ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 100840

FEDERAL AID PROJECT NO. NHPP-0056(36)

DITCH NOS. 1 & 47 STRS. & APPRS. (S)

STATE HIGHWAY 308 **SECTION** 1

IN POINSETT **COUNTY**

The information contained herein was obtained by the Department for design and estimating purposes only. It is being furnished with the express understanding that said information does not constitute a part of the Proposal or Contract and represents only the best knowledge of the Department as to the location, character and depth of the materials encountered. The information is only included and made available so that bidders may have access to subsurface information obtained by the Department and is not intended to be a substitute for personal investigation, interpretation and judgment of the bidder. The bidder should be cognizant of the possibility that conditions affecting the cost and/or quantities of work to be performed may differ from those indicated herein.

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

April 3, 2017

TO: Mr. Trinity Smith, Engineer of Roadway Design

SUBJECT: Job No. 100840
Ditch Nos. 1 & 47 Str. & Apprs. (S)
Route 308 Section 1
Poinsett County

Transmitted herewith is the requested Soil Survey, strength data and Resilient Modulus test results for the above referenced job. The project consists of replacing the bridge for Ditch Numbers 1 and 47 on Highway 308. Samples were obtained in the existing travel lanes and ditch line. There were no paved shoulders within the project limits.

Based on laboratory results of samples obtained, the subgrade soils consist primarily of highly plastic clay with gravel. Cross sections are not currently available; it is assumed that the construction grade line will closely match that of the existing roadway. The subgrade soils will likely require remediation where new embankment crosses existing ditches or shallow fills in agricultural fields. Remediation recommendations can be made when cross sections become available. No slides were observed within the project limits.

Additional earthwork requirements will be made upon request when plans are further developed.

Listed below is the additional information requested for use in developing the plans:

1. The Qualified Products List (QPL) indicates that Aggregate Base Course (Class CL-7) is available from commercial producers located at the river ports near Osceola.

2. Asphalt Concrete Hot Mix

Type	Asphalt Cement %	Mineral Aggregate %
Surface Course	5.2	94.8
Binder Course	4.1	95.9
Base Course	3.9	96.1

Michael C. Benson
Materials Engineer

MCB:pt:bjj
Attachment
cc: State Constr. Eng. – Master File Copy
District 10 Engineer
System Information and Research Div.
G. C. File

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT - LITTLE ROCK, ARKANSAS
MATERIALS DIVISION
MICHAEL BENSON, MATERIALS ENGINEER
*** SOIL SURVEY STRENGTH TEST REPORT ***

DATE - 03/23/2017
JOB NUMBER - 100840

SEQUENCE NO. - 1
MATERIAL CODE - SSRV
SPEC. YEAR - 2014
SUPPLIER ID. - 1
COUNTY/STATE - 56
DISTRICT NO. - 10

JOB NAME - DITCH NOS. 1 & 47 STR. & APPRS. (S)

STATION LIMITS	R-VALUE AT 240 psi
BEGIN JOB - END JOB	LESS THAN 5
RESILIENT MODULUS	
STA. 228+00	10327

REMARKS -
-

AASHTO TESTS : T190

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT
MATERIALS DIVISION

AASHTO T 307-99 - RESILIENT MODULUS OF SUBGRADE SOILS
RECOMPACTED SAMPLES

Job No.	100840	**Material Code**	SSRVPS
Date Sampled:	2/28/17	**Station No.:**	228+00
Date Tested:	March 16, 2017	**Location:**	16LT
Name of Project:	DITCH NOS. 1 & 47 STR. & APPRS. (S)		
County:	**Code:** 56 **Name:** POINSETT		
Sampled By:	DICKERSON/FRAZIER	**Depth:**	0-5
Lab No.:	20170731	**AASHTO Class:**	A-7-6(12)
Sample ID:	RV188	**Material Type (1 or 2):**	2
LATITUDE:		**LONGITUDE:**	

1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No)	N
Testing - Permanent Strain > 5% (Y=Yes or N=No)	N
Number of Load Sequences Completed (0-15)	15

2. Specimen Information:

Specimen Diameter (in):	
Top	3.96
Middle	3.95
Bottom	3.95
Average	3.95
Membrane Thickness (in):	0.01
Height of Specimen, Cap and Base (in):	8.02
Height of Cap and Base (in):	0.00
Initial Length, Lo (in):	8.02
Initial Area, Ao (sq. in):	12.20
Initial Volume, AoLo (cu. in):	97.85

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	2928.40

4. Soil Properties:

Optimum Moisture Content (%):	20.2
Maximum Dry Density (pcf):	98.5
95% of MDD (pcf):	93.6
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	2928.40
Compaction Moisture content (%):	20.5
Compaction Wet Density (pcf):	114.03
Compaction Dry Density (pcf):	94.63
Moisture Content After Mr Test (%):	20.4

6. Quick Shear Test (Y=Yes, N=No, N/A=Not Applicable): #VALUE!

7. Resilient Modulus, Mr: 13140(Sc)^-0.14252(S3)^0.10666

8. Comments ____________________

9. Tested By: G.WENDLAND **Date:** March 16, 2017

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT
MATERIALS DIVISION

AASHTO T 307-99 - RESILIENT MODULUS OF SUBGRADE SOILS
RECOMPACTED SAMPLES

Job No. 100840
Date Sampled: 2/28/17
Date Tested: March 16, 2017
Name of Project: DITCH NOS. 1 & 47 STR. & APPRS. (S)
County: **Code:** 56 **Name:** POINSETT
Sampled By: DICKERSON/FRAZIER
Lab No.: 20170731
Sample ID: RV188
LATITUDE:

Material Code SSRVPS
Station No.: 228+00
Location: 16LT
Depth: 0-5
AASHTO Class: A-7-6(12)
Material Type (1 or 2): 2
LONGITUDE:

PARAMETER	*Chamber Confining Pressure*	*Nominal Maximum Axial Stress*	*Actual Applied Max. Axial Load*	*Actual Applied Cyclic Load*	*Actual Applied Contact Load*	*Actual Applied Max. Axial Stress*	*Actual Applied Cyclic Stress*	*Actual Applied Contact Stress*	*Average Recov Def. LVDT 1 and 2*	*Resilient Strain*	*Resilient Modulus*
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	6.0	2.0	25.2	22.5	2.7	2.1	1.8	0.2	0.00101	0.00013	14,608
Sequence 2	6.0	4.0	47.1	44.3	2.8	3.9	3.6	0.2	0.00211	0.00026	13,800
Sequence 3	6.0	6.0	69.5	66.0	3.6	5.7	5.4	0.3	0.00337	0.00042	12,863
Sequence 4	6.0	8.0	92.8	86.8	6.0	7.6	7.1	0.5	0.00484	0.00060	11,781
Sequence 5	6.0	10.0	114.9	106.6	8.4	9.4	8.7	0.7	0.00648	0.00081	10,819
Sequence 6	4.0	2.0	25.1	22.4	2.7	2.1	1.8	0.2	0.00107	0.00013	13,725
Sequence 7	4.0	4.0	46.9	44.2	2.7	3.8	3.6	0.2	0.00222	0.00028	13,120
Sequence 8	4.0	6.0	68.6	65.9	2.7	5.6	5.4	0.2	0.00348	0.00043	12,437
Sequence 9	4.0	8.0	91.8	86.8	5.0	7.5	7.1	0.4	0.00489	0.00061	11,673
Sequence 10	4.0	10.0	114.5	107.0	7.5	9.4	8.8	0.6	0.00647	0.00081	10,872
Sequence 11	2.0	2.0	25.0	22.4	2.6	2.0	1.8	0.2	0.00121	0.00015	12,180
Sequence 12	2.0	4.0	46.9	44.3	2.7	3.8	3.6	0.2	0.00244	0.00030	11,913
Sequence 13	2.0	6.0	68.5	65.8	2.7	5.6	5.4	0.2	0.00379	0.00047	11,420
Sequence 14	2.0	8.0	90.8	86.6	4.2	7.4	7.1	0.3	0.00524	0.00065	10,873
Sequence 15	2.0	10.0	113.6	107.0	6.6	9.3	8.8	0.5	0.00681	0.00085	10,327

TESTED BY ı.WENDLAND DATE March 16, 2017
REVIEWED BY DATE

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

MATERIALS DIVISION

AASHTO T 307-99 - RESILIENT MODULUS OF SUBGRADE SOILS

RECOMPACTED / THINWALL TUBE SAMPLES

Job No.	100840	**Material Code**	SSRVPS
Date Sampled:	2/28/17	**Station No.:**	228+00
Date Tested:	March 16, 2017	**Location:**	16LT
Name of Project:	DITCH NOS. 1 & 47 STR. & APPRS. (S)		
County:	**Code:** 56 **Name:** POINSETT		
Sampled By:	DICKERSON/FRAZIER	**Depth:**	0-5
Lab No.:	20170731	**AASHTO Class:**	A-7-6(12)
Sample ID:	RV188	**Material Type (1 or 2):**	2
LATITUDE:		**LONGITUDE:**	

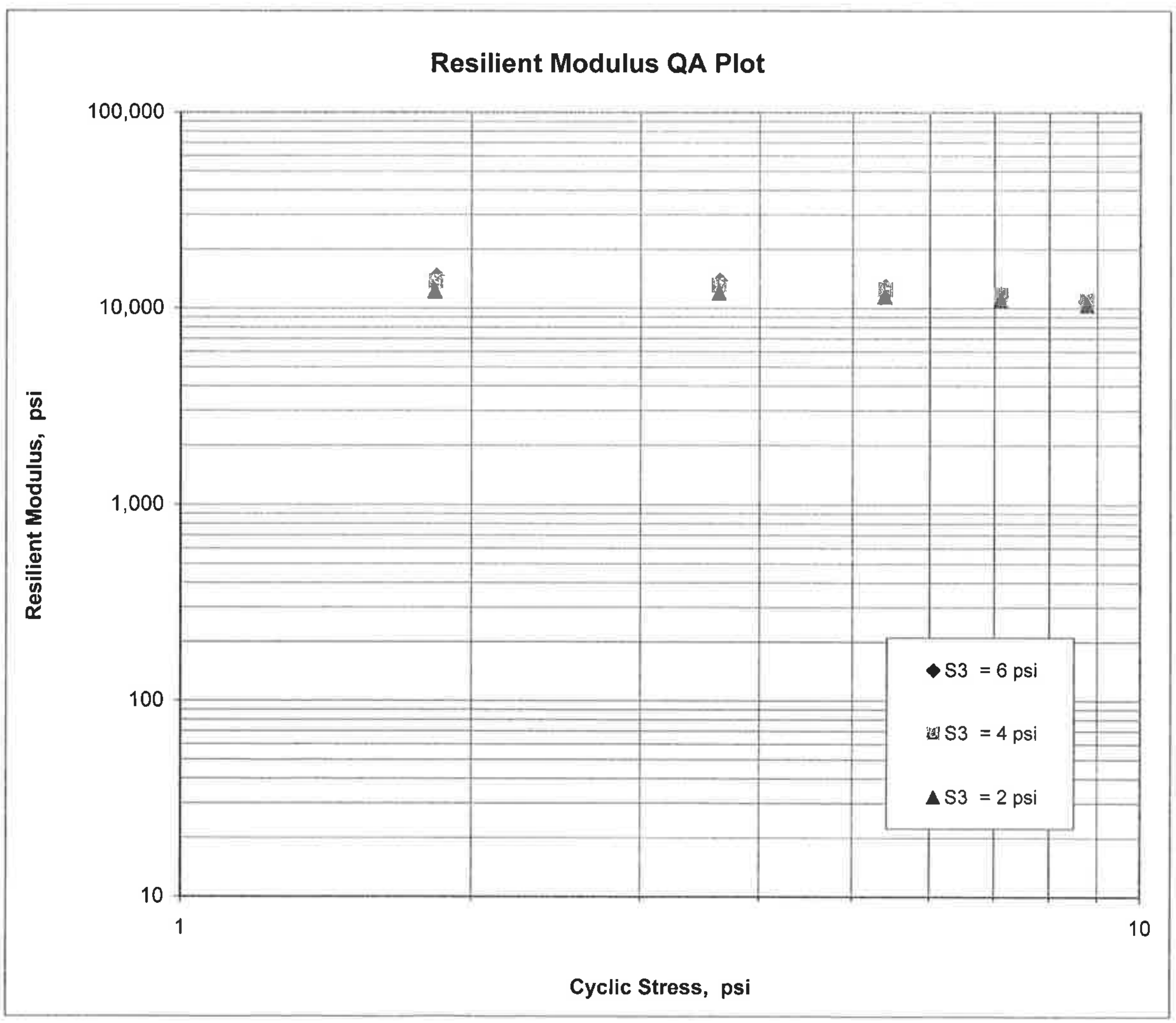
$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 13,140

K2 = -0.14252

K5 = 0.10666

R^2 = 0.88



JOB: **100840** **Arkansas State Highway Transporation Department**

JOB NAME: DITCH NOS. 1 & 47 STR. & APPRS. (S) Materials Division

COUNTY NO. 56 DATE TESTED 3/9/2017 Michael Benson, Materials Engineer

STA.#	LOC.	DEPTH	COLOR	#4	#10	#40	#80	#200	L.L.	P.I.	SOIL CLASS	LAB #:	%MOISTURE
				S	I	E	V	E	S				
228+00	16 LT	0-5	BR/GR	89	88	84	75	60	41	26	A-7-6(12)	RV188	
202+00	05RT	0-5	BR/GR	96	92	86	81	78	57	40	A-7-6(31)	S182	44.3
202+00	16 RT	0-5	GR/BR	92	86	78	72	68	51	34	A-7-6(21)	S183	39.4
210+00	05 LT	0-5	BR/GR	96	92	87	74	64	42	29	A-7-6(15)	S184	23.8
220+00	05 LT	0-5	BR/GR	93	91	86	76	68	46	31	A-7-6(19)	S185	23.8
228+00	05 LT	0-5	BR/GR	96	90	81	66	56	40	28	A-6(12)	S186	34.3
228+00	16 LT	0-5	BR/GR	96	95	92	76	65	38	24	A-6(13)	S187	33.1

comments: W=MULTIPLE LAYERS, X=STRIPPED

Tuesday, March 28, 2017

JOB: 100840
JOB NAME: DITCH NOS. 1 & 47 STR. & APPRS. (S)
COUNTY NO. 56

Arkansas State Highway Transporation Department
Materials Division
Michael Benson, Materials Engineer

DATE TESTED
3/9/2017

PAVEMENT SOUNDINGS

STA.#	LOC.		
202+00	16 RT	ACHMSC ---	AGG. BASE CRS. CL-5 ---
202+00	05RT	ACHMSC **6.0WX**	AGG. BASE CRS. CL-5 **12.0**
210+00	05 LT	ACHMSC **7.0WX**	AGG. BASE CRS. CL-5 **10.0**
220+00	05 LT	ACHMSC **6.75W**	AGG. BASE CRS. CL-5 **10.0**
228+00	16 LT	ACHMSC ---	AGG. BASE CRS. CL-5 ---
228+00	05 LT	ACHMSC **4.5W**	AGG. BASE CRS. CL-5 **13.0**

comments: W=MULTIPLE LAYERS, X=STRIPPED

Tuesday, March 28, 2017

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT - LITTLE ROCK, ARKANSAS
MATERIALS DIVISION
MICHAEL BENSON, MATERIALS ENGINEER
*** SOIL SURVEY / PAVEMENT SOUNDING TEST REPORT ***

DATE - 03/10/17
JOB NUMBER - 100840
FEDERAL AID NO.- TO BE ASSIGNED
PURPOSE - SOIL SURVEY SAMPLE
SPEC. REMARKS - NO SPECIFICATION CHECK
SUPPLIER NAME - STATE
NAME OF PROJECT - DITCH NOS. 1 & 47 STR. & APPRS. (S)
PROJECT ENGINEER - NOT APPLICABLE
PIT/QUARRY - ARKANSAS
LOCATION - POINSETT, COUNTY
SAMPLED BY - DICKERSON/FRAZIER
SAMPLE FROM - TEST HOLE
MATERIAL DESC. - SOIL SURVEY - R VALUE- PAVEMENT SOUNDINGS

SEQUENCE NO. - 1
MATERIAL CODE - SSRVPS
SPEC. YEAR - 2014
SUPPLIER ID. - 1
COUNTY/STATE - 56
DISTRICT NO. - 10

DATE SAMPLED - 02/28/17
DATE RECEIVED - 03/07/17
DATE TESTED - 03/09/17

LAB NUMBER	20170725	20170726	20170727
SAMPLE ID	S182	S183	S184
TEST STATUS	INFORMATION ONLY	INFORMATION ONLY	INFORMATION ONLY
STATION	202+00	202+00	210+00
LOCATION	05RT	16 RT	05 LT
DEPTH IN FEET	0-5	0-5	0-5
MAT'L COLOR	BR/GR	GR/BR	BR/GR
MAT'L TYPE			
LATITUDE DEG-MIN-SEC	35 32 54.60	35 32 54.40	35 32 54.90
LONGITUDE DEG-MIN-SEC	90 21 47.80	90 21 47.80	90 21 38.20
% PASSING 2 IN.			
1 1/2 IN.			
3/4 IN.		100	
3/8 IN.	100	98	100
NO. 4	96	92	96
NO. 10	92	86	92
NO. 40	86	78	87
NO. 80	81	72	74
NO. 200	78	68	64
LIQUID LIMIT	57	51	42
PLASTICITY INDEX	40	34	29
AASHTO SOIL	A-7-6(31)	A-7-6(21)	A-7-6(15)
UNIFIED SOIL			
% MOISTURE CONTENT	44.3	39.4	23.8
ACHMSC (IN)	6.0WX	---	7.0WX
AGG. BASE CRS. CL-5 (IN)	12.0	---	10.0

REMARKS - W=MULTIPLE LAYERS, X=STRIPPED

AASHTO TESTS : T24 T88 T89 T90 T265

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT - LITTLE ROCK, ARKANSAS
MATERIALS DIVISION
MICHAEL BENSON, MATERIALS ENGINEER
*** SOIL SURVEY / PAVEMENT SOUNDING TEST REPORT ***

DATE - 03/10/17
JOB NUMBER - 100840
FEDERAL AID NO.- TO BE ASSIGNED
PURPOSE - SOIL SURVEY SAMPLE
SPEC. REMARKS - NO SPECIFICATION CHECK
SUPPLIER NAME - STATE
NAME OF PROJECT - DITCH NOS. 1 & 47 STR. & APPRS. (S)
PROJECT ENGINEER - NOT APPLICABLE
PIT/QUARRY - ARKANSAS
LOCATION - POINSETT, COUNTY
SAMPLED BY - DICKERSON/FRAZIER
SAMPLE FROM - TEST HOLE
MATERIAL DESC. - SOIL SURVEY - R VALUE- PAVEMENT SOUNDINGS

SEQUENCE NO. - 2
MATERIAL CODE - SSRVPS
SPEC. YEAR - 2014
SUPPLIER ID. - 1
COUNTY/STATE - 56
DISTRICT NO. - 10

DATE SAMPLED - 02/28/17
DATE RECEIVED - 03/07/17
DATE TESTED - 03/09/17

LAB NUMBER	20170728	20170729	20170730
SAMPLE ID	S185	S186	S187
TEST STATUS	INFORMATION ONLY	INFORMATION ONLY	INFORMATION ONLY
STATION	220+00	228+00	228+00
LOCATION	05 LT	05 LT	16 LT
DEPTH IN FEET	0-5	0-5	0-5
MAT'L COLOR	BR/GR	BR/GR	BR/GR
MAT'L TYPE			
LATITUDE DEG-MIN-SEC	35 32 51.30	35 32 52.50	35 32 52.60
LONGITUDE DEG-MIN-SEC	90 21 26.90	90 21 17.40	90 21 17.40
% PASSING 2 IN.			
1 1/2 IN.			
3/4 IN.	100	100	100
3/8 IN.	98	99	98
NO. 4	93	96	96
NO. 10	91	90	95
NO. 40	86	81	92
NO. 80	76	66	76
NO. 200	68	56	65
LIQUID LIMIT	46	40	38
PLASTICITY INDEX	31	28	24
AASHTO SOIL	A-7-6(19)	A-6(12)	A-6(13)
UNIFIED SOIL			
% MOISTURE CONTENT	23.8	34.3	33.1
ACHMSC (IN)	6.75W	4.5W	---
AGG. BASE CRS. CL-5 (IN)	10.0	13.0	---

REMARKS - W=MULTIPLE LAYERS, X=STRIPPED

AASHTO TESTS : T24 T88 T89 T90 T265

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT - LITTLE ROCK, ARKANSAS
MATERIALS DIVISION
MICHAEL BENSON, MATERIALS ENGINEER
*** SOIL SURVEY / PAVEMENT SOUNDING TEST REPORT ***

DATE	- 03/10/17	SEQUENCE NO.	- 1
JOB NUMBER	- 100840	MATERIAL CODE	- RV
FEDERAL AID NO.	- TO BE ASSIGNED	SPEC. YEAR	- 2014
PURPOSE	- SOIL SURVEY SAMPLE	SUPPLIER ID.	- 1
SPEC. REMARKS	- NO SPECIFICATION CHECK	COUNTY/STATE	- 56
SUPPLIER NAME	- STATE	DISTRICT NO.	- 10

NAME OF PROJECT - DITCH NOS. 1 & 47 STR. & APPRS. (S)
PROJECT ENGINEER - NOT APPLICABLE
PIT/QUARRY - ARKANSAS

LOCATION	- POINSETT, COUNTY	DATE SAMPLED	- 02/28/17
SAMPLED BY	- DICKERSON/FRAZIER	DATE RECEIVED	- 03/07/17
SAMPLE FROM	- TEST HOLE	DATE TESTED	- 03/09/17

MATERIAL DESC. - SOIL SURVEY - RESISTANCE R-VALUE ACTUAL RESULTS

LAB NUMBER	20170731	-	-
SAMPLE ID	RV188	-	-
TEST STATUS	INFORMATION ONLY	-	-
STATION	228+00	-	-
LOCATION	16 LT	-	-
DEPTH IN FEET	0-5	-	-
MAT'L COLOR	BR/GR	-	-
MAT'L TYPE		-	-
LATITUDE DEG-MIN-SEC	35 32 52.60	-	-
LONGITUDE DEG-MIN-SEC	90 21 17.40		
% PASSING 2 IN.		-	-
1 1/2 IN.		-	-
3/4 IN.	100	-	-
3/8 IN.	91	-	-
NO. 4	89	-	-
NO. 10	88	-	-
NO. 40	84	-	-
NO. 80	75	-	-
NO. 200	60		
LIQUID LIMIT	41	-	-
PLASTICITY INDEX	26	-	-
AASHTO SOIL	A-7-6(12)	-	-
UNIFIED SOIL		-	-
% MOISTURE CONTENT		-	-

REMARKS - W=MULTIPLE LAYERS, X=STRIPPED

AASHTO TESTS : T24 T88 T89 T90 T265

GEOTECHNICAL EXPLORATION
DITCH NOS. 1 & 47 STRS. & APPRS. (S))
POINSETT COUNTY, ARKANSAS

ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT NO. 100840
FAP NO. NHPP-0046(50)

Prepared for:

GARVER, LLC
NORTH LITTLE ROCK

Prepared by:

GEOTECHNOLOGY, INC.
JONESBORO, ARKANSAS

Date:

JUNE 11, 2020

Geotechnology Project No.:

J034298.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

GEOTECHNOLOGY INC
FROM THE GROUND UP

June 11, 2020

Mr. John Ruddell, P.E., S.E.
Vice President - Bridge Design Manager
Garver, LLC
4701 Northshore Drive
North Little Rock 72118

Re: Geotechnical Exploration
Ditch Nos. 1 & 47 Strs. & Apprs. (S))
Poinsett County, Arkansas
Geotechnology Project No. J034298.01

Dear Mr. Ruddell:

Presented in this report are the results of the geotechnical exploration performed by Geotechnology, Inc. for the referenced project. The report includes our understanding of the project, observed site conditions, conclusions and/or recommendations, and support data as listed in the Table of Contents.

We appreciate the opportunity to provide geotechnical services for this project. If you have any questions regarding this report, or if we can be of any additional service to you, please do not hesitate to contact us.

Respectfully submitted,
GEOTECHNOLOGY, INC.

Dale M. Smith

Dale M. Smith, P.E.
Geotechnical Manager

Duncan Adrian, P.E.
Project Manager

ALY/DMS/DBA/ASE:aly/dba

Copies submitted: Via email

STATE OF ARKANSAS
LICENSED PROFESSIONAL ENGINEER
No. 17917
Dale M. Smith

Dale M. Smith

6/11/20

3312 Winbrook Drive | Memphis, Tennessee 38116
(901) 353-1981 | Fax: (901) 353-2248 | geotechnology.com

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LIST OF TABLES

GEOTECHNICAL EXPLORATION
DITCH NOS. 1 & 47 STRS. & APPRS. (S))
POINSETT COUNTY, ARKANSAS
June 11, 2020 | Geotechnology Project No. J034298.01

CHAPTER 1. SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design, construction, and other related features for the proposed Route 308 improvements in Poinsett County, Arkansas (Station 202+00 to Station 229+33.71). The referenced improvements consist of the construction of an approximately 308-foot-long, 6-span bridge (Station 212+57.00 to Station 215+65.12) to replace both the existing approximately 87-foot long bridge over Ditch No. 1 and the existing approximately 168-foot long bridge over Ditch No. 47. The existing bridge approaches will be modified to facilitate traffic flow over the new bridge. A general overview of the project is shown on Figure 1 included in Appendix B.

The recommendations presented in this report are based on the geology, topography, and the results of the geotechnical exploration. Results of borings, in-situ testing, sampling, cone penetration testing, and laboratory testing are included in the report. The collected data have been analyzed and the physical properties of the in-situ soils summarized. General site conditions are discussed, along with recommendations for subgrade preparation. Important information prepared by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association for studies of this type is presented in Appendix A for your review.

CHAPTER 2. GENERAL INFORMATION

Planned Modifications

The existing Hwy. 308 bridges over Ditch Nos 1 & 47 will be replaced with an approximately 302-foot-long, 6-span bridge. The replacement bridge will be constructed south of the existing bridges, approximately 55 feet centerline to centerline. The existing approaches will be modified to facilitate traffic over the new bridge. It is our understanding the old bridges will be demolished following completion of the new bridge and approaches. Based on the provided plans[1], we have assumed a maximum of approximately 11.5 feet of fill will be required at the west abutment and approximately 5 feet of fill will be required at the east abutment to bring the approaches to design grade. Based on the plans, the fill will be placed on top of the existing embankment at a slope of 2 horizontal units to 1 vertical unit (2H:1V) at the west abutment. The eastern abutment appears to slope approximately 2.5H:1V. Cross sections for the side slopes were not provided.

Topography

According to the provided plans, across the proposed alignment the elevation varies from approximately El 229[2] to 204, a maximum of 25 feet of relief.

Drainage

The drainage system in the project area consists of the Lower St. Francis Watershed. The Lower St. Francis Watershed, in turn, is part of the overall drainage system of the Mississippi River Basin.

Geology

Poinsett County is located in southeastern Arkansas, in the Mississippi Embayment. The Mississippi Embayment is a trough-like depression plunging southward along an axis approximating the present course of the Mississippi River. Geology in the project area is characterized by alluvial, clay, silt, and sand deposits.

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[1] *Arkansas State Highway and Transportation Department Construction Plans for State Highway Ditch Nos. 1 & 47 Strs. & Apprs. (S) Poinsett County Route 308 Section 1 Job 100840, Federal Aid Proj.*, dated April 5, 2019.

[2] All elevations herein are in feet and referenced to Mean Sea Level (MSL).

CHAPTER 3. GEOTECHNICAL EXPLORATION

Cone Penetrometer Testing

Two cone penetrometer test (CPT) soundings were performed at the existing bridge approaches. The location of the soundings, designated as CPT-1 and -2, are shown on Figure 2 in Appendix B.

The CPT soundings were advanced using a 20-ton, track-mounted Vertek direct-push rig on May 6th, 2019. The data was collected using a Vertek 15 square-centimeter end area, seismic piezometric cone with a u_2 pore pressure location (behind the cone). Plots of the CPT measurements are presented in Appendix C along with interpreted soil behavior types. Seismic cone penetration tests (SCPT) were performed at approximately 3-foot depth intervals in Soundings CPT-1 and -2 to collect shear wave velocity data. A plot of shear wave velocity measurements versus depth is in Table 1 and 2 of the site-specific seismic study in Appendix F.

Rotary Drilling and Soil Sampling

Three borings were drilled at the existing bridge approaches and in the small section of pavement between the two existing bridges. No borings were made through the existing bridges. The boring locations, designated as Borings B-1 through -3, are shown on Figure 2 in Appendix B.

The borings were drilled between May 15th and 17th, 2019 using a rotary drill rig (CME 750) with hollow-stem augers and wash-rotary methods to depths of approximately 50 and 100 feet. Wash rotary drilling methods were utilized in Borings B-1 and -2. Sampling procedures included Standard Penetration Test (SPT) and thin-wall (Shelby) tube methods. SPT's were conducted at 2.5 and 5-foot depth intervals using automatic hammers. Thin-walled Shelby tube samples were collected in cohesive soils at selected depths. Groundwater observations were made during drilling operations.

The collected samples were visually examined by field staff and transported to our laboratory for further evaluation and testing. The samples were examined in the laboratory by a project geotechnical engineer who prepared descriptive logs of the materials encountered. The boring logs are presented in Appendix C. An explanation of the terms and symbols used on the boring logs is also provided in Appendix C. The boring elevations provided on the logs were estimated using the provided plans. Included in Table 1 are in situ tests and measurements made as part of the fieldwork and recorded on the boring logs.

Table 1. Field Tests and Measurements

Item	Test Method
Description and Identification of Soils (Visual-Manual Procedures)	ASTM D 2488/ D 3282
Electric Friction Cone and Piezocone Penetration Testing	ASTM D 5778
Standard Penetration Test (SPT)	ASTM D 1586/ AASHTO T206
Thin-Walled (Shelby) Tube Sampling	ASTM D 1587/ AASHTO T207

The boring logs represent conditions observed at the time of exploration and have been edited to incorporate results of the laboratory tests. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials could be gradual or could occur between recovered samples. Stratification lines on the boring logs indicate approximate changes in strata. The transition between strata could be abrupt or gradual. The stratification given on the boring logs, or described herein, is for use by Geotechnology in its analyses and should not be used as the basis of design or construction cost estimates without realizing that there can be variation from that shown or described.

The boring logs and related information depict subsurface conditions only at the specific locations and times where sampling was conducted. The passage of time could result in changes in conditions, interpreted to exist, at or between the locations where sampling was conducted.

CHAPTER 4. LABORATORY REVIEW AND TESTING

Laboratory testing was performed on soil samples to assess engineering and index properties. Most of the laboratory test results are presented on the boring logs in Appendix C. The Atterberg limits, grain size analyses, unconsolidated-undrained triaxial compression (UU), consolidated-undrained triaxial compression (CU), one-dimensional consolidation, pH, resistivity test results are also provided in Appendix D. The laboratory tests and corresponding test method standards are presented in Table 2.

Table 2. Summary of Laboratory Tests and Methods.

Laboratory Test	ASTM	AASHTO
Moisture Content	D 2216	T 265
Atterberg Limits	D 4318	T 98
Grain Size Analysis	D 6913	T 88
Particle Size by Hydrometer	D 7928	T 88
Unconsolidated-Undrained Triaxial Compression	D 2850	T 296
Consolidated-Undrained Triaxial Compression	D 4767	T 297
One-Dimensional Consolidation	D 2435	T 216
pH of Soil	D 4972	T 289
Soil Electrical Resistivity	G 57	T 288

CHAPTER 5. SUBSURFACE CONDITIONS

Existing Pavement

The borings were drilled through the pavement of the existing approaches and in the small pavement area between the two existing bridges. A summary of the pavement materials and thicknesses is provided in Table 3.

Table 3. Summary of Encountered Pavement Materials and Thicknesses.

Boring No.	Surface		Base	
	Material	Thickness (in.)	Material	Thickness (in.)
B-1	Asphalt	2	Clayey Sand	10
			Clayey Gravel	30
B-2	Asphalt	2	--	--
	Concrete	10		
B-3	Asphalt	2	Silt	10

Subgrade Materials

Underlying the pavement, the soils generally consisted of fine-grained, predominately clay soil underlain by coarse-grained soil to the 100-foot maximum depth of exploration. The CPT sounding soil interpretations and the borings logs are included in Appendix C. A summary of the AASHTO and USCS classifications is presented in Appendix E.

The fine-grained, predominately clay soils were classified as low plasticity, “lean” clay (CL) and high plasticity, “fat” clay (CH), AASHTO A-7-6, with some silt (ML) AASHTO A-4. The fine-grained soils ranged in consistency from soft to stiff.

The fine-grained soils were underlain by coarse-grained soil at depths of 18 to 23 feet and classified as poorly-graded sand (SP), AASHTO A-3 and sand with silt (SP-SM), AASHTO A-1-b and A-3. Based on field test results, the coarse-grained soils ranged from medium dense to very dense.

Groundwater

Groundwater was encountered during drilling operations in Boring B-3 at a depth of approximately 25 feet. Based on the pore water pressure data from CPT-1 and -2, groundwater was encountered at an approximate depth of 23 feet. Groundwater was not encountered in Borings B-1 and -2, but may have been obscured by the use of mud rotary drilling methods, which introduces fluid to the borehole. Groundwater levels could vary significantly over time due to water levels in Ditch Nos. 1 & 47, the effects of seasonal variation in precipitation, recharge, or other factors not evident at the time of exploration.

CHAPTER 6. ENGINEERING EVALUATION, ANALYSIS, AND RECOMMENDATIONS

Site Preparation and Earthwork

The following procedures are recommended for site preparation in cut and fill areas. These recommendations do not supersede ARDOT standards and specifications. Site preparation and compaction requirements must conform to the latest ARDOT standards.

Site Preparation. In general, cut areas and areas to receive new fill should be stripped of topsoil, vegetation, and other deleterious materials. Topsoil should be placed in landscape areas or disposed of off-site. Vegetation and tree roots should be over-excavated.

The exposed subgrade should be proof-rolled using a tandem axle dump truck loaded to approximately 20,000 pounds per axle (or equivalent proof-rolling equipment). Soft areas that develop should be over-excavated and backfilled with select fill, which is defined as soil conforming to A-4 or better material, and compacted to the unit weights specified in subsequent paragraphs.

Side Slopes. Existing slopes steeper than 4H:1V must be benched prior to placing new fill. Slope ratios of 3H:1V or flatter are recommended for all cut and fill slopes along the proposed alignment. Based on the results of the global stability analyses, discussed in a subsequent section, some slopes will require either flattening or geosynthetic reinforcement.

Cut Areas. Based on the stratigraphy, excavation will terminate in fat clay, lean clay, or silt. After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 95% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D 698/AASHTO T 99). Areas supporting pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

Fill Materials. Fill material should consist of natural soils classifying as AASHTO A-6 or better. Soils classifying as AASHTO A-4 or better are considered to be select fill. Fine-grained soils (A-4 through A-6) and coarse-grained soils with fines should have a maximum LL of 45 and a PI between 5 and 20 percent. Such materials should be free from organic matter, debris, or other deleterious materials, and have a maximum particle size of 2 inches.

Fill and Backfill Placement. Fill and backfill should be placed in level lifts, up to 8 inches in loose thickness. For fill and backfill exhibiting a well-defined moisture-density relationship, each lift should be moisture-conditioned to within ±2% of the optimum moisture content and compacted with a sheepsfoot roller of self-propelled compactor to a minimum of 98% of the maximum dry unit weight as determined by the standard Proctor test. Moisture-conditioning can include: aeration and drying of wetter soils; wetting drier soils; and/or mixing wetter and drier soils into a uniform blend. The upper three feet of soil beneath the base of pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

For fill and backfill that do not exhibit a well-defined moisture-density relationship, each lift should be compacted to 70% of the minimum relatively density as evaluated from the maximum and minimum index densities measured by ASTM D4253 and D4254, respectively. The upper 3 feet of soil beneath the base of pavement should be compacted to 75% of the minimum relatively density.

Moisture Considerations. Maintaining the moisture content of bearing and subgrade soils within the acceptable range is important during and after construction for the proposed structures. The silty and clayey bearing and subgrade soils should not be allowed to become wet or dry during or after construction, and measures should be taken to hinder water from ponding on these soils and to reduce drying of these soils.

Water from surface runoff, downspouts, and subsurface drains should be collected and discharged through a storm water collection system. Positive drainage should be established around the proposed structures to promote drainage of surface water away from the structures and reduce ponding of water adjacent to these structures.

Seismic Considerations

Earthquake Risk. The project area is located within the New Madrid Seismic Zone (NMSZ). The NMSZ is located in the northern part of the Mississippi Embayment and trends in a northeast to southwest direction from southern Illinois to northeast Arkansas. In December 1811, a series of large magnitude earthquakes occurred, which were centered near New Madrid, Missouri. Three strong earthquakes occurred over the next three months and smaller aftershocks continued until at least 1817. According to researchers, the magnitudes of these three events ranged from 7.5 to 8.0.

Earthquake Forces. It is our understanding the bridge and approaches will be designed in accordance with the AASHTO publication “LRFD Bridge Design Specifications”, seventh edition (2014), with 2016 interim revisions.

Seismic Design Parameters. A site-specific seismic study was conducted using the shear wave velocity profiles obtained in CPT-1 and -2. The process included downhole testing in the CPT soundings to determine near surface shear wave velocities, performing probabilistic seismic hazard analyses, generating synthetic time histories, and evaluating near surface soil effects. The average shear-wave velocity measured at the CPT locations was 735 feet per second (ft/sec). Accordingly, the site is classified as Site Class D, Stiff Soil Profile. Site-specific design spectral acceleration coefficients were calculated for a seismic hazard with 7% probability of exceedance in 75 years. The result of the site-specific seismic study is presented in the following table. The full report is presented in Appendix F.

Seismic design spectral accelerations were estimated for the site using two methods; a code-based approach and a site-specific approach; both are presented in Table 4.

Table 4. Seismic Design Parameters (Probability of Exceedance of 7% in 75 Years).

Parameter	Site-Specific Design Value	Code-Based Values
S_{DS}	1.244g	1.824g
S_{D1}	0.952g	0.770g
As	0.680g	1.020g

Liquefaction and Dynamic Settlement. A study was performed to evaluate the liquefaction and dynamic settlement potential at the site using both the SPT borings and the CPT soundings using an earthquake magnitude (Mw) of 7.7 with a probability of exceedance of 7% in 75 years was considered. A peak ground acceleration of 0.680g was utilized as obtained from the site-specific seismic study. Groundwater was assumed to be at a depth of approximately 23 feet for the analyses.

The SPT based analysis utilized both field and laboratory data which included the assumed depth of the water table, SPT N-values, USCS classifications, and estimated or measured soil unit weights. The CPT based analysis utilized the soundings taken at the locations of CPT-1 and -2, including the soil profile interpreted from the sounding, the measured groundwater depth, and the recorded pore pressure measurements.

Subsurface conditions (as characterized by field and laboratory data) and earthquake characteristics were used to estimate the safety factors against liquefaction in each soil layer, as well as the associated dynamic settlement during the design seismic event. Based on the analysis, there is liquefaction potential at the site. The analysis results are presented in Table 5.

Table 5. Results of Liquefaction Analyses.

Boring No.	Depth of Boring (ft.)	Zones with Liquefaction Factor of Safety Less than 1.0	Estimated Dynamic Settlement (in.)	
			Upper 50 Feet	Total Depth of Boring / CPT Sounding
B-1	50	23 to 33 feet | 38 to 43 feet | 48 to 50 feet	4	4
CPT-1	100	23 to 46 feet | 52 to 63 feet | 71 to 84 feet | 90 to 95 feet	4	6
B-2	100	23 to 28 feet | 33 to 43 feet | 58 to 68 feet	4	7
CPT-2	100	23 to 91 feet | 98 to 100 feet	6	12
B-3	50	38 to 48 feet	2	2

The current state of practice for liquefaction hazard assessment is based on what is known as “the Simplified Method” as introduced by Seed (1971) and subsequent modifications/revisions by many researchers (Seed 1982, Idriss 1999, Youd 2001, and Idriss and Boulanger 2014, among others). The simplified method was based on observations and assessments of soil zones that either liquefied or did not liquefy in the upper 40 feet (12 m). There are reported uncertainties in the values of one of the inputs to the method (the stress reduction factor, or r_d) at depths greater than 50 feet. The occurrence of significant liquefaction in relatively deep sand deposits is unlikely. Therefore, we recommend not considering potentially liquefiable zones below a depth of 50 feet when determining pile embedment lengths.

A discussion of the downdrag potential due to dynamic settlement is included in a subsequent section.

Lateral Spreading. Lateral spreading is triggered and sustained by earthquake ground motions. Based on our seismic slope stability analyses, it is our professional opinion the potential for lateral spreading is low. However, after the earthquake, the soils that liquefy will have residual strengths which can have potentially destabilizing effects on the overlying slope. More information is provided in the global stability section of this report.

Approach Embankment Settlement

Based on the plans provided, it appears up to 11.5 feet of fill will be required at the proposed abutments to bring the site to grade. Up to 6 inches of settlement is estimated to occur under the weight of new fill placed at the bridge approaches and abutments.

Based on the one-dimensional consolidation tests performed, this settlement is expected to be essentially complete after 180 to 220 days.

It should be noted the one-dimensional consolidation test confines the drainage pathway during sample loading to one dimension, in the field drainage takes place in three dimensions; therefore, it is our professional opinion the estimated settlement will occur in a shorter time period. We recommend a settlement monitoring program be implemented and survey data be forwarded to Geotechnology so that construction can commence as soon as settlement is essentially completed. Note that piles may be driven immediately after fill placement if the pile lengths and configurations are based on the post-liquefaction pile capacities.

Settlement Monitoring Program. Settlement plates, or other appropriate methods should be utilized. Settlement plates should be installed approximately 1-foot below the existing ground surface and extended in 5-foot calibrated increments as the height of fill increases. To protect the riser pipes, fill should be hand compacted within a 4-foot radius of each plate. A typical settlement plate detail is presented on Figure 3 in Appendix B. We recommend settlement plates be placed no further than 50-feet apart, with at least one in the deepest areas of fill at both abutments. The project surveyor should be retained to monitor the settlement plate riser pipe. Settlement at the site should be measured twice weekly during fill placement and weekly after filling is completed. Further construction at the abutments should not commence until after the settlement due to the fill placement has dissipated. Driving piles prior to the dissipation

If the estimated settlement due to placement of the approach embankment is not tolerable, then consideration should be given to ground improvement techniques such as rammed aggregate piers.

Global Stability

Based on the provided plans, the west abutment fill will be placed at a 2H:1V slope on top of the existing 4.25H:1V slope and the east abutment will slope approximately 2.5H:1V. Geotechnology performed stability analyses for deep-seated, global failure of bridge abutment slopes using the computer program SLOPE/W. Short-term, long-term, seismic, and post-seismic (residual strength) conditions were considered using the Spencer method to compute factors of safety for the proposed slopes.

The models used in this computation did not consider the relative stabilizing effect of foundation piles driven to support the abutments or cladding of abutments with rip rap or concrete. In general, foundation piles may provide additional stabilizing force to the abutment slopes, resulting in a factor of safety higher than those presented here.

Calculated minimum factors of safety are summarized in the following table. A pseudo-static seismic acceleration of 0.34g, corresponding to one-half the peak ground acceleration (per FHWA Publication HI-99-012) was utilized for the seismic condition. This design horizontal pseudo-static seismic acceleration was further reduced based on the slope height to account for spatially varying ground motions (Anderson et al. 2008), which resulted in a seismic acceleration of about 0.32g. An estimated residual shear strength was used to model the potentially liquefiable sand for the post-seismic analysis. The ordinary high-water elevation, El 211, was used in the analyses. Section profiles with calculated critical failure arcs and utilized soil parameters are presented in Appendix G for the selected analyses.

Table 6. Results of Slope Stability Analyses.

Location	Slope Ratio	Slope Height (ft.)	Calculated Factor of Safety			
			Short-Term Static[a]	Long-Term Static[a]	Seismic[b]	Post-Seismic[b]
North Side Station 212+00	3:1 on top of 3:1	15	3.79	2.10	1.34	1.33
South Side Station 212+00	3:1	12 ½	3.28	1.80	1.31	1.36
West Abutment	2:1 on top of 4.25:1	19	2.91	1.62	1.19	1.16
East Abutment	2.5:1	25	2.40	*1.31*	1.14	1.1
North Side Station 216+00	3:1 on top of 2:1	18	4.08	1.95	1.22	1.50
South Side Station 216+00	3:1	19	2.80	1.62	1.13	1.37

[a] Target factor of safety = 1.5, approximately equivalent to a global stability resistance factor = 0.65.
[b] Target factor of safety = 1.1, approximately equivalent to a global stability resistance factor = 0.9.

An insufficient factor of safety (FOS) against global stability failure was computed for the long-term case for the east abutment. It should be noted the foundations are expected to have a stabilizing effect at the abutments and have not been modeled in the slope stability analyses. We understand that flattening the abutments or side slopes may not be feasible, if so, consideration should be given to geosynthetic reinforcement if the FOS is to be increased above the minimum values. Special attention should be paid to placement of the reinforcement with regards to the location of driven foundations. Regardless of ground improvement performed, the existing slopes should be benched prior to placing new fill to reduce the potential for development of slip planes between the new and existing fill.

Deep Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2014).

It is our understanding the proposed bridge abutments will be supported on driven 18-inch, closed-ended, steel pipe piles and the bents will be supported on driven, 24-inch, closed-ended, steel pipe piles. Prior to driving piles at Bents 3 and 4, a casing will be advanced around the proposed pile locations and soil inside the casing will be excavated down to a proposed pile cut-off elevation approximately matching the cut-off elevation at the other bents (Bents 2, 5 and 6). The purpose of the casing is to provide uniformity of the pile cut-off elevations at each bent. The lateral load of the soil and liquefaction potential above pile cut-off elevation should be neglected. Geotechnology should be notified if a different foundation type is to be considered.

Synthetic profiles have been compiled for each abutment and the bents locations based upon the soil information encountered in the borings, approximate boring elevations, and the proposed final grade. Soil parameters, including LPILE parameters, for each structure are included in Appendix H.

Presented in Appendix I are nominal resistance curves showing the resistance due to skin friction and total compression resistance (skin friction + end bearing) for the abutments and bents. Uplift capacities (tension) may be calculated using the skin friction resistance. Presented in Table 7 are nominal capacities for both static and post-liquefaction cases.

Table 7. Axial Pile Resistance – Static and Post-Liquefaction

Pile Diameter (inches)	Location	Embedment Length (feet)	Nominal Static Resistance (tons)			Nominal Post-Liquefaction Resistance (tons)	
			Skin Friction	End Bearing	Compression Total	Compression Total*	Drag Load
18	East Abutment	60	119	65	184	98	34
		70	159	64	223	138	34
		80	219	132	351	265	34
	West Abutment	60	126	65	191	99	31
		70	167	65	232	140	31
		80	229	132	361	269	31
24	Bents	60	176	235	411	299	36
		70	271	235	506	394	36
		80	381	236	617	505	36

*Nominal post-liquefaction resistance has not been reduced by the drag load

Resistance Factors. Resistance factors should be applied to the nominal resistances provided. In general, a resistance factor of 0.45 may be used for piles in compression and 0.35 in tension. A higher resistance factor may be used in accordance with the level of pile testing performed as indicated in Table 8 based on AASHTO LRFD (2014) Table 10.5.5.2.3-1.

Table 8. Resistance Factors for Driven Piles

Condition/Resistance Determination Method		Resistance Factor
Nominal Bearing Resistance of Single Pile – Dynamic Analysis and Static Load Test Methods	Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing of at least two piles per site, but no less than 2% of the production piles[a]	0.80
	Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing	0.75
	Driving criteria established by dynamic testing conducted on 100% of production piles*	0.75
	Driving criteria established by dynamic testing, quality control by dynamic testing of at least two piles per site condition, but no less than 2% of production piles[a]	0.65
	Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only)	0.40
Uplift Resistance of Single Pile	Dynamic test with signal matching	0.50

[a] Dynamic testing requires signal matching, and estimates of nominal resistance are made from a restrike. Dynamic tests are calibrated to a static load test, when available.

Pile Group Considerations. The settlement of pile groups should be evaluated as per AASHTO LRFD (2014) section 10.7.2.3. Settlement analysis of the pile groups can be performed when the foundation configurations and service loads are available. AASHTO LRFD (2014) section 10.7.3.9 addresses pile group resistance. Group capacity considerations for different pile groups, center-to-center spacings, and other conditions (cap contact with ground, softness of surface soil etc.) are given in AASHTO LRFD (2014) sections 10.7.3.9 and 10.7.3.11.

Driven Pile Construction Considerations. Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. A drivability analysis was performed using the program GRLWEAP Version 2010 produced by Pile Dynamics, Inc. to determine a maximum and minimum recommended open-ended, diesel hammer energy to drive the proposed piles. The unit skin friction and end-bearing values in soil were determined using energy corrected, standard penetration test (SPT) blow counts (i.e., N60) and the static analysis program in GRLWEAP. An 80 percent pile hammer efficiency and a shaft gain/loss factor of 0.8 and a toe gain/loss factor of 1.0 were used in the analysis. A maximum driving stress of 90 percent of the steel yield strength was used to identify the maximum hammer energy and a terminal driving resistance of 20 blows per inch was used to identify the minimum hammer energy. The resulting minimum and maximum hammer energy for the end and intermediate bents is provided in the following table.

Table 9. Results of Drivability Analyses – Maximum and Minimum Hammer Energy

Location	Hammer	Maximum Hammer Energy (kip-ft.)	Minimum Hammer Energy (kip-ft.)
End Bents	D-36	90	50
Interior Bents	D-62	155	70

We recommend a preconstruction wave equation analysis be performed on the actual hammer used during construction prior to production pile driving to confirm drivability. In addition, the results of the wave equation analysis rely on the accuracy of the input data and the validly of the mathematical models to predict the performance and dynamic response of the hammer, pile and soil systems. Therefore, dynamic monitoring should be performed during pile driving to confirm the assumptions used in the wave equation model.

Static Pile Load Testing. If static load testing is required, at least one static pile compression load test should be performed for each bent or abutment location. The testing should be performed in accordance with ASTM D 1143 using the quick loading procedure and AASHTO LRFD (2014) section 10.7.3.8.2. Please refer to the previous Resistance Factors table for additional guidance regarding the minimum number of tests and alternate resistance factors associated with other field methods for determining resistance.

If the piles are to support net uplift loads, at least one tension load test should be performed for each location. The test should be performed in accordance with ASTM D 3689. Piles should be tested to the required nominal uplift resistances.

Load tests are required to verify recommended nominal pile resistance and will not be used to increase the design pile resistance. The piles used in the load tests should not be used for support of any structures. Geotechnology should be consulted regarding the locations of the test piles.

Dynamic Testing of Driven Piles. As an alternative to static pile load testing, high-strain dynamic pile testing can be performed according to AASHTO LRFD (2014)) section 10.7.3.8.3 and the procedures given in ASTM D4945. Different resistance factors correspond to different load testing combinations as illustrated in Table 8.

We recommend that the test piles be identified according to AASHTO LRFD (2014) Table 10.5.5.2.3-1 or 2 percent of the production piles, whichever results in a larger number of tests. We recommend that the identified piles be tested at the end of initial drive (EOID) and a restrike performed at a minimum seven days after EOID.

Pile driving monitoring should be performed by an engineer with a minimum 3 years dynamic pile testing and analysis experience and who has achieved Basic or better certification under the High-Strain Dynamic Pile Testing Examination and Certification process of the Pile Driving Contractors Association and Foundation QA. Pile driving modeling and analyses should be performed by an engineer with a minimum five years dynamic pile testing and analysis experience and who has achieved Advanced or better certification under the High-Strain Dynamic Pile Testing Examination and Certification process of the Pile Driving Contractors Association and Foundation QA.

Dynamic tests are required to monitor hammer and drive system performance, assess driving stresses and structural integrity and to evaluate pile resistance, and should not be used to increase design pile resistance. Dynamic tests should be performed on production piles with the lowest driving resistance. Geotechnology will be available to assist with development of specifications for this program and should be on site to perform or observe the testing and establish the pile driving criteria.

Settlement. Settlement of pile foundations depends on the loads applied and the foundation configuration. In general, settlement of deep foundations designed in accordance with the recommendations provided in this report is expected to be less than 1-inch. However, a calculation of the expected settlement of the pile foundations can be performed when the applied service loads and foundation configuration are available.

Uplift Resistance. Uplift forces can be resisted by the effective weight of the piles and caps, and frictional resistance between the piles and surrounding soil. If the anticipated maximum level of groundwater is higher than the tip of the pile then the buoyant unit weight of the pile must be used in computing uplift resistance for pile lengths extending below the design groundwater level.

Lateral Resistance. The lateral resistance of pile foundations depends on the length and dimensions of the foundation and the soil characteristics. The lateral resistance of pile foundations can be computed using the computer program LPILE to model the behavior of a single pile or shaft. Soil parameters are provided in Appendix H for the various strata and soil strengths present at the site. Soil parameters are based on field and laboratory test results and empirical correlations with SPT N-values.

The effects of group interaction must be considered when evaluating pile/shaft group horizontal movement. The lateral resistance for individual piles calculated by LPILE must be reduced by the P-multipliers provided in Section 10.7.2.4 of the AASHTO LRFD (2014) to determine lateral resistance of a pile group. Alternatively, the GROUP software can be used to evaluate the lateral resistance of the pile/shaft groups. The resistance factor for lateral resistance of single piles or pile groups is 1.0.

Downdrag

The AASHTO LRFD (2014) suggests that settlement of 0.4-inch or greater could produce negative skin friction (downdrag) on pile foundations. Downdrag occurs as the soil strata move downward relative to the foundations due to settlement of the soil layers. The relative movement of the soil layers versus the pile depends on the final foundation configuration.

Downdrag Due to Fill-Induced Settlement. Based on settlement analysis performed for the 11.5-foot maximum fill placement at the abutments, up to 6-inches of settlement is predicted. It is our understanding that the pile lengths and configurations will be based on post-liquefaction pile capacities. Therefore, downdrag due to fill-induced settlement does not need to be considered and pile driving can begin immediately after fill placement.

Downdrag due to Dynamic Settlement. Based on liquefaction analysis results, we expect up to 6 inches of dynamic settlement within the upper 50 feet of soil during the design earthquake event (7% exceedance in 75 years). Additional pile analysis was performed to evaluate post-liquefaction pile capacities. The post-liquefaction pile capacities are presented in Table 7. Pre-drilling or applying bituminous or viscous coatings are not recommended to reduce liquefaction-induced downdrag because such methods will reduce the nominal static compressive resistance of the piles. If more information is desired, please contact Geotechnology.

CHAPTER 7. RECOMMENDED ADDITIONAL SERVICES

The conclusions and recommendations given in this report are based on: Geotechnology's understanding of the proposed design and construction, as outlined in this report; site observations; interpretation of the exploration data; and our experience. Since the intent of the design recommendations is best understood by Geotechnology, we recommend Geotechnology be included in the final design and construction process, and be retained to review the project plans and specifications to confirm the recommendations given in this report have been correctly implemented. We recommend Geotechnology be retained to participate in pre-bid and preconstruction conferences to reduce the risk of misinterpretation of the conclusions and recommendations in this report relative to the proposed construction of the subject project.

Since actual subsurface conditions between boring locations could vary from those encountered in the borings, our design recommendations are subject to adjustment in the field based on the subsurface conditions encountered during construction. Therefore, we recommend Geotechnology be retained to provide construction observation services as a continuation of the design process to confirm the recommendations in this report and to revise them accordingly to accommodate differing subsurface conditions. Construction observation is intended to enhance compliance with project plans and specifications. It is not insurance, nor does it constitute a warranty or guarantee of any type. Regardless of construction observation, contractors, suppliers, and others are solely responsible for the quality of their work and for adhering to plans and specifications.

CHAPTER 8. LIMITATIONS

This report has been prepared on behalf of, and for the exclusive use of, the client for specific application to the named project as described herein. If this report is provided to other parties, it should be provided in its entirety with all supplementary information. In addition, the client should make it clear the information is provided for factual data only, and not as a warranty of subsurface conditions presented in this report.

Geotechnology has attempted to conduct the services reported herein in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions. The recommendations and conclusions contained in this report are professional opinions. The report is not a bidding document and should not be used for that purpose.

Our scope for this phase of the project did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site. Any statements in this report or on the boring logs regarding odors noted or unusual or suspicious items or conditions observed are strictly for the information of our client. Our scope did not include an assessment of the effects of flooding and erosion of creeks or rivers adjacent to or on the project site.

Our scope did not include: any services to investigate or detect the presence of mold or any other biological contaminants (such as spores, fungus, bacteria, viruses, and the by-products of such organisms) on and around the site; or any services, designed or intended, to prevent or lower the risk of the occurrence of an infestation of mold or other biological contaminants.

The analyses, conclusions, and recommendations contained in this report are based on the data obtained from the geotechnical exploration. The field exploration methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Consequently, subsurface conditions could vary gradually, abruptly, and/or nonlinearly between sample locations and/or intervals.

The conclusions or recommendations presented in this report should not be used without Geotechnology's review and assessment if the nature, design, or location of the facilities is changed, if there is a lapse in time between the submittal of this report and the start of work at the site, or if there is a substantial interruption or delay during work at the site. If changes are contemplated or delays occur, Geotechnology must be allowed to review them to assess their impact on the findings, conclusions, and/or design recommendations given in this report. Geotechnology will not be responsible for any claims, damages, or liability associated with any other party's interpretations of the subsurface data or with reuse of the subsurface data or engineering analyses in this report.

The recommendations included in this report have been based in part on assumptions about variations in site stratigraphy that can be evaluated further during earthwork and foundation construction. Geotechnology should be retained to perform construction observation and continue its geotechnical engineering service using observational methods. Geotechnology cannot assume liability for the adequacy of its recommendations when they are used in the field without Geotechnology being retained to observe construction.

APPENDIX A – IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

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

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

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 constructor — 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 this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this 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.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many 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;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the 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 geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this 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 geotechnical-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 confirmation-dependent recommendations included in your report. *Confirmation-dependent 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 confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront 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. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical 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 Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors 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 constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors 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 constructors fail to 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.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental 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 environmental 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, many 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 GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GEOTECHNICAL BUSINESS COUNCIL
of the Geoprofessional Business Association

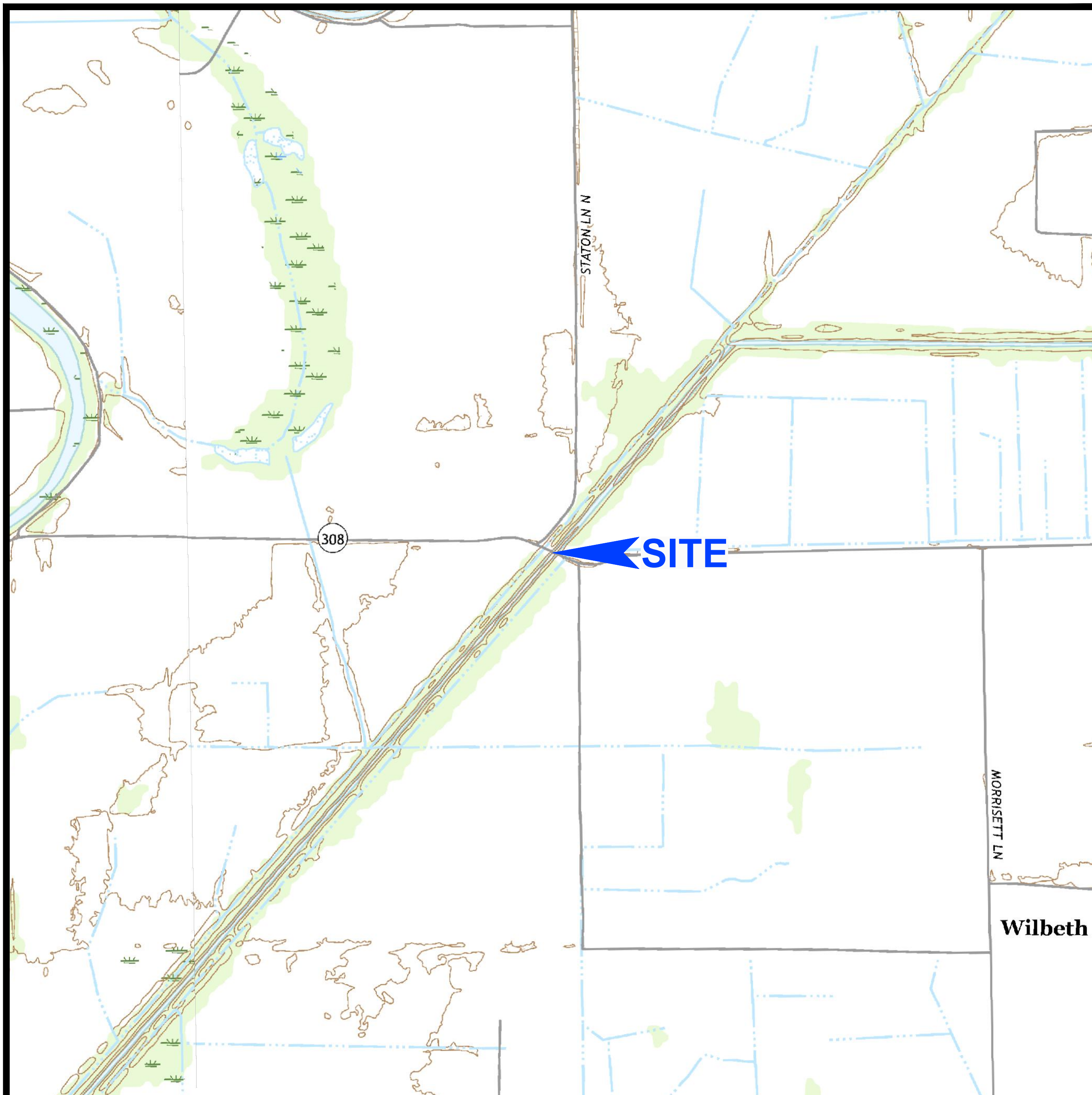
8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B – FIGURES

FIGURE1 - SITE LOCATION AND TOPOGRAPHY

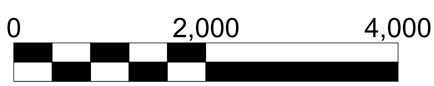
FIGURE 2 - AERIAL PHOTOGRAPH OF SITE AND BORING AND SOUNDING LOCATIONS

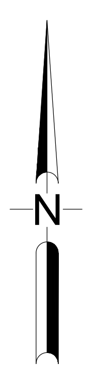
FIGURE 3 - SETTLEMENT PLATE DETAIL



NOTES

1. Plan adapted from 7.5 minute U.S.G.S. maps for Lepanto and Marked Trees, Arkansas quadrangles last revised in 2017.





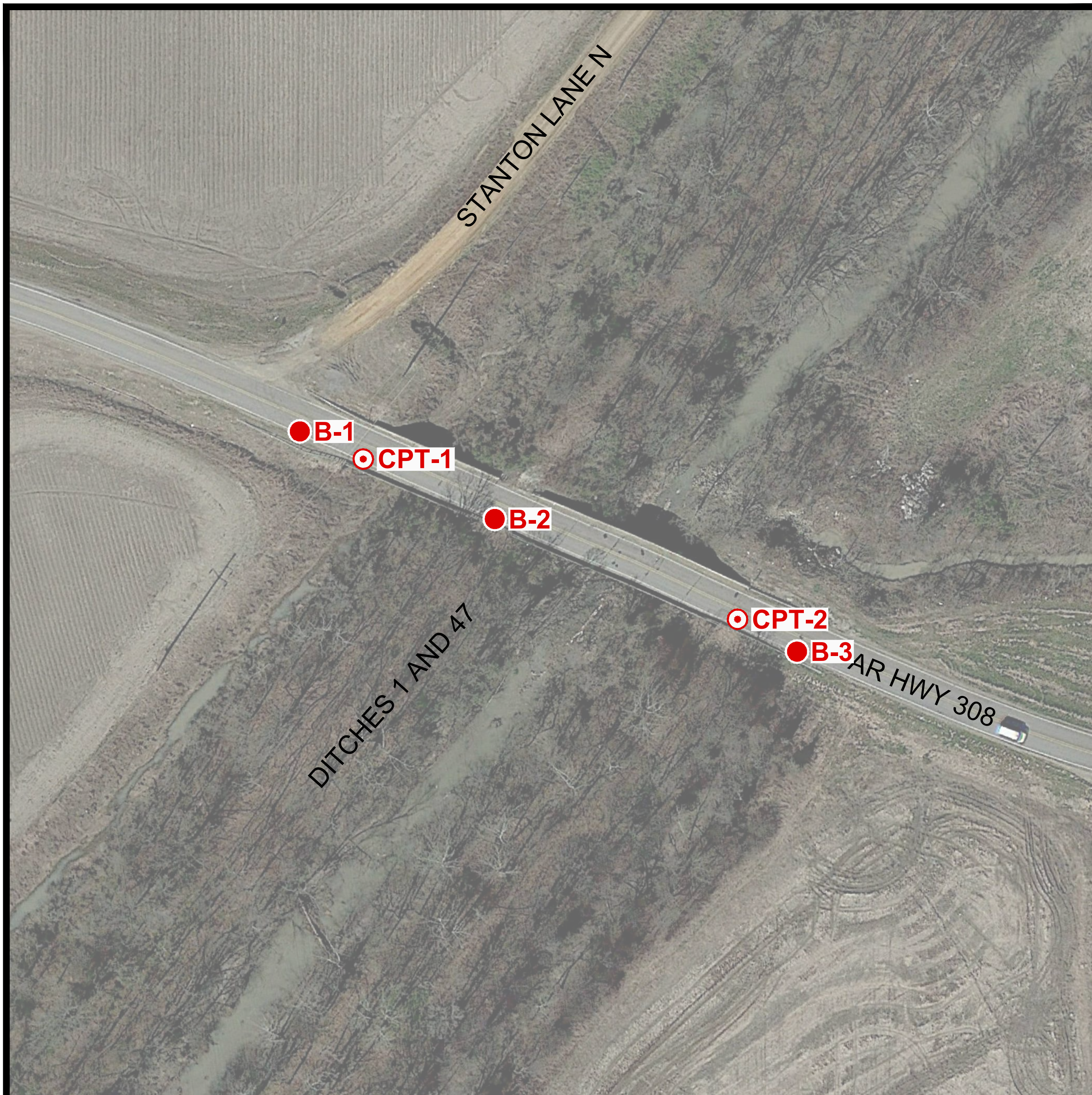
Drawn By: WAH	Ck'd By: ALY	App'vd By: DMS
Date: 7-1-19	Date: 7-1-19	Date: 7-2-19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Project No. 100840
Ditch Nos. 1 & 47 Strs. & Apprs.
Poinsett County, Arkansas

SITE LOCATION AND TOPOGRAPHY

Project Number J034298.01	FIGURE 1



NOTES

1. Plan adapted from a March 14, 2018 aerial photograph courtesy of Google Earth.

2. Borings and CPT Soundings were located in the field with reference to site features and are shown approximate only.

LEGEND

● Boring Location

⊙ CPT Sounding Location

0 100 200

SCALE IN FEET

N

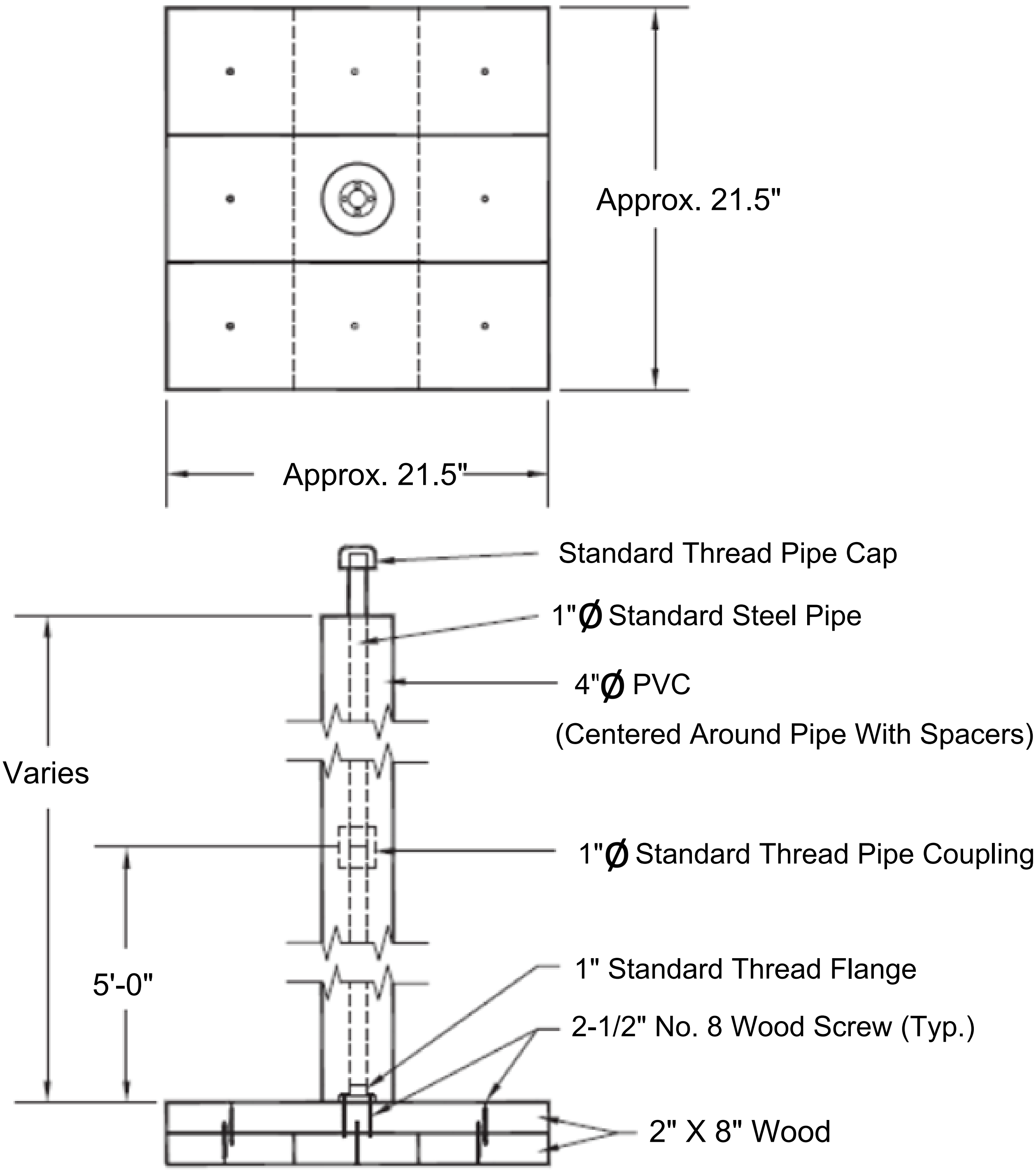
Drawn By: WAH	Ck'd By: ALY	App'vd By: DMS
Date: 7-1-19	Date: 7-1-19	Date: 7-2-19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Project No. 100840
Ditch Nos. 1 & 47 Strs. & Apprs.
Poinsett County, Arkansas

AERIAL PHOTOGRAPH OF SITE AND BORING LOCATIONS

Project Number J034298.01	FIGURE 2



NOTES

Place plate on level surface a minimum of 1 foot below ground level and hand compact backfill adjacent to PVC.

Drawn By: ALY	Ck'd By: DMS	App'vd By:
Date: 6-26-19	Date:6-27-19	Date:

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs.

SETTLEMENT PLATE DETAIL

Project Number J034298.01

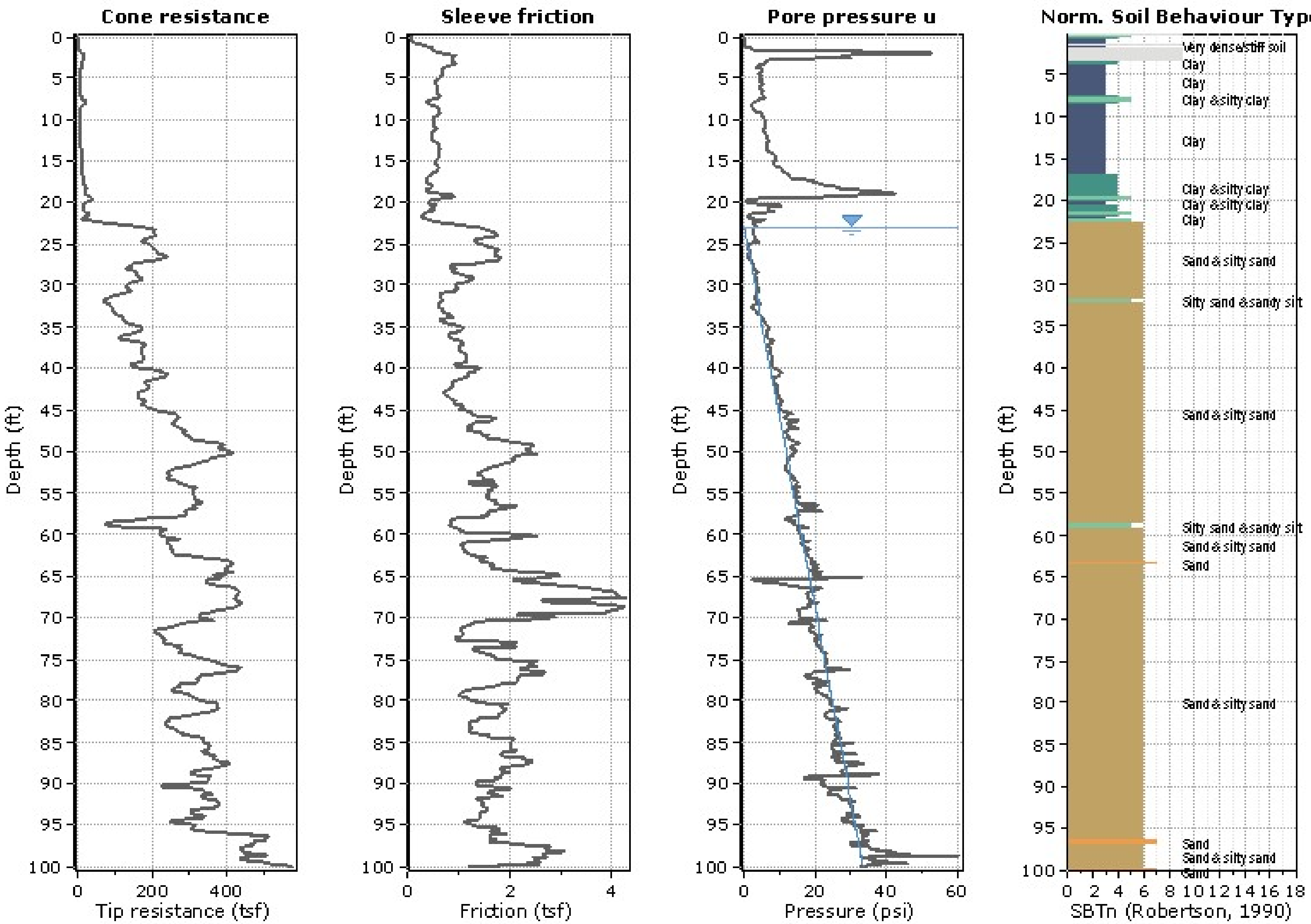
FIGURE 3

APPENDIX C – CPT SOUNDINGS AND BORING INFORMATION BORING LOG TERMS AND SYMBOLS BORING LOGS

Geotechnology, Inc
11816 Lackland Road
Maryland Heights, Missouri

CPT: CPT-1
Total depth: 100.01 ft, Date: 5/7/2019
Coords: lat 35.547905° lon -90.358642°
Cone Type: 15cm2
Cone Operator: DWJ

Project: **Ditches Nos. 1 & 47 Structures and Approaches**
Location: **Poinsett County, Arkansas**

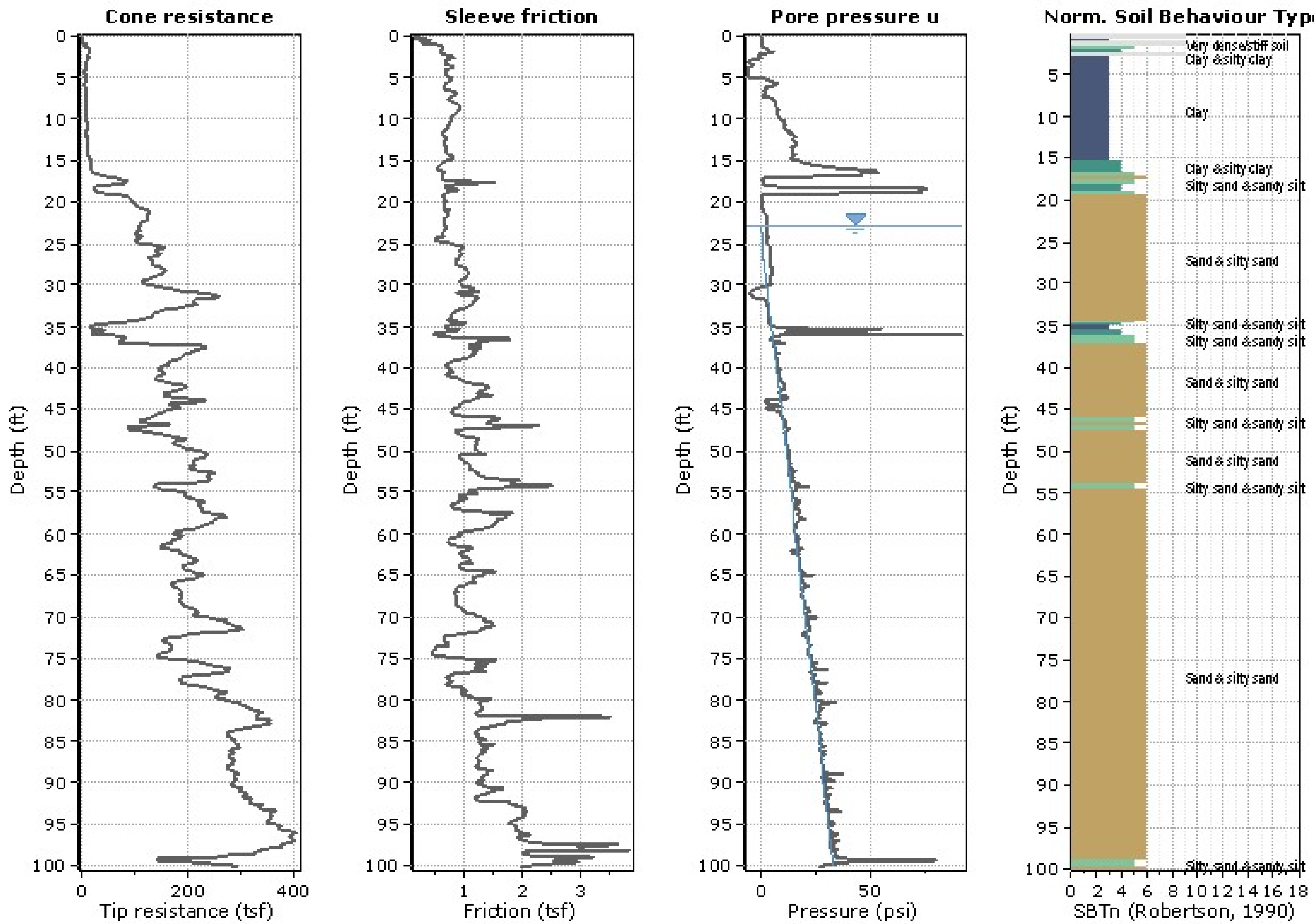


Geotechnology, Inc
11816 Lackland Road
Maryland Heights, Missouri

CPT: CPT-2

Total depth: 100.13 ft, Date: 5/7/2019
Coords: lat 35.548327° lon -90.359757°
Cone Type: 15cm2
Cone Operator: DWJ

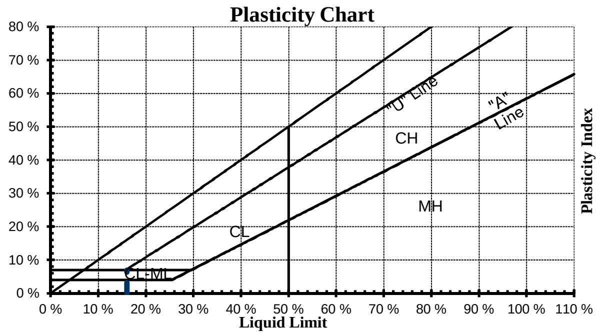
Project: **Ditches Nos. 1 & 47 Structures and Approaches**
Location: **Poinsett County, Arkansas**



BORING LOG: TERMS AND SYMBOLS

LEGEND

CS	Continuous Sampler
GB	Grab Sample
NQ	NQ Rock Core
PST	Three-Inch Diameter Piston Tube Sample
SS	Split-Spoon Sample (Standard Penetration Test)
ST	Three-Inch Diameter Shelby Tube Sample
*	Sample Not Recovered
PL	Plastic Limit (ASTM D4318)
LL	Liquid Limit (ASTM D4318)
SV	Shear Strength from Field Vane (ASTM D2573)
UU	Shear Strength from Unconsolidated-Undrained Triaxial Compression Test (ASTM D2850)
QU	Shear Strength from Unconfined Compression Test (ASTM D2166)



SOIL GRAIN SIZE

US STANDARD SIEVE

12" | 3" | 3/4" | 4 | 10 | 40 | 200

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

300 | 76.2 | 19.1 | 4.76 | 2.00 | 0.42 | 0.074 | 0.005

SOIL GRAIN SIZE IN MILLIMETERS

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			***Symbol***	***Description***
Coarse-Grained Soils (More than 50% Larger than No. 200 Sieve Size)	Gravel and Gravelly Soil	Clean Gravels Little or no Fines	GW	Well-Graded Gravel, Gravel- Sand Mixture
			GP	Poorly-Graded Gravel, Gravel-Sand Mixture
		Gravels with Appreciable Fines	GM	Silty Gravel, Gravel-Sand-Silt Mixture
			GC	Clayey-Gravel, Gravel-Sand-Clay Mixture
	Sand and Sandy Soils	Clean Sands Little or no Fines	SW	Well-Graded Sand, Gravelly Sand
			SP	Poorly-Graded Sand, Gravelly Sand
		Sands with Appreciable Fines	SM	Silty Sand, Sand-Silt Mixture
			SC	Clayey-Sand, Sand-Clay Mixture
Fine-Grained Soils (More than 50% Smaller than No. 200 Sieve Size)	Silts and Clays	Liquid Limit Less Than 50	ML	Silt, Sandy Silt, Clayey Silt, Slight Plasticity
			CL	Lean Clay, Sandy Clay, Silty Clay, Low to Medium Plasticity
			OL	Organic Silts or Lean Clays, Low Plasticity
	Silts and Clays	Liquid Limit Greater Than 50	MH	Silt, High Plasticity
			CH	Fat Clay, High Plasticity
			OH	Organic Clay, Medium to High Plasticity
	Highly Organic Soils		PT	Peat, Humus, Swamp Soil

STRENGTH OF COHESIVE SOILS

Consistency	***Undrained Shear Strength (tsf)***	***Unconfined Comp. Strength (tsf)***
Very Soft	less than 0.125	less then 0.25
Soft	0.125 to 0.25	0.25 to 0.5
Medium Stiff	0.25 to 0.5	0.5 to 1.0
Stiff	0.5 to 1.0	1.0 to 2.0
Very Stiff	1.0 to 2.0	2.0 to 3.0
Hard	greater than 2.0	greater than 4.0

DENSITY OF GRANULAR SOILS

Descriptive Term	***Approximate N_{60}-Value Range***
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	>50

N-Value (Blow Count) is the last two, 6-inch drive increments (i.e. 4/7/9, N = 7 + 9 = 16). Values are shown as a summation on the grid plot and shown in the Unit Dry Weight/SPT column.

RELATIVE COMPOSITION

Trace	0 to 10%
Little	10 to 20%
Some	20 to 35%
And	35 to 50%

OTHER TERMS

- Layer - Inclusion greater than 3 inches thick.
- Seam - Inclusion 1/8-inch to 3 inches thick
- Parting - Inclusion less than 1/8-inch thick
- Pocket - Inclusion of material that is smaller than sample diameter

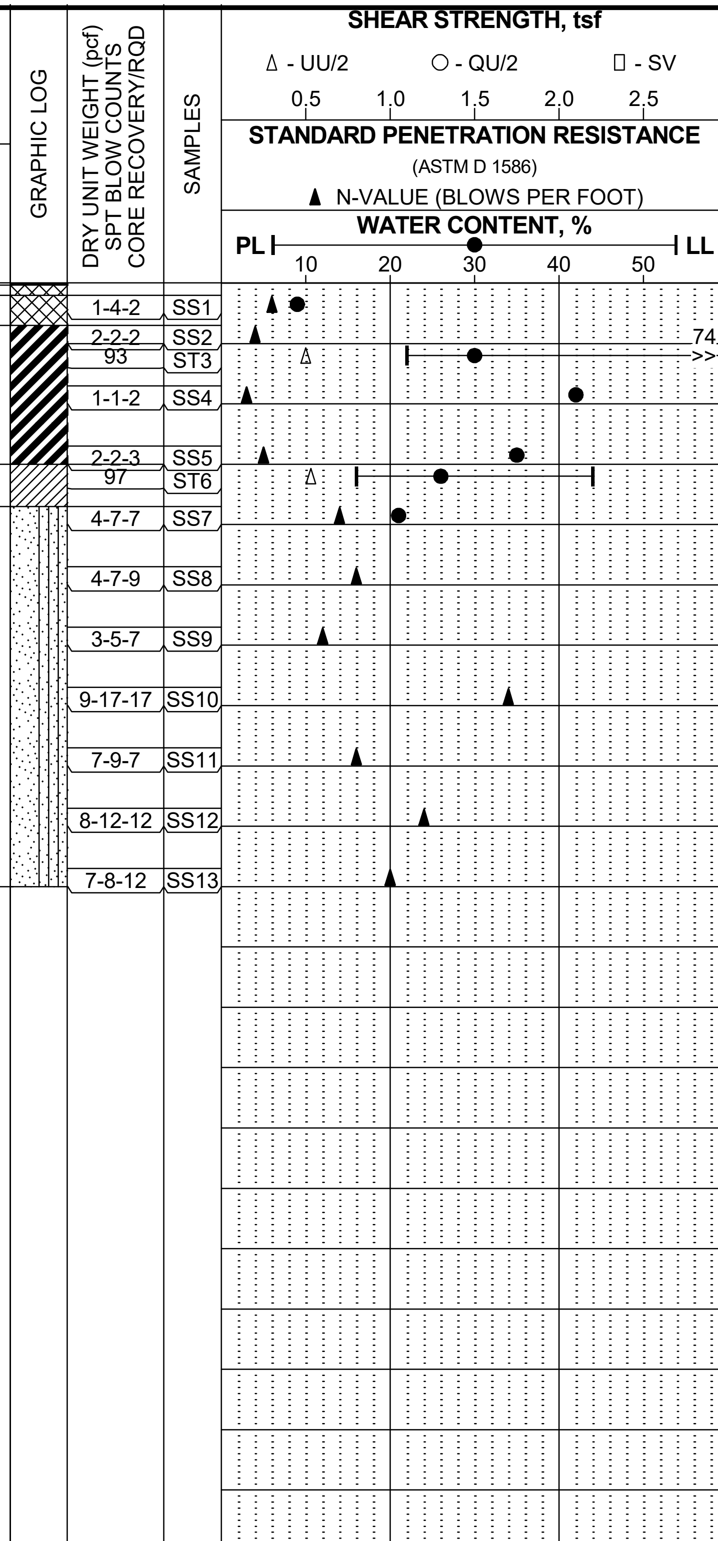
GEOTECHNOLOGY INC
FROM THE GROUND UP

Relative composition and Unified Soil Classification System (USCS) designations are based on visual descriptions and are approximate only. If laboratory tests were performed to classify the soil, the USCS designation is shown in parenthesis.

Surface Elevation: 222 | Completion Date: 5/15/19

Datum MSL

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) / SPT BLOW COUNTS / CORE RECOVERY/RQD	SAMPLES
		ASPHALT: 2 inches		
		FILL: brown and red CLAYEY SAND - SC	1-4-2	SS1
5	217	FILL: brown and red CLAYEY GRAVEL with sand - GC	2-2-2	SS2
			93	ST3
10	212	Soft to medium stiff, gray and brown FAT CLAY - (CH)	1-1-2	SS4
		trace sand		
15	207	trace sand	2-2-3	SS5
			97	ST6
		Stiff, gray and brown LEAN CLAY - (CL)		
20	202	Medium dense to dense, gray SAND WITH SILT - SP-SM	4-7-7	SS7
		Soil Resistivity = 5,130.00 ohms-cm		
25	197		4-7-9	SS8
30	192		3-5-7	SS9
35	187		9-17-17	SS10
40	182		7-9-7	SS11
45	177		8-12-12	SS12
50	172	Boring terminated at 50 feet.	7-8-12	SS13
55	167			
60	162			
65	157			
70	152			
75	147			
80	142			
85	137			
90	132			
95	127			
100	122			



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM

WASHBORING FROM 20 FEET

KJB DRILLER SAS LOGGER

CME 750 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 91 %

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by:
Date: 5/15/19	Date: 6/27/19	Date:

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT 100840
Ditch Nos 1 & 47 Strs. & Apprs. (S)

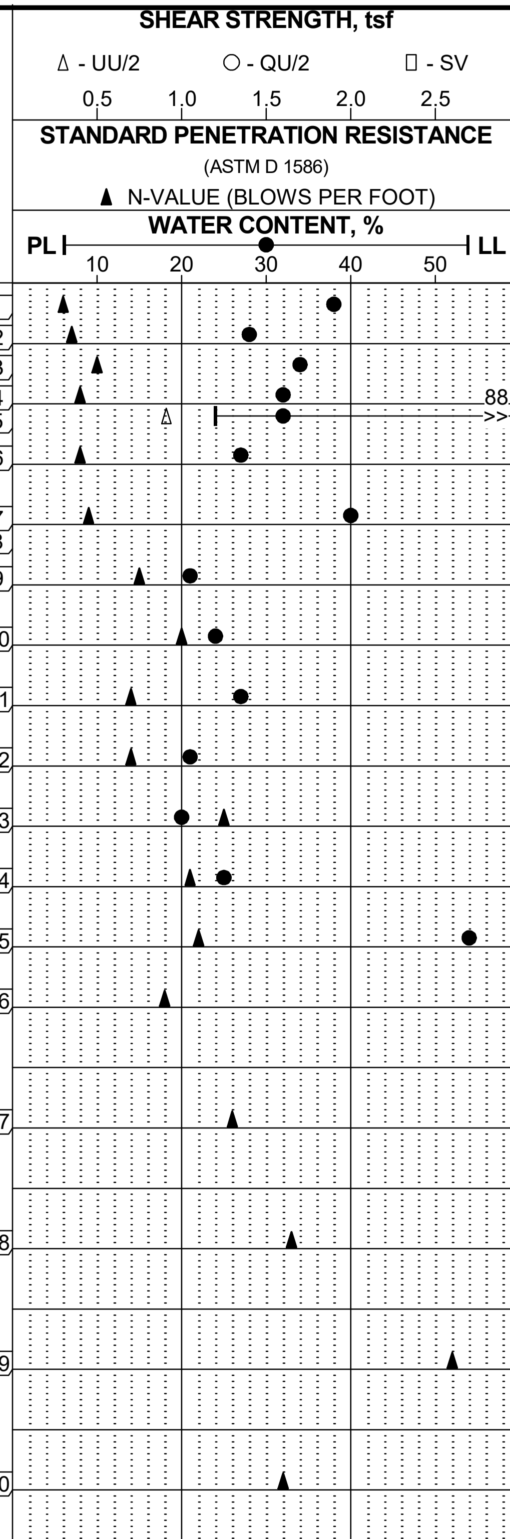
LOG OF BORING: B-1

Geotechnology Project No. J034298.01

Surface Elevation: 222 | Completion Date: 5/16/19

Datum MSL

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	SPT BLOW COUNTS / DRY UNIT WEIGHT (pcf)	SAMPLES
		Asphalt: 2 inches		
		Concrete	2-2-4	SS1
5	217	Medium stiff to stiff, brown to brown and gray FAT CLAY, silt seams - (CH)	2-3-4	SS2
			2-5-5	SS3
10	212	Soil Resistivity = 302.10 ohms-cm; pH = 5.79	2-3-5	SS4
			87	ST5
15	207		2-4-4	SS6
20	202		2-4-5	SS7
				ST8
25	197	Medium dense to very dense, brown to gray SAND WITH SILT - SP-SM	3-7-8	SS9
30	192		7-9-11	SS10
35	187	9.8% passing No. 200 sieve	5-6-8	SS11
40	182		5-6-8	SS12
45	177	Soil Resistivity = 4,389.00 ohms-cm; pH = 8.18	5-10-15	SS13
50	172	trace organics	6-10-11	SS14
55	167	Lense of gray, fat clay	3-10-12	SS15
60	162	5.5% passing No. 200 sieve	9-9-9	SS16
65	157			
70	152	trace gravel	10-12-14	SS17
75	147			
80	142		14-14-19	SS18
85	137			
90	132		12-22-30	SS19
95	127			
100	122	Boring terminated at 100 feet.	15-17-15	SS20



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 20 FEET
KJB DRILLER SAS LOGGER
CME 750 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 91 %

REMARKS:

Drawn by: AIM	Checked by: DMS	App'vd. by:
Date: 5/17/19	Date: 6/27/19	Date:

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT 100840
Ditch Nos 1 & 47 Strs. & Apprs.
(S)

LOG OF BORING: B-2

Geotechnology Project No. J034298.01

Surface Elevation: 222 | Completion Date: 5/17/19

Datum MSL

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) / SPT BLOW COUNTS / CORE RECOVERY/RQD	SAMPLES
		Asphalt: 2 inches	3-4-6	SS1*
5	217	Fill: Brown SILT - ML	2-3-3	SS2
		Medium stiff, brown to gray FAT CLAY - (CH) trace sand and organics	87	ST3
10	212	trace silt	1-1-2	SS4
15	207		1-2-3	SS5
				ST6
20	202	Medium stiff, brown and gray SILT, trace organics - (ML) 99.3% passing No. 200 sieve Soil Resistivity = 5,130.00 ohms-cm	2-3-3	SS7
25	197		5-11-14	SS8
		Medium dense to dense, gray SAND - SP		ST9*
30	192	trace silt Soil Resistivity = 541.50 pH = 7.91	10-10-12	SS10
35	187		8-10-13	SS11
40	182		4-6-9	SS12
45	177	trace gravel	8-8-11	SS13
50	172	Boring terminated at 50 feet.	8-12-23	SS14
55	167			
60	162			
65	157			
70	152			
75	147			
80	142			
85	137			
90	132			
95	127			
100	122			

GRAPHIC LOG

SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 ☐ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ├──●──┤ LL

10 20 30 40 50

83 >>

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 25 FEET ∇

DRILLING DATA

___ AUGER 3 3/4 HOLLOW STEM

WASHBORING FROM ___ FEET

KGB DRILLER EWF LOGGER

CME 750 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 91 %

REMARKS: *No sample recovery

Drawn by: AIM	Checked by: DMS	App'vd. by:
Date: 5/24/19	Date: 6/27/19	Date:

GEOTECHNOLOGY INC FROM THE GROUND UP

ARDOT 100840
Ditch Nos 1 & 47 Strs. & Apprs. (S)

LOG OF BORING: B-3

Geotechnology Project No. J034298.01

LOG OF BORING 2002 WL J034298.01.GPJ GTINC 0638301.GPJ 6/26/19

APPENDIX D – LABORATORY TEST DATA

ATTERBERG LIMITS

GRAIN SIZE DISTRIBUTIONS

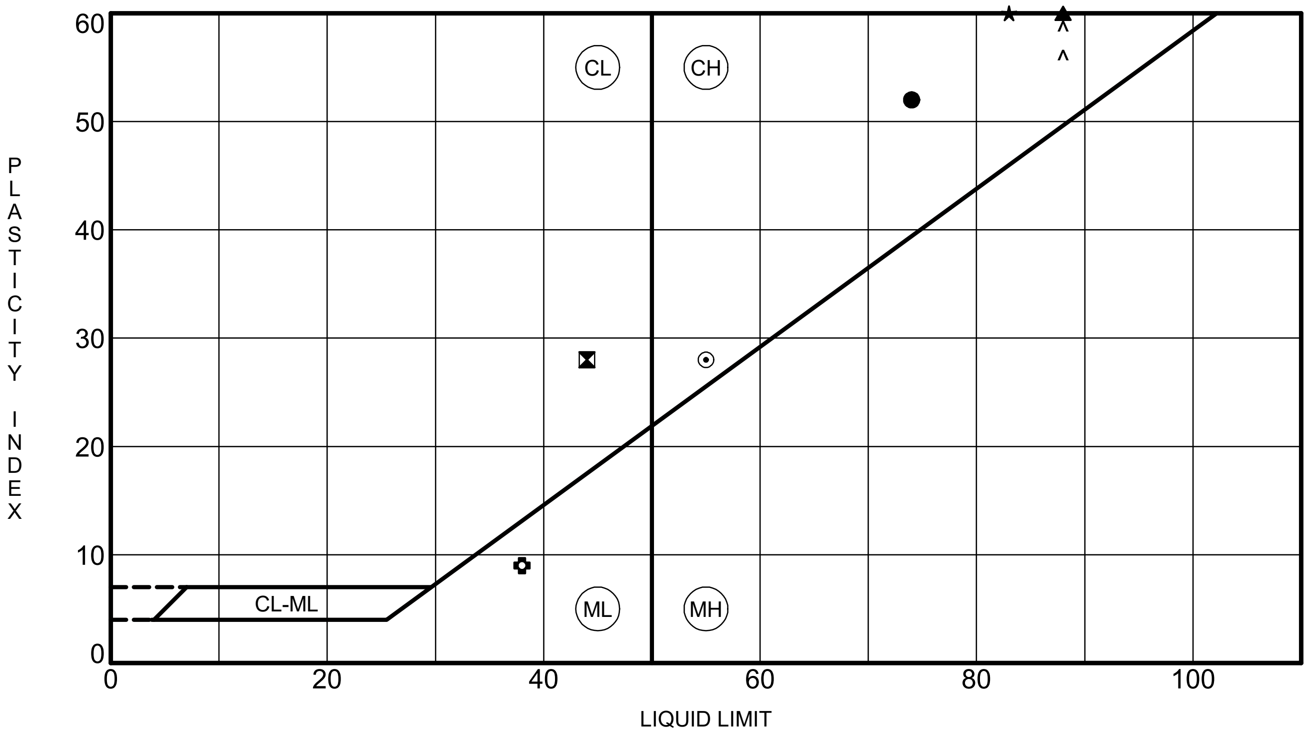
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION

CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION

ONE-DIMENSIONAL CONSOLIDATION

RESISITIVITY

pH



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**5.0**	**74**	**22**	**52**		**FAT CLAY(CH), A-7-6**
⧗	**B-1**	**15.0**	**44**	**16**	**28**		**LEAN CLAY(CL), A-7-6**
▲	**B-2**	**10.0**	**88**	**24**	**64**		**FAT CLAY(CH), A-7-6**
★	**B-3**	**5.0**	**83**	**23**	**60**		**FAT CLAY(CH), A-7-6**
⊙	**B-3**	**15.0**	**55**	**27**	**28**		**FAT CLAY(CH), A-7-6**
✿	**B-3**	**18.5**	**38**	**29**	**9**	**99**	**SILT(ML), A-4(11)**

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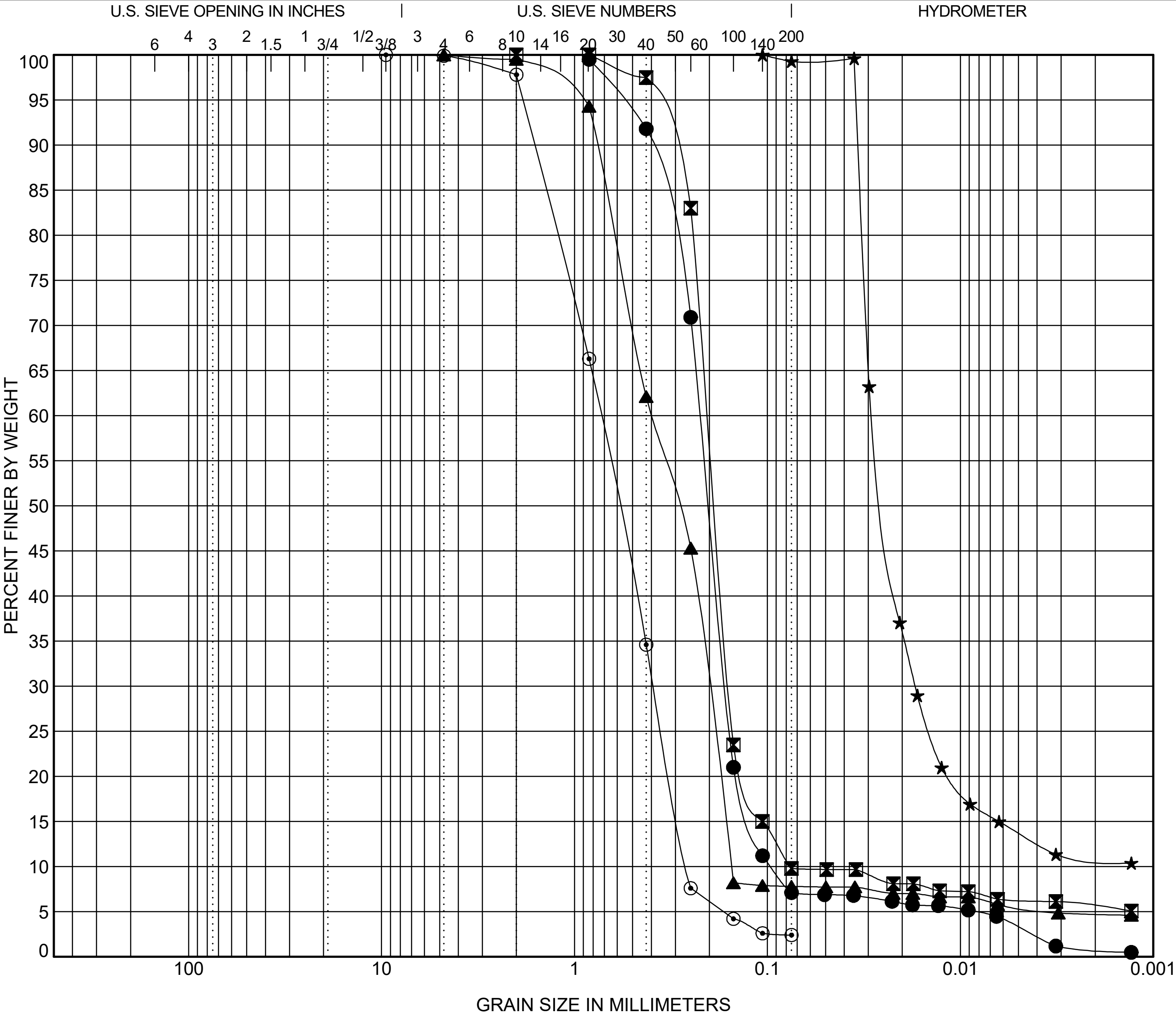
FROM THE GROUND UP

ATTERBERG LIMITS RESULTS

ARDOT 100840
Ditch Nos 1 & 47 Strs. & Apprs. (S)
J034298.01

US ATTERBERG LIMITS J034298.01.GPJ US LAB.GDT 6/26/19



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen ID	Depth	Classification	LL	PL	PI	Cc	Cu
●	B-1	18.5	**POORLY GRADED SAND WITH SILT(SP-SM), A-3**				1.26	2.33
⧗	B-2	33.5	**POORLY GRADED SAND WITH SILT(SP-SM), A-3**				1.61	2.70
▲	B-2	58.5	**POORLY GRADED SAND WITH SILT(SP-SM), A-1-b**				0.67	2.59
★	B-3	18.5	**SILT(ML), A-4(11)**	38	29	9		
⊙	B-3	38.5	**POORLY GRADED SAND(SP), A-3**				0.78	2.80

	Specimen ID	Depth	D100	D60	D50	D30	D10	%Gravel	%Sand	%200-2µm	%<2µm
●	B-1	18.5	0.84	0.224	0.202	0.164	0.096	0.0	92.4	3.8	3.3
⧗	B-2	33.5	2	0.205	0.188	0.159	0.076	0.0	90.2	3.5	6.3
▲	B-2	58.5	4.75	0.398	0.29	0.203	0.154	0.0	92.2	2.3	5.5
★	B-3	18.5	0.106	0.028	0.025	0.017		0.0	0.7	85.5	13.8
⊙	B-3	38.5	9.5	0.734	0.592	0.388	0.262	0.1	97.5	2.4	

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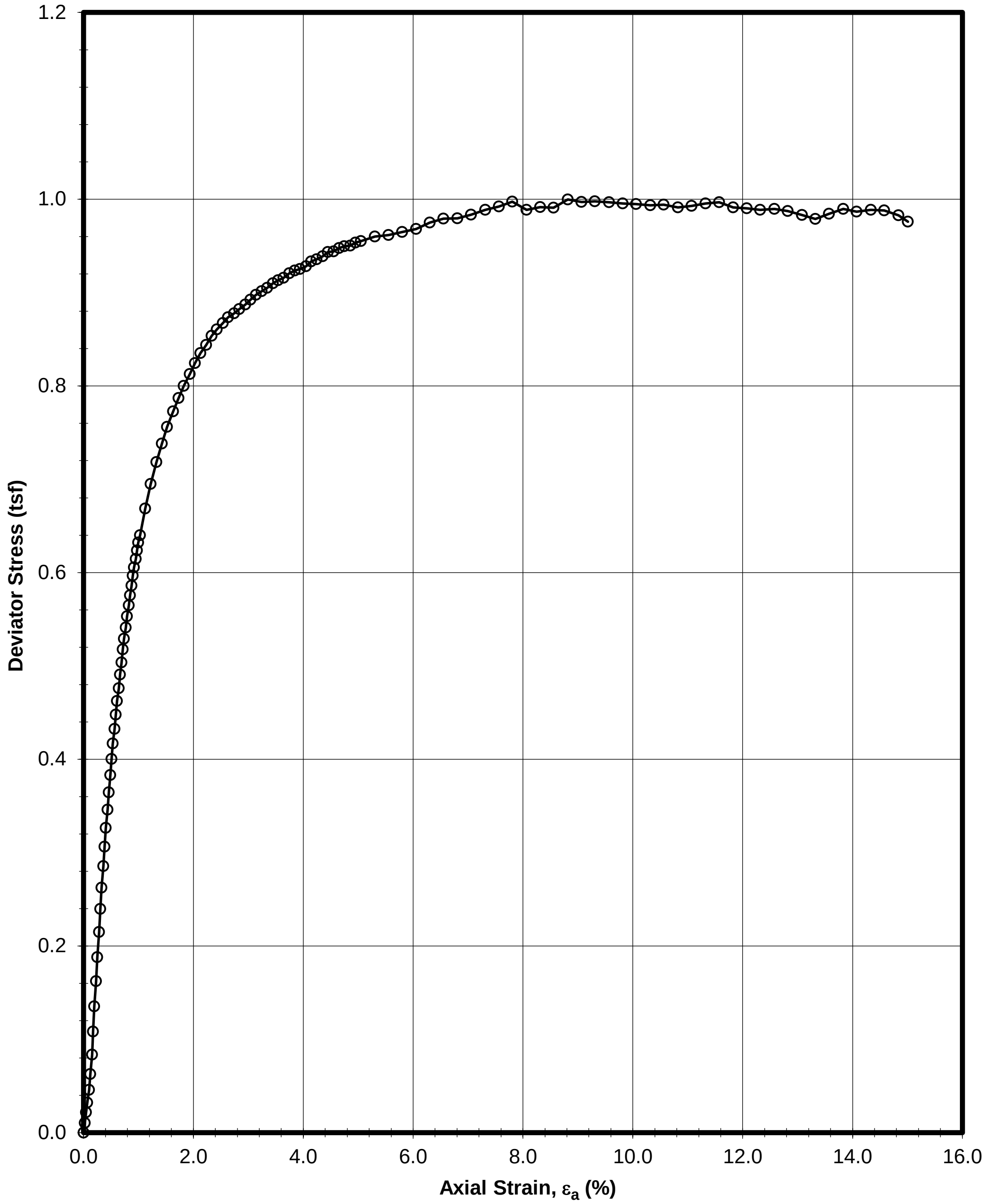
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

ARDOT 100840
Ditch Nos 1 & 47 Strs. & Apprs. (S)
Geotechnology Project No. J034298.01

US GRAIN SIZE J034298.01.GPJ US LAB.GDT 9/6/19



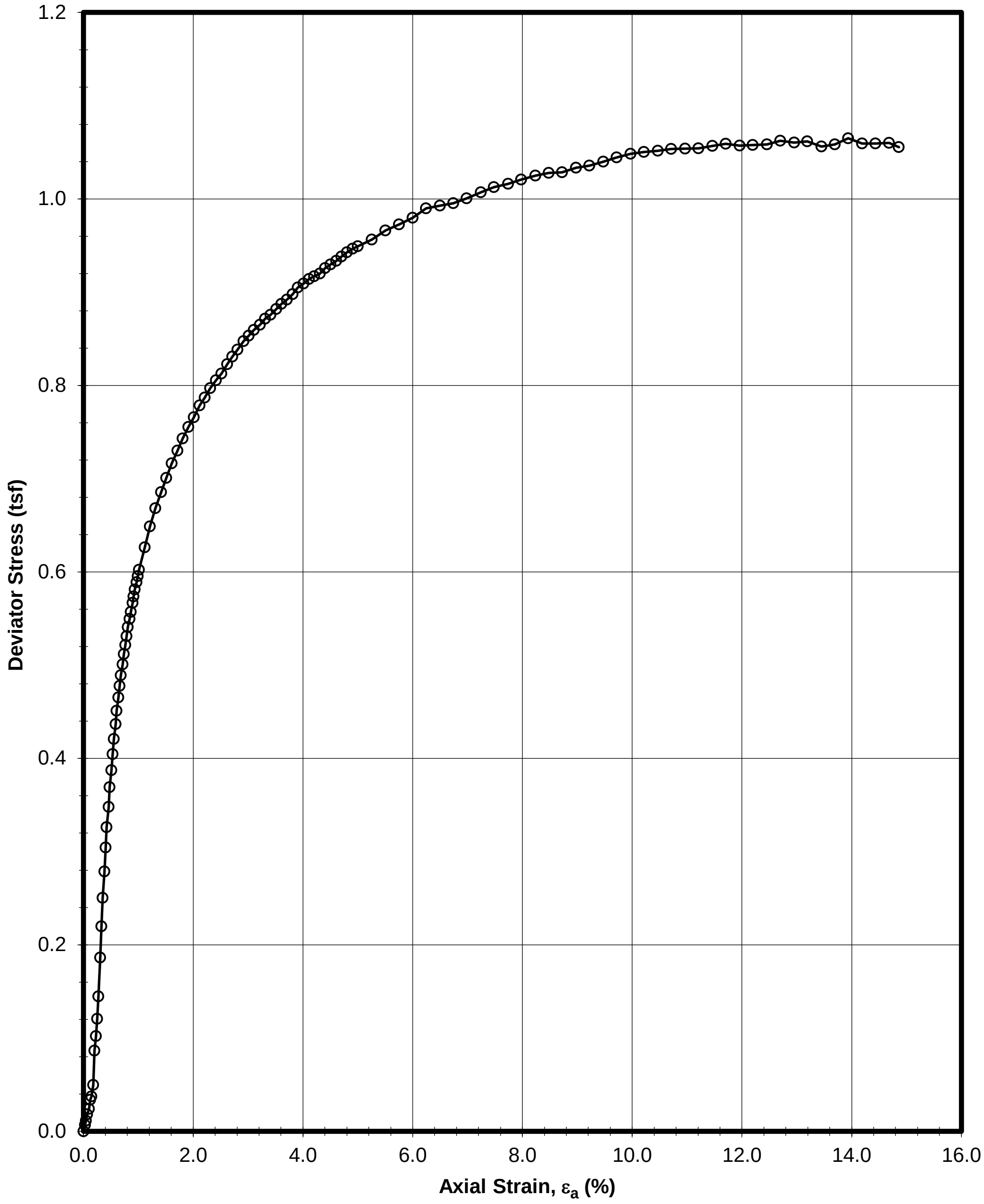
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034298.01

Boring: B-1

Sample: ST-3 - Depth: 5 ft.



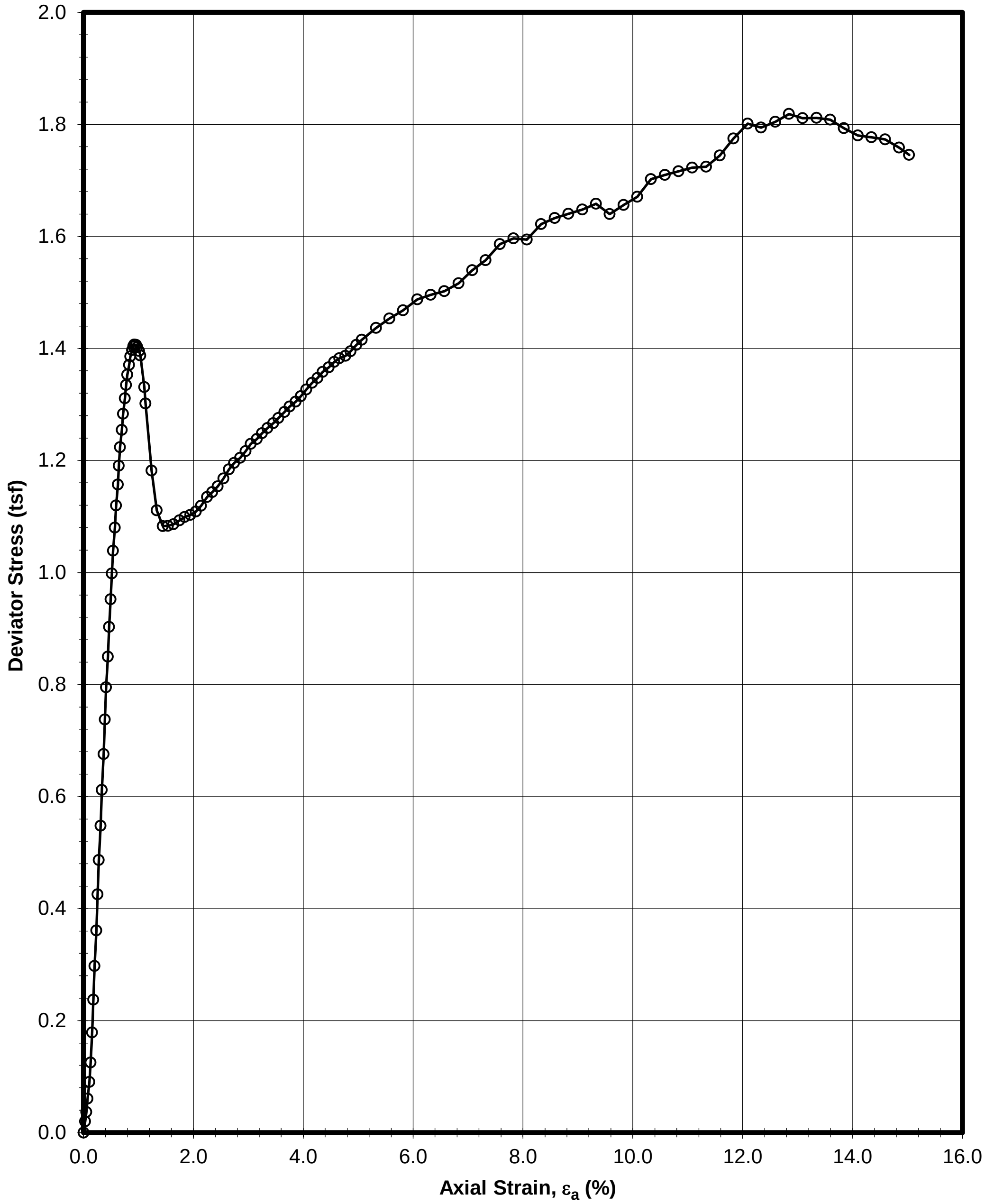
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034298.01

Boring: B-1

Sample: ST-6 - Depth: 15 ft.



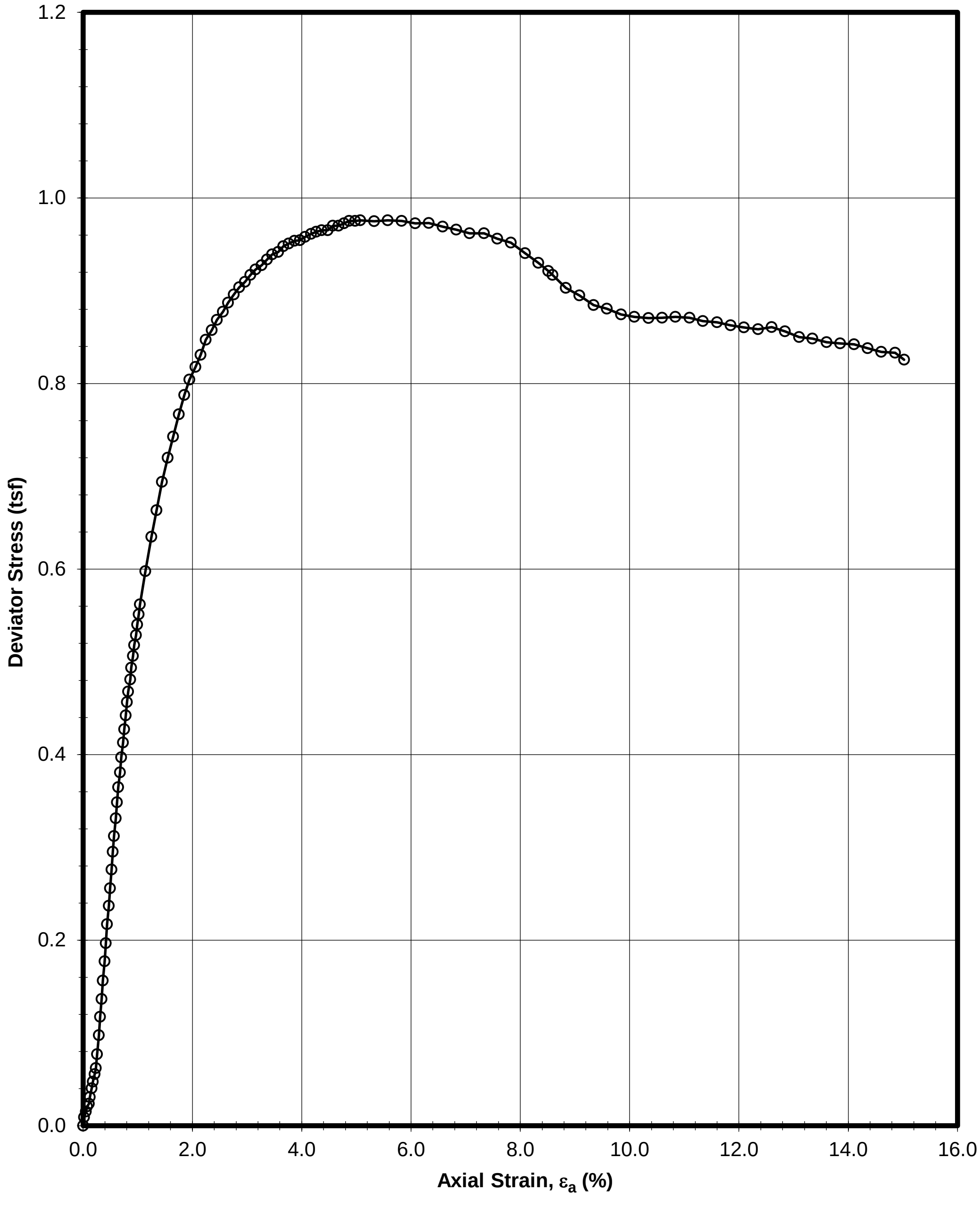
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034298.01

Boring: B-2

Sample: ST-5 - Depth: 10 ft.



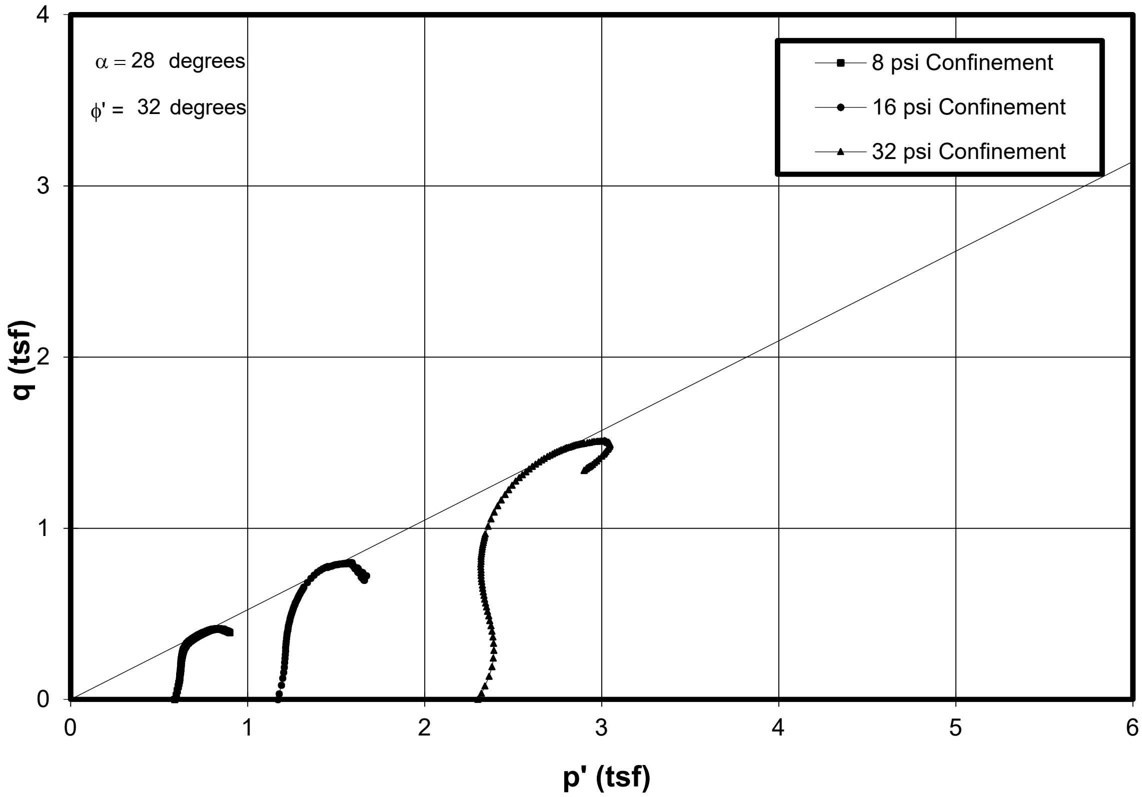
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

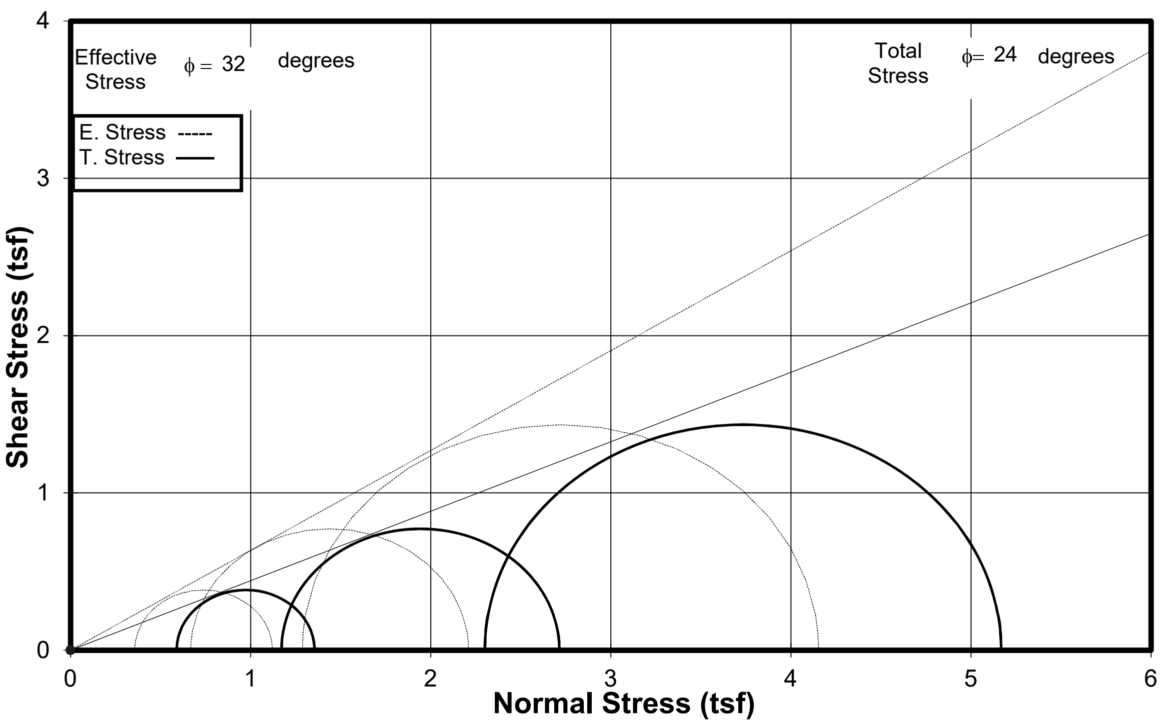
ASTM D 2850

Project No.: J034298.01

Boring: B-3

Sample: ST-1 - Depth: 5 ft.





CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 4767

Project No.: J034298.01

Boring: B-3, B-3, B-3

Sample: ST-2, ST-2, ST-2 - Depth: 15.0, 15.0, 15.0

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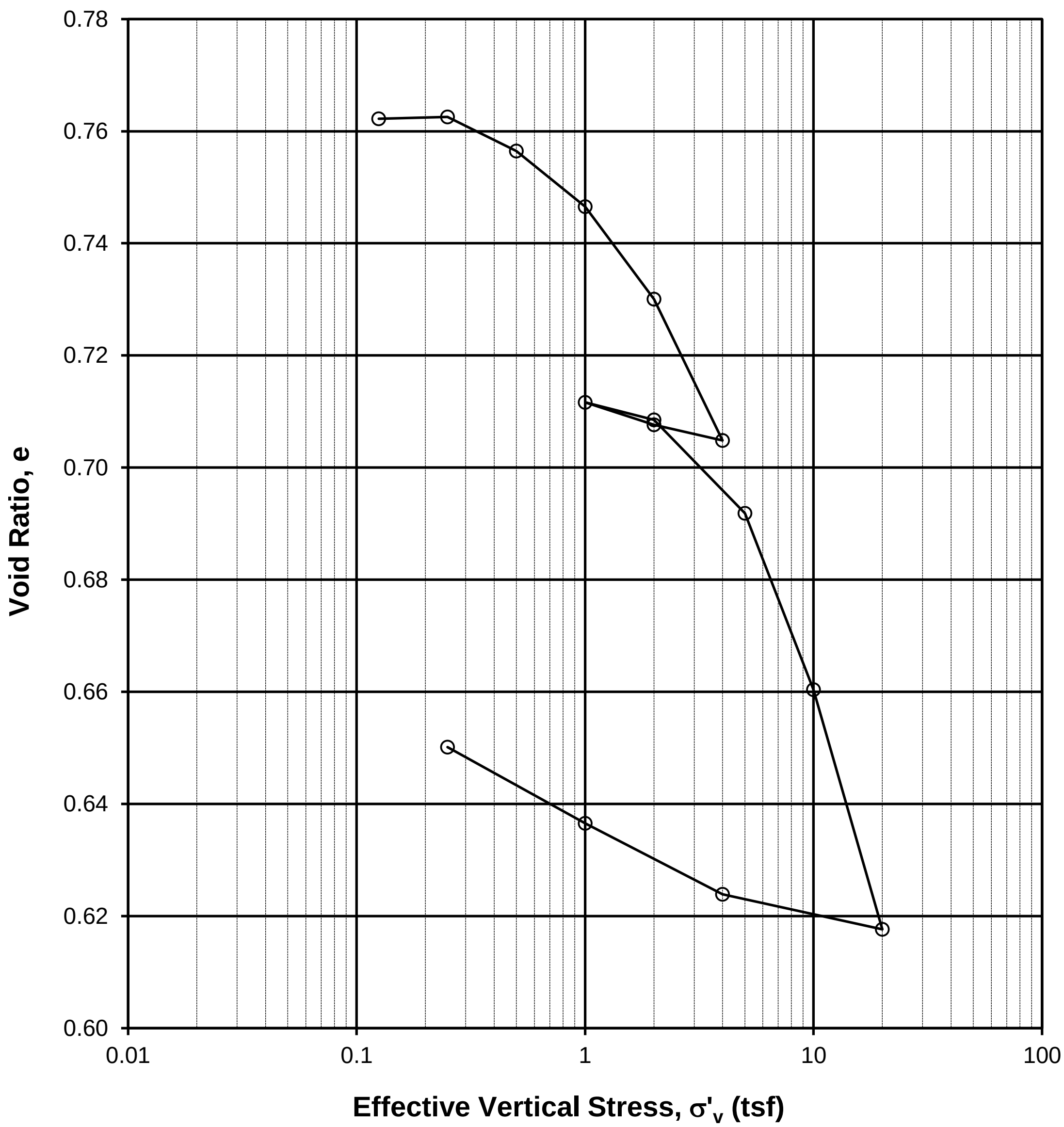
Liquid Limit= 44	Plastic Limit= 16	Plasticity Index = 28	USCS: CL

Compression Index, C_c = 0.14	Void Ratio, e_o = 0.66
Recompression Index, C_r = 0.03	Preconsolidation Pressure = 2.45 tsf



1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435

Project No.: J034298.0

Boring: B-1

Sample: ST-6 - Depth: 15.0

SOIL RESISTIVITY TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034298.01**	**June 26, 2019**
Project Name:	**ARDOT 100840**	
Boring Number:	**B-1**	
Sample ID:	**SS-6**	
Depth (ft):	**18.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	22,000	0.57	12,540.00	9.3
#2	13,000	0.57	7,410.00	10.6
#3	10,000	0.57	5,700.00	24.1
#4	9,000	0.57	5,130.00	30.1
#5	10,000	0.57	5,700.00	31.2

Minimum Soil Resistivity **5,130.00** (ohms-cm)

SOIL RESISTIVITY TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034298.01**	**June 10, 2019**
Project Name:	**ARDOT 100840**	
Boring Number:	**B-2**	
Sample ID:	**SS-3, SS-4, SS-6**	
Depth (ft):	**6**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	6,150	0.57	3,505.50	18.3
#2	1,150	0.57	655.50	27.0
#3	530	0.57	302.10	33.7
#4	535	0.57	304.95	40.0

Minimum Soil Resistivity **302.10** (ohms-cm)

SOIL RESISTIVITY TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034298.01**	**June 10, 2019**
Project Name:	**ARDOT 100840**	
Boring Number:	**B-2**	
Sample ID:	**SS-13, SS-14**	
Depth (ft):	**43.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	15,500	0.57	8,835.00	9.8
#2	87,650	0.57	49,960.50	16.8
#3	7,700	0.57	4,389.00	21.6
#4	9,150	0.57	5,215.50	19.9

Minimum Soil Resistivity **4,389.00** (ohms-cm)

SOIL RESISTIVITY TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034298.01**	**June 21, 2019**
Project Name:	**ARDOT 100840**	
Boring Number:	**B-3**	
Sample ID:	**SS-7**	
Depth (ft):	**18.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	22,000	0.57	12,540.00	9.3
#2	13,000	0.57	7,410.00	10.6
#3	10,000	0.57	5,700.00	24.1
#4	9,000	0.57	5,130.00	30.1
#5	10,000	0.57	5,700.00	31.2

Minimum Soil Resistivity **5,130.00** (ohms-cm)

SOIL RESISTIVITY TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034298.01**	**June 10, 2019**
Project Name:	**ARDOT 100840**	
Boring Number:	**B-3**	
Sample ID:	**SS-10, SS-11**	
Depth (ft):	**28.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	17,500	0.57	9,975.00	9.6
#2	12,000	0.57	6,840.00	16.7
#3	950	0.57	541.50	22.9
#4	10,500	0.57	5,985.00	25.6

Minimum Soil Resistivity **541.50** (ohms-cm)

pH TESTS (ASTM D 4972 or AASHTO T-289)

GEOTECHNOLOGY INC
FROM THE GROUND UP

DATE May 28, 2019	PROJECT NAME ARDOT 100840	PROJECT NO. J034298.01

General Test Information: pH Meter: Humboldt Ph Testr H-4371 or ______
Distilled Water: required pH=5.5 to 7.5 Measured value: ______
Soil/Water Ratio: Typically 1/1 or 1/2, but 1/5 for lime stabilized soils

Boring No.	Sample No.	Depth (ft)	Visual Identification (Color, Group Name & Symbol)	Soil : Water Ratio (g/g) or (g/mL)	pH of Solution (Meter/ Paper)[1]	Tare No. Air Drying	Jar Number	Remarks
B-2	SS-3,4,6	6-13.5		1/2	5.79 ------------- 21.7°	TP-50	1	
B-2	SS-13,14	43.5-48.5		1/1	8.18 ------------- 22.7°	TP-35	3	
B-3	SS-10,11	28.5-33.5		1/1	7.91 ------------- 22.0°	TP-46	4	

					-------------			

[1]pH by Meter is Method A; pH by Paper is Method B

Tested By: ______ Date: ______

Calculated By: AIM Date: 06/10/19

Checked By: ______ Date: ______

APPENDIX E – AASHTO AND USCS CLASSIFICATIONS

SUMMARY OF CLASSIFICATION TEST RESULTS
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
ARDOT 100840

Boring No.	Depth	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Sieve Analysis								AASHTO CLASS.	USCS CLASS.
					2 in.	1 in.	3/4 in.	3/8 in.	#4	#10	#40	#200		
B-1	5	74	22	52	--	--	--	--	--	--	--	--	A-7-6	CH
B-1	15	44	16	28	--	--	--	--	--	--	--	--	A-7-6	CL
B-1	18.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	91.8	7.1	A-3	SP-SM
B-2	10	88	23	60	--	--	--	--	--	--	--	--	A-7-6	CH
B-2	33.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	97.5	9.8	A-3	SP-SM
B-2	58.5	--	--	--	100.0	100.0	100.0	100.0	100.0	99.5		5.5	A-1-b	SP-SM
B-3	5	83	23	60	--	--	--	--	--	--	--	--	A-7-6	CH
B-3	15	55	27	28	--	--	--	--	--	--	--	--	A-7-6	CH
B-3	18.5	38	29	9	100.0	100.0	100.0	100.0	100.0	100.0	100.0	99.3	A-4	ML
B-3	38.5	--	--	--	100.0	100.0	100.0	100.0	99.9	97.8	34.6	2.4	A-1-b	SP

APPENDIX F - SITE SPECIFIC SEISMIC STUDY

Site-Specific Seismic Study
Ditches Nos. 1 & 47 Structures and Approaches
Marked Tree, AR

By

Shahram Pezeshk, Ph.D., P.E.
Email: s.pezeshk@aol.com

May 20, 2018

TABLE OF CONTENTS

Site-Specific Seismic Study
Ditches Nos. 1 & 47 Structures and Approaches
Marked Tree, AR

1.0. EXECUTIVE SUMMARY

The executive summary provides an overview of my understanding of the project and recommendations. Information and recommendations presented in the executive summary should not be used without reviewing of the entire report.

- The location of the study site is at 35.5480083° N and 90.3587944° W.
- Based on the recommendations of the *AASHTO LRFD Bridge Design Specifications*, *7th Edition with 2016 Interim Revisions*, A_s (zero-period), S_{DS} (short period) and S_{D1} (long period) are provided in Table 3 – Site Class D.
- Site-specific recommendations following the AASHTO LRFD Bridge Design Specifications, 7th Edition with 2016 Interim Revisions are provided in Table 7 and Table 8.

2.0. SCOPE OF WORK

The design in the *AASHTO LRFD Bridge Design Specifications, 7th Edition with 2016 Interim Revisions* allows two procedures for determining design ground motions:

1. <u>*General Procedure*</u>. In this method, the response spectrum is determined using the following steps: (1) develop the rock spectrum using seismic design maps for values of Peak Ground Acceleration (PGA), and spectral acceleration at periods of 0.2 and 1.0 seconds; (2) determine the Site Class using shear wave velocity (V_s) measurements from the upper 100 feet of the soil profile; and (3) adjust the rock spectrum for site class to develop the general response spectrum.

2. <u>*Site-Specific Procedure*</u>. In this method, the response spectrum is determined using a combination of probabilistic seismic hazard and site response analyses. The site-specific response spectrum may not be less than 2/3 of the general response spectrum.

Briefly, the scope of our services for the site-specific investigation included the following steps:

1. Perform probabilistic seismic hazard analysis (PSHA) to estimate ground motions in the rock underlying the site;
2. Determine Uniform Hazard Response Spectrum (UHRS) at the rock level considering near fault effects;
3. Determine probabilistic consistent magnitude and distances from deaggregation;
4. Select ground motions consistent with magnitude and distances obtained in step 3 with near-faculty characteristics;
5. Perform spectral matching to match the selected ground motions to the UHRS of step 2;
6. Perform one-dimensional equivalent linear site-specific ground response analysis using the site-specific earthquake time histories by using the computer program SHAKE91 (Idriss and Sun, 1992) and considering the uncertainties associated with the shear-wave velocity and layer thicknesses for the soil profile; and
7. Develop site-specific response spectra for the existing subsurface conditions using the procedure outlined in the *AASHTO LRFD Bridge Design Specifications, 7th Edition with 2016 Interim Revisions,* which include: MCE_R and DBE seismic hazard related to 7% percent probability of exceedance in 75 years, and 5 percent damping for a single degree of freedom (SDOF) structure.

3.0. SUBSURFACE CONDITION

This study is based on the available information of the soil stratigraphy provided by Geotechnology, Inc. The shear-wave velocity was obtained by Geotechnology using a Cone Penetration Testing (CPT) seismic survey. The shear-wave velocity profiles obtained by Geotechnology are provided in Table 1 and Table 2 and are shown in Figure 1. The locations of CPT1 and CPT2 are provided in the Appendix.

Table 1. CPT-1 Shear-Wave Velocity Profile.

Depth (ft)	Average Shear Wave Velocity (ft/sec)
5.18	487.15
8.30	487.15
11.55	381.59
14.83	402.99
18.18	571.28
21.39	719.54
24.67	641.20
27.89	939.50
31.17	650.91
34.45	547.11
37.73	798.96
40.98	755.60
44.29	737.62
47.54	758.33
50.82	913.48
54.07	760.11
57.58	738.66
60.86	867.25
64.17	669.82
67.42	1138.02
70.73	993.61
73.98	986.21
77.2	909.99
80.45	909.22
83.92	773.81
87.14	852.6
90.39	1065.52
93.54	785.41
96.78	852.72
99.93	1227.59

Table 2. CPT-2 Shear-Wave Velocity Profile.

Depth (ft)	**Average Shear Wave Velocity (ft/sec)**
5.31	394.97
8.60	394.97
11.94	442.16
15.19	598.51
18.54	756.53
21.82	829.55
25.13	848.61
28.35	696.94
31.56	599.34
34.84	712.05
38.16	791.57
41.44	1079.02
44.75	676.83
48.00	659.01
51.18	765.75
54.72	720.57
57.78	806.00
60.99	713.34
64.30	628.93
67.55	950.6
70.90	895.57
74.21	877.71
77.46	663.1
80.74	729.82
84.02	869.81
87.24	1214.33
90.68	1321.36
93.96	670.61
97.15	1058.31
100.07	774.87

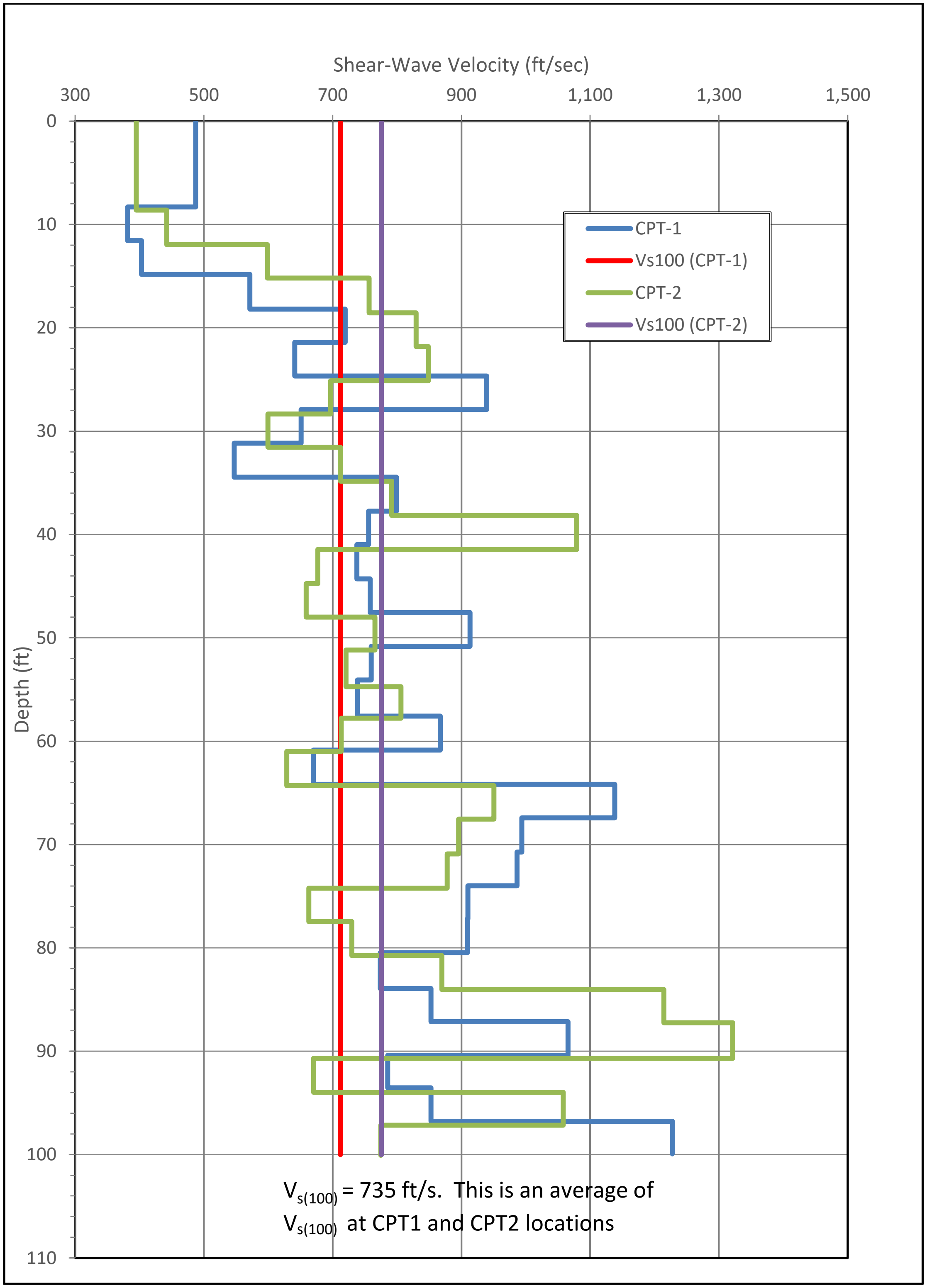


Figure 1. CPT shear-wave velocity profile obtained at CPT1 and CPT2 locations.

4.0 SUBSURFACE GEOLGOY

The study site is located within the Mississippi embayment. For site response analyses we needed data from below the measured shear-wave velocity profiles to B/C boundary. We estimated shear-wave velocity below the CPT values using geologic information at the study site provided by the United States Geological Survey (USGS). Figure 2 shows the location of the study site within the Mississippi embayment. Figure 3 shows the geologic information at the study site that we used for this study.

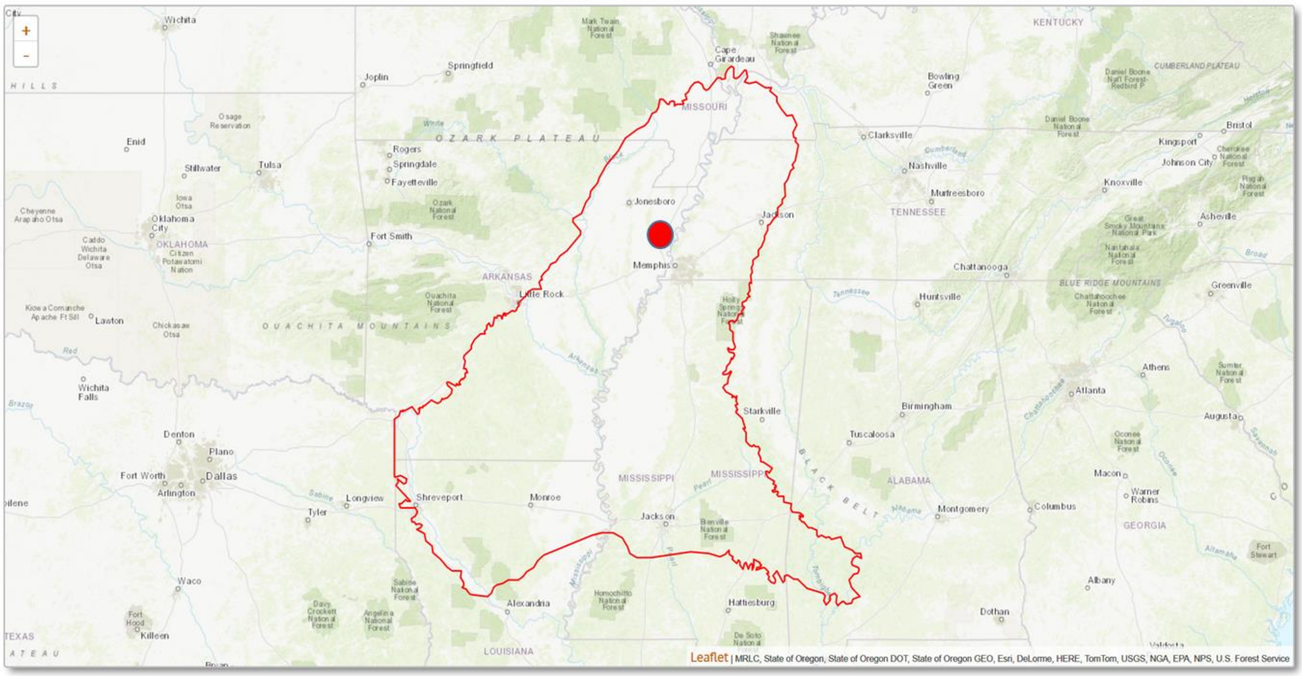
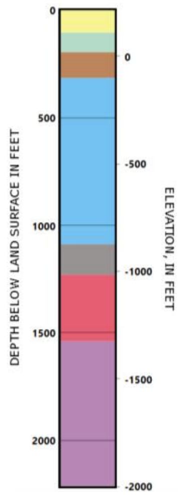


Figure 2. Location of study site within the Mississippi embayment.



Explanation

Top[1]	Bottom[1]	Altitude	Thickness	Geologic Unit
0	104	214	104	**Mississippi River Valley alluvial aquifer**
104	196	110	92	**Upper Claiborne aquifer**
196	315	18	119	**Middle Claiborne confining unit**
315	1088	-101	773	**Middle Claiborne aquifer**
1088	1230	-874	142	**Middle Wilcox aquifer**
1230	1539	-1016	309	**Lower Wilcox aquifer**
1539	2214	-1325	not determined	**Midway confining unit**

[1]Values are depth below land surface, in feet

Geologic correlation

Thickness (ft)	Velocity (ft/s)
104	CPT data
92	1355
119	1355
773	1739
142	2215
309	2543
675	2789

Figure 3. Geologic information at the study site.

5.0 GENERAL INFORMATION

For structural design purposes, the loads imparted to the structure are derived through elastic dynamic structural analysis procedures such as the equivalent lateral force or modal analysis, or if a more advanced dynamic structural analysis is required, by using a procedure such as an inelastic response history analysis. The equivalent lateral force and modal analysis procedures use the response spectrum derived from either code based or site-specific methods, to evaluate the base shear force. The inelastic response history method uses time histories; either modified recorded time histories or synthetic time histories, to evaluate the seismic load demand. For this project, we have been requested to perform a site-specific seismic study to produce a uniform hazard response spectrum based on the seismic parameters used in the *AASHTO LRFD Bridge Design Specifications, 7th Edition with 2016 Interim Revisions* which include: seismic hazard related to 7% percent probability of exceedance in 75 years and 5 percent damping for a single degree of freedom (SDOF) structure.

6.0 *AASHTO LRFD Bridge Design Specifications, 7th Edition with 2016 Interim Revisions* SITE AMPLIFICATIONS

The average shear-wave velocity for this site ($\bar{V}_s$) as per the recommendations of the *AASHTO LRFD Bridge Design Specifications, 7th Edition with 2016 Interim Revisions* was calculated to be 735 ft/sec. As the 2014 *AASHTO* bases the site classification on the average properties in the top

100 feet, the site class for the study site was identified to be a site class “D” according to the $\bar{V}_s$ - value (Table 3.10.3.1-1 Site Class Definitions). According to Tables 3.10.3.2-1, 3.10.3.2-2, and 3.10.3.2-3 and the mapped spectral acceleration, the site coefficients F_{pga}, F_a, and F_v for Site Class “D” were provided in Table 3.

6.1. Dynamic Soil Properties

For seismic ground response analysis, low strain soil shear modulus and damping are the required dynamic soil properties. Brief discussion on these properties is given below.

6.1.1. Low Strain Soil Shear Modulus

A key parameter necessary to evaluate dynamic response of soils is the dynamic shear modulus, G_s or shear wave velocity, which is also related to dynamic shear modulus. Values of shear wave velocity or shear modulus can be determined either by measurement in the laboratory on undisturbed soil samples, or in the field by performing field seismic tests. Shear modulus is not a constant property of soil but decreases nonlinearly with increasing strain. For initial design purposes, shear modulus measured at small shear strain amplitudes (less than 10^{-4} percent), referred to as G_{max}, is a desired design parameter.

Laboratory measurement of shear wave velocity or low strain soil shear modulus was beyond the scope of our services. Various correlations and typical values are available in the literature to estimate the approximate value of shear wave velocity and G_{max}.

6.1.2. Damping

The inelastic behavior of soil (discussed later) also gives rise to energy absorption characteristics of soil known as material damping. Damping is generally expressed as a percentage of the critical damping. Low strain damping of approximately 5 to 10 percent of the critical damping is commonly used for soils. Damping of 5 percent of critical was used for the analysis. However, this damping was modified in the analysis based on the strain levels in the soil, as explained in subsequent sections of this report.

6.1.3. Effect of Strain on Dynamic Soil Properties

It is well understood that the stress-strain relationship of soils is nonlinear. This means that the soil shear modulus is not a constant value but degrades nonlinearly with increasing strain in the soil. Dynamic analyses considering true nonlinear behavior of soil are complicated and are an active and current research area. Accordingly, an equivalent linear analysis is typically used in practice. Equivalent linear analyses consist of performing a series of linear analyses, in an iterative process, using for each analysis soil properties consistent with the strains resulting from the previous one. Equivalent linear site response analysis is used in the present study. Many studies have been performed in the past to establish a relationship between modulus degradation with strain.

7.0. CODE BASED DESIGN APPROACH

7.1. AASHTO Guide Specifications for LRFD Seismic Design

Ground response analysis was performed to obtain representative response spectra at the ground surface based on the time histories at B-C boundary propagated through the site soils. According to the United States Geological Survey (USGS) Hazard Maps, the project location has mapped 0.2 second spectral response acceleration (S_S) of approximately 1.824g, mapped 1.0 second spectral response acceleration (S_1) of approximately 0.513g, and peak ground acceleration (PGA) of 1.020g.

Design Earthquake response spectral acceleration coefficient at the effective peak ground acceleration, A_S, the short period, S_{DS}, and at the 1 second period, S_{D1}, shall be determined from the following equations, respectively:

$$A_s = F_{PGA}PGA = 1.000 \times 1.020 = 1.020 \qquad \text{(Equation 3.10.4.2-2)}$$
$$S_{DS} = F_a S_s = 1.000 \times 1.824 = 1.824 \qquad \text{(Equation 3.10.4.2-3)}$$
$$S_{D1} = F_v S_1 = 1.500 \times 0.513 = 0.770 \qquad \text{(Equation 3.10.4.2-6)}$$

Table 3. Mapped Provisional Design Response Spectrum Parameters at 5% Damping.

Parameter	Value
PGA	1.020
S_s	1.824
S_1	0.513
F_{PGA}	1.000
F_a	1.000
F_v	1.500
A_s	1.020
S_{DS}	1.824
S_{D1}	0.770

8.0. SITE-SPECIFIC PROCEDURE

The probabilistic seismic hazard analysis (PSHA) considers all potential earthquake sources that will contribute hazard at a specific site. The PSHA factors in contributions from all magnitudes, distances, and probability of occurrence for all sources. In this study, probabilistic seismic hazard analysis (PSHA) was used to estimate PGA and spectral acceleration at various periods for a B/C NEHRP site condition (V_{s30} = 760 m/sec) for a 7% probability of exceedance in 75 years.

8.1. Seismic Hazard Analysis

In this section the probabilistic seismic hazard analysis (PSHA) performed for the study site is documented. The uniform hazard response spectrum (UHRS) along with the magnitude and distance deaggregation for 7 percent probability of exceedance in 75 years (equivalent to a return period of about 1034 years) are calculated from the PSHA. The seismic hazard is calculated for the uniform firm rock site condition with 760 m/s shear-wave velocity in the upper 30 m (V_{S30}), representing the boundary between NEHRP site classes B and C. The effects of the near-fault directivity is included in the seismic hazard assessment.

8.1.1. Methodology

The site is located at about 3.6 miles northeast of Marked Tree, Arkansas (Figure 4). To perform the PSHA, the seismic source characterization (SSC) used in development of the 2014 U.S. Geological Survey (USGS) national seismic hazard maps (NSHM) (Petersen et al., 2014) [hereafter referred to as NSHM14] is used. For the study site, the SSC developed for the Central and Eastern United States (CEUS) in NSHM14 is used based on the location of the site. The New Madrid seismic zone (NMSZ) is the source of the 1811-1812 New Madrid earthquake sequence, which includes the three largest earthquakes to have occurred in historical time in the CEUS. The NMSZ contributes significantly to hazard in the CEUS (Petersen et al., 2014). The NMSZ is represented by multiple fault sources in the NSHM14. In NSHM14, two alternative models (separate branches), equally weighted, are used to model earthquakes in the NMSZ. The two alternative models for NMSZ along with the location of the study site are shown in Figure 5. As shown in Figure 5, the study site is very close to the NMSZ. Therefore, the near-fault effects, i.e. directivity effects, should be considered in the seismic hazard assessment. Directivity effects cause pulse-like ground motions that are known to increase the seismic hazard and risk in near-fault region. The directivity effects are included in the PSHA framework through ground motion prediction equations (GMPEs) used to estimate the earthquake ground-motion intensities. For this report, GMPEs used in the CEUS SSC for NSHM14 are adjusted to include the directivity effects. The adjusted GMPEs are used in the PSHA.

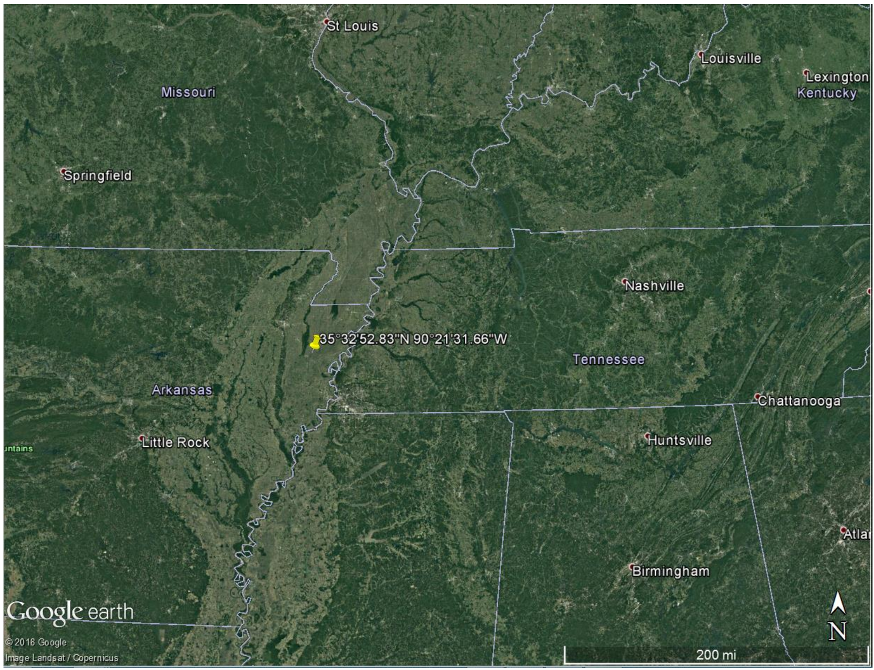


Figure 4. Location of the study site.

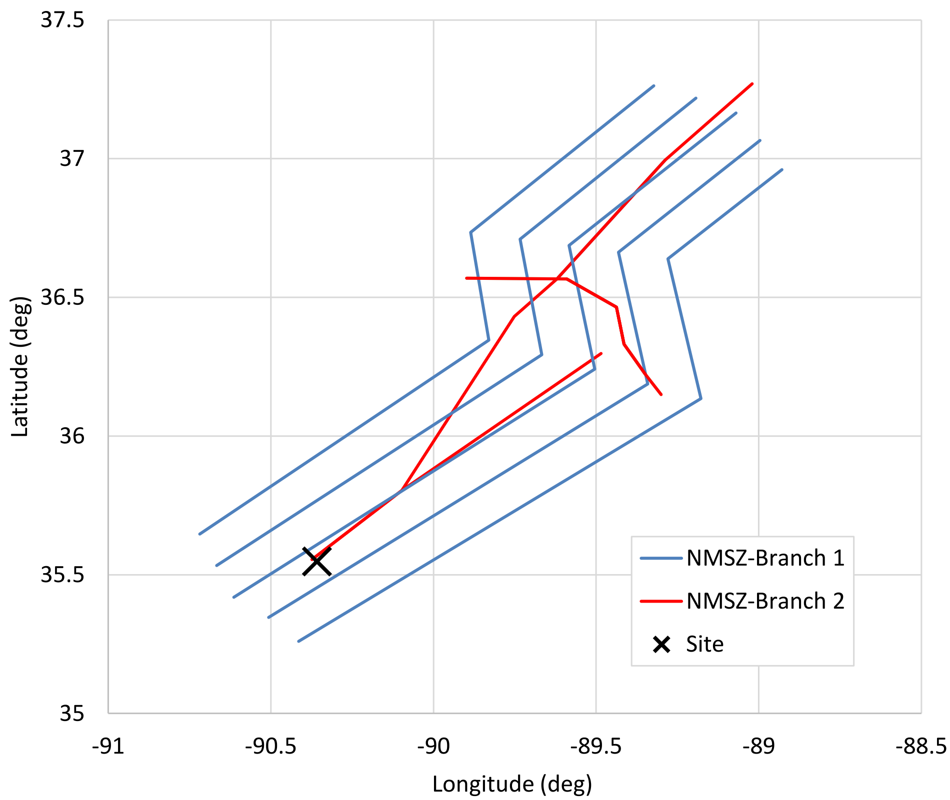


Figure 5. The two alternative models used for NMSZ in NSHM14 along with the location of the study site.

The National Seismic Hazard Mapping Project (NSHMP) code, nshmp-haz (https://github.com/usgs/nshmp-haz), developed and maintained by NSHMP within the USGS earthquake hazards program (EHP) is used for the PSHA. The CEUS SSC is used to run the hazard. Hazard is calculated for the BC boundary site condition with V_{s30}=760 m/s.

Hazard is calculated for the site using nshmp-haz at 7 periods ranging from 0.01 sec to 2.0 sec (0.01, 0.1, 0.2, 0.3, 0.5, 1.0, 2.0 sec) at which CEUS GMPEs are implemented. The 0.01 sec represents the peak ground acceleration (PGA). The UHRS for 7% probability of exceedance in 75 years (equivalent to a return period of about 1034 years) is calculated at the available periods. To extend the UHRS to 10 sec, hazard is calculated using Campbell and Bozorgnia (2014) GMPE for active regions for periods ranging from 2.0 sec to 10 sec. The UHRS obtained using Campbell and Bozorgnia (2014) GMPE is calculated at the return period at which the spectral ordinate at 2.0 sec matches that from 1034 year UHRS using CEUS GMPEs.

The magnitude and distance deaggregation at the seven periods ranging from 0.01 sec to 2.0 sec, at which CEUS GMPEs are implemented, are provided. Deaggregated magnitude and distance are used to smooth the discrete UHRS. Deaggregated magnitude and distances are used in stochastic simulations to generate ground motion spectra applicable to a BC boundary site condition in the CEUS. The stochastic simulations are preformed using the random-vibration theory module *tmrsk_loop_rv_drvr* in the stochastic simulation program SMSIM (Boore, 2005). The same seismological input parameters used in Boore (2015) are also utilized for stochastic simulations in this calculation. The stochastic model uses a single corner frequency source model, the Boore and Thompson (2015) stable continental region finite-fault factor, the Boatwright and Seekins (2011) attenuation model, the path-duration model from Boore and Thompson (2015), the crustal amplifications for a site with BC condition (V_{s30}=760 m/s) from Boore (2015), and the Boore and Thompson (2015) adjustments to the random-vibration-theory (RVT) simulations to account for the finite-duration time series. The spectral shapes obtained from stochastic simulation are used to interpolate the UHRS between the periods used in the PSHA and derive a smooth UHRS.

8.1.2. PSHA Results

The discrete UHRS for 7% probability of exceedance in 75 years obtained from PSHA are provided in Table 4 and shown in Figure 6. The UHRS are provided with and without including the directivity effects.

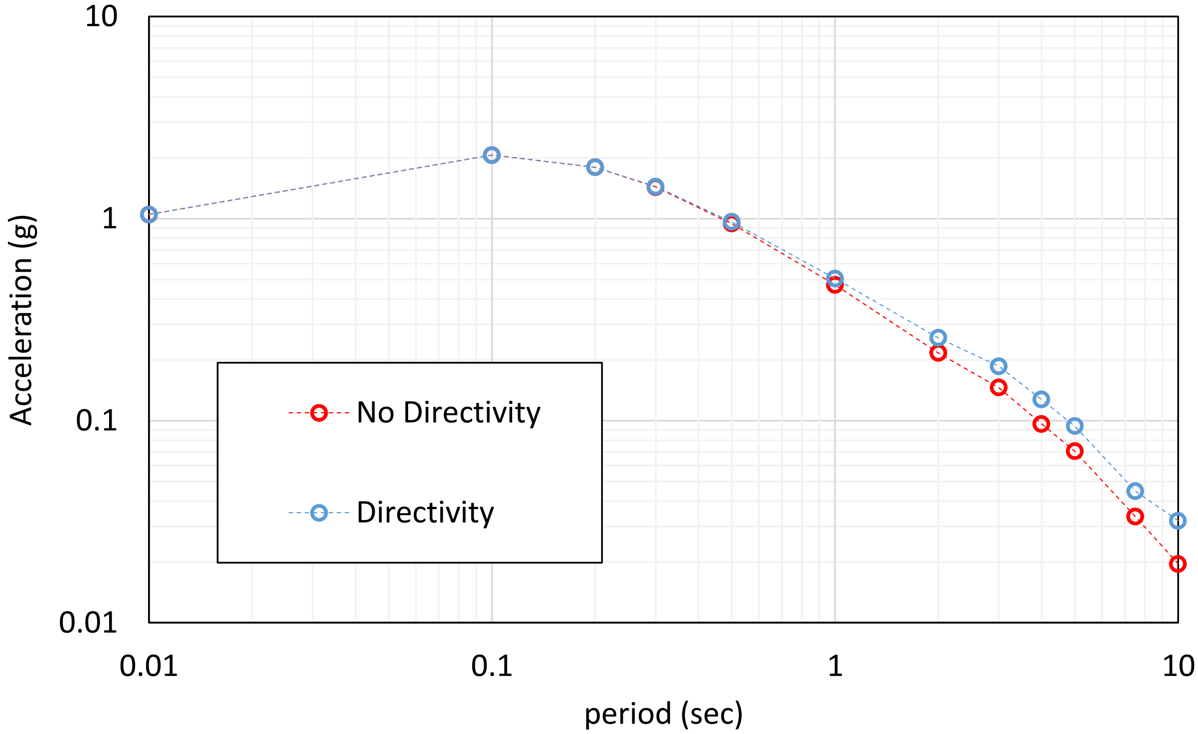


Figure 6. Discrete UHRS for 7% probability of exceedance in 75 years with and without including the directivity effects. Hazard was calculated only at the circle markers. Straight dashed lines just connect the circles and do not represent calculated spectra.

Table 4. Discrete UHRS for 7% probability of exceedance in 75 years with and without directivity effects.

Period (sec)	**UHRS (g)**	
	Without Directivity	**With Directivity**
0.01	1.0482	1.0482
0.1	2.0630	2.0632
0.2	1.7957	1.7978
0.3	1.4341	1.4416
0.5	0.9457	0.9642
1	0.4706	0.5054
2	0.2165	0.2574
3	0.1460	0.1860
4	0.0964	0.1274
5	0.0706	0.0942
7.5	0.0336	0.0448
10	0.0196	0.0320

The mean magnitudes and distances obtained from deaggregation of hazard for 7% probability of exceedance in 75 years at 7 periods are provided in Table 5. As shown in Table 5, the variation in mean magnitudes and distances for different periods is not significant. An average magnitude of 7.44 and average distance of 13.08 km from Table 5 is used in stochastic simulation using SMSIM. The stochastic spectrum is calculated at 309 periods ranging from 0.01 sec to 10 sec, uniformly distributed in logarithmic space.

The simulated spectrum shown in Figure 7 is used to interpolate the discrete spectra (with directivity effects) between periods for which spectral ordinates are available. The smooth UHRS including directivity effects are given in Figure 8 and Table 6.

Table 5. Mean magnitudes and distances obtained from deaggregation of hazard for 7% probability of exceedance in 75 years.

Period (sec)	Mean Magnitude	Mean Distance (km)
2	7.52	15.39
1	7.49	13.94
0.5	7.47	13.04
0.3	7.45	12.63
0.2	7.42	12.39
0.1	7.36	12.32
0.01	7.35	11.84

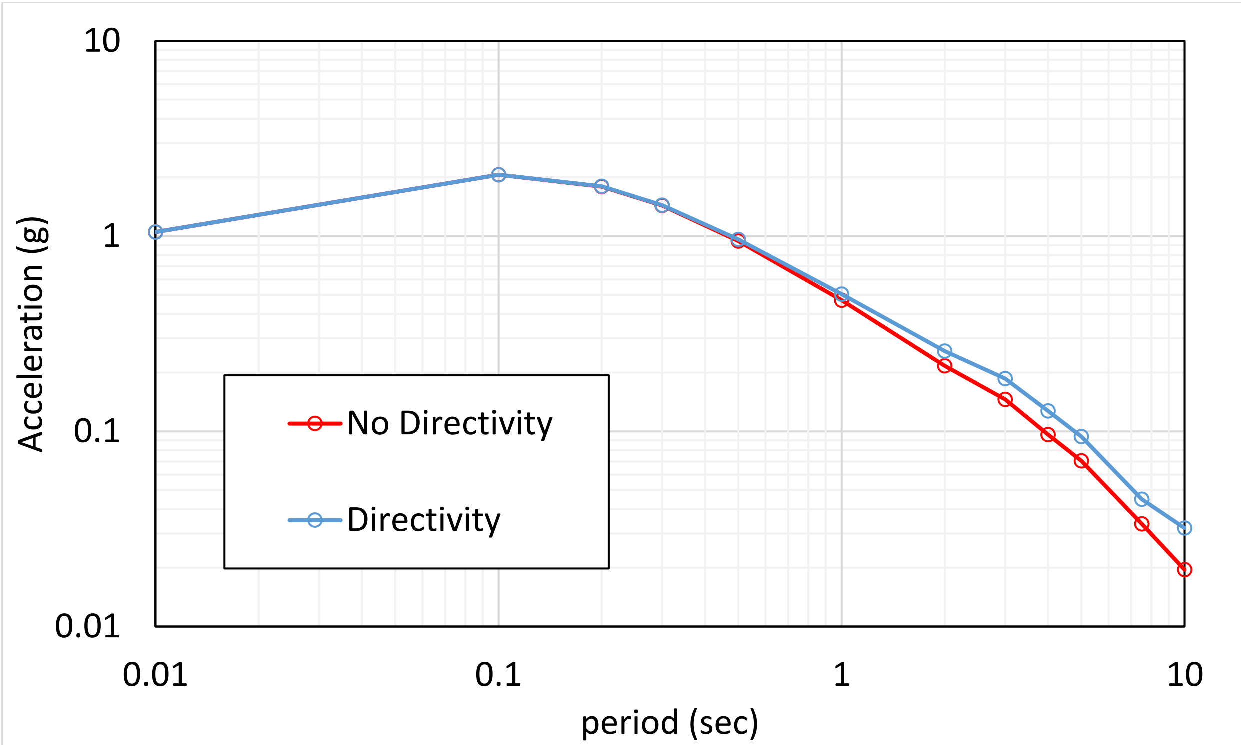


Figure 7. Discrete UHRS for 7% probability of exceedance in 75 years with and without directivity effects.

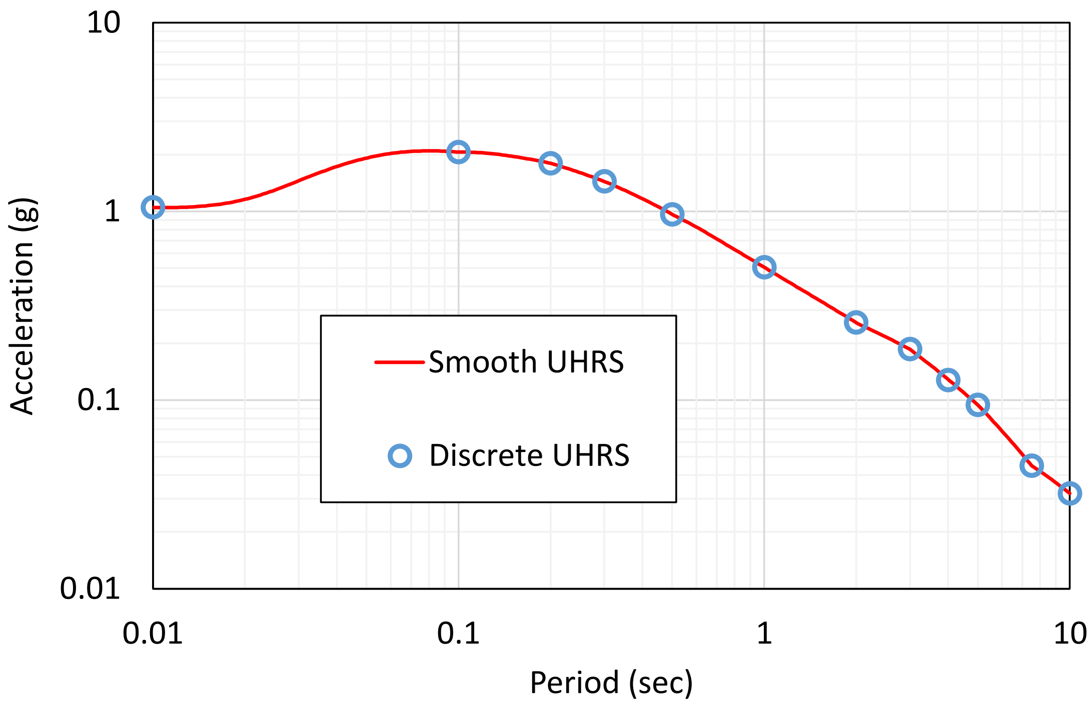


Figure 8. Smooth UHRS for 7% probability of exceedance in 75 years including directivity effects.

Table 6. Smooth UHRS for 7% probability of exceedance in 75 years including directivity effects.

Period (sec)	Smooth UHRS (g)	Period (sec)	Smooth UHRS (g)	Period (sec)	Smooth UHRS (g)	Period (sec)	Smooth UHRS (g)
0.0100	1.0482	0.0241	1.2672	0.0579	2.0100	0.1361	2.0019
0.0102	1.0479	0.0246	1.2841	0.0592	2.0219	0.1393	1.9925
0.0105	1.0475	0.0252	1.3016	0.0606	2.0328	0.1425	1.9824
0.0107	1.0474	0.0258	1.3199	0.0620	2.0429	0.1458	1.9720
0.0110	1.0475	0.0264	1.3388	0.0635	2.0522	0.1493	1.9612
0.0112	1.0475	0.0270	1.3584	0.0650	2.0602	0.1527	1.9502
0.0115	1.0481	0.0276	1.3783	0.0665	2.0673	0.1563	1.9388
0.0118	1.0488	0.0283	1.3988	0.0680	2.0734	0.1600	1.9269
0.0120	1.0493	0.0289	1.4198	0.0696	2.0788	0.1637	1.9150
0.0123	1.0505	0.0296	1.4411	0.0713	2.0832	0.1675	1.9027
0.0126	1.0518	0.0303	1.4628	0.0729	2.0869	0.1714	1.8899
0.0129	1.0534	0.0310	1.4847	0.0746	2.0897	0.1754	1.8771
0.0132	1.0552	0.0317	1.5067	0.0764	2.0918	0.1795	1.8638
0.0135	1.0573	0.0325	1.5290	0.0782	2.0932	0.1837	1.8502
0.0138	1.0597	0.0332	1.5514	0.0800	2.0938	0.1880	1.8365
0.0141	1.0621	0.0340	1.5738	0.0819	2.0936	0.1924	1.8224
0.0145	1.0657	0.0348	1.5964	0.0838	2.0927	0.1969	1.8077
0.0148	1.0689	0.0356	1.6190	0.0857	2.0911	0.2000	1.7978
0.0152	1.0733	0.0365	1.6414	0.0877	2.0889	0.2015	1.7919
0.0155	1.0769	0.0373	1.6638	0.0898	2.0859	0.2062	1.7733
0.0159	1.0820	0.0382	1.6859	0.0919	2.0822	0.2111	1.7539
0.0162	1.0862	0.0391	1.7079	0.0940	2.0779	0.2160	1.7342
0.0166	1.0921	0.0400	1.7296	0.0962	2.0730	0.2210	1.7136
0.0170	1.0983	0.0409	1.7510	0.0985	2.0671	0.2262	1.6931
0.0174	1.1050	0.0419	1.7720	0.1000	2.0632	0.2315	1.6725
0.0178	1.1121	0.0429	1.7927	0.1008	2.0630	0.2369	1.6518
0.0182	1.1196	0.0439	1.8128	0.1031	2.0626	0.2424	1.6310
0.0187	1.1295	0.0449	1.8327	0.1055	2.0615	0.2481	1.6102
0.0191	1.1378	0.0459	1.8518	0.1080	2.0600	0.2539	1.5896
0.0195	1.1464	0.0470	1.8704	0.1105	2.0574	0.2598	1.5691
0.0200	1.1579	0.0481	1.8888	0.1131	2.0546	0.2659	1.5486
0.0205	1.1690	0.0492	1.9062	0.1158	2.0509	0.2721	1.5281
0.0209	1.1808	0.0504	1.9233	0.1185	2.0464	0.2785	1.5076
0.0214	1.1934	0.0516	1.9395	0.1212	2.0411	0.2850	1.4872
0.0219	1.2067	0.0528	1.9550	0.1241	2.0348	0.2917	1.4669
0.0224	1.2207	0.0540	1.9699	0.1270	2.0276	0.2985	1.4464
0.0230	1.2355	0.0553	1.9841	0.1299	2.0196	0.3000	1.4416
0.0235	1.2510	0.0566	1.9974	0.1330	2.0110	0.3055	1.4250

Period (sec)	Smooth UHRS (g)	Period (sec)	Smooth UHRS (g)	Period (sec)	Smooth UHRS (g)	Period (sec)	Smooth UHRS (g)
0.3126	1.4029	0.7696	0.6517	1.8949	0.2715	4.5589	0.1075
0.3199	1.3806	0.7876	0.6374	1.9392	0.2654	4.6655	0.1040
0.3274	1.3580	0.8060	0.6235	1.9845	0.2594	4.7745	0.1007
0.3350	1.3350	0.8249	0.6098	2.0000	0.2574	4.8861	0.0974
0.3429	1.3121	0.8442	0.5963	2.0309	0.2535	5.0000	0.0942
0.3509	1.2896	0.8639	0.5830	2.0784	0.2492	5.0003	0.0941
0.3591	1.2673	0.8841	0.5701	2.1270	0.2448	5.1172	0.0906
0.3675	1.2449	0.9047	0.5573	2.1767	0.2406	5.2368	0.0871
0.3761	1.2230	0.9259	0.5449	2.2275	0.2363	5.3592	0.0837
0.3848	1.2014	0.9475	0.5327	2.2796	0.2322	5.4844	0.0804
0.3938	1.1796	0.9697	0.5209	2.3329	0.2281	5.6126	0.0772
0.4030	1.1587	0.9923	0.5093	2.3874	0.2240	5.7438	0.0741
0.4125	1.1377	1.0000	0.5054	2.4432	0.2200	5.8780	0.0711
0.4221	1.1168	1.0155	0.4979	2.5003	0.2160	6.0154	0.0682
0.4320	1.0960	1.0393	0.4868	2.5587	0.2120	6.1560	0.0654
0.4421	1.0751	1.0635	0.4761	2.6186	0.2081	6.2999	0.0627
0.4524	1.0539	1.0884	0.4655	2.6798	0.2043	6.4471	0.0601
0.4630	1.0327	1.1138	0.4552	2.7424	0.2005	6.5978	0.0575
0.4738	1.0118	1.1399	0.4452	2.8065	0.1967	6.7520	0.0550
0.4849	0.9913	1.1665	0.4353	2.8721	0.1929	6.9098	0.0527
0.4962	0.9708	1.1938	0.4257	2.9392	0.1892	7.0713	0.0504
0.5000	0.9642	1.2217	0.4163	3.0000	0.1860	7.2366	0.0481
0.5078	0.9519	1.2502	0.4072	3.0079	0.1854	7.4057	0.0460
0.5197	0.9338	1.2795	0.3981	3.0782	0.1801	7.5000	0.0448
0.5318	0.9161	1.3094	0.3894	3.1501	0.1750	7.5788	0.0444
0.5442	0.8986	1.3400	0.3807	3.2238	0.1700	7.7559	0.0433
0.5570	0.8814	1.3713	0.3723	3.2991	0.1652	7.9372	0.0422
0.5700	0.8643	1.4033	0.3640	3.3762	0.1604	8.1227	0.0412
0.5833	0.8472	1.4361	0.3560	3.4551	0.1557	8.3125	0.0401
0.5969	0.8302	1.4697	0.3481	3.5359	0.1512	8.5068	0.0391
0.6109	0.8130	1.5040	0.3404	3.6185	0.1467	8.7056	0.0380
0.6252	0.7956	1.5392	0.3328	3.7031	0.1424	8.9091	0.0370
0.6398	0.7785	1.5752	0.3254	3.7896	0.1381	9.1173	0.0360
0.6547	0.7615	1.6120	0.3182	3.8782	0.1340	9.3304	0.0349
0.6700	0.7448	1.6497	0.3111	3.9688	0.1299	9.5485	0.0339
0.6857	0.7284	1.6882	0.3041	4.0616	0.1260	9.7716	0.0329
0.7017	0.7123	1.7277	0.2973	4.1565	0.1221	10.0000	0.0320
0.7181	0.6966	1.7680	0.2907	4.2537	0.1183		
0.7349	0.6813	1.8094	0.2842	4.3531	0.1146		
0.7521	0.6663	1.8517	0.2778	4.4548	0.1110		

The results of the PSHA (the smooth UHRS) and the de-aggregation were used to select earthquakes for the site response analyses. Seven horizontal components (total of 14 time histories) of previously recorded earthquakes within the range of de-aggregation magnitudes and distances and only pulse-like records were selected. The UHRS was selected as the target spectrum and the selected time histories are then matched with the target spectrum. Figure 9 shows A typical plot of one of the selected seed records. Both the seed record and the matched record acceleration, velocity, displacement, intensities are shown in Figure 9.

The top frame of Figure 10 shows all selected records matched with the target spectrum. The bottom frame of Figure 10 shows the target spectrum and the average of all matched Spectra. Figure 11 shows a typical seed ground motion (Chi Chi earthquake), target spectrum, and the matched target spectrum. Figure 12 is the same as Figure 11 but shown in a log scale.

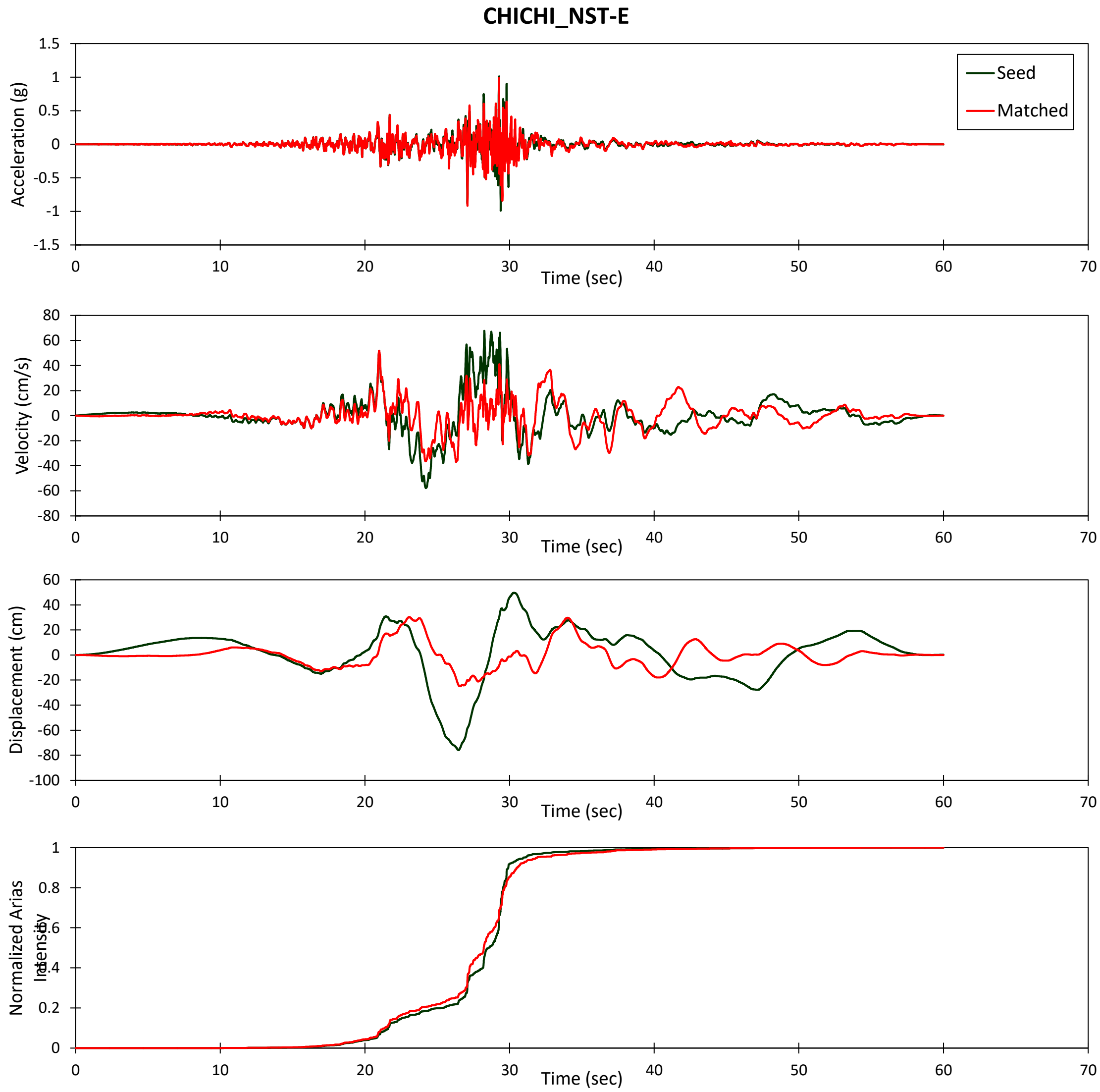


Figure 9. Typical plot of a seed ground motion as well as the matched ground motion: (a) acceleration time series, (b) velocity time series, (c) displacement time series, and (4) The normalized intensity.

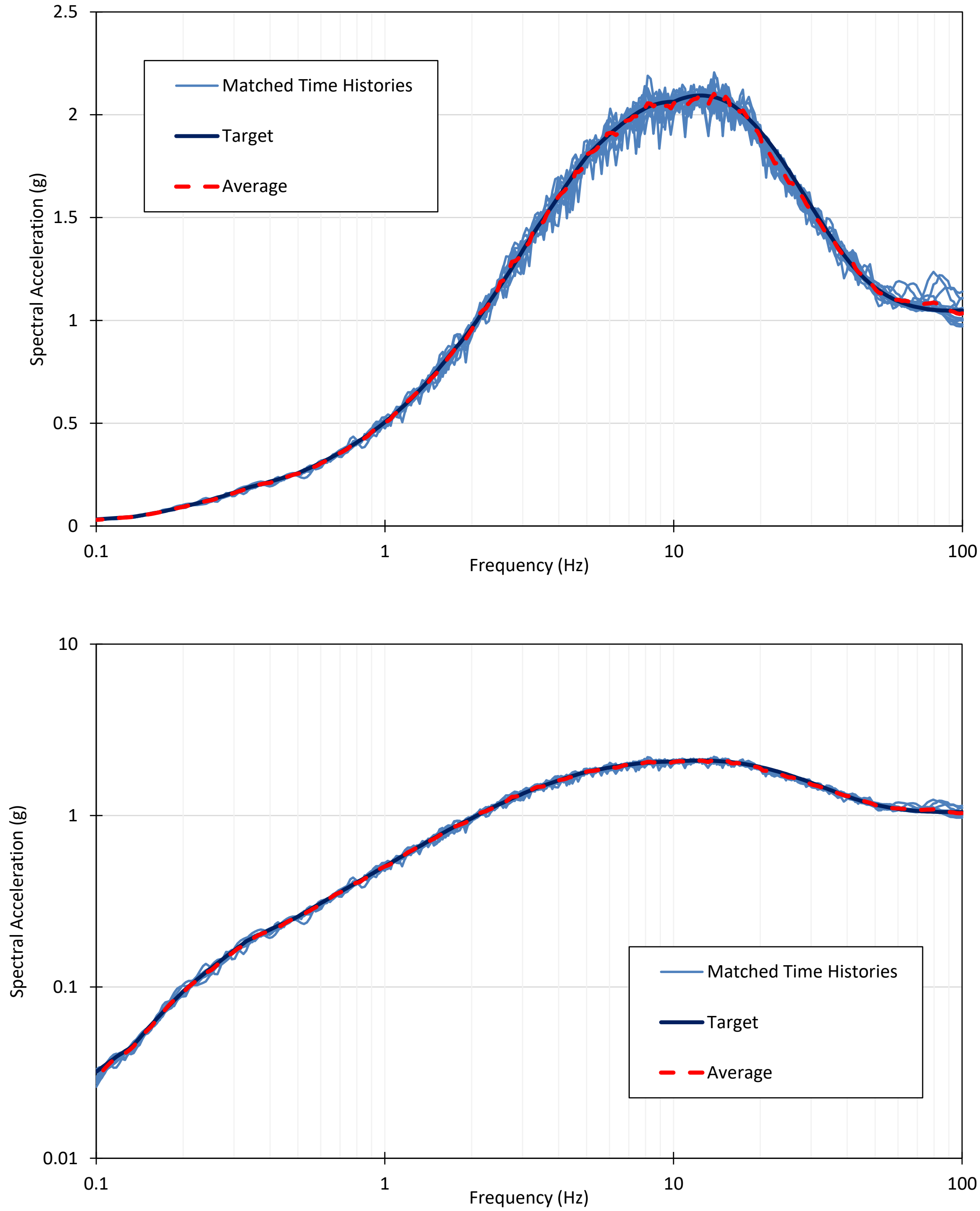


Figure 10. Top frame shows all selected records matched with the target spectrum. The bottom frame shows the target spectrum and the average of all matched Spectra.

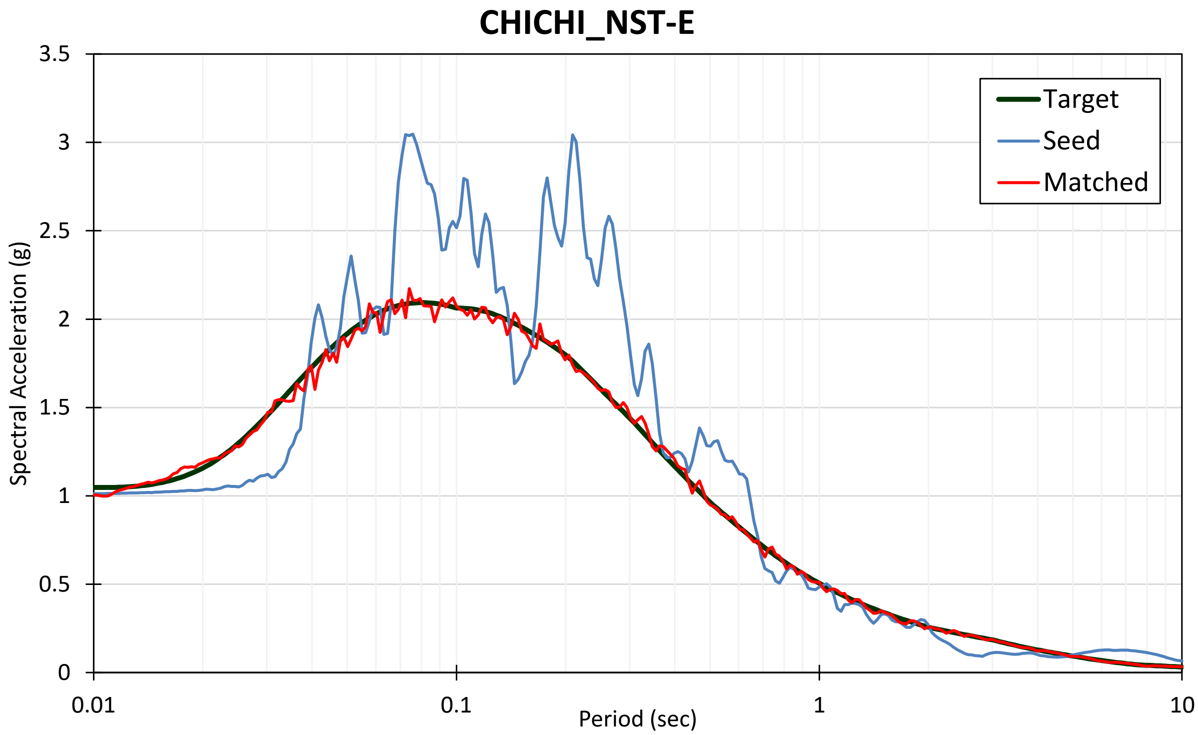


Figure 11. A typical seed ground motion (Chi Chi earthquake), target spectrum, and the matched target spectrum.

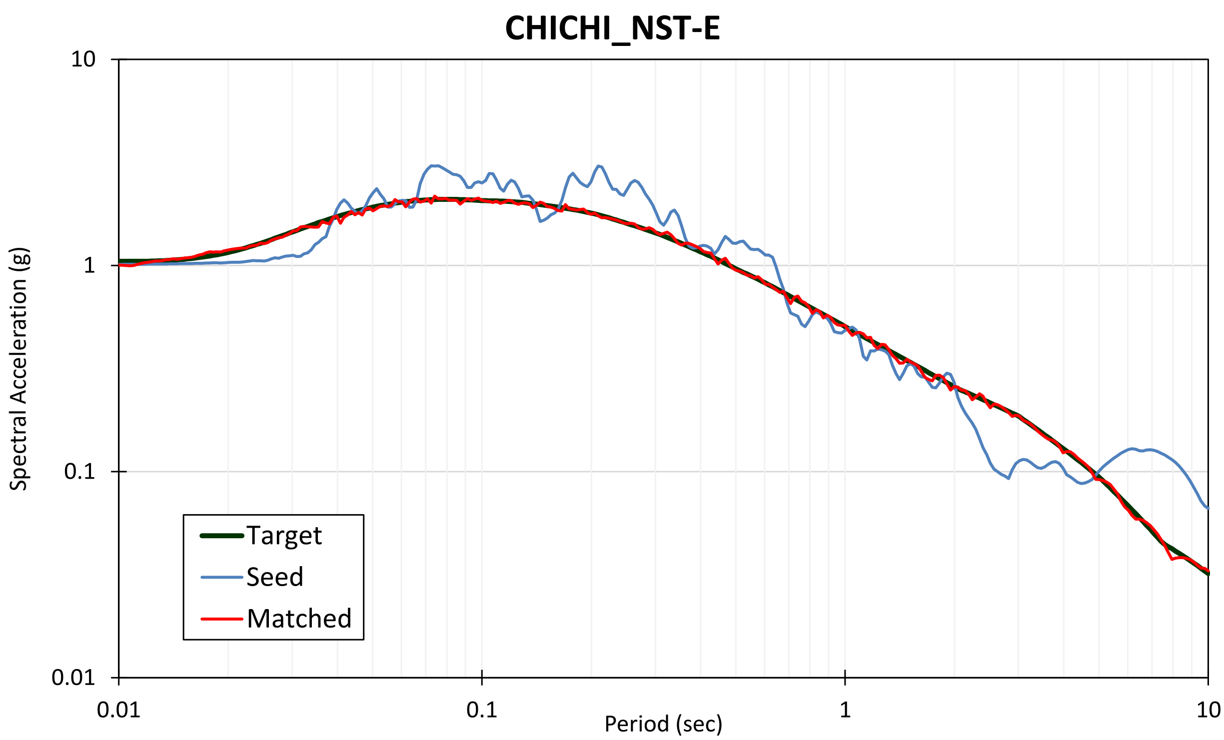


Figure 12. A typical seed ground motion (Chi Chi earthquake), target spectrum, and the matched target spectrum plotted in a log scale.

8.2. Variability in Soil's Shear-Wave and Thickness Profile

Using the EPRI (1993) soil profile database, Toro (1993) developed a probabilistic characterization of a soil shear-wave velocity profile and used the resulting probabilistic model to simulate shear-wave profiles. His probabilistic model consists of two separate components; one for the thickness of each layer called the layering model that captures the variability in the thickness of soil layers; and one for the shear-wave velocity associated with each layer called the velocity model to account for the variability in shear-wave velocity of each layer. Based on the data from EPRI (1993), a non-homogenous Poisson model is used with depth-dependent rate to account for the fact that soil thickness of layers increases with depth.

In this project, the variability in soil thickness and the shear-wave velocity is taken into account which generates a desired number of soil profiles around the base soil profile with a desired probability distribution. This model statistically captures the soil layer shear-wave velocity and thickness uncertainties and their correlation with depth.

Extreme values of shear-wave velocities are rejected by using the truncated distribution model of ε at 2 standard deviations. A coefficient of variation (COV) of 0.15 is used for the shear-wave velocity and a coefficient of variation (COV) of 0.05 is used for the layer thicknesses below the data provided by the downhole seismic survey to generate soil profiles. A total of 60 cases were generated. These 60 soil profiles are used to capture the soil layer shear-wave velocity and thickness uncertainties and their correlation with depth.

8.3. Equivalent Linear Site Response Analyses

Among the available programs for site response analysis, the most widely used is the SHAKE91 computer program (Idriss and Sun, 1992; Cramer, 2006; Hartzel *et al.*, 2004; Wen and Wu, 1999). The computer program SHAKE91 employs the equivalent linear method to compute the response of horizontally layered soil deposits underlain by horizontal bedrock.

8.4. Site-Specific Results

Following the procedure outlined above, the site-specific response spectra were obtained by analyzing 60 profiles for each matched ground motion with the UHRS. Figures are available upon request, but not presented in this report. Table 7 provides the response spectra from site-specific, AASHTO response spectra, and the final site-specific response spectrum. Site-specific analyses were performed using data from both CPT1 and CPT2 locations, the maximum of the two studies have been selected as the recommended site-specific response spectrum and shown in Figure 13.

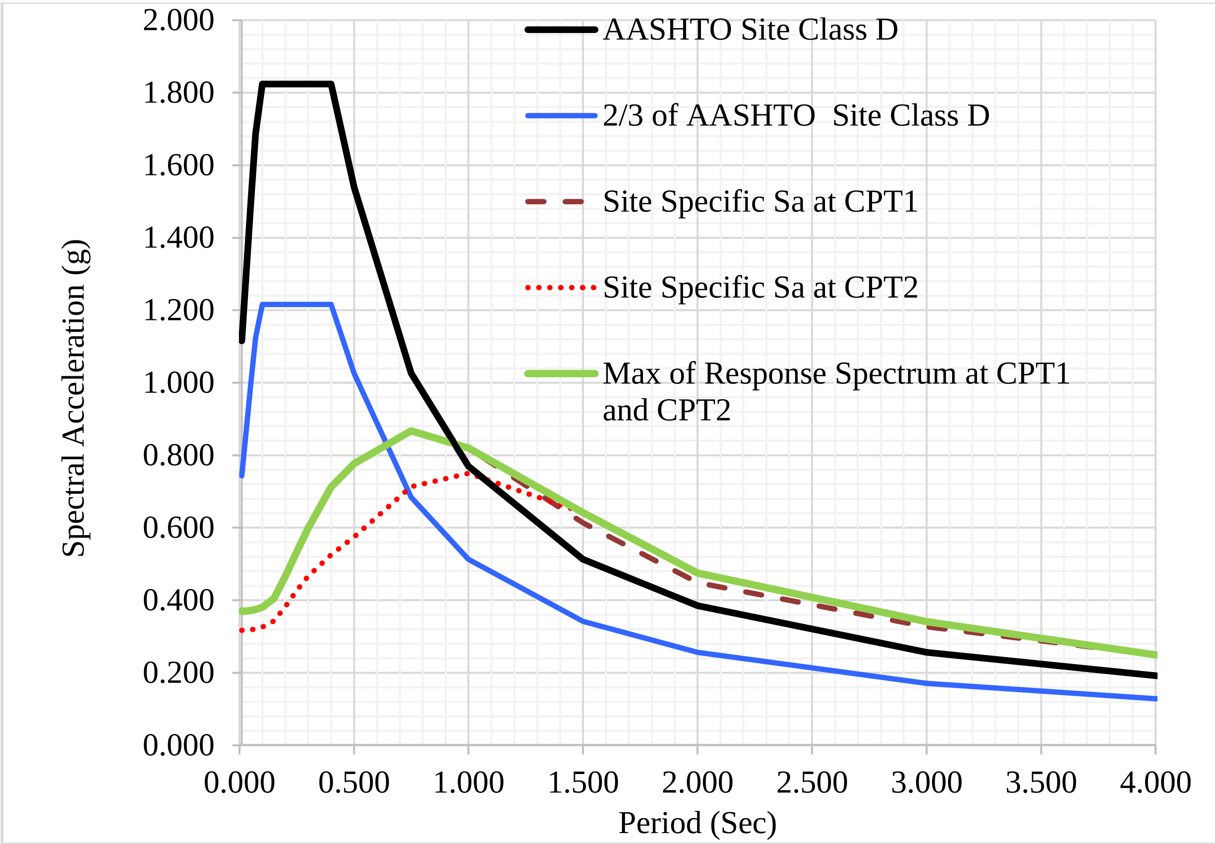


Figure 13. Site-specific response spectrum, AASHTO response spectrum for the site class D, and 2/3 of the AASHTO response spectrum for the site class D.

Figure 14 shows response spectra for the site-specific geometric mean, the AASHTO LRFD Bridge Design Specifications, 7th Edition with 2016 Interim Revisions, and 2/3 of AASHTO. Site-specific analyses were performed using data from both CPT1 and CPT2 locations and the maximum of the two studies are selected as the recommend site-specific response spectrum.

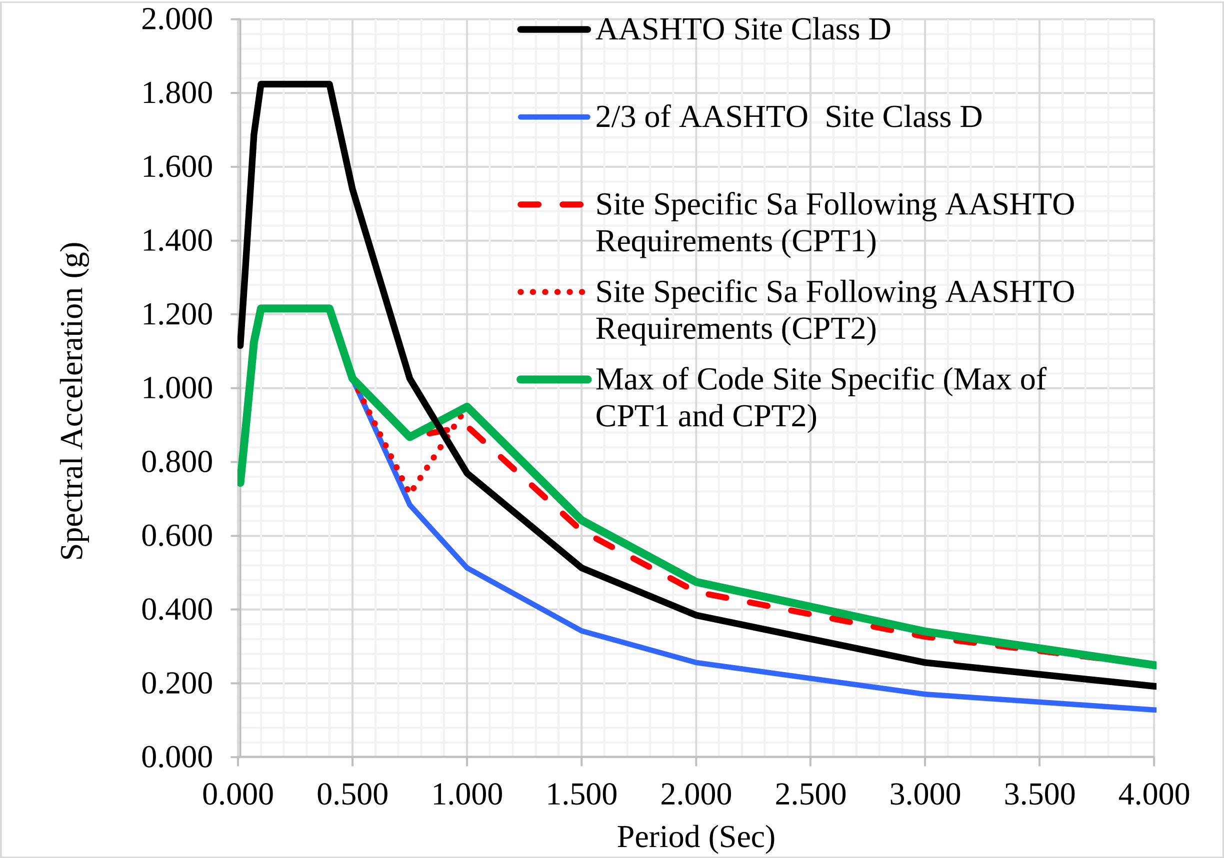


Figure 14. Site-specific response spectrum following AASHTO requirements, AASHTO response spectrum for the site class D, and 2/3 of the AASHTO response spectrum for the site class D.

Table 7. AASHTO, and Site-Specific Response Spectra.

Period	AASHTO Response Sa	2/3 of AASHTO Sa	Site Specific Sa at CPT1	Site Specific Sa at CPT2	Site Specific Sa Maximum of at CPT1 and CPT2
(s)	(g)	(g)	(g)	(g)	(g)
0.010	1.115	**0.744**	0.744	0.744	0.744
0.030	1.306	**0.871**	0.871	0.871	0.871
0.040	1.401	**0.934**	0.934	0.934	0.934
0.050	1.496	**0.998**	0.998	0.998	0.998
0.070	1.687	**1.125**	1.125	1.125	1.125
0.100	1.824	**1.216**	1.216	1.216	1.216
0.150	1.824	**1.216**	1.216	1.216	1.216
0.200	1.824	**1.216**	1.216	1.216	1.216
0.250	1.824	**1.216**	1.216	1.216	1.216
0.300	1.824	**1.216**	1.216	1.216	1.216
0.400	1.824	**1.216**	1.216	1.216	1.216
0.500	1.539	**1.026**	1.026	1.026	1.026
0.750	1.026	**0.684**	0.867	0.713	0.867
1.000	0.770	**0.513**	0.897	0.950	0.950
1.500	0.513	**0.342**	0.613	0.642	0.642
2.000	0.385	**0.257**	0.448	0.475	0.475
3.000	0.257	**0.171**	0.327	0.341	0.341
4.000	0.192	**0.128**	0.249	0.250	0.250
5.000	0.154	**0.103**	0.181	0.180	0.181
7.500	0.103	**0.068**	0.070	0.073	0.073
10.000	0.081	**0.054**	0.054	0.054	0.054

9.0 DESIGN RESPONSE SPECTRAL PARAMETERS

The values of the Design Spectral Response Acceleration parameters are listed in Table 8 and plotted in Figure 15 developed in accordance with AASHTO.

Table 8. Site-Specific Response Accelerations at 5% Damping.

PARAMETER	DESIGN ACCELERATION PARAMETERS (g)
A_s	0.680
S_{DS}	1.244
S_{D1}	0.952

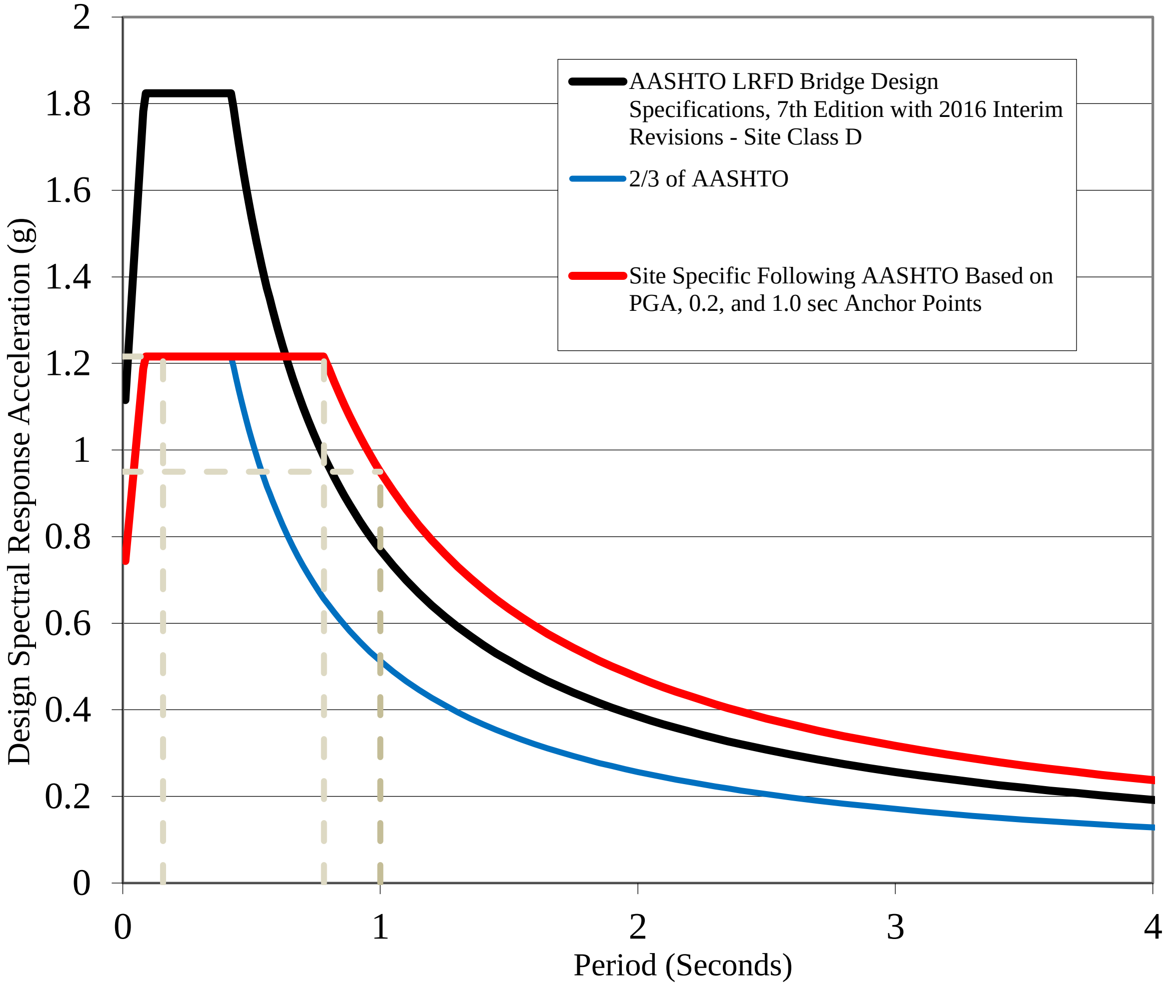


Figure 15. Site-specific response spectrum and AASHTO response spectrum for the site class D.

10.0 LIMITATIONS OF REPORT

The analyses, conclusions, and recommendations presented in this report are professional opinions based on the site conditions and project layout described herein, and further assume that the conditions provided in the Geotechnical report are representative of the subsurface conditions throughout the site, i.e., that the subsurface conditions elsewhere on the site are the same as those disclosed by the borings. If, during construction, subsurface conditions different from those encountered in the exploratory boring are observed or appear to be present, the Client must contact us immediately so that we can make changes to this report if needed. The scope of our services did not include an assessment of the effects of flooding and natural erosion on the project site. No liquefaction studies were performed by the author.

This report was prepared for the exclusive use of the owner, architect, and engineer for evaluating the design of the project as it relates to ground response discussed in this report. *This report is copyrighted and not to be distributed.*

11.0 REFERENCES

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APPENDIX. LOCATION OF CPT1 and CPT2

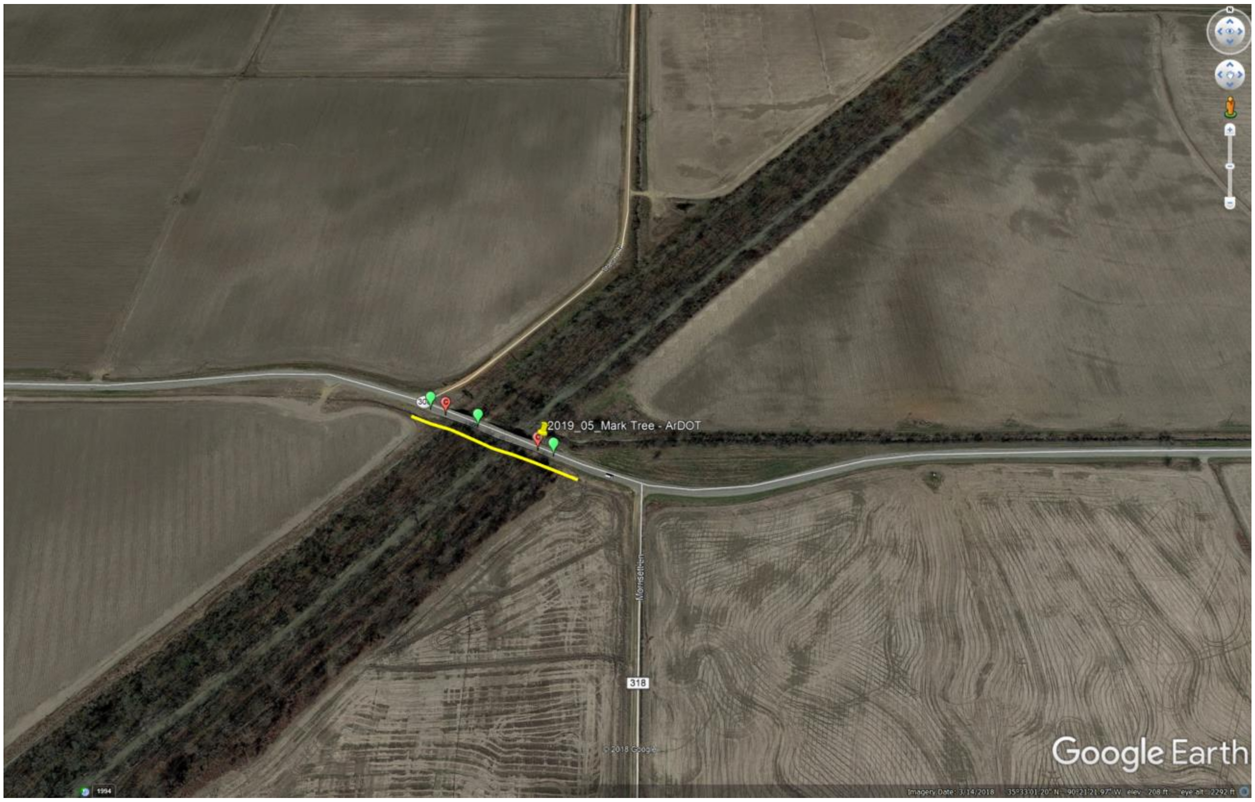
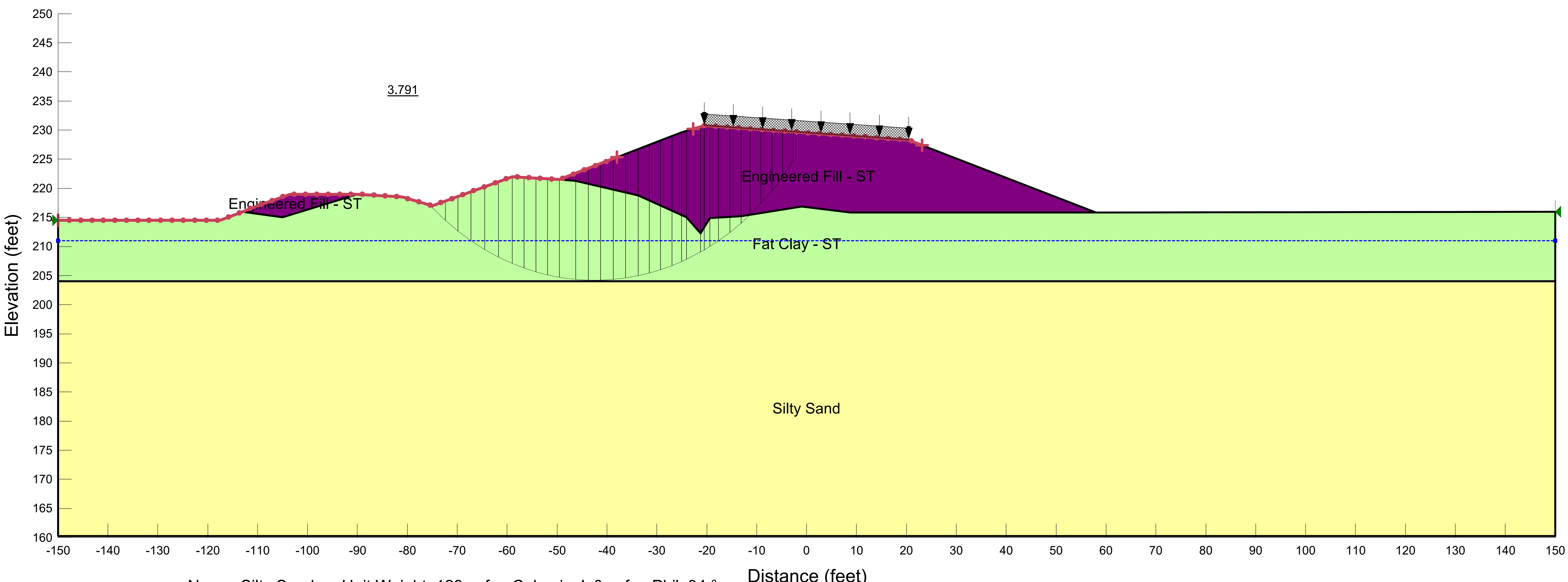


Figure A.1. Location of CPT1 and CPT2 shown by red markers.

APPENDIX G - GLOBAL STABILITY ANALYSES

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: North Side Slope Sta 212+00 ST, Ordinary High Water
Description: Short-Term, GWT El 211



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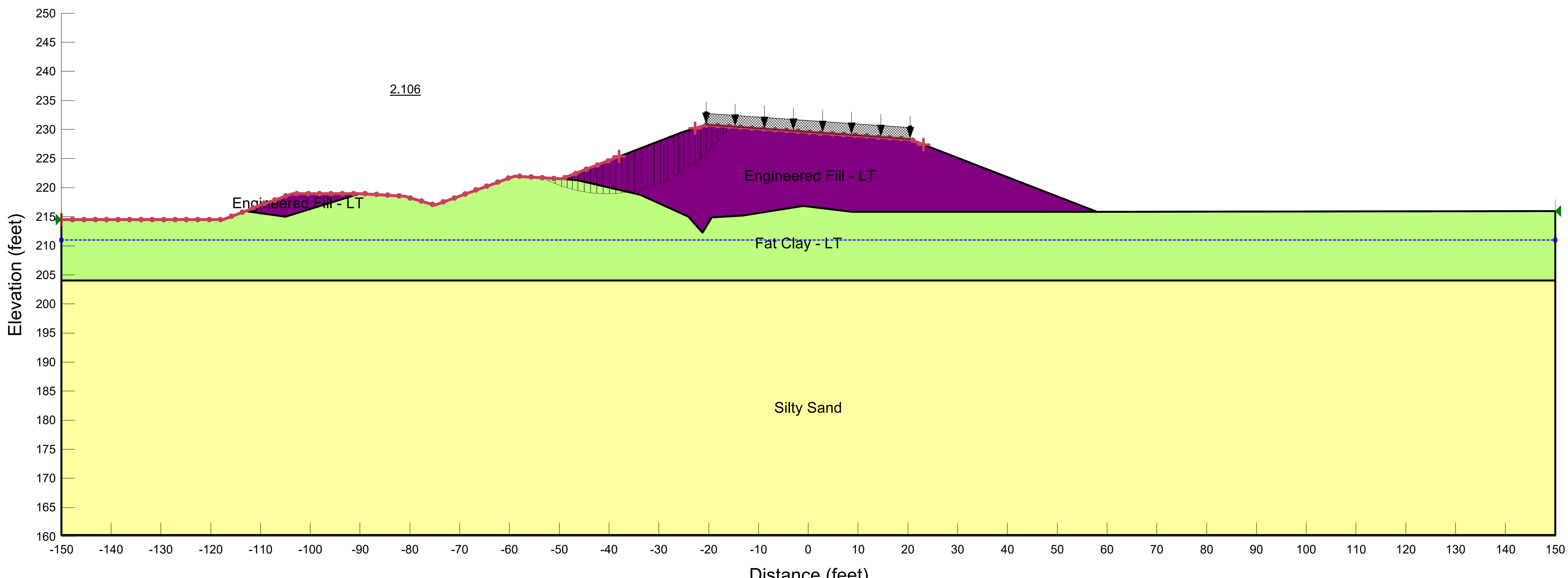
Name: Engineered Fill - ST Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 °
Name: Fat Clay - ST Unit Weight: 116 pcf Cohesion': 900 psf Phi': 0 °

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: North Side Slope Sta 212+00 LT, Ordinary High Water
Description: Long-Term, GWT El 211



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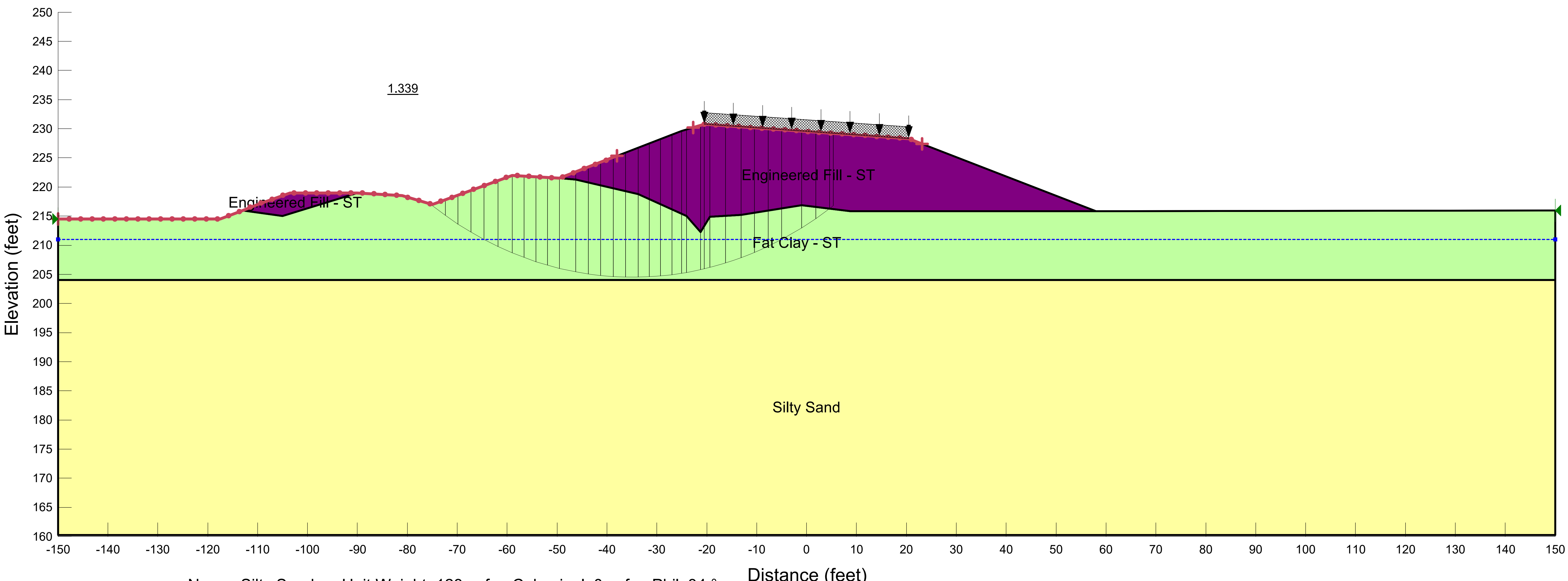
Name: Fat Clay - LT Unit Weight: 116 pcf Cohesion': 50 psf Phi': 21 °
Name: Silty Sand Unit Weight: 123 pcf Cohesion': 0 psf Phi': 34 °

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: North Side Slope Sta 212+00 EQ, Ordinary High Water
Description: Seismic Condition, GWT El 211
Seismic Coef.: 0.32



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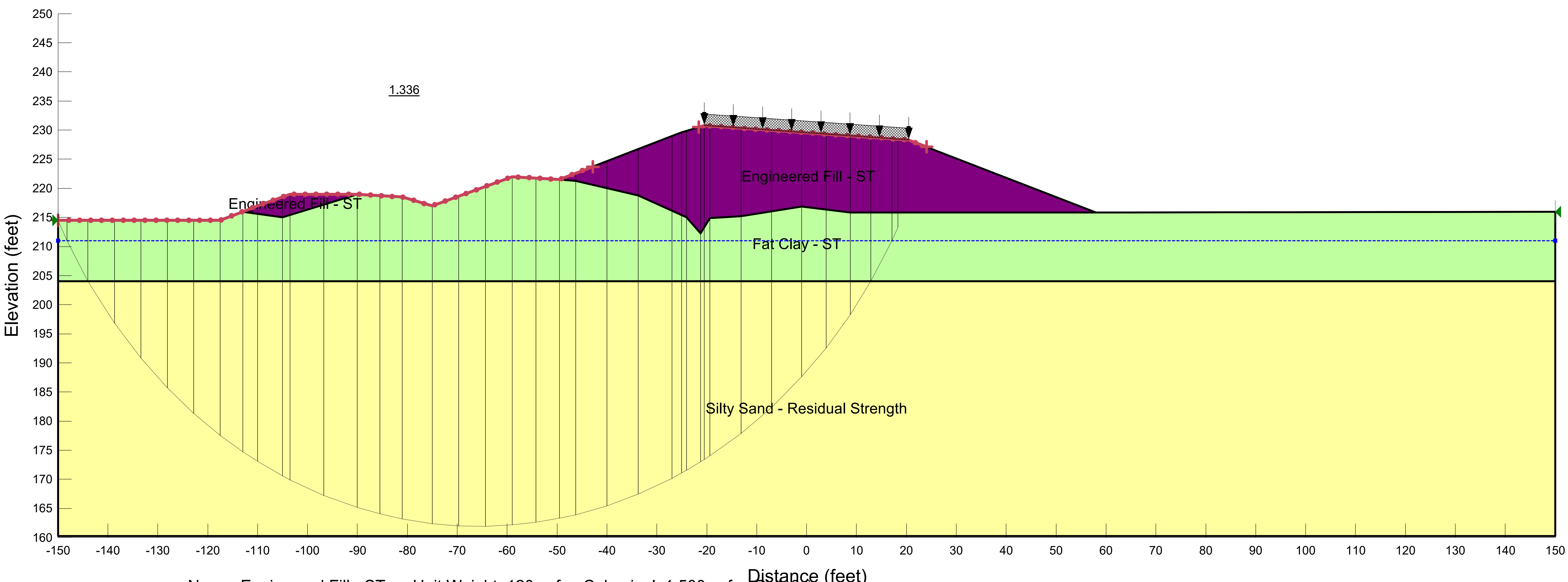
Name: Fat Clay - ST Unit Weight: 116 pcf Cohesion': 900 psf Phi': 0 °

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: North Side Slope Sta 212+00 Residual Strength, Low Groundwater
Description: Post-Seismic Condition, GWT El 203



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Name: Fat Clay - ST Unit Weight: 116 pcf Cohesion': 900 psf Phi': 0 °
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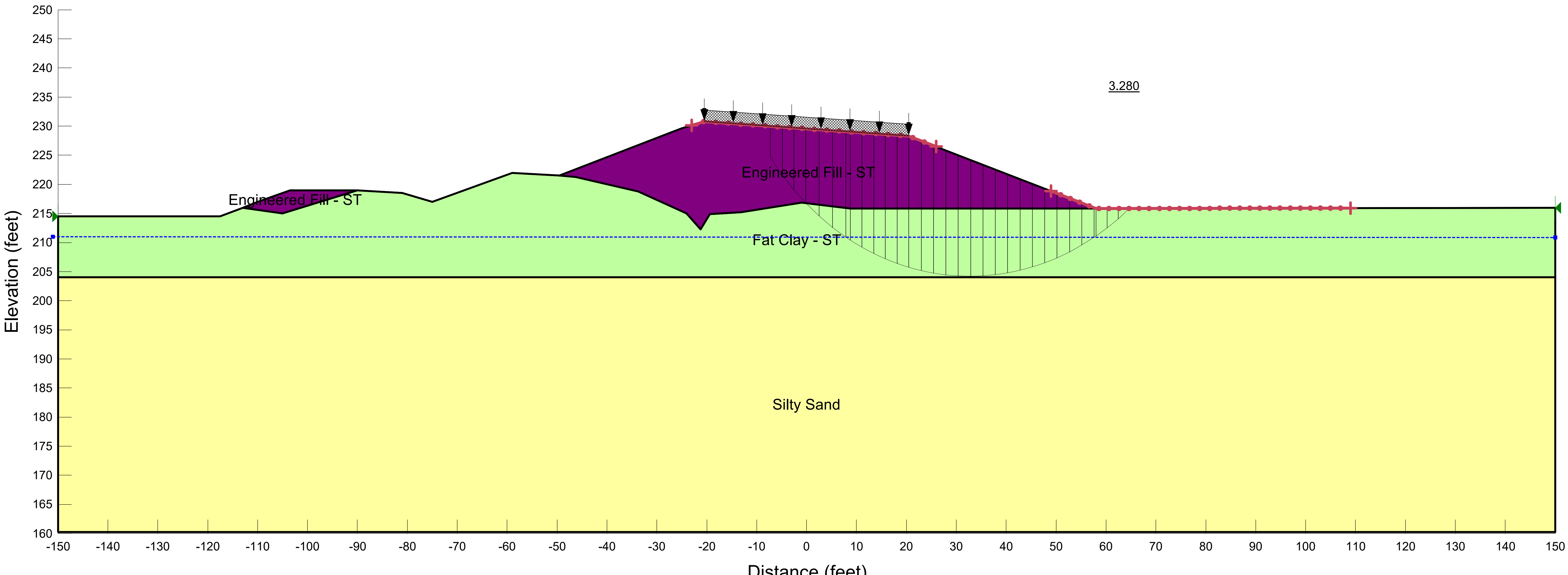
ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: South Side Slope Sta 212+00 ST, Ordinary High Water
Description: Short Term, GWT El 211



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ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
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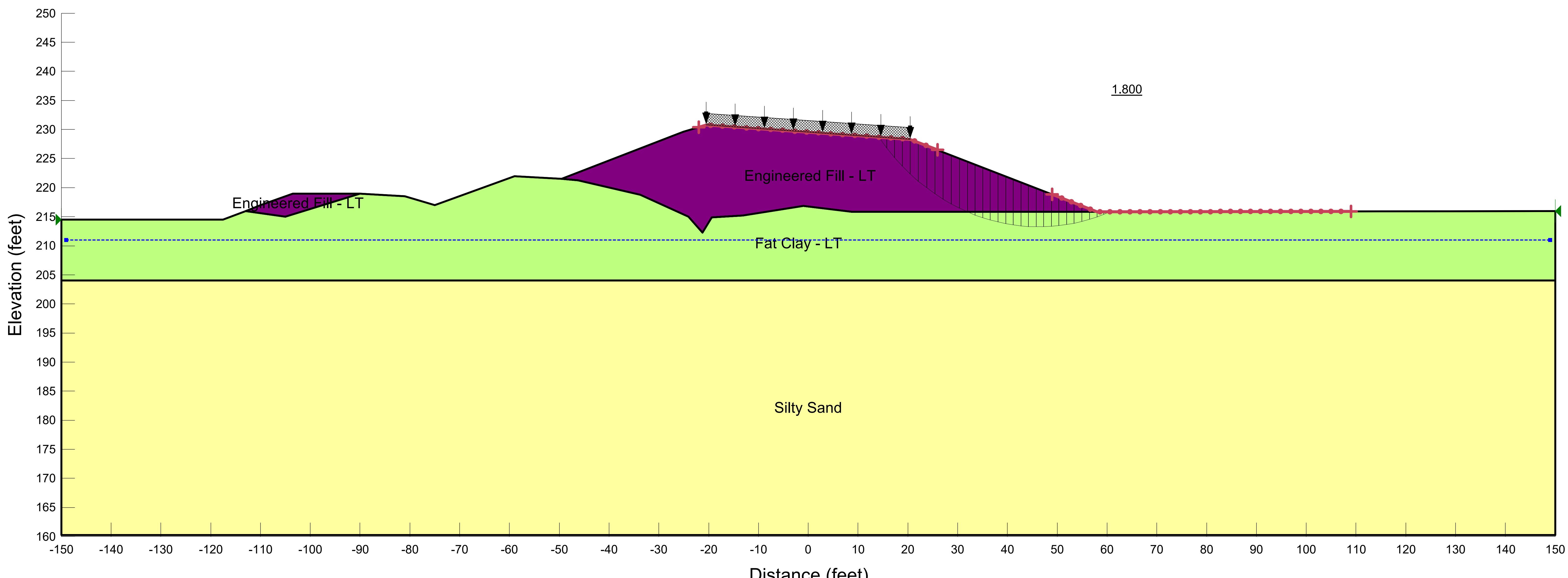
Description: Long Term, GWT EL 211



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ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
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Global Stability Analysis
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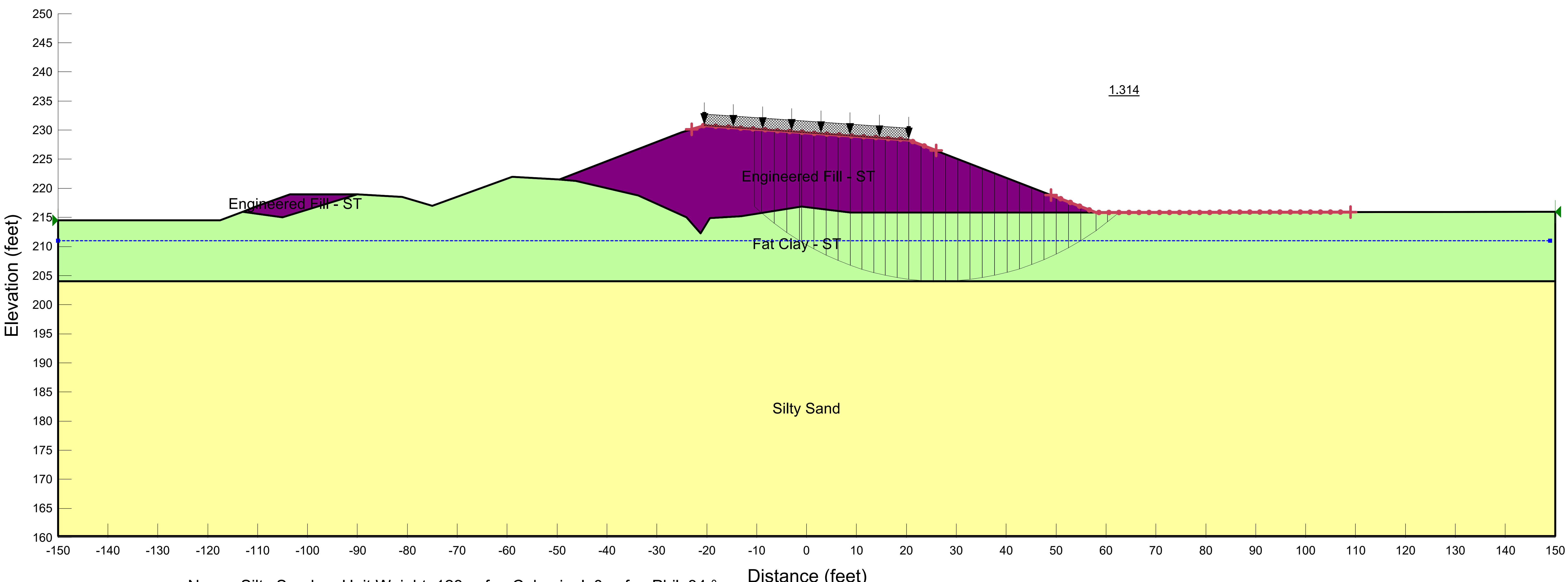
Seismic Coef.: 0.32



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ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
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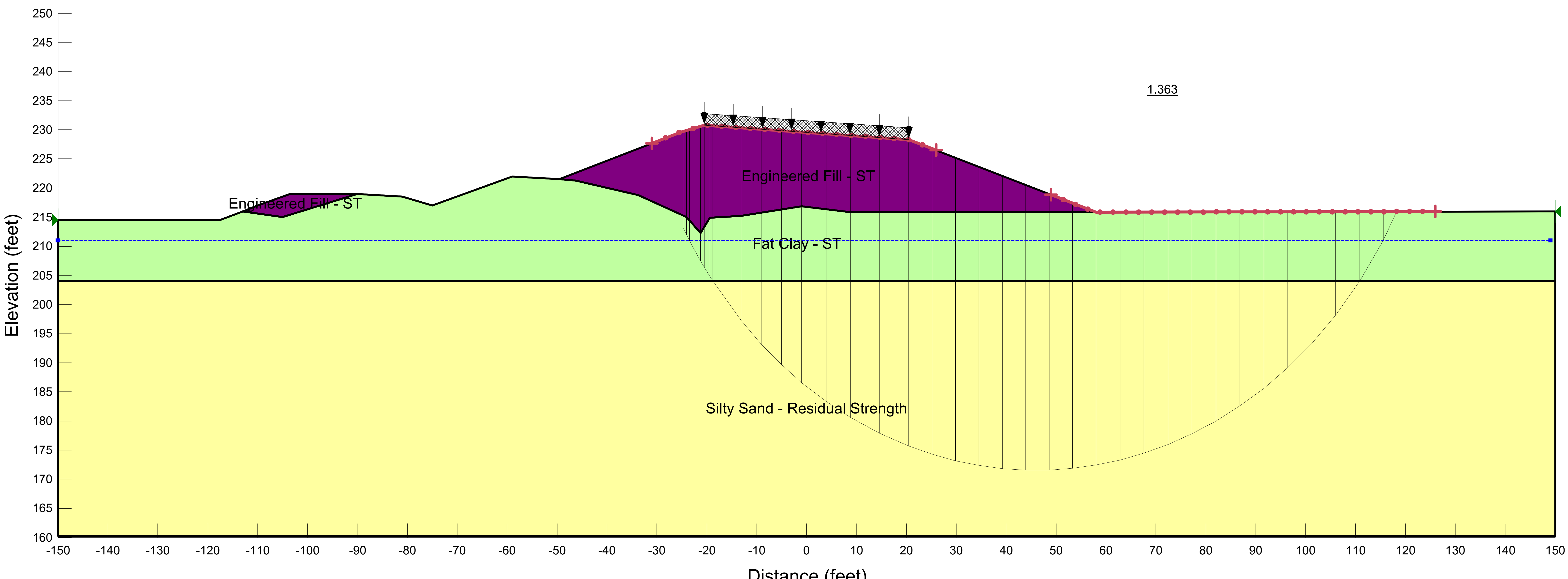
Description: Post-Seismic Condition, GWT El 203



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ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
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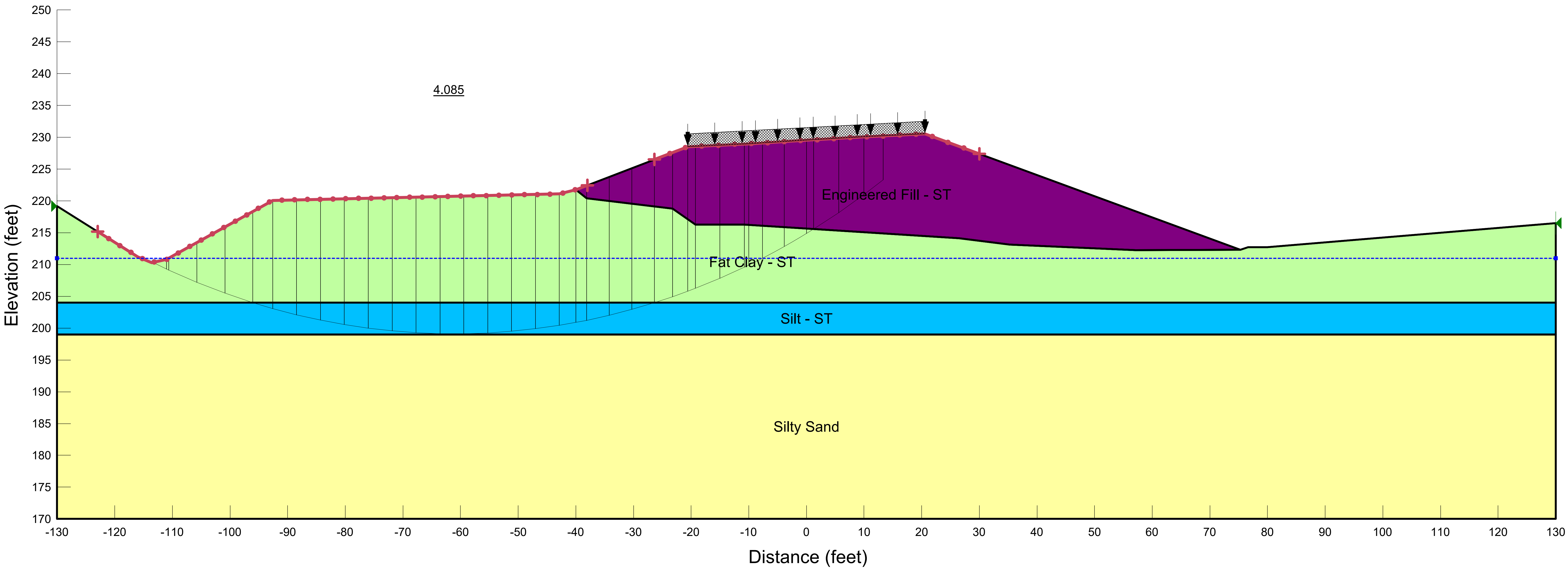

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Ditch Nos. 1 & 47 Strs. & Apprs. (S)
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Geotechnology Project No. J034298.01
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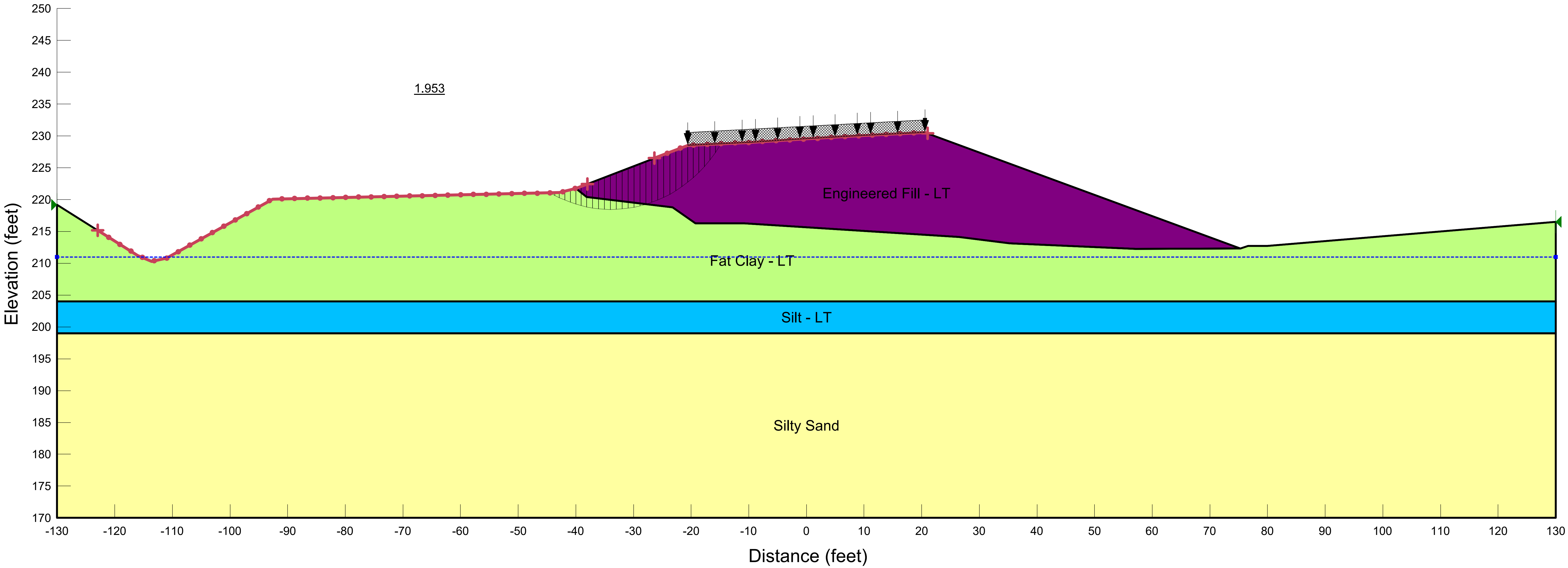

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Poinsett County, Arkansas
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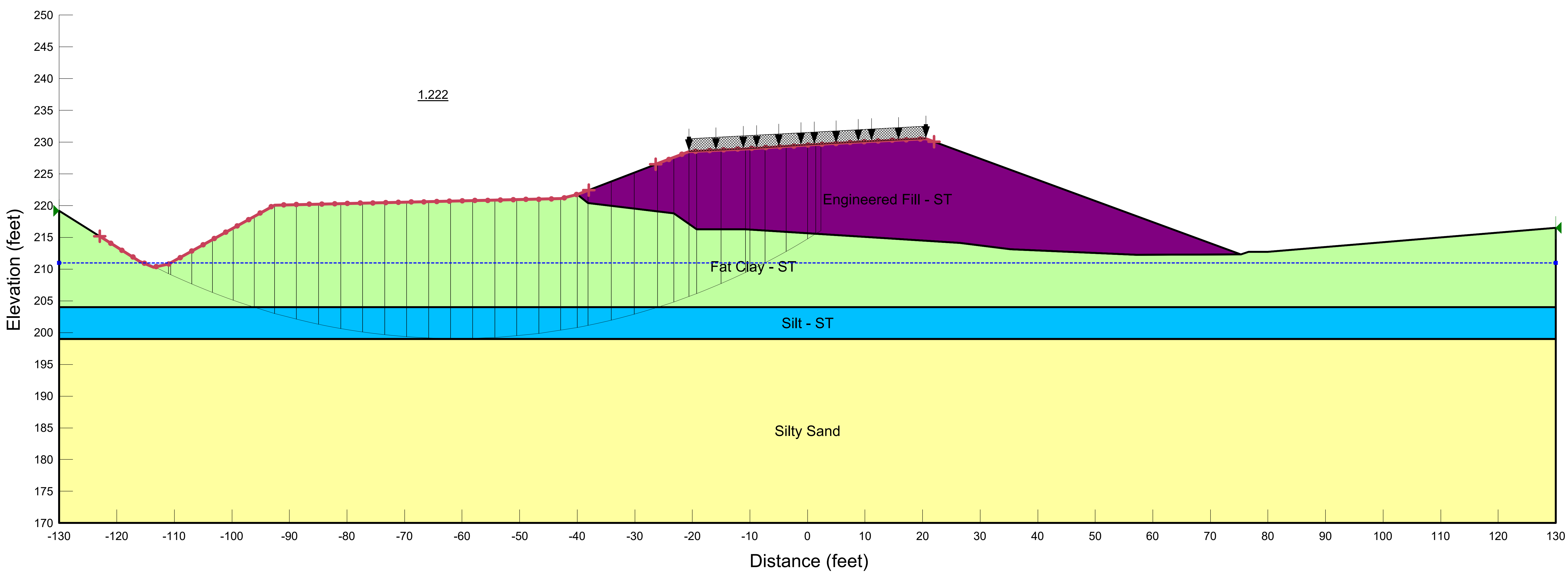
Description: Seismic Condition, GWT El 211
Seismic Coef.: 0.32



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ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
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Global Stability Analysis
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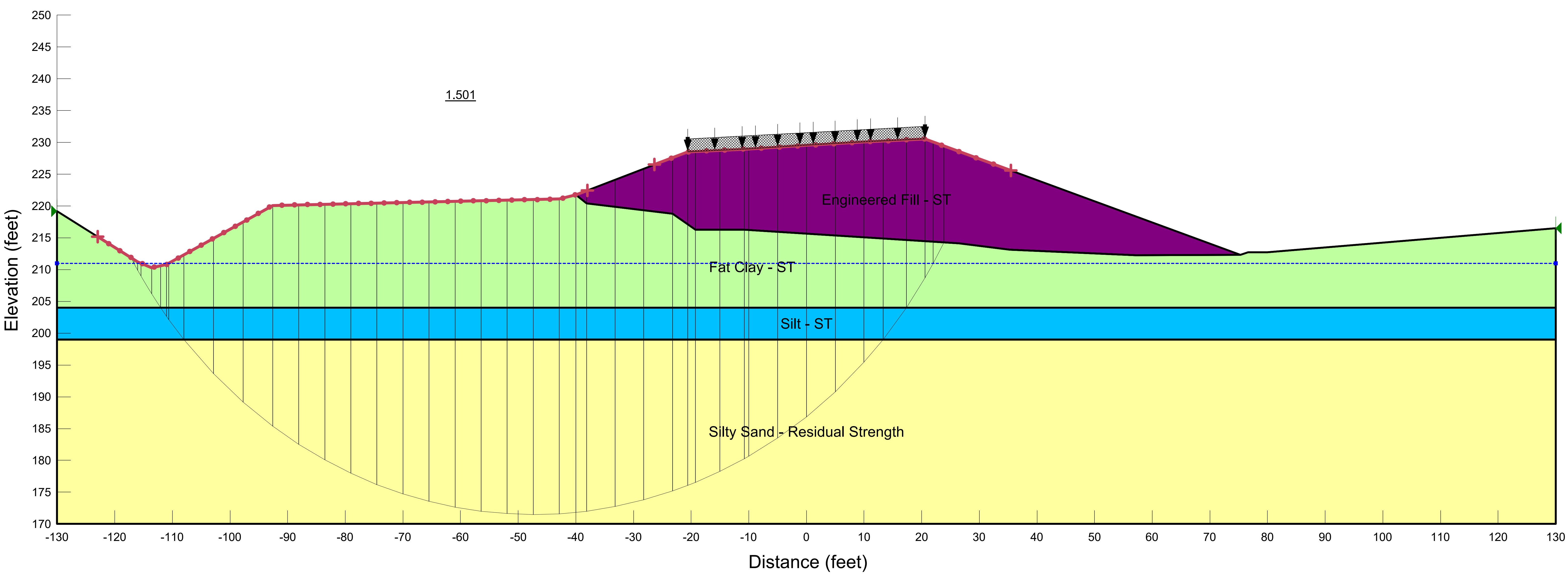
Description: Post-Seismic Condition, GWT El 211



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Name: Silty Sand - Residual Strength Unit Weight: 123 pcf Cohesion': 350 psf Phi': 0 °

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
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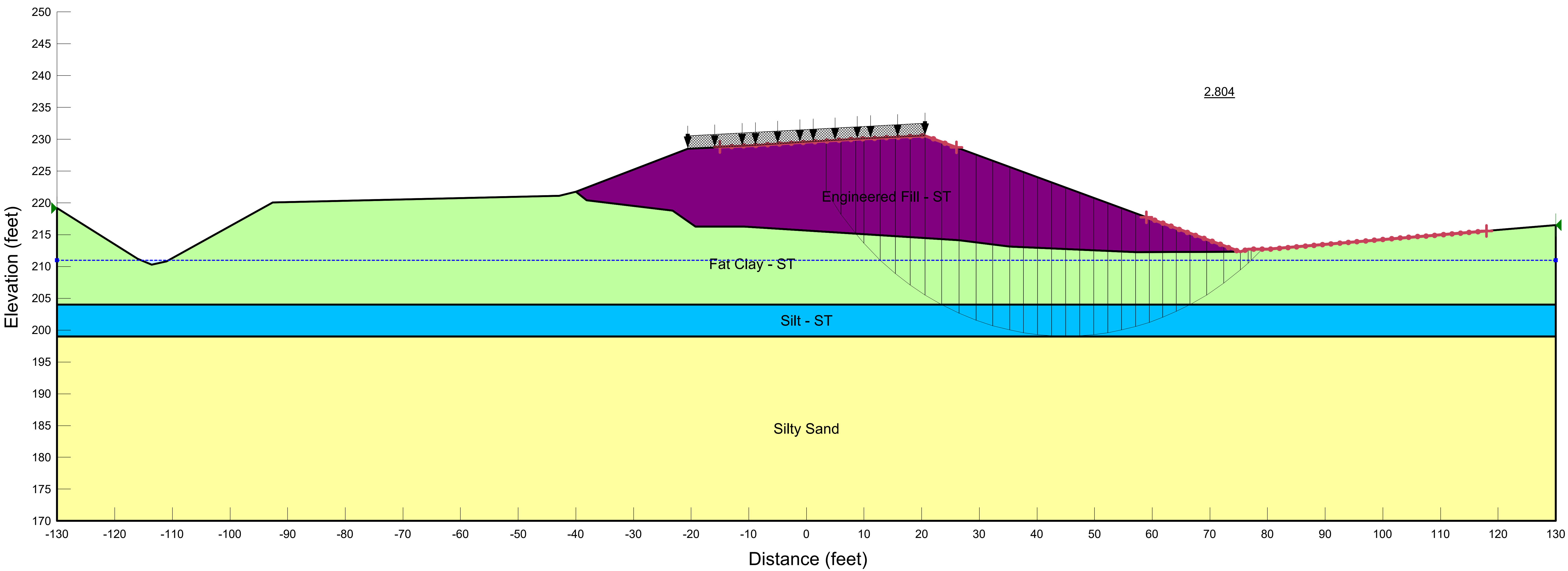
Description: Short Term, GWT El 211



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Ditch Nos. 1 & 47 Strs. & Apprs. (S)
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Geotechnology Project No. J034298.01
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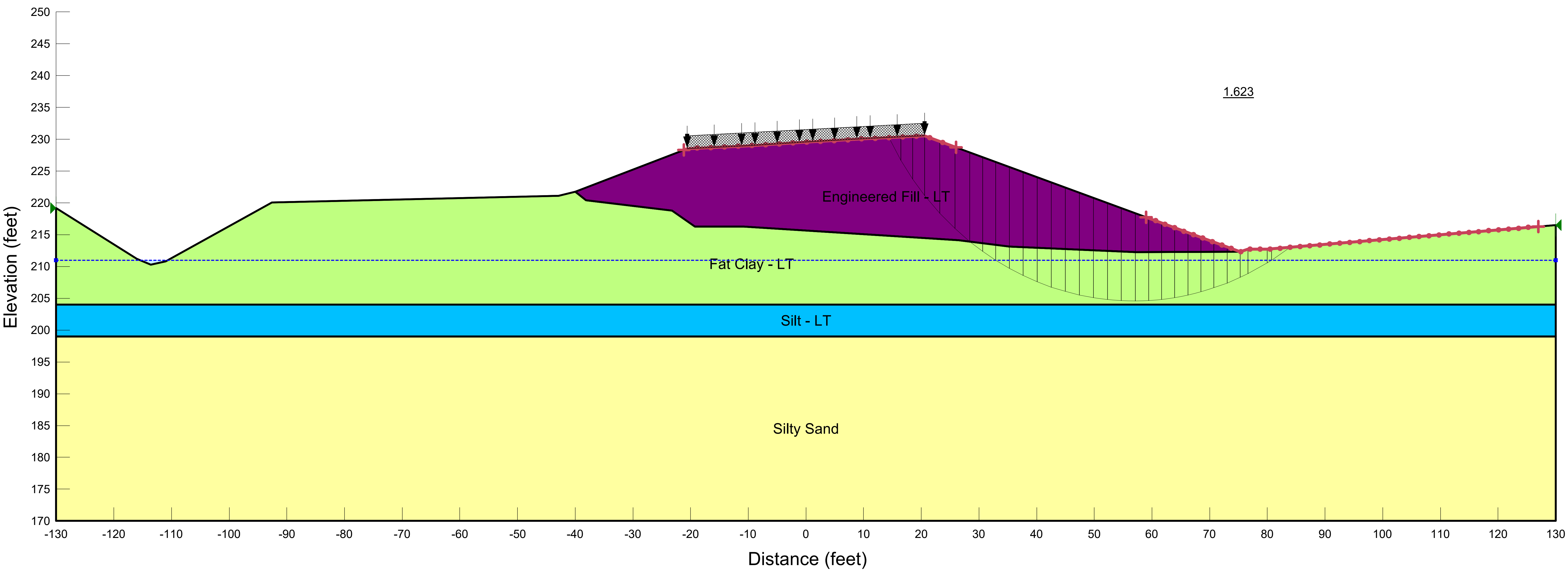

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Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
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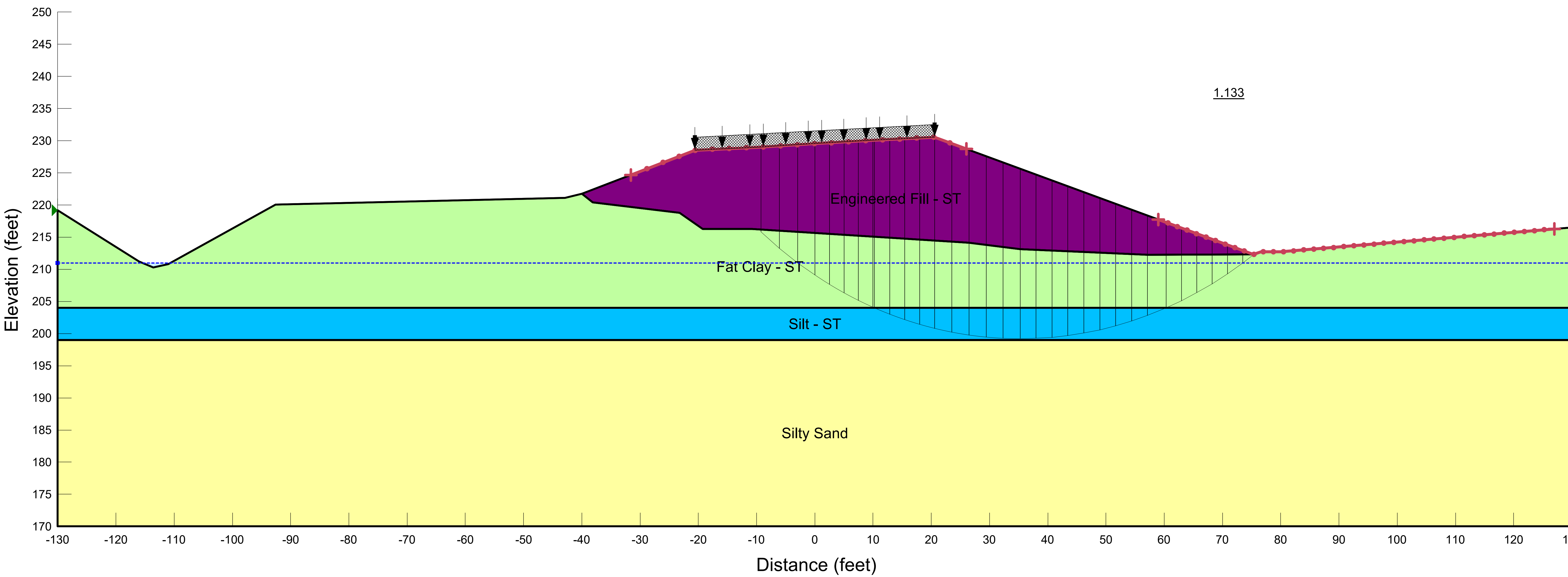

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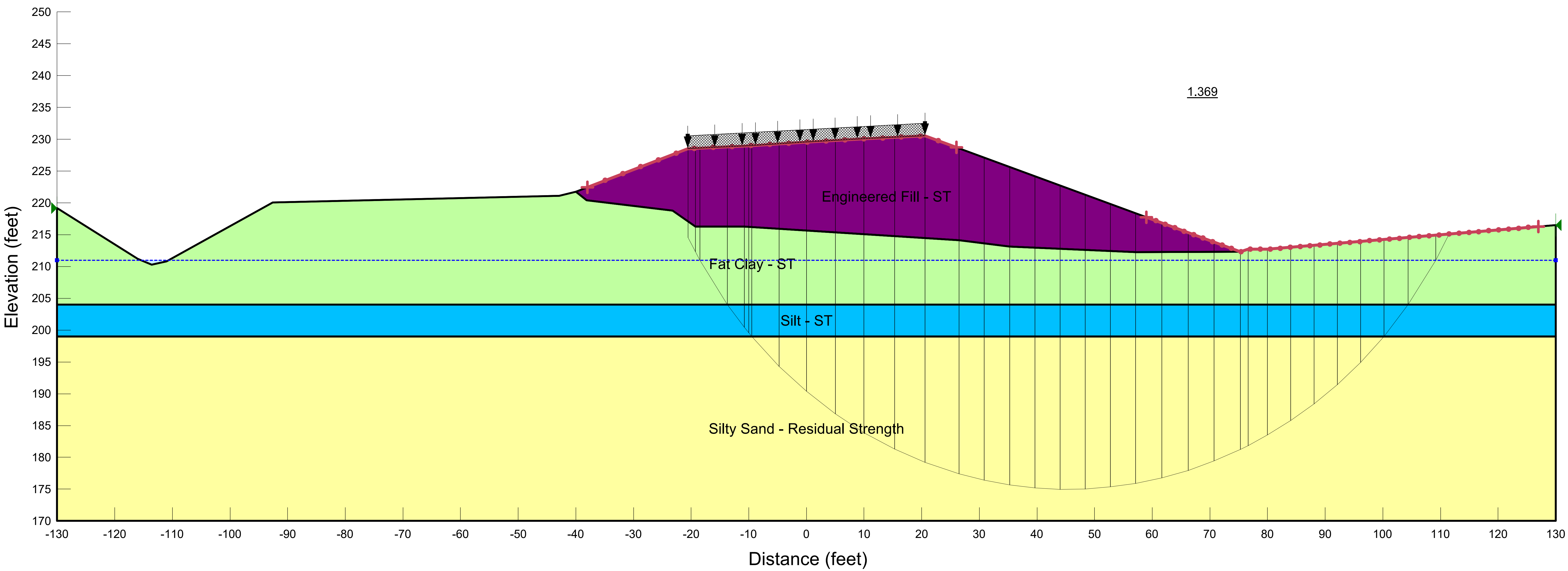
ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: Southern Side Slope Sta 216+00 Residual Strength, Ordinary High Water
Description: Post-Seismic Condition, GWT El 211



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Name: Silt - ST Unit Weight: 120 pcf Cohesion': 1,000 psf Phi': 0 °
Name: Silty Sand - Residual Strength Unit Weight: 123 pcf Cohesion': 350 psf Phi': 0 °

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: East Abutment -ST- Ordinary High Water
Description: Short-Term Conditions / Ordinary High Water

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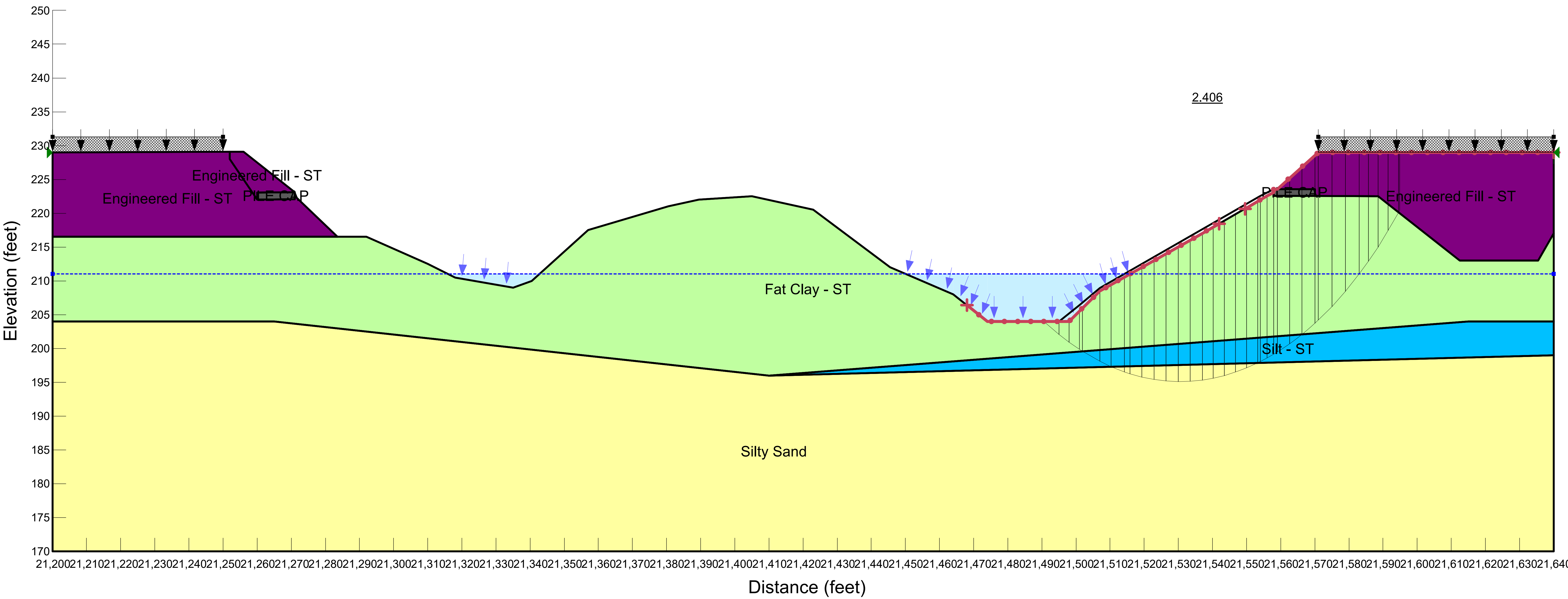
FROM THE GROUND UP



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Name: Fat Clay - ST   Unit Weight: 116 pcf   Cohesion': 900 psf   Phi': 0 °
Name: Silt - ST   Unit Weight: 120 pcf   Cohesion': 1,000 psf   Phi': 0 °

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
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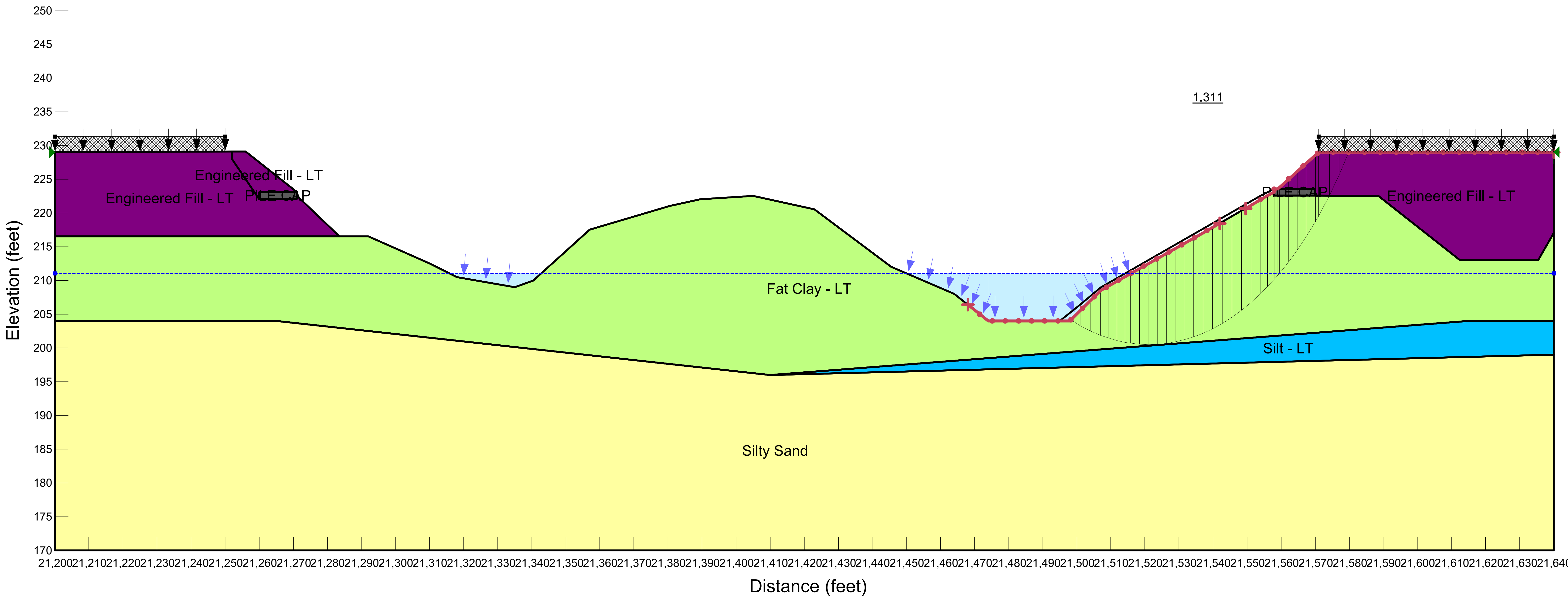
Description: Long-Term Conditions / Ordinary High Water Level



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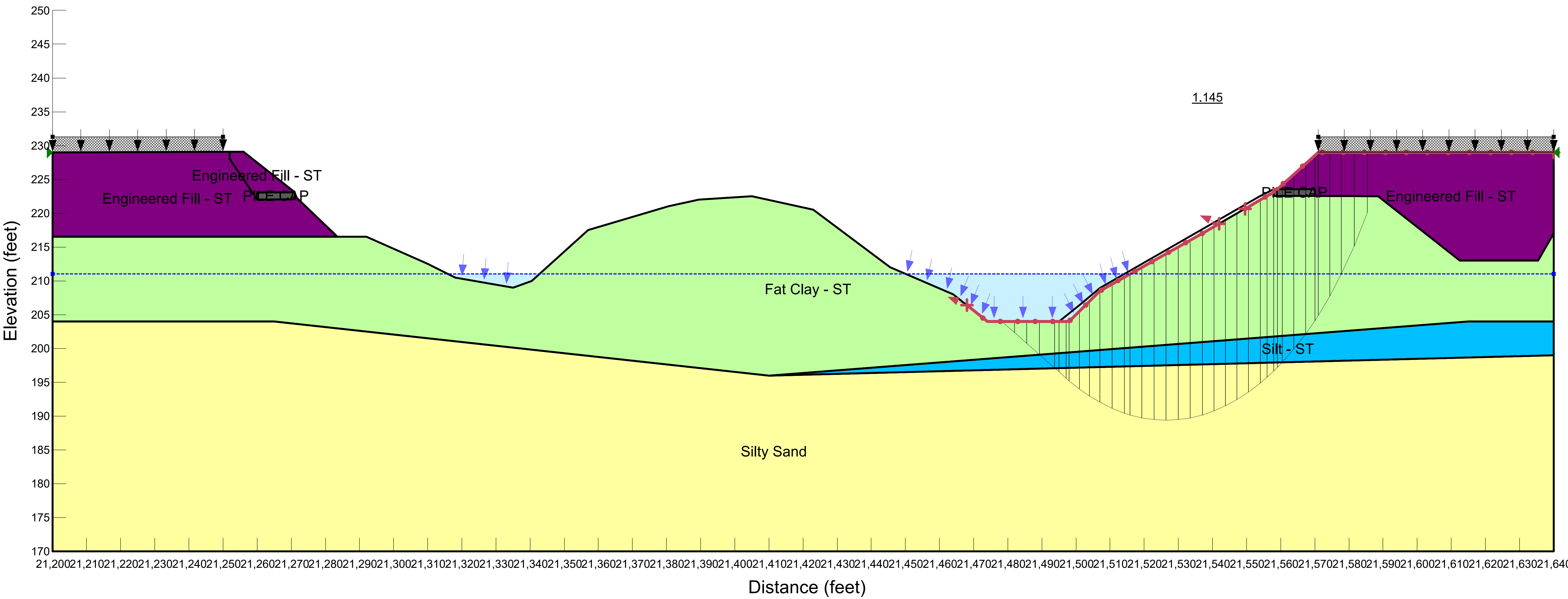
Name: Silty Sand Unit Weight: 123 pcf Cohesion': 0 psf Phi': 34 °
Name: PILE CAP Unit Weight: 150 pcf

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: East Abutment -EQ - Ordinary High Water
Description: Seismic Conditions / Ordinary High Water
Seismic Coef: 0.32



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Name: Engineered Fill - ST Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 °
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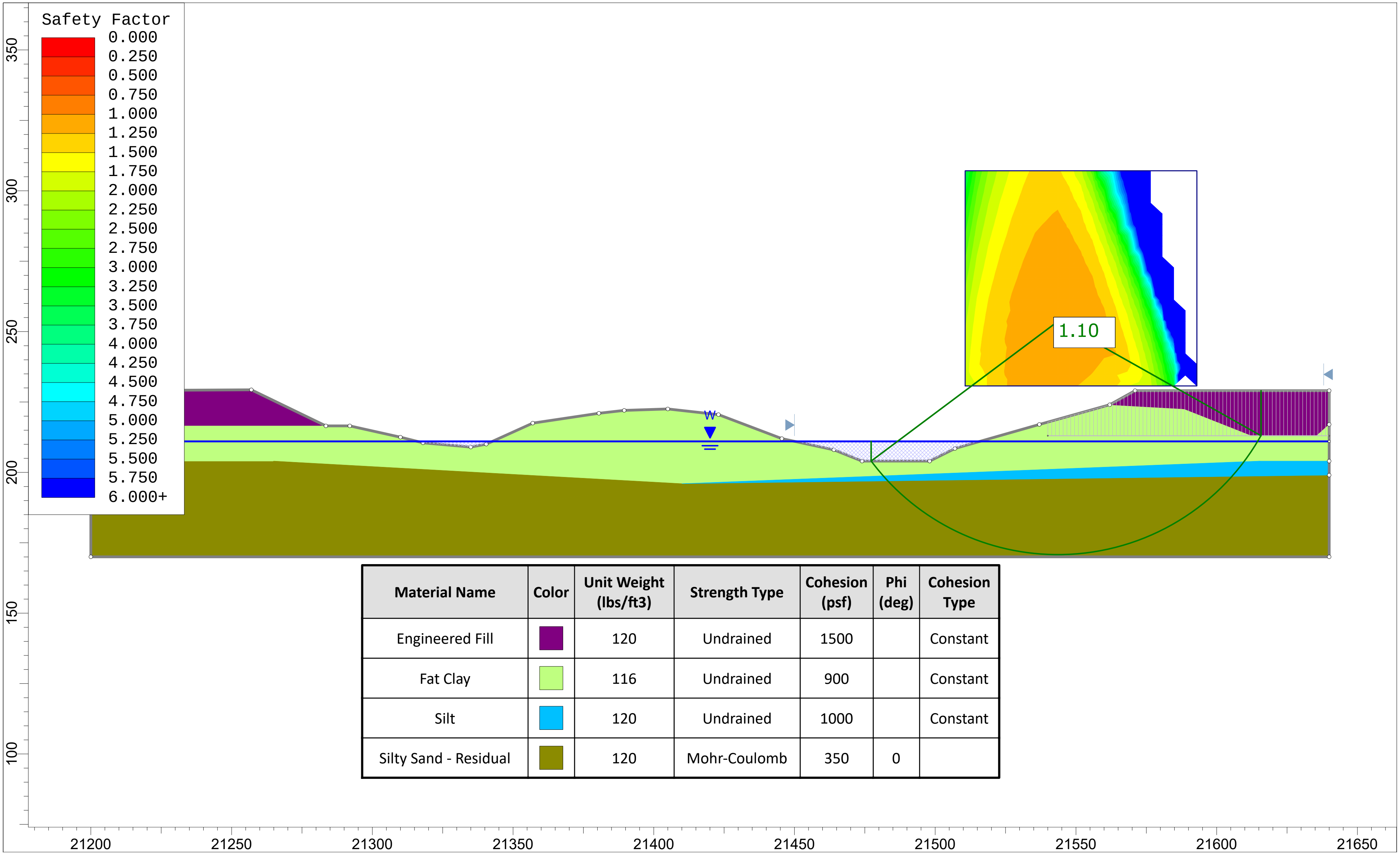

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type
Engineered Fill		120	Undrained	1500		Constant
Fat Clay		116	Undrained	900		Constant
Silt		120	Undrained	1000		Constant
Silty Sand - Residual		120	Mohr-Coulomb	350	0	

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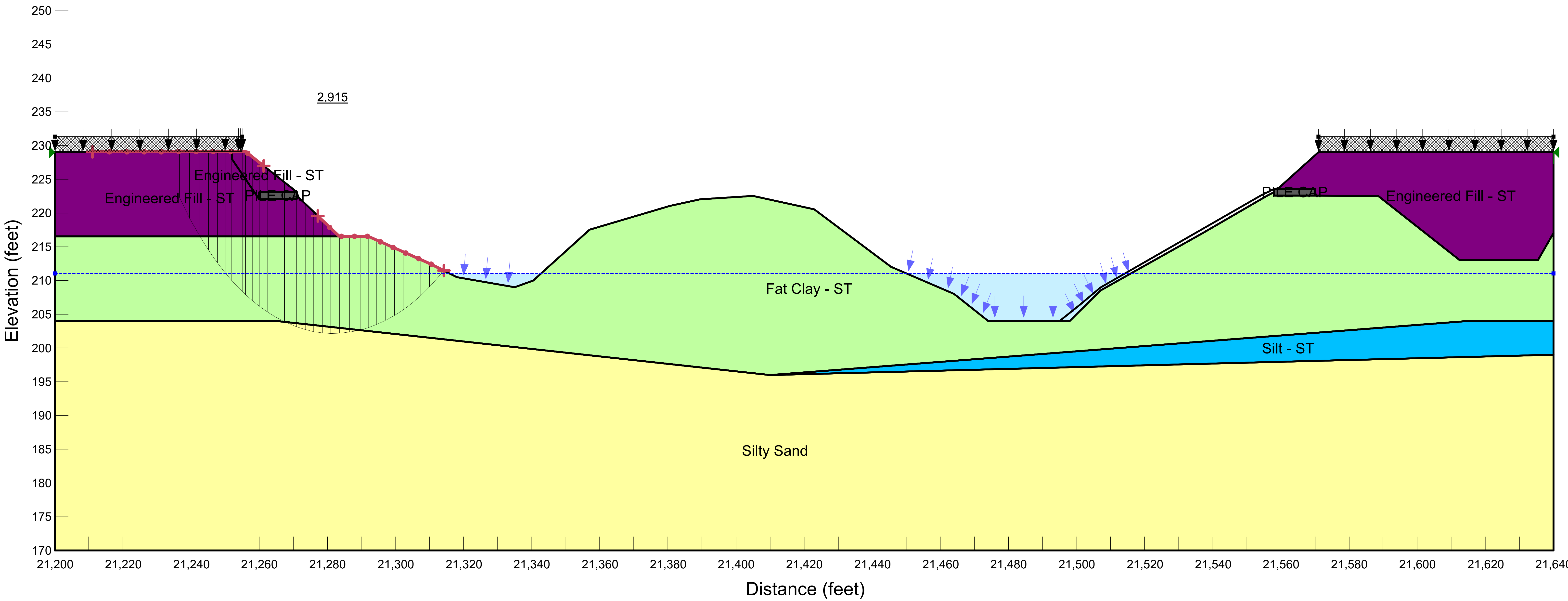
File Name: West Abutment EQ.slmd
Name: East Abutment
Description: Residual shear strength of sand / Ordinary high water
Method: Spencer

Project Number:
Client: Geotechnology
Project: Ditch Nos. 1 & 47 Strs. & Apprs.
Date: 11/1/2019

SLIDEINTERPRET 8.028

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: West Abutment - ST - Ordinary High Water
Description: Short-Term Conditions / Ordinary High Water



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Name: PILE CAP Unit Weight: 150 pcf
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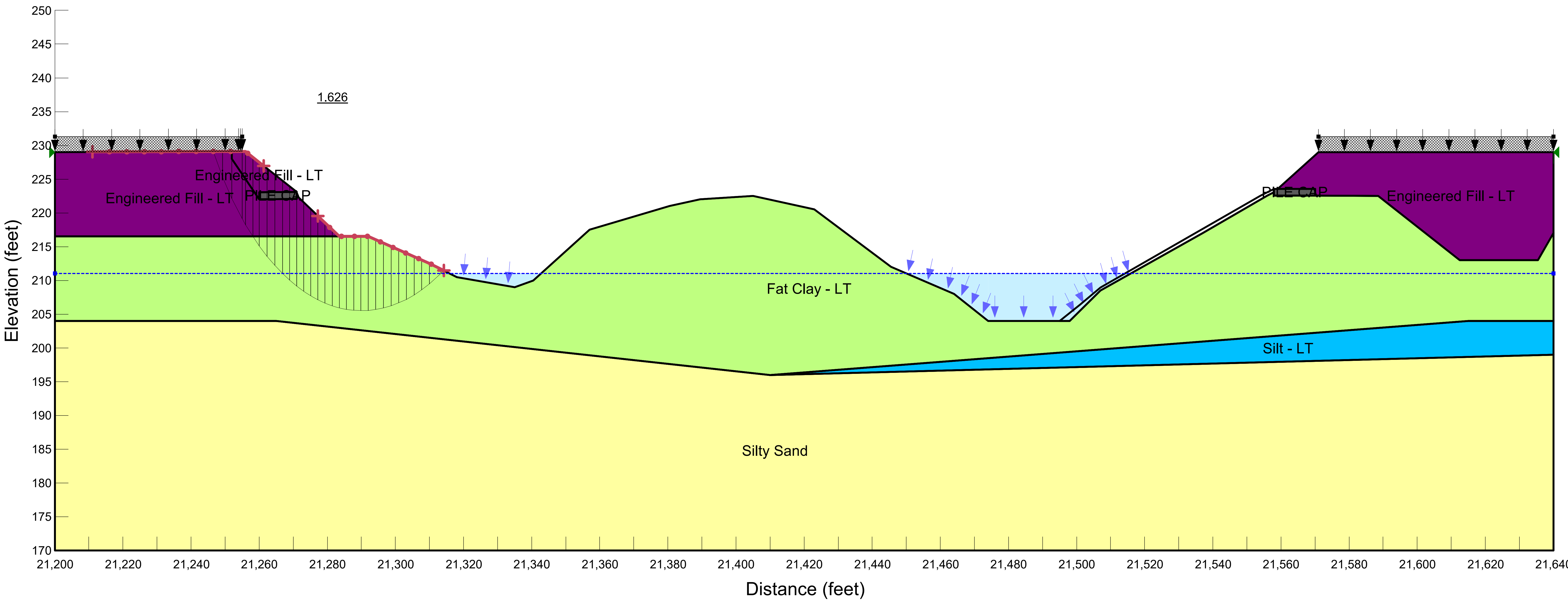
Name: Fat Clay - ST Unit Weight: 116 pcf Cohesion': 900 psf Phi': 0 °
Name: Silt - ST Unit Weight: 120 pcf Cohesion': 1,000 psf Phi': 0 °

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: West Abutment - LT - Ordinary High Water
Description: Long-Term Conditions / Ordinary High Water



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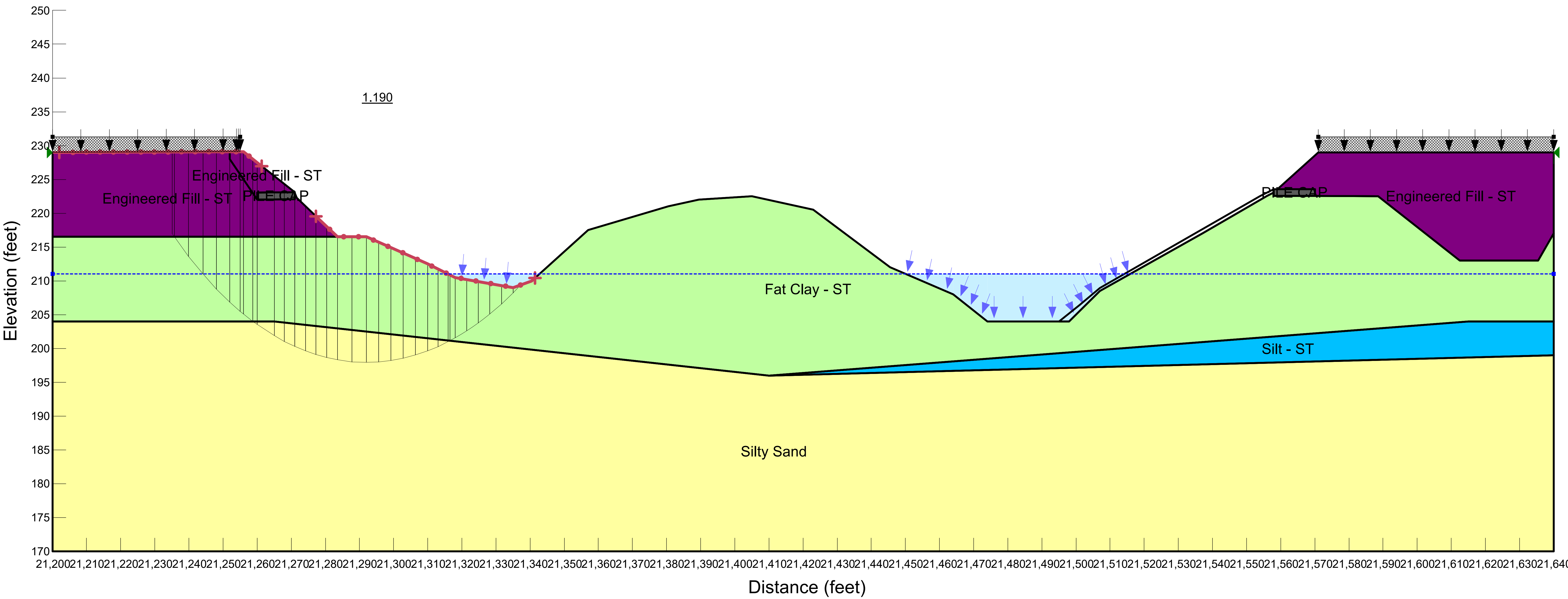
Name: Silty Sand Unit Weight: 123 pcf Cohesion': 0 psf Phi': 34 °
Name: PILE CAP Unit Weight: 150 pcf

ARDOT 100840
Ditch Nos. 1 & 47 Strs. & Apprs. (S)
Poinsett County, Arkansas
Geotechnology Project No. J034298.01
Global Stability Analysis
Name: West Abutment - EQ - Ordinary High Water
Description: Seismic Conditions / Ordinary High Water
Seismic Coef: 0.32



Name: Silty Sand Unit Weight: 123 pcf Cohesion': 0 psf Phi': 34 °
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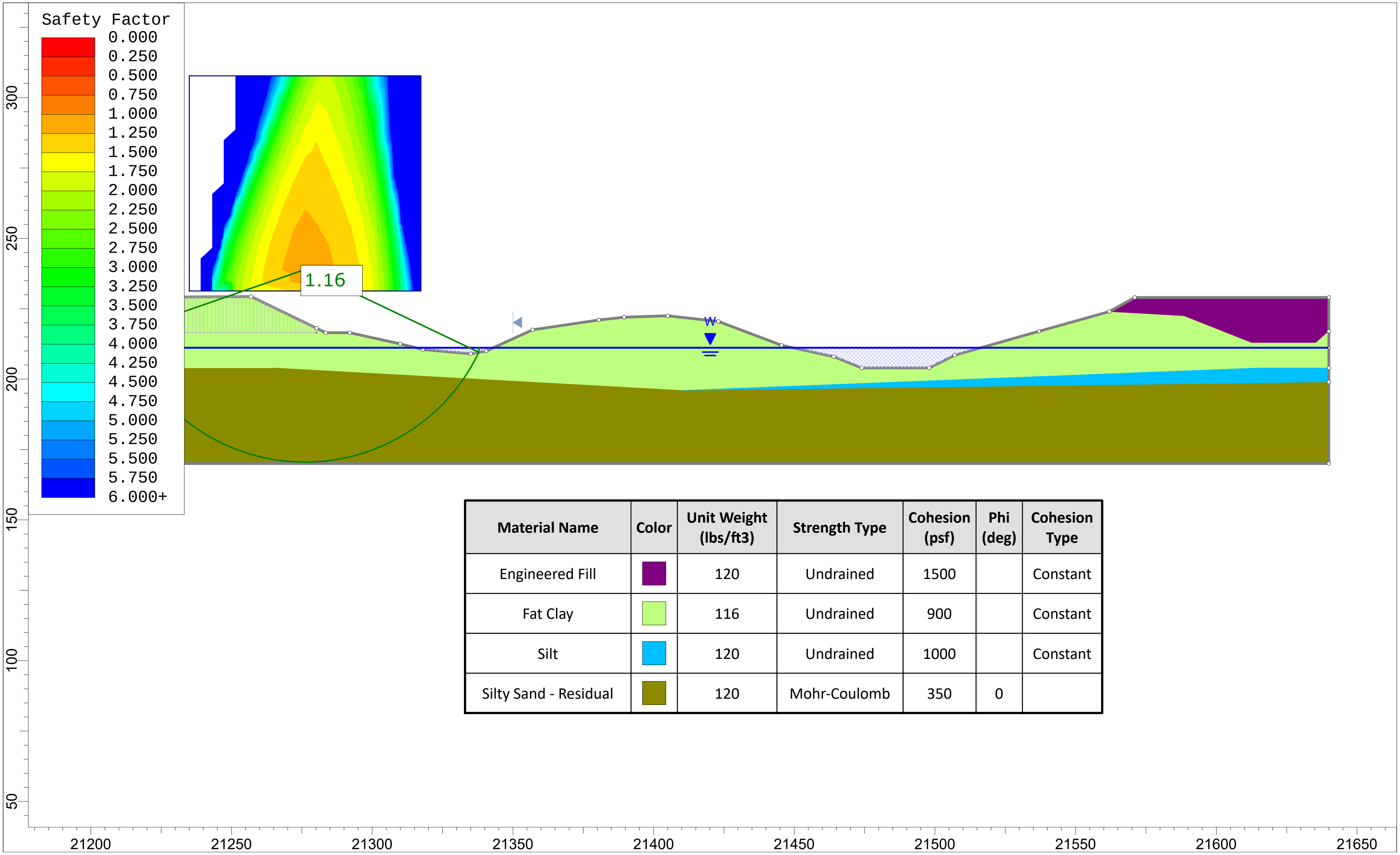
Name: Engineered Fill - ST Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 °
Name: Fat Clay - ST Unit Weight: 116 pcf Cohesion': 900 psf Phi': 0 °
Name: Silt - ST Unit Weight: 120 pcf Cohesion': 1,000 psf Phi': 0 °



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type
Engineered Fill		120	Undrained	1500		Constant
Fat Clay		116	Undrained	900		Constant
Silt		120	Undrained	1000		Constant
Silty Sand - Residual		120	Mohr-Coulomb	350	0	

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: West Abutment EQ.slmd
Name: West Abutment
Description: Residual shear strength of sand / Ordinary high water
Method: Spencer

Project Number:
Client: Geotechnology
Project: Ditch Nos. 1 & 47 Strs. & Apprs.
Date: 11/1/2019

SLIDEINTERPRET 8.028

APPENDIX H - SOIL PARAMETERS FOR SYNTHETIC PROFILES

WEST ABUTMENT - BORING B-1

ZONE	SOIL TYPES	DEPTH[a] (ELEVATION)		TOTAL WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]		LIQUEFACTION SHEAR STRENGTH PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)					
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)	SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	RESIDUAL COHESION (PSF)	RESIDUAL Φ (DEGREE)
1	Engineered Fill (Cohesive)	227[b]	216	120	1,500	--	50	28	0.007	500	1,200	--
2	Fat Clay	216	204	121	900	--	--	20	0.01	100	720	--
3	Silty Sand	204	172	125	--	34	--	34	--	60	--	7

a. Elevations are approximate and determined from the provided drawing
b. Assumed final grade at West Abutment
c. Pounds per cubic inch
d. For Lateral Load Analysis Only

BENTS – BORINGS B-1 THROUGH -3													
ZONE	SOIL TYPES	DEPTH[a] (ELEVATION)		TOTAL WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[c]		LIQUEFACTION SHEAR STRENGTH PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)					
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)	SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[b]	RESIDUAL COHESION (PSF)	RESIDUAL Φ (DEGREE)
1	Fat Clay	220	196	119	1,000	--	--	20	0.01	100	800	--
2	Silty Sand	196	150	125	--	34	--	34	--	60	--	7
3	Silty Sand	150	120	128	--	38	--	38	--	90	--	7

a. Elevations are approximate and determined from the provided drawing
b. Pounds per cubic inch
c. For Lateral Load Analysis Only

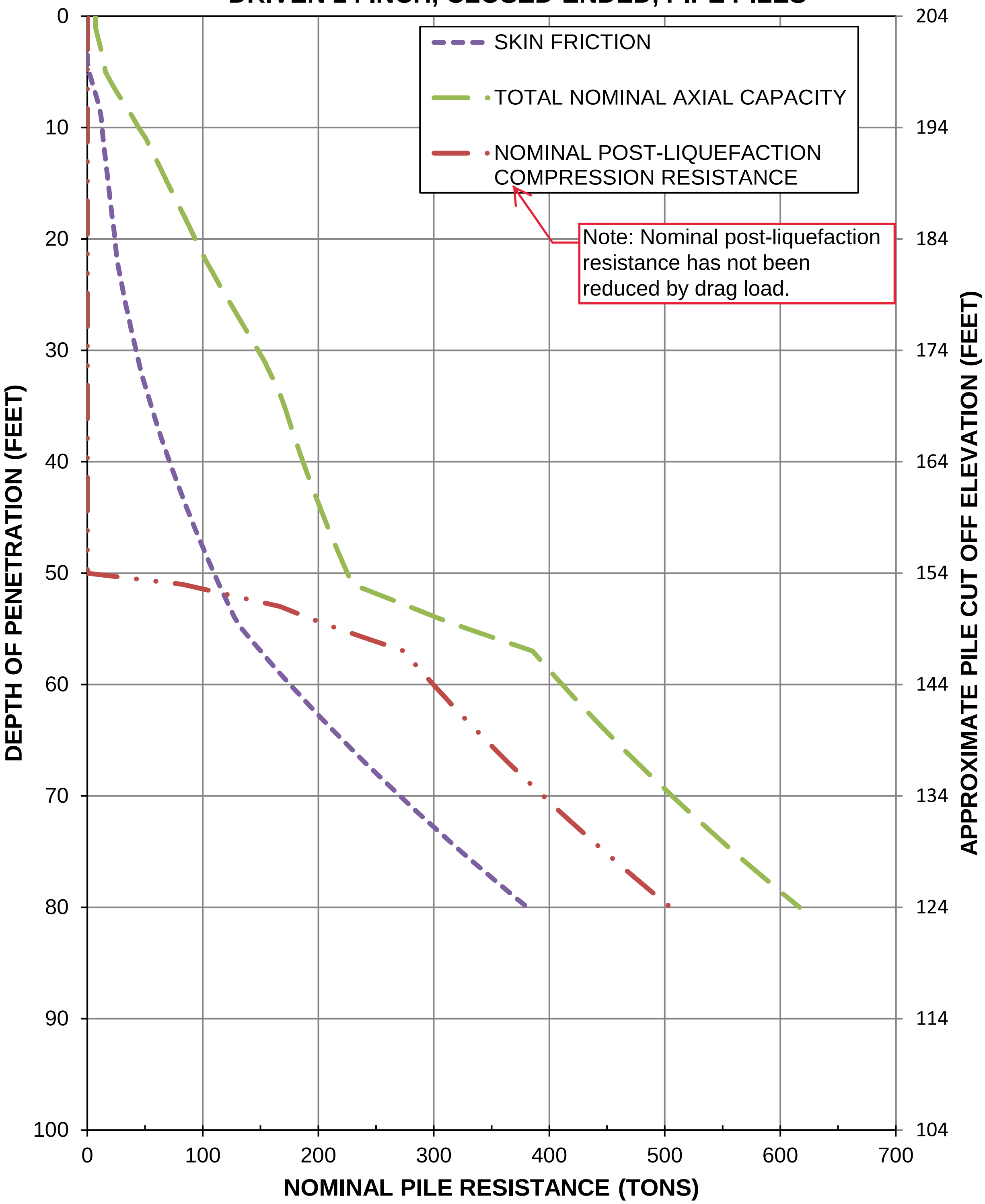
EAST ABUTMENT - BORING B-3													
ZONE	SOIL TYPES	DEPTH[a] (ELEVATION)		TOTAL WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]		LIQUEFACTION SHEAR STRENGTH PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)					
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)	SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[C]	RESIDUAL COHESION (PSF)	RESIDUAL Φ (DEGREE)
1	Engineered Fill (Cohesive)	227[b]	224	120	1,500	--	50	28	0.007	500	1,200	--
2	Fat Clay	224	204	116	900	--	--	20	--	100	720	--
3	Silt	204	199	120	1,000	--	--	30	0.01	125	800	--
4	Silty Sand	199	172	123	--	34	--	34	--	60	--	7

a. Elevations are approximate and determined from the provided drawing
b. Assumed final grade at East Abutment
c. Pounds per cubic inch
d. For Lateral Load Analysis Only

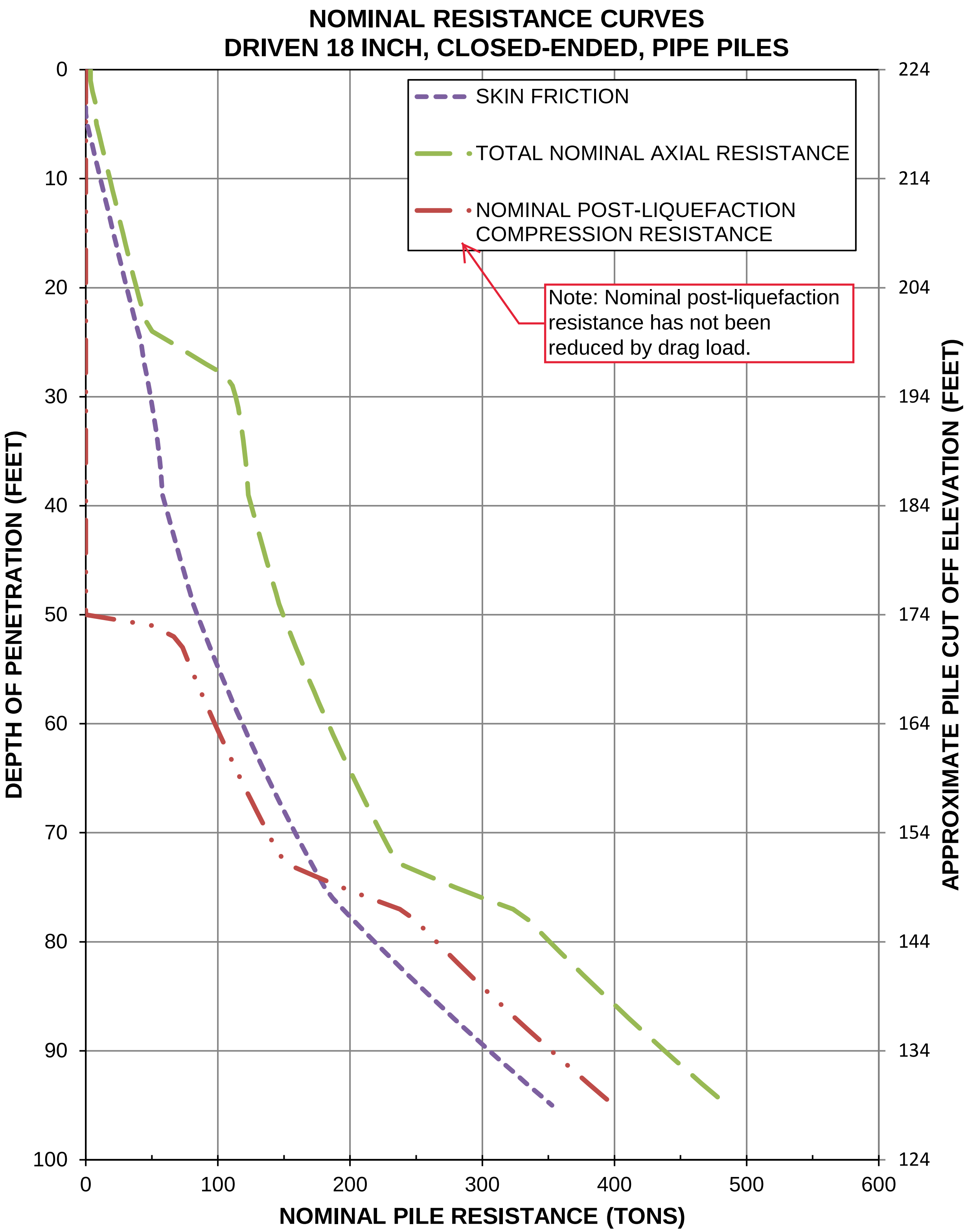
APPENDIX I - NOMINAL RESISTANCE CURVES

BENTS
HWY 308 OVER DITCH NOS. 1 &47

NOMINAL RESISTANCE CURVES
DRIVEN 24 INCH, CLOSED-ENDED, PIPE PILES



EAST ABUTMENT
HWY 308 OVER DITCH NOS. 1 &47



WEST ABUTMENT
HWY 308 OVER DITCH NOS. 1 &47

NOMINAL RESISTANCE CURVES
DRIVEN 18 INCH, CLOSED-ENDED, PIPE PILES