CITY OF SAN ANTONIO
TRANSPORTATION & CAPITAL IMPROVEMENTS
Seeling Channel Phase II Drainage #40-00427

3/20/15

STATE OF TEXAS
MEHMET A. BOZ
102570
LICENSED
PROFESSIONAL ENGINEER

AECOM TBPE #3580

ADDENDUM NO.1

March 20, 2015

TO BIDDER OF RECORD:

The following changes, additions, and/or deletions are hereby made a part of the Contract Documents for the Seeling Channel Phase II Drainage #40-00427 project for the City of San Antonio, Transportation & Capital Improvements, San Antonio, Texas, dated March 2015, as fully and completely as if the same were full set forth therein.

GENERAL

The pre-bid meeting held on Wednesday, March 18, 2015 at 9:00 a.m. was non-mandatory. The list of attendees is attached, and is also posted on the City of San Antonio Bidding & Contracting Opportunities website.

Appended hereto and part of Addendum No. 1 are:

1. Contractor Questions and Clarifications from Pre-Bid Meeting and via E-Mail.
2. Pre-Bid Meeting Sign-In Sheet.

BIDDING AND CONTRACT REQUIREMENTS

Appended hereto and part of Addendum No. 1 is:

1. San Antonio Water System 095 Form - Waterworks and Sanitary Sewer Construction Special Conditions (January 2015). This form shall replace the 095 Form that is included in the bid package.

TECHNICAL SPECIFICATIONS

Not applicable.

DRAWINGS:

Appended hereto and part of Addendum No. 1 is:

1. Revisions to Sheets 131, 132, 133, and 135. Tree #71 is no longer marked to be counted towards the overall removed tree count.

OTHER:

Appended hereto and part of Addendum No. 1 is:

1. Geotechnical Report by Terrance Ian Perez, PE (Raba-Kistner Consultants, Inc.).

CONTRACTOR QUESTIONS AND CLARIFICATIONS FROM PRE-BID MEETING AND VIA EMAIL

1. **Question:** Will the overhead electrical work near the Morning Glory bridge be completed prior to construction, and will the new poles be located on the opposite side of the street?

   Response: The overhead electrical relocations and field adjustments are scheduled to be completed in April 2015. Existing electrical poles in the vicinity of the Morning Glory bridge will be relocated to the opposite side of the street.

2. **Question:** We were reviewing the plans for the above referenced project and noticed that you have called out for rubber gasket joints on the reinforced concrete pipe and box culverts. The City of San Antonio specifications call out for tongue and groove joints with ram nek sealant. This has been the standard for years and there have been no issues that we are aware of. Was there any reason why you were asking for rubber gaskets on this project? We just wanted some clarification so we know how to proceed (via e-mail on 3/19/2015).

   Response: Please bid the project materials as advertised. Considerations for alternative materials can be made during construction.

PRE-BID MEETING SIGN-IN SHEET (1 of 2)

Pre-Bid Meeting Sign-In Sheet **March 18, 2015**

CITY OF SAN ANTONIO
TRANSPORTATION & CAPITAL IMPROVEMENTS

Seeling Channel Phase II Drainage #40-00427

NAME	ORGANIZATION	PHONE	EMAIL
David Rios	**TCI – Contract Services**	**(210) 207-1339**	**david.rios@sanantonio.gov**
Manuel Garcia	**TCI – Contract Services**	**(210) 207-7872**	**manuel.g.garcia@sanantonio.gov**
Tyrel Eckroth	Quest Civil Constructors	623-581-9700	tae@qcsw.com
Fernando Botello	Lane Construction	210 383 2025	fbotello@laneconstruct.com
Ralph Monteros	Lane Construction	210-383-5755	Rmonterosa@laneconstruct.com
Ryan Detru	E-Z Bel Const.	210 736.6595	RyanDetru@EzBel.com
Layne Walker	Capital Excavation	512 440-1717	Lwalker@capitalexcavation.com
Mehmet Boz	AECOM	210.253.7513	mehmet.boz@aecom.com
Brittany Meloni	AECOM	210.296.2152	
Jim O'Reilly	Texas Sterling	210-952-6745	jim.oreilly@strlco.com
Butch Leist	Hill Country Bridge	210 913 7155	bleist@hillcountrybridge.com
Ursula L Villarreal	Tropical Contracting	210 705 1008	uvillarreal@tropicalcontracting.net
Charlie Winters	Tropical Contracting	210 845 4836	cwinters@tropicalcontracting.net
Gerardo Gomez	SAWS	210-233-3469	gxgomez@saws.org
Alberto Olivarez	SAWS	210.233.3474	albert.olivarez@saws.org
John Offer	CPS Energy	210 353 2012	jcoffer@cpsenergy.com

PRE-BID MEETING SIGN-IN SHEET (2 of 2)

Pre-Bid Meeting Sign-In Sheet **March 18, 2015**

CITY OF SAN ANTONIO
TRANSPORTATION & CAPITAL IMPROVEMENTS

Seeling Channel Phase II Drainage #40-00427

NAME	ORGANIZATION	PHONE	EMAIL
Lisa Torres	TCI-Horiz.	207-2109	lisa.torres@sanantonio.gov
DARYL SPILLMANN	SAWS	233-3012	dspillmann@saws.org
PAT DEBNER	J3 COMPANY	(830) 995-5700	INFO@J3Co.Com
ABEL RAMIREZ	SAWS	(210) 233-3254	abramirez@saws.org
Peter Rodriguez	COSA TCI	207-8154	peter.rodriguez@sanantonio.gov
Theresa Larson	COSA TCI	207-1446	theresa.larson@sanantonio.gov
Lucy Barbosa	COSA EDD	2073910	Lucy.Barbosa@sanantonio.gov
Elizabeth Vidal	TCI	207-2072	Elizabeth.vidal@sanantonio.gov
John Wolter	TCI	207-6084	john.wolter@sanantonio.gov
Tom Sirk	Oscar Renda Cont	817-491-2703	tsirk@oscarrenda.com

SAN ANTONIO WATER SYSTEM
WATERWORKS AND SANITARY SEWER CONSTRUCTION
SPECIAL CONDITIONS

The following changes are made to the Contract Documents:

1. Add to the Contract Definitions

   San Antonio Water System: San Antonio Water System Board of Trustees.

2. Add to the Invitation for Bid

   The San Antonio Water System area of construction operations is coincident with the area of construction operations specified in the contract documents for the project. All water and sewer facility adjustment and attendant work as shown on the Plans is considered to be an integral part of the project, and the Contractor shall be responsible for the timely scheduling and accomplishment of all water and sewer main and attendant work in conjunction with the work outlined in paragraph 1 of the City of San Antonio Invitation for Bid.

3. Add to the General Conditions

   All resident inspection of water and sewer facility adjustment and attendant work will be performed by an authorized representative of the San Antonio Water System who will in turn be responsible directly to the inspectors designated above.

4. Add to the General Conditions Paragraph

   Materials for Water and Sanitary Sewer Main Replacement and adjustments: The Contractor shall also furnish all materials required for the installation of all water and sanitary main replacement and adjustments, service lines, sanitary sewer laterals, manholes and attendant work as shown on the drawings and in accordance with the San Antonio Water System Material Specifications.

5. Add to General Conditions

   Water Mains: The Contractor shall be responsible for the establishment in the field of all lines and grades for water works construction utilizing as may be appropriate the survey base control data provided by the Engineer for the work indicated in Paragraph 1 of the City of San Antonio Invitation for Bid. All construction staking, additional survey, layout and measurement work shall also be performed by the Contractor as part of his work.

6. Add to the General Conditions

   Warranty/Correction Period for Water and Sewer Works: During a period of 24 months from and after the date of the final acceptance by the San Antonio Water System of the water and waste water work completed by and through this contract, the Contractor shall make all needed repairs arising out of defective workmanship or materials, or both, which in the judgment of the San Antonio Water System shall become necessary during such period. If within 3 days after the receipt of a notice in writing to the Contractor or his agent, the Contractor shall neglect to make or to undertake with due diligence the aforesaid repairs, the San Antonio Water System is hereby authorized to make such repairs at the Contractor's expense. In case of an emergency where, in the judgment of the San Antonio Water System delay would cause a serious loss or damage, repairs may be made with notice being sent to the Contractor, and the Contractor shall pay the cost thereof.

7. Add to these Contract Documents, the Standard Specifications for Water and Sanitary Sewer Construction, available to the Contractor at the San Antonio Water System or at www.saws.org.
   a. Add the following paragraph to **SAWS Item No. 100 – Mobilization**, to the end of Section 100.1 DESCRIPTION:

      The combined total bids for SAWS Mobilization, Item No. 100 and SAWS Preparing Right-Of-Way, Item No. 101 shall not exceed 15% of the SAWS base bid. A SAWS base bid shall be defined as all SAWS bid items excluding Mobilization, Item No. 100 and Preparing Right-Of-Way, Item No. 101.

   b. Add the following paragraph to **SAWS Item No. 101 – Preparation of Right-of-Way**, to the end of Section 101.1 DESCRIPTION:

      The combined total bids for SAWS Mobilization, Item No. 100 and SAWS Preparing Right-Of-Way, Item No. 101 shall not exceed 15% of the SAWS base bid. A SAWS base bid shall be defined as all SAWS bid items excluding Mobilization, Item No. 100 and Preparing Right-Of-Way, Item No. 101.

8. Add to these Contract Documents, the San Antonio Water System Special Provisions, attached separately.

9. Add to these Contract Documents, the San Antonio Water System Proposals, attached separately.

10. Add to the General Conditions for Article 7 - Changes in Work for San Antonio Water Systems work that is joint bid the COSA the following will apply

    Change Orders allowable markups for SAWS work is as follows:

ACTUAL COST OF THE WORK – Actual Cost incurred by the Contractor to perform the additional Work. Contractor shall provide a complete breakdown of the actual costs to the Owner on a daily basis as follows:

Labor including Foremen

Materials comprising the Work.

The Contractor's actual incremental ownership or rental cost of equipment during the time of use on the extra Work. (Rental cost may be based on current Southwest Regional AGC, Association of Equipment Distributors regional computations or equivalent)

Power and consumable supplies for the operation of power equipment.

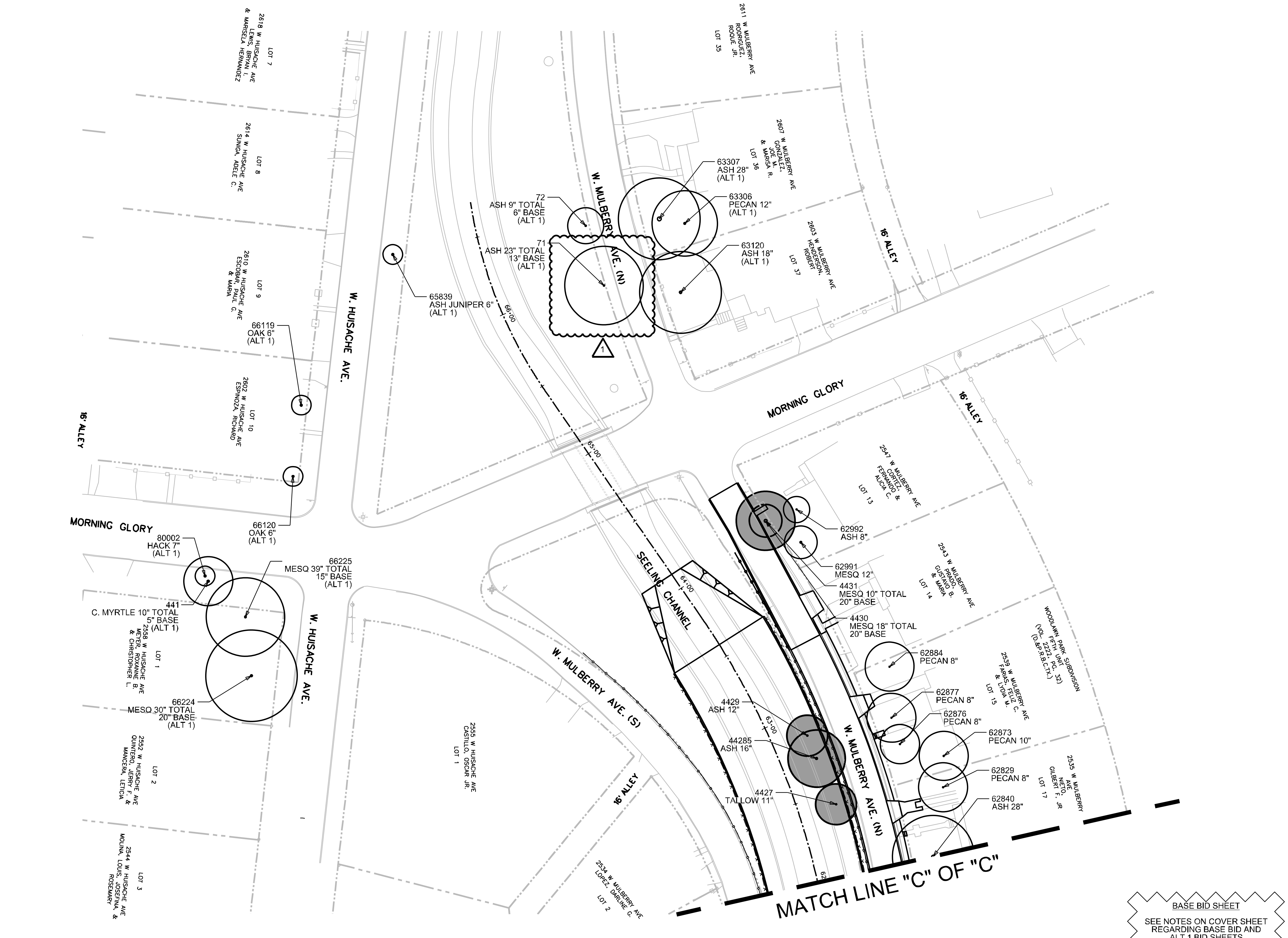
Insurance, any extra bond premiums, Social Security and unemployment contributions, and benefits.

PARTICIPATION ALLOWANCE

Participant	Overhead	Profit	Commission
To Contractor on his Project on Work performed by other than his own forces:	0%	0%	5%
To first tier Subcontractor on Work performed by his subtier Subcontractors:	0%	0%	5%
To Contractor and/or the first tier Subcontractors for that portion of the Work performed with their own respective forces:	10%	10%	0%

Not more than four categories of percentages, not to exceed the maximum percentages shown above, will be allowed regardless of the number of subtier subcontractors: For proposals covering both increases and decreases in the amount of the Contract, the application of overhead and profit percentages shall be on the net increase in Actual for the Contractor or Subcontractor performing the Work. However, where the Contractor or first tier Subcontractor receives proposals for additive and deductive amounts from separate subtier subcontractors, the commission shall be allowed on the added amounts prior to subtraction of the credit amounts. The cost of such extra Work shall be added to the Contract Sum by a Written Change Order

The remaining Article 7 remains as per the COSA General Conditions.



BASE BID SHEET

SEE NOTES ON COVER SHEET REGARDING BASE BID AND ALT 1 BID SHEETS

STATE OF TEXAS
MEHMET A. BOZ
102570
LICENSED PROFESSIONAL ENGINEER

March 19, 2015

NO	DATE	DESCRIPTION	DWG	CHK
1	3/19/2015	Addendum 1, Tree 71 Preserved		

REVISIONS

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TBPE REG. NO. F-3580

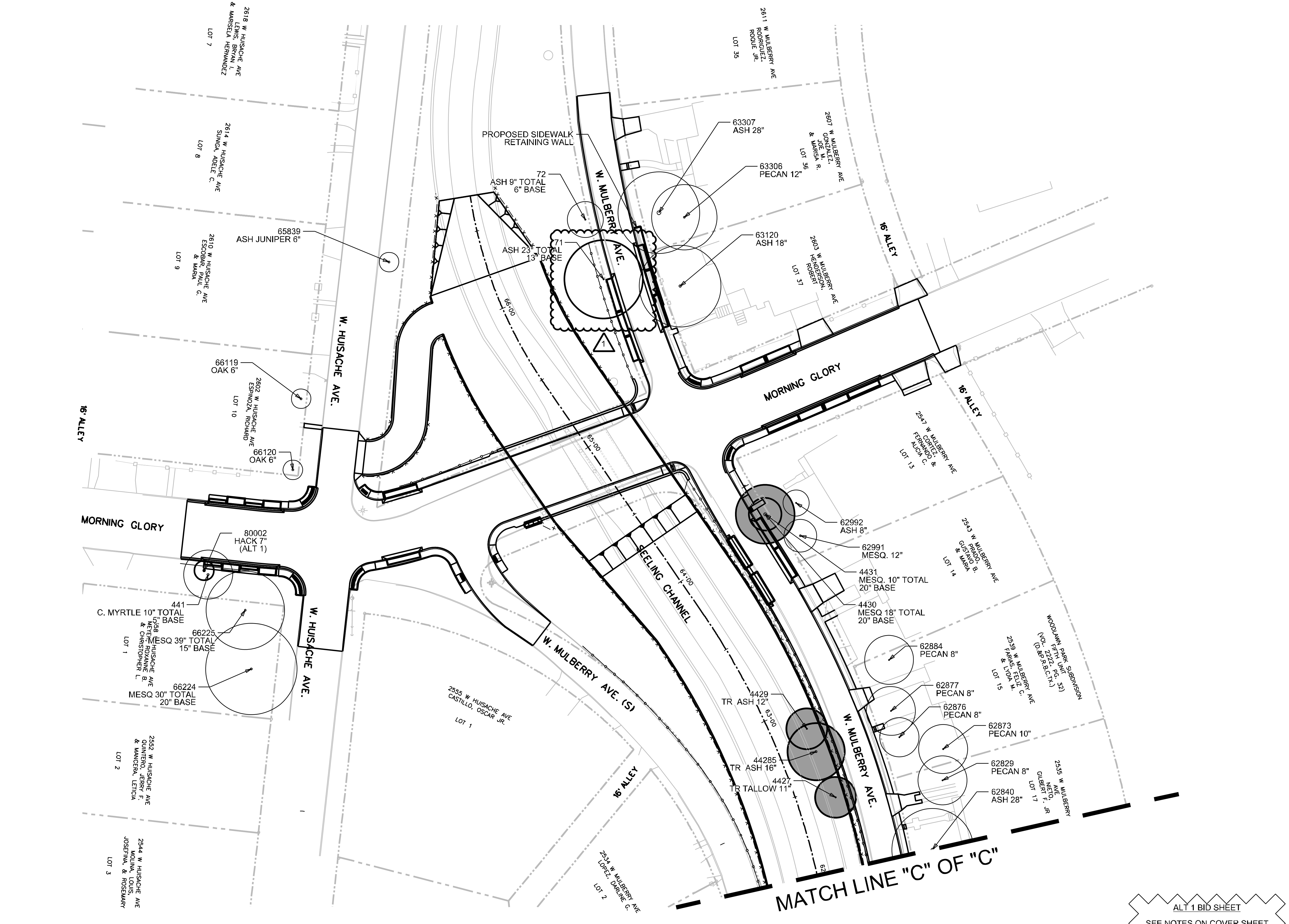
CITY OF SAN ANTONIO
TRANSPORTATION & CAPITAL IMPROVEMENTS

SEELING CHANNEL IMPROVEMENTS PHASE 2 (BASE BID)

TREE PRESERVATION PLAN IV

4 (BASE) OF 4

100% SUBMITTAL	PROJECT NO.: 60312595	DATE: MARCH, 2015	
DRWN. BY: CP	DSGN. BY: BEH	CHKD. BY: EKC	SHEET NO. 131



STATE OF TEXAS
MEHMET A. BOZ
102570
LICENSED
PROFESSIONAL ENGINEER

March 19, 2015

NO	DATE	DESCRIPTION	DWG	CHK
1	3/19/2015	Addendum 1, Tree 71 Preserved		
		REVISIONS		

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ALT 1 BID SHEET
SEE NOTES ON COVER SHEET REGARDING BASE BID AND ALT 1 BID SHEETS

CITY OF SAN ANTONIO
TRANSPORTATION & CAPITAL IMPROVEMENTS
SEELING CHANNEL IMPROVEMENTS PHASE 2 (ALT 1 BID)

TREE PRESERVATION PLAN V
4 (ALT 1) OF 4

100% SUBMITTAL	PROJECT NO.: 60312595	DATE: MARCH, 2015	
DRWN. BY: CP	DSGN. BY: BEH	CHKD. BY: EKC	SHEET NO. 132

TREE INVENTORY SUMMARY TABLE

Tree No.	DBH	Species	Significant Small species		Significant Tree		Significant Tree		Significant Tree		Heritage Small Species		Heritage Large Species (1:1)		Heritage Large Species (3:1)		Additional Inches	Location (Floodplain, Floodplain Buffer, or Uplands/Non-Floodplain)	Comments
			One Trunk 5.0 inches - 11.5 inches (Persimmon, Redbud, Mountain Laurel, Condalia, Possumhaw**, and Crabapple**)		6.0 inches - 23.5 inches		10.0 inches - 23.5 inches (Huisache, Ash Juniper, and Mesquite)		One Trunk 10.0 inches - 23.5 inches (Arizona Ash, Hackberry)		*One Trunk 12 inches or greater (Persimmon, Redbud, Mountain Laurel, Condalia, Possumhaw**, and Crabapple**)		24 inches or greater Ashe Juniper, Arizona Ash, Hackberry, Mesquite, Huisache		24 inches or greater		Preserved for Mitigation		
			Removed	Preserved	Removed	Preserved	Removed	Preserved	Removed	Preserved	Removed	Preserved	Removed	Preserved	Removed	Preserved	Preserved		
30	9	OAK				9												Floodplain	
31	11	OAK				11												Floodplain	
71	23	ASH								23								Floodplain	
72	9	ASH***																Floodplain	
441	16	CRAPE MYRTLE MULTI-TRUNK																Floodplain	NO TRUNK GREATER THAN 6"
4403	16	CRAPE MYRTLE MULTI-TRUNK																Floodplain	NO TRUNK GREATER THAN 6"
4404	19	VITEX MULTI-TRUNK																Floodplain	
4405	10	CRAPE MYRTLE MULTI-TRUNK																Floodplain	NO TRUNK GREATER THAN 6"
4407	8	PECAN			8													Floodplain	
4408	6	PECAN			6													Floodplain	
4409	6	PECAN			6													Floodplain	
4412	16	LIGUSTRUM BUSH																Floodplain	INVASIVE
4413	16	LIGUSTRUM BUSH																Floodplain	INVASIVE
4414	16	LIGUSTRUM BUSH																Floodplain	INVASIVE
4415	10	TALLOW																Floodplain	INVASIVE
4417	18	CRAPE MYRTLE CLUSTER			18													Floodplain	
4419	18	CHINABERRY																Floodplain	INVASIVE
4420	7	ASH																Floodplain	
4421	10	LIGUSTRUM MULTI-TRUNK																Floodplain	INVASIVE
4422	10	LIGUSTRUM MULTI-TRUNK																Floodplain	INVASIVE
4423	10	LIGUSTRUM MULTI-TRUNK																Floodplain	INVASIVE
4424	8	LIGUSTRUM MULTI-TRUNK																Floodplain	INVASIVE
4425	14	LIGUSTRUM MULTI-TRUNK																Floodplain	INVASIVE
4426	21	ASH								21								Floodplain	
4429	12	ASH			12													Floodplain	
4430	18	MESQUITE					18											Floodplain	
4431	10	MESQUITE					10											Floodplain	
4433	8	HACKBERRY																Floodplain	
4434	8	HACKBERRY																Floodplain	
44285	16	ASH			16													Floodplain	
61315	19	PINE			19													Floodplain	SEE SPECIAL CARE NOTE
61365	13	ASH								13								Floodplain	
61366	20	HACKBERRY								20								Floodplain	
61446	24	ASH												24				Floodplain	
Subtotal Inches:			0	0	85	20	28	0	0	77	0	0	0	24	0	0	0		
Subtotal Inches in Floodplain:			0	0	85	20	28	0	0	77	0	0	0	24	0	0	0		
Subtotal Inches in Floodplain Buffer:			0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
Subtotal Inches in Floodplain Uplands/Non-Floodplain:			0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		

Notes:
1) "Mitigation Required" is amount needed to achieve 25% Preservation in Uplands; 80% preservation in the Floodplain and Floodplain Buffer.
2) "Adjusted Mitigation Required" is amount needed to achieve Preservation including Additional Inches Preserved for Mitigation.
3) For "Mitigation Required" and "Adjusted Mitigation Required" values, a positive number indicates inches required for mitigation, negative number indicates surplus inches over minimum required.
4) "Floodplain Buffer" is 30 feet wide if project area is outside the Edwards Aquifer Recharge Zone or Contributing Zone; or 60 feet wide if inside the Edwards Aquifer Recharge Zone or Contributing Zone.
* Value of the 12 inches or greater trunk is the value given to the heritage small tree species.
** Crabapple and Possumhaw In Floodplain Only
1 *** Tree is in decline.

March 19, 2015

NO	DATE	DESCRIPTION	DWG	CHK
1	3/19/2015	Addendum 1, Tree 71 Preserved		

REVISIONS

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TBPE REG. NO. F-3580

CITY OF SAN ANTONIO
TRANSPORTATION & CAPITAL IMPROVEMENTS
SEELING CHANNEL IMPROVEMENTS PHASE 2

SUMMARY OF TREE INVENTORY TABLE I

100% SUBMITTAL | PROJECT NO.: 60312595 | DATE: MARCH, 2015
DRWN. BY: CP | DSGN. BY: BEH | CHKD. BY: EKC | SHEET NO. 133

TREE INVENTORY SUMMARY TABLE

Tree No.	DBH	Species	Significant Small species: One Trunk 5.0 inches - 11.5 inches(Persimmon, Redbud, Mountain Laurel, Condalia, Possumhaw**, and Crabapple**)		Significant Tree: 6.0 inches - 23.5 inches		Significant Tree: 10.0 inches - 23.5 inches (Huisache, Ash Juniper, and Mesquite)		Significant Tree: One Trunk 10.0 inches - 23.5 inches (Arizona Ash, Hackberry)		Heritage Small Species: *One Trunk 12 inches or greater (Persimmon, Redbud, Mountain Laurel, Condalia, Possumhaw**, and Crabapple**)		Heritage Large Species (1:1): 24 inches or greater Ashe Juniper, Arizona Ash, Hackberry, Mesquite, Huisache		Heritage Large Species (3:1): 24 inches or greater		Additional Inches: Preserved for Mitigation	Location (Floodplain, Floodplain Buffer, or Uplands/Non-Floodplain)	Comments
			Removed	Preserved	Removed	Preserved	Removed	Preserved	Removed	Preserved	Removed	Preserved	Removed	Preserved	Removed	Preserved	Preserved		
63306	12	PECAN				12												Floodplain	
63307	28	ASH												28				Floodplain	
64911	13	PECAN				13													
65720	22	ASH						22											
65777	10	PECAN				10													
65778	20	ASH						20											
65839	6	ASH JUNIPER																Floodplain	
66119	6	OAK				6												Floodplain	
66120	6	OAK				6												Floodplain	
66224	30	MESQUITE												30				Floodplain	
66225	39	MESQUITE												39				Floodplain	
66446	20	OAK				20												Floodplain	
66447	8	PALM																Floodplain	NOT SIGNIFCANT TREE
66448	8	PALM																Floodplain	NOT SIGNIFCANT TREE
66449	24	PECAN														24		Floodplain	SEE SPECIAL CARE NOTE
70087	18	TALLOW																Floodplain	INVASIVE
70088	9	MOUNT. LAUREL	9															Floodplain	
70320	8	CHINESE PISTACHE																Floodplain	INVASIVE
80000	10	MOUNT. LAUREL	10															Floodplain	
80001	6	HACKBERRY																Floodplain	
80002	7	HACKBERRY																Floodplain	
		Subtotal Inches:	19	0	0	67	0	42	0	0	0	0	0	97	0	24	0		
		Subtotal Inches in Floodplain:	19	0	0	44	0	0	0	0	0	0	0	97	0	24	0		
		Subtotal Inches in Floodplain Buffer:	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
		Subtotal Inches in Floodplain Uplands/Non-Floodplain:	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		

1

Floodplain			Floodplain Buffer			Upland/Non-Floodplain			Project Total		
Total Removed:	132	Inches	Total Removed:	0	Inches	Total Removed:	0	Inches	Total Removed:	132	Inches
Total Preserved:	641	Inches	Total Preserved:	0	Inches	Total Preserved:	0	Inches	Total Preserved:	641	Inches
Preservation:	83	%	Preservation:	-	%	Preservation:	-	%	Preservation:	83	%
Mitigation Required:	-113	Inches	Mitigation Required:	0	Inches	Mitigation Required:	0	Inches	Mitigation Required:	-113	Inches
Adjusted Mitigation:	-113	Inches	Adjusted Mitigation:	0	Inches	Adjusted Mitigation:	0	Inches	Adjusted Mitigation:	-113	Inches

Notes:

1) "Mitigation Required" is amount needed to achieve 25% Preservation in Uplands; 80% preservation in the Floodplain and Floodplain Buffer.
2) "Adjusted Mitigation Required" is amount needed to achieve Preservation including Additional Inches Preserved for Mitigation.
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1 *** Tree is in decline.

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LICENSED
PROFESSIONAL ENGINEER

March 19, 2015

NO	DATE	DESCRIPTION	DWG	CHK
1	3/19/2015	Addendum 1, Tree 71 Preserved		
		REVISIONS		

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CITY OF SAN ANTONIO
TRANSPORTATION & CAPITAL IMPROVEMENTS
SEELING CHANNEL IMPROVEMENTS PHASE 2

SUMMARY OF TREE INVENTORY TABLE III

100% SUBMITTAL	PROJECT NO.: 60312595		DATE: MARCH, 2015
DRWN. BY: CP	DSGN. BY: BEH	CHKD. BY: EKC	SHEET NO. 135

GEOTECHNICAL ENGINEERING STUDY

FOR

SEELING CHANNEL IMPROVEMENTS, PHASE II
SAN ANTONIO, TEXAS

RABA KISTNER CONSULTANTS

Raba Kistner Consultants, Inc.
12821 W. Golden Lane
San Antonio, TX 78249
P.O. Box 690287
San Antonio, TX 78269
www.rkci.com

P 210 :: 699 :: 9090
F 210 :: 699 :: 6426
TBPE Firm F-3257

Project No. ASA14-003-00
September 12, 2014

Ms. Erin Cavazos, P.E., CFM
AECOM
112 E. Pecan, Suite 400
San Antonio, Texas 78205

RE: Geotechnical Engineering Study
Seeling Channel Improvements, Phase II
San Antonio, Texas

Dear Ms. Cavazos:

Raba Kistner Consultants Inc. (RKCI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with RKCI Proposal No. PSA13-049-00 (5th Revision), dated August 5, 2013. The purpose of this study was to drill borings within the vicinity of proposed transportation and capital improvements to Seeling Channel Phase II, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations for the bridge and retaining wall structures, a global stability analysis for retaining wall structures, as well as to provide pavement design and construction guidelines for the reconstruction of the Phase II roadways.

The following report contains our design recommendations and construction considerations based on our current understanding of the project information provided to us. There may be alternatives for value engineering of the foundation and pavement systems, and RKCI recommends that a meeting be held with the Owner and design team to evaluate these alternatives.

We appreciate the opportunity to be of service to you on this project. Should you have any questions about the information presented in this report, or if we may be of additional assistance with value engineering or on the materials testing-quality control program during construction, please call.

Very truly yours,

RABA KISTNER CONSULTANTS, INC.

Yvonne G. Thomas
Yvonne Garcia Thomas, P.E.
Project Engineer

Ian Perez
T. Ian Perez, P.E.
Area Project Manager

STATE OF TEXAS
TERRANCE IAN PEREZ
101225
LICENSED
PROFESSIONAL ENGINEER

YGT/TIP/dlc
Attachments
Copies Submitted: Above (5)

GEOTECHNICAL ENGINEERING STUDY

For

SEELING CHANNEL IMPROVEMENTS, PHASE II
SAN ANTONIO, TEXAS

Prepared for

AECOM
San Antonio, Texas

Prepared by

RABA KISTNER CONSULTANTS, INC.
San Antonio, Texas

PROJECT NO. ASA14-003-00
September 12, 2014

RABAKISTNER

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Important Information About Your Geotechnical Engineering Report

INTRODUCTION

Raba Kistner Consultants Inc. (RKCI) has completed the authorized subsurface exploration and foundation and pavement analysis for the proposed transportation and capital improvements to Seeling Channel Phase II in San Antonio, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation design and construction considerations for the proposed channel and bridge improvements, as well as for pavement design and construction guidelines.

PROJECT DESCRIPTION

The facilities being considered in this study include transportation and capital improvements to Seeling Channel Phase II in San Antonio, Texas. Based on our review of the Seeling Channel Improvement Phase II 40 percent submittal plans, dated May 2014, as provided by the project civil engineer, Ms. Stephanie D. Blew, P.E. with AECOM, we understand the construction limits for the channel improvements will extend from West Mistletoe Avenue (STA 41+00), approximately 250 ft west of its intersection with Wilson Boulevard, then continue along Manor Drive to West Mulberry Avenue where it ends (STA 66+50), terminating approximately 150 ft west of its intersection with Morning Glory.

Improvements to Phase II of Seeling Channel will include deepening and widening the existing reinforced concrete channel and replacing the existing channel walls with vertical cantilever retaining walls. There will also be an access ramp in the vicinity of Morning Glory. This report includes general retaining wall design and construction recommendations, as well as a global stability analysis for the retaining wall structures.

Seeling Channel bridge improvements will be completed at Morning Glory (at its intersection with West Mulberry Avenue) and West Huisache Avenue (at its intersection with Manor Drive). The Morning Glory bridge is anticipated to span approximately 88-1/2 ft, extending from STA 12+71.5 to 13+60. The West Huisache bridge will span approximately 51 ft, extending from STA 13+33 to 12+70. It is our understanding that the piers supporting the bridges will be 24 in. in diameter and will have 4 piers supporting each abutment with a single interior bent at each of these bridges.

The following information was provided to us regarding the proposed pavement improvements:

Street Name	Approximate Roadway Improvements Limits	Street Classification	Right-of-Way (ft)	No. of Lanes	Lane Widths (ft)
Emory St	STA 10+00 to 10+40	Local Type A w/out busses	50	2	13.5
West Huisache Ave (W)	STA 10+58 to 15+16 STA 21+50 to 21+91 STA 22+48 to 23+00	Local Type A w/out busses	varies	2	13.5
West Magnolia Ave	STA 10+10 to 10+80	Local Type A w/out busses	50	2	13.5

Street Name	Approximate Roadway Improvements Limits	Street Classification	Right-of-Way (ft)	No. of Lanes	Lane Widths (ft)
Manor Dr (E & SW)	STA 120+00 to 127+18 STA 223+95 to 224+75	Local Type A w/out busses	varies	2	10
Manor Dr (NE)	STA 127+18 to 128+00 STA 129+40	Local Type B	80	4 (2 bike lanes)	13/5
West Mistletoe Ave (NW)	STA 112+27 to 120+00	Local Type A w/out busses	varies	2	10
Morning Glory	STA 10+50 to 15+40	Local Type B	60 to 80	4 (2 bike lanes)	12/6
West Mulberry Ave (N)	STA 11+65 to 12+45	Local Type A w/out busses	50	2	13
West Mulberry Ave (N & S)	STA 13+30 to 21+38 STA 22+00 to 23+80 STA 235+25 to 235+87 STA 225+00 to 226+30	Local Type A w/out busses	varies	2	10

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of south/central Texas and for the use of AECOM and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses. This report is not intended for use in determining construction means and methods.

The recommendations submitted in this report are based on the data obtained from fourteen borings drilled at this site and our understanding of the project information provided to us. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site. The nature and extent of variations across the site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the site. No environmental opinions are presented in this report.

If final grade elevations are significantly different from those discussed in this report (more than plus or minus 1 ft), our office should be informed about these changes. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

BORINGS AND LABORATORY TESTS

Subsurface conditions at the site were evaluated by 14 borings drilled at the locations shown on the Boring Location Map, Figure 1. These locations are approximate and distances were measured using a recreational grade, handheld GPS locator; tape; angles; pacing; etc. The borings were drilled using a truck-mounted drilling rig to the approximate maximum depths presented in the table below. The ground surface elevations presented in the table below were provided by AECOM.

Boring No.	Ground Surface Elevation (ft)[1]	Approximate Maximum Depth (ft)
B-101	690.66	55
B-102	690.21	55
B-103	686.54	55
B-104	687.67	55
RW-101	689.36	35
RW-102	686.54	35
P-101	692.05	15
P-102	693.61	15
P-103	692.10	15
P-104	691.45	15
P-105	689.24	15
P-106	691.45	15
P-107	686.28	15
P-108	689.27	15

1) ground surface elevations provided by AECOM

During drilling operations, the following samples were collected:

Type of Sample	Number Collected
Split-Spoon (with Standard Penetration Test)	67
Texas Cone Penetrometer	44
Hand-Collected Grab Sample	38
Undisturbed Shelby Tube	7

In addition to the above, two bulk samples of the representative subgrade were obtained from the proposed pavement areas for use in a California Bearing Ratio (CBR) analyses and for pH-Lime Series testing.

Each sample was visually classified in the laboratory by a member of our Geotechnical Engineering staff. The geotechnical engineering properties of the strata were evaluated by the following tests:

Type of Test	Number Conducted
Natural Moisture Content	112
Atterberg Limits	22
Unconfined Compression	6

The results of all laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 15. A key to classification terms and symbols used on the logs is presented on Figure 16. The results of the laboratory and field testing are also tabulated on Figure 17 for ease of reference. The results of the CBR tests are presented on the Moisture-Density Relationship Curves on Figures 18 and 20 and the pH-Lime Series Curves are presented on Figures 19 and 21.

Texas Cone Penetration (TCP) test results are noted as “blows per ft” on the TxDOT Boring Logs for Borings B-101 through B-104 (Figures 2 through 5), where “blows per ft” refers to the number of blows by a 170 lb. falling hammer. The cone is driven 12 blows or approximately 6 inches, to seat it in the soil or rock. The number of blow counts proceeding is recorded for 1 ft penetration into the soil/weak rock. Where hard or dense materials were encountered, the tests were terminated at 100 blows even if one foot of penetration had not been achieved.

Standard Penetration Test (SPT) results are noted as “blows per ft” on the boring logs for Borings RW-101, RW-102, and P-101 through P-108 (Figures 6 through 15), where “blows per ft” refers to the number of blows by a falling hammer required for 1 ft of penetration into the soil/weak rock. Where hard or dense materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved. When all 50 blows fall within the first 6 in. (seating blows), refusal “Ref” for 6 in. or less will be noted on the boring logs and on Figure 17.

The TCP and SPT results are both summarized in the “Blows per ft” column on Figure 17.

Samples will be retained in our laboratory for 30 days after submittal of this report. Other arrangements may be provided at the request of the Client.

GENERAL SITE CONDITIONS

SITE DESCRIPTION

The Seeling Channel Phase II project site is an existing channel that meanders through a developed residential subdivision in San Antonio, Texas. Project limits for the Phase II channel improvements will extend from West Mistletoe Avenue then continue along Manor Drive to West Mulberry Avenue where it ends. The project site is generally concrete lined and is located northwest of and feeds into the northwest side of Woodlawn Lake.

GEOLOGY

A review of the *Geologic Atlas of Texas, San Antonio Sheet*, indicates that this site is naturally underlain with the soils of the Navarro Group and Marlbrook Marls. This formation typically consists of clays and marly clays and can contain hard layers of marl, sandstone, and siltstone. The clays of this formation are typically highly expansive, montmorillonitic clays. A key geotechnical engineering concern for development supported on this formation is expansive, soil-related movements.

SEISMIC COEFFICIENTS

On the basis of the soil borings conducted for this investigation, the upper 100 feet of soil may be characterized as very dense soil and soft rock and a **Class C** Site Class Definition (Chapter 20 of ASCE 7) has been assigned to this site.

On the basis of the United States Geological Survey (USGS) website[1] which utilizes the International Building Code (IBC) and U.S. Seismic Design Maps to develop seismic design parameters, the following seismic considerations are associated with this site.

- **S_s = 0.105g**
- **S_1 = 0.028g**
- **S_{ms} = 0.127g**
- **S_{m1} = 0.048g**
- **S_{DS} = 0.084g**
- **S_{D1} = 0.032g**

Based on the parameters listed above as well as Tables 1613.3.5(1) and 1613.3.5(2) of the 1012 IBC, the Seismic Design Category for both short period and 1 second response accelerations is **A**. As part of the assumptions required to complete the calculations, a Risk Category of "I or II or III" was selected.

STRATIGRAPHY

The subsurface stratigraphy at this site can generally be described as brown clays overlying tan to tan and gray clays which are underlain by gray clayshale. The boring logs should be consulted for more specific stratigraphic information. Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. The lines designating the interfaces between strata on the boring logs represent approximate boundaries. Transitions between strata may be gradual.

[1] http://geohazards.usgs.gov/designmaps/us/application.php

GROUNDWATER

During drilling operations, groundwater seepage was noted as it was encountered in our borings. Final groundwater measurements were also noted upon completion of our drilling operations. The table below presents groundwater measurements in those borings that encountered groundwater during and upon completion of the drilling operations.

Boring No.	Measurements	
	Groundwater Seepage (ft)	Upon Completion of Drilling (ft)
B-102	20	19
B-104	26	26

It is possible for groundwater to exist beneath this site at shallow depths on a transient basis, particularly following periods of precipitation. Fluctuations in groundwater levels occur due to variation in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

FOUNDATION ANALYSIS

EXPANSIVE SOIL-RELATED MOVEMENTS

The anticipated ground movements due to swelling of the underlying soils at the site were estimated using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). PVR values ranging from 4 to 5-1/4 in. were estimated for the stratigraphic conditions encountered in our borings. A surcharge load of 1 psi (concrete slab and sand cushion), an active zone of 15 ft, and dry moisture conditions were assumed in estimating the above PVR values.

The TxDOT method of estimating expansive soil-related movements is based on empirical correlations utilizing the measured plasticity indices and assuming typical seasonal fluctuations in moisture content. If desired, other methods of estimating expansive soil-related movements are available, such as estimations based on swell tests and/or soil-suction analyses. However, the performance of these tests and the detailed analysis of expansive soil-related movements were beyond the scope of the current study. It should also be noted that actual movements can exceed the calculated PVR values due to isolated changes in moisture content or if water seeps into the soils to greater depths than the assumed active zone depth due to deep trenching or excavations.

Based on our understanding of the project information provided to us, expansive soil-related movements are of concern at the interface between structures such as between the roadway and sidewalks, between the roadway/sidewalks and the bridge abutments, and between the residential driveways and the roadway. In our opinion utilizing the estimated PVR values to determine the expansive potential of the soils is appropriate for these purposes as it relates to this study. We do not feel that additional testing to further quantify the expansive potential of the in-situ soils is necessary for the purposes of this project.

FOUNDATION RECOMMENDATIONS

SITE GRADING

Site grading plans can result in changes in almost all aspects of foundation recommendations. We have prepared all foundation recommendations based on the existing ground surface elevations, the stratigraphic conditions encountered at the time of our study, and the 40 percent plans dated May 2014. It is our understanding that a majority of the grade changes along the project alignment include cut grading on the order of 2 to 4 ft and that isolated areas of fill are anticipated and generally occur in the vicinity of roadway and bridge intersections. If site grading plans differ from those presented on the 40 percent plans dated May 2014, RKCI must be retained to review the site grading plans prior to bidding the project for construction. This will enable RKCI to provide input for any changes in our original recommendations that may be required as a result of site grading operations or other considerations.

AREA FLATWORK

It should be noted that ground-supported flatwork such as walkways, courtyards, etc. will be subject to the same magnitude of potential soil-related movements as discussed previously (see *Expansive Soil-Related Movement* section). Thus, where these types of elements abut rigid foundations or isolated/suspended structures, differential movements should be anticipated. As a minimum, we recommend that flexible joints be provided where such elements abut rigid structures to allow for differential movement at these locations. Where the potential for differential movement is objectionable, it may be beneficial to consider methods of reducing anticipated movements.

DRILLED, STRAIGHT-SHAFT PIERS – BRIDGE FOUNDATIONS

Axial Capacity

We have computed allowable downward vertical capacities for 18, 24, and 30 in. diameter drilled piers for the proposed bridge improvements at Morning Glory and West Huisache Ave. Straight-shaft piers should be designed as friction and end bearing units using the capacities presented graphically on the “Drilled Pier Axial Capacity Curves” on Figure 26. Side shear resistance was neglected to the approximate channel bottom elevations indicated in the notes on the drilled pier capacity curves as well as in the bottom one shaft diameter.

Pier capacity curves were developed using correlations derived from results of the TCP testing and using the *Texas Department of Transportation Geotechnical Manual* dated December 2012. The indicated capacities on these figures are for dead load plus live loads. Dead loads should not exceed two-thirds of the computed capacities.

Uplift Force

The pier shafts will be subject to potential uplift forces if the surrounding expansive soils within the active zone are subjected to alternate drying and wetting conditions. The active zone assumed for calculation of the uplift force was assumed to be 15 ft below the existing ground surface at our boring locations, which is an accepted industry standard depth for expansive soil-related movements in the south/central Texas region. Below this depth, it is assumed that the impact on movement from moisture fluctuation is relatively small and that the overburden pressures will resist potential movements from groundwater infiltration. The maximum potential uplift force acting on the shaft may be estimated by:

$$F_u = 85*D$$

where:

F_u = uplift force in kips; and
D = diameter of the shaft in feet.

Uplift Resistance

Resistance to uplift forces exerted on the drilled, straight-shaft piers will be provided by the sustained compressive axial force (dead load) plus the allowable uplift resistance provided by the soil. The resistance provided by the soil depends on the shear strength of the soils adjacent to the pier shaft and below the depth of the active zone. The allowable uplift resistance provided by the soils at this site may be estimated using the “Drilled Pier Uplift Capacity Curves” presented graphically on Figure 27. Side shear resistance was neglected to the approximate channel bottom elevations presented in the notes on the uplift capacity curves.

Reinforcing steel will be required in each pier shaft to withstand a net force equal to the uplift force minus the uplift resistive force and the sustained compressive load carried by that pier. We recommend that each pier be reinforced to withstand this net force or an amount equal to 1 percent of the cross-sectional area of the shaft, whichever is greater.

To effectively reduce pier group effects and reduction in individual pier capacity, piers should be located with a minimum center-to-center spacing of three shaft diameters.

Based on the maximum allowable loads for a single pier, we estimate total settlements on the order of 1/2 in. to 1 in. to mobilize allowable static capacities. Post-construction settlement is estimated to be on the order of 1/2 in. to 1 in. between adjacent abutments. Post-construction settlement will be dependent on the final structural loading, pier spacing, and group size. We recommend that RKCI be retained to review the final loads and pier group layouts, to review pier capacities and to check estimated foundation settlements.

Lateral Resistance

Resistance to lateral loads and the expected pier behavior under the applied loading conditions will depend not only on subsurface conditions, but also on loading conditions, the pier size, and the engineering properties of the pier. As this information is not yet available, analysis of pier behavior is not possible at this time. Once preliminary pier sizes, concrete strength, and reinforcement are known, piers should be analyzed to determine the resulting lateral deflection, maximum bending moment, and ultimate bending moment. This type of analysis is typically performed utilizing a computer analysis program and usually requires a trial and error procedure to appropriately size the piers and meet project tolerances.

To assist the design engineer in this procedure, we are providing the following soil parameters for use in analysis. These parameters are in accordance with the input requirements of one of the more commonly used computer programs for laterally loaded piles, the LPile program. If a different program is used for analysis, different parameters and limitations may be required than what were assumed in selecting the parameters given below. Thus, if a program other than LPile is used, **RKCI** must be notified of the analysis method, so that we can review and revise our recommendations if required.

The soil-related parameters required for input into the LPile program are summarized in the tables below:

Morning Glory Bridge

Assumed Behavior for Analysis	Elevation (ft)	c (tsf)	k_s (pci)	k_c (pci)	ε_{50}	γ (pcf)
Soft Clay (Matlock)	690 to 671	0.25	-	-	0.02	105
Stiff Clay without Free Water	671 to 635	4.00	2,000	800	0.004	135

West Huisache Bridge

Assumed Behavior for Analysis	Elevation (ft)	c (tsf)	k_s (pci)	k_c (pci)	ε_{50}	γ (pcf)
Soft Clay (Matlock)	687 to 665	0.25	-	-	0.02	105
Stiff Clay without Free Water	665 to 632	4.00	2,000	800	0.004	135

Where:

c = undrained cohesion
k_s = p-y modulus (static)
k_c = p-y modulus (cyclic)
ε_{50} = strain factor
γ = effective unit weight

The depth over which the "Soft Clay (Matlock)" parameters should be utilized generally corresponds to the "neglected" depth presented on the pier capacity curves and does not include other design considerations such as scour. If design considerations will impact the piers to elevations lower than the depths presented in the table above for the "Soft Clay (Matlock)" layers in the tables above, RKCI should be retained to re-evaluate our recommendations.

The values presented above for subgrade modulus and the strain at 50% are based on recommended values for the LPile program for the strength of materials encountered in our borings and are not necessarily based on laboratory test results.

The parameters presented in the above table do **not** include factors of safety. We recommend that a factor of safety of at least 2 be introduced to the analysis by doubling the applied lateral loads and moments.

It should be noted that where piers are spaced closer than three shaft diameters center to center, a modification factor should be applied to the p-y curves to account for a group effect. We recommend the following p-Multipliers for the corresponding center to center pier spacings.

Spacing (in shaft diameters)	p-Multiplier
3	1.0
2	0.75
1	0.50

RETAINING STRUCTURES

RKCI understands that the improvements to Seeling Channel will include widening and deepening the channel and replacing the existing channel walls with cast-in-place concrete walls. This report briefly describes the procedures utilized during this study and presents the results of our global stability analyses. If the details of the retaining wall construction or final site grading are different from those presented in the drawings provided by AECOM then we should be notified of these changes so that we can evaluate the significance of these changes on the global stability of these walls.

These drawings along with the 2 borings drilled during the study were used for the purposes of the global stability analysis. There is 1 wall type presented in the AECOM drawings and will be referred to as Wall Type A. Wall Type A consists of a temporary soldier pile and lagging wall with permanent cast in place concrete wall.

On the basis of the above listed drawings, the following is our understanding of wall :

- Wall Type A will consist of a reinforced, cast-in-place concrete wall and will have a maximum wall height of 14.47 ft from the bottom of the slab. The slab thickness will be 1 ft 4 in. (typical) and the slab will consist of reinforced concrete. The soldier piles will consist of a galvanized W12x106 steel beam with a minimum embedment of 5 ft into a 30 in. diameter concrete pier founded 25 ft below the bottom of the slab. The piles will be constructed at a maximum center to center spacing of 8 ft. Timber lagging will be placed between the steel beams to retain the soil. Mirafi G series drainage mats (or equal) will be placed behind the lagging and any voids between the drain mat and the retained soil will be filled with a suitable fill material. A gravel layer will support the slab and Mirafi 140N filter fabric will be placed underneath the gravel layer.

GLOBAL STABILITY ANALYSIS

Conventional design of engineered works typically assumes that a calculated design factor of safety of 1.5 or greater is sufficient for global stability where the consequences of a global stability failure involve limited damage to property and no reasonably foreseeable risk of loss of life. The selection of an acceptable design factor of safety by the owner involves an evaluation of the level of acceptable risk as well as the cost of the completed project.

Assumed Subsurface Conditions

The evaluation of global stability of the proposed retaining walls were based on a ground surface profile taken from the previously cited plans provided by AECOM; a subsurface profile based on the soil borings drilled during our study; and soil engineering properties based on testing performed for that study and published correlations as well as assumed engineering judgment. In addition, two groundwater conditions were considered in our analyses: a rapid draw down condition where the assumed groundwater level is at the ground surface (drained in the fill material) and a steady state condition where the groundwater level is equal to the normal Woodlawn Lake pool elevation of 672.1 ft (per the provided drawings). Both long term (drained) conditions and short term (undrained) conditions were considered in our analyses.

The following table presents the assumed soil properties for undrained conditions.

Soil	Total Unit Weight, pcf	Cohesion, psf	Angle of Internal Friction, degrees
Gravel	115	0.02	35
Dark Brown Clay	130	600	0
Tan and Gray Clay	131	900	0
Clayshale	134	6000	0

The following table presents the assumed soil properties for drained conditions.

Soil	Total Unit Weight, pcf	Cohesion, psf	Angle of Internal Friction, degrees
Gravel	115	0.02	35
Dark Brown Clay	130	300	20
Tan and Gray Clay	131	500	20
Clayshale	134	500	0

A conservative value was selected for the drained clay shale layer in order to drive the failure plane below the bottom of the soldier pile.

Methods Of Analysis

A wide variety of methods are available for performing global stability analyses. The selection of the method of analysis is important, since the variation in computed factors of safety may vary between methods by 20 percent or more. The U.S. Army Corps of Engineers (USACE) has published a series of Engineering Manuals covering the majority of topics of interest for USACE designed projects. USACE EM 1110-2-1902 deals with slope stability, and directly addresses the issue of global stability calculation methods. The following quote deals with the selection of a method of analysis when performing global stability analyses.

> The various limit equilibrium methods use different assumptions to make the number of equations equal to the number of unknowns. They also differ with regard to which equilibrium equations are satisfied. For example, the Ordinary Method of Slices, the Simplified Bishop Method, and the U.S. Army Corps of Engineers' Modified Swedish Methods do not satisfy all the conditions of static equilibrium. Methods such as the Morgenstern and Price's and Spencer's do satisfy all static equilibrium conditions. Methods that satisfy static equilibrium fully are referred to as "complete" equilibrium methods.[2]

The results presented in this report were performed using a computer program published by Rocscience, Inc. called Slide. Version 6.014 of Slide was employed in the analyses, and Spencer's method was selected for use as the method of analysis.

Slope Stability Considerations

The performance of global stability analyses involves the selection of a variety of assumptions regarding likely modes of failure, external loads, and construction conditions. Non-circular, or general, failure surfaces were used to perform the final evaluation of global stability for the proposed retaining walls. General failure surfaces differ from circular surfaces in that a specific form of global stability is not assumed before the automated search for the global minimum factor of safety is undertaken. Various global failure modes can result from this type of analysis, including circular, block, wedge, translational,

[2] USACE Engineering Manual 1110-2-1902 Engineering and Design - SLOPE STABILITY, 31 October 2003, page C-5.

and combination failure surfaces. The computer selects varied starting and ending points for a large number of trial surfaces, and chooses an initial failure surface from that initial geometry.

In these analyses, we chose to use a large number of initial trial surfaces and had Slide use an optimization scheme on each assumed failure surface to estimate the local minimum calculated factor of safety. The program then presents the failure surfaces with the minimum calculated factor of safety, which are presented on Figures 22 through 25. We have reviewed the results of the slide analyses, and believe that the calculated failure surfaces are kinematically permissible (i.e. could reasonably occur) and the associated calculated factors of safety are within the range we would expect. The surfaces chosen by Slide have calculated factors of safety for the undrained and drained cases, and for steady state and rapid draw down considerations. The factors of safety for each case are presented in the table below. The factors of safety presented are considered acceptable for the purposes of these retaining walls.

Factor of Safety Table		Rapid Draw Down	Steady-State
Wall Type A	Short Term (Undrained)	3.1	3.1
	Long Term (Drained)	2.4	2.4

LATERAL EARTH PRESSURES

Equivalent fluid density values for computation of lateral soil pressures acting on retaining walls were evaluated for various types of backfill materials that may be placed behind the retaining walls. These values, as well as corresponding lateral earth pressure coefficients and estimated unit weights, are presented below in preferential order for use as backfill materials.

Back Fill Type	**Estimated Total Unit Weight (pcf)**	**Active Condition**		**At Rest Condition**	
		Earth Pressure Coefficient, k_a	**Equivalent Fluid Density (pcf)**	**Earth Pressure Coefficient, k_o**	**Equivalent Fluid Density (pcf)**
Washed Gravel	135	0.29	40	0.45	60
Crushed Limestone	145	0.24	35	0.38	55
Clean Sand	120	0.33	40	0.5	60
Pit Run Clayey Gravels or Sands	135	0.32	45	0.48	65
Clays	120	0.59	70	0.74	90

The values tabulated above under “Active Conditions” pertain to flexible retaining walls free to tilt outward as a result of lateral earth pressures. For rigid, non-yielding walls the values under “At-Rest Conditions” should be used.

The values presented above assume the surface of the backfill materials to be level. Sloping the surface of the backfill materials will increase the surcharge load acting on the structures. The above values also do not include the effect of surcharge loads such as construction equipment, vehicular loads, or future storage near the structures. If any of these loading occur within a few feet of the back of these walls then an appropriate surcharge load should be added at the top of the wall to account for these loadings. The recommended values in the table do not account for possible hydrostatic pressures resulting from groundwater seepage entering and ponding within the backfill materials. However, these surcharge loads and groundwater pressures should be considered in designing any structures subjected to lateral earth pressures.

The on-site clays exhibit significant shrink/swell characteristics. The use of these soils as backfill against the proposed retaining structures is not recommended. These soils generally provide higher design active earthen pressures, as indicated above, but may also exert additional active pressures associated with swelling. Controlling the moisture and density of these materials during placement will help reduce the likelihood and magnitude of future active pressures due to swelling, but this is no guarantee.

BACKFILL COMPACTION

Placement and compaction of backfill behind the retaining walls will be critical, particularly at locations where backfill will support adjacent near-grade foundations and/or flatwork. If the backfill is not properly compacted in these areas, the adjacent foundations/flatwork can be subject to settlement.

To reduce potential settlement of adjacent foundations/flatwork, the backfill materials should be placed and compacted as recommended in the *Select Fill* section of this report. Each lift or layer of the backfill should be tested during the backfilling operations to document the degree of compaction. Within at least a 5-ft zone of the walls, we recommend that compaction be accomplished using hand-guided compaction equipment capable of achieving the maximum density in a series of 3 to 5 passes.

DRAINAGE

The use of drainage systems is a positive design step toward reducing the possibility of hydrostatic pressure acting against the retaining structures. Drainage may be provided by the use of a drain trench and pipe. The drain pipe should consist of a slotted, heavy duty, corrugated polyethylene pipe and should be installed and bedded according to the manufacturer's recommendations. The drain trench should be filled with gravel (meeting the requirements of ASTM D 448 coarse concrete aggregate Size No. 57 or 67) and extend from the base of the structure to within 2 ft of the top of the structure. The bottom of the drain trench will provide an envelope of gravel around the pipe with minimum dimensions consistent with the pipe manufacturer's recommendations. The gravel should be wrapped with a suitable geotextile fabric (such as Mirafi 140N or equivalent) to help reduce the intrusion of fine-grained soil particles into the drain system. The pipe should be sloped and equipped with clean-out access fittings consistent with state-of-the-practice plumbing procedures.

As an alternative to a full-height gravel drain trench behind the proposed retaining structures, consideration may be given to utilizing a manufactured geosynthetic material for wall drainage. A number of products are available to control hydrostatic pressures acting on earth retaining structures, including Amerdrain (manufactured by American Wick Drain Corp.), Miradrain (manufactured by Mirafi, Inc.), Enkadrain (manufactured by American Enka Company), and Geotech Insulated Drainage Panel (manufactured by Geotech Systems Corp.). The geosynthetics are placed directly against the retaining structures and are hydraulically connected to the gravel envelope located at the base of the structures. Any of the above systems are appropriate for conventional, cantilevered retaining walls. If other wall systems are used then the wall manufacturer should be consulted regarding specific drainage systems that might be required.

Weepholes may be provided along the length of the proposed retaining structures, if desired, in addition to one of the two alternative drainage measures presented above. Based on our experience, weepholes, as the only drainage measure, often become clogged with time and do not provide the required level of drainage from behind retaining structures. We recommend that RKCI review the final retaining structure drainage design before construction.

FOUNDATION CONSTRUCTION CONSIDERATIONS

SELECT FILL

Materials used as select fill preferably should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT 2004 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 247, Flexible Base, Type A, Grade 2.

Soils classified as CH, CL, MH, ML, SM, GM, OH, OL and Pt under the USCS are **not** considered suitable for use as select fill materials at this site. The native soils at this site are **not** considered suitable for use as select fill materials.

Select fill should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-113-E, Compaction Test. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction.

TEMPORARY CASING AND SLURRY TECHNIQUES

Due to the close proximity to Woodlawn Lake, we anticipate that groundwater will be encountered during drilled pier construction. Groundwater seepage and/or side sloughing is very likely to be encountered at the time of construction, depending on climatic conditions prevalent at the time of construction. Therefore, we recommend that the bid documents require the foundation contractor to specify unit costs for different lengths of casing and unit costs for slurry drilling techniques that may be required.

DRILLED PIERS

Each drilled pier excavation should be examined by a geotechnical engineer who is familiar with the geotechnical aspects of the subsurface stratigraphy, the structural configuration, foundation design details and assumptions, prior to placing concrete. This is to observe that:

- The shaft has been excavated to the specified dimensions at the correct depth established by the previously mentioned criteria;
- The shaft has been drilled plumb within specified tolerances along its total length;

- Excessive cuttings, buildup and soft, compressible materials have been removed from the bottom of the excavation.

REINFORCEMENT AND CONCRETE PLACEMENT

Reinforcing steel should be checked for size and placement prior to concrete placement. Placement of concrete should be accomplished as soon as possible after excavation to reduce changes in the moisture content or the state of stress of the foundation materials. No foundation element should be left open overnight without concreting.

LOAD TESTS

Load tests, if required by the City, should be performed in accordance with TxDOT 2004 Standard Specifications for Construction of Highways, Streets and Bridges, Item 405, Foundation Test Load procedures. The General Contractor should be responsible for providing all equipment, personnel, jacks, and construction (including reaction piles) necessary to conduct these tests.

EXCAVATION SLOPING AND BENCHING

If utility trenches or other excavations extend to or below a depth of 5 ft below construction grade, the contractor or others shall be required to develop a trench safety plan to protect personnel entering the trench or trench vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring, are beyond the scope of the current study. Any such designs and safety plans shall be developed in accordance with current OSHA guidelines and other applicable industry standards.

EXCAVATION EQUIPMENT

Our boring logs are not intended for use in determining construction means and methods and may therefore be misleading if used for that purpose. We recommend that earth-work and utility contractors interested in bidding on the work perform their own tests in the form of test pits to determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this site.

ROADWAY PAVEMENT RECOMMENDATIONS

Recommendations for both flexible and rigid pavements are presented in this report. The Owner and/or design team may select either pavement type depending on the performance criteria established for the project. In general, flexible pavement systems have a lower initial construction cost as compared to rigid pavements. However, maintenance requirements over the life of the pavement are typically much greater for flexible pavements. This typically requires regularly scheduled observation and repair, as well as overlays and/or other pavement rehabilitation at approximately one-half to two-thirds of the design life. Rigid pavements are generally more "forgiving", and therefore tend to be more durable and require less maintenance after construction.

For either pavement type, drainage conditions will have a significant impact on long term performance, particularly where permeable base materials are utilized in the pavement section. Drainage considerations are discussed in more detail in a subsequent section of this report.

SUBGRADE STRENGTH CHARACTERIZATION

We have assumed the pavement subgrade will consist of recompacted on-site clays. Two CBR's were measured using ASTM D 1883 Standard Test Method for CBR of Laboratory-Compacted Soils and were determined to be 5.7 and 6.2 using the soaked sample methodology. Swell was also measured as part of the CBR procedure and were determined to be 0.8 and 1.2 percent. Based on these results and our experience with the soils in this area, we have assumed a design CBR value of 5.0 for use in our pavement section analysis. If clay soils are imported for the purpose of constructing the roadbed then imported materials must be selected that have a CBR value of at least 5.0. If lower quality clay fill materials are utilized, the pavement sections will have to be increased based on the quality (tested CBR value) of the clays imported.

In order to investigate the potential for adverse reaction to lime in certain sulfate-containing soils, the concentration of soluble sulfates in the subgrade soils was determined on a bulk sample. The adverse reaction, referred to as sulfate-induced heave, has been known to cause cohesive subgrade soils to swell in short periods of time, resulting in pavement heaving and possible failure. The sulfate content of the CBR sample was determined to be on the order of 100 ppm. The sulfate content of the tested sample is a relatively low concentration and is not anticipated to cause any significant sulfate-induced heave at this site.

Swell/Heave Potential

As discussed in the *Expansive Soil-Related Movements* section of this report, PVR values ranging from 4 to 5-1/4 in. were estimated for the stratigraphic conditions encountered in our borings. Subgrade soils that are highly expansive will expand and heave when moisture is allowed to infiltrate these materials, causing the pavement to become rough or uneven over time. When edge drying occurs these soils shrink and longitudinal cracking occurs which typically reflects up through the base and asphalt. Pavement roughness is generally defined as an expression of irregularities in the pavement surface (bumps, dips, cracks, etc.) that adversely affect the ride quality of a vehicle and thus the user. Roughness is an important pavement characteristic because it affects not only ride quality but also vehicle costs, fuel consumption, and maintenance costs. Pavement heave and longitudinal cracking can be reduced through various measures but cannot be totally eliminated without full removal of the problematic soil. Measures available for reducing heave include:

- Soil Treatment with Lime or Other Chemicals
- Removal and Replacement of High PI Soils
- Drains or Barriers to Collect or Inhibit Moisture Infiltration

Soil treatment with lime (or other chemicals) is typically used to reduce the swelling potential of the upper portion of the pavement subgrade containing moderately to highly plastic soils. Lime and water are mixed into the top 6 to 12 inches (or possibly more) of the subgrade and allowed to cure for a period of time. After curing, the soil-lime mixture is compacted to form a strong soil matrix that can improve pavement performance by making the treated subgrade clays less susceptible to moisture infiltration and thus helping to reduce the potential for soil heave. However, in deep, highly plastic soils, lime treatment of only the top portion of the expansive subgrade may not provide an acceptable reduction in PVR. For a more substantial reduction in PVR, removal and replacement of the high PI soil may be the only method available to permanently reduce the potential vertical rise of the pavement to an acceptable level. As previously stated, partial removal of expansive clay soil only reduces the potential (or risk) of swell induced damage but it does not completely eliminate this risk.

In addition, capturing water infiltration via French drains, pavement edge drains, or inhibiting water through the use of vertical moisture barriers will help to reduce the potential for heave by reducing or eliminating water infiltration. Geocomposites, such as geogrid, and membranes (moisture barriers), are tools that are also available to help reduce the damage that heaving subgrades inflict on flexible pavements. These tools may be considered in addition to or as an alternative to other mitigation techniques discussed. There may be other means by which swell or heave damage to the pavement may be mitigated and RKCI recommends that the Client review these methods and discuss further with RKCI, if deemed necessary for a specific application.

It should be noted that the pavement sections derived in the following sections are structurally adequate for the given traffic levels and existing clay subgrade strength, but do not consider the long-term effects of pavement roughness due to heave, which can only be addressed by the measures discussed in this section.

AREA FLATWORK

It should be noted that ground-supported flatwork such as sidewalks and driveways will be subject to the same magnitude of potential soil-related movements as discussed above. Thus, where these types of elements abut rigid foundations, isolated/suspended structures, or structures overlying treated soils, differential movements should be anticipated. As a minimum, we recommend that flexible joints be provided where such elements abut rigid structures to allow for differential movement at these locations. Where the potential for differential movement is objectionable, it may be beneficial to consider methods of reducing anticipated movements or to consider extending the flexible (granular) base beneath the sidewalks and driveways to match the adjacent roadway performance. Select fill may also be used beneath sidewalks in lieu of the flexible (granular) base. Recommendations for selection and placement of both the select fill and the flexible (granular) base are presented in subsequent sections of this report. In the instances of approaches to bridge abutments a short structural ramp, hinged at the abutment and bridging to the subgrade approach can be considered. Other TxDOT methods might also be applicable to this detail.

CITY OF SAN ANTONIO DESIGN PARAMETERS – HOT MIX ASPHALT PAVEMENTS

The following information was provided to us by AECOM regarding the proposed pavement improvements:

Street Name	Approximate Roadway Improvements Limits	Street Classification	Right-of-Way (ft)	No. of Lanes	Lane Widths (ft)
Emory St	STA 10+00 to 10+40	Local Type A w/out busses	50	2	13.5
West Huisache Ave (W)	STA 10+58 to 15+16 STA 21+50 to 21+91 STA 22+48 to 23+00	Local Type A w/out busses	varies	2	13.5
West Magnolia Ave	STA 10+10 to 10+80	Local Type A w/out busses	50	2	13.5
Manor Dr (E & SW)	STA 120+00 to 127+18 STA 223+95 to 224+75	Local Type A w/out busses	varies	2	10
Manor Dr (NE)	STA 127+18 to 128+00 STA 129+40	Local Type B	80	4 (2 bike lanes)	13/5
West Mistletoe Ave (NW)	STA 112+27 to 120+00	Local Type A w/out busses	varies	2	10
Morning Glory	STA 10+50 to 15+40	Local Type B	60 to 80	4 (2 bike lanes)	12/6
West Mulberry Ave (N)	STA 11+65 to 12+45	Local Type A w/out busses	50	2	13
West Mulberry Ave (N & S)	STA 13+30 to 21+38 STA 22+00 to 23+80 STA 235+25 to 235+87 STA 225+00 to 226+30	Local Type A w/out busses	varies	2	10

<u>City of San Antonio Guidelines – Flexible Pavements</u>

Based on information provided by the City of San Antonio, we understand that the following design parameters are required for use in the design of flexible pavements for these types of streets.

Street Classification	Equivalent 18-kip Single Axle Load Applications (ESALs)	Reliability	Serviceability Initial/Terminal	Standard Deviation	Structural Number Minimum/Maximum
Local Type A Without Bus Traffic	100,000	70	4.2/2.0	0.45	2.02/3.18
Local Type B	2,000,000	90	4.2/2.0	0.45	2.92/5.08

The required structural number is related to the CBR value of the pavement subgrade and the amount of traffic that the pavement will carry over its service life. The CBR provides an estimate of the relative strength of the subgrade and consequently indicates the ability of the pavement section to carry load. This site specific CBR value is utilized in conjunction with the above specified parameters to determine the required Structural Number (SN) for use in the design of the pavement section.

To determine the required design SN value, we utilized a software program entitled "AASHTOWare DARWin 3.1.01, Pavement Design and Analysis System," which is published by the American Association of State Highway and Transportation Officials (AASHTO) and is based on the 1993 edition of the AASHTO "Guide for the Design of Pavement Structures."

The calculated design SN value is presented below. Also shown is the minimum value subsequently determined in the design of the pavement sections for this site.

Structural Number Recommendations

	Structural Number			
		Minimum Value Provided by Design		
Description	Required	Flex Base Option	Mechanically Stabilized Layer Option	Full Depth Asphalt Option
Local Type A without Bus Traffic	2.05	2.20	-	-
Local Type B	3.68	3.78	3.72	4.02

The City of San Antonio pavement guidelines state that subgrade soils with a plasticity index (PI) greater than 20 must be treated with lime or other proven methods of treatment to reduce the PI of the soil to less than 20. Based on the results of our Atterberg Limits testing, the plasticity index of the upper 5 ft of the existing subgrade ranged from 44 to 52. Thus, per the City of San Antonio, pavements at this site will need to include a minimum of 6 in. of lime-treated subgrade. However, it is our understanding that both the City of San Antonio and the CLIENT would prefer to also have options that do not include a lime-treated subgrade layer.

The following input variables are utilized to design flexible base pavements (commonly referred to as Asphaltic Cement Concrete or Asphalt pavements) when using the procedures detailed in the *1993 AASHTO Guide for Design of Pavement Structures*:

- Performance Period
- Roadbed Soil Resilient Modulus psi
- Serviceability Indices
- Overall Standard Deviation
- Reliability, %
- Design Traffic, 18-kip ESALs

Performance Period

The pavement structure was designed for a 20-year performance period which is typical for most flexible pavements.

Roadbed Soil Resilient Modulus

The Resilient Modulus (M_R) is the material property used to characterize the support characteristics of the roadbed soils in flexible pavement design. It is a measure of the soil's deformation response to cyclic applications of loads much smaller than a failure load. Using conventional correlations, local experience and a design CBR values of 5.0, a Resilient Modulus of 7,500 psi has been used for this project.

To determine the resilient modulus (M_r) of the subgrade, we utilized the equation specified in the Bexar County Design Criteria. The equation is shown below:

M_r = 1,500 x CBR

Serviceability Indices

See the recommended Initial and Terminal Serviceability Indices on the table presented in the *City of San Antonio Guidelines – Flexible Pavements* section of this report.

Overall Standard Deviation

Overall standard deviation accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given traffic. Higher values represent more variability; thus, the pavement thickness increases with higher overall standard deviations. A value of 0.45 is used for all flexible pavement designs.

Reliability, %

The reliability value represents a "safety factor," with higher reliabilities representing pavement structures with less chance of failure. The AASHTO Guide recommends values ranging from 50 to 99.9%, depending on the functional classification and the location (urban vs. rural) of the roadway. See the recommended Reliability values on the table presented in the *City of San Antonio Guidelines – Flexible Pavements* section of this report.

Design Traffic 18-kip ESAL

The 18-kip ESALs were determined from the traffic data specified in the Unified Development Code for the City of San Antonio. See the recommended values on the table presented in the *City of San Antonio Guidelines – Flexible Pavements* section of this report.

RECOMMENDED PAVEMENT SECTIONS – HOT MIX ASPHALT ROADWAY PAVEMENTS

Utilizing the design SN values discussed above and minimum layer thicknesses, the optional pavement sections presented below are recommended. If clay fill is placed for fill grading in pavement areas, it should be placed and compacted as discussed in the *On-Site Clay Fill* section of this report. For this site, the following options for pavement sections are available.

Flexible Base Option

CBR = 5.0	Layer Description	Layer Thickness	Recommended SN Coeff.	S.N. Extension
Local Type A without Bus Traffic	Type D Surface Course Flexible (Granular) Base[1] Lime-Treated Subgrade **Combined Total**	2.0 in. 6.0 in. 6.0 in. **14.0 in.**	0.44 0.14 0.08	0.88 0.84 0.48 **2.20**
Local Type B	Type D Surface Course Type C/D Surface Course Flexible (Granular) Base[1] Lime-Treated Subgrade **Combined Total**	2.0 in. 2.0 in. 11.0 in. 6.0 in. **21.0 in.**	0.44 0.44 0.14 0.08	0.88 0.88 1.54 0.48 **3.78**

1) in the above sections, the flexible (granular) base layer may be increased by 4 in. in lieu of utilizing the lime-treated subgrade layer and the pavement sections will be structurally adequate (meet or exceed the required by design structural number). However, the issues addressed in the *Swell/Heave Potential* section of this report will not be addressed.

Mechanically Stabilized Layer (MSL) Option

CBR = 5.0	Layer Description	Layer Thickness	Recommended SN Coeff.	S.N. Extension
Local Type B	Type D Surface Course Mechanically Stabilized Layer[1] Lime-Treated Subgrade **Combined Total**	3.0 in. 8.0 in. 6.0 in. **17.0 in.**	0.44 0.24 0.08	1.32 1.92 0.48 **3.72**

1) in the above sections, the MSL layer may be increased by 2 in. in lieu of utilizing the lime-treated subgrade layer and the pavement sections will be structurally adequate (meet or exceed the required by design structural number). However, the issues addressed in the *Swell/Heave Potential* section of this report will not be addressed.

A Mechanically Stabilized Layer (MSL) is a composite layer consisting of flexible (granular) base and a Tensar TriAx product. TriAx geogrid provides lateral restraint to the flexible base by confining aggregate particles within the plane of the geogrid, thereby creating a reinforced, or mechanically stabilized layer. The unique design of the TriAx geogrid allows the thickness of the reinforced layer to be optimized which reduces the thickness of the required flexible base and provides a stronger, more resilient structure. For this particular application, we recommend Tensar TriAx TX-5 geogrid. We do not recommend that an alternative geogrid be utilized in this section as the performance of the final pavement structure may be inferior which could result in premature pavement distress. If an alternate geogrid is to be utilized in these pavement sections, we should be retained to review the properties of the material proposed and revise our recommendations as may be necessary.

Full-Depth Asphalt Option

CBR = 5.0	Layer Description	Layer Thickness	Recommended SN Coeff.	S.N. Extension
Local Type B	Type D Surface Course Type B Base Course[1] Lime-Treated Subgrade **Combined Total**	2.0 in. 7.0 in. 6.0 in. **15.0 in.**	0.44 0.38 0.08	0.88 2.66 0.48 **4.02**

1) in the above sections, the Type B Base Course layer may be increased by 2 in. in lieu of utilizing the lime-treated subgrade layer and the pavement sections will be structurally adequate (meet or exceed the required by design structural number). However, the issues addressed in the *Swell/Heave Potential* section of this report will not be addressed.

CITY OF SAN ANTONIO DESIGN PARAMETERS – PORTLAND CEMENT CONCRETE PAVEMENTS

City of San Antonio Guidelines – Rigid Pavements

Based on information provided by the City of San Antonio, we understand that the following design parameters are required for use in the design of rigid pavements for these types of streets.

Street Classification	Equivalent 18-kip Single Axle Load Applications (ESALs)	Reliability	Serviceability Initial/Terminal	Standard Deviation	Thickness Minimum/Maximum
Local Type A Without Bus Traffic	150,000	70	4.5/2.0	0.35	5 in./6 in.
Local Type B	3,000,000	90	4.5/2.0	0.35	7 in./9 in.

The following input variables are utilized to design rigid pavements (commonly referred to as Portland Cement Concrete or PCC pavements) when using the procedures detailed in the *1993 AASHTO Guide for Design of Pavement Structures*:

- Performance period
- Design traffic, 18-kip equivalent single axle loads (ESALs).
- 28-day concrete modulus of rupture, psi
- 28-day concrete elastic modulus, psi
- Effective modulus of subbase/subgrade reaction, pci
- Serviceability indices
- Load transfer coefficient
- Drainage coefficient
- Overall standard deviation
- Reliability, %

Performance Period

The pavement structure was designed for a 30-year performance period which is in accordance with Appendix 10-A of the COSA Design Guidance Manual.

Design Traffic 18-kip ESAL

The 18-kip ESALs were determined from the street classifications as discussed previously in the *City of San Antonio Guidelines – Rigid Pavements* section of this report.

28-day Concrete Modulus of Rupture, M_r

The M_r of concrete is a measure of the flexural strength of the concrete as determined by breaking concrete beam test specimens. A M_r of approximately 600 psi at 28 days was used in the analysis and is typical of local concrete production.

28-day Concrete Elastic Modulus

Elastic modulus of concrete is an indication of concrete stiffness and varies depending on the coarse aggregate type used in the concrete. A modulus of 4,000,000 psi is used for this pavement design.

Effective Modulus of Subbase/Subgrade Reaction: k-value

Concrete slab support is characterized by the modulus of subgrade/subbase reaction, otherwise known as the k-value with units typically shown as psi/in. A k-value of 140 psi/in. was used in the rigid pavement design procedure and is based upon a CBR value of 5.0 as discussed above.

Construction Note: It is recommended that the clay subgrade, be treated with lime to facilitate construction of the concrete pavement as well as to provide additional support to the pavement structure. More detail is provided in the *Pavement Construction Considerations* section of this report.

Serviceability Indices

See the recommended Initial and Terminal Serviceability Indices on the table presented in the *City of San Antonio Guidelines – Rigid Pavements* section of this report.

Load Transfer Coefficient

The load transfer coefficient is used to incorporate the effect of dowels, reinforcing steel, tied shoulders, and tied curb and gutter on reducing the stress in the concrete slab due to traffic loading and therefore causing a reduction in the required concrete slab thickness. The coefficients recommended in the AASHTO Guide are based on findings from the AASHO Road Test.

The load transfer coefficient used in this pavement design is 2.9.

Drainage Coefficient

The drainage coefficient characterizes the quality of drainage of the subbase layers under the concrete slab. Good draining pavement structures do not give water the chance to saturate the subbase and subgrade; thus, pumping is not as likely to occur.

There is no subbase recommended for this pavement structure. Therefore, the drainage coefficient used in this pavement design is 0.90 and is based upon local design experience for slabs without subbases on expansive clay subgrade.

Overall Standard Deviation

Overall standard deviation accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given traffic. Higher values represent more variability; thus, the pavement thickness increases with higher overall standard deviations. A value of 0.35 is used for this rigid pavement design.

Reliability, %

The reliability value represents a "safety factor," with higher reliabilities representing pavement structures with less chance of failure. The AASHTO Guide recommends values ranging from 50 to 99.9%, depending on the functional classification and the location (urban vs. rural) of the roadway. The reliability values utilized in our design are presented in the City of San Antonio Design Guidelines section of this report.

RECOMMENDED PAVEMENT SECTIONS – PORTLAND CEMENT CONCRETE ROADWAY PAVEMENTS

The recommended concrete slab thicknesses determined with the inputs discussed above are presented in the table below. An optional lime treated subgrade is recommended to facilitate construction but is not required. Typical cross sections will be provided in the construction documentation.

Portland Cement Concrete Design - Cross Sections	Layer Description	COSA Spec. Item	Layer Thickness
Local Type A without Bus Traffic	PCC Surface Subbase Lime Treated Subgrade[1] **Combined Total**	209 ---- 108	5.0 in. 0.0 in. 6.0 in. 11.0 in.
Local Type B	PCC Surface Subbase Lime Treated Subgrade[1] **Combined Total**	209 ---- 108	9.0 in. 0.0 in. 6.0 in. 15.0 in.

1) Used as a working or construction platform only, if constructed on clay subgrades.

PAVEMENT CONSTRUCTION CONSIDERATIONS

SITE PREPARATION

All existing paving materials should be completely removed in accordance with the 2008 City of San Antonio Standard Specification Item 104 – *Street Excavation*. Preparation for widening of street and areas for sidewalks, utilities, etc. should be performed in accordance with the 2008 City of San Antonio Standard Specification Item 101 – *Preparing of Right-of-Way*. Exposed subgrades should be thoroughly proofrolled in order to locate and densify any weak, compressible zones. A minimum of 5 passes of a fully-loaded dump truck or a similar heavily-loaded piece of construction equipment should be used for planning purposes. Proofrolling operations should be observed by the Geotechnical Engineer or his representative to document subgrade condition and preparation. Weak or soft areas identified during proofrolling should be removed and replaced with a suitable, compacted backfill.

The exposed clay subgrade should be moisture conditioned. This should be completed after proofrolling operations and just prior to flexible base placement. Moisture conditioning is done by scarifying to a minimum depth of 6 in. and recompacting to a minimum of 95 percent of the maximum density determined from the Texas Department of Transportation Compaction Test (TxDOT, Tex-114-E). The moisture content of the subgrade should be maintained within the range of optimum moisture content to 3 percentage points above optimum until permanently covered.

For areas that require fill, upon completion of fill grading using on-site clays, the final 6 in. of fill should be lime treated (see section below *Lime Treatment of Subgrade*). If non-expansive fill or imported fill is used for fill grading, RKCI should be notified so that we may re-evaluate our recommendations if necessary. If fill grading is not planned, then lime treatment of the exposed clay subgrade should be performed in conjunction with the scarifying, moisture conditioning, and recompaction described previously.

SELECT FILL

If utilized beneath sidewalk/driveway or pavement sections, select fill preferably should be either crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the City of San Antonio 2008 Standard Specifications Item 200 - *Flexible Base*, Types A or C, Grades 1 through 3.

Select fill should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-113-E, Compaction Test. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction.

ON-SITE CLAY FILL

If on-site clay fill is required under portions of the pavement reconstruction, we recommend that the on-site soils be placed to conform to the 2008 City of San Antonio Standard Specifications Item 107 – *Embankment* Type B and should be placed in lifts not exceeding 6 in. (compacted) in thickness and compacted to the requirements of Table 2 in Item 107 based on the maximum density and optimum moisture content as determined by TxDOT, Tex-114-E. The moisture content of the fill should be maintained to be at least equal to the optimum water content, but not exceed 3 percentage points above the optimum water content until permanently covered. Fill materials shall be free of roots and other organic or degradable material. We recommend that the maximum particle size not exceed 3 in. or one half the completed lift thickness, whichever is smaller.

It is imperative that the subgrade modulus utilized in the pavement design process be met or exceeded by the fill material. In the event that the clay fill used is different than the existing subgrade, the recommendations in this report could be invalidated and the design engineer must be consulted to determine if additional CBR testing and thicker pavement sections are required.

LIME TREATMENT OF SUBGRADE

Lime treatment of the subgrade soils with PIs greater than 20 should be in accordance with the 2008 City of San Antonio Standard Specification, Item 108 – *Lime Treatment for Subgrade*. Lime-treated subgrade soils should be compacted to a minimum of 95 percent of the maximum density at a moisture content within the range of optimum moisture content to 3 percentage points above the optimum moisture content as determined by Tex-114-E. Based on the results of the Lime Series Curve developed in the laboratory, we recommend that a minimum of 4 percent hydrated lime by weight be used to reduce the PI of the subgrade clays or the minimum required by the City of San Antonio of 15 pounds/S.Y. for 6 in. of lime treated subgrade. If dry placement of lime is used during construction, an additional 1 percent of lime should be added to account for expected loss.

GEOGRID

The geogrid reinforcement should be Tensar TX-5. An approved source of geogrid is The Tensar Corporation, Morrow, GA or their designated representative. The geogrid component shall be integrally formed and produced from a punched sheet of polypropylene which is then oriented in three substantially equilateral directions so that the resulting ribs shall have a high degree of molecular orientation, which continues at least in part through the mass of the integral node. The resulting geogrid structure shall have apertures that are triangular in shape, and shall have ribs with a depth-to-width ratio greater than 1.0.

The geogrid shall have the nominal characteristics shown in the table below, and shall be certified in writing by the manufacturer to be TX-5:

Properties	Longitudinal	Diagonal	Transverse	General
Rib pitch, mm (in.)	40 (1.60)	40 (1.60)		
Mid-rib depth, mm (in.)		1.3 (0.05)	1.2 (0.05)	
Mid-rib width, mm (in.)		0.9 (0.04)	1.2 (0.05)	
Rib shape				Rectangular
Aperture shape				Triangular

The geogrid should be placed at the bottom of the flexible (granular) base section in all cases. An alternative to the above geogrid should not be considered without approval from **RKCI**.

GRANULAR BASE COURSE

The flexible base course should be crushed limestone conforming to the 2008 City of San Antonio Standard Specification, Item 200 – *Flexible Base*, Type A, Grade 2. The base course should be placed in lifts with a maximum compacted thickness of 8 in. (10 inches loose) and compacted to a minimum of 95 percent of the maximum density determined by Tex-113-E at a moisture content within the range of 2 percentage points below to 2 percentage points above the optimum moisture content as determined by Tex-113-E.

PRIME COAT

A prime coat should be placed on top of a flexible base course (if used) and should be a MC-30 or AE-P conforming to the 2008 City of San Antonio Standard Specification for Construction Item 202 – *Prime Coat* as well as TxDOT Standard Specifications 2004, Item 300 – *Asphalts, Oils or Emulsions*. Prime coat application rates are typically between 0.1 to 0.3 gal/yd^2 and are generally dependent upon the absorption rate of the granular base and other environmental conditions at the time of placement. City of San Antonio Standard Specification Item 202 – *Prime Coat* states that the application rate shall not exceed 0.2 gal/yd^2.

TACK COAT

A tack coat should be placed between asphaltic concrete base and/or surface lifts and should be a PG binder with a minimum high-temperature grade of PG 58, SS-1H, CSS-1H, or EAP&T conforming to TxDOT Standard Specifications 2004, Item 300 – *Asphalts, Oils or Emulsions*. For construction, City of San Antonio Standard Specification Item 203 – *Tack Coat* shall be specified and the application rate shall not exceed 0.1 gal/yd^2. See additional requirements for tack coats in the appropriate City of San Antonio Standard Specification for Asphaltic Concrete Materials.

ASPHALTIC CONCRETE SURFACE AND/OR BINDER[3] COURSES

The asphaltic concrete surface and/or binder courses should conform to the 2008 City of San Antonio Standard Specification Item 205 – *Hot Mix Asphaltic Concrete Pavement* Types C or D for the surface and binder, and Type B for the base, if the full depth asphalt section is selected for construction. WMAC may also be considered for construction of this roadway and should conform to the TxDOT SS3267 specifications. Recycled asphalt pavement (RAP) should be limited to 20 percent of the total weight of the mix for Types C and D mixes and 30 percent for Type B mixes. Higher percentages of RAP may be permissible depending on the material source. If higher percentages of RAP are desired, contact **RKCI** for consideration. Asphalt cement grades should conform to the table shown below, which conforms to the requirements of Item 205.

[3] A binder course is defined as the asphaltic concrete layer (HMAC or WMAC) placed directly beneath the HMAC or WMAC surface or wearing course but is not an asphalt treated base layer.

<table>
<tr><th rowspan="2">Street Classifications</th><th colspan="3">Minimum PG Asphalt Cement Grade</th></tr>
<tr><th>Surface Courses</th><th>Binder & Level Up Courses</th><th>Base Courses</th></tr>
<tr><td>Primary and Secondary Arterials</td><td>PG 76-22</td><td rowspan="2">PG 70-22</td><td rowspan="4">PG 64-22</td></tr>
<tr><td>Collector and Local Type B Streets</td><td rowspan="2">PG 70-22</td></tr>
<tr><td>Local Type A Street With Bus Traffic</td><td rowspan="2">PG 64-22</td></tr>
<tr><td>Local Type A Street Without Bus Traffic</td><td>PG 64-22</td></tr>
</table>

The asphaltic concrete should be compacted on the roadway to contain from 5 to 9 percent air voids computed using the maximum theoretical specific gravity (Rice) of the mixture determined according to Test Method Tex-227-F. Pavement specimens, which shall be either cores or sections of asphaltic pavement, will be tested according to Test Method Tex-207-F. The nuclear-density gauge or other methods which correlate satisfactorily with results obtained from project roadway specimens may be used when approved by the Engineer. Unless otherwise shown on the plans, the Contractor shall be responsible for obtaining the required roadway specimens at their expense and in a manner and at locations selected by the Engineer.

It is recommended that the hot mix asphalt concrete pavement be placed with a paving machine only and not with a motor grader unless prior approval is granted by the Engineer for special circumstances.

PORTLAND CEMENT CONCRETE

The Portland cement concrete should conform to the requirements of 2008 City of San Antonio Standard Specification Item 209 – *Concrete Pavement* section 209.2.A. Hydraulic Cement Concrete Class P. Liquid membrane-forming curing compound should be applied as soon as practical after broom finishing the concrete surface and conform to section 209.2.D. Curing Compound. The curing compound will help reduce the loss of water from the concrete. The reduction in the rapid loss in water will help reduce shrinkage cracking of the concrete.

CONCRETE PAVEMENT CONSTRUCTION CONTROL

Construction of Portland Cement Concrete Pavements should be controlled by the 2008 City of San Antonio Standard Specification Item 209 – *Concrete Pavement*. The surface of all concrete pavements should be textured or tined. Texturing using carpet dragging or tining should be in accordance with Item 209 Section 3, paragraph D, sub-paragraphs 1 and 2. Other texturing techniques may be utilized as described in ACI 330.1-03 Section 3 Subparagraph 9.

CONCRETE PAVEMENT TYPE

Jointed Plain Concrete Pavement (which is referred to by TxDOT as Concrete Pavement Contraction Design or CPCD) is suggested for roadways with crosswalks, adjacent parking, or sidewalks and is recommended as the pavement type for these city streets. It is recommended that a shoulder be used if curbs are not placed on the concrete slab. The shoulder may consist of a 2 ft widened edge.

JOINT SPACING AND DETAILS

Construction joint spacing in PCC pavements should not exceed 15 ft in either the longitudinal or transverse direction. The depth of sawcut should be a minimum of 1/4 of the slab depth if utilizing a conventional saw or 1 in. when using an early entry saw (early entry sawing is recommended). The width of the joint will be a function of the sealant chosen to seal the joint. It is recommended that a joint seal be utilized to reduce the introduction of incompressible material into the joint.

It is recommended that dowel bars be used to provide load transfer and reduce differential movement (or faulting) across transverse joints. Dowels should be smooth #9 bars (Grade 60 steel) spaced 12 in. on center with an embedment length of at least 8 in.

Tie bars should be used to tie longitudinal joints within the pavement lanes and at the shoulder, if used. Tie bars should be deformed #4 bars at a minimum (Grade 60 steel) spaced 36 in. on center with a minimum length of 30 in.

Isolation joints must be used around fixed structures including light standard foundations and drainage inlets to offset the effects of differential horizontal and vertical movements. Premolded joint fillers should be used around the fixed structures prior to placing the concrete pavement to prevent bonding of the slab to the structure and should extend through the depth of the slab but slightly recessed from the pavement surface to provide room for the joint sealant.

SUGGESTED PAVEMENT DETAILS

Suggested details (see Figure 28 of the Attachments) that can be utilized for construction are:

- CPCD-94, Concrete Pavement Details, Contraction Design (CPCD) – standard for plain jointed concrete pavement and covers pavement thickness from 8 to 15 in.
- JS-94, Concrete Paving Details, Joint Seals - specifies joint sealing requirements for concrete pavement.

MISCELLANEOUS PAVEMENT RELATED AND OTHER CONSIDERATIONS

Drainage Considerations

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surface and subsurface drainage. Insufficient drainage which allows saturation of the pavement subgrade and/or the supporting granular pavement materials will greatly reduce the performance and service life of the pavement systems.

Surface and subsurface drainage considerations crucial to the performance of pavements at this site include (but are not limited to) the following:

- Any known natural or man-made subsurface seepage at the site which may occur at sufficiently shallow depths as to influence moisture contents within the subgrade should be intercepted by drainage ditches or below grade French drains.
- Final site grading should eliminate isolated depressions adjacent to curbs, which may allow surface water to pond and infiltrate into the underlying soils. Curbs should be installed to a sufficient depth to reduce infiltration of water beneath the curbs and into the pavement base materials.
- Pavement surfaces should be maintained to help reduce surface ponding and to provide rapid sealing of any developing cracks. These measures will help reduce infiltration of surface water downward through the pavement section.

Utilities

Our experience indicates that significant settlement of backfill can occur in utility trenches, particularly when trenches are deep, when backfill materials are placed in thick lifts with insufficient compaction, and when water can access and infiltrate the trench backfill materials. The potential for water to access the backfill is increased where water can infiltrate flexible base materials due to insufficient penetration of curbs, and at sites where geological features can influence water migration into utility trenches (such as fractures within a rock mass or at contacts between rock and clay formations). It is our belief that another factor which can significantly impact settlement is the migration of fines within the backfill into the open voids in the underlying free-draining bedding material.

To reduce the potential for settlement in utility trenches, we recommend that consideration be given to the following:

- All backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized and all backfilling procedures should be tested and documented in accordance with the *Select Fill* section of this report.
- Consideration should be given to wrapping free-draining bedding gravels with a geotextile fabric (similar to Mirafi 140N) to reduce the infiltration and loss of fines from backfill material into the interstitial voids in bedding materials.

Curb and Gutter

It is good practice to construct curbs such that the depth of the curb extends through the entire depth of the granular base material to act as a protective barrier against the infiltration of water into the granular base. Pavements that do not have this protective barrier to moisture tend to develop longitudinal cracks 1 to 2 ft from the edge of the pavement. Once these cracks develop, further degradation and weakening of the underlying granular base may occur due to water seepage through the cracks. Similar results can be achieved by installing PVC sheeting the full depth of the pavement section and into the natural subgrade, behind the curb. Additionally, consideration can be given to installing a horizontal moisture barrier by way of a sidewalk or other impermeable layer immediately adjacent to the back of the curbs.

Pavement Maintenance

Regular pavement maintenance is critical in maintaining pavement performance over a period of several years. All cracks that develop in asphalt pavements should be regularly sealed. Areas of moderate to severe fatigue cracking (also known as alligator cracking) should be sawcut and removed. The underlying base should be checked for contamination or loss of support and any insufficiencies fixed or removed and the entire area patched. All cracks that develop in concrete pavements should be routed and sealed regularly. Joints in concrete pavements should be maintained to reduce the influx of incompressible materials that restrain joint movement and cause spalling and/or cracking. Other typical TxDOT or City of San Antonio maintenance techniques should be followed as required.

Construction Traffic

Construction traffic on prepared subgrade, granular base or asphalt treated base (black base) should be restricted as much as possible until the protective asphalt surface pavement is applied. Significant damage to the underlying layers resulting in weakening may occur if heavily loaded vehicles are allowed to use these areas.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

As presented in the attachment to this report, *Important Information About Your Geotechnical Engineering Report*, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.

Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, Raba Kistner, is retained to perform construction observation and testing services during the construction of the project. This is because:

- RKCI has an intimate understanding of the geotechnical engineering report's findings and recommendations. RKCI understands how the report should be interpreted and can provide such interpretations on site, on the client's behalf.
- RKCI knows what subsurface conditions are anticipated at the site.
- RKCI is familiar with the goals of the owner and project design professionals, having worked with them in the development of the geotechnical workscope. This enables RKCI to suggest remedial measures (when needed) which help meet the owner's and the design teams' requirements.
- RKCI has a vested interest in client satisfaction, and thus assigns qualified personnel whose principal concern is client satisfaction. This concern is exhibited by the manner in

which contractors' work is tested, evaluated and reported, and in selection of alternative approaches when such may become necessary.

- RKCI cannot be held accountable for problems which result due to misinterpretation of our findings or recommendations when we are not on hand to provide the interpretation which is required.

BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that RKCI and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. RKCI looks forward to the opportunity to provide continued support on this project, and would welcome the opportunity to meet with the Project Team to develop both a scope and budget for these services.

* * * * * * * * * * * * * * * * * *

ATTACHMENTS

RABA KISTNER CONSULTANTS

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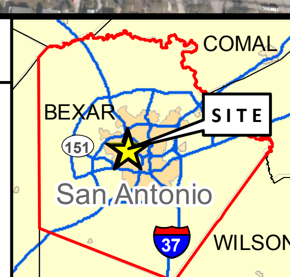
TBPE Firm Number 3257

SOURCE: 2013 Aerial Photograph Provided by the City of San Antonio (COSA)

BORING LOCATION MAP

SEELING CHANNEL IMPROVEMENTS, PHASE II
SAN ANTONIO, TEXAS



PROJECT No.: ASA14-003-00

ISSUE DATE:	06/25/2014
DRAWN BY:	CCL
CHECKED BY:	YLG
REVIEWED BY:	TIP

FIGURE

1

NOTE: This Drawing is Provided for Illustration Only, May Not be to Scale and is Not Suitable for Design or Construction Purposes

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DRILLING LOG

WinCore Version 3.0	County	Hole B-101	District
	Highway	Structure	Date 02/10/14
	CSJ	Station	Grnd. Elev. 690.66 ft
		Offset	GW Elev. N/A

Elev. (ft)	Texas Cone Penetrometer	Strata Description	Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	Additional Remarks
		CLAY, Stiff, Brown (CH)			13	57	41		
685.7 5	13 (6) 11 (6)	CLAY, Stiff to Very Hard, Tan (CH)							
								102	UC = 0.96 tsf, Dry Unit Weight = 102 pcf
10	11 (6) 15 (6)								
					21	58	42		
15	13 (6) 17 (6)								
					17				
20	28 (6) 32 (6)								
								113	UC = 2.63 tsf, Dry Unit Weight = 113 pcf
25	35 (6) 50 (5)								additional TCP test 15/1"
					18				
30	50 (3) 50 (1.5)								
					16				
659.7		SHALE, Very Hard, Gray, with clayey seams							
35	50 (0.25) 50 (0.25)				13				
40	50 (0.5) 50 (0)								

Remarks:

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

Driller: **Logger:**

Organization: Raba-Kistner Consultants, Inc.

FIGURE: 2a

DRILLING LOG

WinCore	County		Hole	B-101	District	
Version 3.0	Highway		Structure		Date	02/10/14
	CSJ		Station		Grnd. Elev.	690.66 ft
			Offset		GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	Properties: MC	Properties: LL	Properties: PI	Properties: Wet Den. (pcf)	Additional Remarks
			SHALE, Very Hard, Gray, with clayey seams			13				
45		50 (0.75) 50 (0)				15				
50		50 (0.75) 50 (0.25)				14				
635.7 55		50 (0.25) 50 (0)								
60										
65										
70										
75										
80										

Remarks:

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

Driller: **Logger:**

Organization: Raba-Kistner Consultants, Inc.

FIGURE: 2b

DRILLING LOG

WinCore Version 3.0	County	Hole B-102	District
	Highway	Structure	Date 01/28/14
	CSJ	Station	Grnd. Elev. 690.21 ft
		Offset	GW Elev. N/A

Elev. (ft)	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	Additional Remarks
		CLAY, Soft, Brown (CH)			21	68	52		
687.2		CLAY, Soft to Very Stiff, Tan (CH)							
5	5 (6) 5 (6)								
								129	UC = 1.27 tsf, Dry Unit Weight = 106 pcf
10	9 (6) 15 (6)								
					21	65	49		
15	14 (6) 20 (6)								
					17				
20	26 (6) 28 (6)								DRILLER'S NOTE: WATER encountered at 20 ft
					18				
665.2 25	43 (6) 50 (5.25)	SHALE, Hard to Very Hard, Gray, with clayey seams							additional TCP test 7/.5"
					13				
30	50 (5.5) 50 (3.5)								
					26				
35	50 (1) 50 (1.75)				31				
40	50 (0.5) 50 (0.25)								

Remarks:

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

Driller: **Logger:**

Organization: Raba-Kistner Consultants, Inc.

FIGURE: 3a

DRILLING LOG

	County	Hole	B-102	District	
WinCore	Highway	Structure		Date	01/28/14
Version 3.0	CSJ	Station		Grnd. Elev.	690.21 ft
		Offset		GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	Properties: MC	Properties: LL	Properties: PI	Properties: Wet Den. (pcf)	Additional Remarks
			SHALE, Hard to Very Hard, Gray, with clayey seams			19				
45		50 (0.25) 50 (0.5)				20				
50		50 (0) 50 (0)				19				
635.2 55		50 (1) 50 (0.25)								
60										
65										
70										
75										
80										

Remarks:

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

Driller: **Logger:**

Organization: Raba-Kistner Consultants, Inc.

FIGURE: 3b

DRILLING LOG

WinCore
Version 3.0

County		Hole	B-103	District	
Highway		Structure		Date	02/10/14
CSJ		Station		Grnd. Elev.	686.54 ft
		Offset		GW Elev.	N/A

Elev. (ft)	Depth (ft)	Texas Cone Penetrometer	Strata Description	Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	Properties: MC	Properties: LL	Properties: PI	Properties: Wet Den. (pcf)	Additional Remarks
			CLAY, Soft, Brown (CH)			24	61	44		
681.5	5	4 (6) 5 (6)	CLAY, Soft to Hard, Tan (CH)							
									126	UC = 0.45 tsf, Dry Unit Weight = 101 pcf
	10	10 (6) 12 (6)								
						23				
	15	16 (6) 17 (6)								
						21	57	42		
	20	19 (6) 24 (6)								
									135	UC = 4.36 tsf, Dry Unit Weight = 115 pcf
	25	50 (6) 50 (5.5)								
660.5			SHALE, Very Hard, Gray, with clayey seams			19				
	30	50 (2) 50 (1.25)				17				
	35	50 (0) 50 (0)				22				
	40	50 (0.25) 50 (0)								

Remarks:

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

Driller: Logger:

Organization: Raba-Kistner Consultants, Inc.

FIGURE: 4a

DRILLING LOG

WinCore
Version 3.0

County		Hole	B-103
Highway		Structure	
CSJ		Station	
		Offset	
District			
Date	02/10/14		
Grnd. Elev.	686.54 ft		
GW Elev.	N/A		

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	Properties: MC	Properties: LL	Properties: PI	Properties: Wet Den. (pcf)	Additional Remarks
			SHALE, Very Hard, Gray, with clayey seams			20				
45		50 (0) 50 (0)				11				
50		50 (0.75) 50 (0.5)				15				
631.5 55		50 (0.25) 50 (0)								
60										
65										
70										
75										
80										

Remarks:

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

Driler: Logger:

Organization: Raba-Kistner Consultants, Inc.

FIGURE: 4b

DRILLING LOG

WinCore	County	Hole	B-104	District	
Version 3.0	Highway	Structure		Date	02/11/14
	CSJ	Station		Grnd. Elev.	687.67 ft
		Offset		GW Elev.	N/A

Elev. (ft)	Texas Cone Penetrometer	Strata Description	Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	Additional Remarks
		CLAY, Soft, Brown (CH)			24	64	45		
5	7 (6) 7 (6)								
681.7		CLAY, Soft to Hard, Tan (CH)			22	64	46		
10	9 (6) 11 (6)								
								131	UC = 1.59 tsf, Dry Unit Weight = 108 pcf
15	11 (6) 14 (6)								
					20	54	37		
20	41 (6) 45 (6)								
					18				
25	35 (6) 50 (5)								additional TCP test 15/1"
661.7		SHALE, Very Hard, Gray, with clayey seams							DRILLER'S NOTE: WATER encountered at 26 ft
					21				
30	50 (0.75) 50 (0.25)				18				
35	50 (0.5) 50 (0.25)				18				
40	50 (0.25) 50 (0.25)								

Remarks:

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

Driller: Logger:

Organization: Raba-Kistner Consultants, Inc.

FIGURE: 5a

DRILLING LOG

	County	Hole	B-104	District	
WinCore	Highway	Structure		Date	02/11/14
Version 3.0	CSJ	Station		Grnd. Elev.	687.67 ft
		Offset		GW Elev.	N/A

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	Properties: MC	Properties: LL	Properties: PI	Properties: Wet Den. (pcf)	Additional Remarks
			SHALE, Very Hard, Gray, with clayey seams			23				
45		50 (0) 50 (0)				19				
50		50 (0) 50 (0)				13				
632.7 55		50 (0) 50 (0.25)								
60										
65										
70										
75										
80										

Remarks:

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

Driller: **Logger:**

Organization: Raba-Kistner Consultants, Inc.

FIGURE: 5b

LOG OF BORING NO. RW-101

Seeling Channel Improvements, Phase II
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.45852; W 98.54449

SURFACE ELEVATION: 689.36 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	CLAY, Hard, Brown, with gravel	43			
		41		43	
5		50/10"			
	CLAY, Hard, Tan and Gray, with calcareous deposits and ferrous staining	40			
10		35		52	
15					
	- with gypsum crystals below 18 ft	43			
20					
25		50/9"			
		50/5"			
30	CLAYSHALE, Hard, Gray				
		Ref/3"			
35	Boring Terminated				

SHEAR STRENGTH, TONS/FT^2: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10, 20, 30, 40, 50, 60, 70, 80

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 33.8 ft
DATE DRILLED: 1/22/2014
DEPTH TO WATER: DRY
DATE MEASURED: 1/22/2014
PROJ. No.: ASA14-003-00
FIGURE: 6

LOG OF BORING NO. RW-102

Seeling Channel Improvements, Phase II
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.45674; W 98.54251

SURFACE ELEVATION: 686.54 ft

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	CLAY, Stiff to Very Stiff, Brown, with gravel	13		41	
		19			
4	CLAY, Very Stiff to Hard, Tan and Gray, with calcareous deposits and ferrous staining	22		46	
		28			
		37			
		37		43	
	- with gypsum crystals below 19 ft	50/12"			
		50/9"			
		50/6"			
30	CLAYSHALE, Hard, Gray				
		Ref/6"			
34	Boring Terminated				

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	34.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ASA14-003-00
DATE DRILLED:	1/23/2014	**DATE MEASURED:**	1/23/2014	**FIGURE:**	7

LOG OF BORING NO. P-101

Seeling Channel Improvements, Phase II
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.45873; W 98.54619

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

SURFACE ELEVATION: 692.05 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	ASHPALT (6 in.)				
	BASE MATERIAL (8 in.)				
	CLAY, Stiff to Very Stiff, Brown	13		48	
		17			
	CLAY, Very Stiff, Tan				
5		19			
		18			
10		18			
		23			
15	Boring Terminated				
20					
25					
30					
35					

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 15.0 ft
DATE DRILLED: 2/4/2014

DEPTH TO WATER: DRY
DATE MEASURED: 2/4/2014

PROJ. No.: ASA14-003-00
FIGURE: 8

LOG OF BORING NO. P-102

Seeling Channel Improvements, Phase II
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.45712; W 98.54569

SURFACE ELEVATION: 693.61 ft

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	ASHPALT (2 in.)				
	BASE MATERIAL (4 in.)				
	CLAY, Stiff, Brown	11		52	
		12			
5		13			
	CLAY, Stiff to Very Stiff, Tan	18			
10		14			
		20			
15	Boring Terminated				

DEPTH, FT: 5, 10, 15, 20, 25, 30, 35

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 15.0 ft	**DEPTH TO WATER:** DRY	**PROJ. No.:** ASA14-003-00	
DATE DRILLED: 2/7/2014	**DATE MEASURED:** 2/7/2014	**FIGURE:** 9	

LOG OF BORING NO. P-103

Seeling Channel Improvements, Phase II
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.45922; W 98.54333

SURFACE ELEVATION: 692.1 ft

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	ASHPALT (3 in.)				
	BASE MATERIAL (9 in.)	8			
	CLAY, Stiff, Brown				
	CLAY, Very Stiff to Hard, Tan	18			
5		24		42	
		25			
		26			
14		38			
15	Boring Terminated				

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ASA14-003-00
DATE DRILLED:	2/4/2014	**DATE MEASURED:**	2/4/2014	**FIGURE:**	10

LOG OF BORING NO. P-104

Seeling Channel Improvements, Phase II
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.45863; W 98.54252

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

SURFACE ELEVATION: 691.45 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	ASHPALT (3 in.)				
	BASE MATERIAL (7 in.)				
	CLAY, Very Stiff, Brown, with gravel	18		52	
		22			
	CLAY, Hard, Tan				
5		40			
		48			
		37			
10					
		32			
15	Boring Terminated				
20					
25					
30					
35					

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ASA14-003-00
DATE DRILLED:	2/7/2014	**DATE MEASURED:**	2/7/2014	**FIGURE:**	11

LOG OF BORING NO. P-105

Seeling Channel Improvements, Phase II
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.45747; W 98.54346

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

SURFACE ELEVATION: 689.24 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	ASHPALT (4 in.)				
	BASE MATERIAL (10 in.)				
	CLAY, Stiff, Brown	11			
		13		45	
5		13			
	CLAY, Very Stiff, Tan	18			
		15			
10					
		23			
15	Boring Terminated				
20					
25					
30					
35					

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 15.0 ft
DATE DRILLED: 2/7/2014
DEPTH TO WATER: DRY
DATE MEASURED: 2/7/2014
PROJ. No.: ASA14-003-00
FIGURE: 12

LOG OF BORING NO. P-106

Seeling Channel Improvements, Phase II
San Antonio, Texas

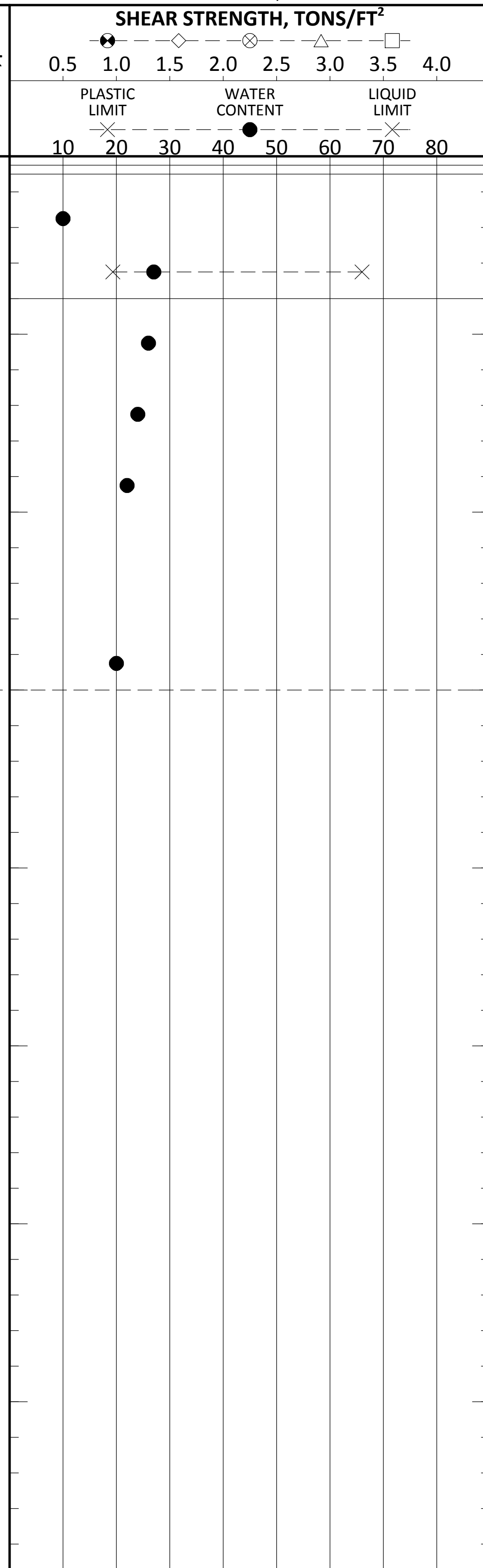
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.45659; W 98.54345

SURFACE ELEVATION: 691.45 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	ASHPALT (3 in.)				
	BASE MATERIAL (3 in.)				
	CLAY, Very Stiff, Brown	25			
		22		47	
4	CLAY, Very Stiff to Hard, Tan	18			
		23			
		25			
		39			
15	Boring Terminated				



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ASA14-003-00
DATE DRILLED:	2/5/2014	**DATE MEASURED:**	2/5/2014	**FIGURE:**	13

LOG OF BORING NO. P-107

Seeling Channel Improvements, Phase II
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.45697; W 98.54123

SHEAR STRENGTH, TONS/FT²: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	SURFACE ELEVATION: 686.28 ft				
	ASHPALT (4 in.)				
	BASE MATERIAL (3 in.)				
	CLAY, Stiff to Very Stiff, Brown	17			
		16		44	
5		13			
	CLAY, Very Stiff, Tan	16			
		19			
10					
		21			
15	Boring Terminated				
20					
25					
30					
35					

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ASA14-003-00
DATE DRILLED:	2/7/2014	**DATE MEASURED:**	2/7/2014	**FIGURE:**	14

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

LOG OF BORING NO. P-108

Seeling Channel Improvements, Phase II
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.45567; W 98.54132

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	SURFACE ELEVATION: 689.27 ft				
	ASHPALT (6 in.)				
	BASE MATERIAL (10 in.)				
	CLAY, Stiff to Very Stiff, Brown	12		46	
		16			
	CLAY, Very Stiff to Hard, Tan				
5		20			
		27			
		24			
10					
		40			
15	Boring Terminated				
20					
25					
30					
35					

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 15.0 ft
DATE DRILLED: 2/5/2014
DEPTH TO WATER: DRY
DATE MEASURED: 2/5/2014
PROJ. No.: ASA14-003-00
FIGURE: 15

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

- CALCAREOUS
- CALICHE
- CLAY
- CLAYEY
- GRAVEL
- GRAVELLY
- PEAT
- SAND
- SANDY
- SILT
- SILTY
- FILL

ROCK TERMS

- CHALK
- CLAYSTONE
- CLAY-SHALE
- CONGLOMERATE
- DOLOMITE
- IGNEOUS
- LIMESTONE
- MARL
- METAMORPHIC
- SANDSTONE
- SHALE
- SILTSTONE

OTHER

- ASPHALT
- BASE
- CONCRETE/CEMENT
- BRICKS / PAVERS
- WASTE
- NO INFORMATION

WELL CONSTRUCTION AND PLUGGING MATERIALS

- BLANK PIPE
- SCREEN
- BENTONITE
- CEMENT GROUT
- BENTONITE & CUTTINGS
- CONCRETE/CEMENT
- CUTTINGS
- GRAVEL
- SAND
- VOLCLAY

SAMPLE TYPES

- AIR ROTARY
- GRAB SAMPLE
- CORE
- GEOPROBE SAMPLER
- ROTOSONIC -DAMAGED
- MUD ROTARY
- NO RECOVERY
- NX CORE
- PITCHER
- ROTOSONIC -INTACT
- SHELBY TUBE
- SPLIT BARREL
- SPLIT SPOON
- TEXAS CONE PENETROMETER
- DISTURBED

STRENGTH TEST TYPES

- POCKET PENETROMETER
- TORVANE
- UNCONFINED COMPRESSION
- TRIAXIAL COMPRESSION UNCONSOLIDATED-UNDRAINED
- TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED

NOTE: VALUES SYMBOLIZED ON BORING LOGS REPRESENT SHEAR STRENGTHS UNLESS OTHERWISE NOTED

PROJECT NO. ASA14-003-00

RABAKISTNER

REVISED 04/2012

FIGURE 16a

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

Terms used in this report to describe soils with regard to their consistency or conditions are in general accordance with the discussion presented in Article 45 of SOILS MECHANICS IN ENGINEERING PRACTICE, Terzaghi and Peck, John Wiley & Sons, Inc., 1967, using the most reliable information available from the field and laboratory investigations. Terms used for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in American Society for Testing and Materials D2487-06 and D2488-00, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics; 2005.

The depths shown on the boring logs are not exact, and have been estimated to the nearest half-foot. Depth measurements may be presented in a manner that implies greater precision in depth measurement, i.e 6.71 meters. The reader should understand and interpret this information only within the stated half-foot tolerance on depth measurements.

RELATIVE DENSITY

Penetration Resistance Blows per ft	Relative Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
> 50	Very Dense

COHESIVE STRENGTH

Resistance Blows per ft	Consistency	Cohesion TSF
0 - 2	Very Soft	0 - 0.125
2 - 4	Soft	0.125 - 0.25
4 - 8	Firm	0.25 - 0.5
8 - 15	Stiff	0.5 - 1.0
15 - 30	Very Stiff	1.0 - 2.0
> 30	Hard	> 2.0

PLASTICITY

Plasticity Index	Degree of Plasticity
0 - 5	None
5 - 10	Low
10 - 20	Moderate
20 - 40	Plastic
> 40	Highly Plastic

ABBREVIATIONS

B = Benzene
T = Toluene
E = Ethylbenzene
X = Total Xylenes
BTEX = Total BTEX
TPH = Total Petroleum Hydrocarbons
ND = Not Detected
NA = Not Analyzed
NR = Not Recorded/No Recovery
OVA = Organic Vapor Analyzer
ppm = Parts Per Million

Qam, Qas, Qal = Quaternary Alluvium
Qat = Low Terrace Deposits
Qbc = Beaumont Formation
Qt = Fluviatile Terrace Deposits
Qao = Seymour Formation
Qle = Leona Formation
Q-Tu = Uvalde Gravel
Ewi = Wilcox Formation
Emi = Midway Group
Mc = Catahoula Formation
EI = Laredo Formation
Kknm = Navarro Group and Marlbrook Marl
Kpg = Pecan Gap Chalk
Kau = Austin Chalk

Kef = Eagle Ford Shale
Kbu = Buda Limestone
Kdr = Del Rio Clay
Kft = Fort Terrett Member
Kgt = Georgetown Formation
Kep = Person Formation
Kek = Kainer Formation
Kes = Escondido Formation
Kew = Walnut Formation
Kgr = Glen Rose Formation
Kgru = Upper Glen Rose Formation
Kgrl = Lower Glen Rose Formation
Kh = Hensell Sand

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy.
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 type.
Interlayered	Soil sample composed of alternating layers of different soil type.
Intermixed	Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of carbonate.
Carbonate	Having more than 50% carbonate content.

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

Cohesive soil samples are to be collected using three-inch thin-walled tubes in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587) and granular soil samples are to be collected using two-inch split-barrel samplers in general accordance with the Standard Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). Cohesive soil samples may be extruded on-site when appropriate handling and storage techniques maintain sample integrity and moisture content.

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-in.-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARREL SAMPLER DRIVING RECORD

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

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

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Seeling Channel Improvements, Phase II
San Antonio, Texas

FILE NAME: ASA14-003-00.GPJ 8/12/2014

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-101	0.0 to 5.0		13	57	16	41	CH				
	5.0 to 6.5	24									
	8.0 to 10.0		24					102		0.96	UC
	10.0 to 11.8	26									
	11.8 to 15.0		21	58	16	42	CH				
	15.0 to 16.5	30									
	16.5 to 20.0		17								
	20.0 to 21.3	60									
	23.0 to 25.0		18					113		2.63	UC
	25.0 to 26.3	100									
	26.3 to 30.0		18								
	30.0 to 30.7	100/4.5"									
	30.7 to 35.0		16								
	35.0 to 35.1	100/0.5"									
	35.1 to 40.0		13								
	40.0 to 40.1	100/0.5"									
	40.1 to 45.0		13								
	45.0 to 45.2	100/0.75"									
	45.2 to 50.0		15								
	50.0 to 50.2	100/1"									
	50.2 to 55.0		14								
	55.0 to 55.1	100/0.25"									
B-102	0.0 to 5.0		21	68	16	52	CH				
	5.0 to 7.0	11									
	8.0 to 10.0		22					106		1.27	UC
	10.0 to 12.0	24									
	12.0 to 15.0		21	65	16	49	CH				
	15.0 to 16.5	34									
	16.5 to 20.0		17								
	20.0 to 21.3	54									
	21.3 to 25.0		18								
	25.0 to 25.2	100/11.75"									
	28.0 to 28.5		13								
	30.0 to 30.9	100/9"									
	30.9 to 35.0		26								
	35.0 to 35.3	100/2.75"									
	35.3 to 40.0		31								
	40.0 to 40.1	100/0.75"									
	40.1 to 45.0		19								

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

PROJECT NO. ASA14-003-00

RABA KISTNER

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Seeling Channel Improvements, Phase II
San Antonio, Texas

FILE NAME: ASA14-003-00.GPJ 8/12/2014

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-102	45.0 to 45.1	100/0.75"									
	45.1 to 50.0		20								
	50.0 to 50.1	100/0"									
	50.1 to 55.0		19								
	55.0 to 55.2	100/1.25"									
B-103	0.0 to 5.0		24	61	17	44	CH				
	5.0 to 7.0	9									
	8.0 to 10.0		25					101		0.45	UC
	10.0 to 12.0	22									
	12.0 to 15.0		23								
	15.0 to 16.5	33									
	16.5 to 20.0		21	57	15	42	CH				
	20.0 to 21.4	43									
	23.0 to 25.0		17					115		4.36	UC
	25.0 to 26.3	100/11.5"									
	26.3 to 30.0		19								
	30.0 to 30.4	100/3.25"									
	30.4 to 35.0		17								
	35.0 to 35.0	100/0"									
	35.0 to 40.0		22								
	40.0 to 40.1	100/0.25"									
	40.1 to 45.0		20								
	45.0 to 45.1	100/0"									
	45.1 to 50.0		11								
	50.0 to 50.2	100/1.25"									
	50.2 to 55.0		15								
	55.0 to 55.1	100/0.25"									
B-104	0.0 to 5.0		24	64	19	45	CH				
	5.0 to 7.0	14									
	7.0 to 10.0		22	64	18	46	CH				
	10.0 to 11.8	20									
	13.0 to 15.0		21					108		1.59	UC
	15.0 to 16.7	25									
	16.7 to 20.0		20	54	17	37	CH				
	20.0 to 21.4	86									
	21.4 to 25.0		18								
	25.0 to 26.3	100									
	28.0 to 30.0		21								
	30.0 to 30.2	100/1"									

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

PROJECT NO. ASA14-003-00

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Seeling Channel Improvements, Phase II
San Antonio, Texas

FILE NAME: ASA14-003-00.GPJ 8/12/2014

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-104	30.2 to 35.0		18								
	35.0 to 35.1	100/0.75"									
	35.1 to 40.0		18								
	40.0 to 40.1	100/0.5"									
	40.1 to 45.0		23								
	45.0 to 45.0	100/0"									
	45.1 to 50.1		19								
	50.0 to 50.1	100/0"									
	50.1 to 55.0		13								
	55.0 to 55.1	100/0.25"									
RW-101	0.0 to 1.5	43	13								
	2.5 to 4.0	41	15	61	18	43	CH				
	4.5 to 5.8	50/10"	15								
	6.5 to 8.0	40	16								
	8.5 to 10.0	35	21	69	17	52	CH				
	13.0 to 15.0		19							1.88	PP
	18.5 to 20.0	43	18								
	23.5 to 24.8	50/9"	19								
	28.5 to 29.4	50/5"	17								
	33.5 to 33.8	Ref/3"	13								
RW-102	0.0 to 1.5	13	20	59	18	41	CH				
	2.5 to 4.0	19	25								
	4.5 to 5.8	22	21	63	17	46	CH				
	6.5 to 8.0	28	21								
	8.5 to 10.0	37	22								
	13.5 to 15.0	37	21	60	17	43	CH				
	18.5 to 20.0	50/12"	19								
	23.5 to 24.8	50/9"	18								
	28.5 to 29.5	50/6"	18								
	33.5 to 34.0	Ref/6"	21								
P-101	1.0 to 2.5	13	25	66	18	48	CH				
	2.5 to 4.0	17	25								
	4.5 to 6.0	19	51								
	6.5 to 8.0	18	25								
	8.5 to 10.0	18	24								
	13.5 to 15.0	23	20								
P-102	1.0 to 2.5	11	32	75	23	52	CH				
	2.5 to 4.0	12	30								
	4.5 to 6.0	13	29								

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Seeling Channel Improvements, Phase II
San Antonio, Texas

FILE NAME: ASA14-003-00.GPJ 8/12/2014

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
P-102	6.5 to 8.0	18	25								
	8.5 to 10.0	14	26								
	13.5 to 15.0	20	22								
P-103	1.0 to 2.5	8	24								
	2.5 to 4.0	18	26								
	4.5 to 6.0	24	21	65	23	42	CH				
	6.5 to 8.0	25	21								
	8.5 to 10.0	26	22								
	13.5 to 15.0	38	20								
P-104	1.0 to 2.5	18	24	70	18	52	CH				
	2.5 to 4.0	22	16								
	4.5 to 6.0	40	18								
	6.5 to 8.0	48	16								
	8.5 to 10.0	37	21								
	13.5 to 15.0	32	20								
P-105	1.0 to 2.5	11	9								
	2.5 to 4.0	13	26	64	19	45	CH				
	4.5 to 6.0	13	28								
	6.5 to 8.0	18	17								
	8.5 to 10.0	15	22								
	13.5 to 15.0	23	22								
P-106	1.0 to 2.5	25	10								
	2.5 to 4.0	22	27	66	19	47	CH				
	4.5 to 6.0	18	26								
	6.5 to 8.0	23	24								
	8.5 to 10.0	25	22								
	13.5 to 15.0	39	20								
P-107	1.0 to 2.5	17	24								
	2.5 to 4.0	16	27	62	18	44	CH				
	4.5 to 6.0	13	26								
	6.5 to 8.0	16	24								
	8.5 to 10.0	19	22								
	13.5 to 15.0	21	23								
P-108	1.0 to 2.5	12	27	66	20	46	CH				
	2.5 to 4.0	16	27								
	4.5 to 6.0	20	27								
	6.5 to 8.0	27	23								
	8.5 to 10.0	24	23								
	13.5 to 15.0	40	25								

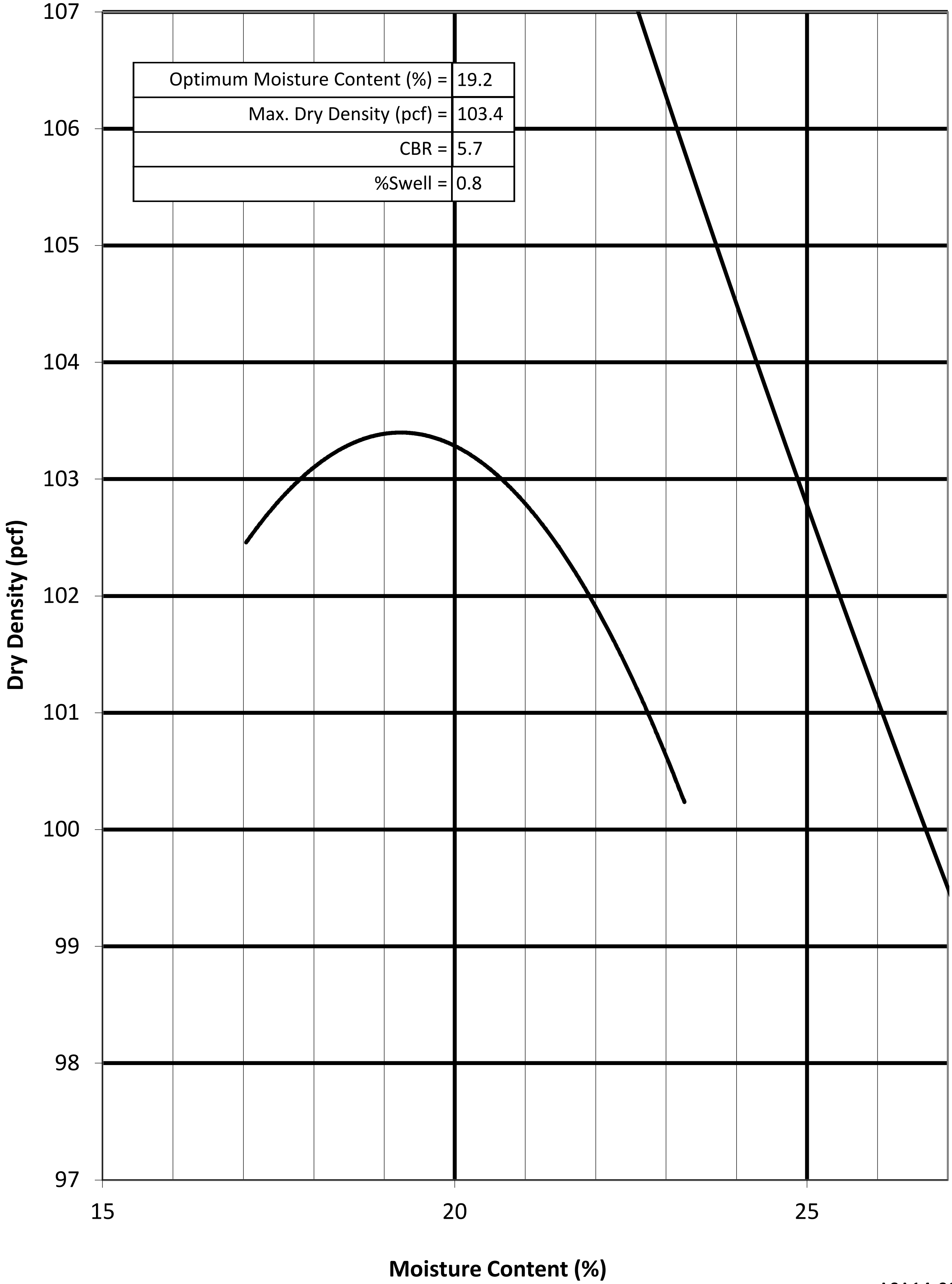
PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

PROJECT NO. ASA14-003-00

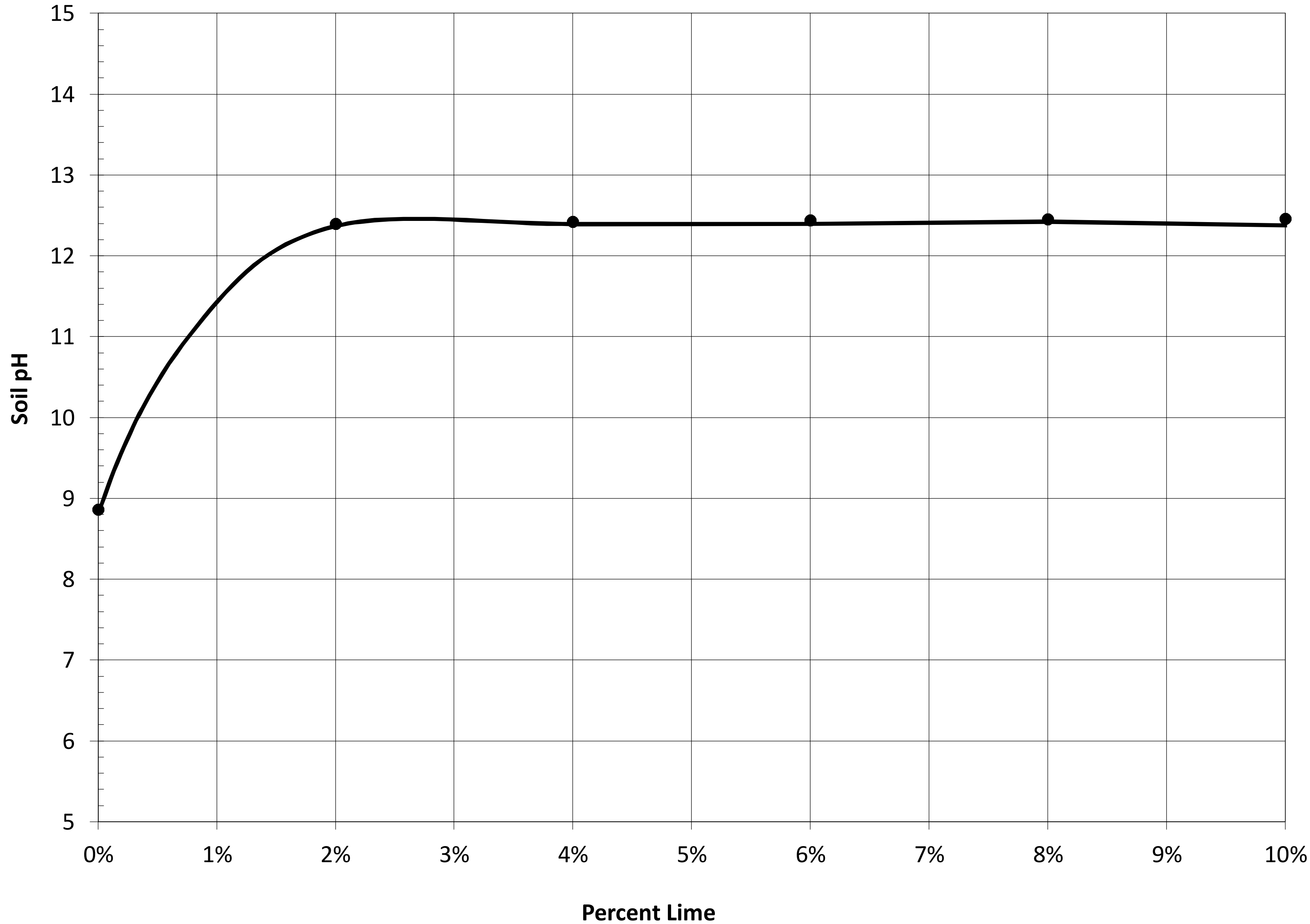
MOISTURE DENSITY RELATIONSHIP CURVE - CBR I

Seeling Channel Improvements, Phase II



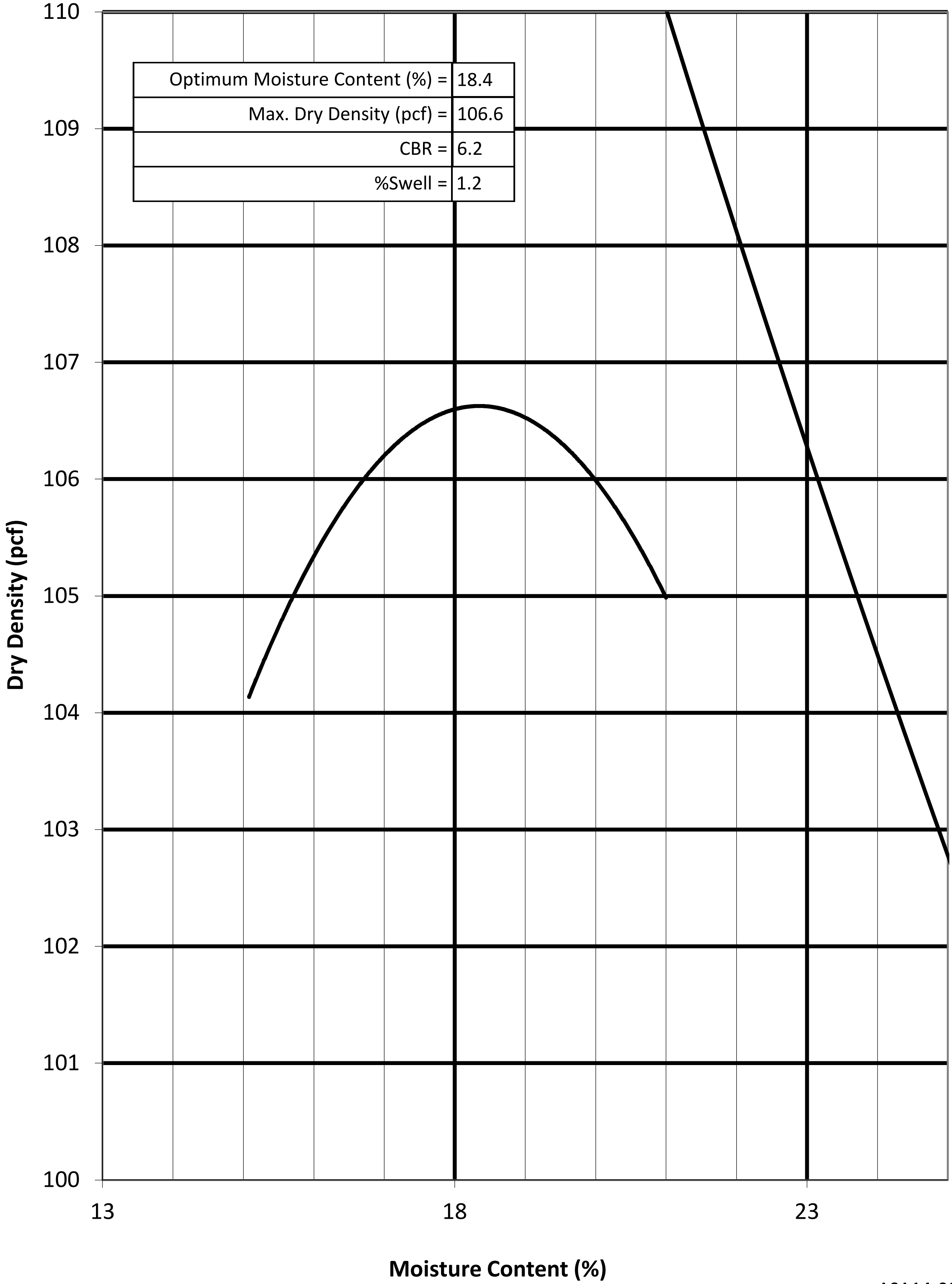
pH-LIME SERIES CURVE - CBR I

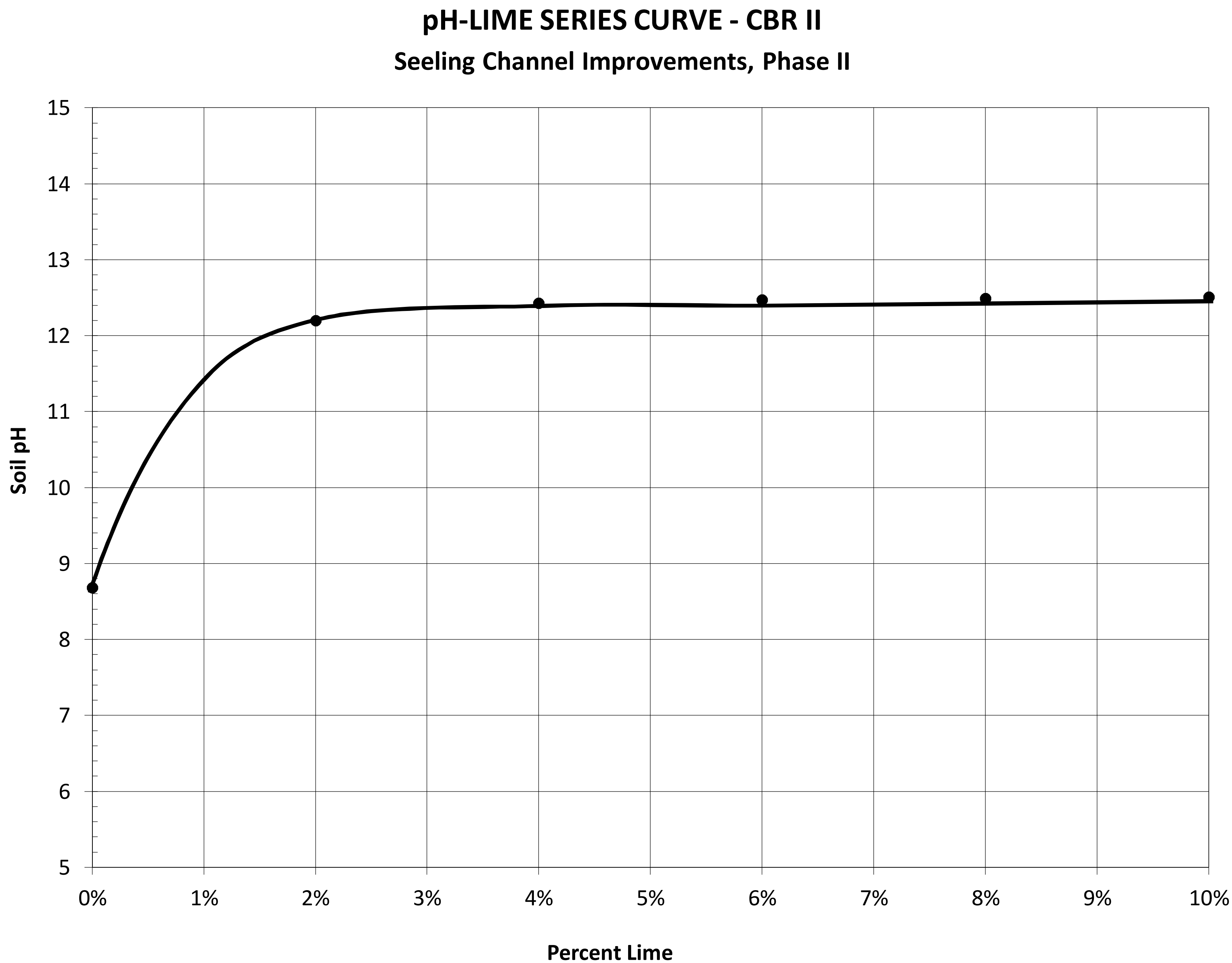
Seeling Channel Improvements, Phase II

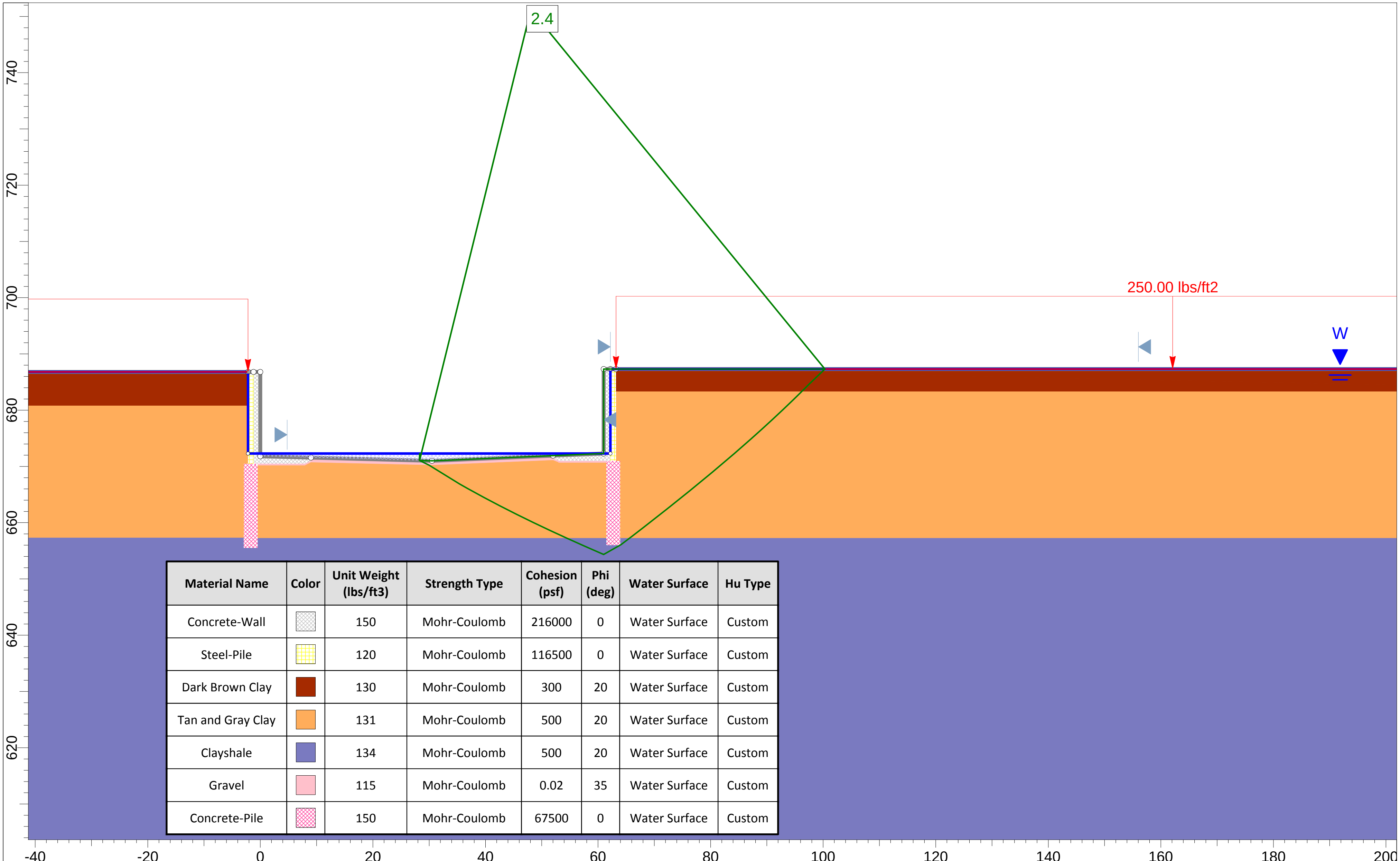


MOISTURE DENSITY RELATIONSHIP CURVE - CBR II

Seeling Channel Improvements, Phase II




pH-LIME SERIES CURVE - CBR II
Seeling Channel Improvements, Phase II
Soil pH
15
14
13
12
11
10
9
8
7
6
5
0%
1%
2%
3%
4%
5%
6%
7%
8%
9%
10%
Percent Lime



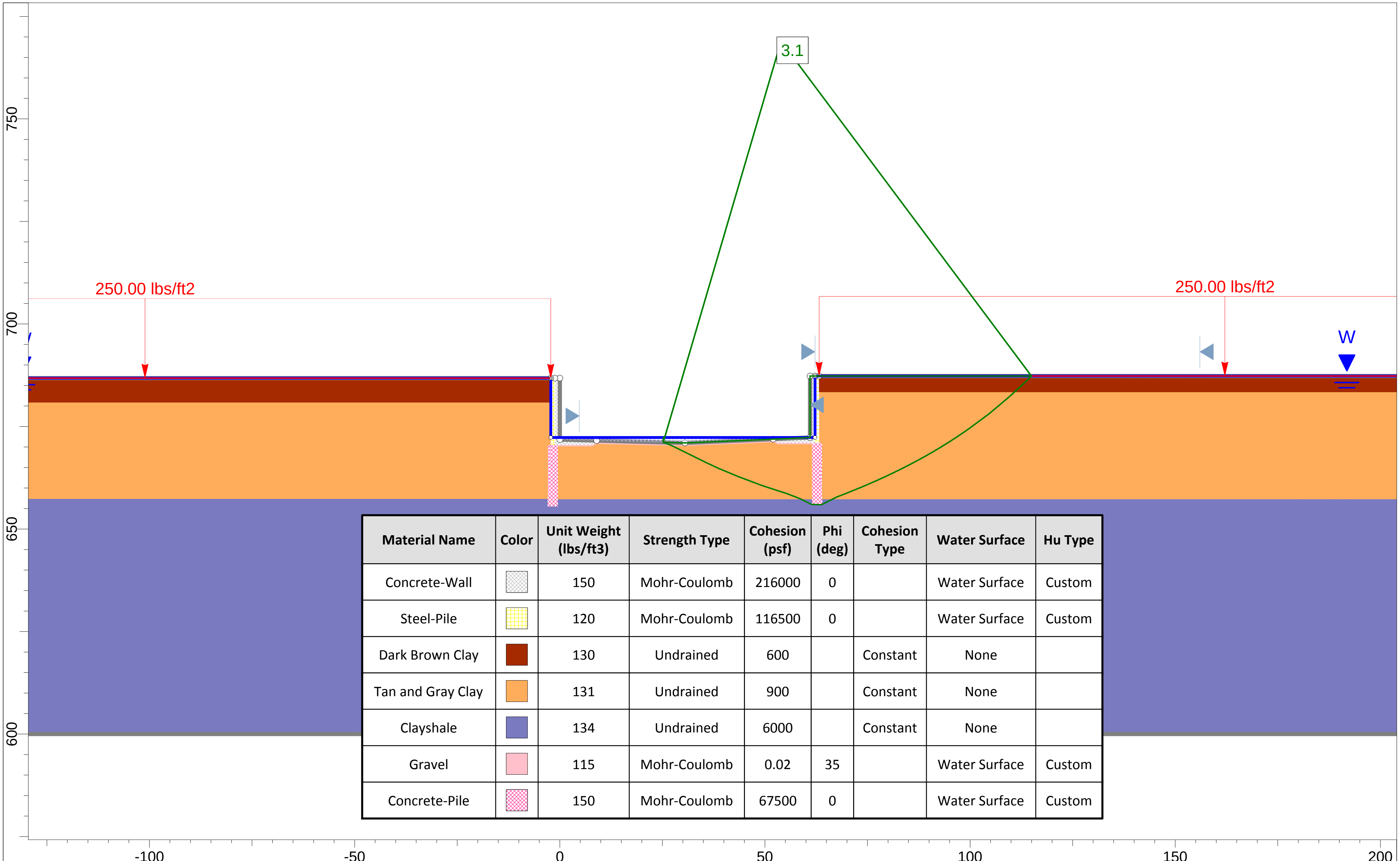
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Concrete-Wall		150	Mohr-Coulomb	216000	0	Water Surface	Custom
Steel-Pile		120	Mohr-Coulomb	116500	0	Water Surface	Custom
Dark Brown Clay		130	Mohr-Coulomb	300	20	Water Surface	Custom
Tan and Gray Clay		131	Mohr-Coulomb	500	20	Water Surface	Custom
Clayshale		134	Mohr-Coulomb	500	20	Water Surface	Custom
Gravel		115	Mohr-Coulomb	0.02	35	Water Surface	Custom
Concrete-Pile		150	Mohr-Coulomb	67500	0	Water Surface	Custom

Global Stability Analysis
Drained - Rapid Draw Down

Seeling Channel Improvements, Phase II
San Antonio, Texas

Figure 22

RABA KISTNER CONSULTANTS



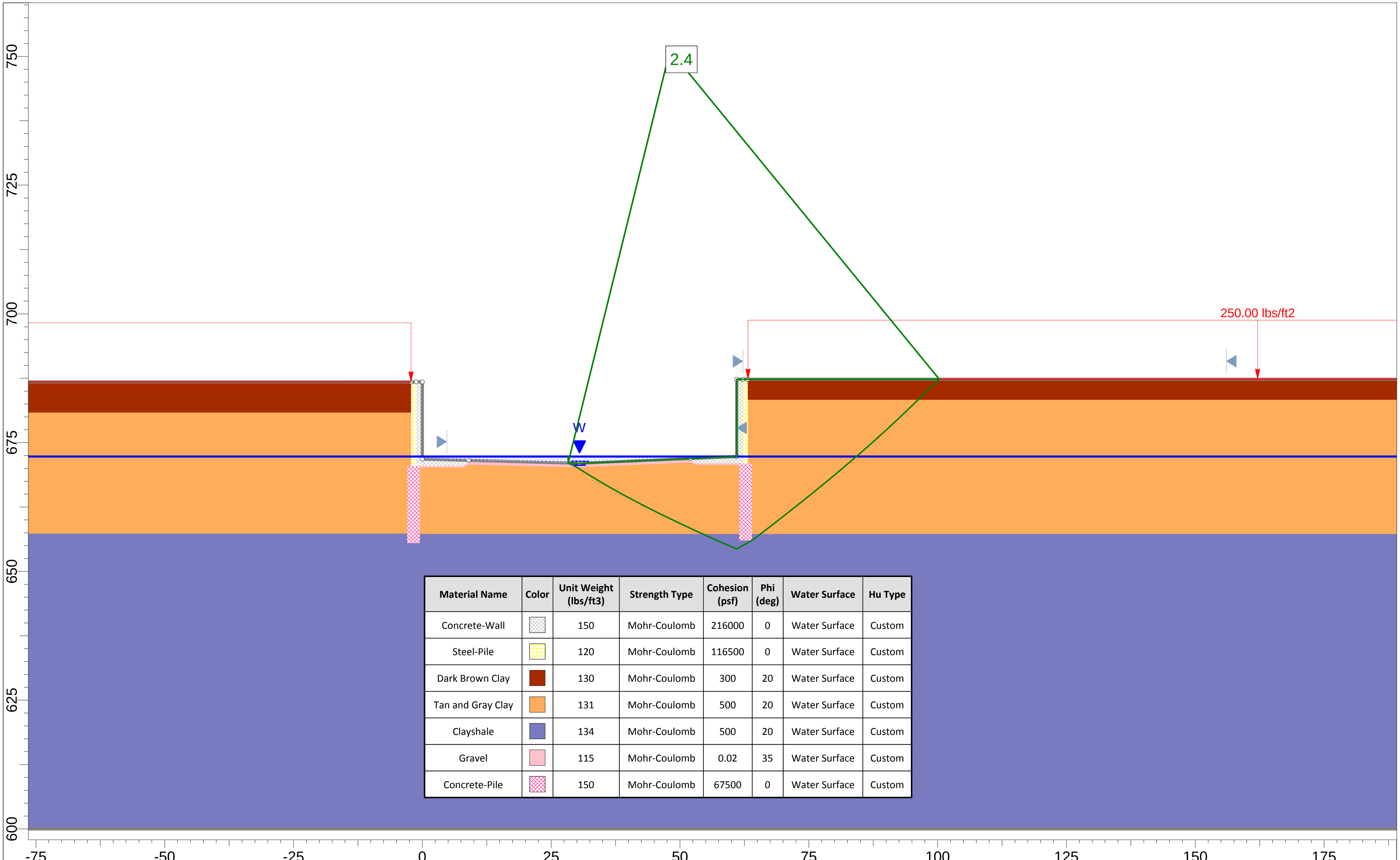
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type
Concrete-Wall		150	Mohr-Coulomb	216000	0		Water Surface	Custom
Steel-Pile		120	Mohr-Coulomb	116500	0		Water Surface	Custom
Dark Brown Clay		130	Undrained	600		Constant	None	
Tan and Gray Clay		131	Undrained	900		Constant	None	
Clayshale		134	Undrained	6000		Constant	None	
Gravel		115	Mohr-Coulomb	0.02	35		Water Surface	Custom
Concrete-Pile		150	Mohr-Coulomb	67500	0		Water Surface	Custom

Global Stability Analysis
Undrained - Rapid Draw Down

Seeling Channel Improvements, Phase II
San Antonio, Texas

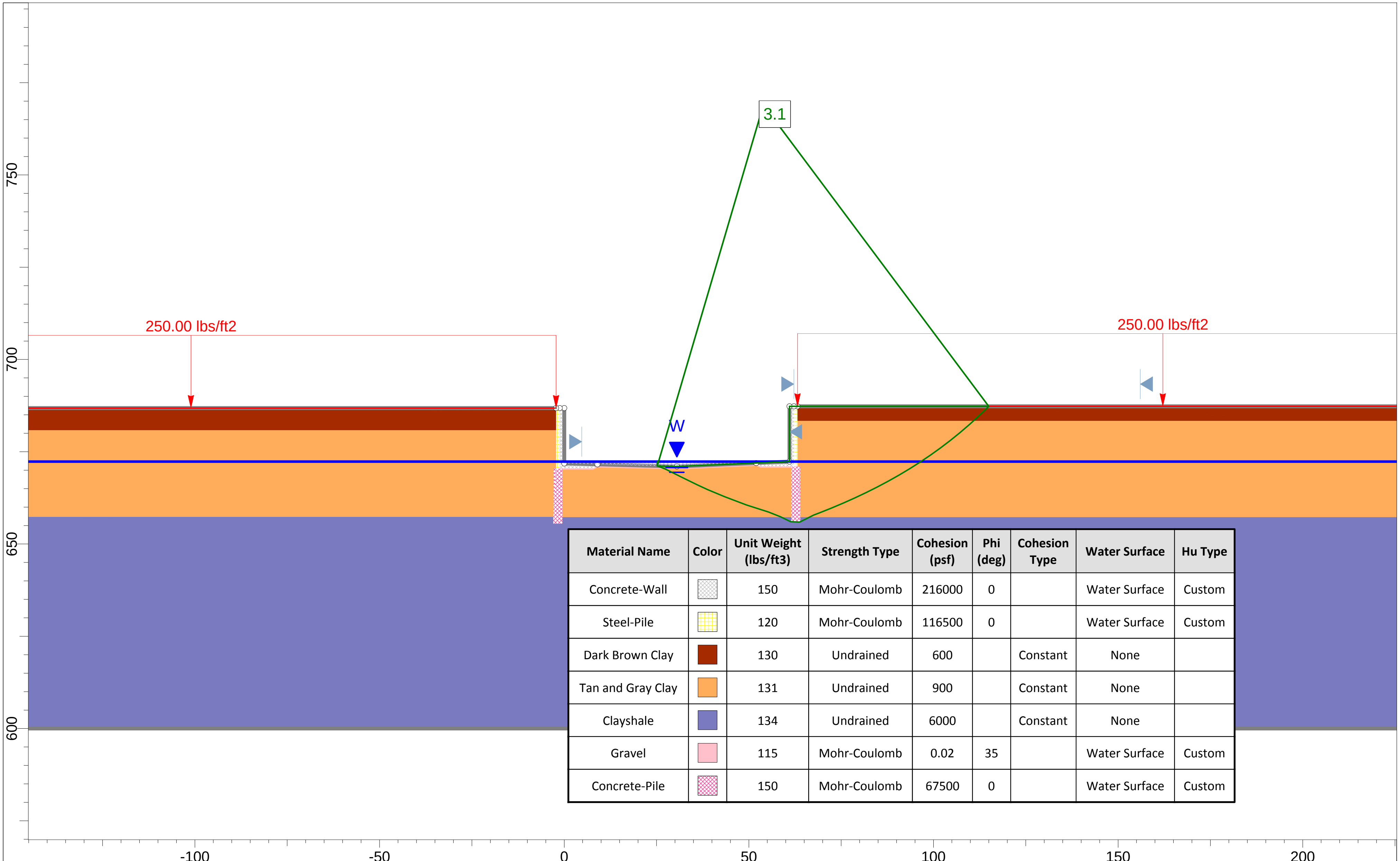
Figure 23

RABA KISTNER CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Concrete-Wall		150	Mohr-Coulomb	216000	0	Water Surface	Custom
Steel-Pile		120	Mohr-Coulomb	116500	0	Water Surface	Custom
Dark Brown Clay		130	Mohr-Coulomb	300	20	Water Surface	Custom
Tan and Gray Clay		131	Mohr-Coulomb	500	20	Water Surface	Custom
Clayshale		134	Mohr-Coulomb	500	20	Water Surface	Custom
Gravel		115	Mohr-Coulomb	0.02	35	Water Surface	Custom
Concrete-Pile		150	Mohr-Coulomb	67500	0	Water Surface	Custom

Global Stability Analysis Drained - Steady State	Seeling Channel Improvements, Phase II San Antonio, Texas	Figure 24 RABA KISTNER CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type
Concrete-Wall		150	Mohr-Coulomb	216000	0		Water Surface	Custom
Steel-Pile		120	Mohr-Coulomb	116500	0		Water Surface	Custom
Dark Brown Clay		130	Undrained	600		Constant	None	
Tan and Gray Clay		131	Undrained	900		Constant	None	
Clayshale		134	Undrained	6000		Constant	None	
Gravel		115	Mohr-Coulomb	0.02	35		Water Surface	Custom
Concrete-Pile		150	Mohr-Coulomb	67500	0		Water Surface	Custom

Global Stability Analysis
Undrained - Steady State

Seeling Channel Improvements, Phase II
San Antonio, Texas

Figure 25

RABA KISTNER CONSULTANTS

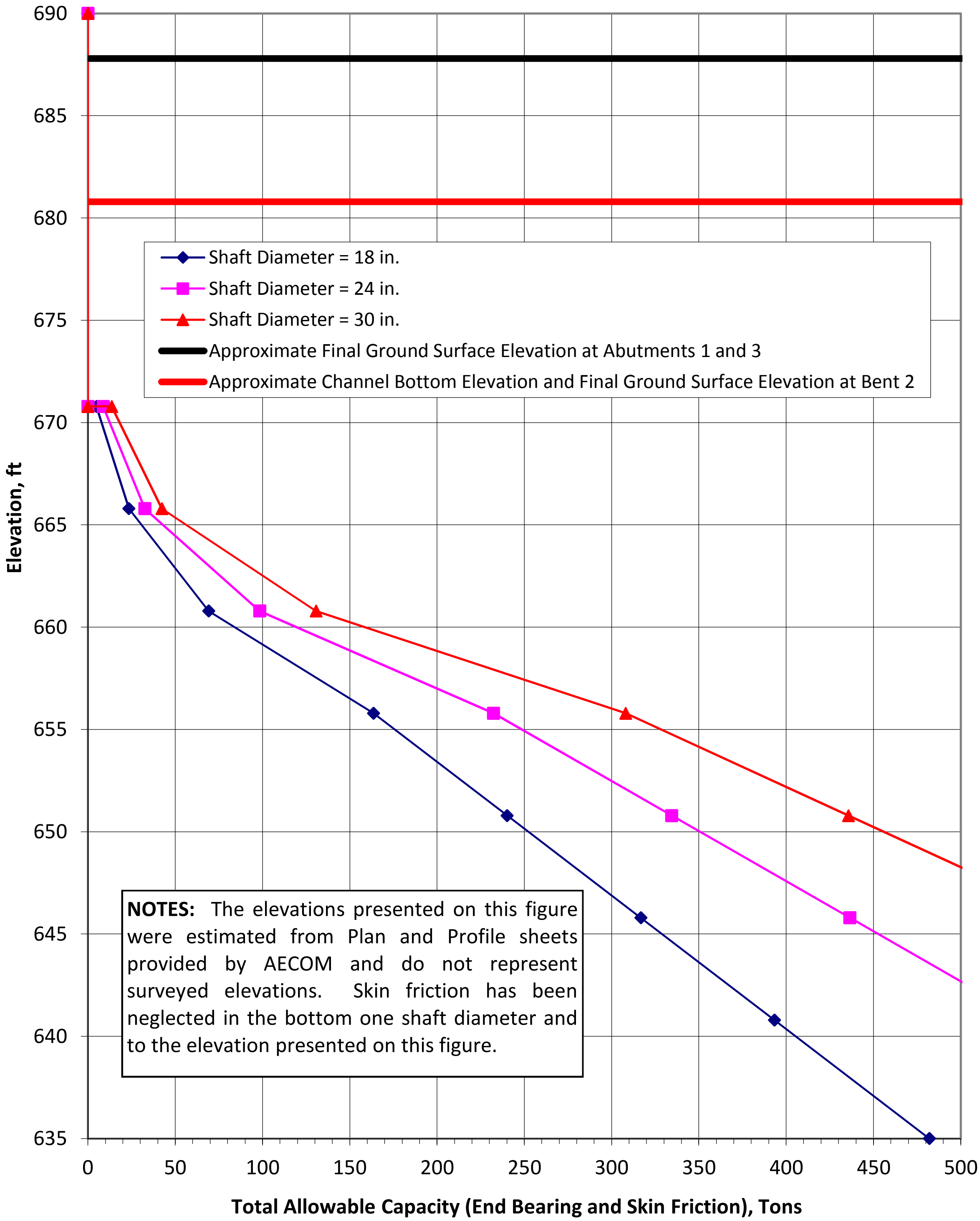
DRILLED PIER AXIAL CAPACITY CURVE

Straight Shaft Piers

Seeling Channel Improvements, Phase II

Morning Glory Bridge, San Antonio, Texas

(Borings B-101 and B-102 - Abutment 1, Bent 2, and Abutment 3)



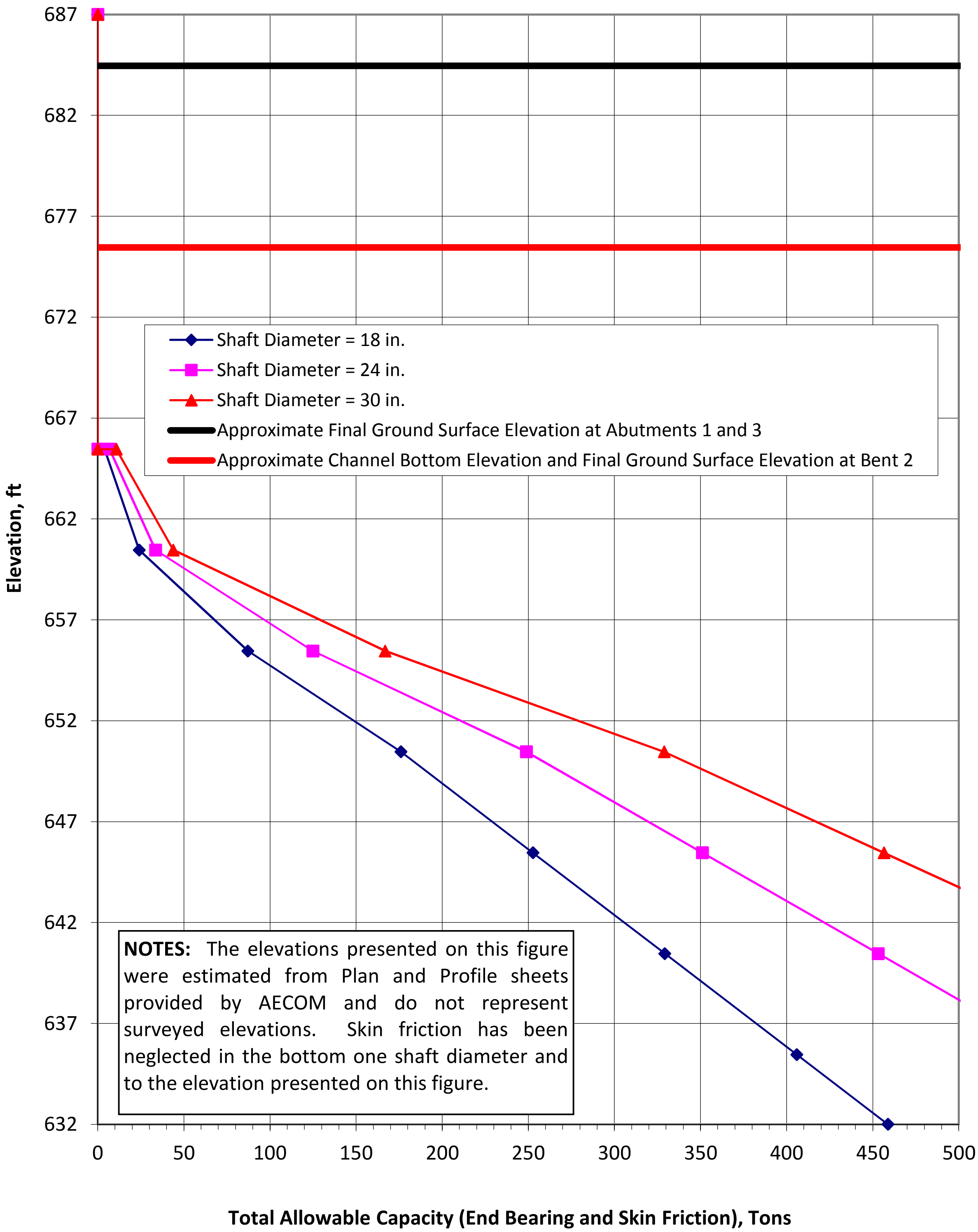
DRILLED PIER AXIAL CAPACITY CURVE

Straight Shaft Piers

Seeling Channel Improvements, Phase II

West Huisache Bridge, San Antonio, Texas

(Borings B-103 and B-104 - Abutment 1, Bent 2, and Abutment 3)



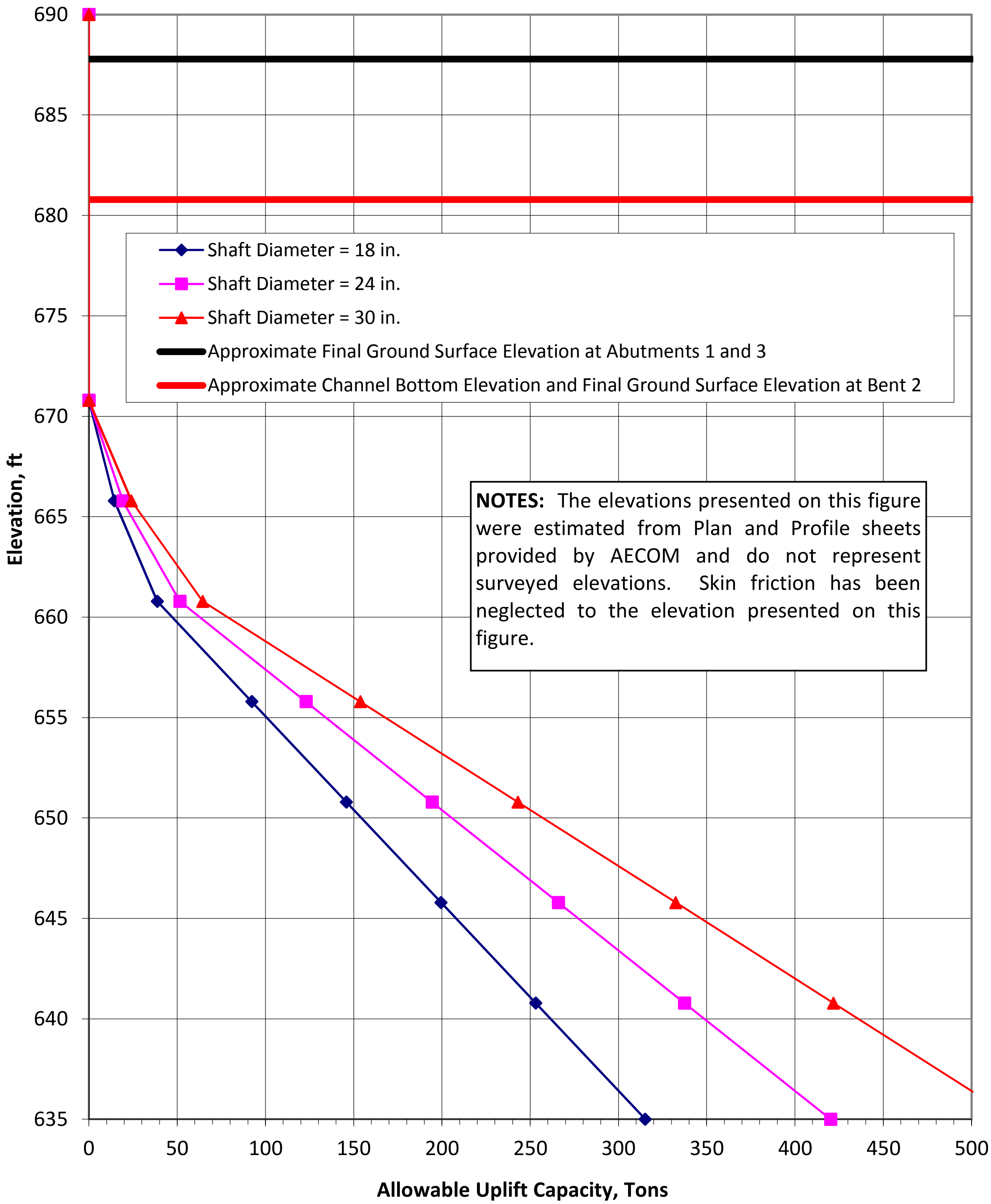
DRILLED PIER UPLIFT CAPACITY CURVE

Straight Shaft Piers

Seeling Channel Improvements, Phase II

Morning Glory Bridge, San Antonio, Texas

(Borings B-101 and B-102 - Abutment 1, Bent 2, and Abutment 3)



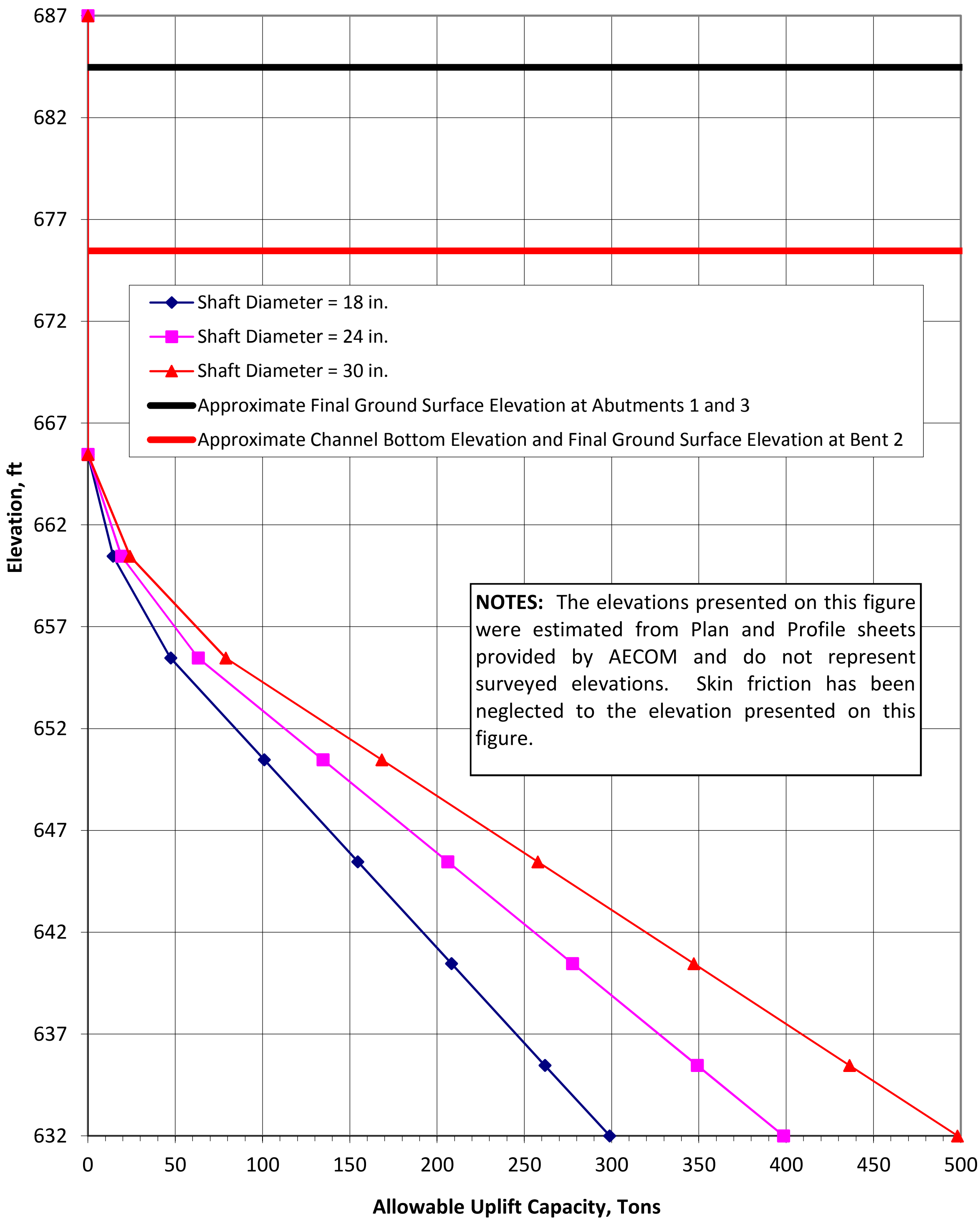
DRILLED PIER UPLIFT CAPACITY CURVE

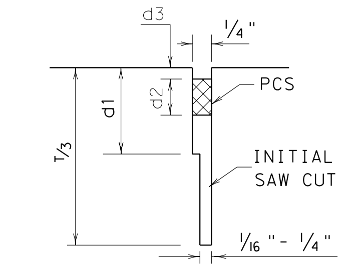
Straight Shaft Piers

Seeling Channel Improvements, Phase II

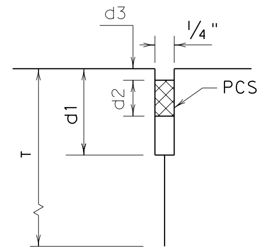
West Huisache Bridge, San Antonio, Texas

(Borings B-103 and B-104 - Abutment 1, Bent 2, and Abutment 3)



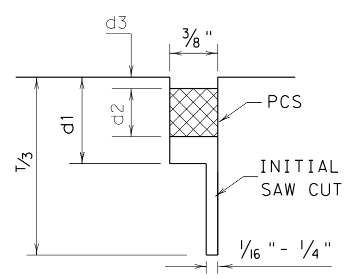


SAWED
LONGITUDINAL JOINT

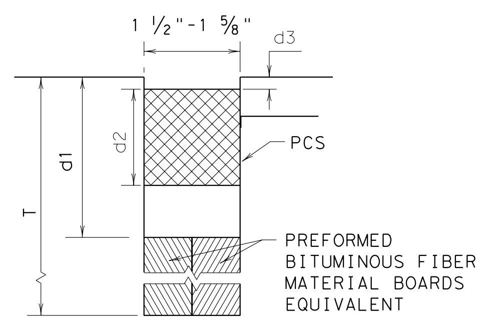


LONGITUDINAL
CONSTRUCTION JOINT

LONGITUDINAL JOINT SEALS



SAWED
CONTRACTION JOINT



FORMED
FORMED EXPANSION JOINT

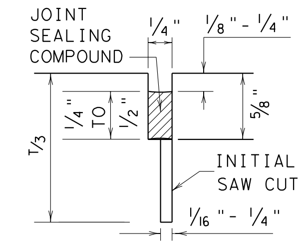
TRANSVERSE JOINT SEALS

METHOD A: PREFORMED COMPRESSION SEALS (PCS)

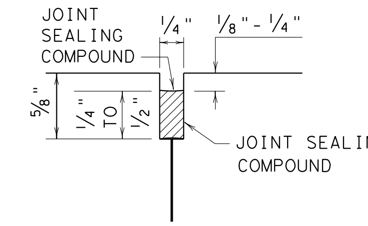
(CLASS 6 PREFORMED JOINT SEALANT)

GENERAL NOTES FOR METHOD "A"

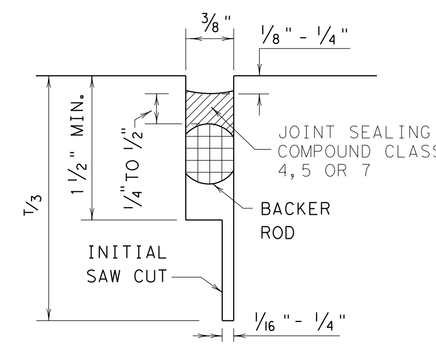
1. UNLESS OTHERWISE SHOWN IN THE PLANS, EITHER METHOD "A" OR METHOD "B" MAY BE USED.
2. THE LOCATION OF JOINTS SHALL BE AS SHOWN ELSEWHERE IN THE PLANS.
3. DIMENSIONS d1, d2, AND d3 SHALL BE IN ACCORDANCE WITH THE PREFORMED COMPRESSION SEAL MANUFACTURES RECOMMENDATION.
4. THE JOINT RESERVOIR FOR SEALANT SHALL BE SAWED UNLESS OTHERWISE SHOWN ON THE PLANS FOR THE LONGITUDINAL AND TRANSVERSE CONSTRUCTION AND THE TWO SAWED JOINTS.
5. THE JOINTS SHALL BE CLEANED IN ACCORDANCE WITH THE ITEM 438 AND PRIOR TO BEGINNING OPERATIONS, THE CONTRACTOR SHALL SUBMIT A STATEMENT FROM THE SEALANT MANUFACTURER SHOWING THE RECOMMENDED EQUIPMENT AND INSTALLATION PROCEDURES TO BE USED.
6. THE SAW CUT FOR THE LONGITUDINAL JOINT SHALL BE ONE FOURTH THE SLAB THICKNESS WHEN CRUSHED LIMESTONE IS USED AS THE COARSE AGGREGATE.



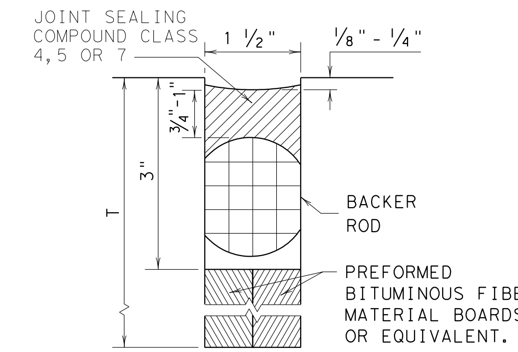
SAWED
LONGITUDINAL JOINT



LONGITUDINAL OR TRANSVERSE
CONSTRUCTION JOINT



TRANSVERSE SAWED
CONTRACTION JOINT



TRANSVERSE FORMED
EXPANSION JOINT

METHOD B: JOINT SEALING COMPOUND

GENERAL NOTES FOR METHOD "B"

1. UNLESS OTHERWISE SHOWN IN THE PLANS, EITHER METHOD "A" OR METHOD "B" MAY BE USED.
2. THE LOCATION OF JOINTS SHALL BE AS SHOWN ELSEWHERE IN THE PLANS.
3. THE ENGINEER SHALL SELECT A TARGET PLACEMENT THICKNESS FOR THE SEALANT DETAILS WHICH SHOW RANGES IN THICKNESS. THE TARGET THICKNESS WILL NORMALLY BE THE MIDPOINT OF THE RANGE.
4. THE JOINT RESERVOIR FOR SEALANT SHALL BE SAWED UNLESS OTHERWISE SHOWN ON THE PLANS FOR THE LONGITUDINAL AND TRANSVERSE CONSTRUCTION AND THE TWO SAWED JOINTS.
5. THE JOINTS SHALL BE CLEANED IN ACCORDANCE WITH THE ITEM 438 AND PRIOR TO BEGINNING OPERATIONS, THE CONTRACTOR SHALL SUBMIT A STATEMENT FROM THE SEALANT MANUFACTURER SHOWING THE RECOMMENDED EQUIPMENT AND INSTALLATION PROCEDURES TO BE USED.
6. THE SAW CUT FOR THE LONGITUDINAL JOINT SHALL BE ONE FOURTH THE SLAB THICKNESS WHEN CRUSHED LIMESTONE IS USED AS THE COARSE AGGREGATE.

PROJECT No. ASA14-003-00
Figure 28A

Texas Department of Transportation
Construction Division (Pavement)

CONCRETE PAVING DETAILS JOINT SEALS

JS-94

©TxDOT SEPTEMBER 1994	DN: LJB	CK: LJB	DW: BGD	CK: GLG
MODIFICATIONS	DISTRICT	FEDERAL AID PROJECT		SHEET
	COUNTY	CONTROL	SECTION	JOB	HIGHWAY

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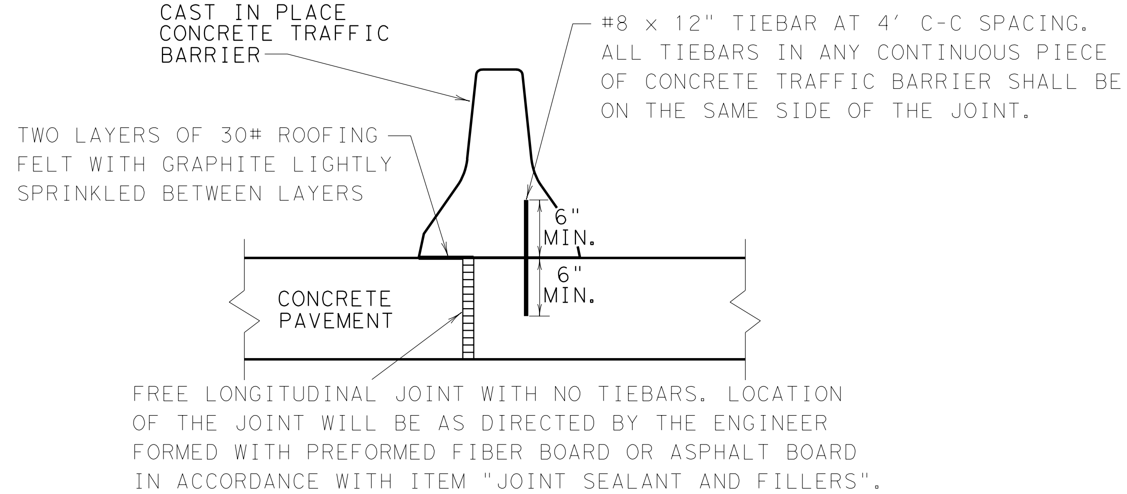
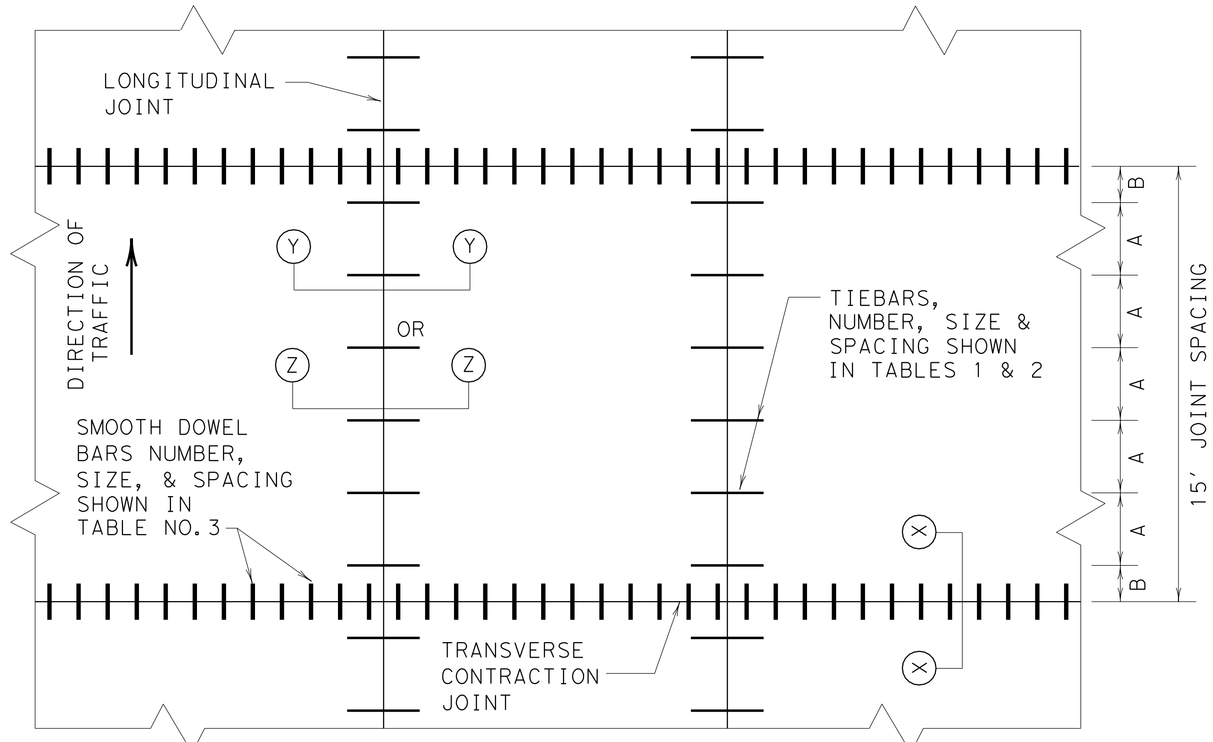
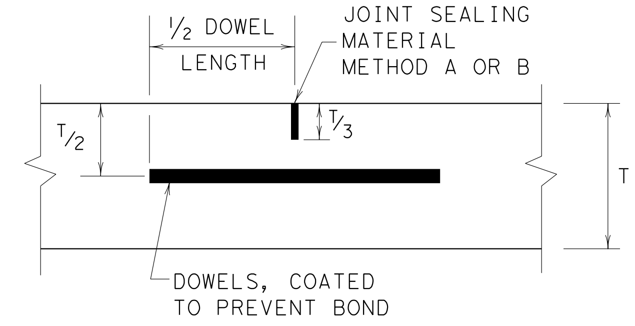


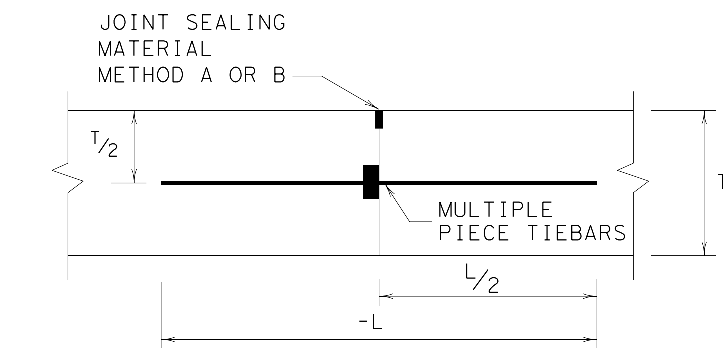
FREE LONGITUDINAL JOINT DETAIL



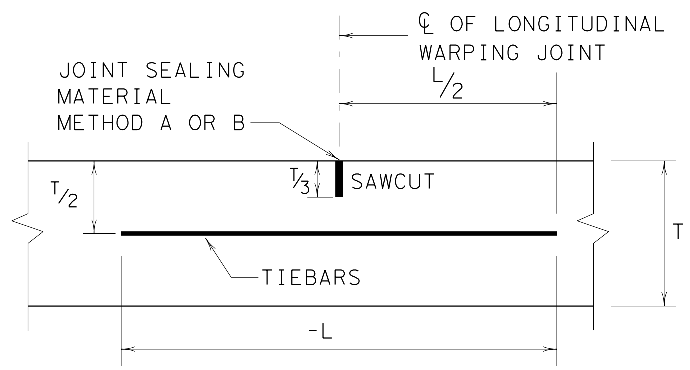
PAVEMENT DETAIL LAYOUT



TRANSVERSE CONTRACTION JOINT
SECTION X-X



LONGITUDINAL CONSTRUCTION JOINT
SECTION Y-Y



LONGITUDINAL WARPING JOINT
SECTION Z-Z

GENERAL NOTES

1. CONCRETE SLABS WIDER THAN 100′ WITHOUT A FREE JOINT, ARE NOT COVERED BY THIS STANDARD.
2. FOR FURTHER INFORMATION REGARDING THE PLACEMENT OF CONCRETE AND LOAD TRANSFER DEVICES REFER TO THE GOVERNING SPECIFICATIONS FOR "CONCRETE PAVEMENT" AND "REINFORCING STEEL."
3. DETAILS FOR PAVEMENT WIDTH, PAVEMENT THICKNESS, AND CROWN CROSS SLOPE SHALL BE AS SHOWN ELSEWHERE IN THE PLANS.
4. THE DETAIL FOR THE JOINT SEALANT AND RESERVOIR WILL BE SHOWN IN CONCRETE PAVEMENT DETAIL, JOINT SEALANT STANDARD (JS-94).
5. PAVEMENT WIDTHS IN EXCESS OF 16′ SHALL BE PROVIDED WITH A LONGITUDINAL JOINT (SECTION Z-Z OR Y-Y). THESE JOINTS SHALL BE LOCATED WITHIN 6" OF THE LANE LINES UNLESS SHOWN ELSEWHERE ON THE PLANS.
6. THE JOINT BETWEEN OUTSIDE LANE AND SHOULDER SHALL BE A LONGITUDINAL WARPING JOINT (SECTION Z-Z) UNLESS OTHERWISE SHOWN IN THE PLANS.
7. THE SPACING BETWEEN TRANSVERSE JOINTS SHALL BE 15 FEET UNLESS OTHERWISE SHOWN IN THE PLANS.
8. WHERE A MONOLITHIC CURB IS SPECIFIED, THE JOINT IN THE CURB SHALL COINCIDE WITH PAVEMENT JOINTS AND MAY BE FORMED BY ANY MEANS APPROVED BY THE ENGINEER.
9. TRANSVERSE CONSTRUCTION JOINTS MAY BE FORMED BY USE OF METAL OR WOOD FORMS EQUAL IN DEPTH TO THE NOMINAL DEPTH OF THE PAVEMENT, OR BY METHODS APPROVED BY THE ENGINEER.
10. THE ENGINEER WILL ADJUST THE REQUIRED NUMBER OF TIEBARS FOR SLABS SHORTER OR LONGER THAN 15′. SPACING "B" WILL BE ADJUSTED TO MAINTAIN A MINIMUM CLEARANCE OF 2" BETWEEN THE TIEBAR AND THE DOWEL BARS AT THE TRANSVERSE JOINT AND THE "A" SPACING WILL REMAIN AS REQUIRED FOR THE PAVEMENT SLAB WIDTH.
11. MULTIPLE PIECE TIEBARS SHALL BE USED AT LONGITUDINAL CONSTRUCTION JOINTS UNLESS OTHERWISE SPECIFIED IN THE PLANS.
12. THE SAW CUT FOR LONGITUDINAL WARPING AND THE TRANSVERSE CONSTRUCTION JOINTS MAY BE ONE FOURTH THE SLAB THICKNESS WHEN CRUSHED LIMESTONE IS USED AS THE COARSE AGGREGATE.

TABLE NO.1 TIEBARS REQUIRED FOR LONGITUDINAL JOINT JOINTS FOR EACH 15′ SLAB

ASTM A-616 OR A-615 (GRADE 60) STRAIGHT OR MULIPLE PIECE REINFORCING TIEBARS		CONCRETE SLAB THICKNESS	DISTANCE FROM THE LONGITUDINAL JOINT TO THE NEAREST LONGITUDINAL FREE EDGE			
			< OR =16′	< OR =24′	< OR =34′	< OR =50′
BAR LENGTH, "L" INCHES	BAR SIZE	"T" INCHES	REQUIRED NO. OF BARS	REQUIRED NO. OF BARS	REQUIRED NO. OF BARS	REQUIRED NO. OF BARS
42	#5 (5/8")	8	5	5	6	9
		9	5	5	7	10
		10	5	5	7	11
		11	5	6	8	12
		12	5	6	9	13
		13	5	7	9	13
		14	6	7	10	NA
		15	6	8	11	NA
50	#6 (3/4")	8	5	5	5	6
		9	5	5	5	7
		10	5	5	5	8
		11	5	5	6	8
		12	5	5	6	9
		13	5	5	7	10
		14	5	5	7	10
		15	5	6	8	11

THE DISTANCE TO THE FREE EDGE WILL BE DETERMINED BY THE ENGINEER AND THE DISTANCE WILL BE BASED ON THE NOMINAL WIDTHS OF THE LANES AND SHOULDERS PLUS ANY TIED RAMPS OR CONNECTING ROADWAYS.

TABLE NO.2 TIEBAR SPACINGS
SPACING REQUIREMENT FOR 15′SLAB FOR REQUIRED NUMBER OF BARS

REQUIRED NO. OF BARS	REGUALR SPACING "A" INCHES	FIRST AT JOINT "B" INCHES
5	36	18
6	30	15
7	25	15
8	21	16.5
9	18	18
10	16	18
11	15	15
12	13	18.5
13	12	18

TABLE NO.3 DOWELS REQUIEMENTS

T, IN.	DOWELS (SMOOTH BARS)	
	SIZE AND LENGTH	AVERAGE SPACING (INCHES)
8	1" X 18"	12
9	1 1/8" X 18"	12
10	1 1/4" X 18"	12
11	1 3/8" X 18"	12
12	1 1/2" X 18"	12
13	1 5/8" X 18"	12
14	1 3/4" X 18"	12
15	1 7/8" X 18"	12

PROJECT No. ASA14-003-00
Figure 28B

Texas Department of Transportation
Construction Division (Pavements)

CONCRETE PAVEMENT DETAILS
CONTRACTION DESIGN
T-8 THROUGH 15 INCHES
CPCD-94

©TxDOT SEPTEMBER 1994
DN: LJB | CK: LJB | DW: BGD | CK: GLG
MODIFICATIONS | DISTRICT | FEDERAL AID PROJECT | SHEET
COUNTY | CONTROL | SECTION | JOB | HIGHWAY

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LEVELS DISPLAYED
1

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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City of San Antonio

TRANSPORTATION AND CAPITAL IMPROVEMENTS

RECEIPT OF ADDENDUM NUMBER(S) **1** IS HEREBY ACKNOWLEDGED FOR PLANS AND

SPECIFICATIONS FOR CONSTRUCTION OF THE **Seeling Channel Phase II Drainage #40-00427**

FOR WHICH BIDS WILL BE OPENED ON **April 7, 2015 at 2:00pm**

THIS ACKNOWLEDGEMENT MUST BE SIGNED AND RETURNED WITH THE BID PACKAGE.

Company Name: ________________________

Address: ______________________________

City/State/Zip Code: _____________________

Date: _______________

____
Signature

____
Print Name/Title