1:1, Long Term Condition, Based on Boring B-1
Plate E-13a Slope Stability Analysis on CS-CC Existing Landside Slope (Global Slide for North Embankment), H:V=3.1:1, Rapid Drawdown Condition, Based on Boring B-1
Plate E-13b Slope Stability Analysis on CS-CC Existing Landside Slope (Local Slide for North Embankment), H:V=3.1:1, Rapid Drawdown Condition, Based on Boring B-1

Table 1. Design Soil Parameters for Slope Stability Analysis (CS-CC and DD) Based on Boring B-1

Depth Below the Top of Levee (ft)	**Soil Type**	**γ (pcf)**	**Short-Term (UU)**		**Long-Term (CD)**		**Rapid Drawdown (CU)**	
			C_u (psf)	**ϕ_u (deg)**	**C' (psf)**	**ϕ' (deg)**	**C_{cu} (psf)**	**ϕ_{cu} (deg)**
n/a	Compacted Select Fill	123	1600	0	180	22	210	18
0 to 10	Fill: soft CH	115	600 (C_r=65)	0 (ϕ_r=21)	100 (C_r=65)	20 (ϕ_r=21)	110 (C_r=65)	15 (ϕ_r=21)
10 to 27	Stiff to very stiff CH	125	1400	0	270	18	270	13

Table 2. Design Soil Parameters for Slope Stability Analysis (CS-AA and BB) Based on Boring B-2

Depth Below the Top of Levee (ft)	**Soil Type**	**γ (pcf)**	**Short-Term (UU)**		**Long-Term (CD)**		**Rapid Drawdown (CU)**	
			C_u (psf)	**ϕ_u (deg)**	**C' (psf)**	**ϕ' (deg)**	**C_{cu} (psf)**	**ϕ_{cu} (deg)**
n/a	Compacted Select Fill	123	1600	0	180	22	210	18
0 to 8	Fill: firm to very stiff CH	115	1000 (C_r=65)	0 (ϕ_r=21)	100 (C_r=65)	20 (ϕ_r=21)	110 (C_r=65)	15 (ϕ_r=21)
8 to 18	Firm to stiff CH	114	600	0	100	20	110	15
18 to 32	Firm to very stiff CH	121	1200	0	250	18	250	13

Notes:
(1) γ = wet unit weight of soil;
(2) C_u =undrained cohesion, ϕ_u = angle of internal friction, under short term conditions. UU = strength parameters that were determined from Unconsolidated-Undrained triaxial tests;
(3) C' =effective cohesion, ϕ' =effective friction angle, under long term condition; CD = Consolidated-Drained strength parameters that were determined from CU triaxial tests with pore pressure measurements;
(4) C_r = cohesion for desiccated fat clay, ϕ_r = friction angle for desiccated fat clay;
(5) C_{cu} = total cohesion, ϕ_{cu} = friction angle, under rapid drawdown condition; CU = strength parameters developed from Consolidated-Undrained triaxial tests;
(6) CH = fat clay.

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION A-A - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, SHORT TERM CONDITION

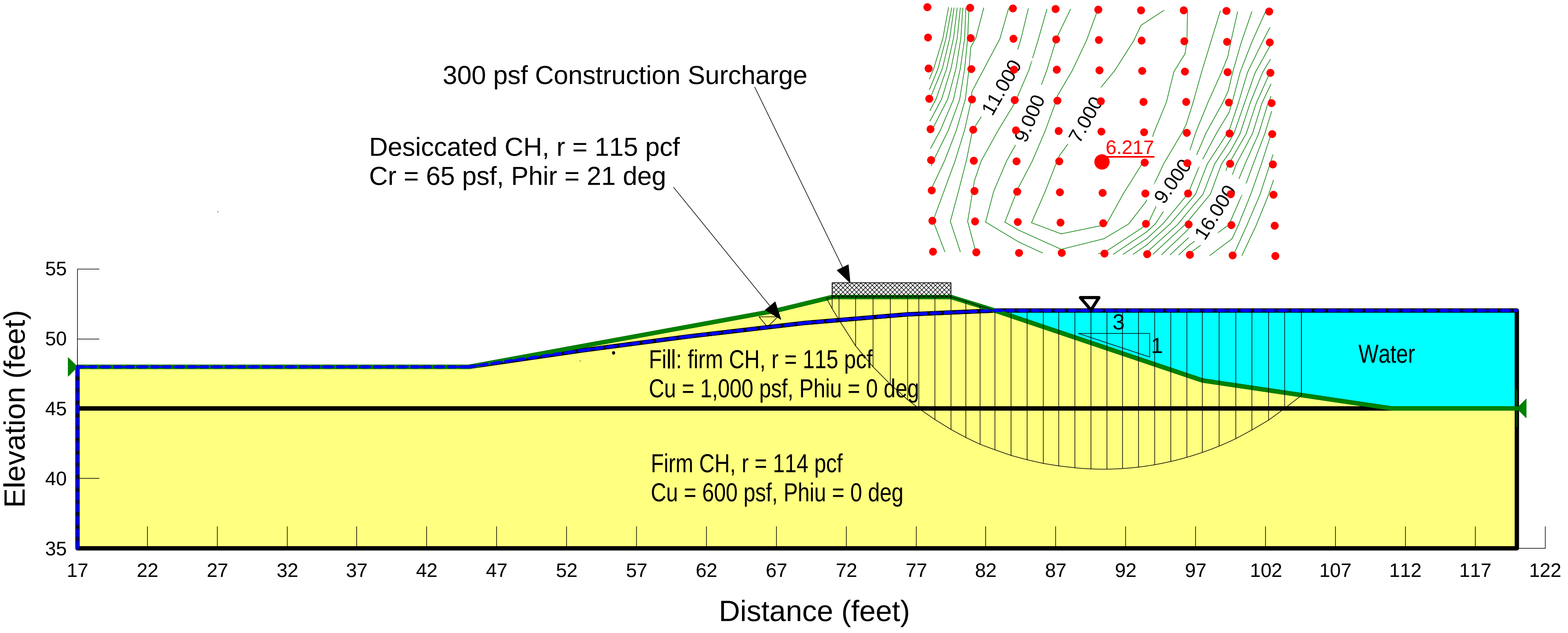


PLATE E-2a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION A-A - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, SHORT TERM CONDITION

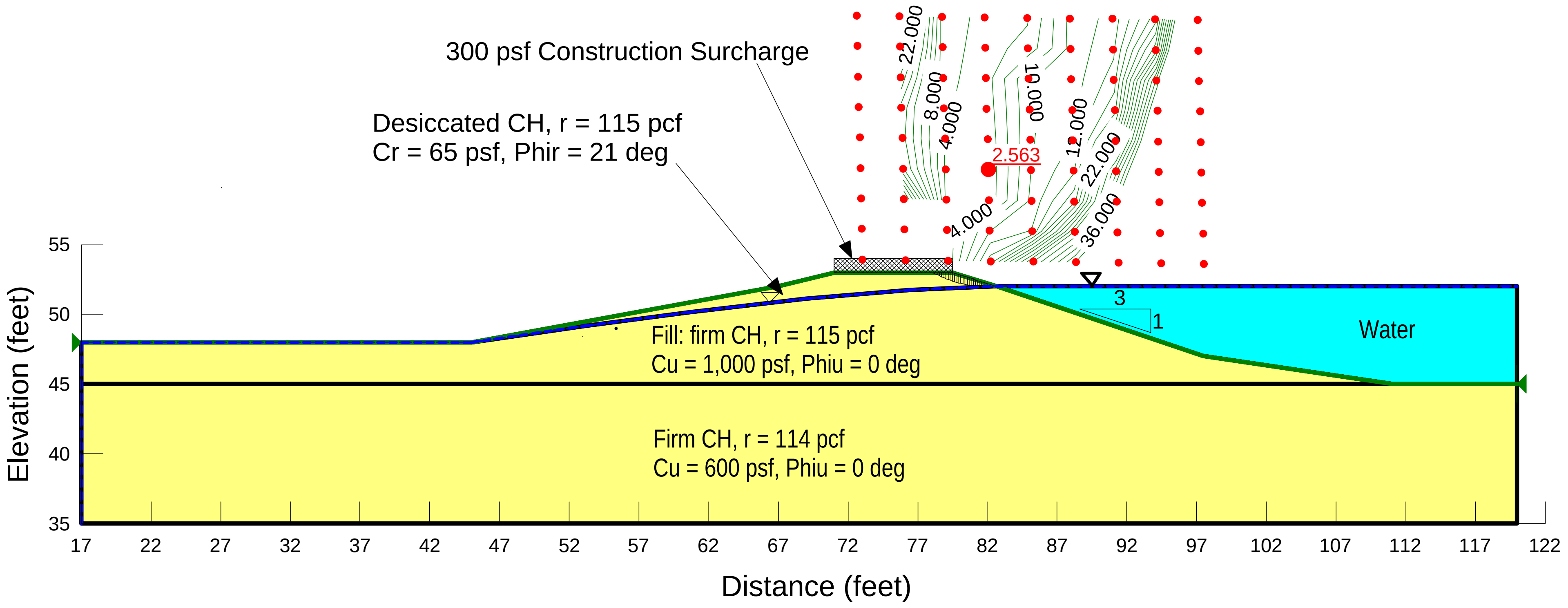


PLATE E-2b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION A-A - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, LONG TERM CONDITION

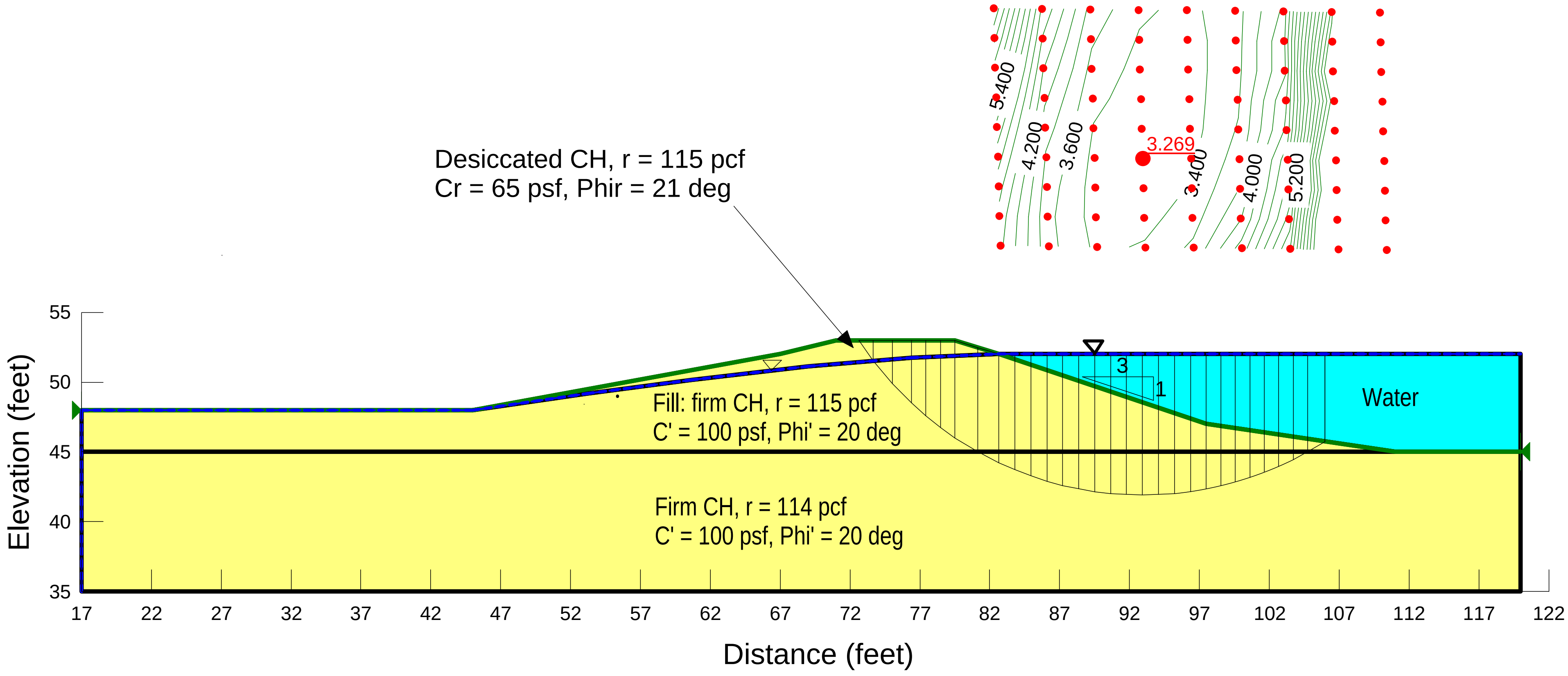


PLATE E-3a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION A-A - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, LONG TERM CONDITION

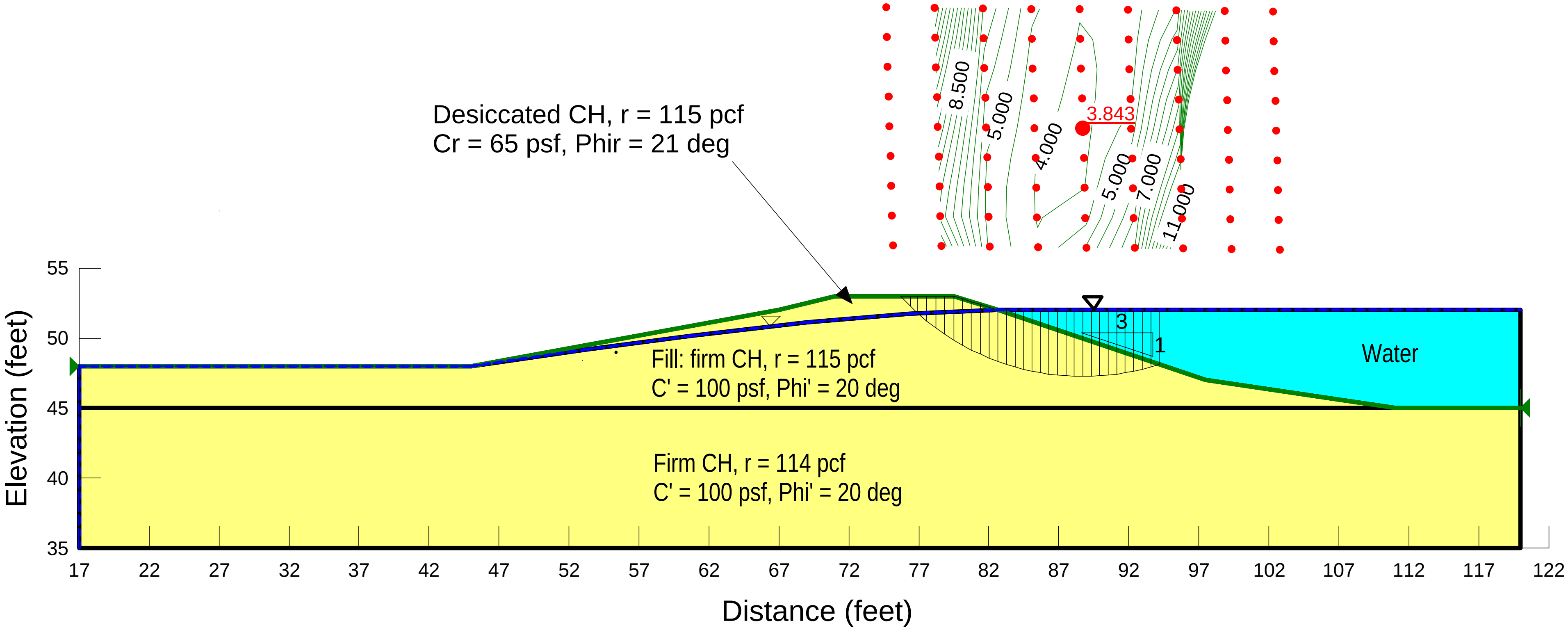


PLATE E-3b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION A-A - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, RAPID DRAWDOWN CONDITION (DURING MAINTENANCE)

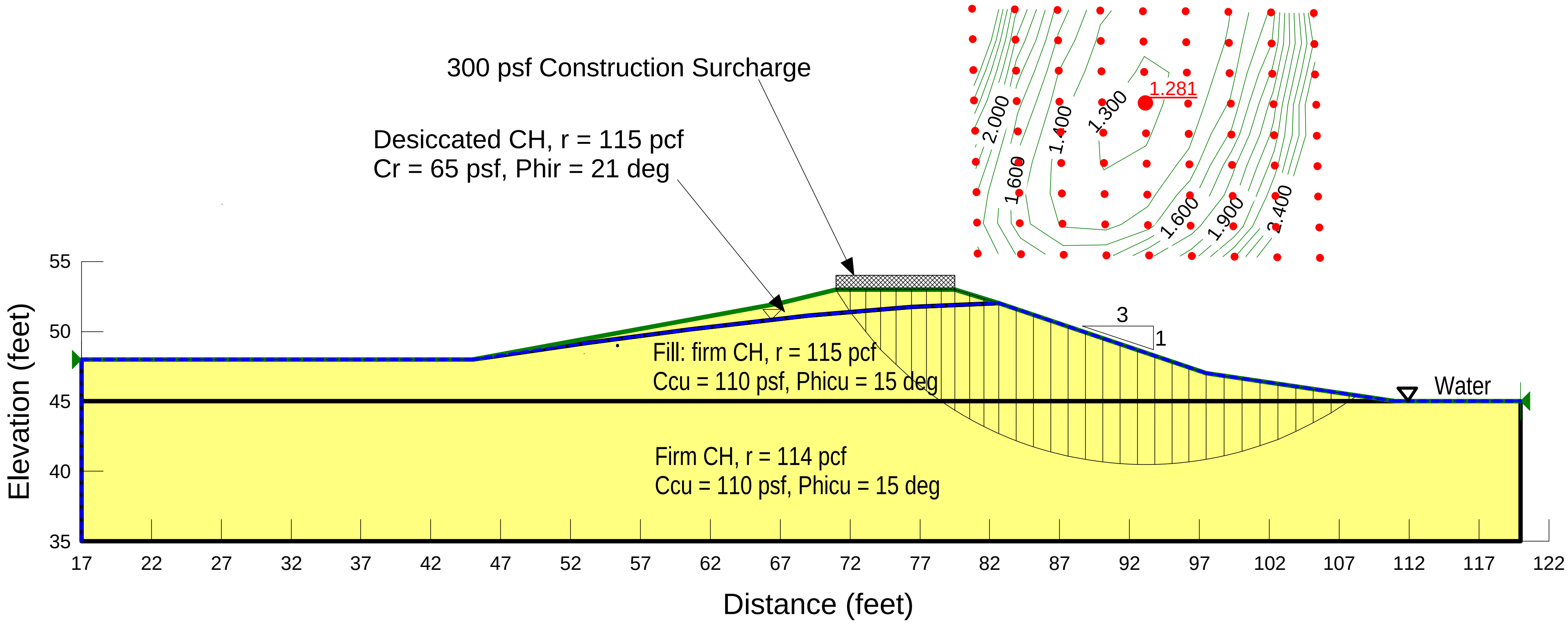


PLATE E-4a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION A-A - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, RAPID DRAWDOWN CONDITION (DURING MAINTENANCE)

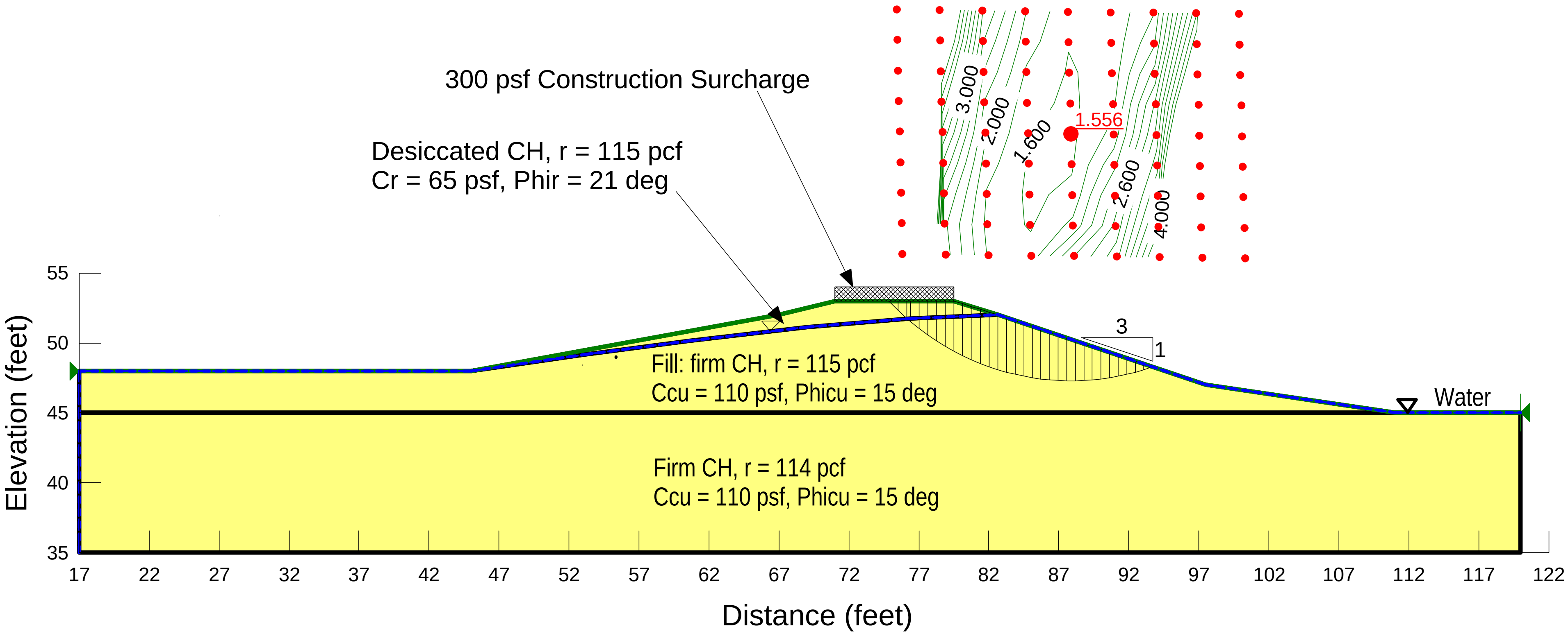


PLATE E-4b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION B-B - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, SHORT TERM CONDITION

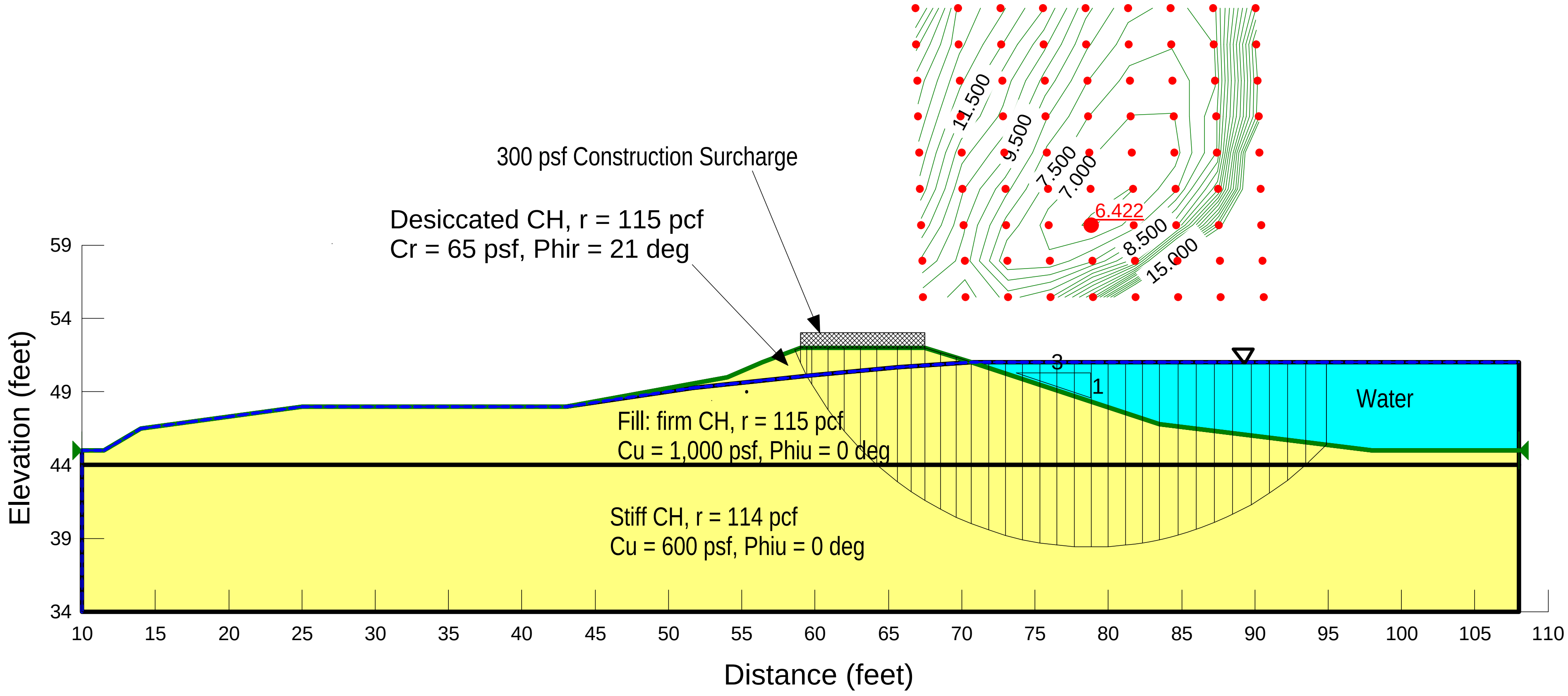


PLATE E-5a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION B-B - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, SHORT TERM CONDITION

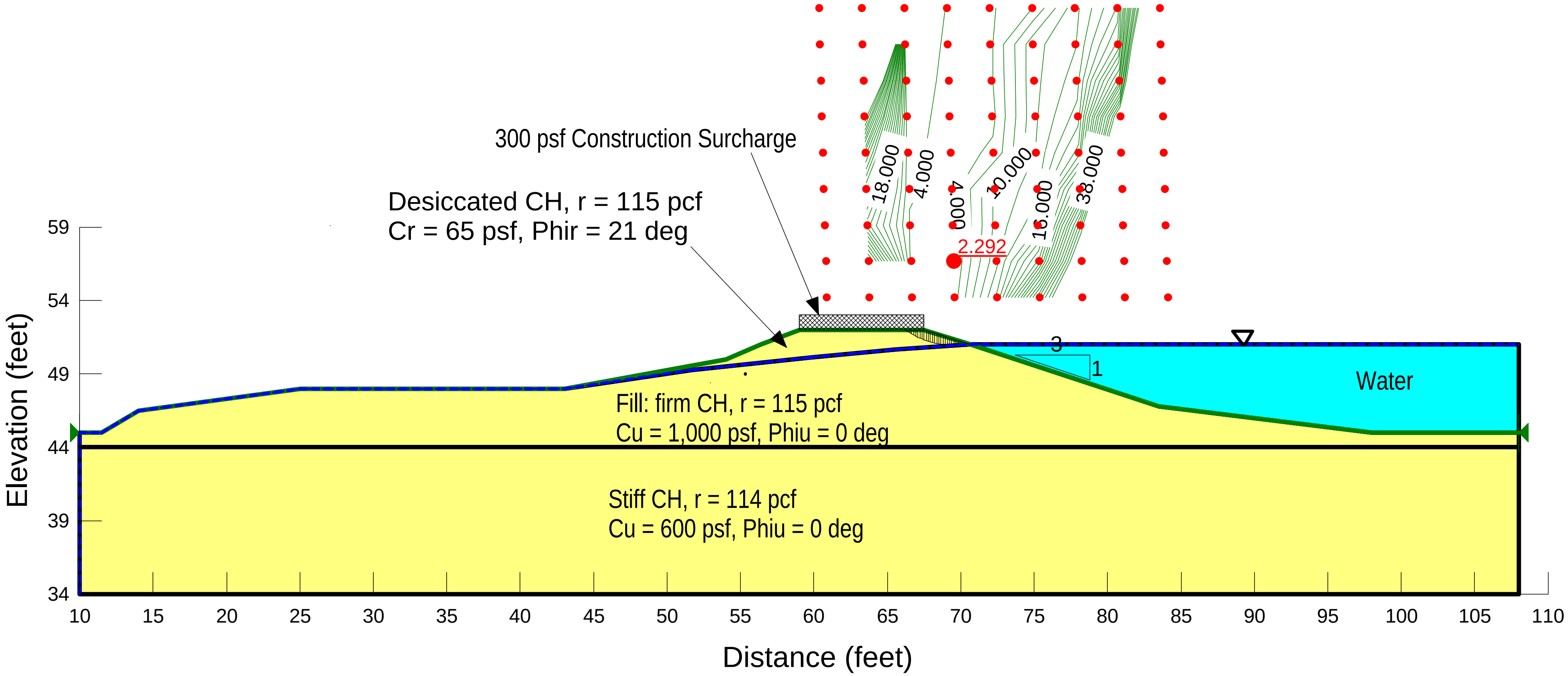


PLATE E-5b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION B-B - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, LONG TERM CONDITION

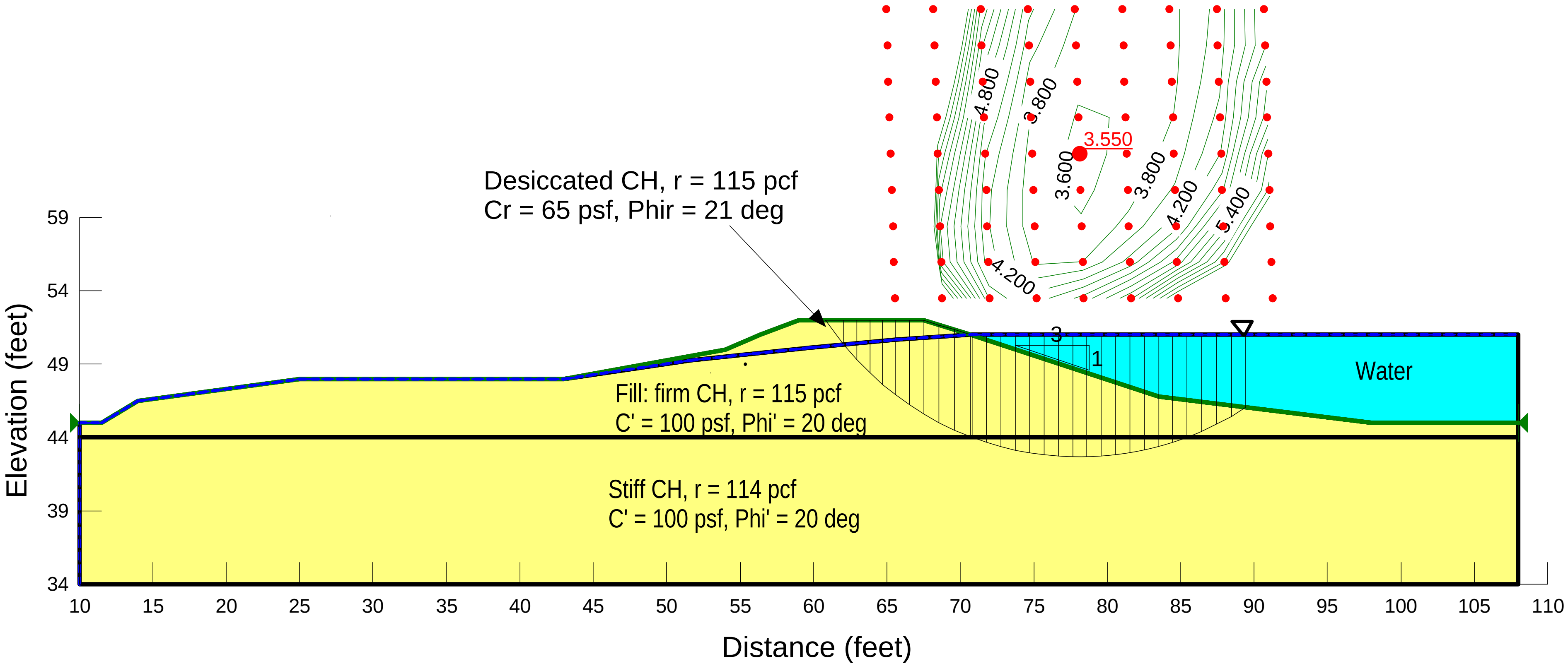


PLATE E-6a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION B-B - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, LONG TERM CONDITION

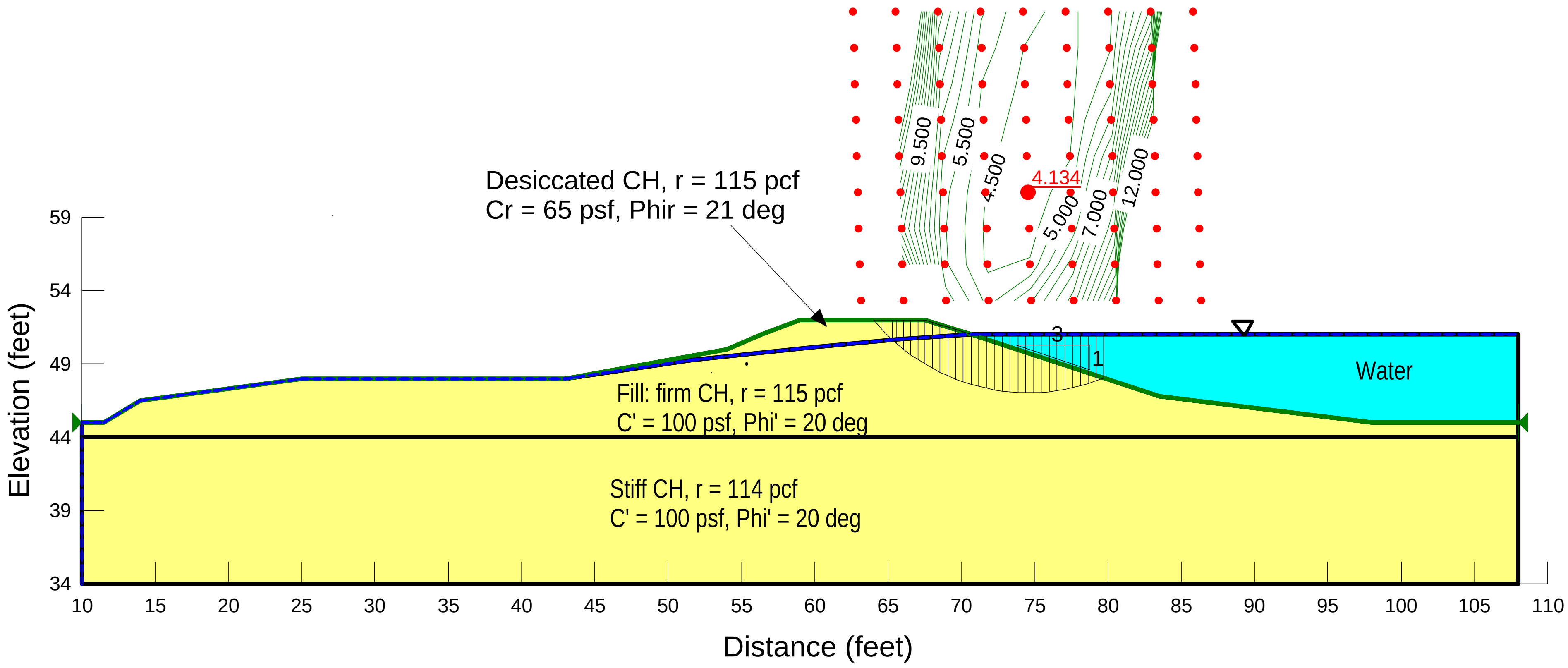


PLATE E-6b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION B-B - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, GLOBAL SLIDE FOR NORTH EMBANKMENT BASED ON BORING B-2, RAPID DRAWDOWN CONDITION (DURING MAINTENANCE)

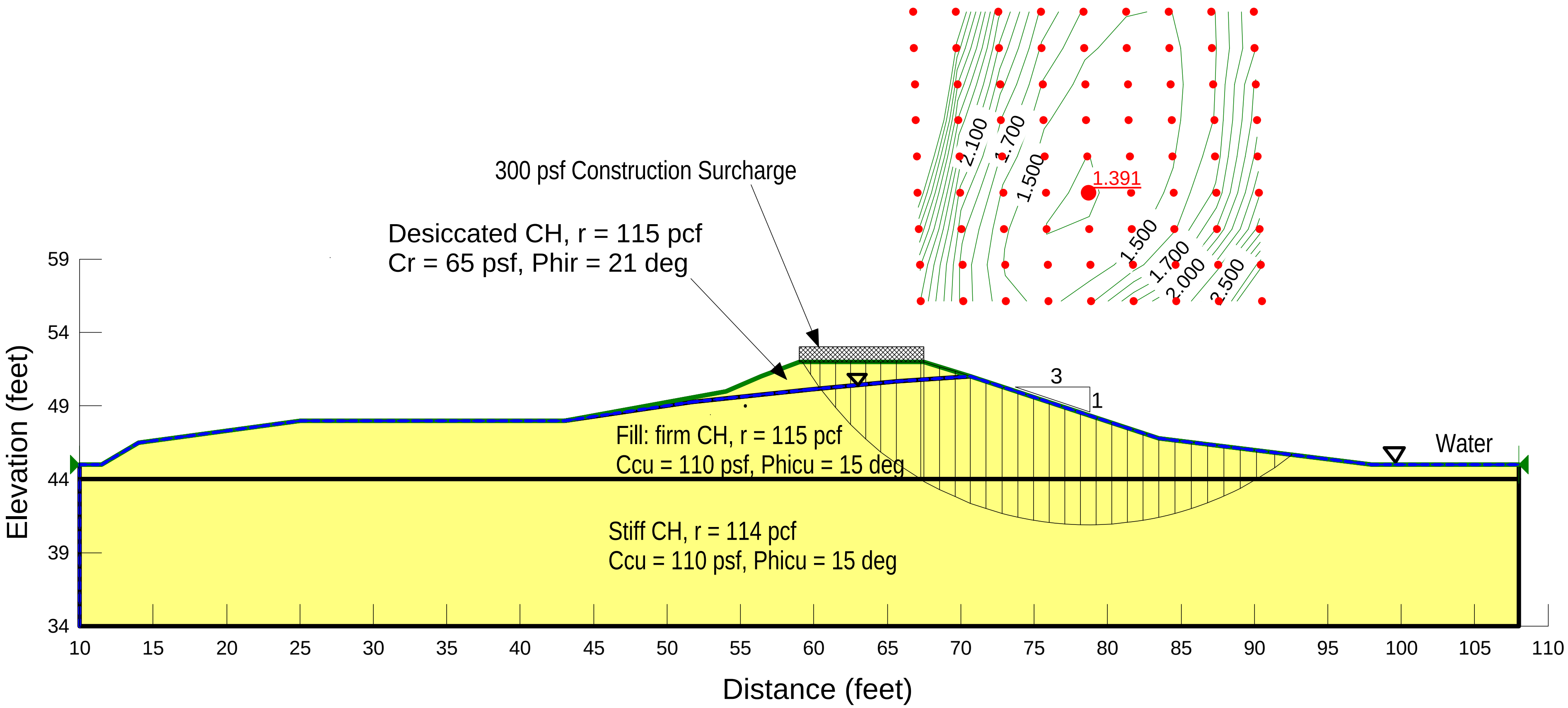


PLATE E-7a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION B-B - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 3:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-2, RAPID DRAWDOWN CONDITION (DURING MAINTENANCE)

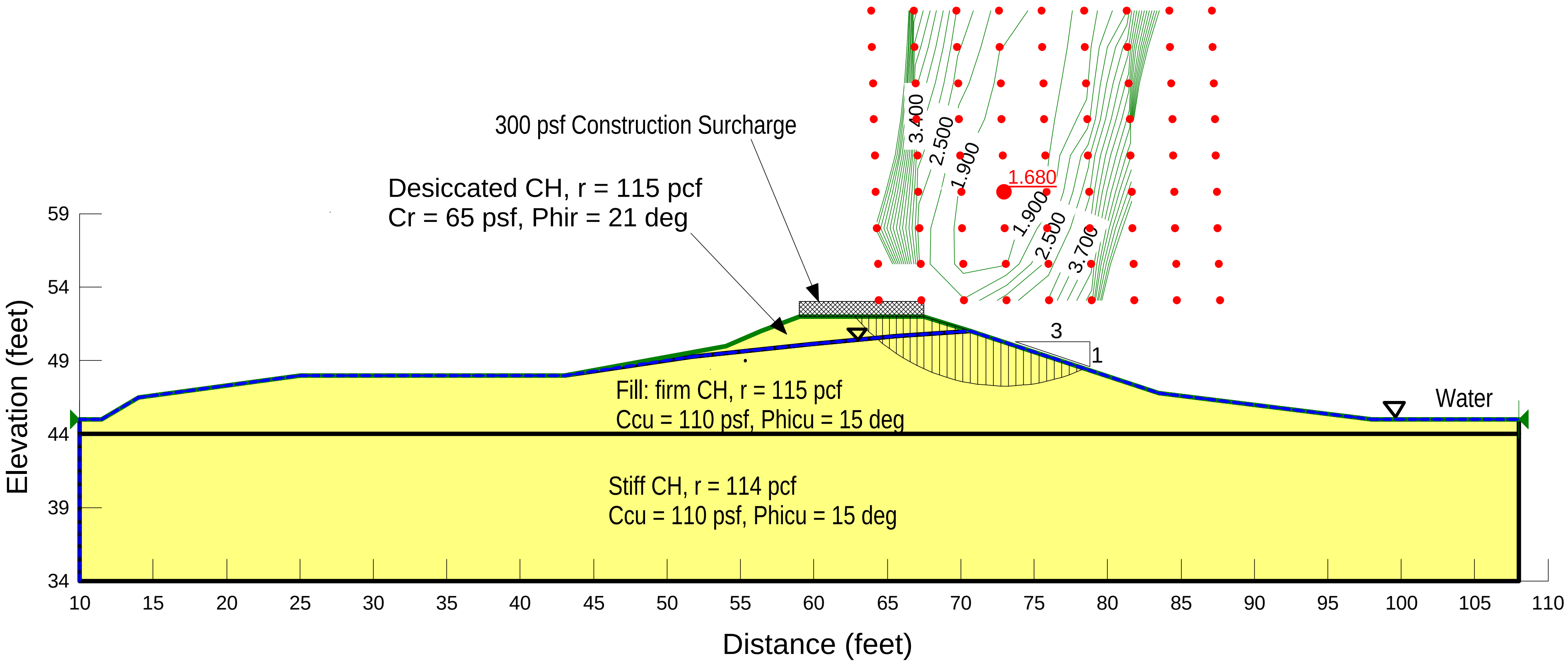


PLATE E-7b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 1.9:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, SHORT TERM CONDITION

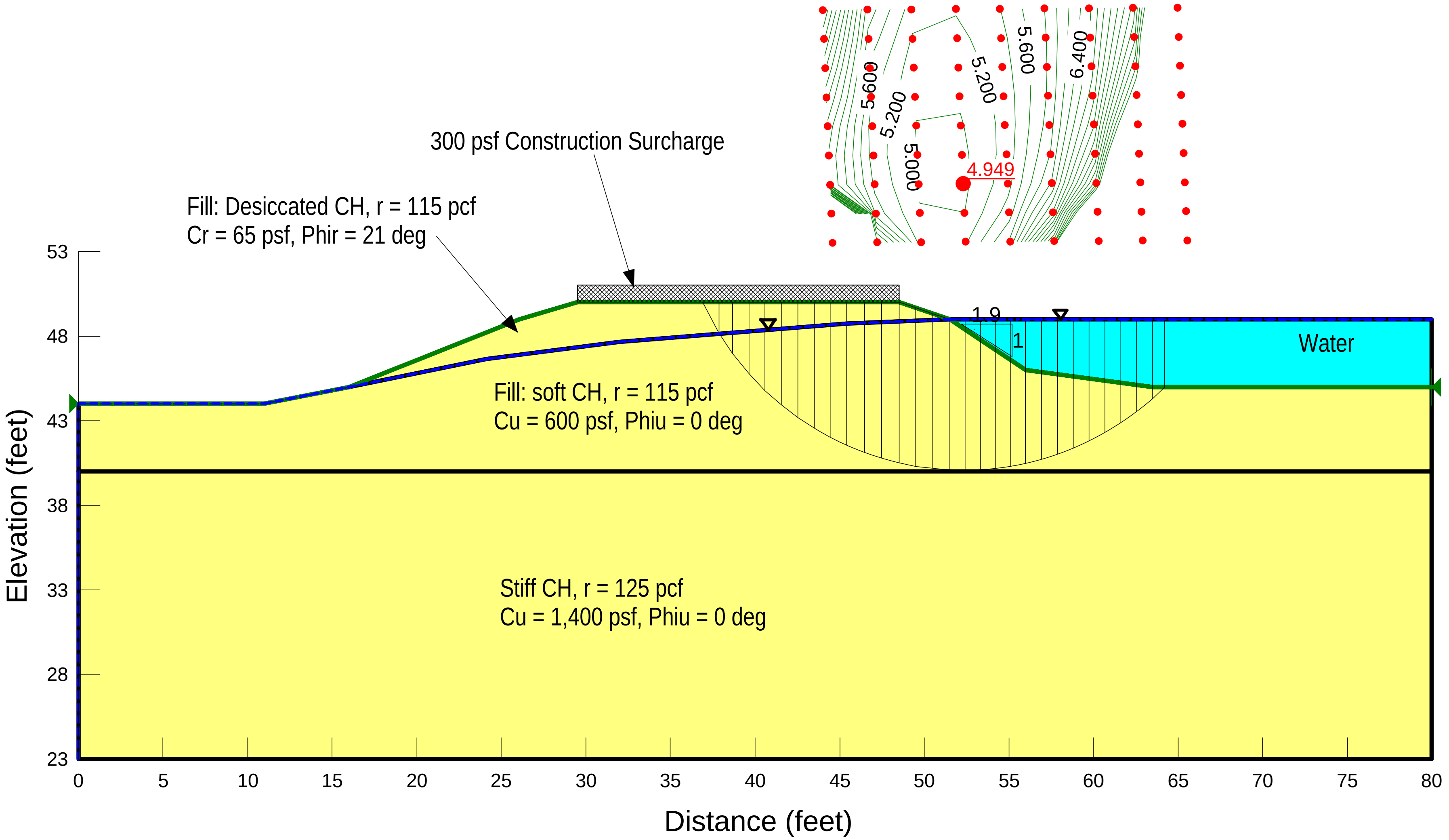


PLATE E-8a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 1.9:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, SHORT TERM CONDITION

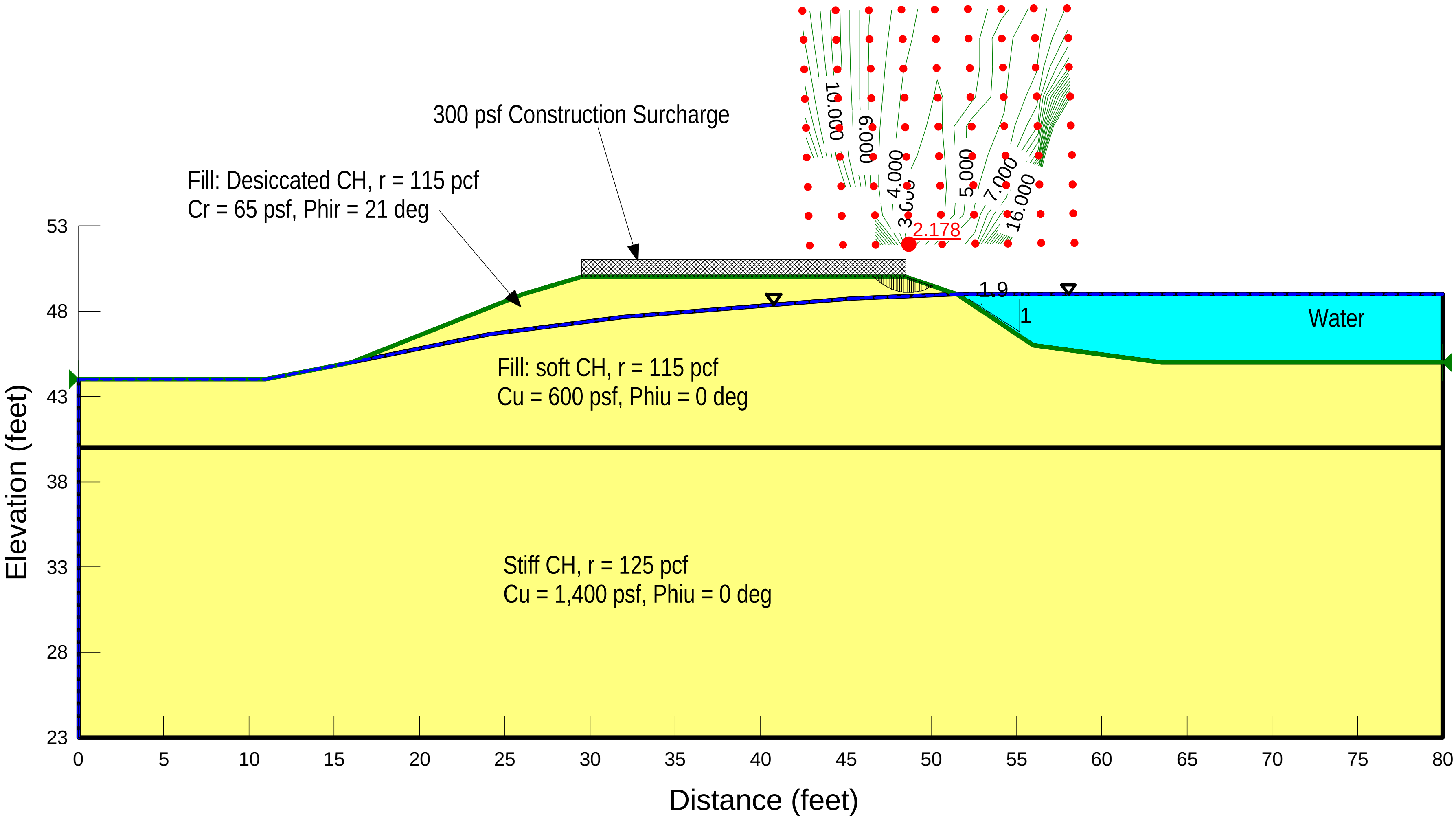


PLATE E-8b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 1.9:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, LONG TERM CONDITION

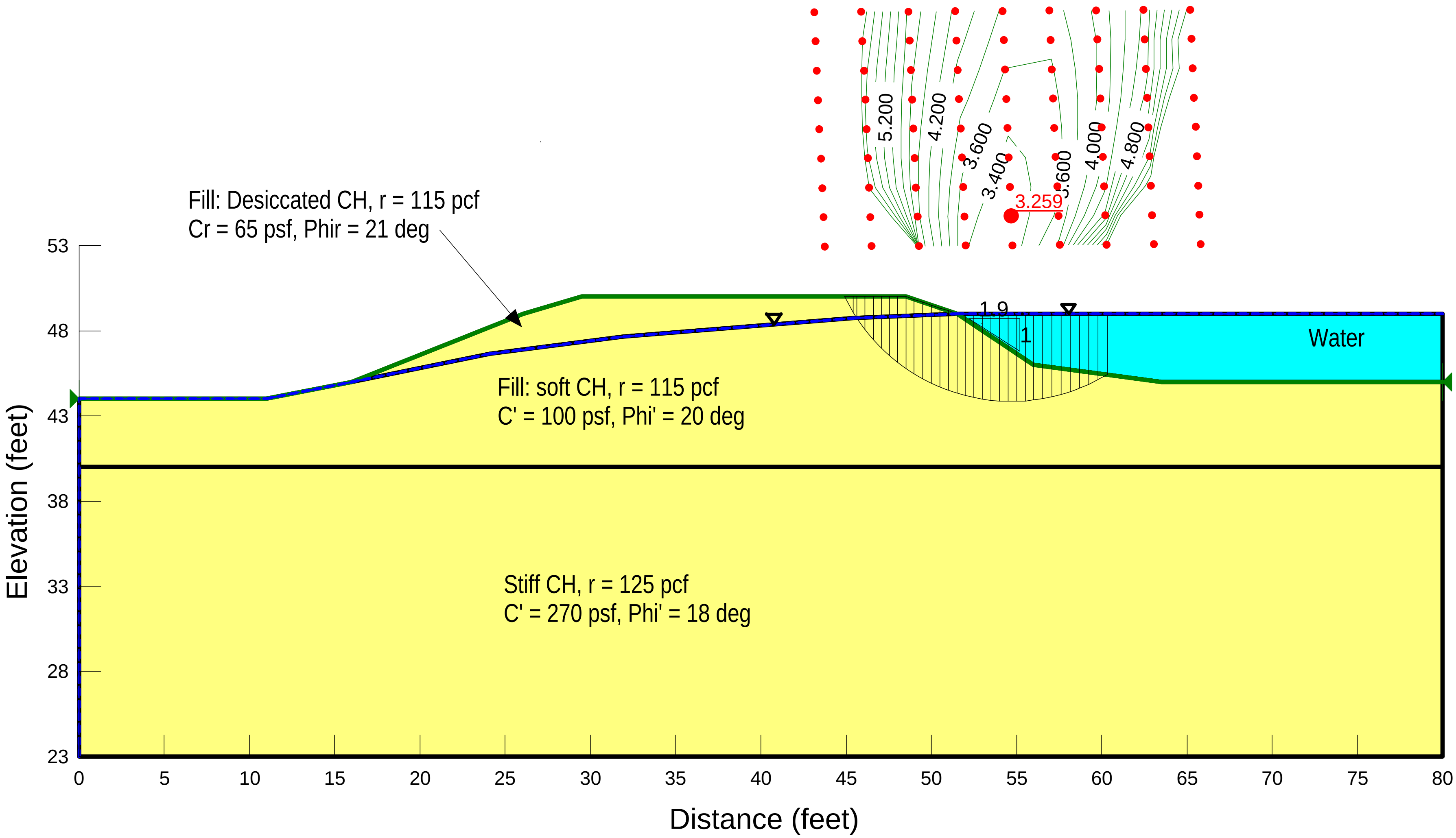


PLATE E-9a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 1.9:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, LONG TERM CONDITION

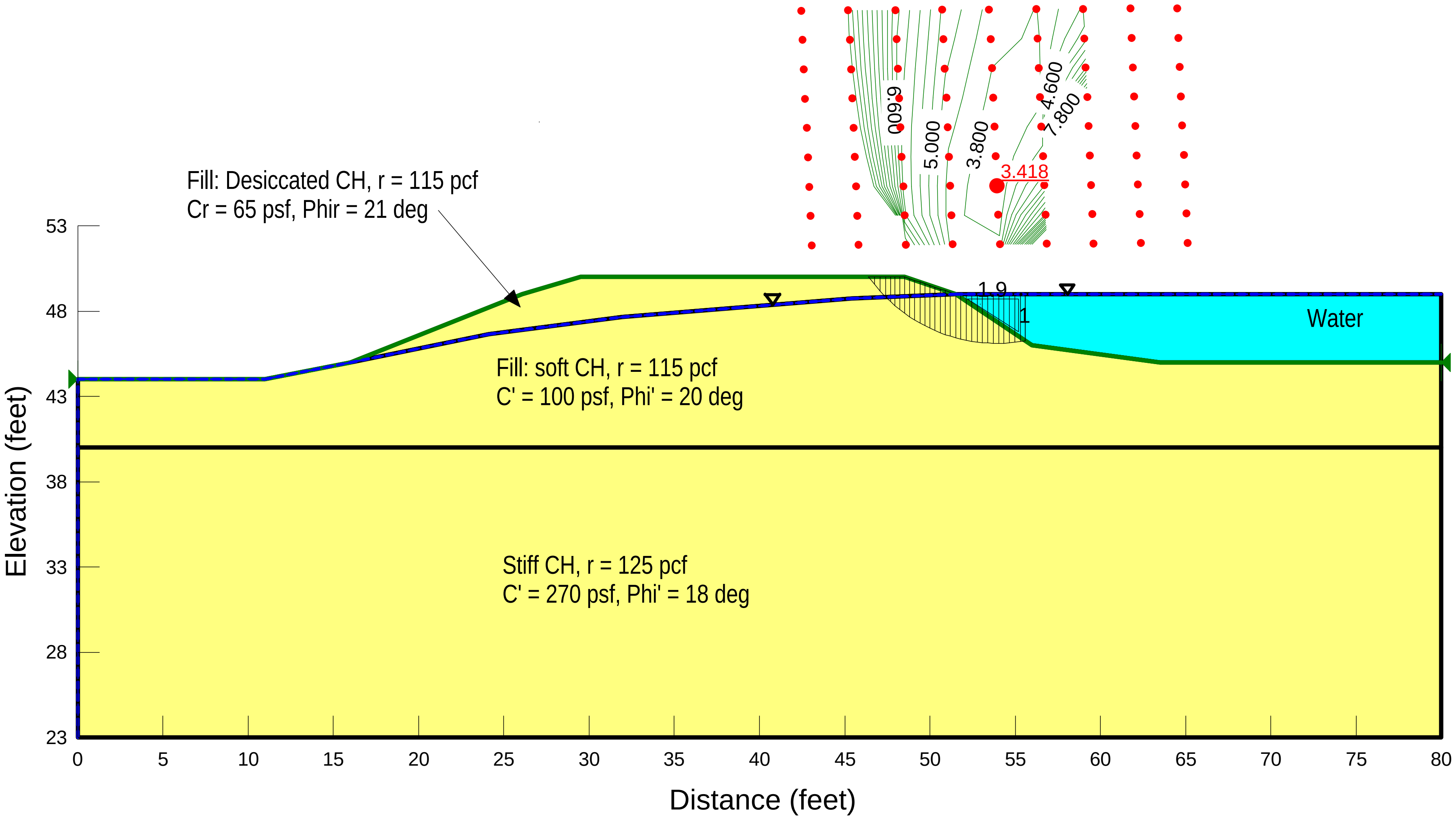


PLATE E-9b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 1.9:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, RAPID DRAWDOWN CONDITION (DURING MAINTENANCE)

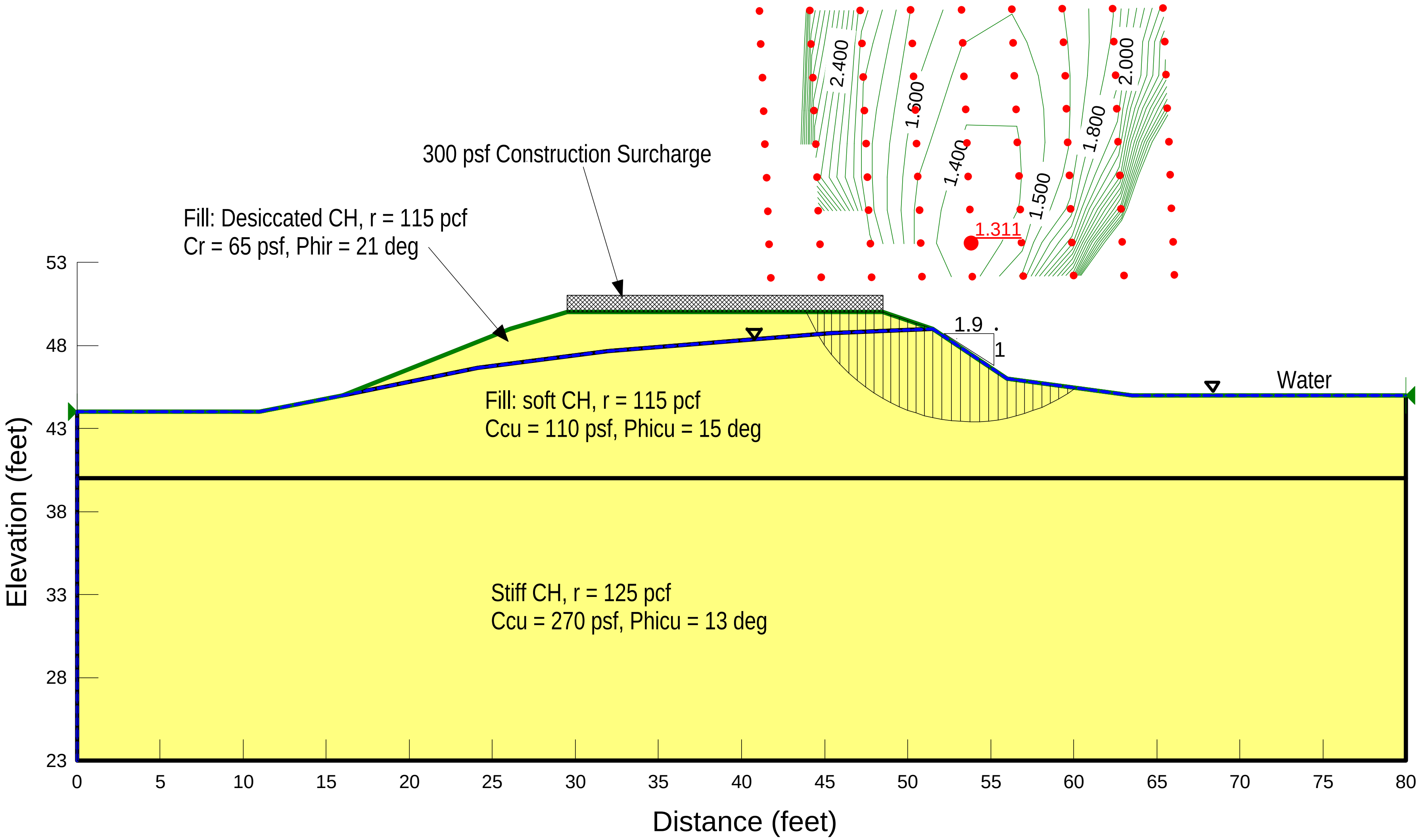


PLATE E-10a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING WATERSIDE SLOPE, UPPER SLOPE H:V = 1.9:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, RAPID DRAWDOWN CONDITION (DURING MAINTENANCE)

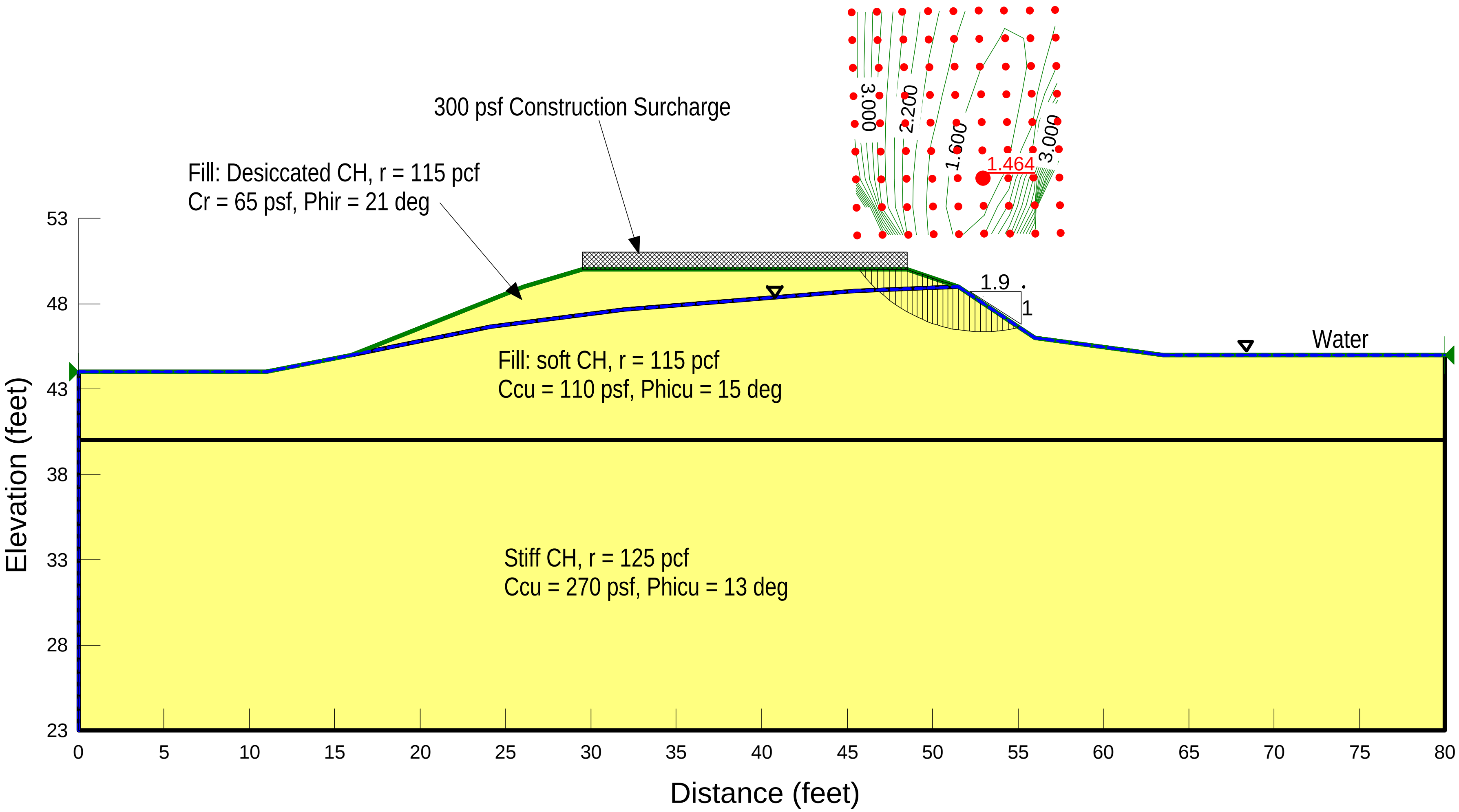


PLATE E-10b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING LANDSIDE SLOPE, UPPER SLOPE H:V = 3.1:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, SHORT TERM CONDITION

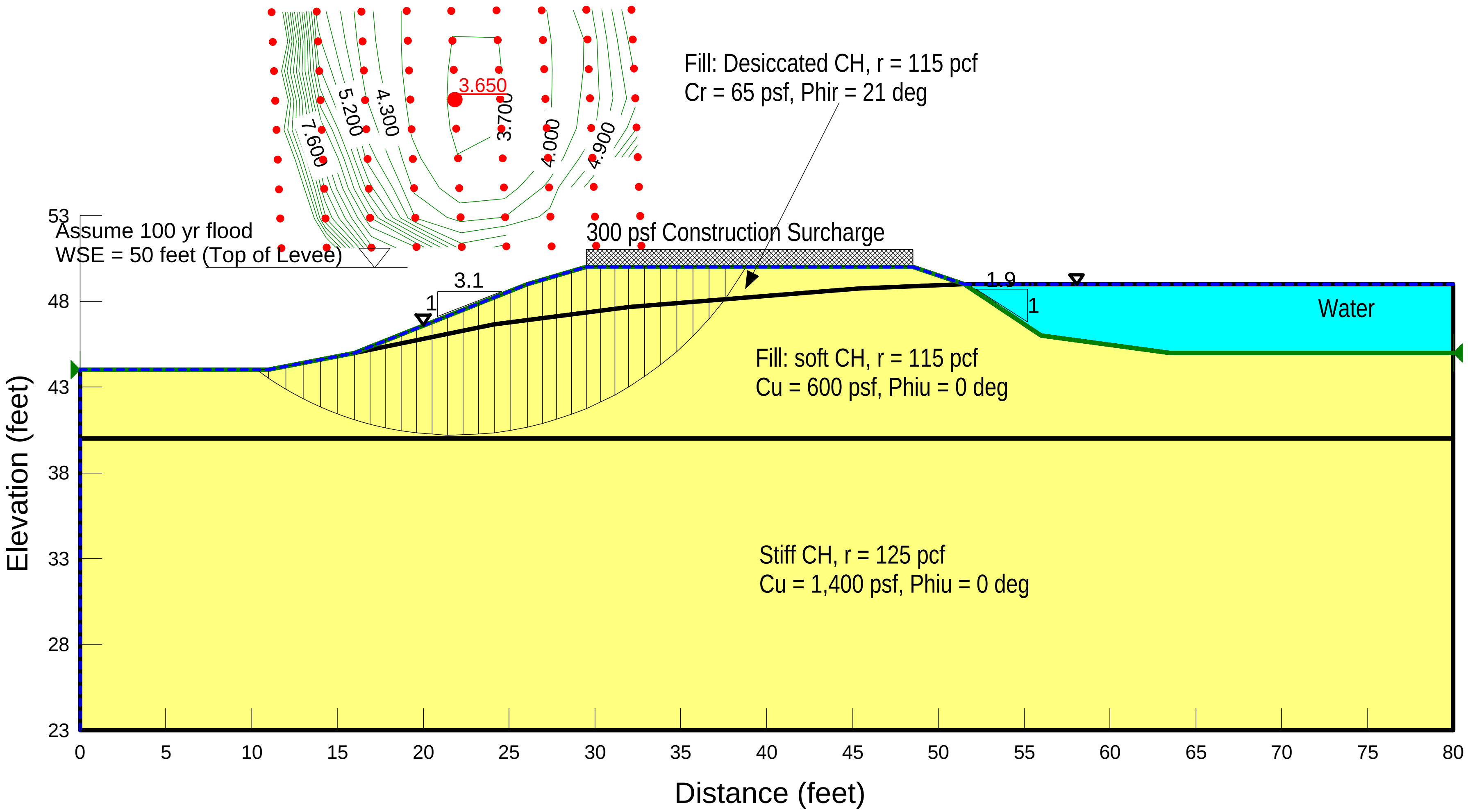


PLATE E-11a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING LANDSIDE SLOPE, UPPER SLOPE H:V = 3.1:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, SHORT TERM CONDITION

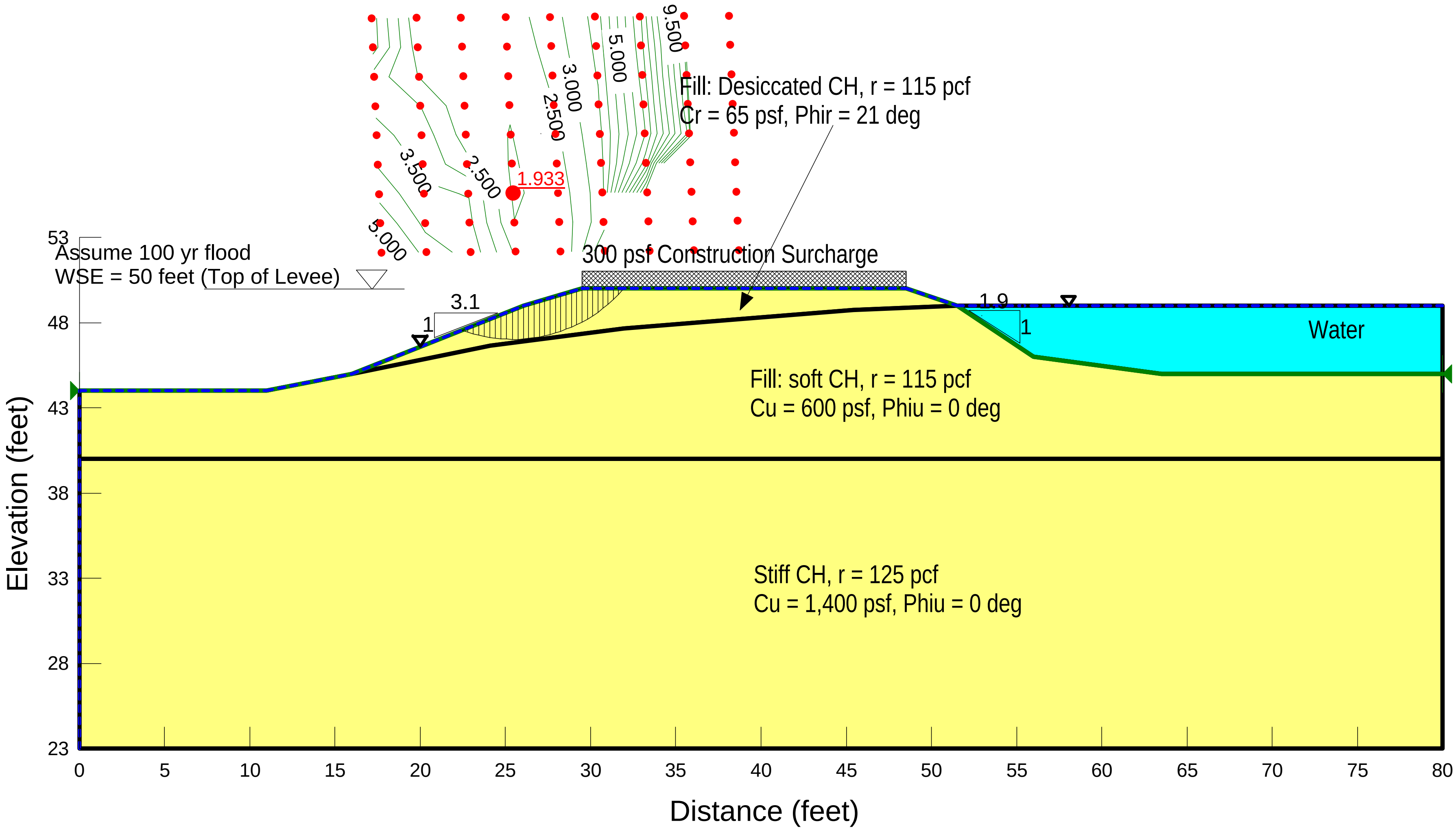


PLATE E-11b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING LANDSIDE SLOPE, UPPER SLOPE H:V = 3.1:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, LONG TERM CONDITION

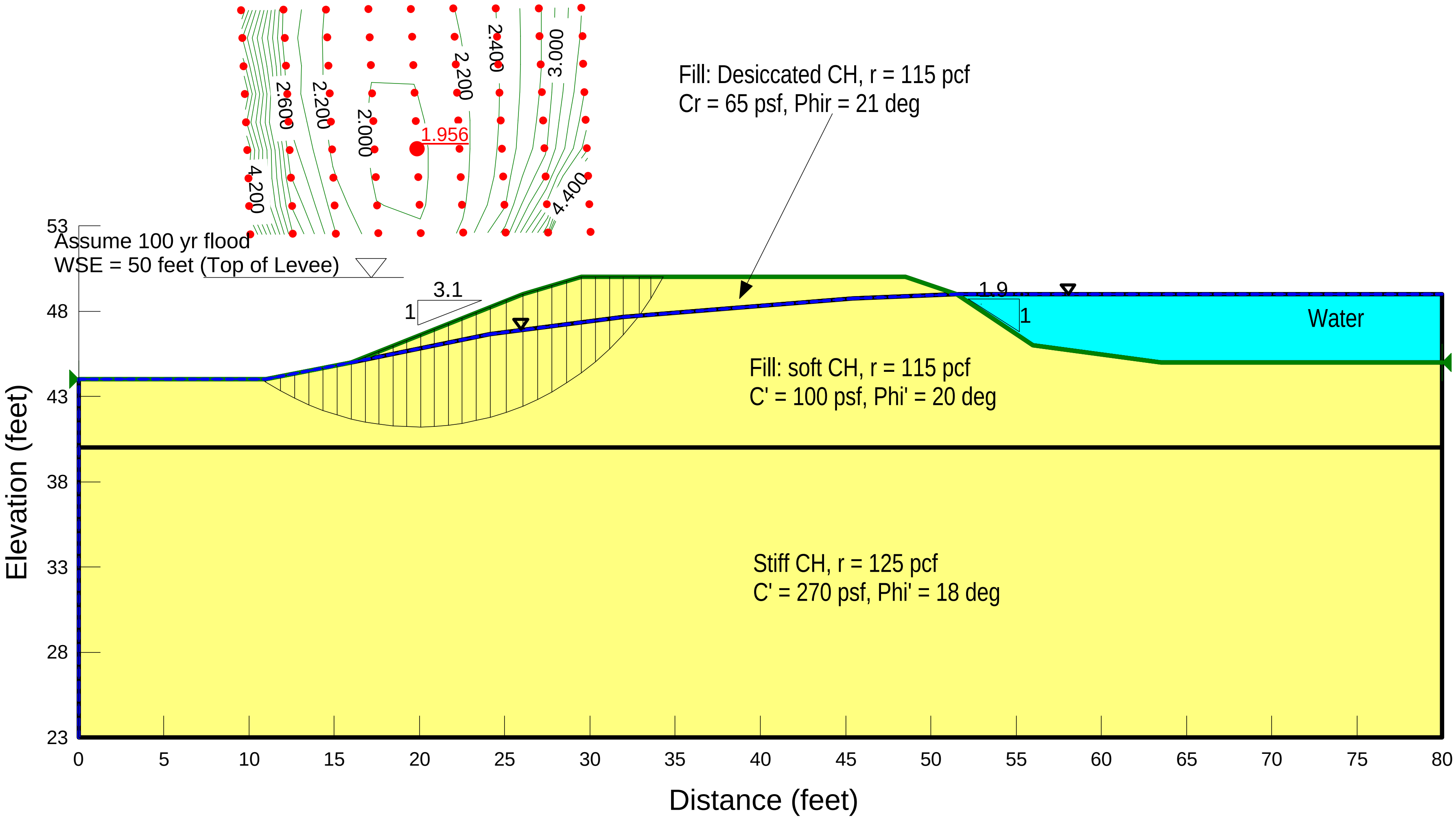


PLATE E-12a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING LANDSIDE SLOPE, UPPER SLOPE H:V = 3.1:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, LONG TERM CONDITION

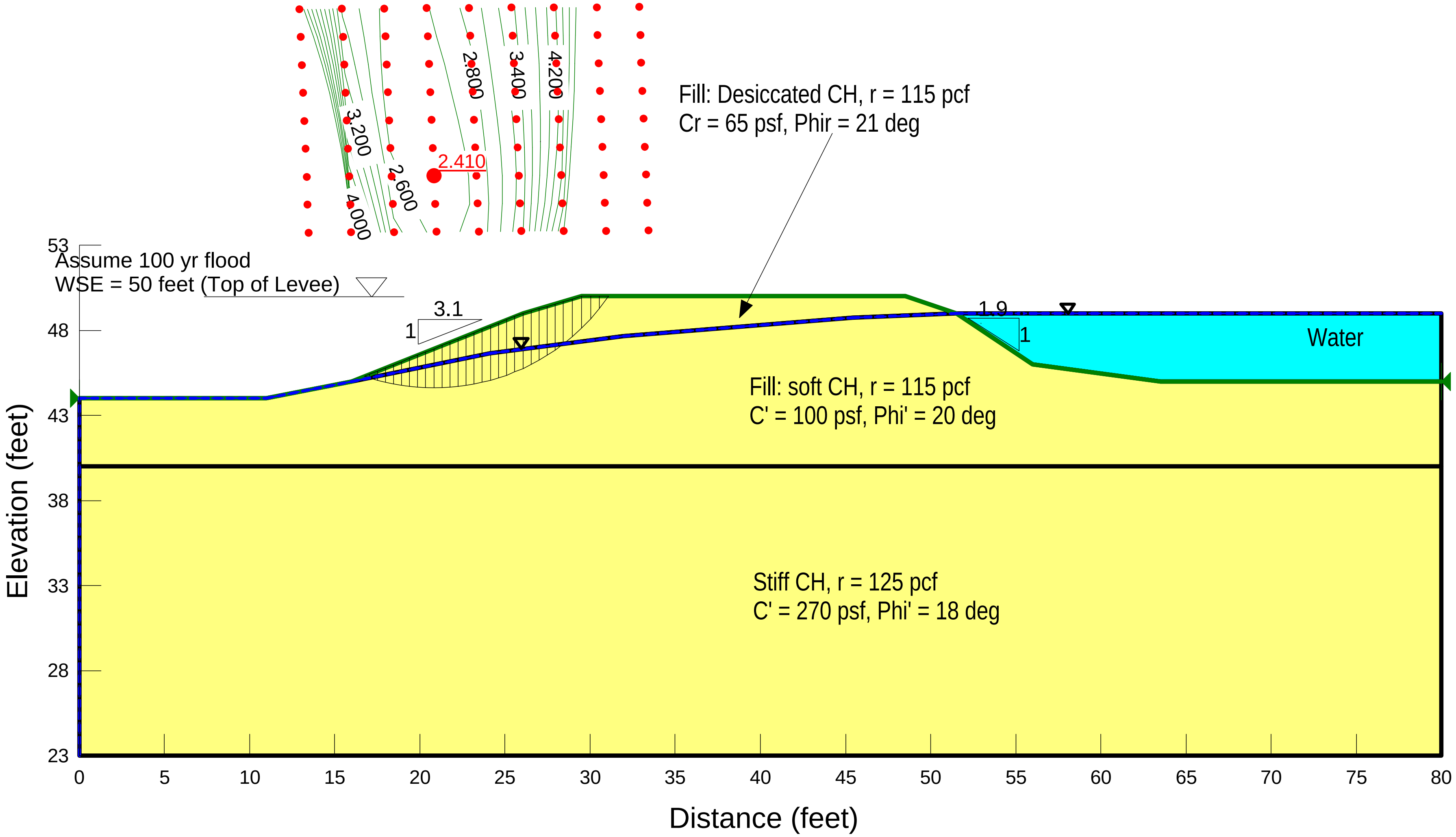


PLATE E-12b

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING LANDSIDE SLOPE, UPPER SLOPE H:V = 3.1:1, GLOBAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, RAPID DRAWDOWN CONDITION

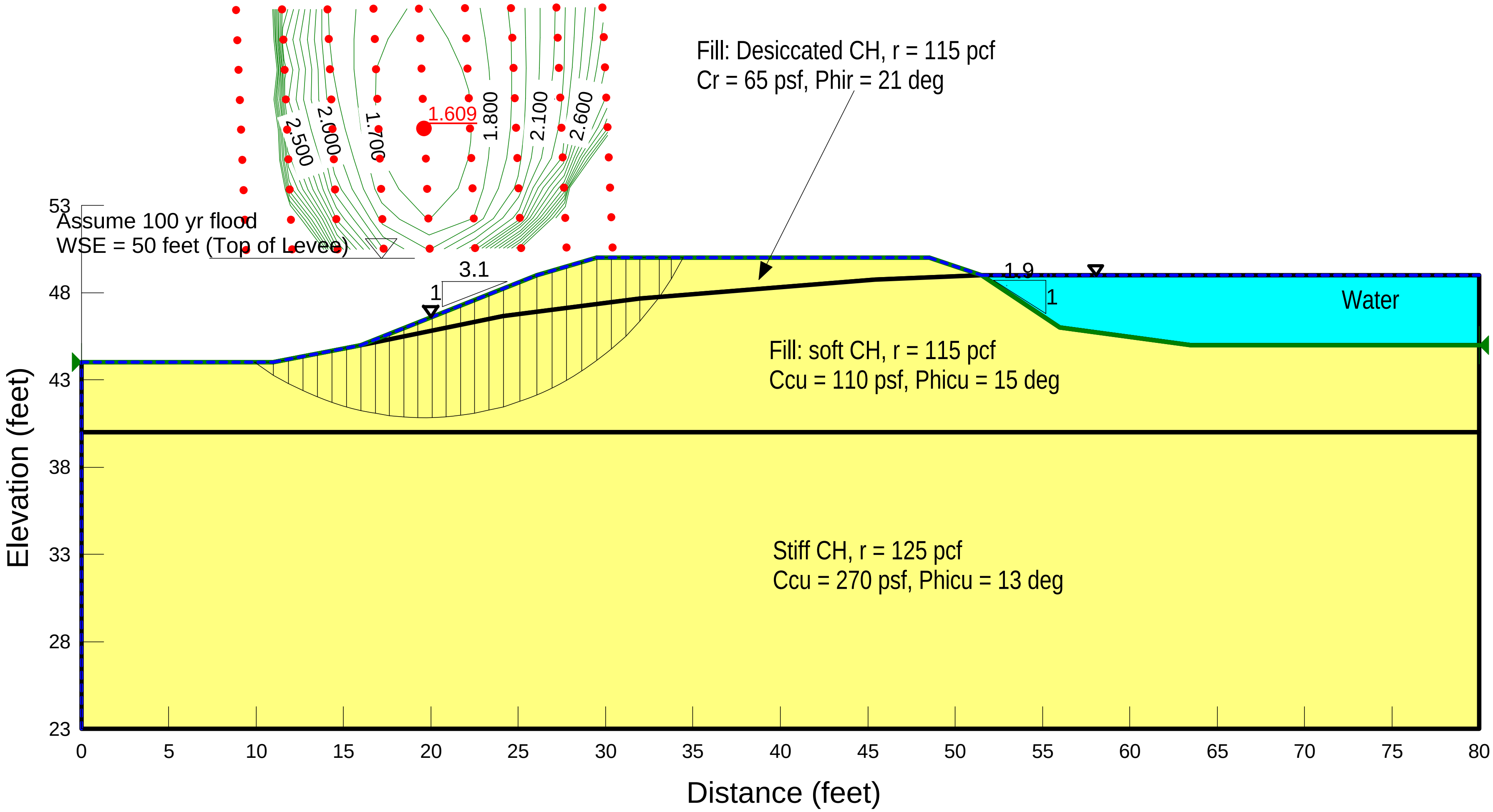


PLATE E-13a

G134-15 SJRA MAIN CANAL SIPHON NO. 7
CROSS SECTION C-C - SLOPE STABILITY ANALYSIS
EXISTING LANDSIDE SLOPE, UPPER SLOPE H:V = 3.1:1, LOCAL SLIDE FOR NORTH EMBANKMENT
BASED ON BORING B-1, RAPID DRAWDOWN CONDITION

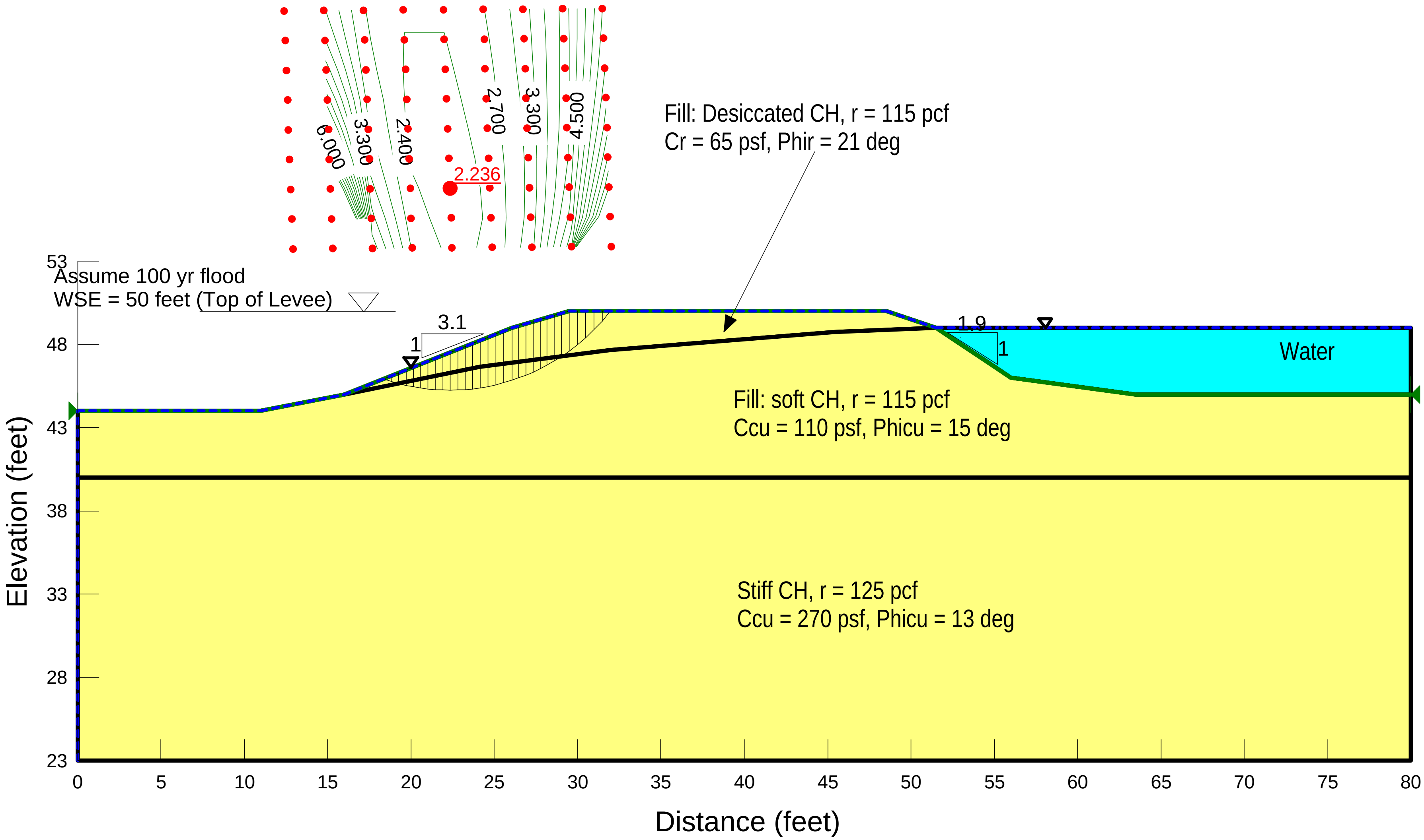


PLATE E-13b

2018 PIPE INSPECTION TECHNICAL MEMORANDUM BY V&A CONSULTING ENGINEERS

Technical Memorandum

San Jacinto River Authority Siphon 7 Investigation

Prepared for:	Daniel Hilderbrandt, P.E. Engineer 4 San Jacinto River Authority PO Box 329 Conroe, TX 77305
Prepared by:	Chris Hunniford, P.E.
Reviewed by:	Brian Huang, P.E.
Date:	June 2018

CHRISTOPHER D. HUNNIFORD
107411

6/13/2018

V&A
consulting engineers

Texas Registered Engineering Firm F-9154

V&A Project No. 18-0099

This document has been designed for double-sided printing; blank pages are intentional.

1 Introduction

V&A Consulting Engineers (V&A) was retained by San Jacinto River Authority (SJRA) for the condition assessment of the SJRA Siphon 7 located near 16900 Crosby Huffman Road, Crosby, TX. Siphon 7 consists of dual 48-inch diameter Reinforced Concrete Pipes (RCP) and a 60-inch diameter RCP approximately 125 Linear Feet (LF) and 135 LF in length, respectively. The siphon pipes share an inlet and outlet structure headwall and convey flow from the SJRA canal system under Crosby Huffman Road.

The purpose of the assessment was to verify the size of each pipe and location of observed defects. The condition assessment methods included pipe diameter measurement, surface pH measurement, visual observations, non-destructive concrete penetration testing, and photographic documentation. V&A performed a confined space manned-entry assessment of the three siphon pipes. This report summarizes the findings of this assessment.

Figure 1-1 shows the arrangement of the siphon. Flow is conveyed from West to East. The 60-inch line is on the south side. The two 48-inch lines (North and South) are on the North side.

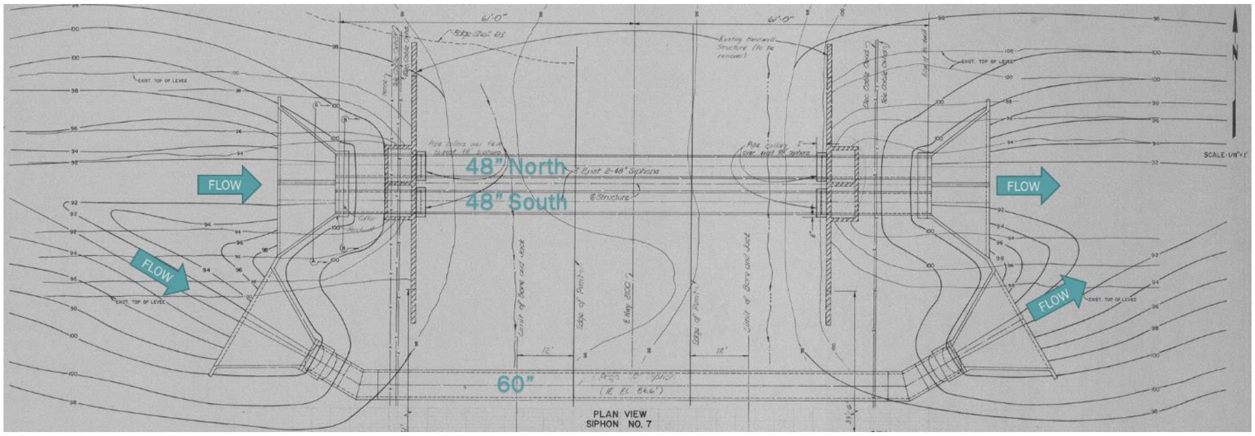


Figure 1-1. Siphon 7 Plan View

Figure 1-2 shows the arrangement of the outlet structure (downstream/east end). The inlet structure is a mirror image (left to right along page orientation) of the outlet structure.

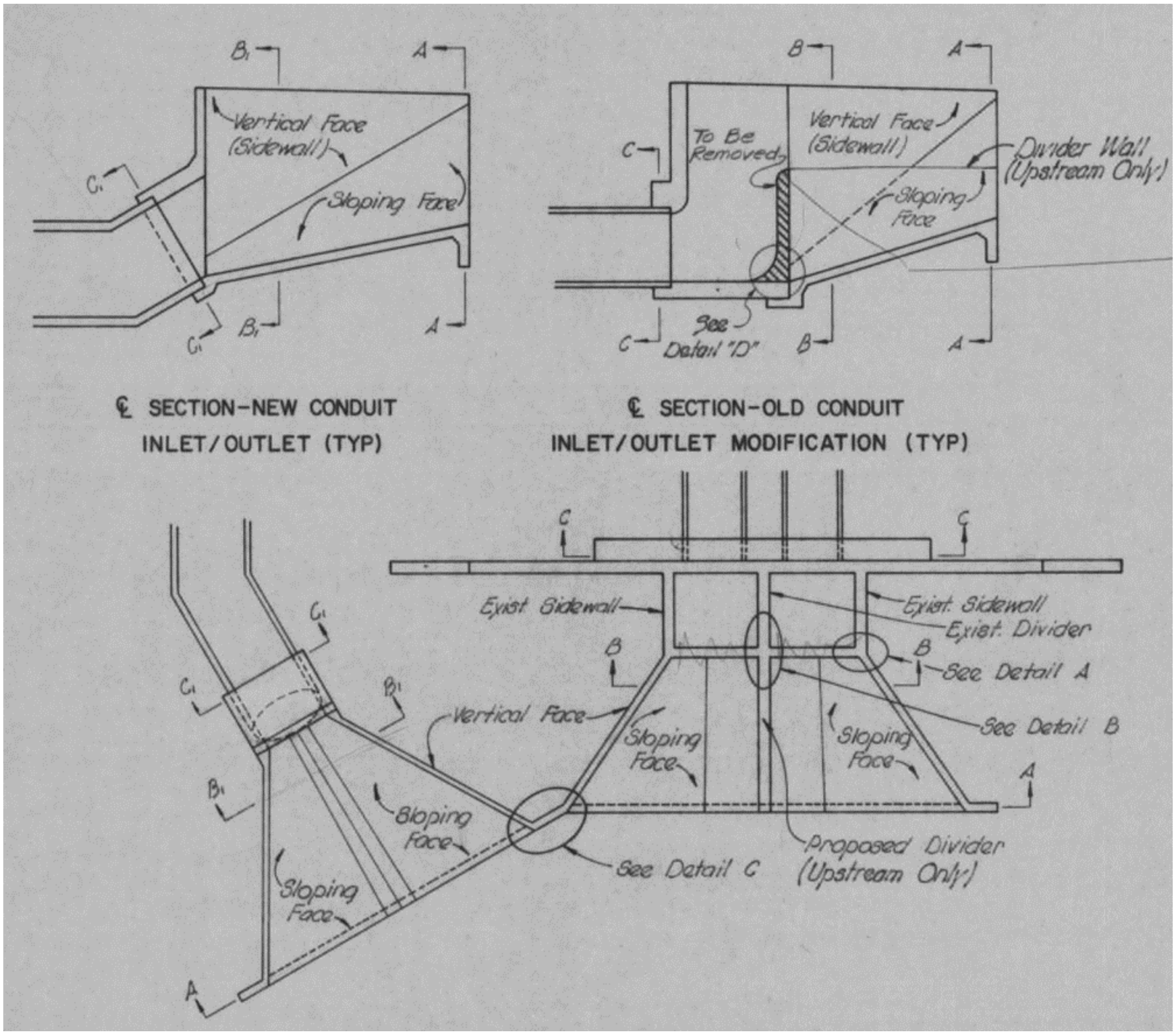


Figure 1-2. Siphon 7 Outlet Structure

2 Approach

2.1 Access

The Siphon 7 condition assessment was conducted by personnel trained in confined space entry (CSE). The entry was performed on May 18, 2018 during the daytime with the canal system shut down and the siphon dewatered.

The siphon pipes were treated as permit-required confined spaces (PRCS) due to the difficulty of ingress and egress to the structures and the potential atmospheric and engulfment hazards that may exist. A health and safety plan was prepared and a pre-entry tailgate meeting was conducted immediately prior to the entry. For the assessment of the siphon pipes, the entrant made access to each pipe from the upstream/west side and exited on the downstream/east side. All entries were performed using rope and harness assembly (Photo 2-1). Entrants remained connected to a safety line while performing condition assessment activities within the confined space and maintained 2-way radio communication throughout the assessment.

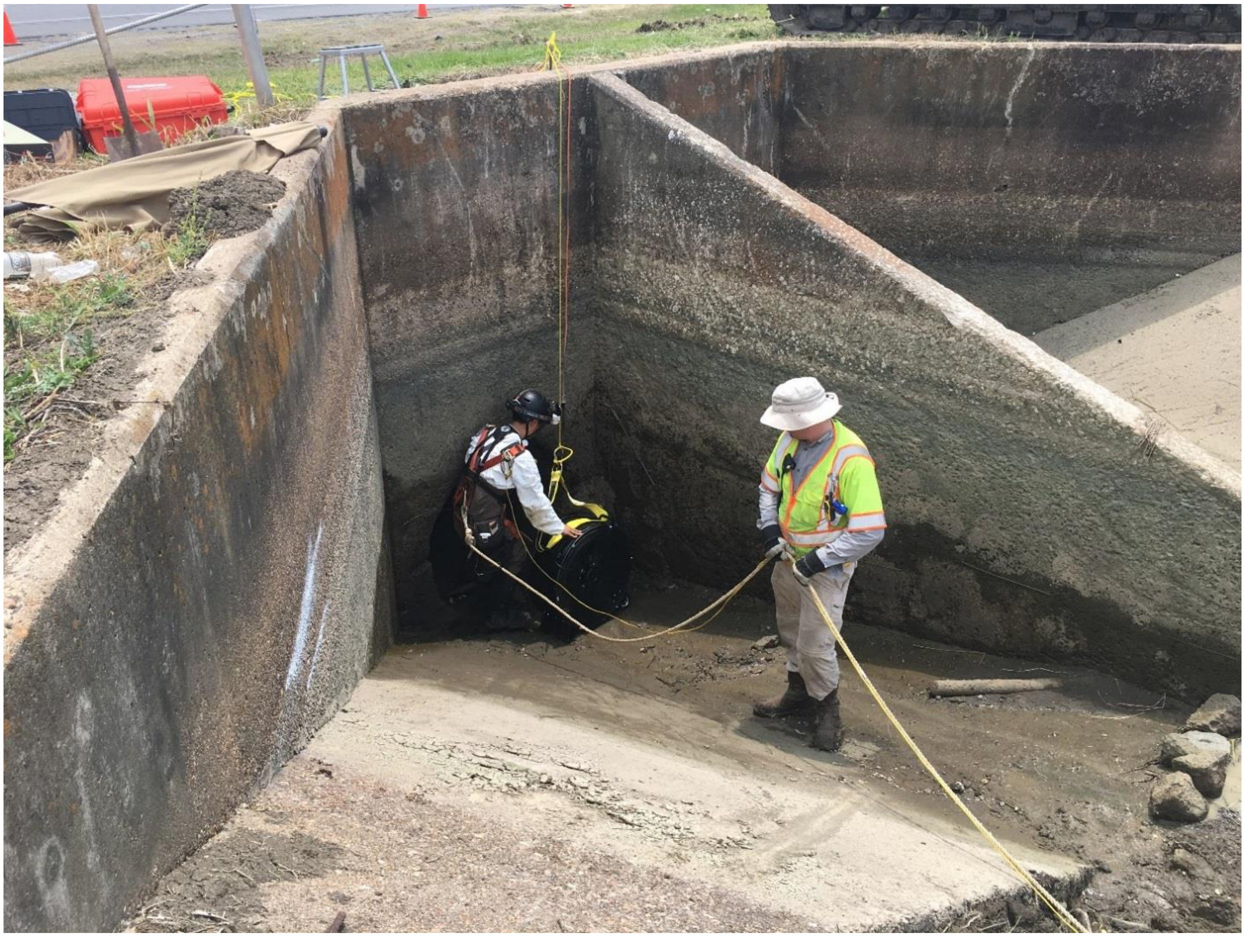


Photo 2-1. Confined Space Entry at SJRA Siphon 7 48-inch North

Appropriate personal protective equipment (PPE) was worn by the entrant and included a 4-gas monitor to continuously sample the atmosphere in the confined spaces. The monitors were calibrated to alarm if threshold values of hydrogen sulfide (H_2S), carbon monoxide (CO), and/or LEL (explosive) gases were present, or if safe oxygen (O_2) levels were not present in the air. Forced air ventilation was used to mitigate atmospheric hazards.

One V&A staff entered the siphon pipe, and two V&A staff provided supporting roles as attendants on each end of the siphon. SJRA provided dewatering of the inlet/outlet structures and the majority of the siphon pipes. Sand bags and a bypass pump were used to prevent backwatering from the downstream side of the canal into the siphon. V&A dewatered the remaining water within the pipes using transfer pumps. V&A followed local, state, federal, and industry standard health and safety guidelines.

Photo 2-2. Dewatering at Downstream End of Siphon 7

2.2 Visual Assessment

Observations made during the condition assessment of the siphon pipes were documented with digital photographs. The visual assessment focused on the condition of concrete and metallic surfaces comprising the pipe walls. Observations such as spalling, holes, and exposed/corroded reinforcing steel were recorded when found. Pipeline joints were evaluated for offsets, infiltration, gaps, and other notable items. It should be noted that much of the condition assessment data is subjective and is based upon V&A's extensive experience evaluating concrete facilities in the wastewater industry.

2.3 Dimension Measurements

V&A measured the internal pipe diameter at various locations for verification and assessment purposes. Diameter measurements were generally taken at the inlet, mid-point, and outlet.

2.4 Debris Removal

V&A assisted SJRA with debris removal from within the siphon pipes. Debris consisted primarily of rocks less than 24-inches in any one dimension, wood products (including tree debris and finished boards), and other miscellaneous trash. Debris was placed at the inlet and outlet structures for removal from the canal by SJRA.

2.5 Concrete Penetration Testing

Penetration measurements involve applying a consistent level of force from a pointed tool to the concrete surface until sound, hard material is reached, and then measuring the depth of the resulting cavity. The cavity depth provides quantitative data on the integrity and condition of the concrete surfaces. Typically, as concrete deteriorates, the cement paste begins to lose integrity and becomes soft. Carbonation and exposure to aggressive water chemistry (high sulfate, low pH) are typical causes of degraded concrete surface hardness. A measure of the loss of concrete surface hardness based on depth of penetration measurements is displayed in Table 2-1. While the test is subjective due to variations in applied force, it does provide a means of comparison between different portions of the study area and general trends over time.

Table 2-1. Evaluation of Concrete Surface Hardness

Penetration Depth (in.)	Loss of Surface Hardness
> 1/4	Significant
1/8 – 1/4	Moderate
1/16 – 1/8	Minor
< 1/16	Negligible

2.6 Concrete Surface pH Testing

The corrosion of concrete and other cementitious materials is of primary concern in water pipelines that rely on this material to provide passivation of the underlying steel. Concrete is an extremely versatile and inexpensive construction material, particularly for large hydraulic structures and pipes. Therefore, when this universal building material cannot perform adequately, it presents a significant challenge for the designer.

In general, with conventional concrete mix designs using common Type II Portland cements, concrete has the ability to withstand moderately low pH surfaces (≈ 6.0) for long periods of time. The generally accepted ranges for corrosion categories and surface pH values are listed below:

1. **Severe Corrosion.** This category of concrete corrosion is characterized by significant measurable concrete loss or active corrosion. There is exposed aggregate and occasional exposed reinforcing steel. The original concrete surface is not distinguishable. The surface is covered with soft, pasty corrosion

products where active scouring is not present. There is generally a depressed wall pH (< 3.0) indicating active corrosion.

2. **Moderate Corrosion.** This category of concrete corrosion is characterized by some concrete loss with aggregate slightly exposed but the original concrete surface is still distinguishable. The surface may have a thin covering of pasty material which is easily penetrated. There is generally a depressed wall pH (< 5.0) indicating moderately corrosive conditions.
3. **Light Corrosion.** This category of concrete corrosion is characterized by a slightly depressed pH (< 6.0) and a concrete surface that can be scratched with a sharp instrument under moderate hand pressure with the removal of some concrete material. The original concrete surface is fully recognizable and aggregate may or may not be exposed.
4. **Negligible Corrosion.** This category of concrete corrosion is characterized by normal pH ranges (>6.0) and a normal concrete surface which cannot be penetrated or removed by a sharp instrument under moderate hand pressure. The surface of the concrete may have biological growth and moisture but the concrete is normal and the aggregate is not exposed.

Table 2-2 summarizes the surface pH criteria to determine the severity of corrosion on a concrete pipe.

Table 2-2. pH and Corrosivity Correlation for Reinforced Concrete

pH	Degree of Corrosivity
< 3	Severe
3 to 5	Moderate
5 to 6	Light
> 6	Negligible

2.7 VANDA® Reinforced Concrete Condition Index

The VANDA® Reinforced Concrete Condition Index was created by V&A to provide consistent reporting of corrosion damage based on qualitative, objective criteria. As shown in Table 2-3, the condition of concrete corrosion can vary from Level 1 to Level 4 based upon visual observations and field measurements, with Level 1 indicating the best condition and Level 4 indicating severe damage. In general, Level 1 and 2 conditions do not require remedial action. However, sometimes recommendations are presented for Level 2 observations to prolong the useful life of a structure. Level 3 warrants remedial action such as minor repairs or coating to prolong useful life. Level 4 warrants repair and/or replacement. Note that these guidelines are based on generally acceptable industry standards and do not represent an engineering analysis of the Siphon 7 conditions.

Table 2-3. VANDA® Reinforced Concrete Condition Index

Condition Rating	Description	Representative Photograph
Level 1	**None/Minimal Damage to Concrete** • Hardness: No Loss • Surface Profile: No Loss • Cracking: Shrinkage Cracks • Spalling: None • Reinforcing Steel (Rebar): Not Exposed or Damaged	
Level 2	**Damage to Concrete Mortar** • Hardness: Damage to Concrete Mortar • Surface Profile: Some Loss • Cracking: Thumbnail Sized Cracks of Minimal Frequency • Spalling: Shallow Spalling of Minimal Frequency, Related Rebar Damage • Reinforcing Steel (Rebar): May Be Exposed but Not Damaged	
Level 3	**Loss of Concrete Mortar/Damage to Rebar** • Hardness: Complete Loss • Surface Profile: Large Diameter Exposed Aggregate • Cracking: ¼-inch to ½-inch Cracks, Moderate Frequency • Spalling: Deep Spalling of Moderate Frequency, Related Rebar Damage • Reinforcing Steel (Rebar): Exposed and Damaged, Can Be Rehabilitated	
Level 4	**Rebar Severely Corroded/Significant Damage to Structure** • Hardness: Complete Loss • Surface Profile: Large Diameter Exposed Aggregate • Cracking: ½-inch Cracks or Greater, High Frequency • Spalling: Deep Spalling at High Frequency, Related Rebar Damage • Reinforcing Steel (Rebar): Damaged or Consumed, Loss of Structural Integrity	

2.8 Infiltration

Water infiltration was noted and coded based on the National Association of Sewer Service Companies (NASSCO) Pipeline Assessment Certification Program (PACP) infiltration guidelines. The descriptors for the infiltration from the PACP manual are as follows:

1. **Stain** – No moisture present during the inspection but a watermark indicates water has entered in the past
2. **Weeper** – The slow ingress of water through a defective or faulty joint or pipe wall. No visible drips.
3. **Dripper** – Water dripping through a defect of faulty joint or pipe wall. Not a continuous flow.
4. **Runner** – Water running into the sewer through a faulty joint or pipe wall. A continuous flow will be visible
5. **Gusher** – Water entering the pipe “under pressure” through a defect of faulty joint.

3 Findings

The overall findings indicate that the Siphon 7 pipes are comprised of 48-inch and 60-inch diameter reinforced concrete pipes with a VANDA concrete rating of level 1 for the majority of pipe. The concrete surface was hard with no observable penetration and exhibited a pH between 6 and 8. Joints with missing mortar, offset joints, and separated joints were observed throughout. Most of the joint gaps were found near the inlet and outlet side of the pipe where the siphon pipe slopes down/up towards the pipe outlet. Some areas of surface delamination and damaged mortar were also observed near joints. A sag in the pipe was observed in the 48-inch South pipe, near the location of a diagonal cold joint, resulting in approximately 9-inches of standing water. Observed pipeline lengths were consistent with values reported by SJRA. Figure 3-1 presents a summary of the findings.

The following section provides detailed observations for each siphon pipe and the results of testing performed. Clock and photo positions are with respect to the downstream direction, unless otherwise noted. Distances are measured relative to the inlet of the pipe. Findings are presented in the order in which each pipe was assessed. All photos from the internal assessment are presented in Appendix A.

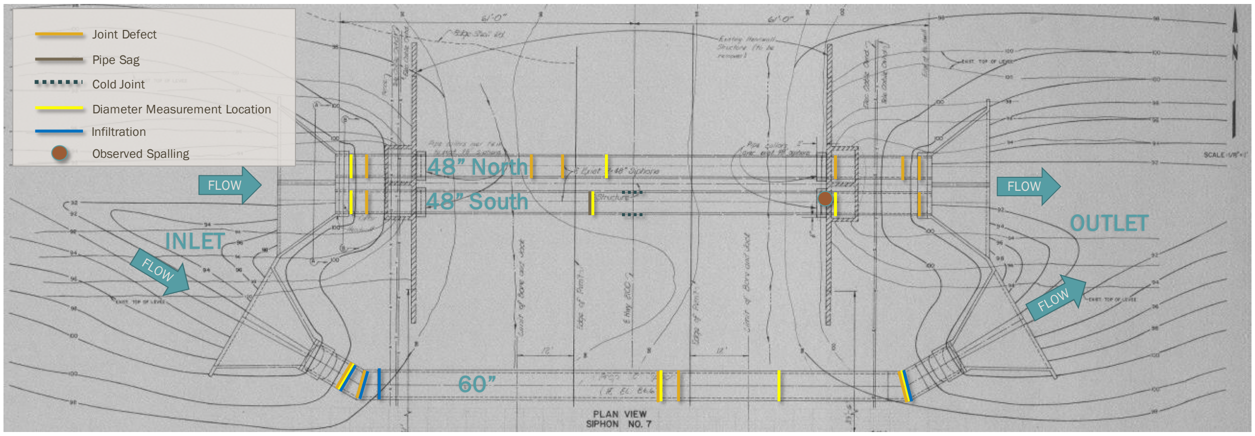


Figure 3-1. Siphon 7 Investigation Findings

3.1 60-Inch

The 60-inch pipe was observed to have an internal diameter of 60 inches (59 inches at 10 ft). The pipe wall was observed to have a VANDA concrete rating of level 1. The surface was generally smooth, was hard when subjected to penetration testing, and no observable voids were found during random concrete sounding. The surface had an observed pH of 6.

Photos 3-1 to 3-6 show observations within the 60-inch pipe. Water infiltration was observed at multiple joints at approximately 15 ft (runner), 20 ft (runner), 25 ft (dripper and runner), and 125 ft (dripper). At the first joint, a 3-inch wide gap was observed at the 5 o'clock position and a 7-inch wide gap was observed at the 7 o'clock position. The joint gap was varied between 2 to 3-inches in depth. A 3.5-inch wide gap with 2 to 3-inches of joint depth was observed at the 3rd joint (approximately 15 ft) from the 12 to 5 o'clock positions, near the change in direction (Photo 3-2). A similar 3-inch wide gap, and 1-inch joint depth was observed at the change in direction near the outlet (approximately 125 ft). Smaller gaps of 1-inch or less were observed at 20 ft and 80 ft. A 0.5-inch pipe joint offset was observed near 75 ft. Small areas of surface delamination were observed in various locations (Photo 3-6). Small debris (rocks and bottles) were observed at the inlet structure (Photo 3-1), invert of the pipe (Photo 3-3), and outlet structure.

Photo 3-1. 60-inch Inlet Structure

Photo 3-2. Pipe Joint Gap (15 ft)

Photo 3-3. Debris (Typ.)

Photo 3-4. 60-inch Pipe Condition (Typ.)

Photo 3-5. Surface Profile and Pipe Joint (Typ.)

Photo 3-6. Surface Delamination (Typ.)

Following completion of the condition assessment, SJRA installed a steel plate across the inlet structure to facilitate a flow test through the two 48-inch pipes (Photo 3-7).

Photo 3-7. Steel Plate Installation at 60-inch Inlet Structure (Perfomed by SJRA)

3.2 48-Inch South

The 48-inch South pipe was observed to have an internal diameter of 48 inches. The pipe wall was observed to have a VANDA concrete rating of level 1, with the exception of small section of spalling that was observed as a level 2. The surface was generally smooth, was hard when subjected to penetration testing, and no observable voids were found during random concrete sounding. The surface had an observed pH of 6.

Photos 3-8 to 3-15 show observations within the 48-inch South pipe. A possible sag in the pipe was observed near 50 ft (Photo 3-12), indicated by increased water depth at this location (9 inches). A an inconsistency in the pipe surface similar to diagonal cold joint was observed on each side of the pipe (3 and 9 o'clock) at approximately 60 ft (Photo 3-13), typical of cast-in-place construction. The surface inconsistency did not appear to be a crack, did not appear to have any obvious repair materials present, and appeared to be an inherent part of concrete matrix. The profile of the pipe itself was consistent on either side. Assuming the pipe is precast as reported, it is possible this surface inconsistency occurred at the manufacturing facility.

A 1.5-inch wide gap and 1" joint depth was observed at the 2nd joint (approximately 10 ft) with missing mortar (Photo 3-11). A similar 1-inch wide gap and 0.5" joint depth was observed at the last pipe joint between the 1 and 10 o'clock positions. A 4-inch by 8-inch area and 1-inch depth of surface spalling (VANDA level 2) was observed at approximately 100 ft at the 7 o'clock position (Photo 3-14 and Photo 3-15). At the deepest point, the defect was 1-inch deep relative to the adjacent pipe surface. Other smaller areas of surface delamination near pipe joints were also observed (Photo 3-10). Small debris (rocks, boards, and bottles) were observed at the inlet structure, invert of the pipe (Photo 3-9), and outlet structure.

Photo 3-8. 48-inch South Inlet Structure

Photo 3-9. Pipe Profile and Debris (Typ.)

Photo 3-10. Surface Delamination at Joint (Typ.)

Photo 3-11. 1.5-inch Joint Gap at 10 ft

Photo 3-12. Possible Pipe Sag

Photo 3-13. Possible Diagonal Cold Joint

Photo 3-14. Surface Spalling at 100 ft (A)

Photo 3-15. Surface Spalling at 100 ft (B)

3.3 48-Inch North

The 48-inch North pipe was observed to have an internal diameter of 48 inches. The pipe wall was observed to have a VANDA concrete rating of level 1. The surface was generally smooth, was hard when subjected to penetration testing, and no observable voids were found during random concrete sounding. The surface had an observed pH between 7 and 8.

Photos 3-16 to 3-20 show observations within the 48-inch North pipe. A ¾-inch wide gap with ¼-inch depth and 1-inch wide gap with 3.5-inch depth were observed at the 1st and last joints, respectively, with missing mortar. Joint gaps of 1-inch or less and missing mortar were also observed at 30, 40, 100, and 110 ft (Photo 3-18). Small debris (rocks, boards, and bottles) were observed at the inlet structure, invert of the pipe, and outlet structure (Photo 3-20). A piece of debris approximately 18-inches long located near the inlet structure appeared to be cast into the pipe invert and could not be removed (Photo 3-19).

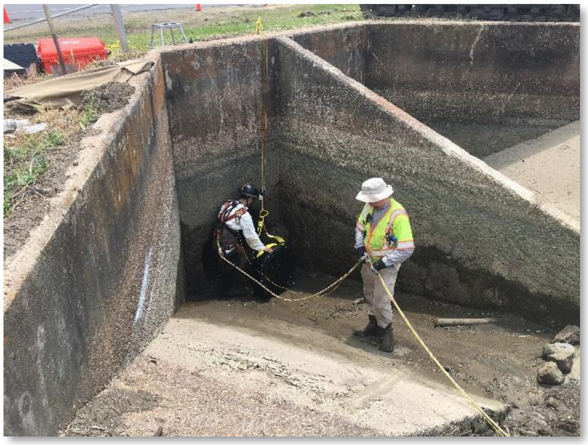


Photo 3-16. 48-inch North Inlet Structure

Photo 3-17. Pipe Surface (Typ.)

Photo 3-18. Small Joint Gap (Typ.)

Photo 3-19. Debris Cast in Pipe Invert

Photo 3-20. Removed Debris (Typ.)

4 Recommendations

The Siphon 7 pipes have an overall VANADA concrete condition rating of level 1. The observed defects do not appear to be impacting the functionality of the pipes. While some water is likely leaking out of the joint gaps, there were no observed areas of erosion occurring behind the joint and the canal system is earthen bank (i.e. water loss to surrounding soil is much higher along the entirety of the canal). Filling of joint gaps with a non-shrink grout could reduce the risk of future erosion. The joint gaps, location of observed spalling, and surface defect similar to a cold joint should be observed on a regular basis for any changes (a minimum of every 3 to 5 years).

Appendix A
Photo Log

60-Inch

IMG_3953. 60-inch Inlet Structure

IMG_3954. Pipe Inlet

IMG_3955. Pipe Inlet (Upstream View)

IMG_3956. Pipe Joint Missing Mortar @ 10ft

IMG_3957. 3.5-Inch Joint Gap at Inlet Change in Direction (A)

IMG_3958. 3.5-Inch Joint Gap at Inlet Change in Direction (B)

IMG_3959. 3.5-Inch Joint Gap at Inlet Change in Direction (C)

IMG_3960. 3.5-Inch Joint Gap at Inlet Change in Direction (D)

IMG_3961. 60-Inch Pipe and Debris

IMG_3962. Pipe Joint Missing Mortar @ 80 ft

IMG_3963. Debris

IMG_3964. 60-inch Pipe (A)

IMG_3966. Pipe Joint (Typ.)

IMG_3967. 60-Inch Pipe Wall

IMG_3970. 60-inch Pipe (B)

IMG_3971. 60-Inch Pipe Crown

IMG_3972. 60-inch Pipe (C)

IMG_3973. 60-inch Pipe Outlet

IMG_3974. Pipe Invert

IMG_3978. Surface Delamination (Typ.)

IMG_3979. 60-inch Pipe Outlet and 3.5-inch Joint Gap at Change in Direction

IMG_3980. Pipe Joint Missing Mortar at Last Joint

IMG_3981. Debris at 60-inch Pipe Outlet (A)

IMG_3982. Debris at 60-inch Pipe Outlet (A))

48-Inch South

IMG_4004. Debris at Pipe Inlet

IMG_4005. Delamination at Pipe Joint (Typ.)

IMG_4007. Joint Gap (Typ.)

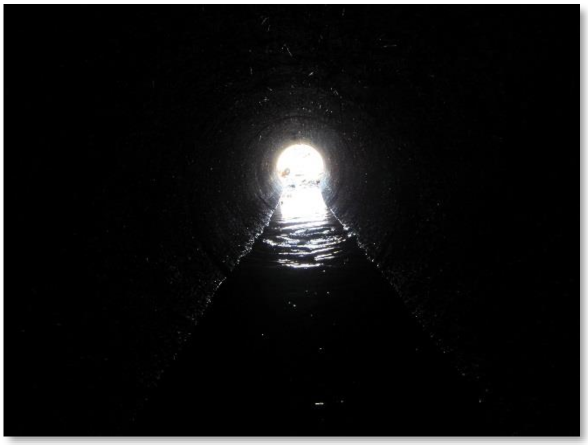
IMG_4009. Pipe Joint Missing Mortar @ 10ft

IMG_4011. 48-inch South Pipe (A)

IMG_4017. Pipe Invert @ 50 ft

IMG_4018. Pipe Crown

IMG_4019. Diagonal Cold Joint at 60 ft



IMG_4022. 48-inch South Pipe (B)

IMG_4024. Surface Spalling @ 100 ft (A)

IMG_4025. Surface Spalling @ 100 ft (A)

IMG_4026. Surface Spalling @ 100 ft (A)

IMG_4027. Surface Spalling @ 100 ft (A)

IMG_4028. Surface Spalling @ 100 ft (A)

IMG_4029. 48-inch South Pipe Wall (A)

IMG_4030. 48-Inch South Pipe Wall (B)

48-Inch North

IMG_4037. 48-inch North Pipe

IMG_4038. 48-inch North Pipe Wall (A)

IMG_4044. Joint Gap (Typ.)

IMG_4045. Joint Gap (Typ.)

IMG_4051. 48-inch North Pipe Wall (B)

IMG_4052. Pipe Crown (A)

IMG_4053. 48-inch North Pipe Surface

IMG_4054. Pipe Joint (Typ.)

IMG_4057. 48-inch North Pipe Wall (C)

IMG_4059. 48-inch North Pipe Wall (D)

IMG_4061. Pipe Crown (B)

IMG_4069. 48-inch North Pipe Looking Upstream

2018 FLOW TEST TECHNICAL MEMORANDUM BY TEXAS WATER ENGINEERING, PLLC

TECHNICAL MEMORANDUM

TO: David Parkhill, Matt Barrett, Kenneth Forrest, Daniel Hilderbrandt, Kimberly Wright

FROM: Victoria Foss

DATE: June 8, 2018

SUBJECT: Siphon 7 Flow Test Field Memorandum

THIS DOCUMENT IS RELEASED FOR THE PURPOSE
INFORMATION ONLY UNDER THE AUTHORITY OF
VICTORIA FOSS, P.E., TEXAS NO. 91952 ON 6/8/18.
IT IS NOT TO BE USED FOR CONSTRUCTION,
BIDDING OR PERMIT PURPOSES.
TEXAS WATER ENGINEERING, PLLC.
TEXAS REGISTERED ENGINEERING FIRM F- 8482

1.0 INTRODUCTION

On May 18, 2018, the San Jacinto River Authority (SJRA) with the assistance of Texas Water Engineering (TWE) and V&A Consultants performed a multi task field investigation in support of the upcoming Siphon 7 bypass project. The purpose of the investigation was to determine the condition of the 60-inch bypass and dual 48-inch siphon pipes and to install a steel plate to block off the 60-inch bypass in preparation of a field flow test. Prior to the field investigation, the SJRA developed an In-House work plan that outlined the procedures, participants, roles, responsibilities, safety procedures, and equipment to be used. The pumps at the LHPS were shut down on May 17th to allow the canal to dewater prior to the inspections that began at 8:00 AM on May 18th.

2.0 PIPE INSPECTIONS

At 6:00 AM on May 18th, the SJRA mobilized pumps to the downstream area of Siphon 7 to begin dewatering the pipes in preparation for the manned inspections (refer to photographs 1 and 2 of Appendix A). The pipe inspections were performed by V&A Consulting Engineers under Contract 18-0063, Work Order No. 1. Detailed results from the inspections are summarized under a separate report developed by V&A. The SJRA staff completed the dewatering of the 60-inch bypass around 8:00 AM and the V&A inspector entered the pipe at the upstream end around 9:35 AM. TWE was on-site during the inspection and was able to listen to the inspector (via radio communication) narrate what he observed. TWE was able to communicate with the inspector during the inspection to ask questions and obtain additional information as needed. The inspector noted the concrete to be in good overall condition. However, several joints were observed to have some separation. A few joints were also observed to be missing joint material and had water infiltration from the surrounding soils. There was no indication of large debris present in the bypass. Once the inspector exited the pipe at the downstream end, the SJRA staff began the installation of the steel plate to temporarily block off the bypass for the flow test. While the SJRA was installing the steel plate at the bypass, the V&A inspector entered the southern-most 48-inch pipe for inspection. Victoria Foss with TWE was observing the steel plate installation and Abby Crockett with TWE was listening to and communicating with the V&A inspector in the 48-inch pipes. The two 48-inch pipes were noted to be in good overall condition. Some joint separation and missing joint material was noted in both 48-inch pipes. The southern 48-inch pipe was observed to have standing water and a sag in the pipe approximately 50 feet from the upstream end. The southern pipe was also observed to have some delamination of the pipe 100 feet from the upstream end. There was no indication of large debris present in the pipes.

3.0 STEEL PLATE INSTALLATION

Prior to May 18th, the SJRA fabricated a steel plate to be installed across the intake structure of the Siphon 7 60-inch bypass pipe. Photograph 4 of Appendix A shows a picture of the fabricated steel plate. The SJRA used the Gradall to remove earthen material approximately 6 to 8-inches below the bottom slab of the intake structure and approximately 3 to 4 feet into the south levee to accommodate the plate (see photograph 3 of Appendix A). Once the earth material was removed, the Gradall was used to lift the steel plate into position in front of the headwall (see photographs 5 through 7 of Appendix A). Once the plate was in place, the staff used the Gradall to install and compact backfill material along the sides and bottom of the plate (see photographs 8 through 10 of Appendix A) with the intent of providing a watertight installation. Once V&A completed the inspection of both 48-inch pipes, the SJRA performed final grading upstream of Siphon 7 prior to placing the canal back in service.

The normal operating water surface elevation (WSEL) of the canal was marked on the north side of the intake structure for the 48-inch pipes at Siphon 7 with white spray paint. It was noted that the normal WSEL range is 48 to 40 inches below the top of the intake structure wall (see photograph 11 of Appendix A). These marks were used as reference points during the flow test.

4.0 FLOW TEST

The purpose of the flow test was to determine the flow rate from the Lake Houston Pump Station (LHPS) that will produce a normal WSEL upstream of Siphon 7 with the 60-inch bypass blocked. This information will be used to determine the range of flows that the SJRA would be comfortable allowing the Siphon 7 bypass contractor to pass through the two existing 48-inch pipes during construction. The intent of maintaining the normal water surface elevation is to avoid introducing the 48-inch siphon pipes to additional pressure during construction.

The pipe inspections and steel plate installation were complete by 3:00 PM on May 18th and the pumps at the LHPS were turned on to place the canal back in service. The pumps were initially set to pump 65 MGD in an effort to quickly fill the main canal, and later that evening the pumps were set to pump approximately 45 MGD and the system was left to operate at that rate over the weekend in preparation for the data collection that began on Monday May 21st. WSELs were recorded upstream and downstream of Siphon 7, Siphon 11, and Siphon 23; measurements were taken using a tape measure from the top of the headwall down to the water surface. Figures 1 through 3 show the measurement locations at each siphon. The WSEL measurements are recorded in Appendix B. Two measurements were taken at each location along with the date, time of reading and current pumping rate at the LHPS. Once the WSEL measurements were observed to be consistent at a given flow rate, the pumping rates were increased, and the system allowed to stabilize. Stabilization of the system was defined as achieving the same WSEL readings for a given flow rate for at least two consecutive readings, which was typically observed to take approximately 24 hours. Once the canal system appeared to stabilize, measurements were taken for the increased flow rate. This process was repeated until the normal target water surface elevation (as marked on the intake structure of Siphon 7) was reached.

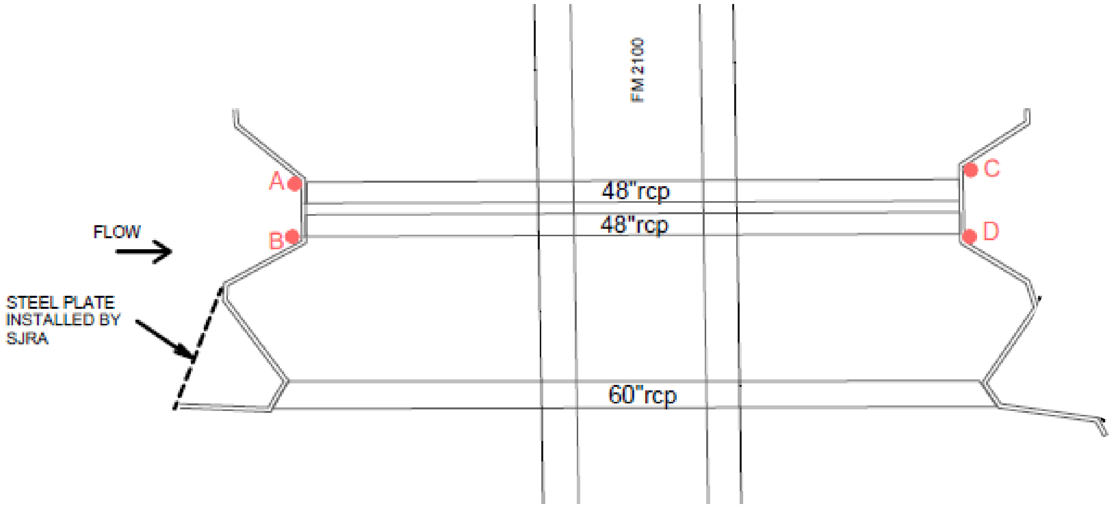


Figure 1 - Water Surface Measurement Locations – Siphon 7

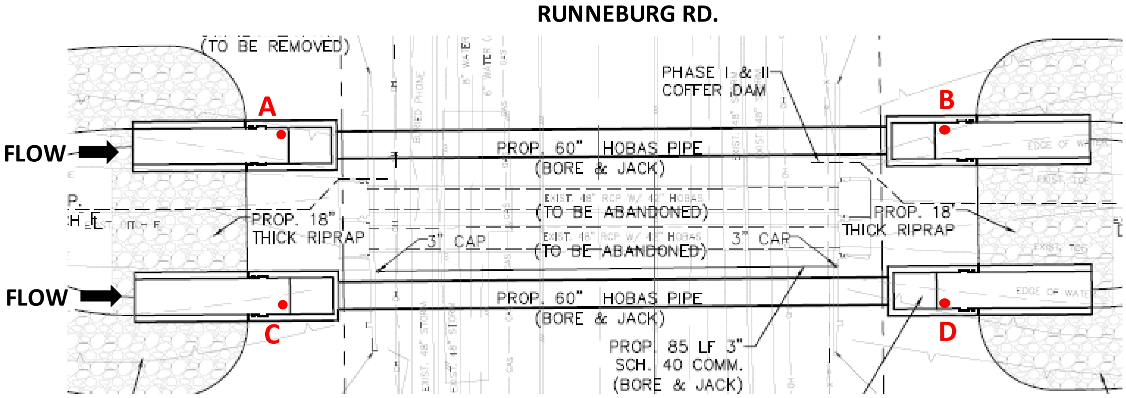


Figure 2 - Water Surface Measurement Locations – Siphon 11

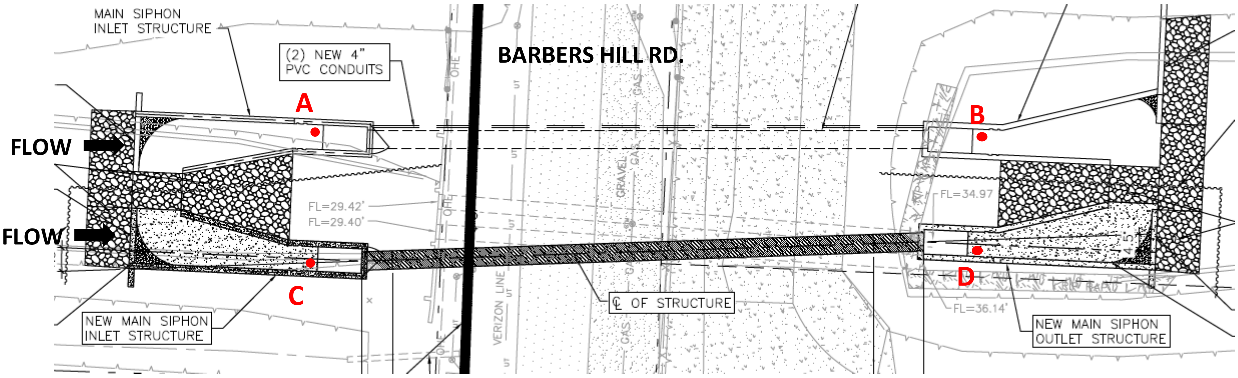


Figure 3 - Water Surface Measurement Locations – Siphon 23

5.0 SUMMARY

Data was collected during the flow test from May 21st to June 7th and recorded in an Excel spreadsheet. The raw data collection spreadsheet is shown in Appendix B of this memorandum. It was observed during the testing period, that the WSELs at the collection points took about 24 hours to stabilize and reach steady state. It was also observed that only at flows higher than 60 MGD during the testing period did the canal WSEL upstream of Siphon 7 reach higher than the target normal operating range. Figure 4 below provides a graphical representation of the flow test results.

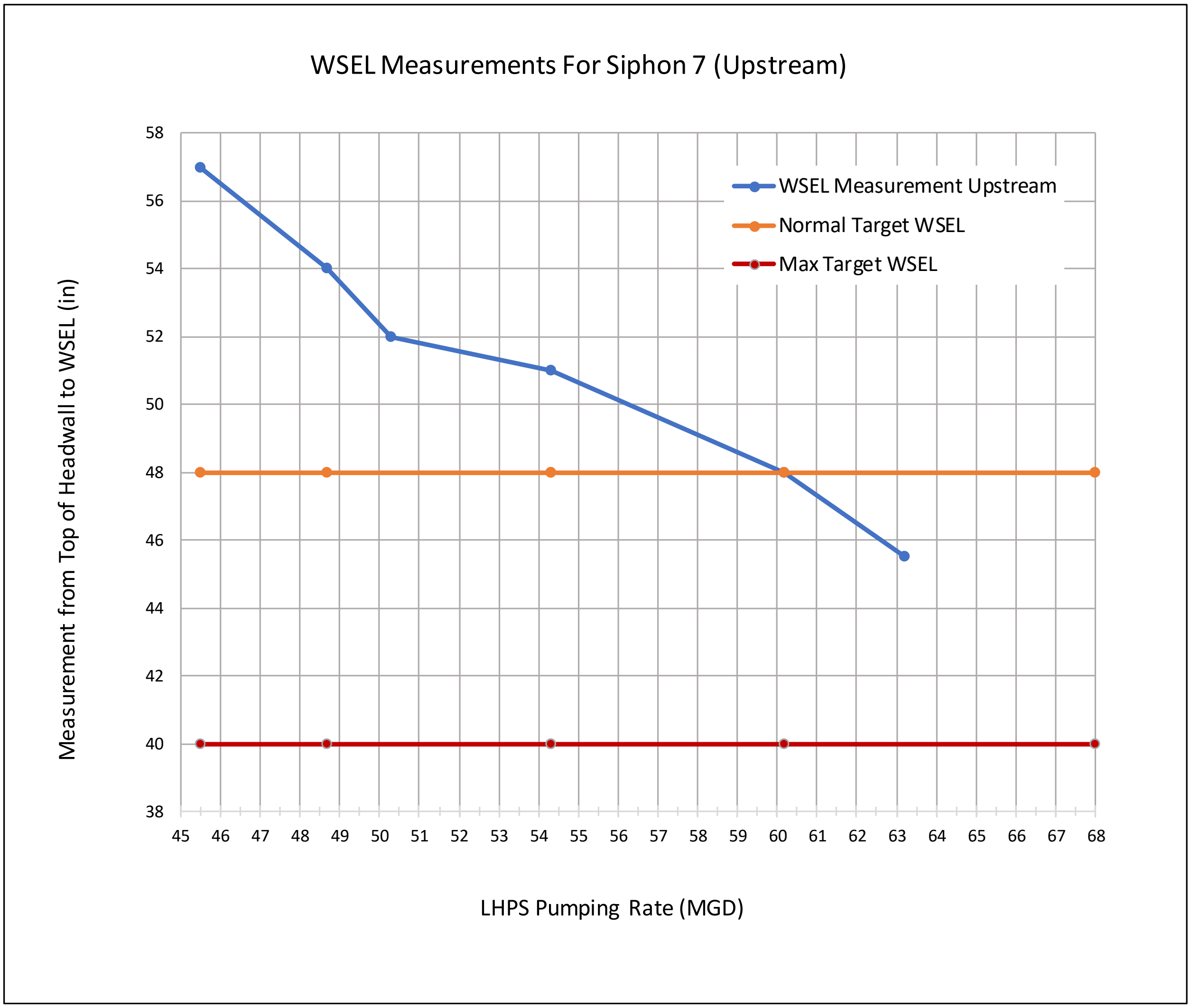


Figure 4 – Results from Flow Test

Appendix A

Photographs

1 Dewatering pump suction line installation.

2 Cofferdam and pump suction at 60-inch downstream structure (looking upstream).

3

Grading for plate installation at the upstream of 60-in bypass.
(looking downstream/southeast)

4

Mobilization of steel plate.

5

Steel plate installation at 60-inch bypass intake structure.
(looking downstream/southeast)

6

Steel plate installation at 60-inch bypass intake structure.
(looking downstream/southeast)

7

Steel plate installation at 60-inch bypass intake structure.
(looking downstream/southeast)

8

Backfilling behind steel plate.
(looking downstream/southeast)


9

Backfill and re-grading behind steel plate.
(looking downstream/southeast)


10

Final installation of steel plate at 60-inch bypass intake structure.
(looking upstream)

11

Target water surface elevations for flow test.

Appendix B

Flow Testing Measurements

Flow Testing for Main Canal - Data Collection Sheet

Location		Date Time LHPS	Date Time LHPS	Date Time LHPS	Date Time LHPS	Date Time LHPS	Date Time LHPS	Date Time LHPS	Date Time LHPS	Date Time LHPS	Date Time LHPS	Date Time LHPS	Date Time LHPS	Date Time LHPS
		5/21/2018 7:00 AM 46.3 MGD	5/21/2018 2:15 PM 46.3 MGD	5/22/2018 7:00 AM 45.5 MGD	5/22/2018 2:45 PM 45.5 MGD	5/23/2018 7:30 AM 48.7 MGD	5/23/2018 2:45 PM 48.7 MGD	5/24/2018 7:30 AM 50.3 MGD	5/24/2018 2:45 PM 50.3 MGD	5/25/2018 7:00 AM 54.6 MGD	5/29/2018 6:15 AM 54.3 MGD	5/30/2018 7:00 AM 54.3 MGD	5/31/2018 7:00 AM 60.34 MGD	6/7/2018 7:00 AM 63.2 MGD
Siphon 7	A	56.5	57	57	57	54	No data collected	54	52	51	51	51	48	45.5
	B	56.5	57	57	57	54		54	52	51	51	51	48	45.5
	C	60	61	60	60	58		58	56	56	56	56	54	50.5
	D	60	61	60	60	58		58	56	56	56	56	54	50.5
Siphon 11	A	No data collected	78	77	78	74	No data collected	75	75	73	71	71	68	66
	C		77	77	78	74		75	75	73	72	71	68	66
	B		74.5	76	76	72		72	73	70	69	70	69	64
	D		75.75	75	77	73		73.5	73	71	70	70	69	64
Siphon 23	A	79	82.8	83	83	84	No data collected	85	85	83.5	84	85	85	83
	C	79	84.5	84	84	85		85	85	83.5	84	85	85	83
	B	81.5	82	83	83	83		87	87	85.5	86	87	87	85
	D	81.5	84.5	84	84	85		87	87	85.5	86	87	87	85

NOTES:

🟦 Upstream measurements 🟧 Downstream measurements

5/21/18 - the gate going into the south reservoir was set at 30-inches. The gate at the bypass canal was set at 15-inches.

5/23/18 - the gate going into the south reservoir was set at 45-inches. The gate at the bypass canal was set at 14-inches.

5/24/18 - the gate going into the south reservoir was set at 60-inches. The gate at the bypass canal was set at 11-inches.

5/25/18 - received 1/10 -inches of rain, closed the gate at the bypass to 8-inches and kept the gate into the south reservoir at 60-inches.

6/7/18 - The pumps were turned up to 63 MGD on 6/4/18 and the canal was allowed to stabilized before taking the readings on the June 7th.

EXCERPT FROM 2008 DIVE INSPECTION REPORT BY INTERNATIONAL DIVING SERVICES, LLC

Inspection #3—Site #7 Hwy 2100:

Two pipes, side by side, measuring 42 inches in diameter and 100 feet in length.

Pipe#1-Siphon=100 feet:
This pipe is fitted with five joints. Heavy sediment and debris was found at the Entry Point of this penetration. Rock and 4X6 pieces of wood filled half of the pipe.
At first joint, a gap of ½-inch was noted.
At second joint, the sedimentation dissipated and erosion was found on the bottom portion in this area.
On the right side of this joint, approximately half way up, heavy erosion, cracking, large rocks, and concrete loss were reported. Cracking and worn concrete due to flow was documented as well.
At third joint, heavy erosion and large rocks were found. Large gaps and possible separation were noted. Half of pipe was cracked and exhibited concrete loss and heavy corrosion.
At the fourth joint, the bottom portion maintained a tight seat but the upper portion had a ½-inch gap. Erosion was noted on the bottom of pipe and the concrete was extremely brittle.
At the fifth joint, heavy erosion, large rocks, and debris were found on the bottom portion. The upper portion maintained a tight seat.

Pipe #2-Siphon=100 feet:
Heavy debris was found at the Entry Point of this penetration. The pipe was 1/3 full of this debris.
At the first joint, heavy erosion and consistent gapping of ½-inch was noted. Concrete was very brittle in this area and sedimentation and brush were filling pipe approximately half.
At 8 feet, concrete lifted approximately 5 inches.
At second joint, minor gapping and erosion were noted. Eight feet from this joint, a 1-foot area exhibited heavy erosion, concrete loss, and marine growth.
At third joint, a ½-inch gap and erosion were reported. The concrete was fair in this area.
At fourth joint, erosion, severe cracking, and concrete loss were documented. ½-inch to ¾-inch gap was noted. Sedimentation, sand, and debris consisting of large rocks were found.
At fifth joint, minor erosion was noted. A tight seat was maintained on the upper portion and no debris was found. A crack was found on the lower portion of this pipe.
At sixth joint, erosion, sedimentation, brush, and large rocks or bricks were found. No other abnormalities were noted.
At seventh joint, minor erosion, sedimentation, bricks, rocks, and chunks of asphalt were found. No other abnormalities were noted.
At eighth joint, heavy corrosion, major debris filled 1/3 of the pipe.
A ½-inch gap that extends to the end of pipe was reported.
IDS recommend a cleaning of this pipe.

RFCSP No. 18-0036 - Siphon 7 Improvements - Phase 1

PROPOSAL FORM: OPTION NO. 4: EXTENSION AND GEOPLOYMER LINING OF EXISTING 60-INCH RCP, NEW 60-INCH STEEL PIPE ON NORTHERN SIDE OF EASEMENT

The respondent shall complete the following Proposal Form template, which directly corresponds to the project specifications. The contractor shall not make changes to the format of this file.

	Offeror's Name:	

A4. Proposal Option No. 4

Item No.	Qty.	Unit	Spec. Reference	Description	Unit Price (this column controls)	Proposal Price
1	1	LS	01 71 13	Mobilization: 5% (Maximum) of total proposal price. See Specification Section 01 71 13 – Mobilization for measurement and payment.	$0.00	$0.00
2	1	LS	01 55 26 01 57 13.02 10 14 53	Installation of stabilized construction access and traffic control as shown on Drawings, complete in place and maintained during entire project.	$0.00	$0.00
3	1	LS	01 57 23	Installation of silt fence (filter fabric fence), as shown on the Drawings, complete in place, maintained during entire project, and removed at final completion of project.	$0.00	$0.00
4	1	LS	01 56 39	Installation of tree protection, as shown on the Drawings, complete in place, maintained during entire project, and removed at final completion of project.	$0.00	$0.00
5	1	LS	31 41 00	Trench Safety	$0.00	$0.00

6	1	LS	Division 31	Grade canal and project site as shown on the Drawings and compact all fill areas to 95% standard proctor density in applicable specified lifts. (Includes the installation of crushed concrete base material to show future cofferdam limits, and import of select fill material, if necessary).	$0.00	$0.00
7	1	LS	02 41 13.13	Removal of existing reinforced concrete headwall structures and associated reinforced concrete pipe as shown on Drawings.	$0.00	$0.00
8	1	LS	01 74 23 32 92 13	Hydromulch, seeding, and restoration of all disturbed areas.	$0.00	$0.00
9	1	LS	Division 31 33 11 16	Installation of Geopolymer lining of existing 60-inch reinforced concrete bypass pipe, and pipe extensions, including all fittings, gaskets, etc. as shown on Drawings. Complete in place.	$0.00	$0.00
10	1	LS	Division 31 33 05 23.19 33 11 13.02	Installation of **Northern** 60-inch nominal diameter, steel siphon pipe as shown on Drawings, including but not limited to jacking, excavation, shoring, backfill, fittings, post-installation inspection, and all other incidentals, complete in place.	$0.00	$0.00
11	1	LS	31 37 01	Installation of 18-inch thick broken concrete riprap (as specified) as shown in the Drawings, including geotextile, placement of material, and any backfill necessary, complete in place.	$0.00	$0.00

12	1	LS	01 57 23.02	Care of Water, including but not limited to control of ground, surface, and canal water or any other water encountered throughout the contract duration, as detailed in Specification Section 01 57 23.02 – Control of Ground Water and Surface Water and all applicable notes on Drawings, complete in place. **The minimum cost for this item shall be equal or greater than $250,000.**	$0.00	$0.00
<u>A4. Total for Proposal Option No. 4:</u>					**$0.00**	

B. EXTRA UNIT PRICE ITEMS (NOT USED)	
<u>B. Total Extra Unit Price Items:</u>	**$0.00**

C. CASH ALLOWANCES (NOT USED)	
<u>C. Cash Allowances:</u>	**$0.00**

D. ALTERNATE ITEMS (NOT USED)	
<u>D. Total Alternate Items:</u>	**$0.00**