ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 020590

FEDERAL AID PROJECT NO. NHPP-0021(40)

UPRR STR. & APPRS. (MCGEHEE) (S)

STATE HIGHWAY 278 **SECTION** 16

IN DESHA **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 DEPARTMENT OF TRANSPORTATION

July 6, 2017

TO: Mr. Trinity Smith, Engineer of Roadway Design

SUBJECT: Job No. 020590
UPRR Str. & Apprs. (S)
Route 278 Section 16
Desha 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 Union Pacific Railroad overpass on Highway 278 on new location. Samples were obtained in the existing travel lanes, shoulders and ditch line. Locations are measured from centerline of the existing roadway.

Based on laboratory results of samples obtained, the subgrade soils consist primarily of non-plastic sands and moderately plastic sandy clay. The subgrade soils are expected to provide a stable working platform with conventional processing if the weather is favorable during construction.

Based on currently available cross sections the maximum embankment height is approximately 29 feet. Approximately 60 feet right of construction centerline are large drainage ditches that run parallel to the existing road. At the time of investigation these ditches held standing water. Prior to embankment construction the ditches must be drained and all soft unstable organic material should be undercut to a maximum depth of 3 feet. The embankment may be constructed with locally available unspecified material utilizing the 3:1 slope configuration shown in the cross sections.

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 port near Pendleton.

2. Asphalt Concrete Hot Mix

Type	Asphalt Cement %	Mineral Aggregate %
Surface Course	5.2	94.8
Binder Course	4.2	95.8
Base Course	3.5	96.5

Michael C. Benson
Materials Engineer

MCB:pt:bjj
Attachment
cc: State Constr. Eng. – Master File Copy
District 2 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       - 06/29/2017                              SEQUENCE NO.   - 1
JOB NUMBER - 020590                                  MATERIAL CODE - SSRV
                                                     SPEC. YEAR     - 2014
                                                     SUPPLIER ID.   - 1
                                                     COUNTY/STATE   - 21
                                                     DISTRICT NO.   - 02
JOB NAME - UPRR STR. & APPRS. (S)
*****************************************************************************
*                  STATION LIMITS          R-VALUE AT 240 psi               *
*****************************************************************************

                 BEGIN JOB - END JOB          LESS THAN 5

                 RESILIENT MODULUS
                 STA. 109+90                  8236

-----------------------------------------------------------------------------
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.	020590	**Material Code**	SSRVPS
Date Sampled:	6/14/17	**Station No.:**	109+90
Date Tested:	June 22, 2017	**Location:**	33'RT
Name of Project:	UPRR STR. & APPRS. (S)		
County:	**Code:** 21 **Name:** DESHA		
Sampled By:	THORNTON/CAMPBELL	**Depth:**	0-5
Lab No.:	20171953	**AASHTO Class:**	A-6(13)
Sample ID:	RV428	**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	3.95
Middle	3.94
Bottom	3.95
Average	3.95
Membrane Thickness (in):	0.01
Height of Specimen, Cap and Base (in):	8.03
Height of Cap and Base (in):	0.00
Initial Length, Lo (in):	8.03
Initial Area, Ao (sq. in):	12.16
Initial Volume, AoLo (cu. in):	97.64

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	2998.40

4. Soil Properties:

Optimum Moisture Content (%):	16.7
Maximum Dry Density (pcf):	105.8
95% of MDD (pcf):	100.5
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	2998.40
Compaction Moisture content (%):	17.0
Compaction Wet Density (pcf):	117.01
Compaction Dry Density (pcf):	100.01
Moisture Content After Mr Test (%):	17.0

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

7. Resilient Modulus, Mr: 11725(Sc)^-0.19488(S3)^0.11583

8. Comments ______________________

9. Tested By: GW **Date:** June 22, 2017

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT
MATERIALS DIVISION

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

Job No. 020590
Date Sampled: 6/14/17
Date Tested: June 22, 2017
Name of Project: UPRR STR. & APPRS. (S)
County: **Code:** 21 **Name:** DESHA
Sampled By: THORNTON/CAMPBELL
Lab No.: 20171953
Sample ID: RV428
LATITUDE:

Material Code SSRVPS
Station No.: 109+90
Location: 33'RT
Depth: 0-5
AASHTO Class: A-6(13)
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	25.1	22.2	2.8	2.1	1.8	0.2	0.00115	0.00014	12,733
Sequence 2	6.0	4.0	47.0	44.2	2.8	3.9	3.6	0.2	0.00245	0.00030	11,936
Sequence 3	6.0	6.0	69.3	65.7	3.6	5.7	5.4	0.3	0.00394	0.00049	11,016
Sequence 4	6.0	8.0	92.1	86.0	6.1	7.6	7.1	0.5	0.00585	0.00073	9,720
Sequence 5	6.0	10.0	113.7	105.2	8.5	9.4	8.7	0.7	0.00799	0.00099	8,699
Sequence 6	4.0	2.0	25.0	22.3	2.8	2.1	1.8	0.2	0.00126	0.00016	11,717
Sequence 7	4.0	4.0	46.9	44.0	2.8	3.9	3.6	0.2	0.00264	0.00033	11,008
Sequence 8	4.0	6.0	68.3	65.5	2.8	5.6	5.4	0.2	0.00423	0.00053	10,232
Sequence 9	4.0	8.0	91.0	85.8	5.2	7.5	7.1	0.4	0.00600	0.00075	9,437
Sequence 10	4.0	10.0	113.3	105.7	7.6	9.3	8.7	0.6	0.00805	0.00100	8,672
Sequence 11	2.0	2.0	25.0	22.1	2.8	2.1	1.8	0.2	0.00136	0.00017	10,781
Sequence 12	2.0	4.0	46.7	43.8	2.8	3.8	3.6	0.2	0.00288	0.00036	10,061
Sequence 13	2.0	6.0	67.9	65.1	2.8	5.6	5.4	0.2	0.00454	0.00057	9,473
Sequence 14	2.0	8.0	89.8	85.5	4.3	7.4	7.0	0.4	0.00638	0.00079	8,850
Sequence 15	2.0	10.0	111.9	105.2	6.7	9.2	8.7	0.6	0.00844	0.00105	8,236

TESTED BY GW DATE June 22, 2017
REVIEWED BY DATE

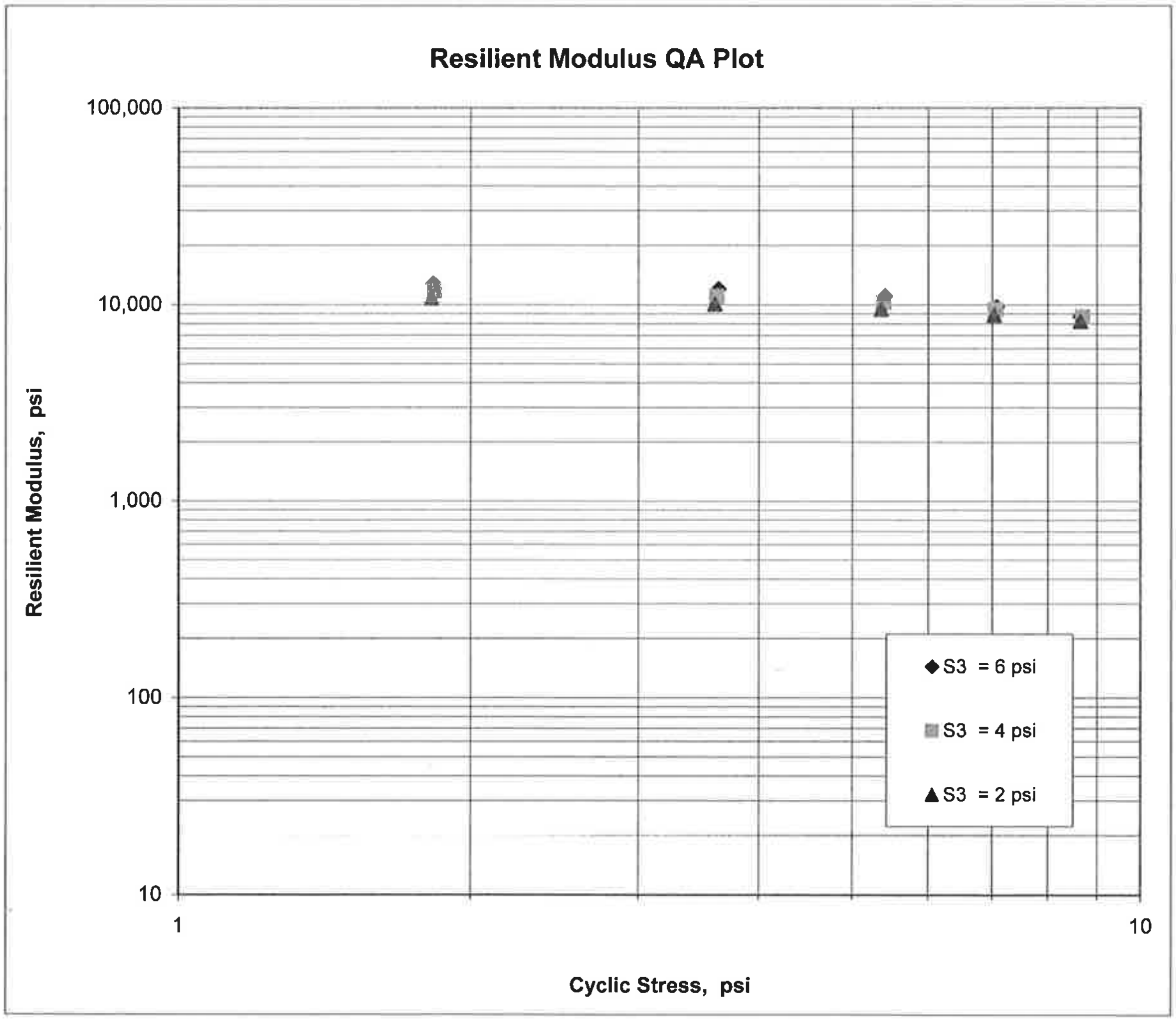
ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT
MATERIALS DIVISION

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

Job No. 020590
Material Code SSRVPS
Date Sampled: 6/14/17
Station No.: 109+90
Date Tested: June 22, 2017
Location: 33'RT
Name of Project: UPRR STR. & APPRS. (S)
County: **Code:** 21 **Name:** DESHA
Sampled By: THORNTON/CAMPBELL
Depth: 0-5
Lab No.: 20171953
AASHTO Class: A-6(13)
Sample ID: RV428
Material Type (1 or 2): 2
LATITUDE:
LONGITUDE:

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

K1 = 11,725
K2 = -0.19488
K5 = 0.11583
R^2 = 0.90



JOB: **020590**

JOB NAME: **UPRR STR. & APPRS. (S)**

COUNTY NO. **21** *DATE TESTED* **6/21/2017**

Arkansas State Highway Transporation Department

Materials Division

Michael Benson, Materials Engineer

STA.#	*LOC.*	*DEPTH*	*COLOR*	*#4*	*#10*	*#40*	*#80*	*#200*	*L.L.*	*P.I.*	*SOIL CLASS*	*LAB #:*	*%MOISTURE*
				SIEVES									
109+90	33 RT	0-5	BR/GR	99	98	95	82	78	35	24	A-6(16)	RV428	
102+00	06 RT	0-5	BROWN	100				92	ND	NP	A-4(0)	S421	27.7
102+00	15 RT	0-5	BROWN	98	92	89	86	81	ND	NP	A-4(0)	S422	21.2
102+00	24 RT	0-5	BROWN	98	95	88	82	77	ND	NP	A-4(0)	S423	19.1
110+00	33 RT	0-5	BROWN	99	99	98	91	90	43	33	A-7-6(29)	S424	30.8
119+00	30 RT	0-5	BROWN	100				98	35	18	A-6(18)	S425	47.5
127+00	06 LT	0-5	BR/GR	81	71	59	47	42	ND	NP	A-4(0)	S426	16.4
127+00	15 LT	0-5	BR/GR	92	86	78	74	68	ND	NP	A-4(0)	S427	14.5

comments: W=MULTIPLE LAYERS,X=STRIPPED

Thursday, July 06, 2017

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT - LITTLE ROCK, ARKANSAS
MATERIALS DIVISION
MICHAEL BENSON, MATERIALS ENGINEER

*** SOIL SURVEY / PAVEMENT SOUNDING TEST REPORT ***

DATE - 07/05/17
JOB NUMBER - 020590
FEDERAL AID NO.- TO BE ASSIGNED
PURPOSE - SOIL SURVEY SAMPLE
SPEC. REMARKS - NO SPECIFICATION CHECK
SUPPLIER NAME - STATE
NAME OF PROJECT - UPRR STR. & APPRS. (S)
PROJECT ENGINEER - NOT APPLICABLE
PIT/QUARRY - ARKANSAS
LOCATION - DESHA, COUNTY
SAMPLED BY - THORNTON/CAMPBELL
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 - 21
DISTRICT NO. - 02

DATE SAMPLED - 06/14/17
DATE RECEIVED - 06/14/17
DATE TESTED - 06/21/17

LAB NUMBER	20171945	20171946	20171947
SAMPLE ID	S421	S422	S423
TEST STATUS	INFORMATION ONLY	INFORMATION ONLY	INFORMATION ONLY
STATION	102+00	102+00	102+00
LOCATION	06 RT	15 RT	24 RT
DEPTH IN FEET	0-5	0-5	0-5
MAT'L COLOR	BROWN	BROWN	BROWN
MAT'L TYPE			
LATITUDE DEG-MIN-SEC	33 37 5.00	33 37 4.90	33 37 4.90
LONGITUDE DEG-MIN-SEC	91 23 28.10	91 23 28.20	91 23 28.30
% PASSING 2 IN.			
1 1/2 IN.			
3/4 IN.		100	100
3/8 IN.		98	99
NO. 4	100	98	98
NO. 10		92	95
NO. 40		89	88
NO. 80		86	82
NO. 200	92	81	77
LIQUID LIMIT	ND	ND	ND
PLASTICITY INDEX	NP	NP	NP
AASHTO SOIL	A-4(0)	A-4(0)	A-4(0)
UNIFIED SOIL			
% MOISTURE CONTENT	27.7	21.2	19.1
ACHMSC (IN)	7.75W	6.0W	---
PCCP (IN)	7.0	---	---
AGG BASE CRS. CL-5 (IN)	---	5.0	---

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 - 07/05/17
JOB NUMBER - 020590
FEDERAL AID NO.- TO BE ASSIGNED
PURPOSE - SOIL SURVEY SAMPLE
SPEC. REMARKS - NO SPECIFICATION CHECK
SUPPLIER NAME - STATE
NAME OF PROJECT - UPRR STR. & APPRS. (S)
PROJECT ENGINEER - NOT APPLICABLE
PIT/QUARRY - ARKANSAS
LOCATION - DESHA, COUNTY
SAMPLED BY - THORNTON/CAMPBELL
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 - 21
DISTRICT NO. - 02

DATE SAMPLED - 06/14/17
DATE RECEIVED - 06/14/17
DATE TESTED - 06/21/17

LAB NUMBER	20171948	20171949	20171950
SAMPLE ID	S424	S425	S426
TEST STATUS	INFORMATION ONLY	INFORMATION ONLY	INFORMATION ONLY
STATION	110+00	119+00	127+00
LOCATION	33 RT	30 RT	06 LT
DEPTH IN FEET	0-5	0-5	0-5
MAT'L COLOR	BROWN	BROWN	BR/GR
MAT'L TYPE			
LATITUDE DEG-MIN-SEC	33 36 57.10	33 36 49.30	33 36 45.20
LONGITUDE DEG-MIN-SEC	91 23 26.30	91 23 21.20	91 23 13.30
% PASSING 2 IN.			
1 1/2 IN.			
3/4 IN.			100
3/8 IN.	100		91
NO. 4	99	100	81
NO. 10	99		71
NO. 40	98		59
NO. 80	91		47
NO. 200	90	98	42
LIQUID LIMIT	43	35	ND
PLASTICITY INDEX	33	18	NP
AASHTO SOIL	A-7-6(29)	A-6(18)	A-4(0)
UNIFIED SOIL			
% MOISTURE CONTENT	30.8	47.5	16.4
ACHMSC (IN)	---	---	2.0
ACHMSC (IN)	---	---	5.0X
ASPHALT TREATED BASE (IN)	---	---	26
AGG BASE CRS CL-5 (IN)	---	---	26

REMARKS - X=STRIPPED,W=MULTIPLE LAYERS

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 - 07/05/17
JOB NUMBER - 020590
FEDERAL AID NO.- TO BE ASSIGNED
PURPOSE - SOIL SURVEY SAMPLE
SPEC. REMARKS - NO SPECIFICATION CHECK
SUPPLIER NAME - STATE
NAME OF PROJECT - UPRR STR. & APPRS. (S)
PROJECT ENGINEER - NOT APPLICABLE
PIT/QUARRY - ARKANSAS
LOCATION - DESHA, COUNTY
SAMPLED BY - THORNTON/CAMPBELL
SAMPLE FROM - TEST HOLE
MATERIAL DESC. - SOIL SURVEY - R VALUE- PAVEMENT SOUNDINGS

SEQUENCE NO. - 3
MATERIAL CODE - SSRVPS
SPEC. YEAR - 2014
SUPPLIER ID. - 1
COUNTY/STATE - 21
DISTRICT NO. - 02

DATE SAMPLED - 06/14/17
DATE RECEIVED - 06/14/17
DATE TESTED - 06/21/17

LAB NUMBER	20171951		
SAMPLE ID	S427		
TEST STATUS	INFORMATION ONLY		
STATION	127+00		
LOCATION	15 LT		
DEPTH IN FEET	0-5		
MAT'L COLOR	BR/GR		
MAT'L TYPE			
LATITUDE DEG-MIN-SEC	33 36 45.20		
LONGITUDE DEG-MIN-SEC	91 23 13.20		
% PASSING 2 IN.			
1 1/2 IN.			
3/4 IN.	100		
3/8 IN.	97		
NO. 4	92		
NO. 10	86		
NO. 40	78		
NO. 80	74		
NO. 200	68		
LIQUID LIMIT	ND		
PLASTICITY INDEX	NP		
AASHTO SOIL	A-4(0)		
UNIFIED SOIL			
% MOISTURE CONTENT	14.5		
ACHMSC (IN)	4.5W		
AGG BASE CRS CL-5 (IN)	8.0		

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 - 07/05/17
JOB NUMBER - 020590
FEDERAL AID NO.- TO BE ASSIGNED
PURPOSE - SOIL SURVEY SAMPLE
SPEC. REMARKS - NO SPECIFICATION CHECK
SUPPLIER NAME - STATE
NAME OF PROJECT - UPRR STR. & APPRS. (S)
PROJECT ENGINEER - NOT APPLICABLE
PIT/QUARRY - ARKANSAS
LOCATION - DESHA, COUNTY
SAMPLED BY - THORNTON/CAMPBELL
SAMPLE FROM - TEST HOLE
MATERIAL DESC. - SOIL SURVEY - RESISTANCE R-VALUE ACTUAL RESULTS

SEQUENCE NO. - 1
MATERIAL CODE - RV
SPEC. YEAR - 2014
SUPPLIER ID. - 1
COUNTY/STATE - 21
DISTRICT NO. - 02

DATE SAMPLED - 06/14/17
DATE RECEIVED - 06/14/17
DATE TESTED - 06/21/17

LAB NUMBER - 20171953 - -
SAMPLE ID - RV428 - -
TEST STATUS - INFORMATION ONLY - -
STATION - 109+90 - -
LOCATION - 33 RT - -
DEPTH IN FEET - 0-5 - -
MAT'L COLOR - BR/GR - -
MAT'L TYPE - - -
LATITUDE DEG-MIN-SEC - 33 36 57.20 - -
LONGITUDE DEG-MIN-SEC - 91 23 26.30

% PASSING 2 IN. - - -
1 1/2 IN. - - -
3/4 IN. - - -
3/8 IN. - 100 - -
NO. 4 - 99 - -
NO. 10 - 98 - -
NO. 40 - 95 - -
NO. 80 - 82 - -
NO. 200 - 78

LIQUID LIMIT - 35 - -
PLASTICITY INDEX - 24 - -
AASHTO SOIL - A-6(16) - -
UNIFIED SOIL - - -
% MOISTURE CONTENT - - -

\- - -
\- - -
\- - -
\- - -
\- - -
\- - -
\- - -
\- - -
\- - -
\- - -

REMARKS - X=STRIPPED, W=MULTIPLE LAYERS
\-
\-
\-
\-

AASHTO TESTS : T24 T88 T89 T90 T265
:

JOB: 020590

JOB NAME: UPRR STR. & APPRS. (S)

COUNTY NO. 21

Arkansas State Highway Transporation Department

Materials Division

Michael Benson, Materials Engineer

DATE TESTED 6/21/2017

STA.#	LOC.	PAVEMENT SOUNDINGS			
102+00	06 RT	ACHMSC	PCCP	AGG BASE CRS. CL-5	
		7.75W	**7.0**	---	
102+00	15 RT	ACHMSC	PCCP	AGG BASE CRS. CL-5	
		6.0W	---	**5.0**	
102+00	24 RT	ACHMSC	PCCP	AGG BASE CRS. CL-5	
		---	---	---	
110+00	33 RT	ACHMSC	ACHMSC	ASPHALT TREATED BA	AGG BASE CRS CL-5
		---	---	---	---
119+00	30 RT	ACHMSC	ACHMSC	ASPHALT TREATED BA	AGG BASE CRS CL-5
		---	---	---	---
127+00	06 LT	ACHMSC	ACHMSC	ASPHALT TREATED BA	AGG BASE CRS CL-5
		2.0	**5.0X**	**26**	**26**
127+00	15 LT	ACHMSC	AGG BASE CRS CL-5		
		4.5W	**8.0**		

comments: W=MULTIPLE LAYERS,X=STRIPPED

GEOTECHNICAL REPORT
HIGHWAY 278 IMPROVEMENTS
Bridge Over Union Pacific Railroad
DESHA COUNTY, ARKANSAS

ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT NO. 020590

Prepared for:

GARVER, LLC
NORTH LITTLE ROCK

Prepared by:

GEOTECHNOLOGY, INC.
MEMPHIS, TENNESSEE

Date:

JUNE 5, 2020

Geotechnology Project No.:

J033659.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

GEOTECHNOLOGY INC
FROM THE GROUND UP

June 5, 2020

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

Re: Geotechnical Report
Highway 278 Improvements
Bridge Over Union Pacific Railroad
Desha County, Arkansas
Geotechnology Project No. J033659.01

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

ALY/DMS/ASE:aly/dms

Copies submitted: Client (email/2 mail)

No. 17917
Dale M. Smith

Dale M. Smith

6/5/20

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

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LIST OF TABLES

GEOTECHNICAL REPORT
HIGHWAY 278 IMPROVEMENTS
Bridge Over Union Pacific Railroad
DESHA COUNTY, ARKANSAS
June 5, 2020 | Geotechnology Project No. J033659.01

CHAPTER 1. SCOPE OF SERVICES

Presented in this report are the results of the subsurface exploration and recommendations for design, construction, and other related features for the proposed improvements to US Highway 278 (Hwy 278) in Desha County, Arkansas (Station 102+04.00 to Station 129+25.00). The referenced improvements consist of the construction of an approximately 922-foot-long, 9-span bridge (Station 110+14.87 to Station 119+37.13) over Union Pacific Railroad, south of the existing Hwy 278 bridge which will be demolished. It is our understanding the existing Hwy 278 bridge over Union Pacific Railroad will remain in use through construction of the replacement bridge. 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 subsurface exploration. Results of borings, in-situ testing, sampling and laboratory testing are included in the report. A total of 14 borings were drilled at intervals along the proposed US Highway 278 bridge over Union Pacific Railroad as shown in Figure 2 included in Appendix B. The boring logs, along with field and laboratory test results, are enclosed. 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 modifications to the approaches will require realignment of the existing roadway; beginning at Station 102+04.00, the existing road way will be widened to the south and the new alignment of Hwy 278 will merge off the existing at Station 104+30 and extend to the beginning of the bridge at Station 110+14.87. Based on the provided plans[1], it is our understanding the construction of the new western bridge approach will require up to 17 feet of fill and less than 3 feet of cut. The planned side slopes of the western approach are 3 horizontal units for every 1 vertical unit (3H:1V).

The proposed nine-span bridge will cross the Union Pacific Railroad, including future planned tracks, and a pedestrian tunnel. It is our understanding that minimal grade changes will be required at the bent locations. A 2H:1V slope is planned for the bridge abutments.

From the end of the bridge at Station 119+37.13 the new alignment of Hwy 278 will continue northeast until merging with existing Hwy 278 at Station 125+00. Widening of existing Hwy 278 will extend from Station 125+00 to the end of project Station 129+25.00. Approximately 25 feet of fill will be required in the eastern bridge approach and up to 9 feet of cut will be required near the base of the proposed southern side slope. Between Stations 124+75 and 126+00, up to 14 feet will be required near the base of the proposed southern side slope. The planned side slopes of the eastern bridge approach are 3V:1H.

Topography

According to the provided plans, the elevations at the west and east abutments are El 163.03[2] and 162.70, respectively, with a maximum of approximately 27 feet of relief across the proposed alignment.

Drainage

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

Geology

Desha 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-dominant deposits.

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[1] *Arkansas State Highway and Transportation Department Construction Plans for State Highway UPRR STR & APPRS. (S) Desha County Route 278 Section 16, Federal Aid Proj. JOB 020590.* Provided by Arkansas Department of Transportation, dated May 20, 2018.

[2] Elevations are referenced to Mean Sea Level (MSL) in units of feet.

CHAPTER 3. GEOTECHNICAL EXPLORATION

A total of 14 borings were drilled at selected locations near the bridge approaches and the alignment of the proposed bridge. The borings were drilled to a depth of 10 to 100 feet. Two pavement cores were drilled to depths of approximately 8 to 11 inches.

The borings were drilled between January 15th and 24, 2019 using a rotary drill rig (CME 55LC) and hollow-stem augers. 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 geotechnical professional 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. Included on each boring log are elevation, latitude and longitude, and station and offset data estimated from 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
Soil Classification	ASTM D 2488/ D 3282
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. 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), direct shear, one-dimensional consolidation, pH, resistivity, standard proctor, and California Bearing Ratio (CBR) 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 422	T 88
Percent Finer Than No. 200 Sieve	D 1140	T 11
Unconsolidated-Undrained Triaxial Compression	D 2850	T 296
Direct Shear	D 3080	T 236
One-Dimensional Consolidation	D 2435	T 216
pH of Soil	D 4972	T 289
Soil Electrical Resistivity	G 57	T 288
Moisture-Density (Standard Effort)	D 698	T 99
California Bearing Ratio (CBR)	D 1883	T 193

The boring logs were prepared by a project geotechnical engineer from the field logs, visual classification of the soil samples in the laboratory, and laboratory test results. Terms and symbols used on the boring logs are presented on the Boring Log: Terms and Symbols in Appendix C. Stratification lines on the boring logs indicate approximate changes in strata. The transition between strata could be abrupt or gradual.

CHAPTER 5. SUBSURFACE CONDITIONS

Existing Pavement

Borings B-1, -13, -14, C-1 and -2 were drilled in the existing bridge approaches for the purpose of obtaining pavement thickness and subgrade information beneath the existing road way. 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	7	Red Sand	17
B-13	Asphalt	9.5	Gravel	3
			Red Sand	17.5
B-14	Asphalt	6	Gravel	6
			Red Sand	18
C-1*	Asphalt	11	--	--
C-2*	Asphalt	8	--	--

*Asphalt Core Only

Subgrade Materials

The borings were drilled in the alignment of the proposed bridge and approaches, with the exception of Boring B-9 which was offset to the north of the existing bridge. The borings were drilled through approximately 3 to 6 inches of topsoil. Underlying the topsoil or 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 borings logs, with more detailed soil descriptions are included in Appendix C. The laboratory testing used to determine AASHTO and USCS classifications is presented in Appendix E.

The fine-grained, predominately clay soils were classified as low plasticity, “lean”, clay (CL), AASHTO A-7-6, 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 hard. Based on the cross-sections provided, cut areas planned near the bridge abutments will terminate in CH soil.

The fine-grained soils were underlain by coarse-grained soil at depths of 33 to 48 feet and classified as poorly-graded sand (SP), AASHTO A-3, A-1-b, sand with silt (SP-SM), AASHTO A-1-b, A-3, A-2-4, and silty sand (SM), AASHTO A-2-4, with some clayey sand (SC), AASHTO A-2-6. Based on field test results, the coarse-grained soils ranged from loose to very dense.

Groundwater

Groundwater was encountered during drilling operations in the borings at the depths indicated in Table 4. The presence of groundwater in Boring B-9 was obscured by the use of mud rotary drilling methods, which introduces fluid to the borehole. Groundwater levels could vary significantly over time due to the effects of seasonal variation in precipitation, recharge, or other factors not evident at the time of exploration.

Table 4. Summary of Groundwater Depths.

Boring No.	Groundwater Depth (ft.)	Groundwater Elevation
B-2	28.5	132.5
B-3	53.5	90.5
B-4	48.5	92.5
B-6	50	91
B-7	3.5	138.5
B-8	8	134
B-10	38.5	107.5
B-11	35	111
B-12	20	127

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 1V:4H must be benched prior to placing new fill. Slope ratios of 1V:3H 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 and PI of 40 and 20 percent, respectively. 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 three 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.

Pavement Design Information

Composite bulk samples of the auger cuttings were collected from selected borings. Atterberg limits and standard Proctor compaction tests (ASTM D 698/AASHTO T99) were performed on each composite sample. California Bearing Ratio (CBR) tests (ASTM D 1883/ AASHTO T193) were conducted on soaked samples remolded in standard CBR molds using compaction efforts of 25 and 56 blows per layer. The test results are summarized in Table 5.

Table 5. Summary of Compaction and CBR Test Results.

Boring No.	Depth (ft.)	USCS/ AAHSTO	Liquid Limit (%)	Plastic Limit (%)	Proctor Results		CBR Results				Percent Compaction (%)
					Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)	Blows per Layer	Dry Unit Weight (pcf)	Moisture Content (%)	CBR	
B-2	1 – 5	CL	42	20	107.6	15.2	25	95.5	23.06	1.5	88.7
		A-7-6(15)					56	104.9	19.41	3.3	97.5
B-12	1 – 5	CL	49	21	101.8	19.1	25	93.4	26.95	1.6	91.7
		A-7-6(29)					56	101.5	23.86	2.3	99.7

The results in the previous table were interpolated/extrapolated to estimate the CBR values at 95 percent compaction, which is typically considered a minimum compaction value to be achieved in the field. The mean and standard deviation of the interpretation were also calculated. The results are presented in Table 6.

Table 6. CBR Interpolation/Extrapolation.

Boring No.	Depth (ft.)	USCS/ AASHTO	CBR at 95% Compaction
B-2	1 – 5	CL	3.2
		A-7-6(15)	
B-12	1 – 5	CL	2.2
		A-7-6(29)	
Average CBR			2.7

Based on the test results and the data presented in the previous table, a CBR of 2.0 is recommended for design of pavements for this project. A CBR value of this magnitude will result in a relatively thick, expensive pavement structure. We recommend a 3-foot undercut below the base of pavements and backfilling with better (larger CBR) materials. Two materials are considered herein: A-4 (design CBR value of 8.0) and A-3 (design CBR value of 10.0).

The design CBR values mentioned in the previous paragraph were correlated to Resilient Modulus (M_R) and Resistance (R) values. The correlation was performed using information provided by Mr. Paul Tinsley of ARDOT and is presented in Table 7.

Table 7. Soil Design Parameter Recommendations for Pavement Design.

	Soil Classification/Source		
	A-7-6 (In-Situ)	**A-4 (Import)**	**A-3 (Import)**
	CBR = 2	**CBR = 8**	**CBR = 10**
M_R (psi)	3,120	8,059	9,389
Resistance (R) Value	4	12	15

Seismic Considerations

Earthquake Risk. The project area is located in the vicinity of 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 2015 interims.

Seismic Design Parameters. Seismic design parameters based on a seismic hazard with 7% probability of exceedance in 75 years and field and laboratory testing is presented in Table 8.

Table 8. Seismic Design Parameters (7% Probability of Exceedance in 75 years).

Latitude 33.614784°N/Longitude 91.390037°W		
Category/ Parameter	Designation/ Value	Reference
Seismic Site Class	D	AASHTO LRFD 2014 Table 3.10.3.1-1
S_S	0.211g	Computed using design maps provided by the USGS (http://earthquake.usgs.gov/ws/designmaps) using the indicated latitude and longitude coordinates of the project site. The USGS tool used references AASHTO 2009.
S_1	0.074g	
F_a	1.600	
F_v	2.400	
F_{PGA}	1.600	
t_s	0.524g	
t_0	0.105g	
S_{DS}	0.337g	
S_{D1}	0.177g	
PGA	0.090g	
As	0.145g	

Liquefaction and Dynamic Settlement. A study was performed to evaluate the liquefaction and dynamic settlement potential at the site. Both field and laboratory data were used to perform the analysis. The field measurements included the assumed depth of the water table and the SPT N-values. The laboratory data included USCS classification and soil unit weight. An earthquake magnitude (Mw) of 7.7 with a probability of exceedance of 7% in 75 years was considered. A peak ground acceleration of 0.145g was utilized as obtained from the referenced Seismic Design Maps. Groundwater was assumed to be at a depth of approximately 20 feet.

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 9.

Table 9. 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
B-3	80	NA	Less than ¼	¾
B-4	100	NA	Less than ¼	Less than ¼
B-5	80	NA	½	¾
B-6	100	78 to 88 feet	Less than ¼	5 ¾
B-7	80	NA	¾	¾
B-8	100	NA	Less than ¼	¼
B-9	80	NA	Less than ¼	Less than ¼
B-10	100	NA	Less than ¼	Less than ¼
B-11	80	NA	Less than ¼	Less than ¼
B-11	80	NA	Less than ¼	Less than ¼

Please note 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.

Approach Embankment Settlement

Based on the cross sections provided and the proposed pile cap elevations, up to 25 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 approach completion in 200 to 250 days.

Note the one-dimensional consolidation test confines the soil-moisture drainage pathway to one dimension; field drainage is a three-dimensional phenomenon and typically occurs in three directions. It has been our experience the estimated settlement occurs over a shorter period than anticipated. Therefore, it is our professional opinion the estimated settlement will approach completion over a period of 100 to 120 days. It is also important to realize field drainage can be impacted by actual soil conditions and construction methods and sequencing. We recommend a settlement monitoring program be implemented and the survey data be forwarded to Geotechnology for review. The settlement pattern will be used to decide when construction activities can commence.

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 practically dissipated.

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 plans provided by Garver, the abutment slopes for the existing bridge are covered in rip rap and slope 2H:1V. Riprap will be placed to construct similar slopes at the proposed locations of abutment widening. Geotechnology performed stability analyses for deep-seated, global failure of bridge abutment slopes using the computer program SLOPE/W. Short-term, long-term and seismic 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 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 restraining 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 0.073g, corresponding to one-half the peak ground acceleration (per FHWA Publication HI-99-012) was utilized. Section profiles with calculated critical failure arcs and utilized soil parameters are presented in Appendix F for the selected analyses.

Table 10. Results of Slope Stability Analyses.

Location	Description	Slope Height (ft.)	Calculated Factor of Safety		
			Short-Term Static[a]	Long-Term Static[a]	Seismic[b]
West Abutment	2:1 Fill Slope	20	1.854	1.209	1.487
Side Slope Station 110+18	3:1 Fill Slope	21	1.822	1.863	1.416
East Abutment	2:1 Fill Slope	17	2.235	1.109	1.768
Side Slope Station 119+20	3:1 Fill Slope	31	1.400	1.369	1.158

[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.

Insufficient factors of safety (FOS) against global stability failure were computed for the side-slope at Station 119+20 in the short- and long-term conditions and in the long-term condition at each of the abutment slopes.

Reinforced Slopes. An analysis for the global stability of reinforced slopes was performed for the East and West Abutment slopes and the Side Slope at Sta. 119+37 using the computer program ReSSA, the slope configurations and soil parameters utilized in the global stability analysis, and the Bishop method. The design was based on the reinforcement properties presented in Table 11. If a different reinforcement is desired, Geotechnology should be contacted to revise recommendations.

Table 11. Geogrid Design Properties.

Reinforcement Properties	
Reinforcement Type	Geosynthetic
Material Type	Geogrid
Design Tensile Strength (T_{Design})	1,828 lb./ft.
Reduction Factor: Installation Damage	1.10
Reduction Factor: Durability	1.20
Reduction Factor: Creep	1.45
Reduction Factor: Coverage Ratio	1.00
Interaction Parameter: Direct Sliding	0.67
Interaction Parameter: Pullout	0.67
Interaction Parameter: Scale Effect Correction Factor	0.80

In the analysis we modeled the use of cohesive fill meeting the requirements in the Site Preparation and Earthwork Section of this report. The soil parameters used in the analysis for the Short Term, Long Term, and Seismic Condition are presented in Table 12. If granular materials are to be used, Geotechnology should be contacted to revise recommendations.

Table 12. Utilized Soil Parameters.

Soil Layer	Unit Weight (pcf)	Soil Parameters					
		Short Term		Long Term		Seismic	
		Cohesion (psf)	Phi (°)	Cohesion (psf)	Phi (°)	Cohesion (psf)	Phi (°)
Engineered Fill	120	1,500	--	50	30	1,500	--
Soft Fat Clay	116	800	--	--	24	800	--
Fat Clay	116	1,200	--	--	24	1,200	--
Sand	123	--	36	--	36	--	36

Based on the analysis, slopes reinforced as indicated in Table 13 through Table 19 meet the target factors of safety for the short term, long term, and seismic condition for the East and West Abutments and the Side Slope at Sta 119+37. It should be noted the models did not consider the relative stabilizing effect of foundation piles driven to support the abutments or cladding of abutments with rip rap or concrete. The number and length of the reinforcement required is dependent on the planned height of the slope and slope of the ground surface; if configuration of the slope changes, Geotechnology should be contact to revise these recommendations.

Table 13. East Abutment Slope (2:1) Reinforcement Configurations.

Reinforcement Layer No.	Elevation	Length (feet)
Toe of Slope	148.0	--
1	150	20
2	152	
3	154	
4	156	
5	158	
6	160	
7	162	
Top of Slope	165.0	--

Table 14. Side Slope Sta. 119+37 (2:1) Reinforcement Configurations.

Reinforcement Layer No.	Elevation	Length (feet)
Toe of Slope	135.7	--
1	136	40
2	138	
3	140	
4	142	
5	144	
6	146	
7	148	
8	150	
9	152	
10	154	
11	156	
12	158	
13	160	
14	162	
Top of Slope	165.0	--

Table 15. Side Slope Sta. 119+85 (2:1) Reinforcement Configurations.

Reinforcement Layer No.	Elevation	Length (feet)
Toe of Slope	135.5	--
1	136	30
2	138	
3	140	40
4	142	
5	144	
6	146	
7	148	
8	150	
9	152	
10	154	
11	156	
12	158	
13	160	
Top of Slope	164.0	--

Table 16. Side Slope Sta. 121+00 (2:1) Reinforcement Configurations.

Reinforcement Layer No.	Elevation	Length (feet)
Toe of Slope	135.2	--
1	136	25
2	138	
3	140	30
4	142	
5	144	
6	146	
7	148	
8	150	40
9	152	
10	154	
11	156	
Top of Slope	159.3	--

Table 17. Side Slope Sta. 122+00 (2:1) Reinforcement Configurations.

Reinforcement Layer No.	Elevation	Length (feet)
Toe of Slope	135.0	--
1	136	25
2	138	
3	140	
4	142	
5	144	
6	146	
7	148	30
8	150	
9	152	
Top of Slope	156.0	--

Table 18. Side Slope Sta. 122+55 (2:1) Reinforcement Configurations.

Reinforcement Layer No.	Elevation	Length (feet)
Toe of Slope	134.9	--
1	136	20
2	138	
3	140	
4	142	
5	144	
6	146	
7	148	
8	150	
Top of Slope	154.5	--

Extend reinforcement from Sta. 122+55 to Sta. 123+00 where the side slope begins a transition to a 3:1 slope at Sta. 123+13.

Table 19. West Abutment Slope (2:1) Reinforcement Configurations.

Reinforcement Layer No.	Elevation	Length (feet)
Toe of Slope	143.6	--
1	145.5	20
2	147.5	
3	149.5	
4	151.5	
5	153.5	
6	155.5	
7	157.5	
8	159.5	
9	161.5	
Top of Slope	164.0	--

Construction Considerations. Based on the required lengths of the geogrid reinforcement, over excavation of the existing slopes will be required for installation of the geogrid reinforcement.

At the east abutment where the soil will be reinforced along the abutment and side slope, the reinforcement should be installed so the reinforcement overlaps in a radial pattern around the corner to create continuous reinforcement for both the abutment and side slope.

Special attention should be paid to placement of the reinforcement with regards to the location of driven foundations. During installation of the geogrid, for reinforcement in the upper 10 feet of soil, a hole should be cut to allow for the driving of piles through the geogrid. At depths greater than 10 feet, the overburden pressure should allow for the driving of piles through the geogrid without wrinkling the geogrid mat.

We recommend a working platform of 6 inches or more between the reinforcement and the wheels or track of construction equipment. The abutment slopes should be protected from erosion/sloughing during construction prior to the placement of cladding or rip rap.

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 and bents will be supported using 18-inch, closed-ended, steel pipe piles. 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 profile encountered in the borings, approximate boring elevations, and the proposed final grade. Downdrag forces due to liquefaction during the design earthquake event were not considered. Nominal resistance curves showing the capacity due to skin friction and the total capacity (skin friction + end bearing) for the abutments and bents are presented in Appendix H. Uplift capacities (tension) may be calculated using the resistance provided by skin friction.

<u>Resistance Factors</u>. Resistance factors should be applied to the nominal resistances provided. In general, a factor of 0.45 may be used for piles in compression and 0.35 in tension. Based on AASHTO LRFD (2014) higher resistance factor may be used in accordance with the level of pile testing performed as indicated in Table 20.

Table 20. 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*	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*	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

* 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.

Static Pile Load Testing. 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 the previous table. 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 three 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.

Driven Pile Construction Considerations. A WEAP analysis using the program GRLWEAP was performed for the proposed piles based on the soil profile encountered, the pile cut-off elevations provided, and a Delmag D25-32 hammer. Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. Specifications should include minimum pile hammer energies of:

- 36 kip-feet for the 18-inch, 203-ton nominal capacity piles at Bents 1 and 4, and
- 47 kip-feet for the 18-inch, 294-ton nominal capacity piles at Bents 2 and 3.

Alternatively, potential driving criteria can be developed using wave equation analyses after the actual pile hammer is selected.

An additional design phase analysis using the design loads, pile lengths, and the hammer to be used should be performed to determine pile drivability and minimum hammer energy.

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 F 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 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 shaft depends on the final foundation configuration.

Downdrag Due to Fill-Induced Settlement. Based on settlement analysis performed for the 25-foot maximum fill placement, up to 6-inches of settlement is predicted. Piles driven immediately after fill placement will be subject to downdrag loads as the soil consolidates under the load of the fill. The survey data recorded for the settlement monitoring program recommended in the Abutment Settlement Section, should be forwarded to Geotechnology to determine if the settlement due to fill placement is complete prior to driving piles.

Downdrag due to Dynamic Settlement. Based on liquefaction analysis results, our professional opinion, and the uncertainties described in the Seismic Considerations section, we expect up to ¾ of an inch of dynamic settlement within the upper 50 feet of soil during the design earthquake event (7% exceedance in 75 years) at one boring location. Liquefaction induced downdrag can be reduced by performing ground improvement on the potentially liquefiable soil layers. It is our professional opinion that driving the 18-inch and 24-inch closed-ended, steel pipe piles through the identified liquefiable layers will sufficiently densify susceptible layers in the vicinity of the piles such that liquefaction induced downdrag is not expected to occur in the design earthquake. However, the extent of the densification cannot be determined at this stage. 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.*

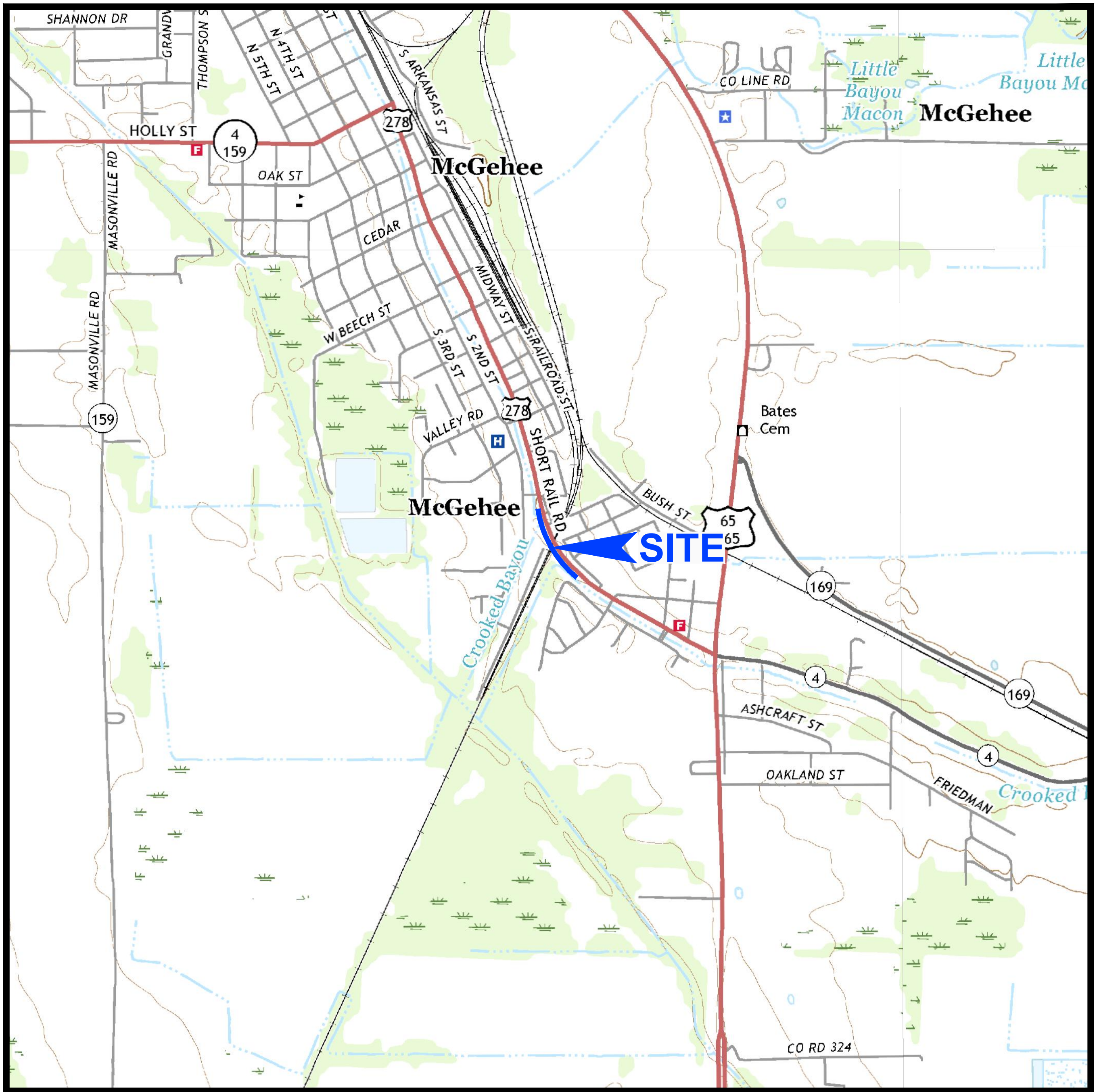
Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

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.

GBC 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



NOTES

1. Plan adapted from 7.5 minute U.S.G.S. maps for McGehee South, McGehee North, McArthur, and Halley, Arkansas quadrangles last revised in 2017.

0 2,000 4,000

SCALE IN FEET

N

Drawn By: WAH	Ck'd By: ALY	App'vd By: DMS
Date: 2-4-19	Date: 2-4-19	Date: 2-21-19

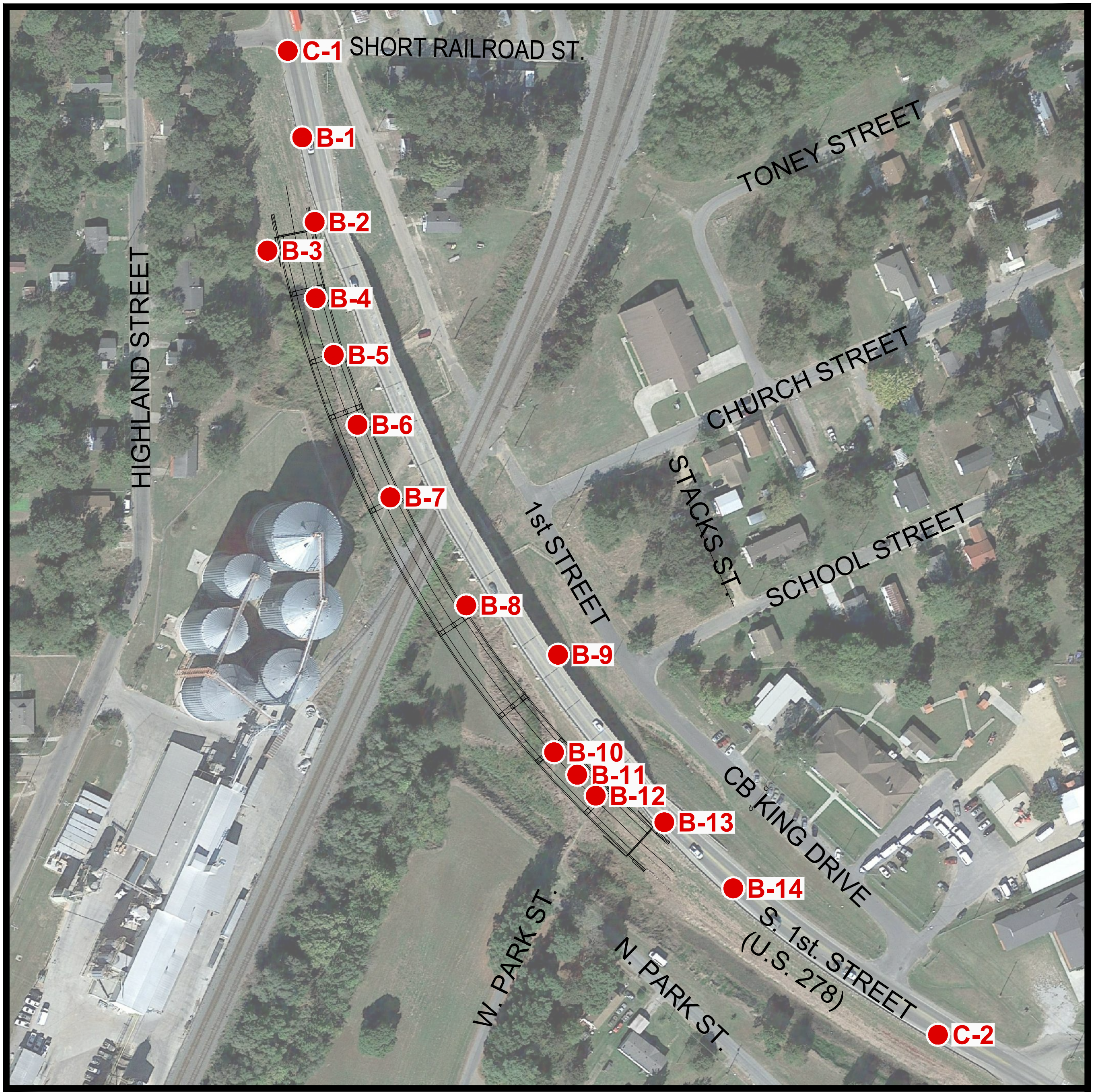
GEOTECHNOLOGY INC
FROM THE GROUND UP

Hwy 278 Over Union Pacific Railroad
McGehee, Arkansas

SITE LOCATION AND TOPOGRAPHY

Project Number
J033659.01

FIGURE 1



NOTES

1. Plan adapted from an October 30, 2016 aerial photograph courtesy of Google Earth and an undated, untitled drawing, supplied by the client.

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

LEGEND

● Boring Location

0 200 400

SCALE IN FEET

N

Drawn By: WAH	Ck'd By: ALY	App'vd By: DMS
Date: 2-4-19	Date: 2-4-19	Date: 2-21-19

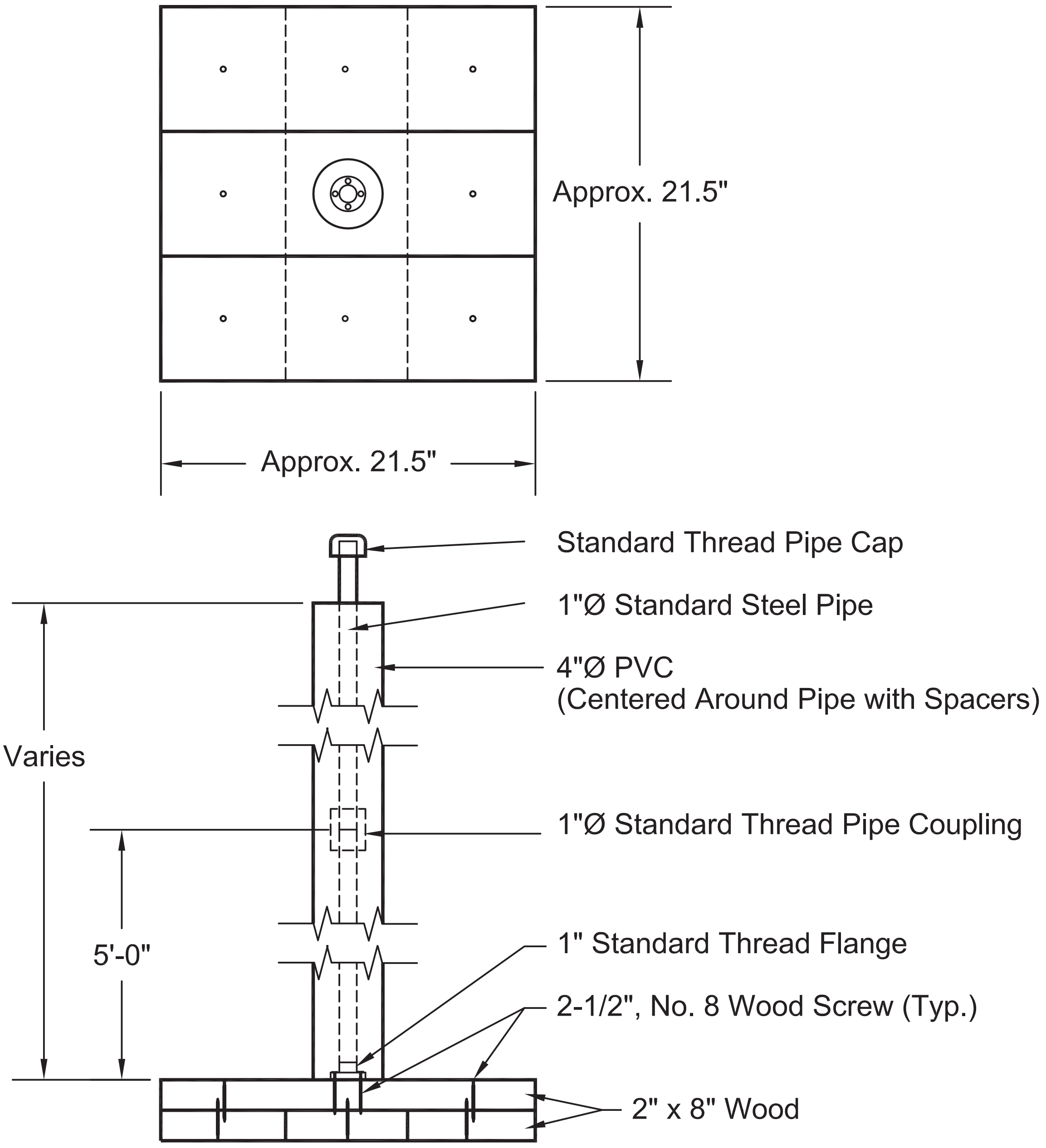
GEOTECHNOLOGY INC
FROM THE GROUND UP

Hwy 278 Over Union Pacific Railroad
McGehee, Arkansas

AERIAL PHOTOGRAPH OF SITE AND BORING LOCATIONS

Project Number
J033659.01

FIGURE 2



NOTES

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

Drawn By: NAA	Ck'd By: ASE	App'vd By: DMS
Date: 11/6/2012	Date: 11/6/2012	Date: 3/18/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Highway 278 Over Union Pacific Railroad
Desha County, Arkansas

SETTLEMENT PLATE DETAIL

Project Number
J033659.01

FIGURE 3

APPENDIX C – BORING INFORMATION

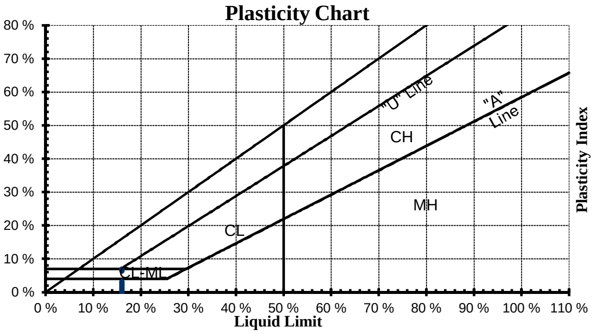
Boring Log Terms and Symbols

Boring Logs

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

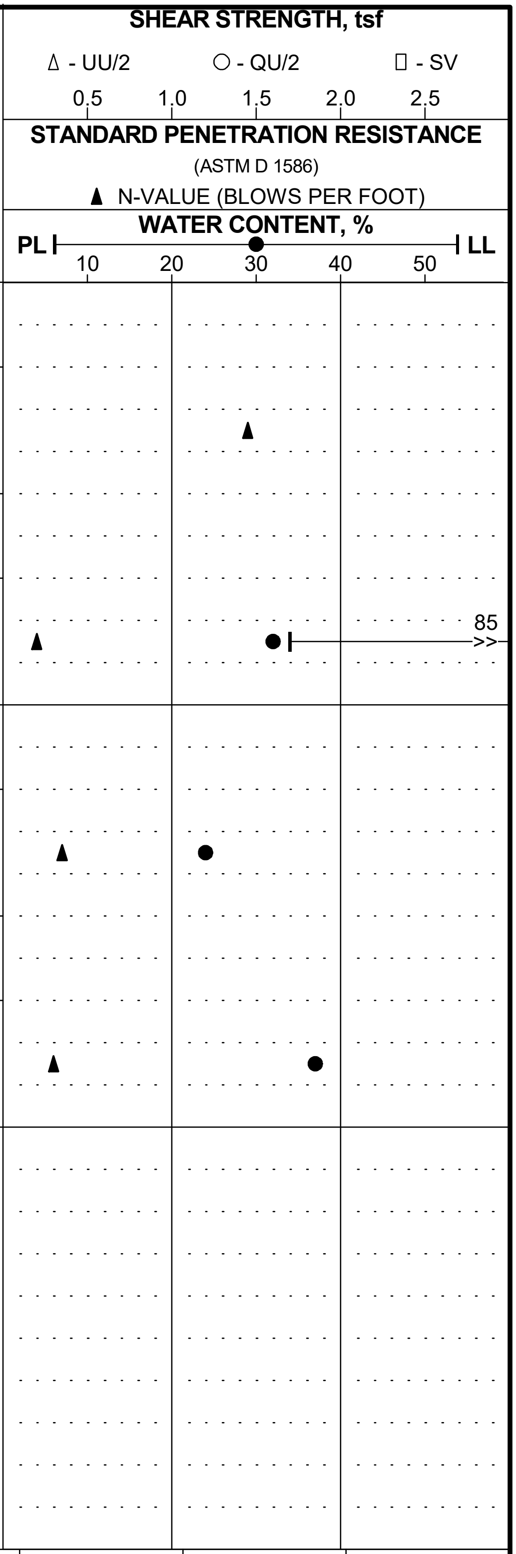
Relative composition and Unified Soil Classification System (USCS) designations are based on visual descriptions and are approximate only. If laboratory tests were performed to classify the soil, the USCS designation is shown in parenthesis.

Surface Elevation: 155

Datum MSL

Completion Date: 1/17/19
Station: 109+00
Offset: 28 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		ASPHALT: 7 inches	■		
		FILL: red sand	▩	9-15-14	SS1
		Soft to medium stiff, red-brown and gray, FAT CLAY - (CH)	▨		
			▨	2-2-2	SS2
5	150		▨		
			▨	2-3-4	SS3
			▨	1-3-3	SS4
10	145	Boring terminated at 10 feet.			



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 __ FEET
TPD DRILLER KLR LOGGER
CME 55LC DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: B- 1

Geotechnology Project No.
J033659.01

LOG OF BORING 2002 WL J033659.01.GPJ GTINC 0638301.GPJ 3/22/19

Surface Elevation: 164

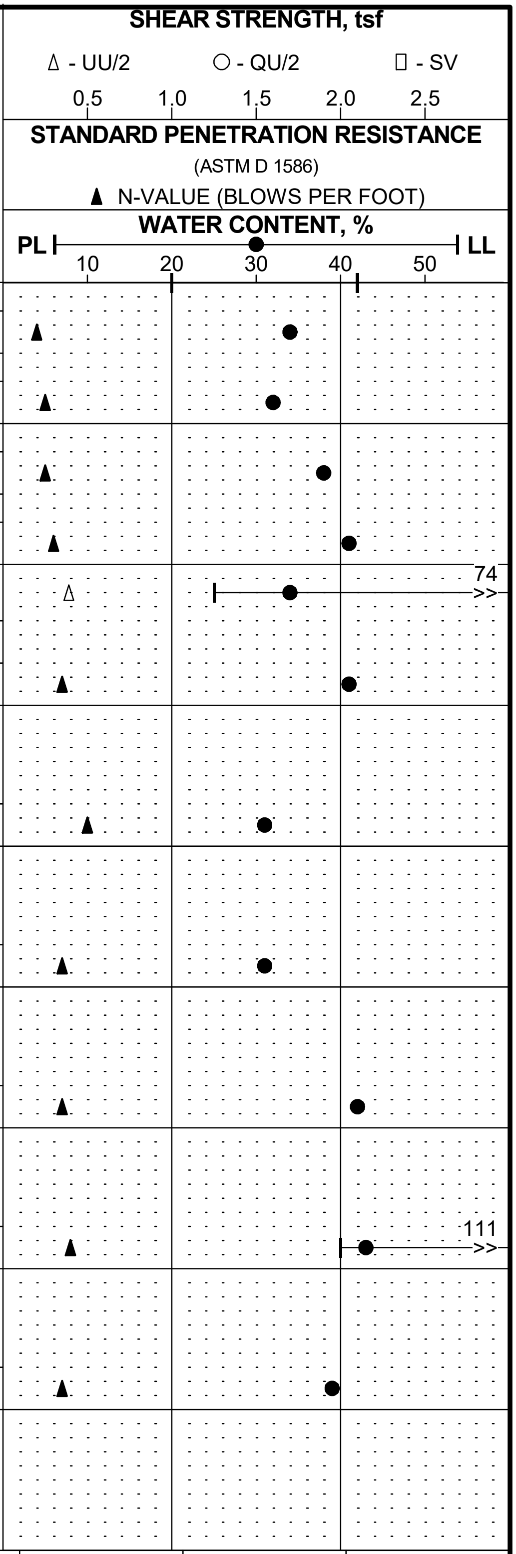
Datum MSL

Completion Date: 1/15/19

Station: 110+16

Offset: 30 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches		
		Soft to medium stiff, gray, LEAN CLAY with sand - (CL) 74.5% passing No. 200 sieve	1-2-2	SS1
			2-2-3	SS2
5	159	Medium stiff, gray and brown, FAT CLAY - (CH)		
			2-3-2	SS3
			2-2-4	SS4
10	154		88	ST5
		Medium stiff to stiff, brown to gray, FAT CLAY, trace oxide nodules - CH pH = 7.61 Soil Resistivity: 570 ohms-cm	2-3-4	SS6
15	149			
			3-4-6	SS7
20	144			
			2-3-4	SS8
25	139			
		⊻	2-3-4	SS9
30	134			
		Medium stiff, maroon, FAT CLAY - CH	3-3-5	SS10
35	129			
			3-3-4	SS11
40	124	Boring terminated at 40 feet.		



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 28.5 FEET ⊻

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM __ FEET
TPD DRILLER KLR LOGGER
CME 55LC DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: B- 2

Geotechnology Project No.
J033659.01

Surface Elevation: 147

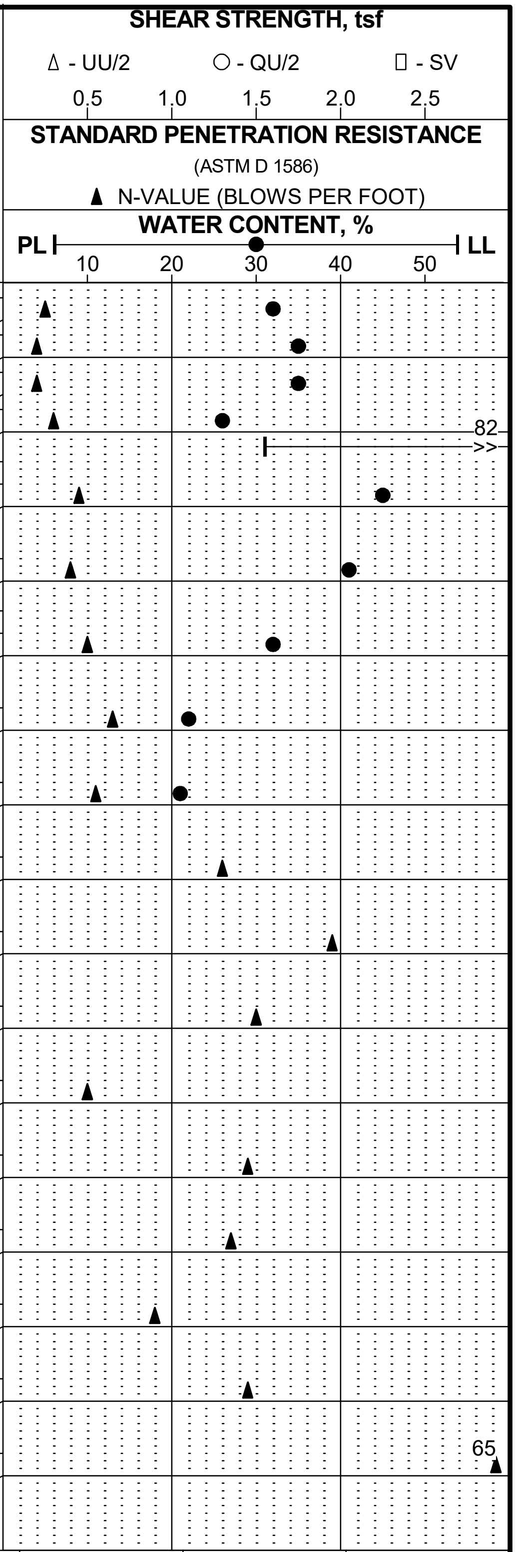
Datum MSL

Completion Date: 1/15/19

Station: 110+16

Offset: 40 RT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches		
		Medium stiff to soft, brown, LEAN CLAY - CL	1-2-3	SS1
5	142		2-2-2	SS2
			1-2-2	SS3
		Medium stiff, brown and gray, clayey SILT - ML	3-2-4	SS4
10	137	Stiff to medium stiff, brown to gray, FAT CLAY - (CH)		ST5
15	132	Soil Resistivity: 6,270 ohms-cm	2-4-5	SS6
20	127		2-3-5	SS7
				ST8
25	122		2-5-5	SS9
30	117		3-6-7	SS10
35	112	Stiff, gray and brown, FAT CLAY, little sand - CH	4-5-6	SS11
40	107	Loose to dense, gray SAND - SP pH = 7.05	8-12-14	SS12
45	102		11-16-23	SS13
50	97		9-10-20	SS14
55	92	3.3% passing No. 200 sieve ⊽	4-7-3	SS15
60	87		13-13-16	SS16
65	82		17-15-12	SS17
70	77		10-10-8	SS18
75	72	Medium dense to very dense, gray SAND, little gravel - SP	16-17-12	SS19
80	67		18-32-33	SS20
		Boring terminated at 80 feet.		



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 53.5 FEET ⊻

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 53.5 FEET
TPD DRILLER KLR LOGGER
CME 55LC DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: B- 3

Geotechnology Project No.
J033659.01

Surface Elevation: 141

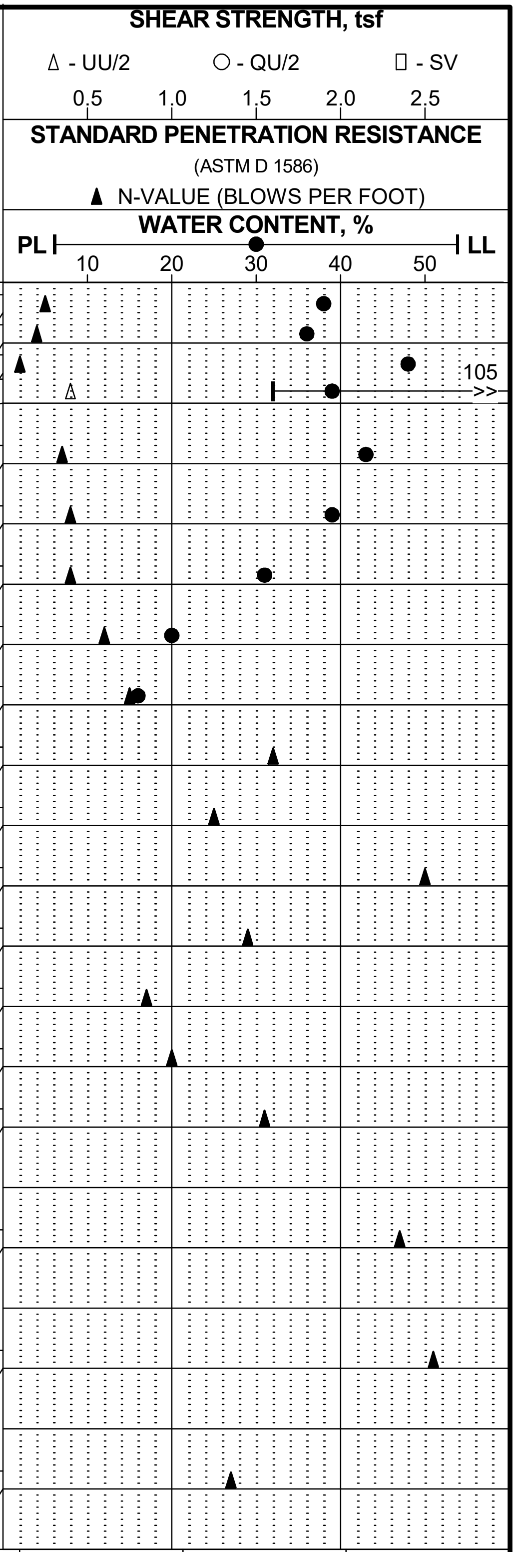
Datum MSL

Completion Date: 1/16/19

Station: 111+00

Offset: 8 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches			
		FILL: red-brown to black, lean clay, trace wood		1-2-3	SS1
5	136			1-2-2	SS2
				1-1-1	SS3
10	131	Medium stiff to stiff, red-brown to gray, FAT CLAY - (CH) 27.0% Gravel 65.1% passing No. 200 sieve		84	ST4
15	126			2-3-4	SS5
20	121			3-3-5	SS6
25	116			4-4-4	SS7
30	111	Stiff, gray, FAT CLAY, little sand - CH		5-4-8	SS8
35	106	Medium dense to dense, gray CLAYEY SAND, trace silt - SC 24.8% passing No. 200 sieve		4-7-8	SS9
40	101			9-13-19	SS10
45	96			7-8-17	SS11
			▽		
50	91			15-25-25	SS12
55	86	pH = 7.13 Soil Resisitivity: 2,679 ohms-cm		13-14-15	SS13
60	81	Medium dense to dense, gray, GRAVELLY SAND, trace silt - (SP) 31.0% Gravel 4.1% passing No. 200 sieve		8-9-8	SS14
65	76			11-9-11	SS15
70	71	Medium dense to very dense, gray, CLAYEY SAND - SC 15.7% passing No. 200 sieve		10-13-18	SS16
75	66				
80	61			16-22-25	SS17
85	56				
90	51			32-25-26	SS18
95	46				
100	41	Boring terminated at 100 feet.		10-12-15	SS19



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 48.5 FEET ▽

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 48.5 FEET
TPD DRILLER KLR LOGGER
CME 55LC DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

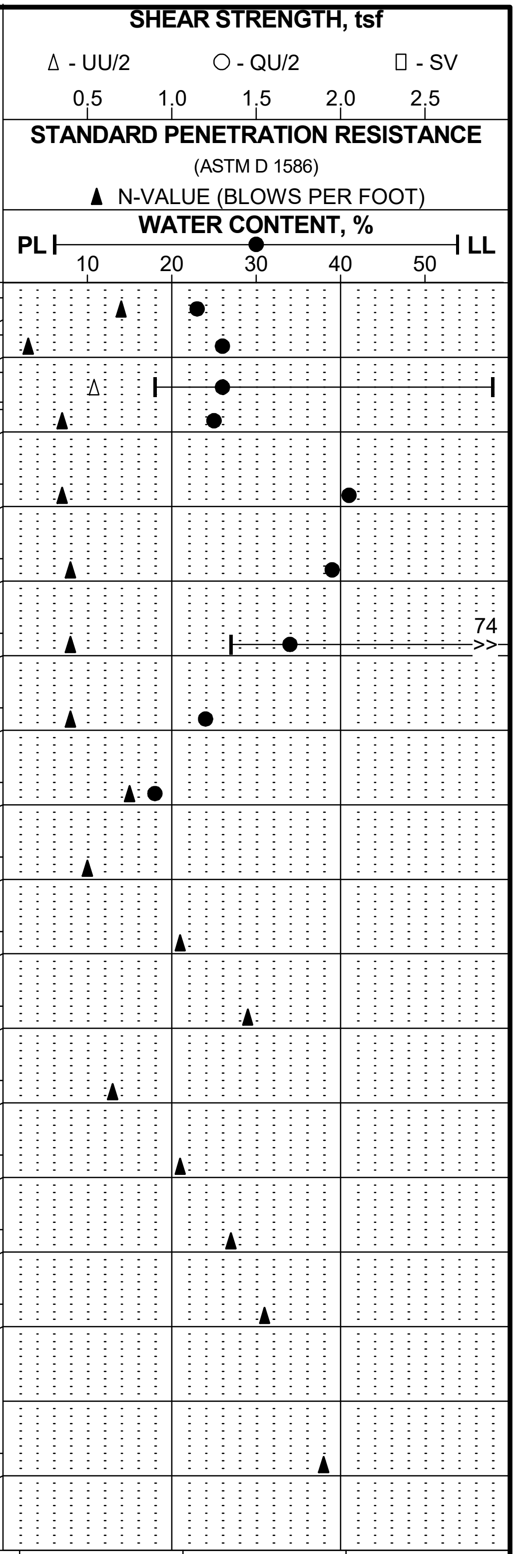
LOG OF BORING: B- 4

Geotechnology Project No.
J033659.01

Surface Elevation: 140
Datum MSL
Completion Date: 1/17/19
Station: 111+90
Offset: 8LT

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

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches			
		Stiff, brown SILT - ML		3-4-10	SS1
5	135	Soft, gray, FAT CLAY - (CH)		2-2-1	SS2
				98	ST3
10	130	Medium stiff, gray and brown, FAT CLAY - (CH)		2-3-4	SS4
15	125			2-3-4	SS5
20	120	pH = 7.32 Soil Resistivity: 912 ohms-cm		2-3-5	SS6
25	115			3-4-4	SS7
30	110			3-3-5	SS8
35	105	Medium dense to loose, gray to brown SAND, trace silt - SP		5-6-9	SS9
40	100			7-5-5	SS10
45	95			10-10-11	SS11
50	90			11-14-15	SS12
55	85	0.8% Gravel 3.7% passing No. 200 sieve		10-5-8	SS13
60	80			13-11-10	SS14
65	75	Medium dense to dense, gray SAND, trace gravel - SP pH = 8.01 Soil Resistivity: 2,850 ohms-cm		13-13-14	SS15
70	70			12-14-17	SS16
75	65				
80	60			14-20-18	SS17
		Boring terminated at 80 feet.			



GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 33.5 FEET
TPD DRILLER KLR LOGGER
CME 55LC DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

**ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas**

LOG OF BORING: B- 5

**Geotechnology Project No.
J033659.01**

Surface Elevation: 141

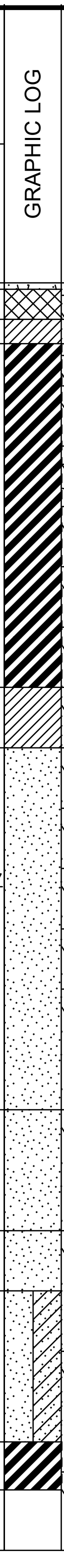
Datum MSL

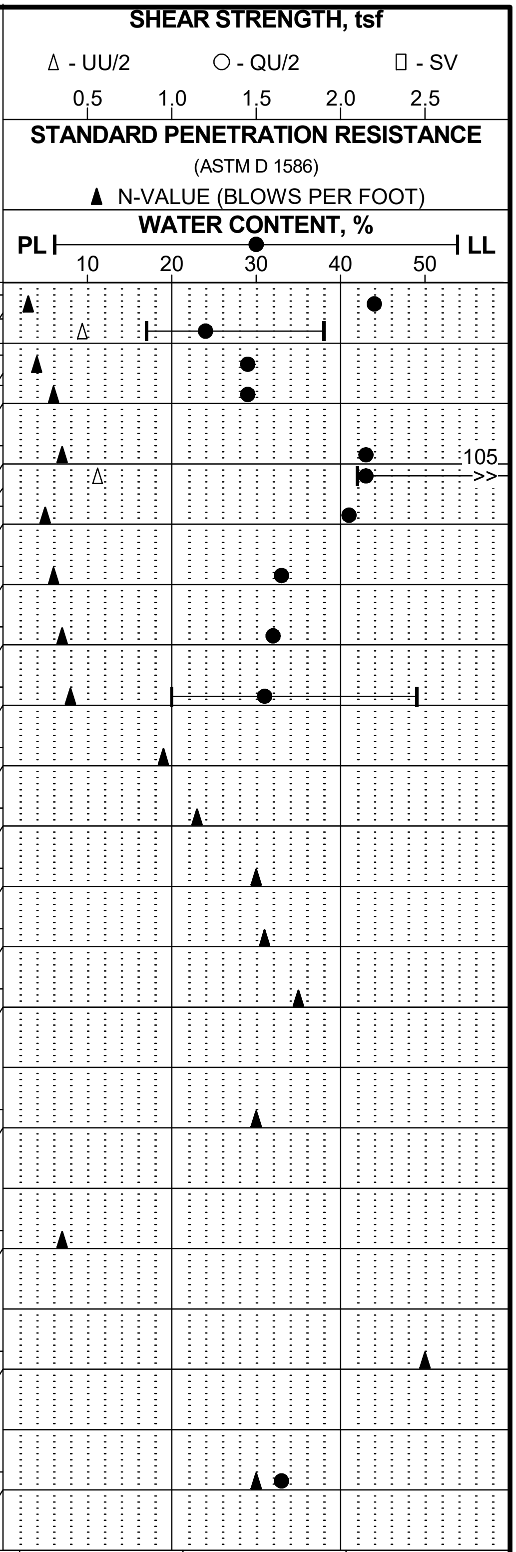
Completion Date: 1/18/19

Station: 112+80

Offset: 8 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches	2-2-1	SS1
		FILL: black, lean clay	100	ST2
5	136	Medium stiff, gray, LEAN CLAY - (CL)		
		Soft to medium stiff, gray and brown to red-brown, FAT CLAY - (CH)	1-2-2	SS3
10	131		2-3-3	SS4
15	126		3-3-4 79	SS5 ST6
20	121		2-2-3	SS7
25	116		2-3-3	SS8
30	111		2-3-4	SS9
35	106	Medium stiff, gray, LEAN CLAY - (CL) 75.1% passing No. 200 sieve	5-3-5	SS10
40	101	Medium dense to dense, gray SAND - SP pH = 6.91 Soil Resistivity: 3,477 ohms-cm	3-8-11	SS11
45	96		5-10-13	SS12
50	91	∇	3-14-16	SS13
55	86		12-13-18	SS14
60	81		10-17-18	SS15
65	76			
70	71	Medium dense, gray SAND, trace gravel - SP	10-14-16	SS16
75	66			
80	61	Loose, gray SAND and wood - SP	1-1-6	SS17
85	56	Very dense, gray SAND WITH CLAY - SP-SC		
90	51	7.4% passing No. 200 sieve	50=5"	SS18
95	46			
		Very stiff, gray, FAT CLAY - CH		
100	41	Boring terminated at 100 feet.	8-14-16	SS19





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 50 FEET ∇

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM

WASHBORING FROM 50 FEET

TPD DRILLER KLR LOGGER

CME 55LC DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: B- 6

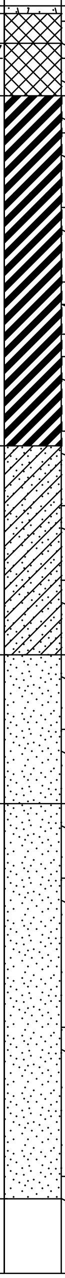
Geotechnology Project No.
J033659.01

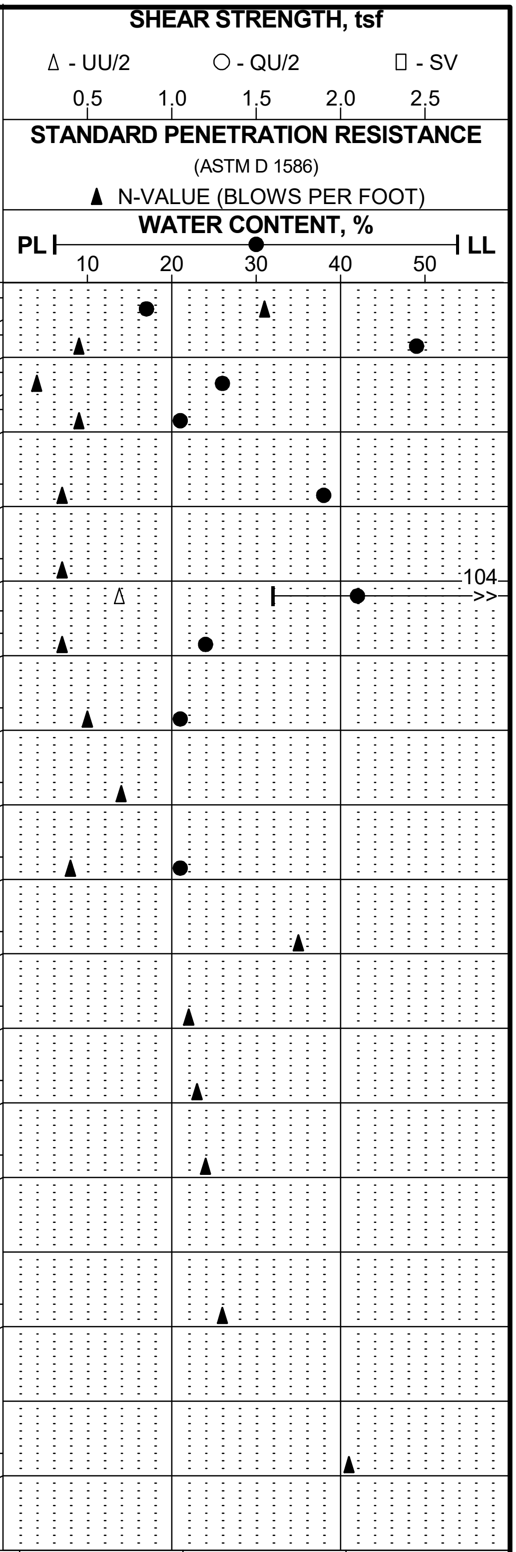
Surface Elevation: 142
Datum MSL

Completion Date: 1/20/19
Station: 113+86
Offset: 6 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches		
			9-16-15	SS1
		FILL: brown silt ∇		
			2-5-4	SS2
5	137	FILL: black, lean clay and gravel with wood		
		Soft to stiff, gray to red-brown, FAT CLAY - (CH)	1-2-2	SS3
			2-4-5	SS4
10	132			
			2-3-4	SS5
15	127			
			2-3-4	SS6
20	122			
			80	ST7
			2-3-4	SS8
25	117			
			3-5-5	SS9
30	112	Loose, red and gray, CLAYEY SAND - SC		
			3-8-6	SS10
35	107			
		32.7% passing No. 200 sieve	2-4-4	SS11
40	102			
		Loose to dense, gray SAND - SP	9-15-20	SS12
45	97			
			13-11-11	SS13
50	92			
		Medium dense to dense, gray SAND, trace gravel - SP	10-11-12	SS14
55	87			
			10-12-12	SS15
60	82			
65	77			
		2.7% Gravel 4.0% passing No. 200 sieve	8-13-13	SS16
70	72			
75	67			
			17-19-22	SS17
80	62	Boring terminated at 80 feet.		

GRAPHIC LOG





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 3.5 FEET ∇

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 45 FEET
TPD DRILLER KLR LOGGER
CME 55LC DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: B- 7

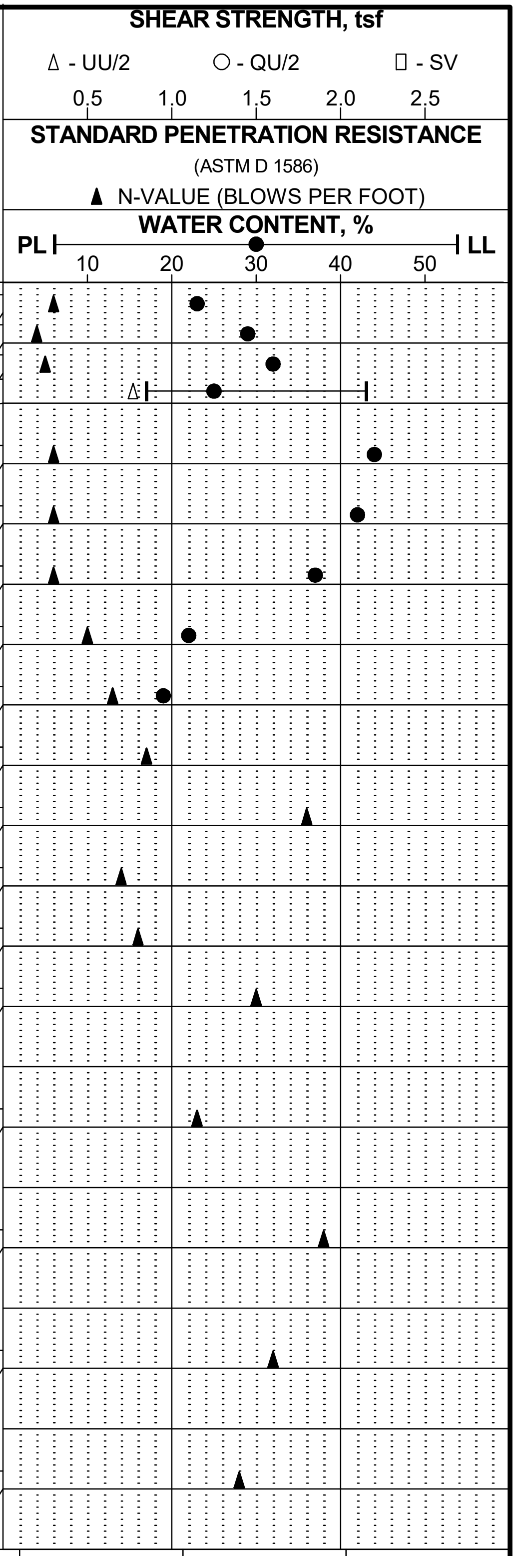
Geotechnology Project No.
J033659.01

LOG OF BORING 2002 WL J033659.01.GPJ GTINC 0638301.GPJ 3/22/19

Surface Elevation: 142
Datum MSL

Completion Date: 1/20/19
Station: 115+50
Offset: 24 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches		
		Medium stiff to soft, red-brown SILT - ML	2-3-3	SS1
5	137	Medium stiff, gray, LEAN CLAY - (CL)	1-2-2	SS2
		∇	1-2-3	SS3
10	132		102	ST4
15	127	Medium stiff, red-brown and gray, FAT CLAY - CH	2-3-3	SS5
20	122	pH = 7.68 Soil Resistivity: 741 ohms-cm	2-3-3	SS6
25	117		2-2-4	SS7
30	112	Stiff, red-brown and gray, FAT CLAY, little sand - CH	4-4-6	SS8
35	107	Medium dense to dense, brown to gray, SILTY SAND - SM 19.5% passing No. 200 sieve	4-7-6	SS9
40	102		8-11-6	SS10
45	97		12-19-17	SS11
50	92	11.7% passing No. 200 sieve	3-5-9	SS12
55	87		6-7-9	SS13
60	82		13-14-16	SS14
65	77			
70	72	Medium dense to dense, gray SAND, trace gravel - SP pH = 8.45	10-11-12	SS15
75	67	Soil Resistivity: 3,192 ohms-cm		
80	62		17-17-21	SS16
85	57			
90	52		17-19-13	SS17
95	47			
100	42	Very stiff, gray, FAT CLAY - CH Boring terminated at 100 feet.	20-12-16	SS18



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 8 FEET ∇

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 40 FEET
TPD DRILLER KLR LOGGER
CME 55LC DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: B- 8

Geotechnology Project No.
J033659.01

Surface Elevation: 145

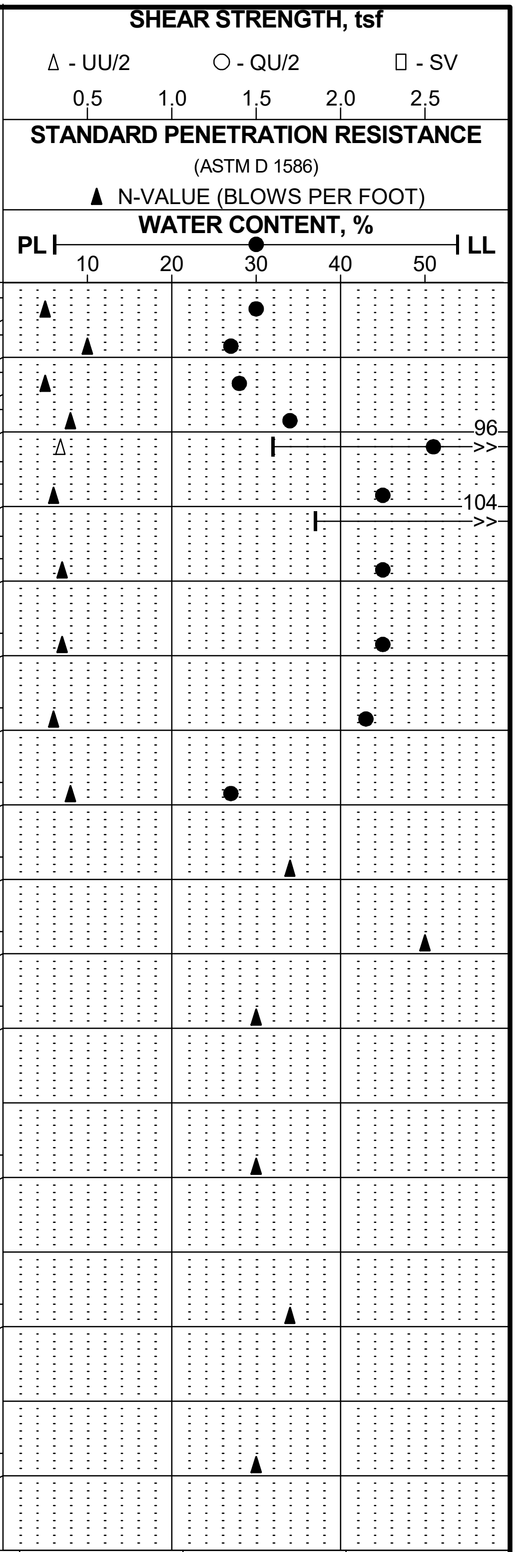
Datum MSL

Completion Date: 1/24/18

Station: 117+00

Offset: 88 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches		
			3-2-3	SS1
		Medium stiff, red-brown, LEAN CLAY - CL		
			5-5-5	SS2
5	140	Stiff to medium stiff, brown and gray, FAT CLAY - (CH)		
			3-3-2	SS3
			1-5-3	SS4
10	135			
			71	ST5
			2-2-4	SS6
15	130			
				ST7
			2-3-4	ST8
20	125			
			2-3-4	ST9
25	120			
			3-3-3	ST10
30	115			
		59.4% passing No. 200 sieve	3-3-5	ST11
35	110			
		Dense to medium dense, gray SAND - SP	12-16-18	ST12
40	105			
			21-27-23	ST13
45	100			
			12-15-15	ST14
50	95			
55	90			
			10-14-16	ST15
60	85			
65	80			
		Dense to medium dense, gray SAND WITH SILT, trace gravel - SP-SM	13-17-17	ST16
70	75			
75	70			
		8.7% passing No. 200 sieve	11-13-17	ST17
80	65			
		Boring terminated at 80 feet.		



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 __ FEET

TPD DRILLER KLR LOGGER

CME 55LC DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

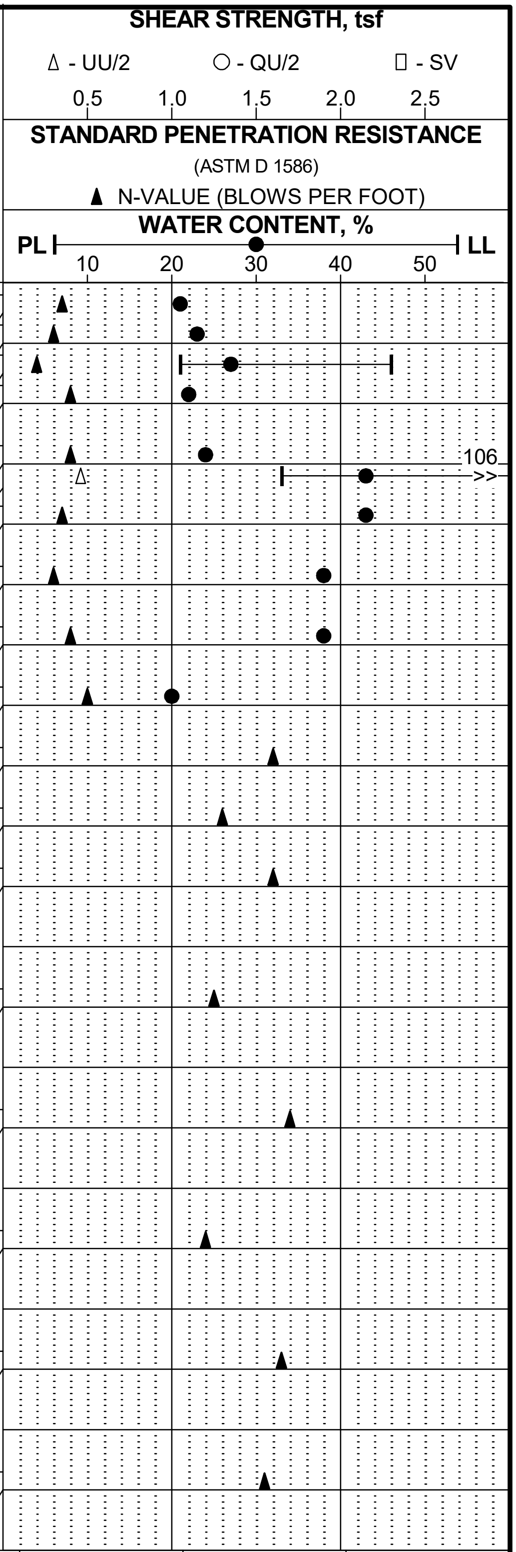
ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: B- 9

Geotechnology Project No.
J033659.01

Surface Elevation: 146
Datum MSL
Completion Date: 1/21/19
Station: 117+70
Offset: 6 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches		
		Medium stiff to soft, red-brown to gray, LEAN CLAY - (CL)	3-3-4	SS1
5	141		2-3-3	SS2
			1-2-2	SS3
10	136		2-3-5	SS4
15	131	Medium stiff, gray, FAT CLAY, some sand - CH	2-3-5	SS5
		Medium stiff, red-brown to gray, FAT CLAY - (CH)	79	ST6
20	126		2-3-4	SS7
25	121		2-2-4	SS8
30	116		3-4-4	SS9
35	111	Stiff, gray, FAT CLAY, some sand - CH	4-5-5	SS10
40	106	Medium dense to dense, gray SAND WITH SILT - SP-SM	10-14-18	SS11
45	101	5.0% passing No. 200 sieve	10-12-14	SS12
50	96	pH = 8.41 Soil Resistivity: 2,793 ohms-cm	18-16-16	SS13
55	91			
60	86		10-12-13	SS14
65	81			
70	76	Dense to medium dense, gray SAND, trace gravel - SP	15-15-19	SS15
75	71			
80	66		11-12-12	SS16
85	61			
90	56		19-16-17	SS17
95	51			
100	46	Boring terminated at 100 feet.	17-15-16	SS18



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 38.5 FEET ▽

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 40 FEET
TPD DRILLER KLR LOGGER
CME 55LC DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge over Union Pacific Railroad Desha County, Arkansas

LOG OF BORING: B-10

Geotechnology Project No. J033659.01

Surface Elevation: 146

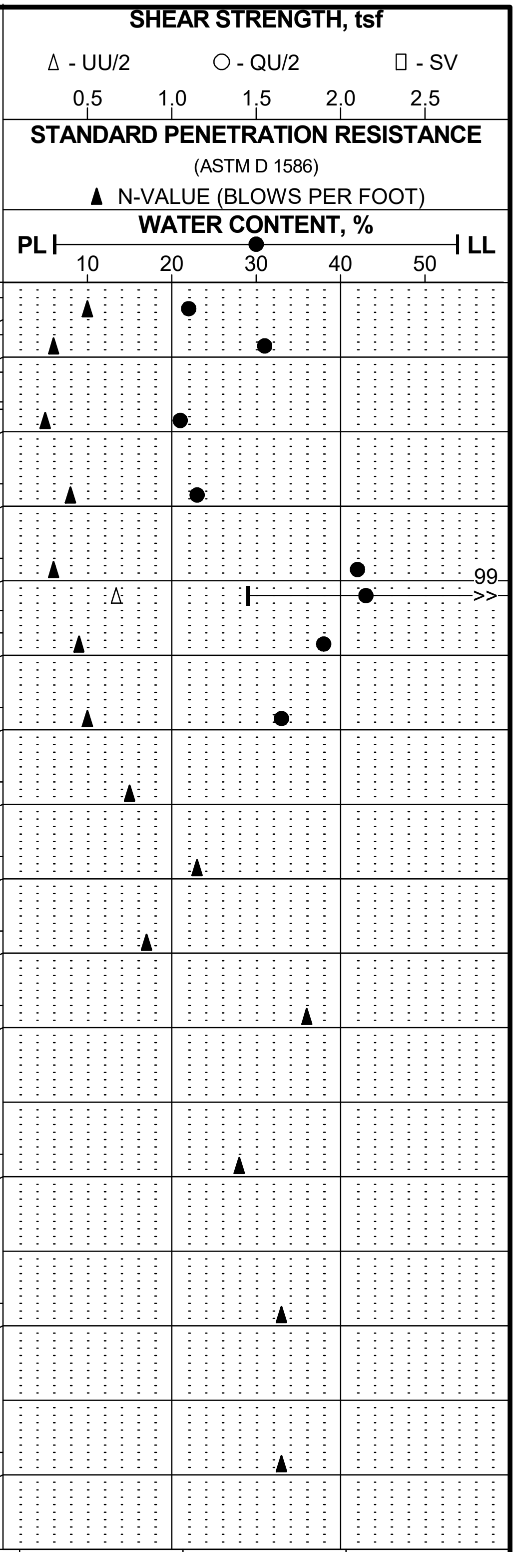
Datum MSL

Completion Date: 1/21/19

Station: 118+20

Offset: 6 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches		
		Stiff to medium stiff, red-brown SILT - ML	3-5-5	SS1
			2-3-3	SS2
5	141			
		Medium stiff, red-brown to gray, LEAN CLAY - CL		ST3
		Medim stiff to stiff, gray and brown, FAT CLAY - (CH)		
10	136		2-2-3	SS4
15	131		2-4-4	SS5
20	126		3-3-3	SS6
			82	ST7
25	121		3-4-5	SS8
30	116		3-5-5	SS9
35	111	Medium dense to dense, brown and gray, SILTY SAND - SM ∇ 16.0% passing No. 200 sieve pH = 8.37 Soil Resistivity: 6,840 ohms-cm	6-6-9	SS10
40	106		10-10-13	SS11
45	101		9-8-9	SS12
50	96		11-16-20	SS13
55	91			
60	86	10.6% passing No. 200 sieve	10-12-16	SS14
65	81			
70	76		17-17-16	SS15
75	71			
80	66		13-15-18	SS16
		Boring terminated at 80 feet.		



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 35 FEET ∇

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM

WASHBORING FROM 35 FEET

TPD DRILLER KLR LOGGER

CME 55LC DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

**ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas**

LOG OF BORING: B-11

**Geotechnology Project No.
J033659.01**

Surface Elevation: 147

Datum MSL

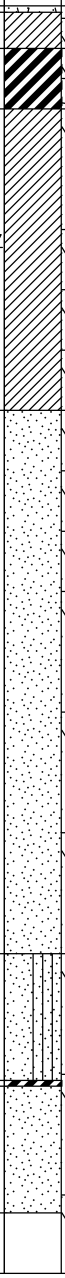
Completion Date: 1/21/19

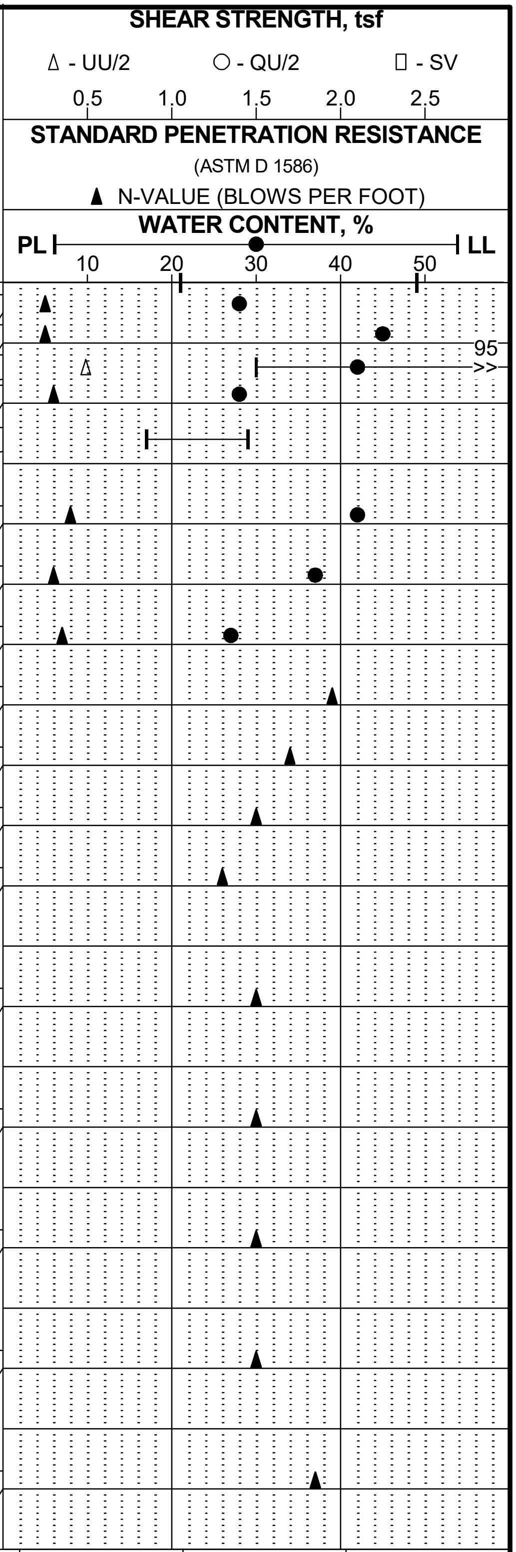
Station: 118+60

Offset: 3 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		TOPSOIL: 6 inches		
		Medium stiff, red-brown, LEAN CLAY, trace sand and roots - (CL) 95.0% passing No 200 sieve	1-2-3	SS1
5	142		1-2-3	SS2
			79	ST3
10	137	Medium stiff, red-brown to gray, FAT CLAY, trace roots - (CH)	1-2-4	SS4
		Medium stiff, gray to red-brown, LEAN CLAY - (CL)		ST5
15	132			
20	127	▽	3-3-5	SS6
25	122		3-3-3	SS7
30	117		3-3-4	SS8
35	112	Dense to medium dense, brown to gray SAND - SP	6-17-22	SS9
40	107		14-16-18	SS10
45	102		13-13-17	SS11
50	97	0.2% Gravel 4.2% passing No. 200 sieve	10-12-14	SS12
55	92			
60	87	pH = 7.72 Soil Resistivity: 2,508 ohms-cm	10-14-16	SS13
65	82			
70	77		13-14-16	SS14
75	72			
80	67	Medium dense, gray SAND WITH SILT, trace gravel - SP-SM 6.1% Gravel 5.9% passing No. 200 sieve	12-13-17	SS15
85	62			
90	57	Very stiff, gray, FAT CLAY - CH	13-14-16	SS16
		Dense, gray SAND, trace gravel - SP		
95	52			
100	47	Boring terminated at 100 feet.	18-15-22	SS17

GRAPHIC LOG





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 20 FEET ▽

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 35 FEET
TPD DRILLER KLR LOGGER
CME 55LC DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: B-12

Geotechnology Project No.
J033659.01

Surface Elevation: 159

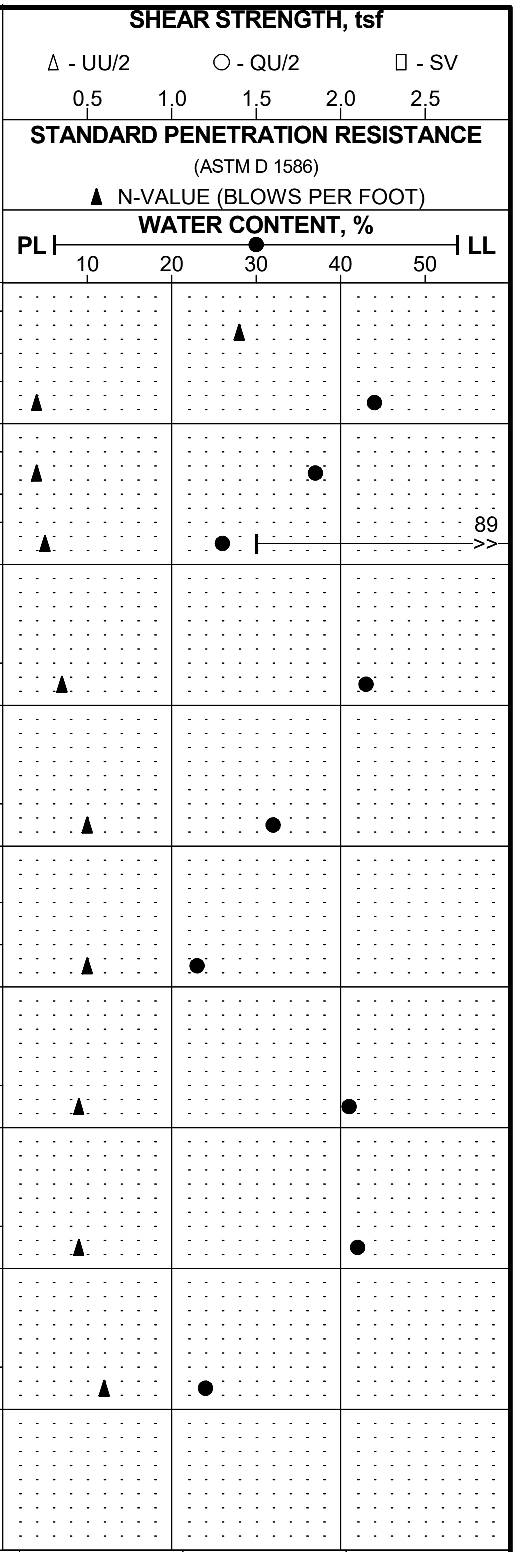
Datum MSL

Completion Date: 1/17/19

Station: 119+50

Offset: 40 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		ASPHALT: 9.5 inches			
		BASE: 3 inches of gravel		10-13-15	SS1
		FILL: red sand			
		Soft to medium stiff, red-brown and gray, FAT CLAY - (CH)		2-1-3	SS2
5	154				
				1-2-2	SS3
				2-2-3	SS4
10	149				
				3-2-5	SS5
15	144				
		Stiff, brown and gray, FAT CLAY - CH		3-4-6	SS6
20	139				
					ST7
		pH = 7.16			
		Soil Resistivity: 513 ohms-cm		3-5-5	SS8
25	134				
				3-4-5	SS9
30	129				
				2-4-5	SS10
35	124				
				4-5-7	SS11
40	119	Boring terminated at 40 feet.			



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 __ FEET

TPD DRILLER KLR LOGGER

CME 55LC DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

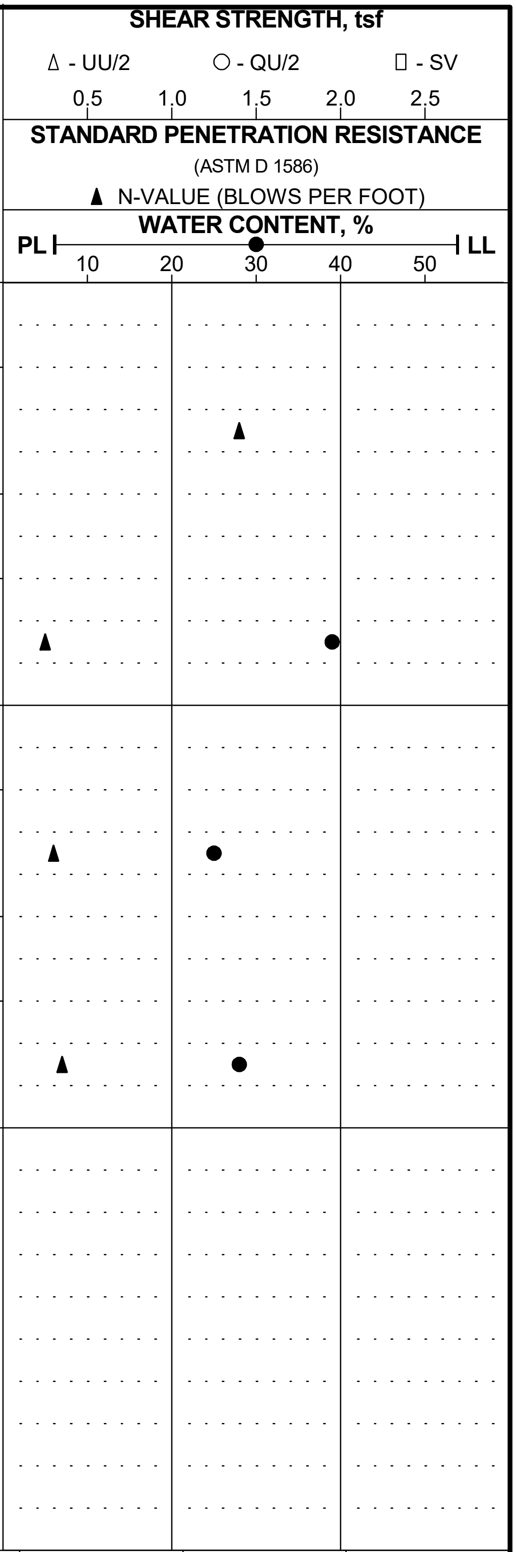
ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: B-13

Geotechnology Project No.
J033659.01

Surface Elevation: 152
Datum MSL
Completion Date: 1/17/19
Station: 120+75
Offset: 30 LT

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		ASPHALT: 6 inches		
		BASE: 6 inches of gravel		
		FILL: red sand	10-15-13	SS1
		Medium stiff, red-brown to gray, FAT CLAY - CH		
			2-2-3	SS2
5	147			
			2-3-3	SS3
			2-3-4	SS4
10	142	Boring terminated at 10 feet.		



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 __ FEET
TPD DRILLER KLR LOGGER
CME 55LC DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: B-14

Geotechnology Project No.
J033659.01

Surface Elevation: ______

Datum MSL

Completion Date: 1/17/19

GRAPHIC LOG

DRY UNIT WEIGHT (pcf)
SPT BLOW COUNTS
CORE RECOVERY/RQD

SAMPLES

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

DEPTH IN FEET	ELEVATION IN FEET	**DESCRIPTION OF MATERIAL**
		ASPHALT: 11 inches Boring terminated at 11 inches.
5		
10		
15		
20		
25		
30		
35		
40		
45		
50		
55		
60		
65		
70		
75		
80		
85		
90		
95		
100		

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 __ FEET

TPD DRILLER KLR LOGGER

CME 55LC DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: C- 1

Geotechnology Project No.
J033659.01

Surface Elevation: ______

Datum MSL

Completion Date: 1/17/19

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES	SHEAR STRENGTH, tsf ∆ - 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
		ASPHALT: 8 inches Boring terminated at 8 inches				
5						
10						
15						
20						
25						
30						
35						
40						
45						
50						
55						
60						
65						
70						
75						
80						
85						
90						
95						
100						

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 __ FEET

TPD DRILLER KLR LOGGER

CME 55LC DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: ALY	Checked by: DMS	App'vd. by: ASE
Date: 1/24/19	Date: 3/12/19	Date: 3/13/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas

LOG OF BORING: C- 2

Geotechnology Project No.
J033659.01

APPENDIX D – LABORATORY TEST DATA

Atterberg Limits

Grain Size Distributions

Unconsolidated-Undrained Triaxial Compression

Direct Shear

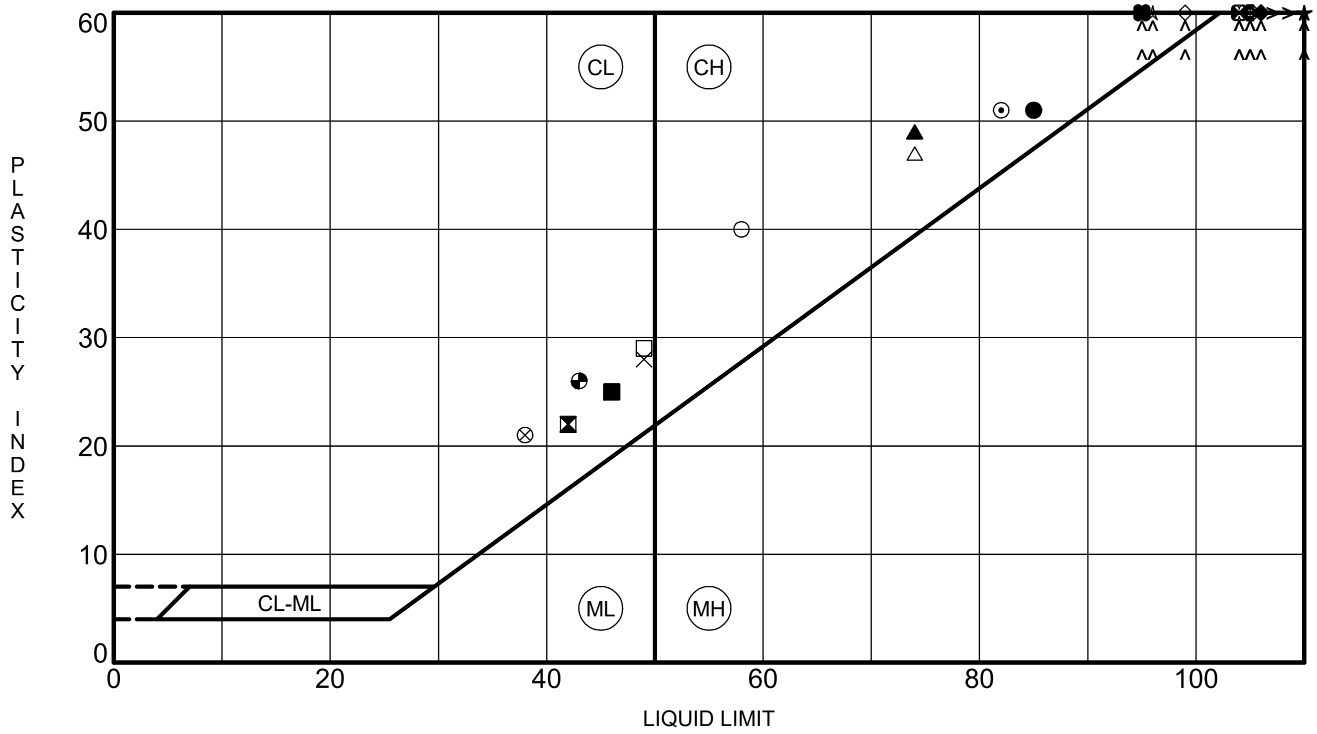
Consolidation

Resistivity

pH

Standard Proctor Curves

CBR Results

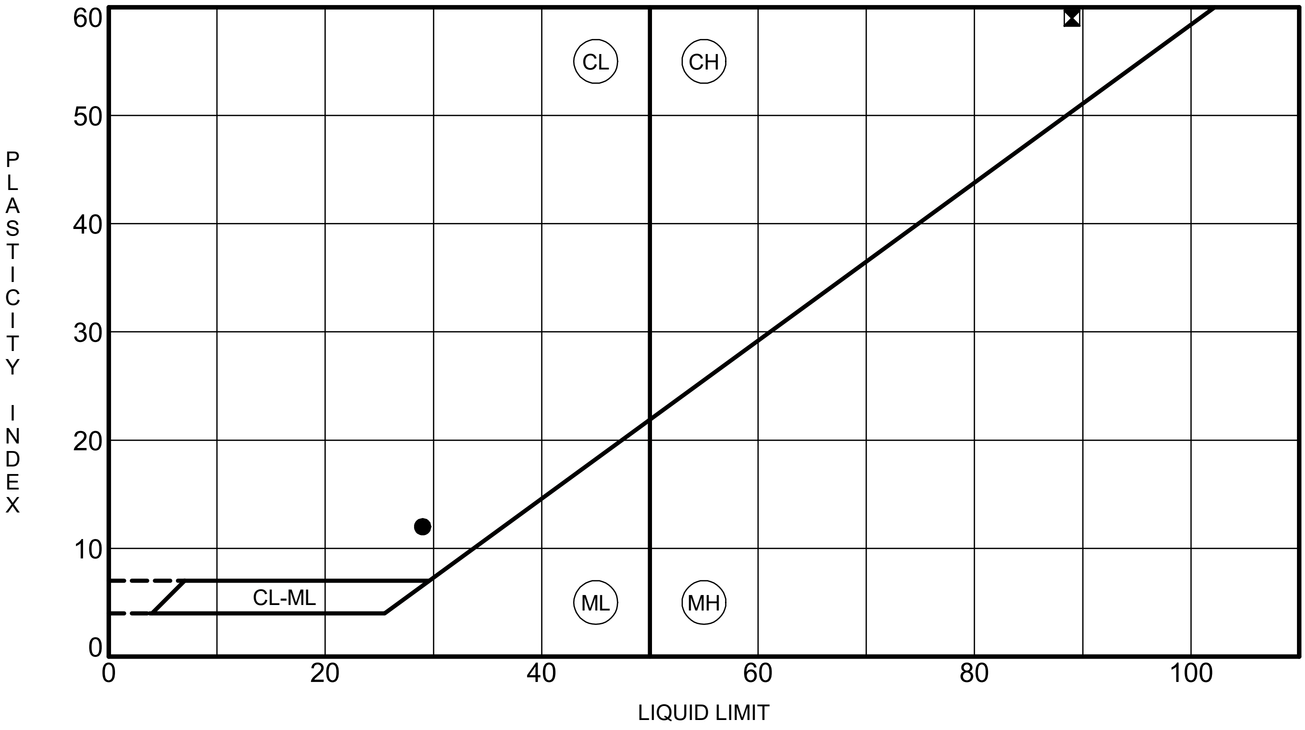


	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B- 1**	**3.5**	**85**	**34**	**51**		**FAT CLAY(CH), A-7-5**
⧗	**B- 2**	**0.0**	**42**	**20**	**22**	**75**	**LEAN CLAY(CL), A-7-6(15)**
▲	**B- 2**	**10.0**	**74**	**25**	**49**		**FAT CLAY(CH), A-7-6**
★	**B- 2**	**33.5**	**111**	**40**	**71**		**FAT CLAY(CH), A-7-5**
⊙	**B- 3**	**10.0**	**82**	**31**	**51**		**FAT CLAY(CH), A-7-5**
✿	**B- 4**	**8.0**	**105**	**32**	**73**	**65**	**FAT CLAY(CH), A-7-5(47)**
○	**B- 5**	**6.0**	**58**	**18**	**40**		**FAT CLAY(CH), A-7-6**
△	**B- 5**	**23.5**	**74**	**27**	**47**		**FAT CLAY(CH), A-7-6**
⊗	**B- 6**	**3.0**	**38**	**17**	**21**		**LEAN CLAY(CL), A-6**
⊕	**B- 6**	**15.0**	**105**	**42**	**63**		**FAT CLAY(CH), A-7-5**
□	**B- 6**	**33.5**	**49**	**20**	**29**	**75**	**LEAN CLAY(CL), A-7-6**
⊛	**B- 7**	**20.0**	**104**	**32**	**72**		**FAT CLAY(CH), A-7-5**
◕	**B- 8**	**8.0**	**43**	**17**	**26**		**LEAN CLAY(CL), A-7-6**
☆	**B- 9**	**10.0**	**96**	**32**	**64**		**FAT CLAY(CH), A-7-5**
✣	**B- 9**	**15.0**	**104**	**37**	**67**		**FAT CLAY(CH), A-7-5**
■	**B-10**	**6.0**	**46**	**21**	**25**		**LEAN CLAY(CL), A-7-6**
◆	**B-10**	**15.0**	**106**	**33**	**73**		**FAT CLAY(CH), A-7-5**
◇	**B-11**	**20.0**	**99**	**29**	**70**		**FAT CLAY(CH), A-7-6**
×	**B-12**	**0.0**	**49**	**21**	**28**	**95**	**LEAN CLAY(CL), A-7-6(29)**
✖	**B-12**	**6.0**	**95**	**30**	**65**		**FAT CLAY(CH), A-7-5**

GEOTECHNOLOGY INC
FROM THE GROUND UP

ATTERBERG LIMITS RESULTS

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas
J033659.01

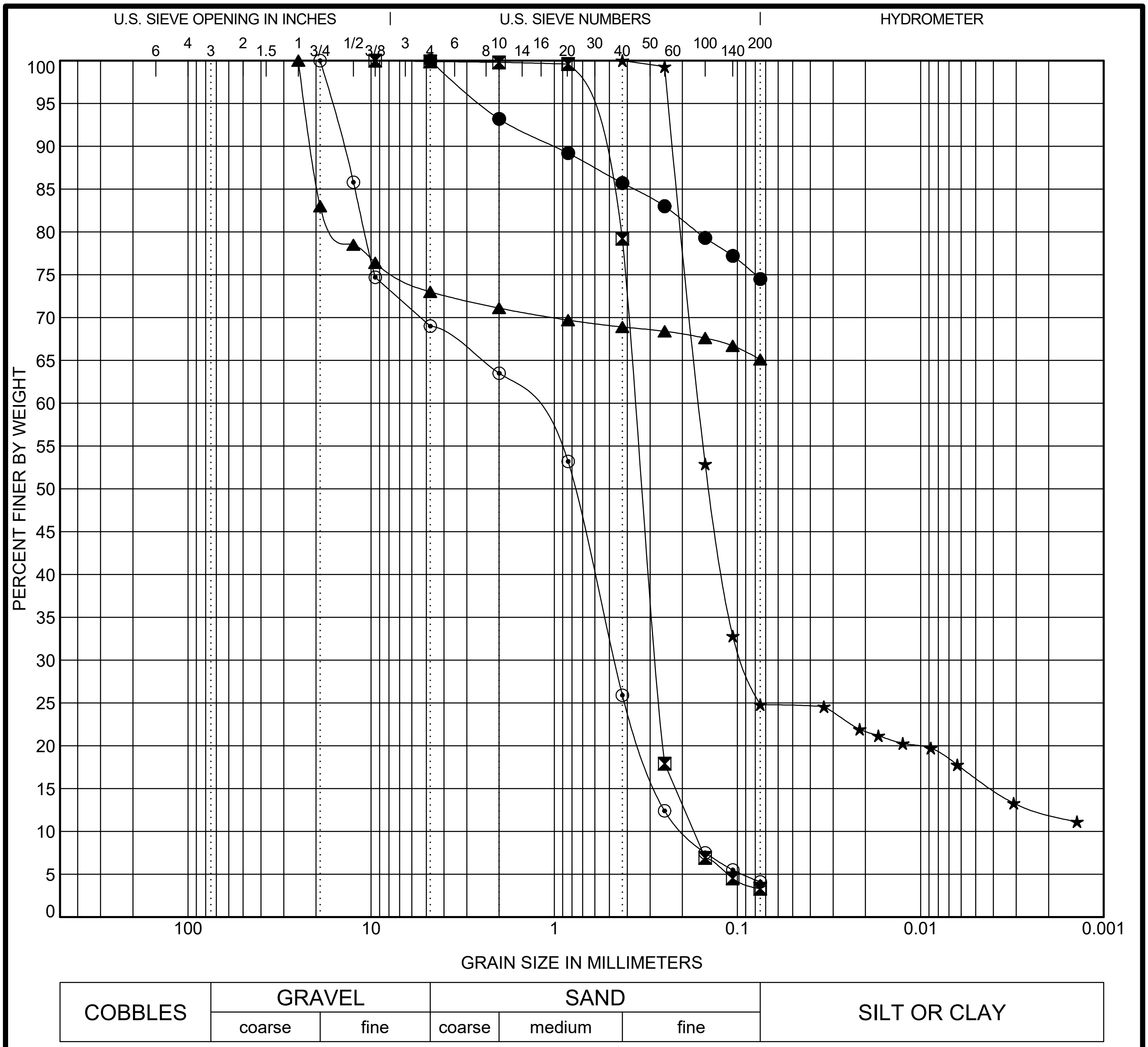


	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-12**	**12.0**	**29**	**17**	**12**		**LEAN CLAY(CL), A-6**
☒	**B-13**	**8.5**	**89**	**30**	**59**		**FAT CLAY(CH), A-7-5**

GEOTECHNOLOGY INC
FROM THE GROUND UP

ATTERBERG LIMITS RESULTS

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas
J033659.01



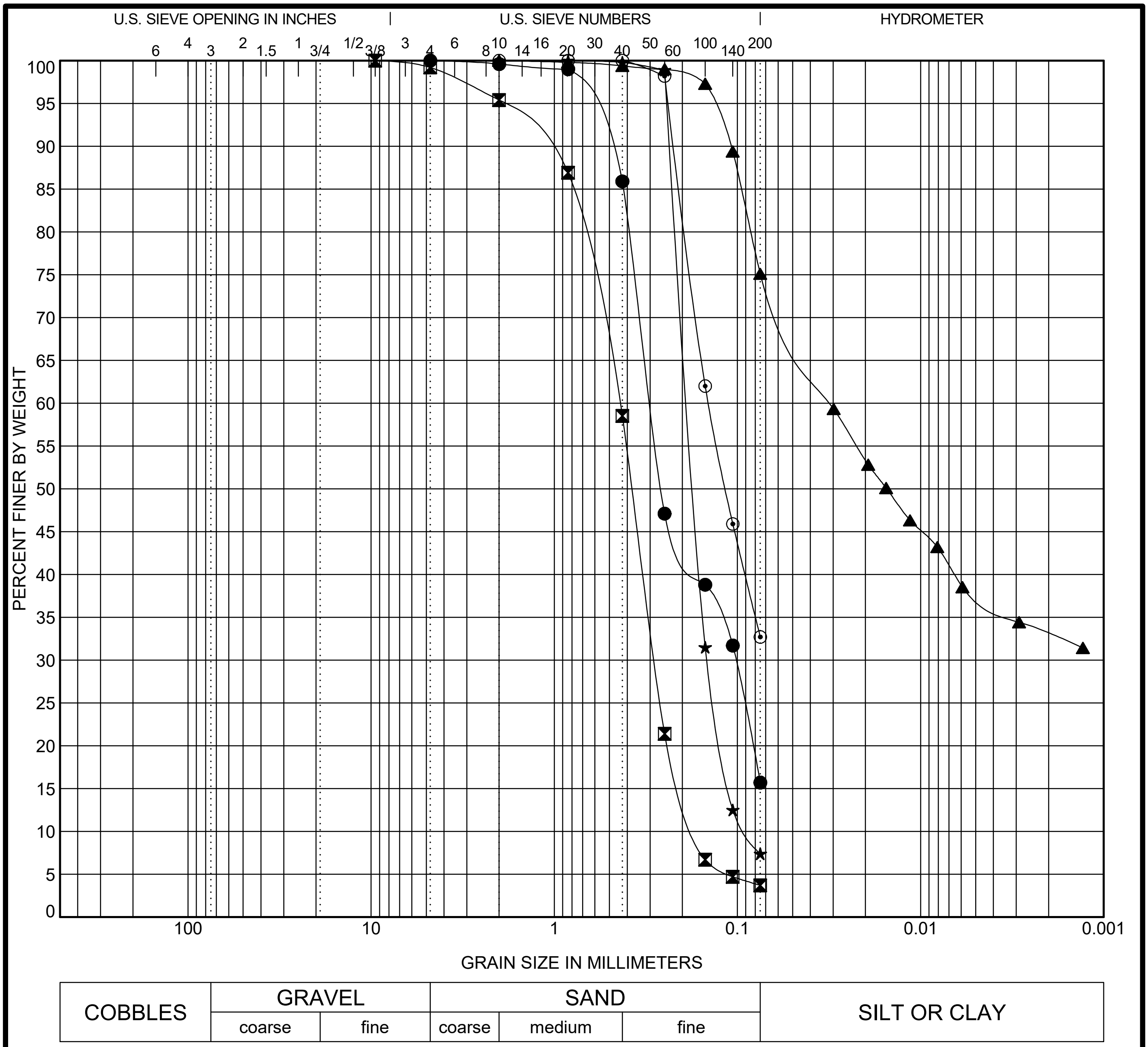
Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**B- 2**	**0.0**	**LEAN CLAY(CL), A-7-6(15)**	**42**	**20**	**22**		
☒	**B- 3**	**53.5**	**POORLY GRADED SAND(SP), A-2-6**				**1.24**	**2.08**
▲	**B- 4**	**8.0**	**FAT CLAY(CH), A-7-5(47)**	**105**	**32**	**73**		
★	**B- 4**	**33.5**	**FAT CLAY(CH), A-7-5**					
⊙	**B- 4**	**58.5**	**POORLY GRADED SAND(SP), A-2-6**				**0.76**	**7.65**

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B- 2**	**0.0**	**4.75**				**0.0**	**25.5**	**74.5**	
☒	**B- 3**	**53.5**	**9.5**	**0.36**	**0.278**	**0.173**	**0.1**	**96.6**	**3.3**	
▲	**B- 4**	**8.0**	**25**				**27.0**	**7.9**	**65.1**	
★	**B- 4**	**33.5**	**0.425**	**0.162**	**0.094**		**0.0**	**75.2**	**8.5**	**16.3**
⊙	**B- 4**	**58.5**	**19**	**1.489**	**0.471**	**0.195**	**31.0**	**64.9**	**4.1**	

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas
J033659.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	B- 4	68.5	CLAYEY SAND(SC), A-2-6					
⧗	B- 5	53.5	POORLY GRADED SAND(SP), A-2-6				1.08	2.62
▲	B- 6	33.5	LEAN CLAY(CL), A-7-6	49	20	29		
★	B- 6	88.5	SAND WITH CLAY(SP-SC), A-2-6				1.28	2.08
⊙	B- 7	38.5	CLAYEY SAND(SC), A-2-6					

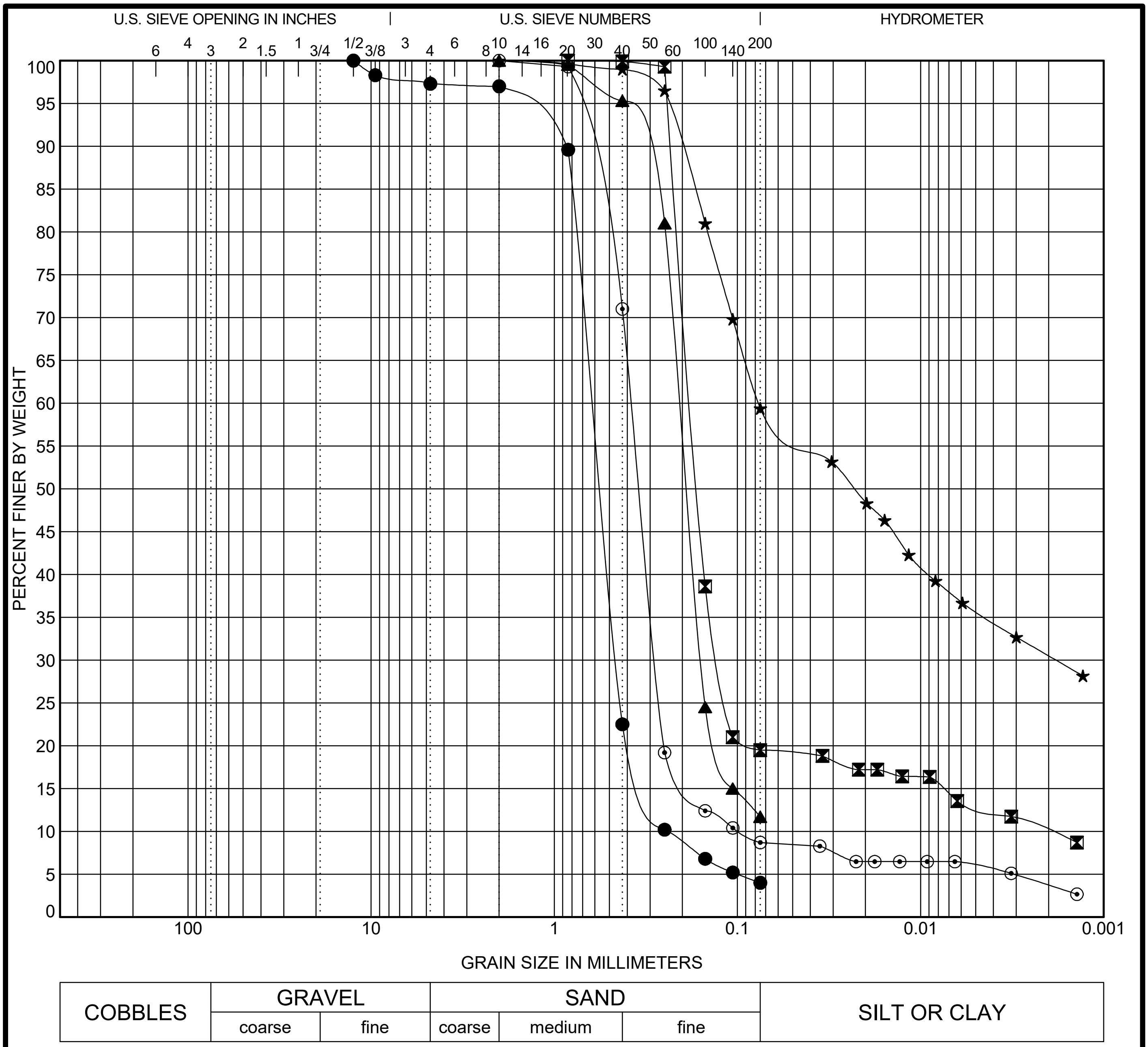
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B- 4	68.5	4.75	0.298	0.102		0.0	84.3	15.7	
⧗	B- 5	53.5	9.5	0.441	0.283	0.168	0.8	95.5	3.7	
▲	B- 6	33.5	2	0.031			0.0	24.9	37.5	37.6
★	B- 6	88.5	2	0.186	0.146	0.089	0.0	92.6	7.4	
⊙	B- 7	38.5	2	0.144			0.0	67.3	32.7	

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas
J033659.01

US GRAIN SIZE J033659.01.GPJ US LAB.GDT 3/21/19



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	B- 7	68.5	**POORLY GRADED SAND(SP), A-2-6**				1.39	2.56
⧗	B- 8	33.5	**SILTY SAND(SM), A-2-4**				44.66	89.88
▲	B- 8	48.5	**SAND WITH SILT(SP-SM), A-2-4**				1.92	3.29
★	B- 9	33.5	**FAT CLAY(CH), A-7-5**					
⊙	B- 9	78.5	**SAND WITH SILT(SP-SM), A-3**				2.10	3.89

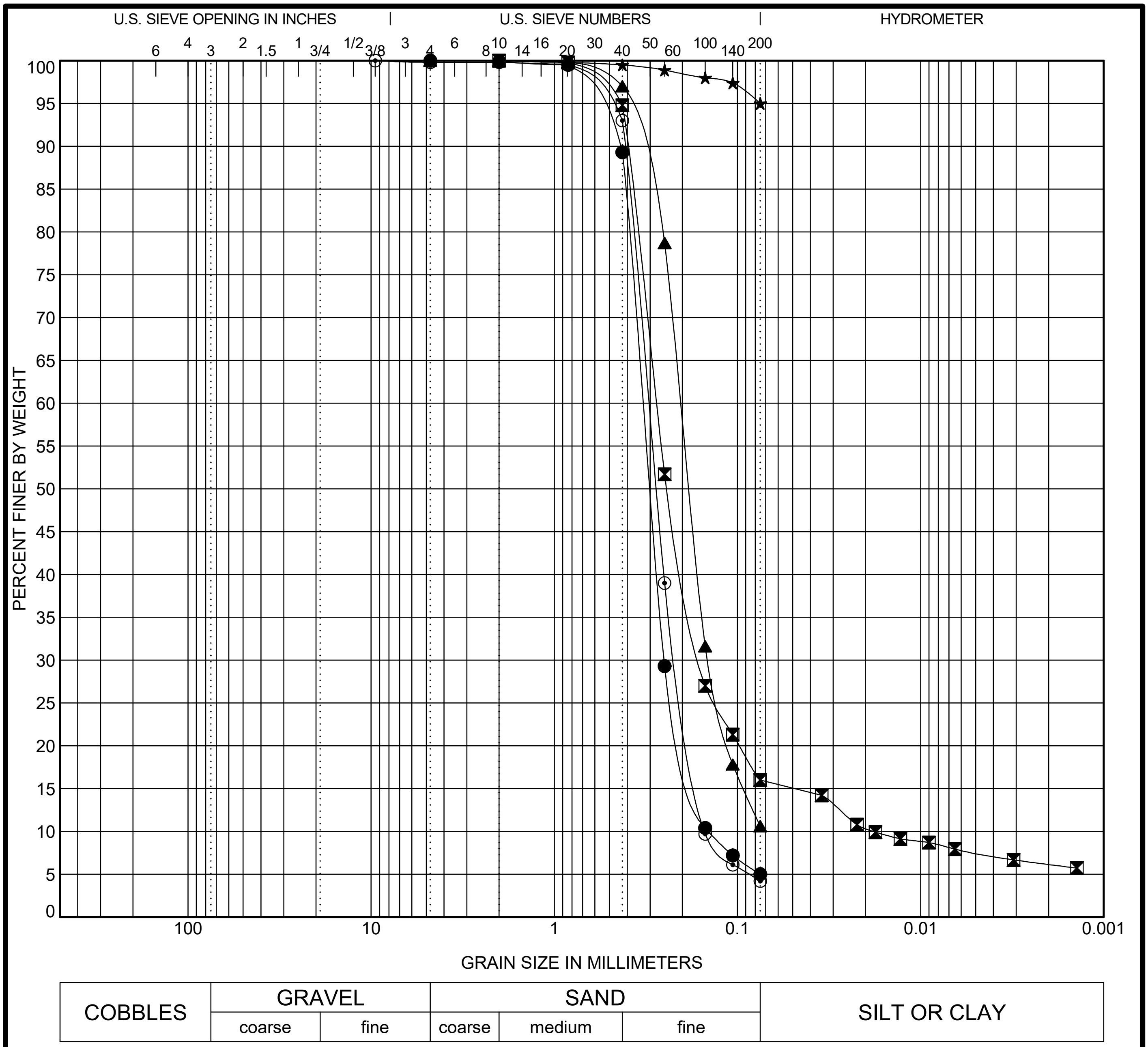
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B- 7	68.5	12.5	0.622	0.459	0.243	2.7	93.3	4.0	
⧗	B- 8	33.5	0.84	0.18	0.127	0.002	0.0	80.5	6.6	12.9
▲	B- 8	48.5	2	0.207	0.158		0.0	88.3	11.7	
★	B- 9	33.5	2	0.077	0.002		0.0	40.6	23.7	35.7
⊙	B- 9	78.5	2	0.38	0.279	0.098	0.0	91.3	2.7	6.0

GEOTECHNOLOGY INC

FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas
J033659.01



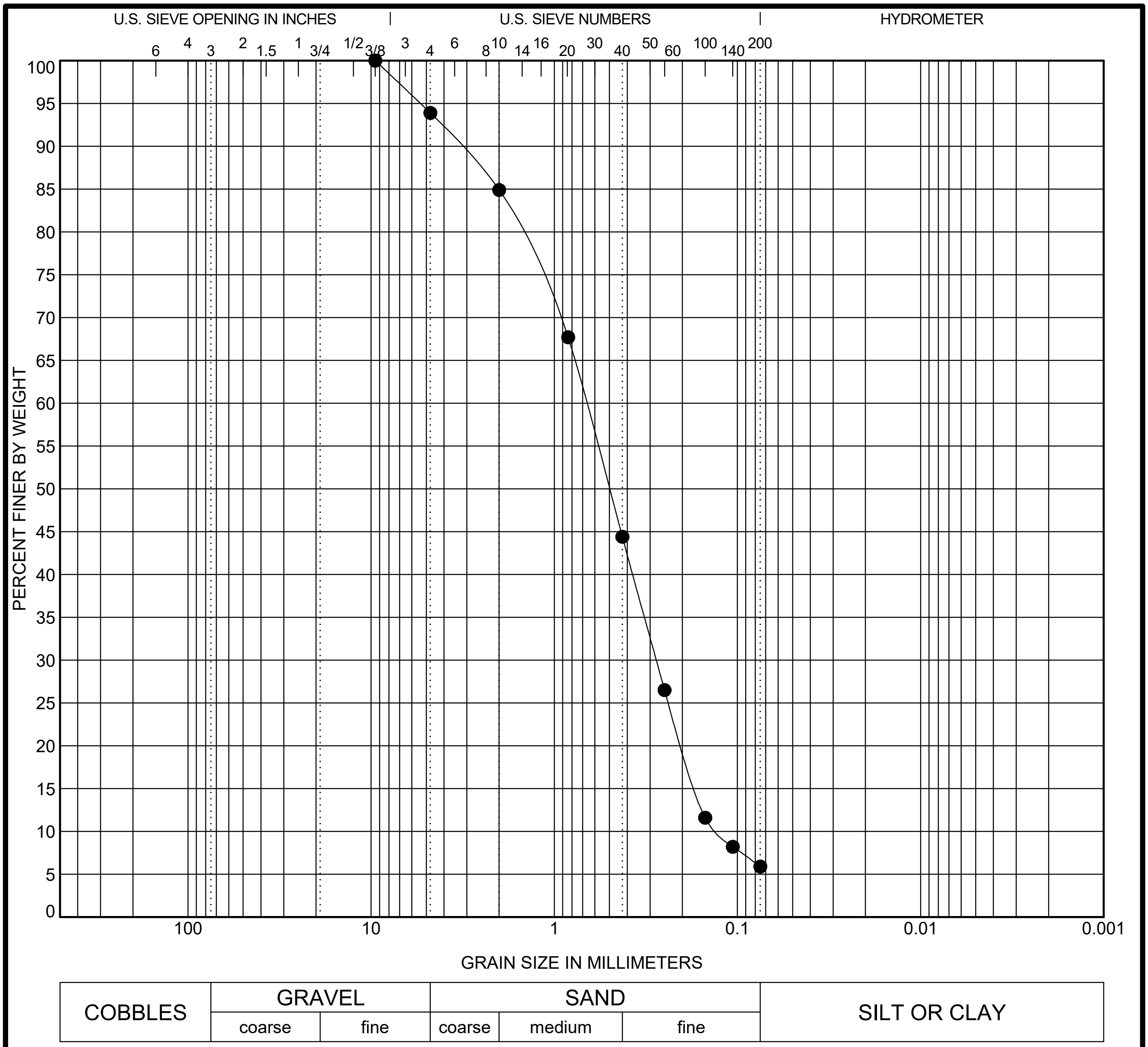
Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	B-10	43.5	SAND WITH SILT(SP-SM), A-3				1.34	2.28
⧗	B-11	33.5	SILTY SAND(SM), A-2-4				5.12	15.41
▲	B-11	58.5	SAND WITH SILT(SP-SM), A-2-4				1.40	2.80
★	B-12	0.0	LEAN CLAY(CL), A-7-6(29)	49	21	28		
⊙	B-12	48.5	POORLY GRADED SAND(SP), A-2-6				0.99	2.04

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-10	43.5	4.75	0.328	0.252	0.144	0.0	95.0	5.0	
⧗	B-11	33.5	2	0.277	0.16	0.018	0.0	84.0	8.5	7.5
▲	B-11	58.5	4.75	0.204	0.144		0.0	89.4	10.6	
★	B-12	0.0	2				0.0	5.0	95.0	
⊙	B-12	48.5	9.5	0.307	0.214	0.151	0.2	95.6	4.2	

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas
J033659.01



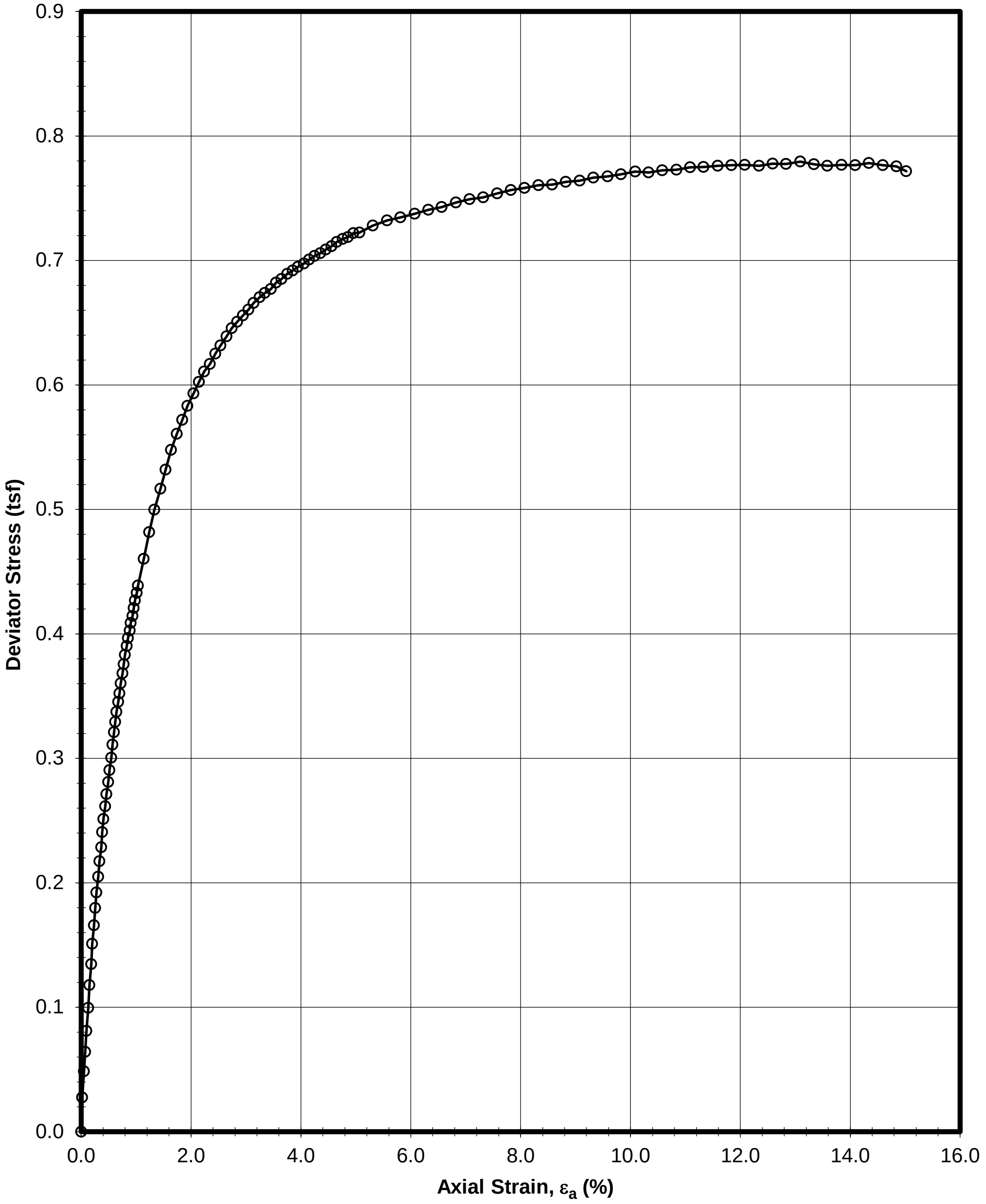
Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**B-12**	**78.5**	**SAND WITH SILT(SP-SM), A-1-b**				**0.90**	**5.26**

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-12**	**78.5**	**9.5**	**0.671**	**0.277**	**0.127**	**6.1**	**88.0**	**5.9**	

GEOTECHNOLOGY INC
FROM THE GROUND UP

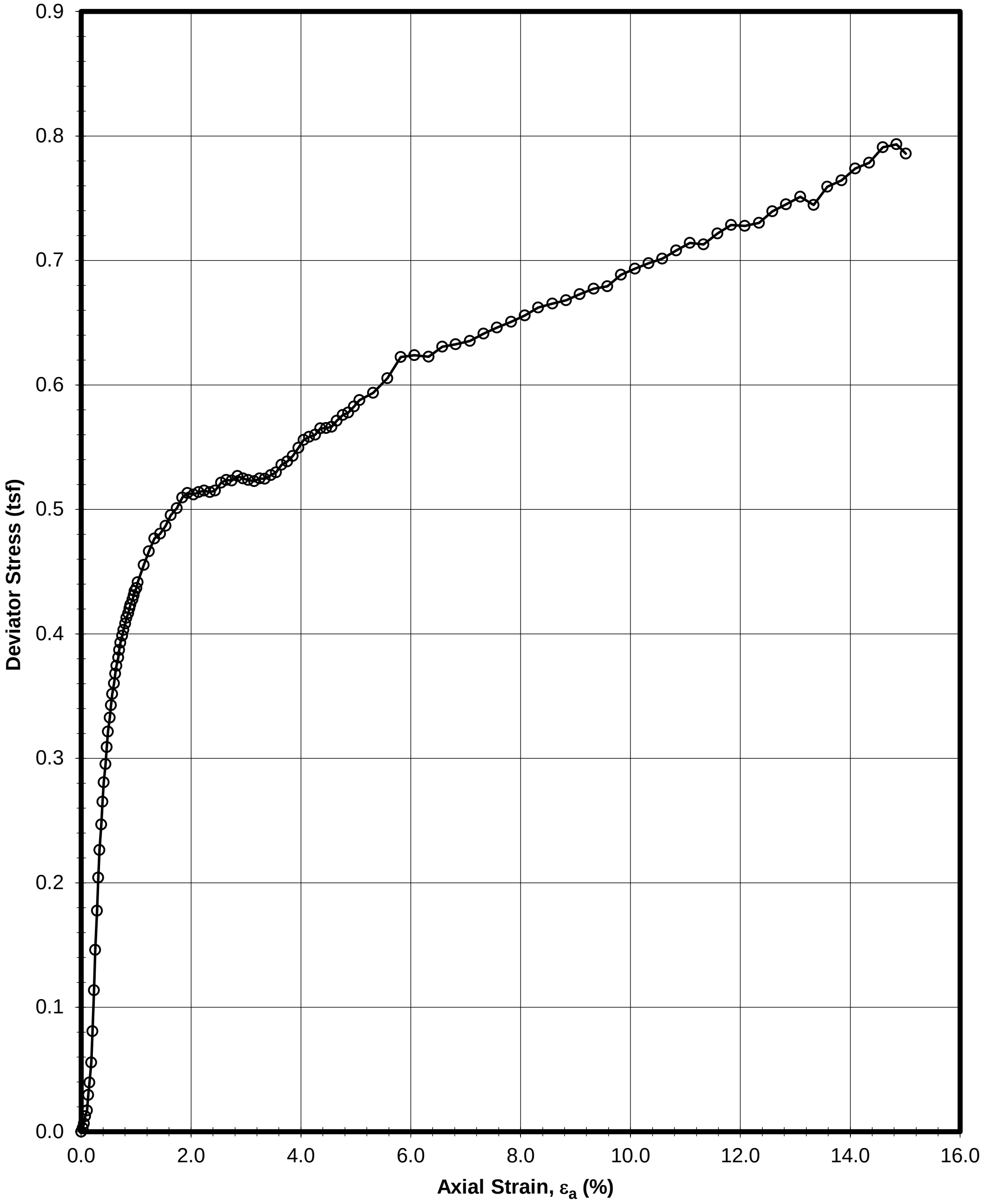
GRAIN SIZE DISTRIBUTION

ARDOT Hwy 278 Bridge
over Union Pacific Railroad
Desha County, Arkansas
J033659.01



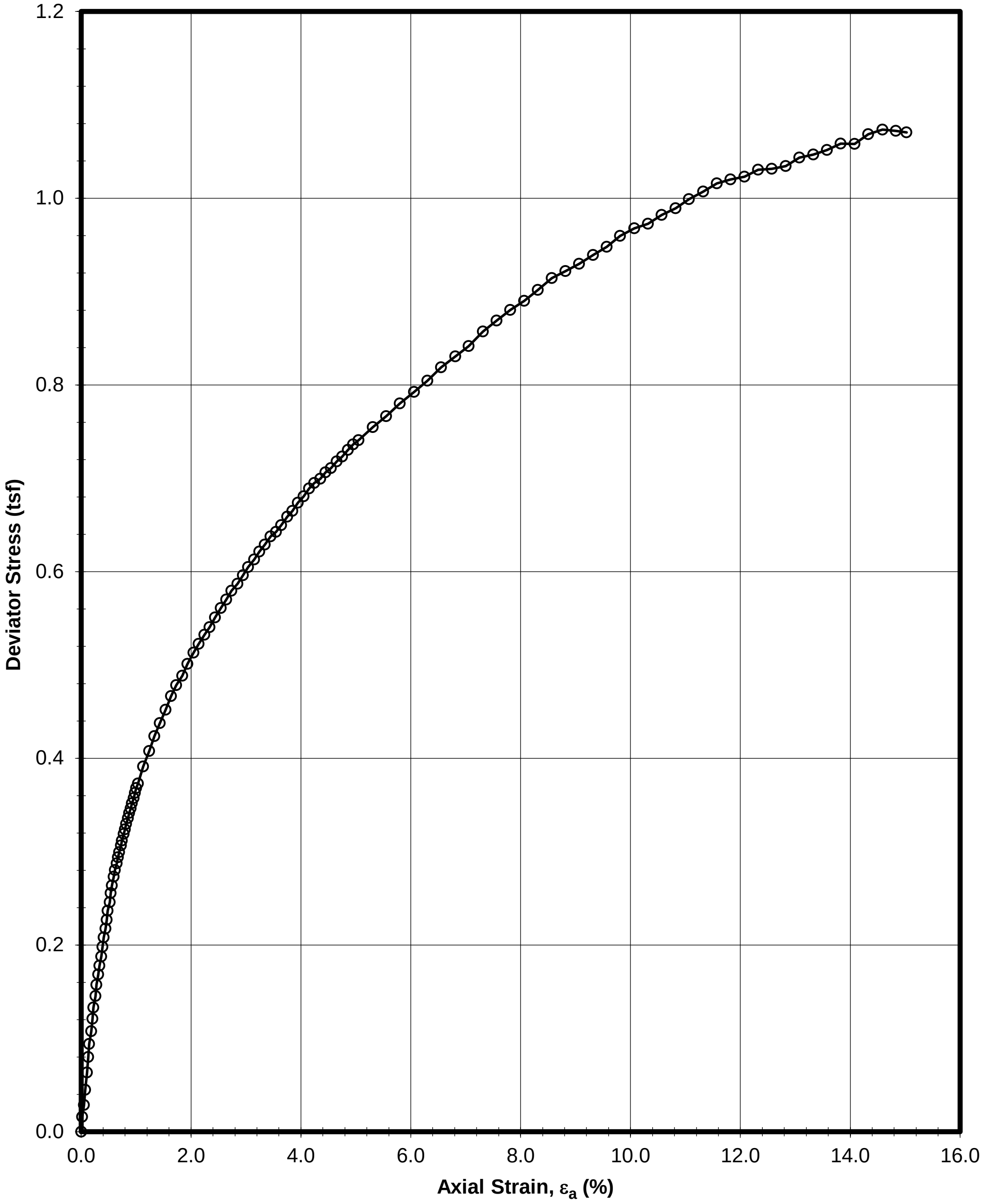
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J033659.01
Boring: B-2
Sample: ST-5 - Depth: 10 ft.



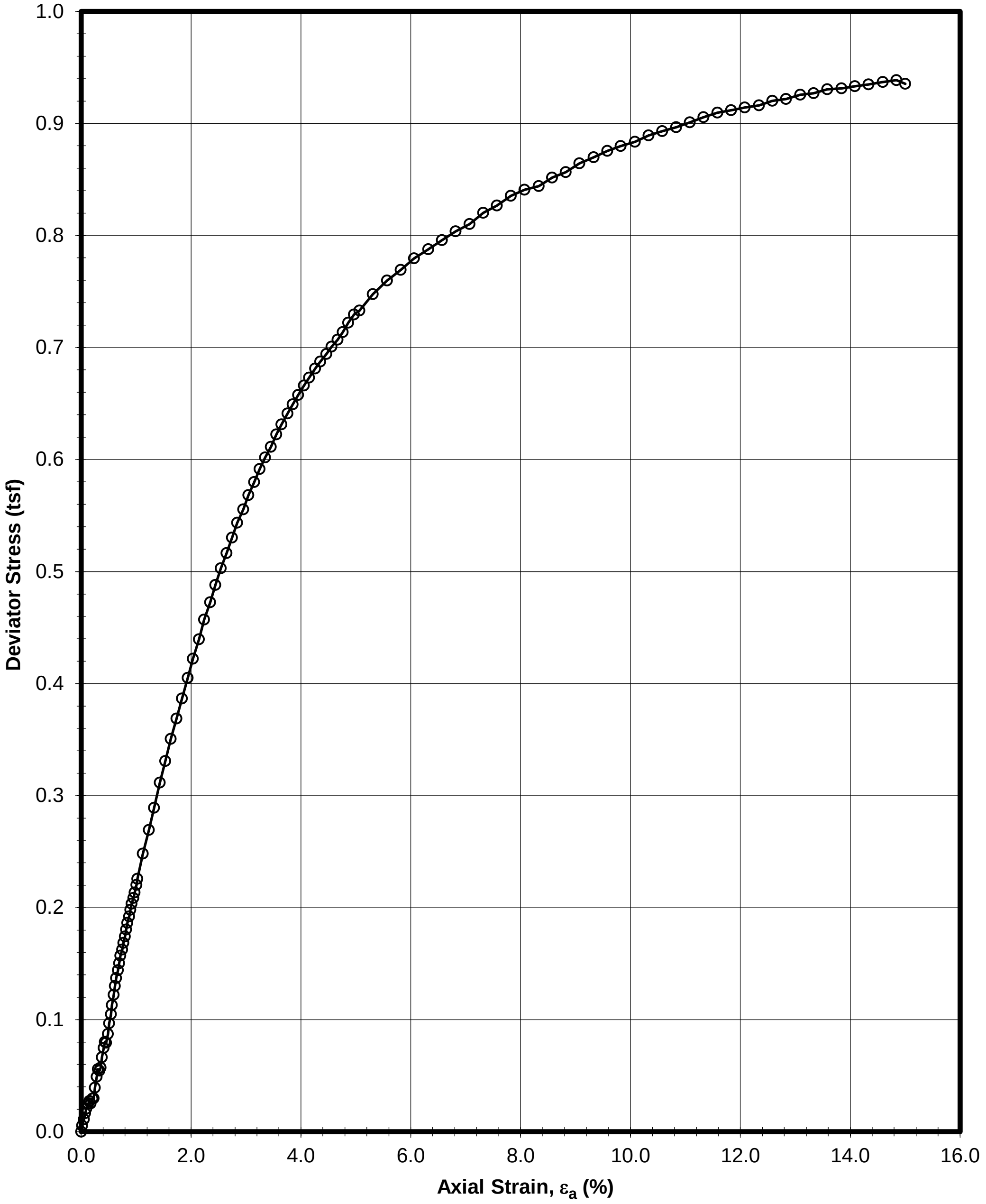
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J033659.01
Boring: B-4
Sample: ST-4 - Depth: 8 ft.



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J033659.01
Boring: B-5
Sample: ST-3 - Depth: 6 ft.



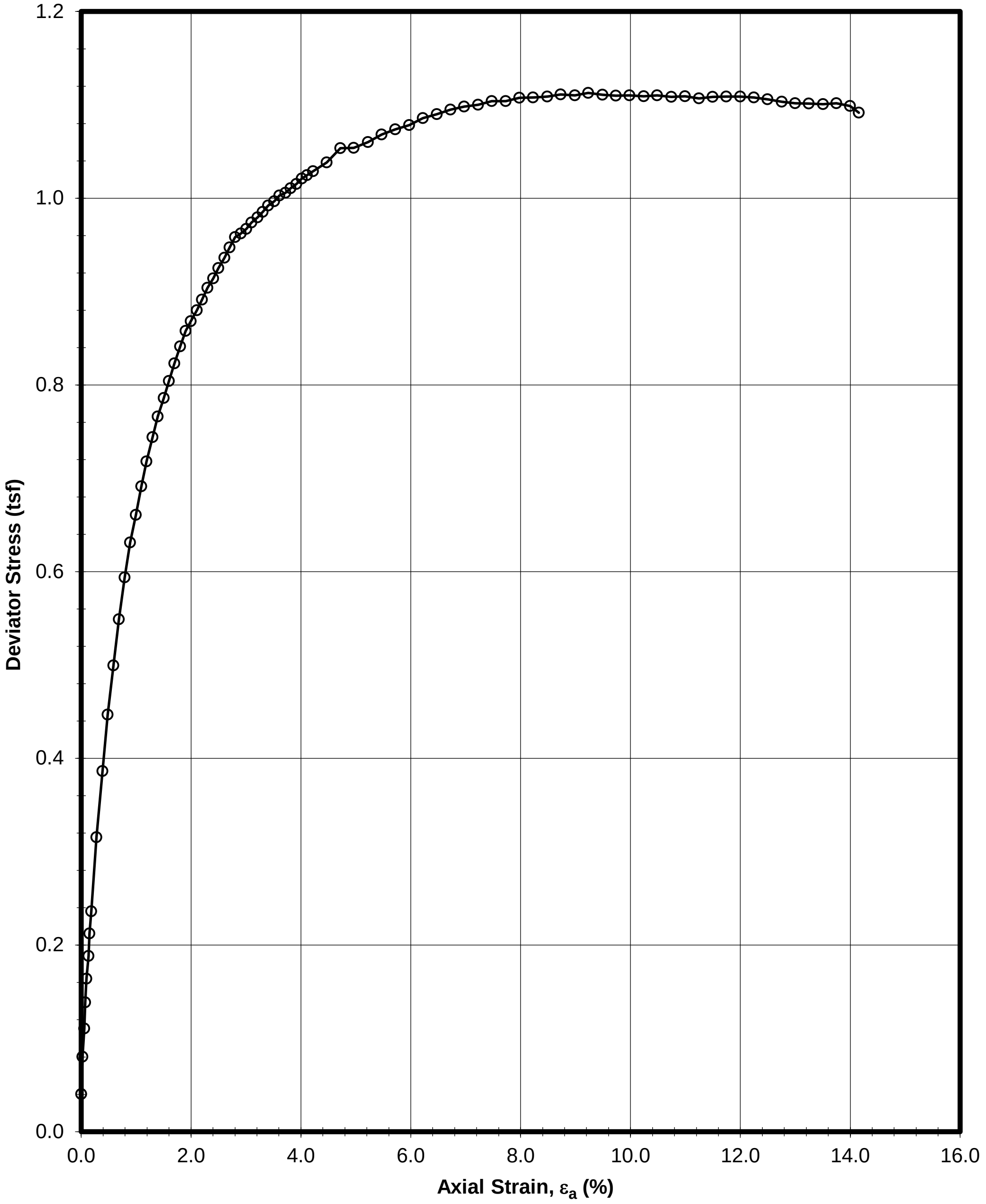
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J033659.01

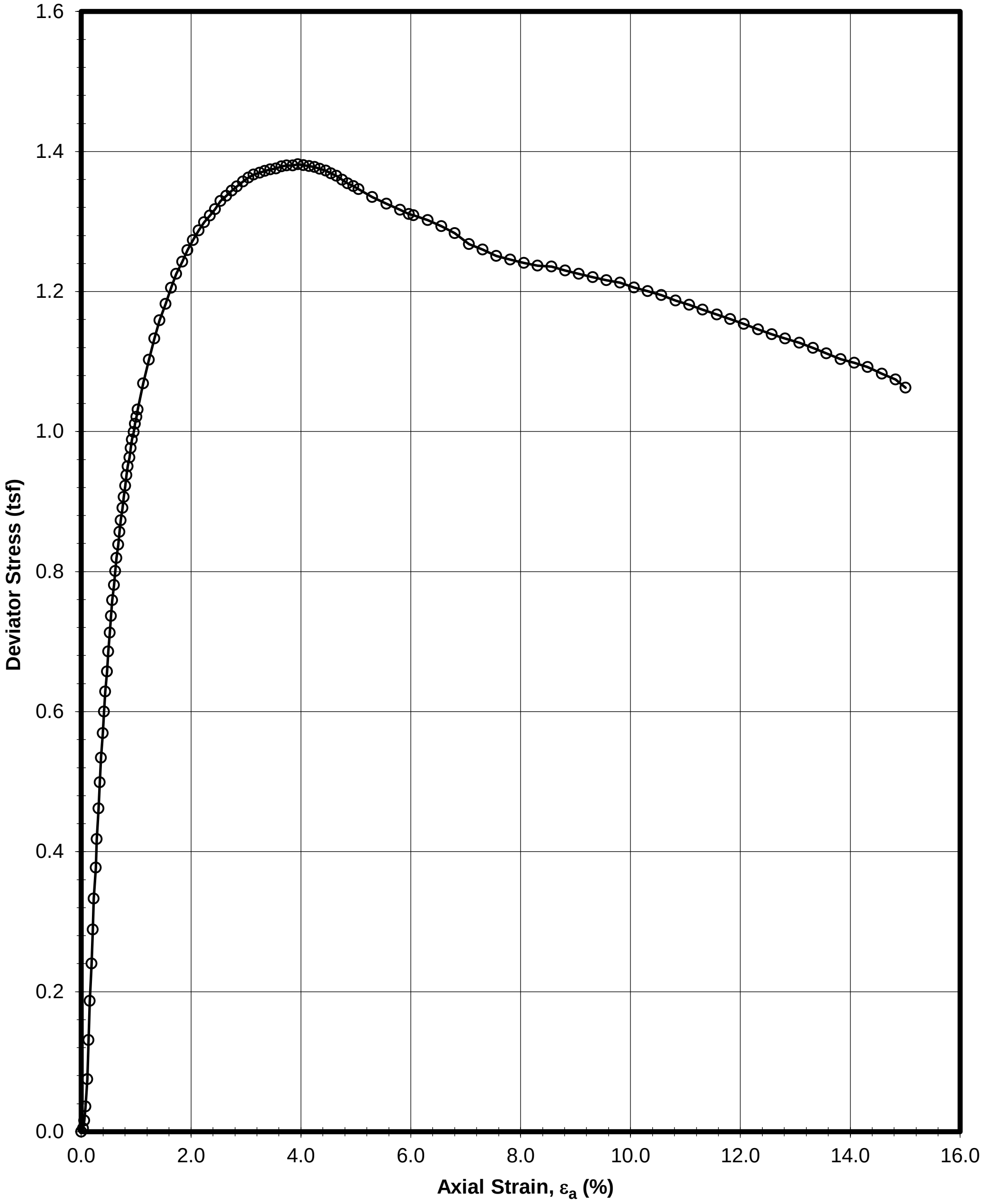
Boring: B-6

Sample: ST-2 - Depth: 3 ft.



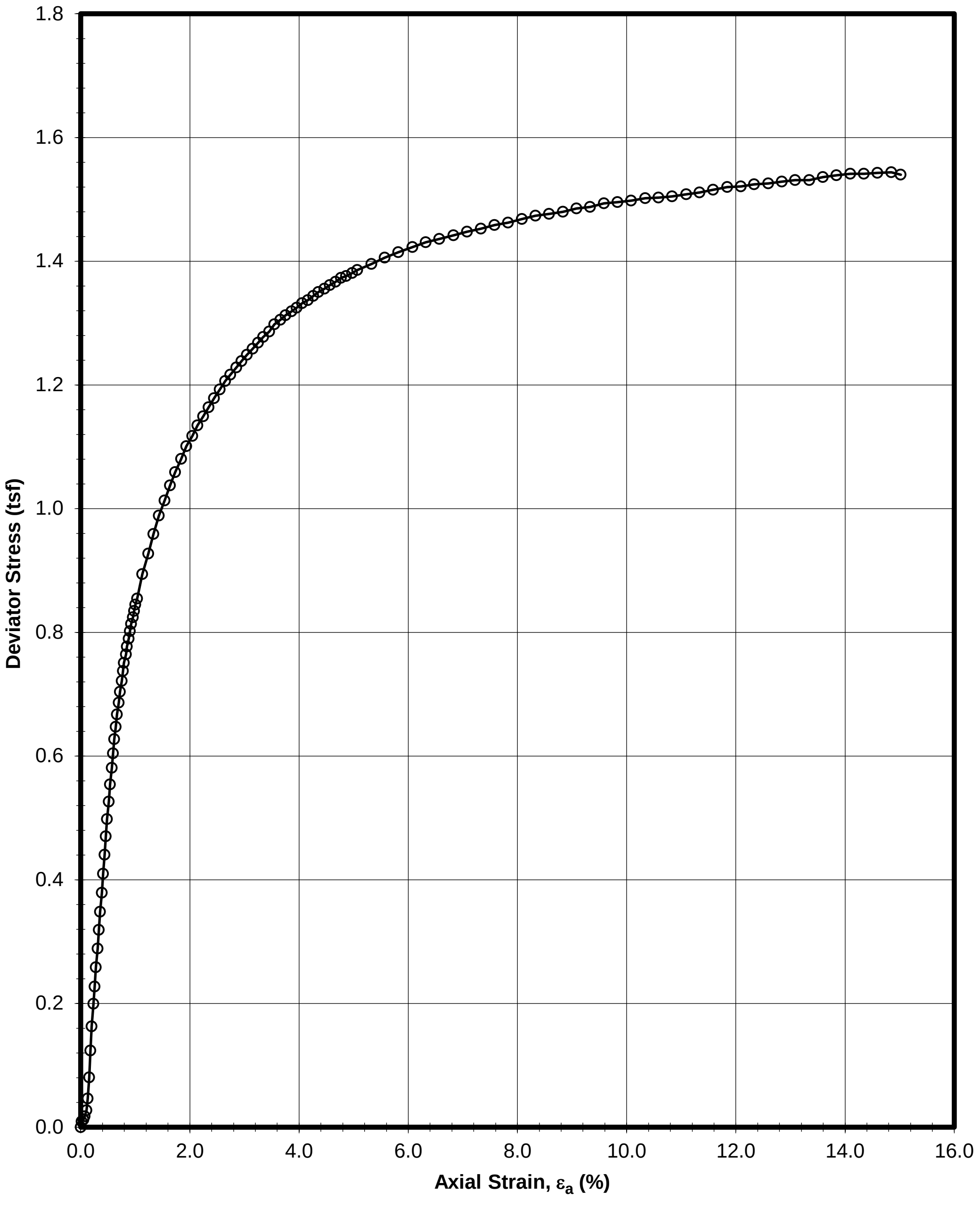
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J033659.01
Boring: B-6
Sample: ST-6 - Depth: 15 ft.



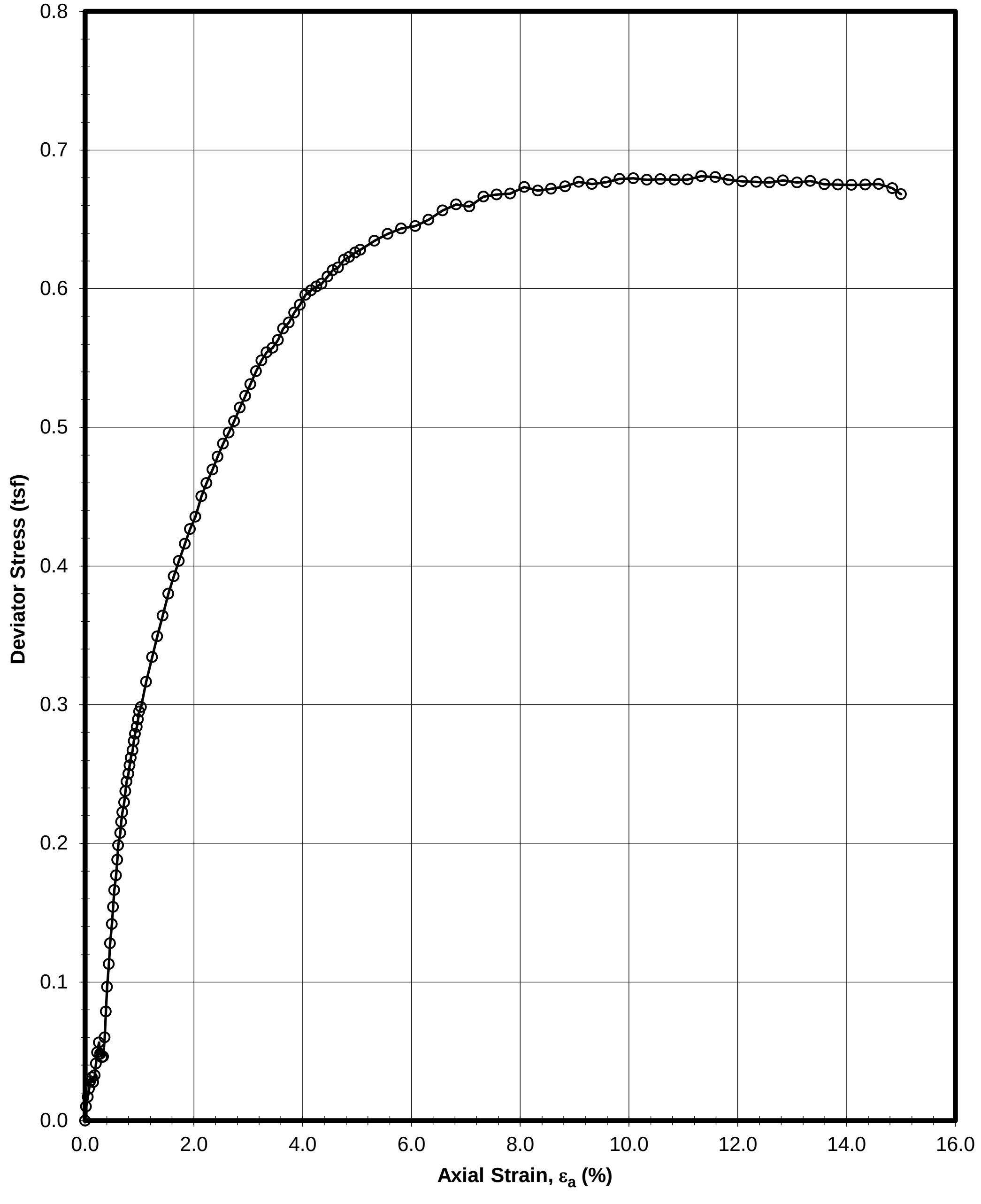
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J033659.01
Boring: B-7
Sample: ST-7 - Depth: 20 ft.



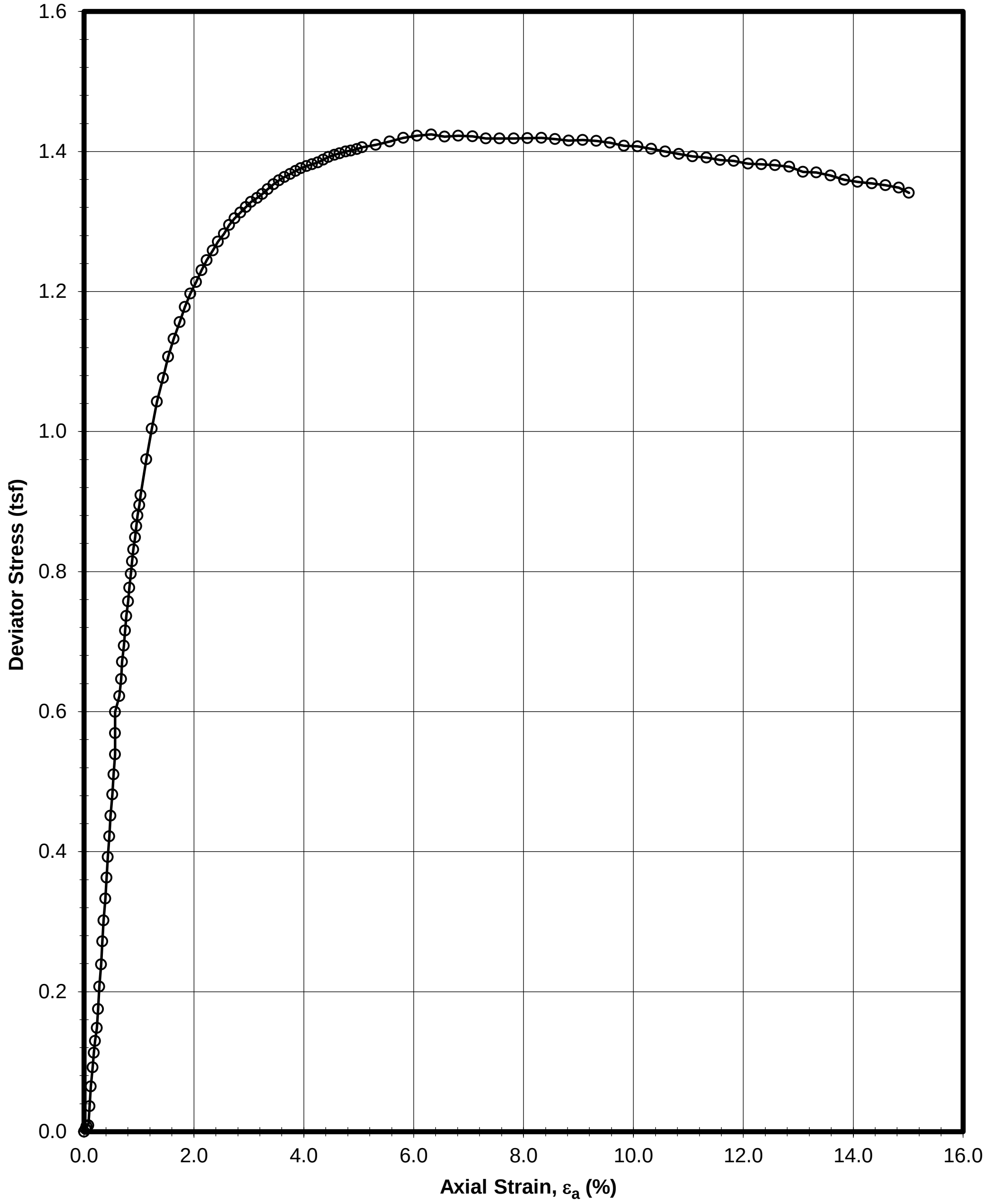
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J033659.01
Boring: B-8
Sample: ST-4 - Depth: 8 ft.



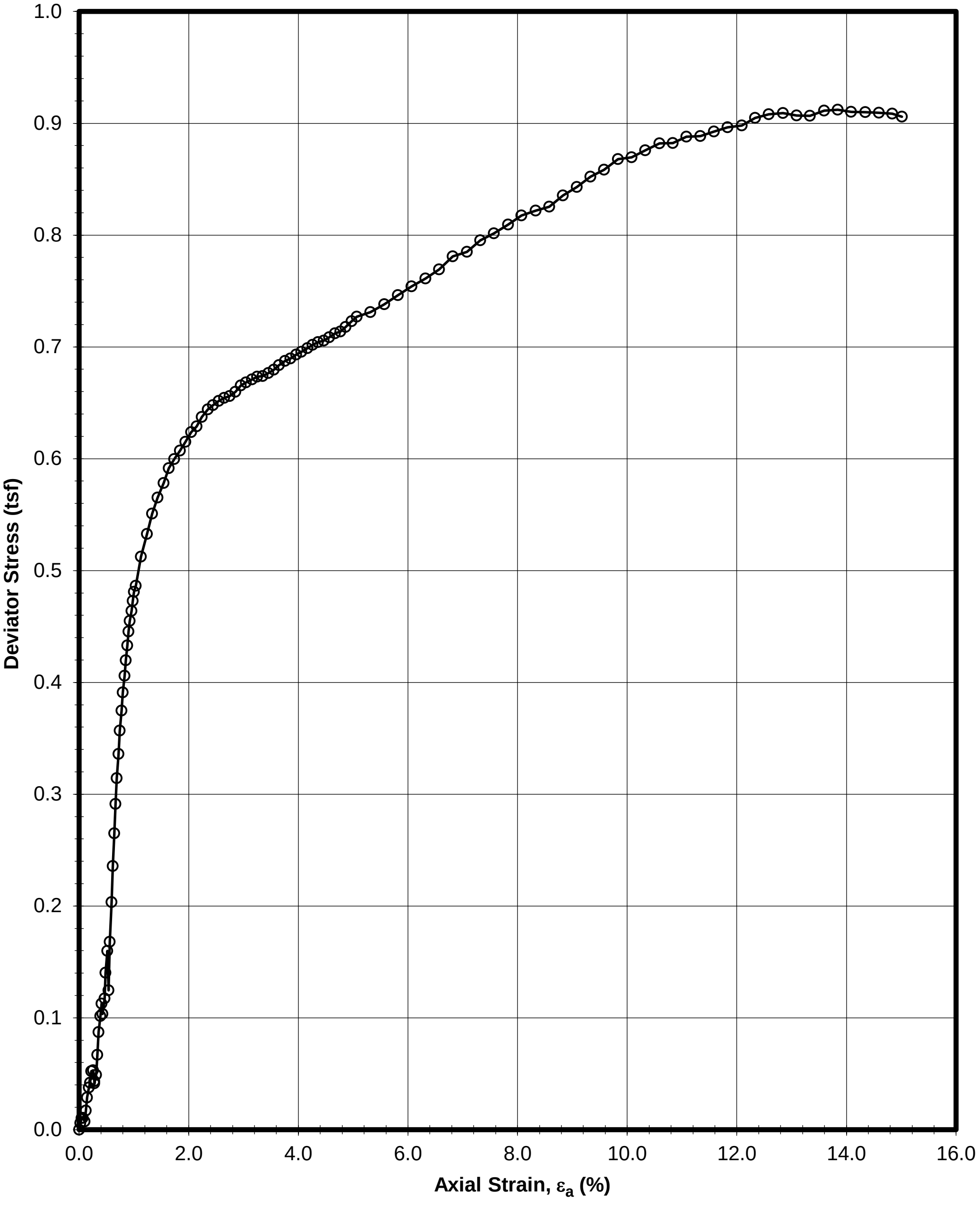
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J033659.01
Boring: B-9
Sample: ST-5 - Depth: 10 ft.



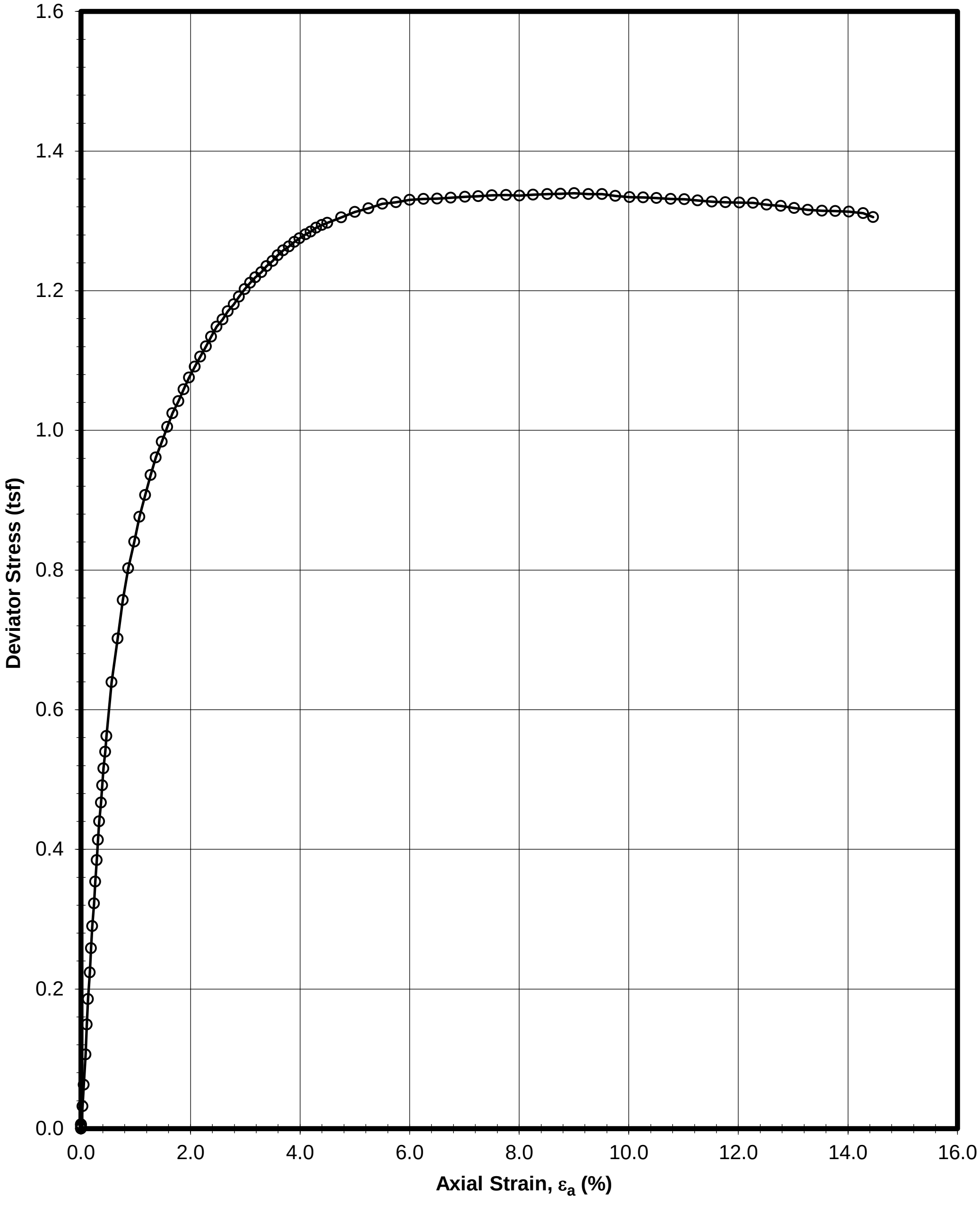
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J033659.01
Boring: B-9
Sample: ST-7 - Depth: 15 ft.



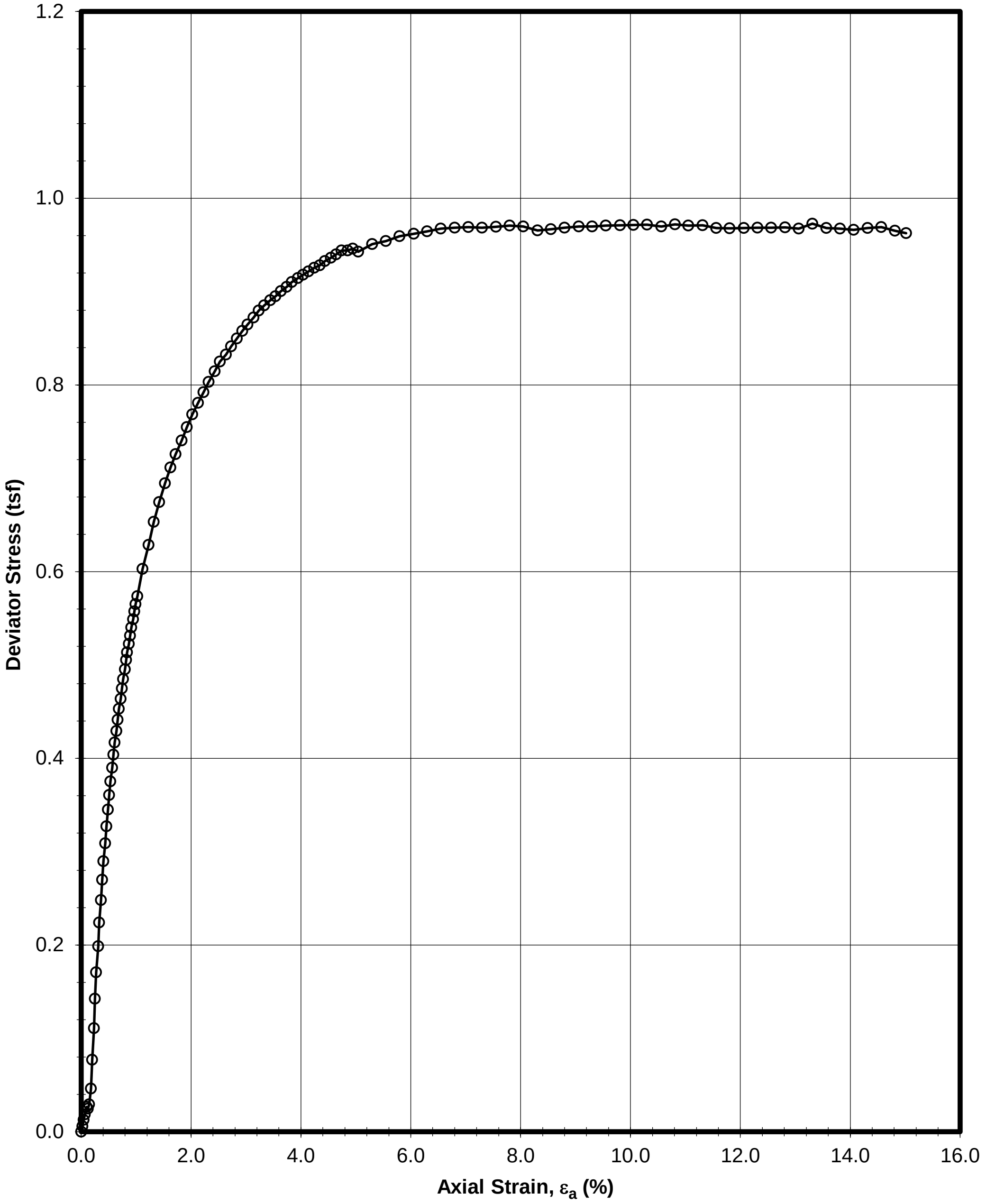
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J033659.01
Boring: B-10
Sample: ST-6 - Depth: 15 ft.



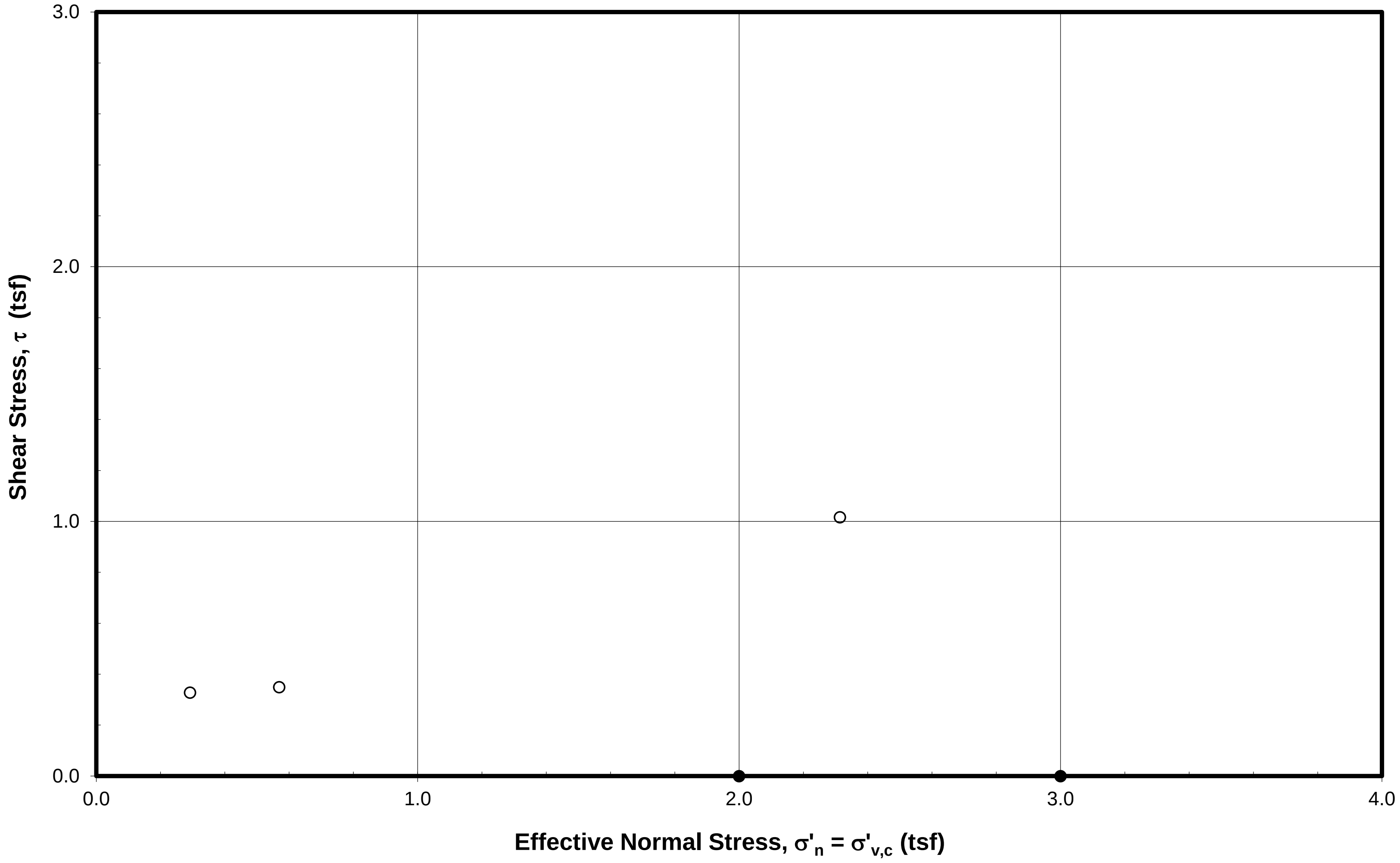
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J033659.01
Boring: B-11
Sample: ST-7 - Depth: 20 ft.



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

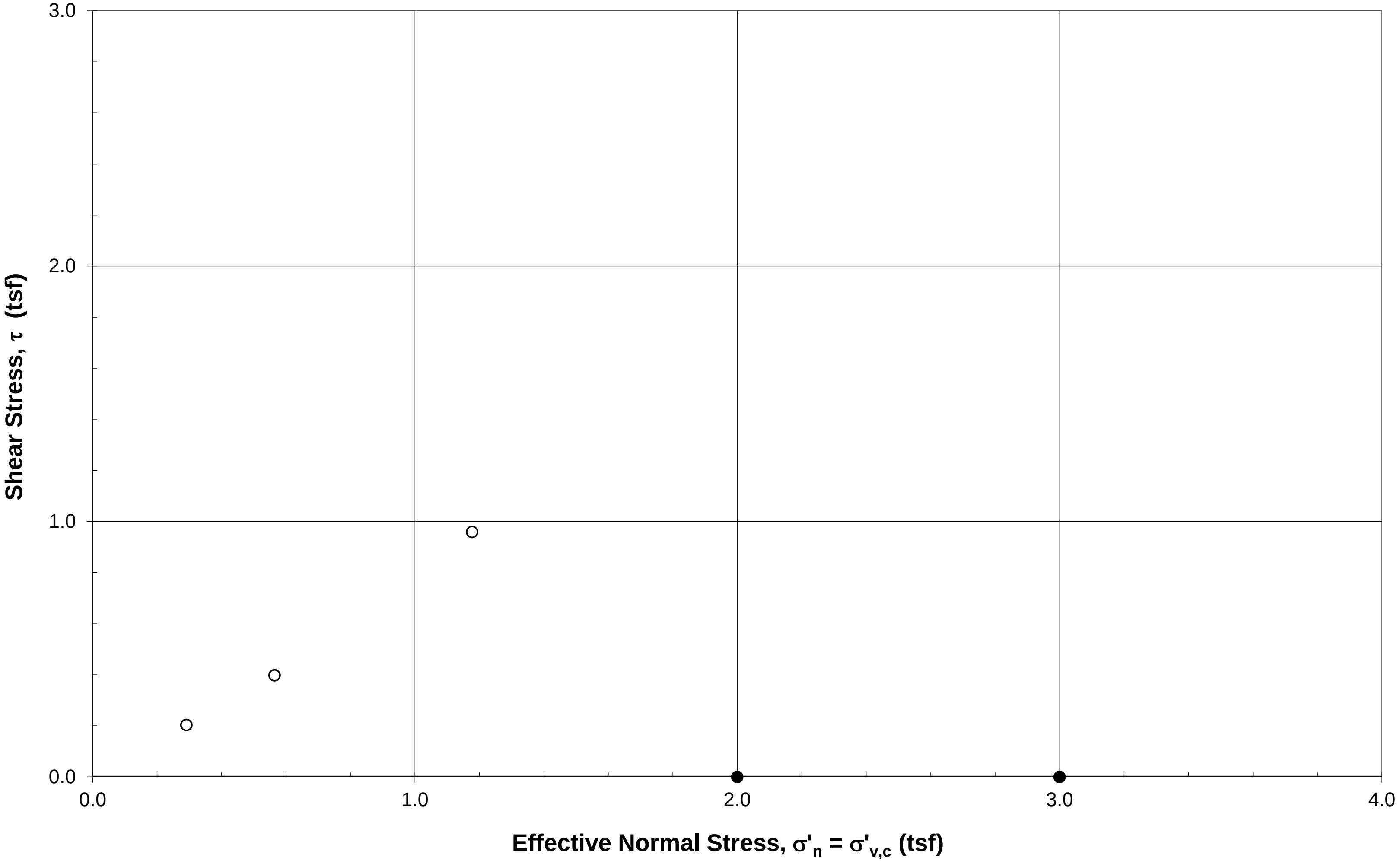
ASTM D 2850
Project No.: J033659.01
Boring: B-12
Sample: ST-3 - Depth: 6 ft.



DRAINED DIRECT SHEAR TEST

ASTM D 3080

Boring: B-5 Sample: ST-3 -Depth: 6.0ft



DRAINED DIRECT SHEAR TEST

ASTM D 3080

Boring: B-12 Sample: ST-6 -Depth: 12.0ft

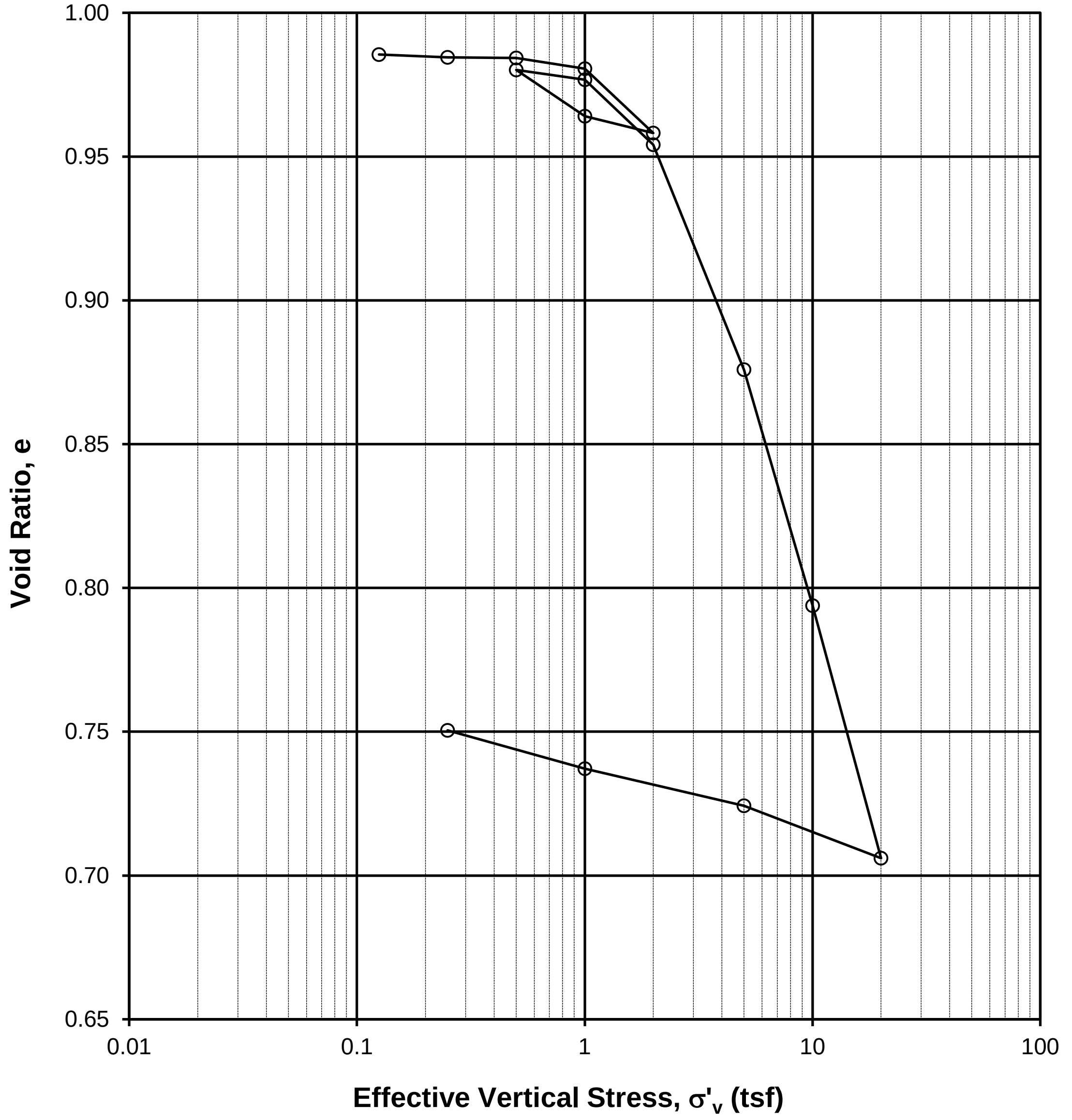
Liquid Limit= 74 Plastic Limit= 25 Plasticity Index = 49 USCS: CH

Compression Index, C_c = 0.29

Void Ratio, e_o = 0.81

Recompression Index, C_r = 0.6

Preconsolidation Pressure = 2.15 tsf



1-D CONSOLIDATION TEST: INCREMENTAL

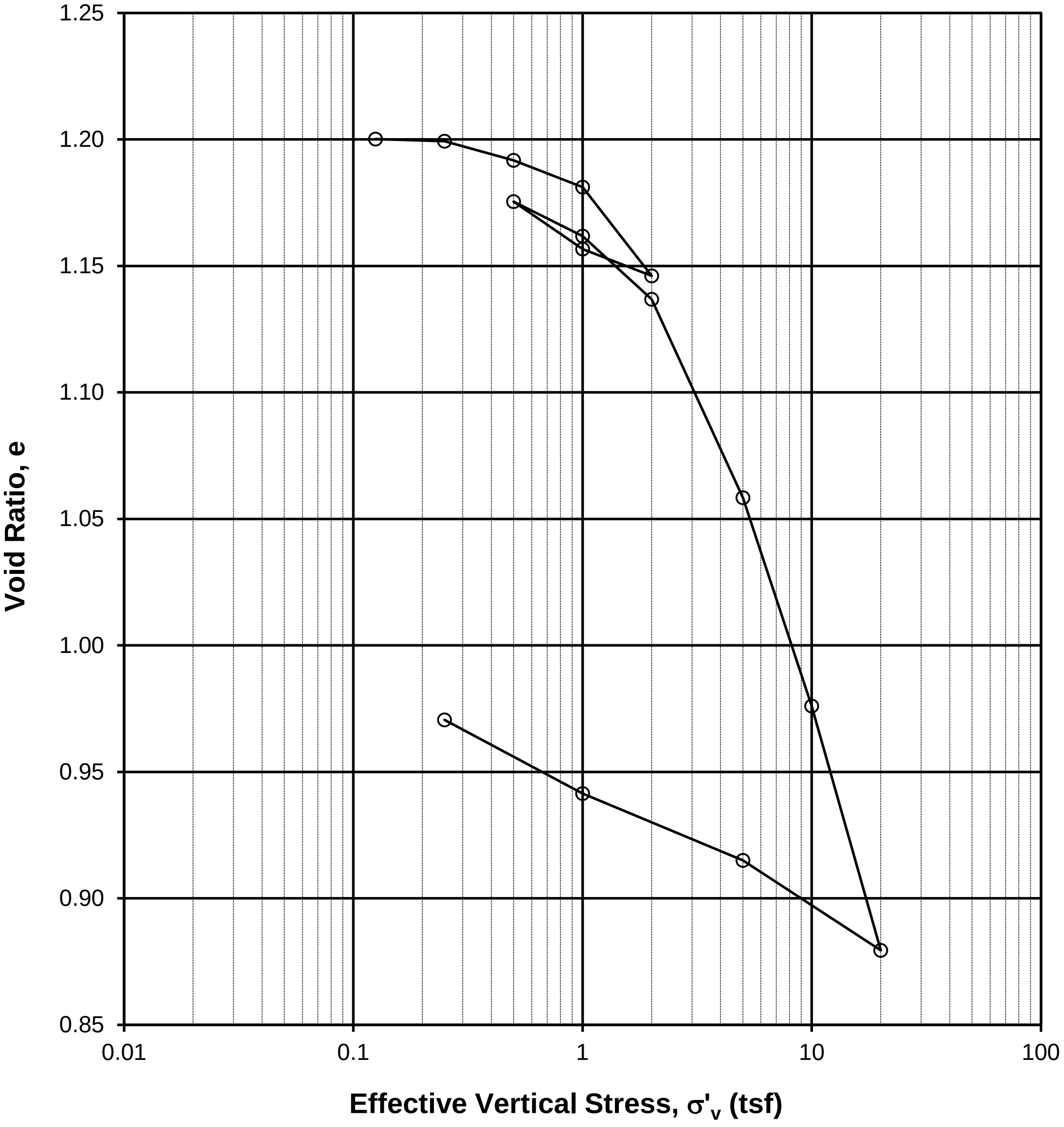
ASTM D 2435
Project No.: J033659.01
Boring: B-2
Sample: ST-5 - Depth: 10.0

GEOTECHNOLOGY INC
FROM THE GROUND UP

Liquid Limit= 105 Plastic Limit= 42 Plasticity Index = 63 USCS: CH

Compression Index, C_c = 0.32 Void Ratio, e_o = 1.03

Recompression Index, C_r = 0.6 Preconsolidation Pressure = 2.71 tsf



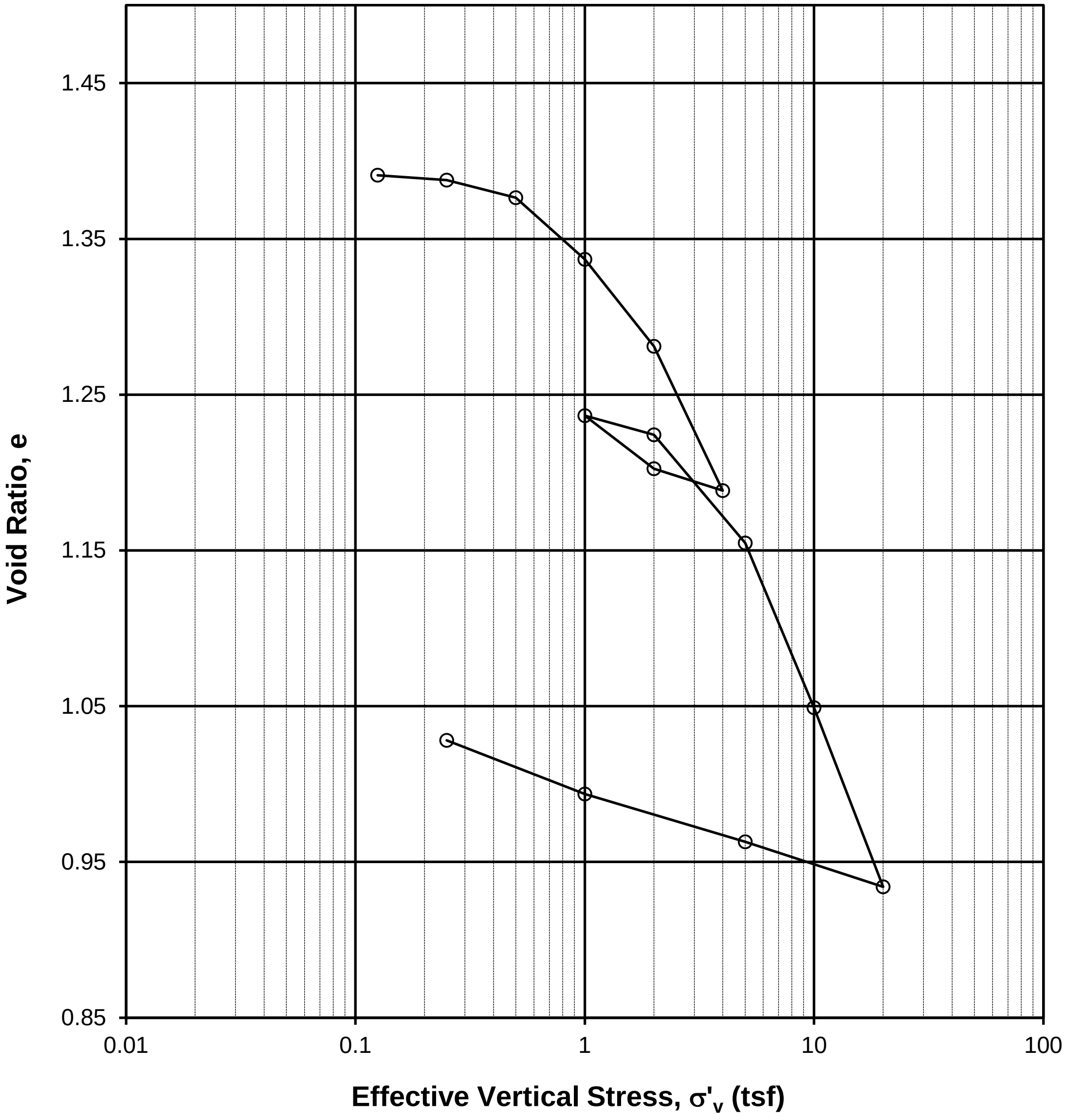
1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435
Project No.: J033659.01
Boring: B-6
Sample: ST-6 - Depth: 15.0

Liquid Limit= 96 Plastic Limit= 32 Plasticity Index = 64 USCS: CH

Compression Index, C_c = 3.73 Void Ratio, e_o = 0.70

Recompression Index, C_r = 0.75 Preconsolidation Pressure = 2 tsf



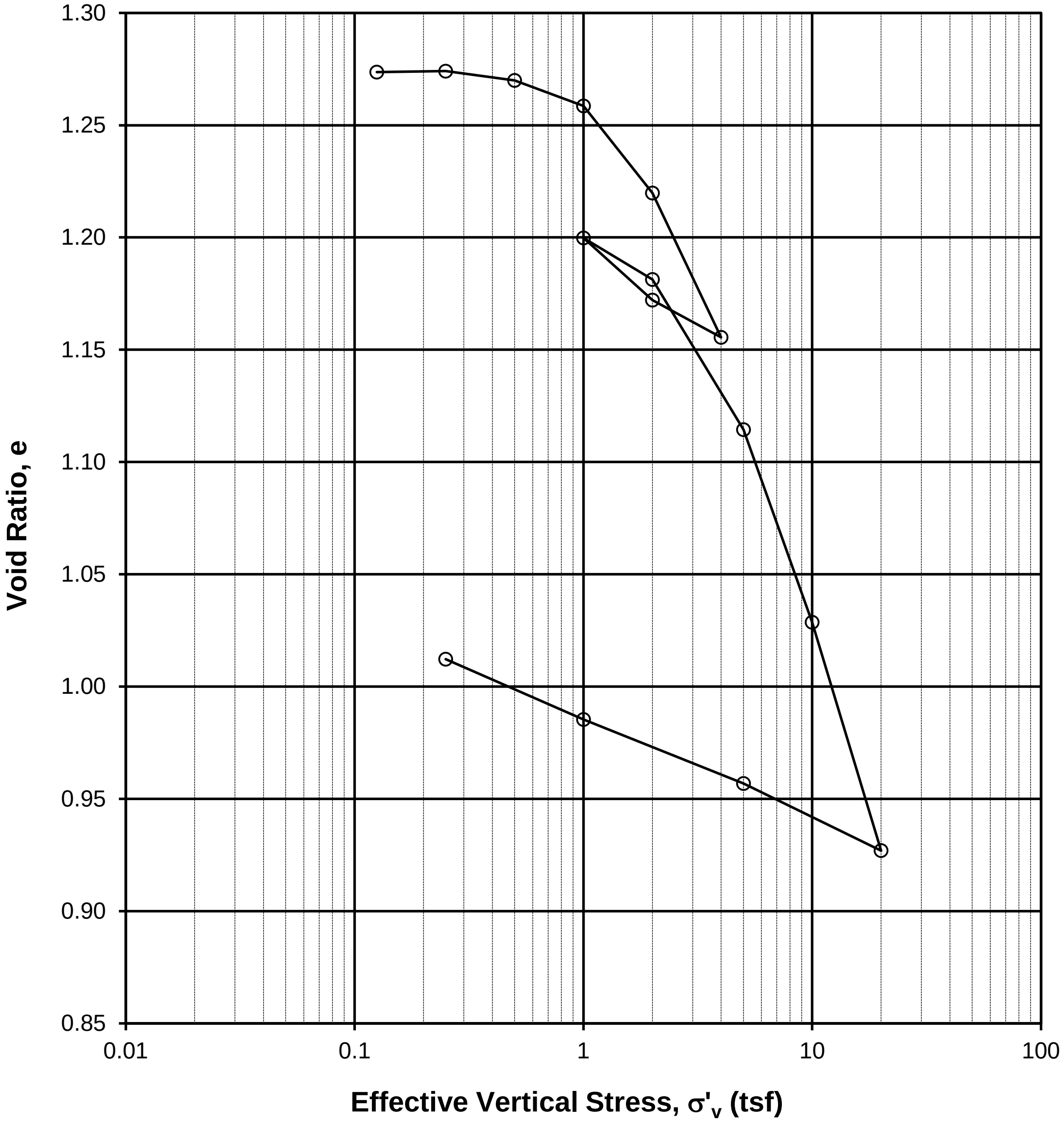
1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435
Project No.: J033659.01
Boring: B-9
Sample: ST-5 - Depth: 10

Liquid Limit= 99 Plastic Limit= 29 Plasticity Index = 70 USCS: CH

Compression Index, C_c = 0.34 Void Ratio, e_o = 1.05

Recompression Index, C_r = 0.07 Preconsolidation Pressure = 2.45 tsf



1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435
Project No.: J033659.01
Boring: B-11
Sample: ST-7 - Depth: 20

SOIL RESISTIVITY TEST REPORT

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

Project No.:	**J033659.01**	**February 20, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-2**	
Sample ID:	**SS6-SS8**	
Depth (ft):	**13.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	6,800	0.57	3,876.00	14.8
#2	1,200	0.57	684.00	24.0
#3	1,000	0.57	570.00	28.4
#4	1,100	0.57	627.00	36.3

Minimum Soil Resistivity **570.00**

SOIL RESISTIVITY TEST REPORT

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

Project No.:	**J033659.01**	**February 12, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-3**	
Sample ID:	**SS12, SS13**	
Depth (ft):	**13.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	23,000	0.57	13,110.00	10.3
#2	15,000	0.57	8,550.00	18.1
#3	13,000	0.57	7,410.00	24.7
#4	11,000	0.57	6,270.00	30.0
#5	13,000	0.57	7,410.00	32.7

Minimum Soil Resistivity **6,270.00**

SOIL RESISTIVITY TEST REPORT

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

Project No.:	**J033659.01**	**February 20, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-4**	
Sample ID:	**SS13-SS15**	
Depth (ft):	**53.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	9,800	0.57	5,586.00	9.8
#2	6,000	0.57	3,420.00	16.1
#3	4,700	0.57	2,679.00	23.9
#4	5,200	0.57	2,964.00	28.1

Minimum Soil Resistivity **2,679.00**

SOIL RESISTIVITY TEST REPORT

Prepared For:

Garver USA
4701 Northshore Drive North Little Rock, AR 72118

Project No.:	**J033659.01**	**February 21, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-5**	
Sample ID:	**SS6-SS8**	
Depth (ft):	**18.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	3,100	0.57	1,767.00	18.3
#2	1,600	0.57	912.00	25.8
#3	1,700	0.57	969.00	34.0

Minimum Soil Resistivity **912.00**

SOIL RESISTIVITY TEST REPORT

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

Project No.: J033659.01
Project Name: ARDOT (McGehee)
Boring Number: B-5
Sample ID: SS15-SS17
Depth (ft): 63.5

February 21, 2019
Page 1 of 1

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	10,900	0.57	6,213.00	10.3
#2	6,200	0.57	3,534.00	17.8
#3	5,000	0.57	2,850.00	24.1
#4	5,900	0.57	3,363.00	25.1

Minimum Soil Resistivity **2,850.00**

SOIL RESISTIVITY TEST REPORT

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

Project No.:	**J033659.01**	**February 21, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-6**	
Sample ID:	**SS11-SS13**	
Depth (ft):	**38.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	16,000	0.57	9,120.00	11.1
#2	8,200	0.57	4,674.00	19.3
#3	6,100	0.57	3,477.00	27.4
#4	6,900	0.57	3,933.00	48.4

Minimum Soil Resistivity **3,477.00**

SOIL RESISTIVITY TEST REPORT

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

Project No.:	**J033659.01**	**February 25, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-8**	
Sample ID:	**SS6-SS8**	
Depth (ft):	**18.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	1,700	0.57	969.00	24.0
#2	1,400	0.57	798.00	31.5
#3	1,300	0.57	741.00	40.9
#4	1,500	0.57	855.00	59.9

Minimum Soil Resistivity **741.00**

SOIL RESISTIVITY TEST REPORT

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

Project No.:	**J033659.01**	**February 25, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-8**	
Sample ID:	**SS15-SS16**	
Depth (ft):	**68.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	10,700	0.57	6,099.00	11.6
#2	5,900	0.57	3,363.00	19.6
#3	5,700	0.57	3,249.00	27.7
#4	5,600	0.57	3,192.00	49.4
#5	8,800	0.57	5,016.00	110.0

Minimum Soil Resistivity **3,192.00**

TEST REPORT

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

Project No.:	**J033659.01**	**February 25, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-10**	
Sample ID:	**SS13-SS14**	
Depth (ft):	**48.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	9,500	0.57	5,415.00	11.2
#2	5,100	0.57	2,907.00	20.0
#3	4,900	0.57	2,793.00	27.6
#4	6,000	0.57	3,420.00	102.9

Minimum Soil Resistivity **2,793.00**

SOIL RESISTIVITY TEST REPORT

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

Project No.:	**J033659.01**	**February 25, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-11**	
Sample ID:	**SS10-SS12**	
Depth (ft):	**33.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	18,000	0.57	10,260.00	10.4
#2	14,000	0.57	7,980.00	17.2
#3	12,000	0.57	6,840.00	24.1
#4	14,000	0.57	7,980.00	25.6

Minimum Soil Resistivity **6,840.00**

SOIL RESISTIVITY TEST REPORT

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

Project No.:	**J033659.01**	**February 25, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-12**	
Sample ID:	**SS13-SS14**	
Depth (ft):	**58.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	8,500	0.57	4,845.00	15.8
#2	4,800	0.57	2,736.00	24.3
#3	4,400	0.57	2,508.00	32.3
#4	6,200	0.57	3,534.00	31.7

Minimum Soil Resistivity **2,508.00**

SOIL RESISTIVITY TEST REPORT

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

Project No.:	**J033659.01**	**February 25, 2019**
Project Name:	**ARDOT (McGehee)**	**Page 1 of 1**
Boring Number:	**B-13**	
Sample ID:	**SS8-SS10**	
Depth (ft):	**23.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	1,300	0.57	741.00	28.8
#2	900	0.57	513.00	35.4
#3	1,100	0.57	627.00	45.1

Minimum Soil Resistivity **513.00**

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

GEOTECHNOLOGY INC
FROM THE GROUND UP

DATE February 8, 2019	PROJECT NAME ARDOT McGehee	PROJECT NO. J033659.01

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	SS6	13.50		1:2	7.61 ------------- 21.6			
B-3	SS12	38.50		1:1	7.05 ------------- 21.4			
B-4	SS13	53.50		1:1	7.13 ------------- 21.4			
B-5	SS6	18.50		1:2	7.32 ------------- 21.6			
B-5	SS15	63.50		1:1	8.01 ------------- 21.4			
B-6	SS11	38.50		1:1	6.91 ------------- 21.4			
B-8	SS6	18.50		1:2	7.68 ------------- 21.5			
B-8	SS15	68.50		1:1	8.45 ------------- 21.2			
B-13	SS8	23.50		1:1	7.16 ------------- 21.2			
B-10	SS13	48.50		1:1	8.41 ------------- 20.8			
B-11	SS10	33.50		1:1	8.37 ------------- 20.8			
B-12	SS13	58.50		1:1	7.72 ------------- 21.1			

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

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

Tested By: AIM
Date: 02/13/19

Calculated By: HP
Date: 02/13/19

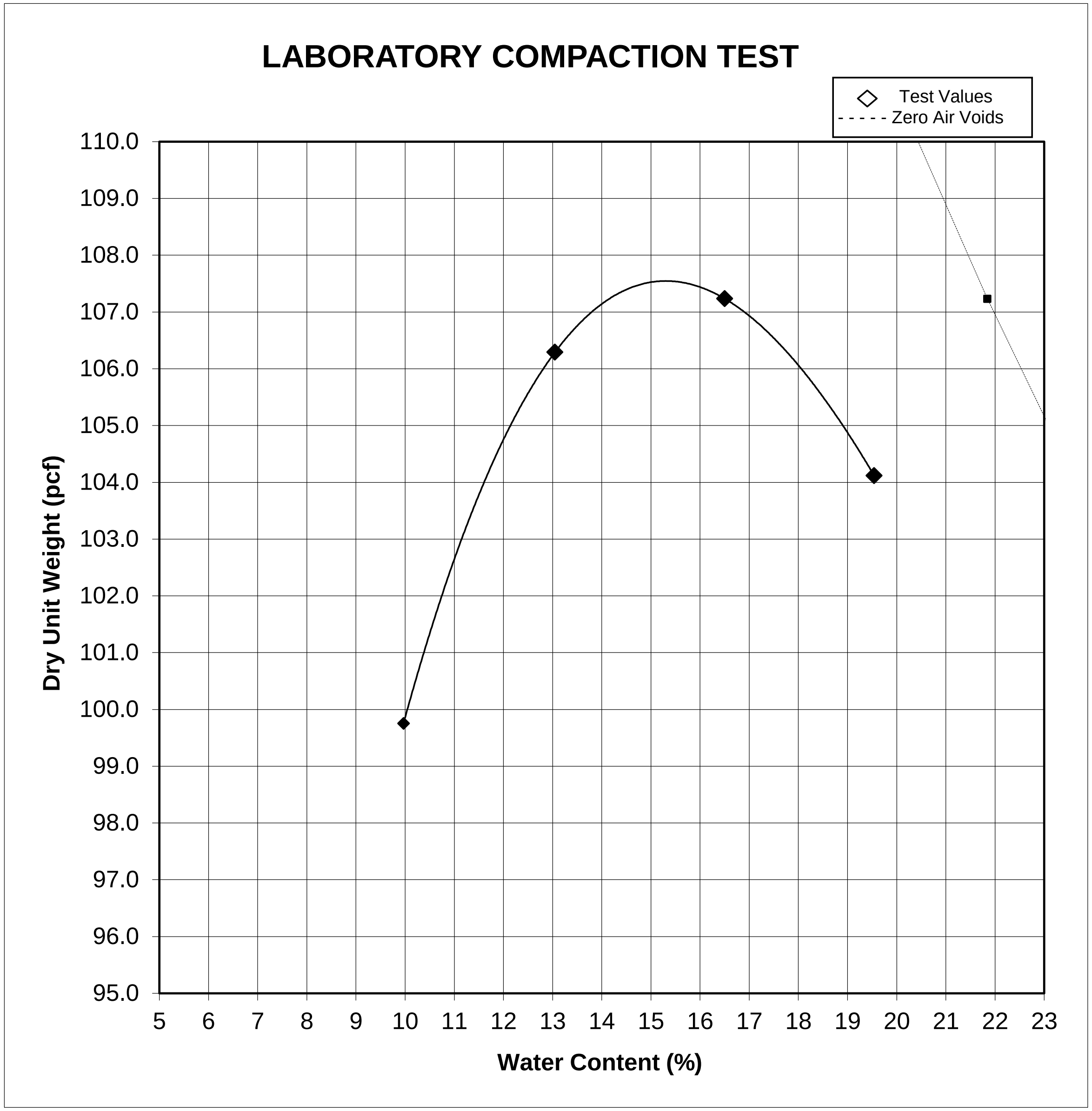
Checked By: ALY
Date: 02/26/19

3312 Winbrook Dr
Memphis, TN 38116
Ph: 901-353-1981
Fax: 901-353-2248

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: ARDOT (McGehee)
Client: Garver USA
Sample Source: B-2, 1.0'-5.0'
Supplier:

LABORATORY COMPACTION TEST



Test Information

Project No.: J033659.01
Test Date: 02/11/19
Proctor No.: B-2

Test Method: ASTM D 698 Method A
Rammer Type: Mechanical
Prep. Method: Dry

Sample Description

Dark Brown, Lean Clay with sand

Sample Properties

Moisture Content NA
Liquid Limit 42
Plastic Limit 20
Plasticity Index 22
Specific Gravity: 2.750 Estimated
Classification CL

Test Results:

Maximum Dry Unit Weight (pcf): 107.6
Optimum Water Content (%): 15.2

Oversize Correction Values:

Maximum Dry Unit Weight (pcf): --
Optimum Water Content (%): --

Tested By: TA
Date: 02/11/19

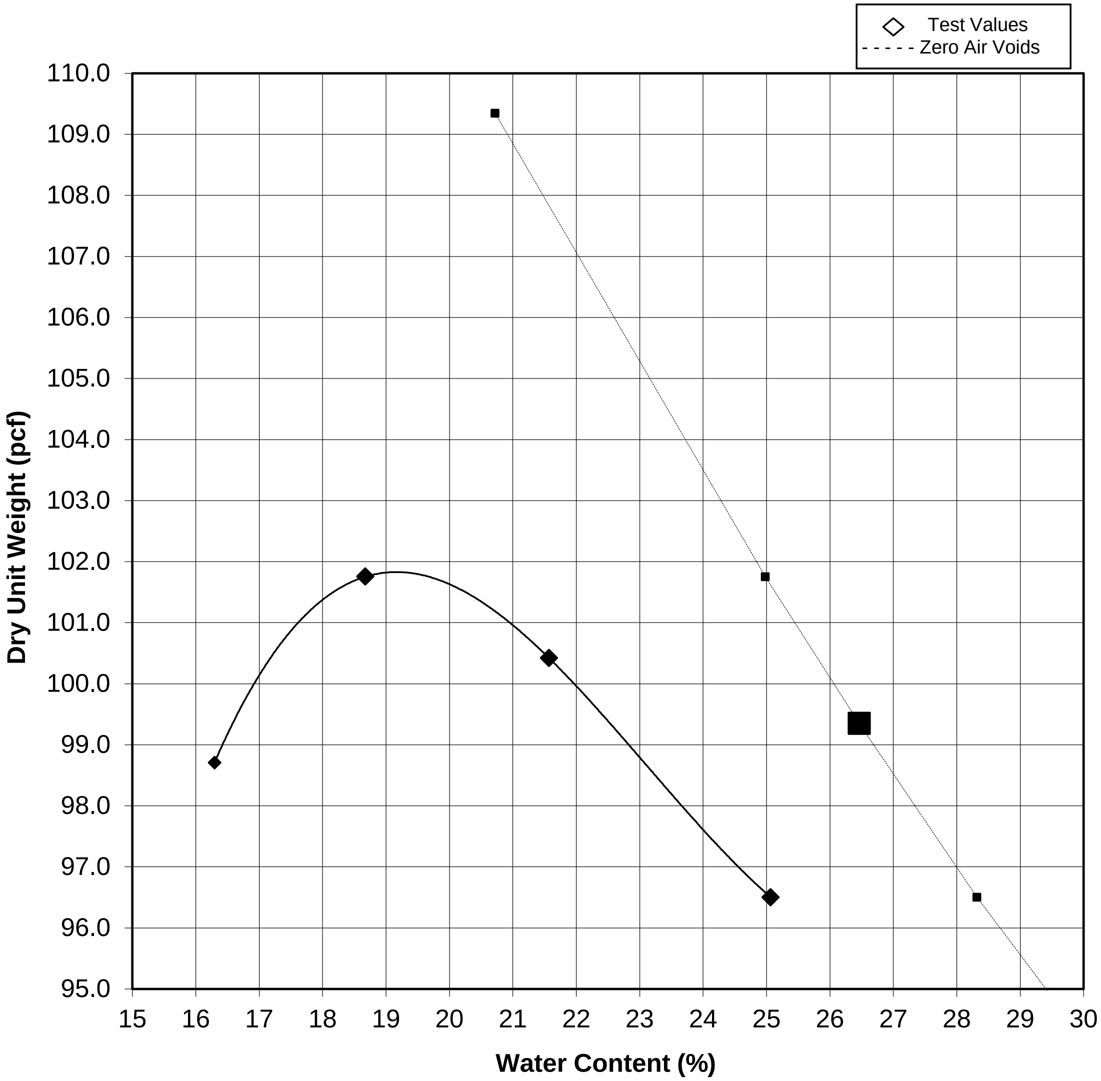
Input By: HP
Date: 02/12/19

Checked By: HP
Date: 02/12/19

3312 Winbrook Dr
Memphis, TN 38116
Ph: 901-353-1981
Fax: 901-353-2248

Project: ARDOT (McGehee)
Client: Garver USA
Sample Source: B-12, 1.0'-5.0'
Supplier:

LABORATORY COMPACTION TEST



Test Information

Project No.: J033659.01
Test Date: 02/13/19
Proctor No.: B-12

Test Method: ASTM D 698 Method A
Rammer Type: Mechanical
Prep. Method: Dry

Sample Description

Red/Brown Lean Clay, trace sand

Sample Properties

Moisture Content NA
Liquid Limit 49
Plastic Limit 21
Plasticity Index 28
Specific Gravity: 2.750 Estimated
Classification CL

Test Results:

Maximum Dry Unit Weight (pcf): 101.8
Optimum Water Content (%): 19.1

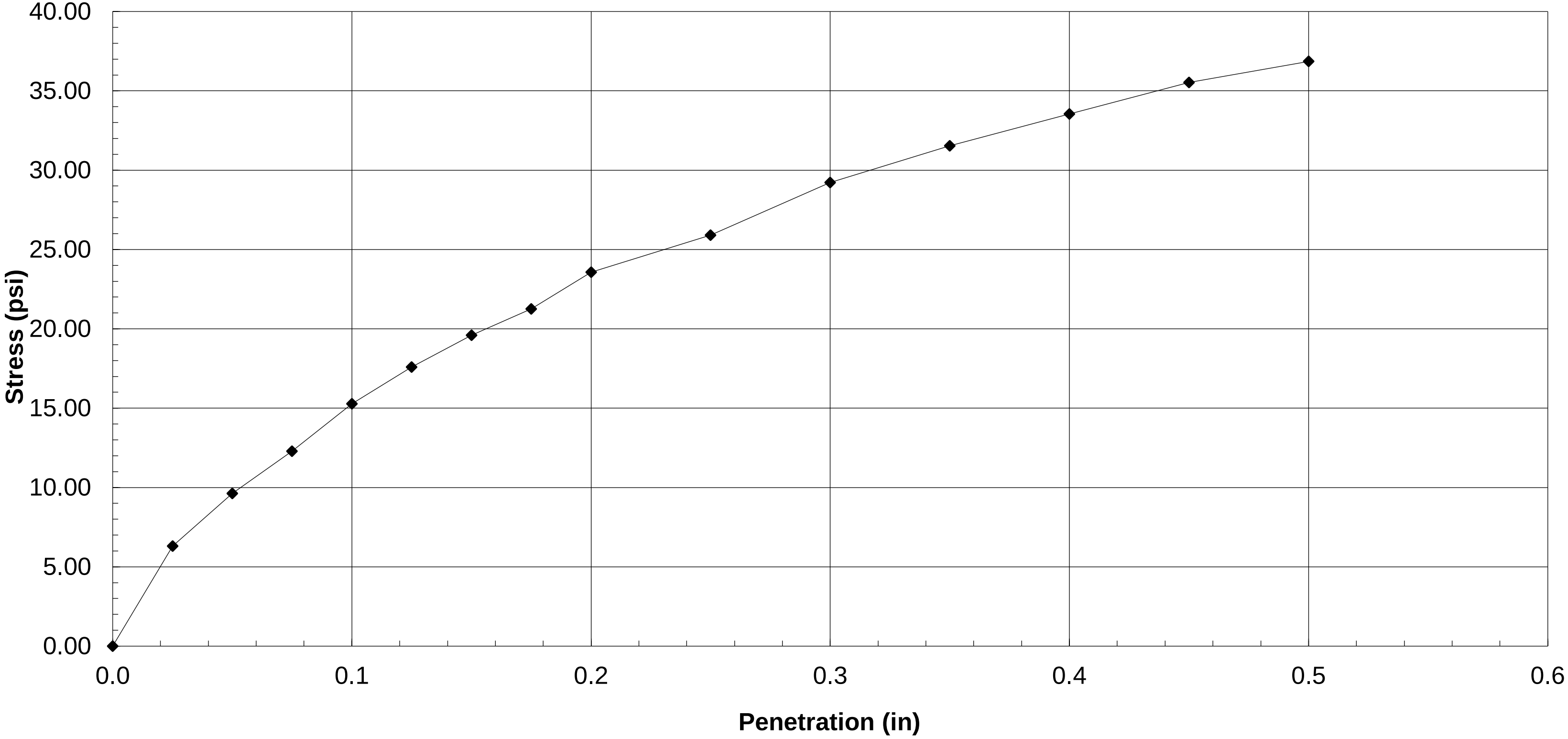
Oversize Correction Values:
Maximum Dry Unit Weight (pcf): --
Optimum Water Content (%): --

Tested By: TA
Date: 02/13/19

Input By: HP
Date: 02/18/19

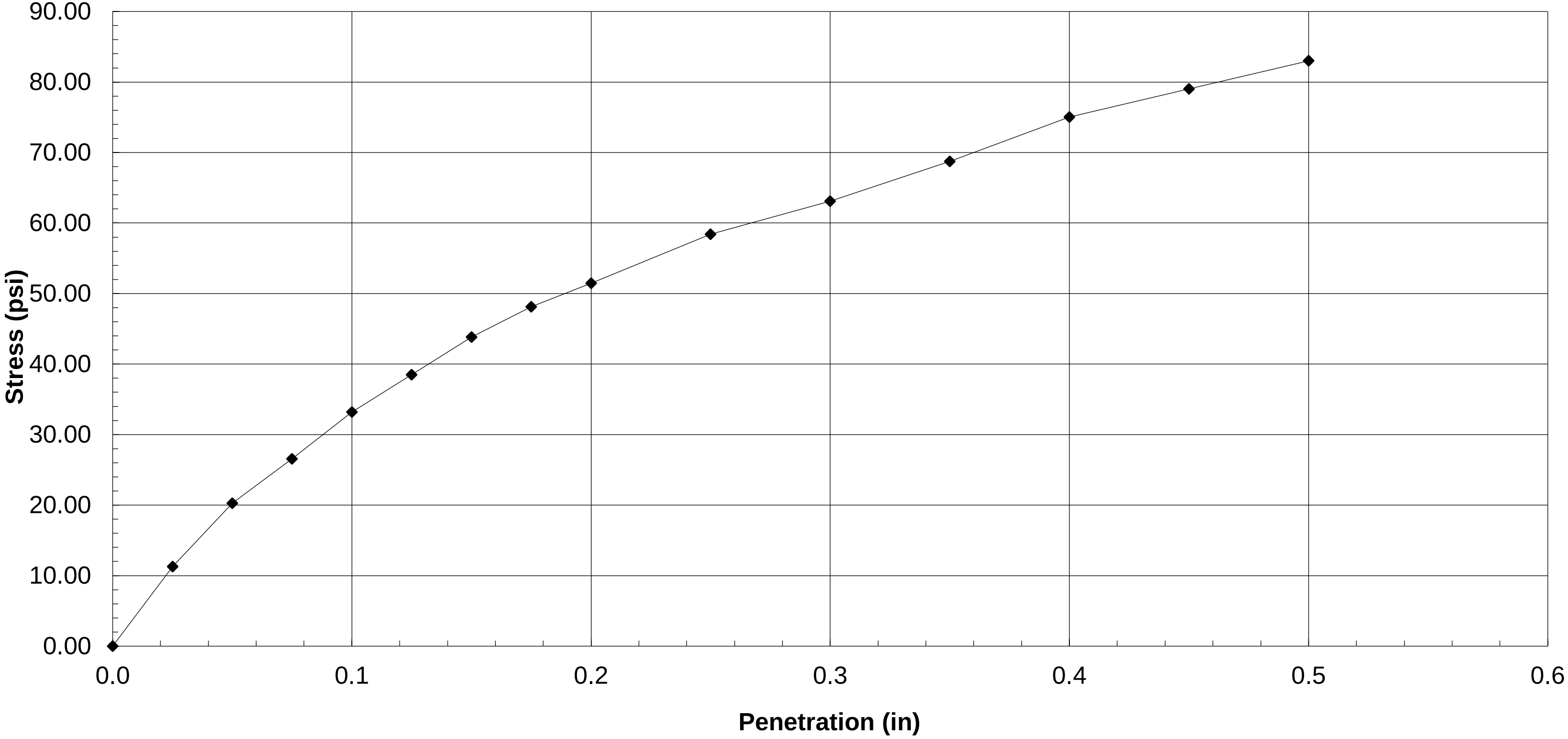
Checked By: HP
Date: 02/18/19

P401 (04/17/12)



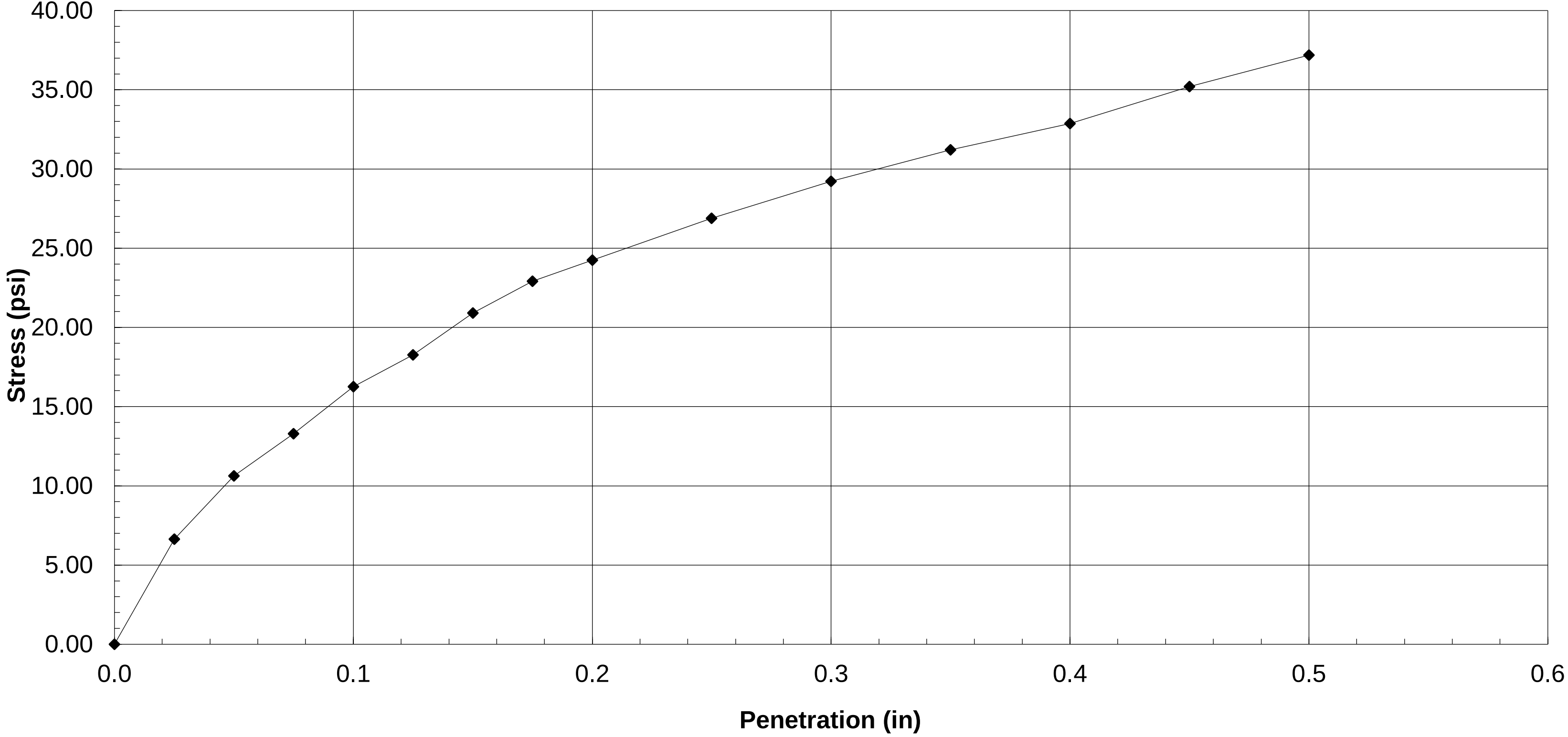
CALIFORNIA BEARING RATIO (CBR) TEST

ASTM D 1883
Project No.: J033659.01
Boring: B-2
Sample: 25 Blows - Depth: 0 ft.



CALIFORNIA BEARING RATIO (CBR) TEST

ASTM D 1883
Project No.: J033659.01
Boring: B-2
Sample: 56 Blows - Depth: 0 ft.



CALIFORNIA BEARING RATIO (CBR) TEST

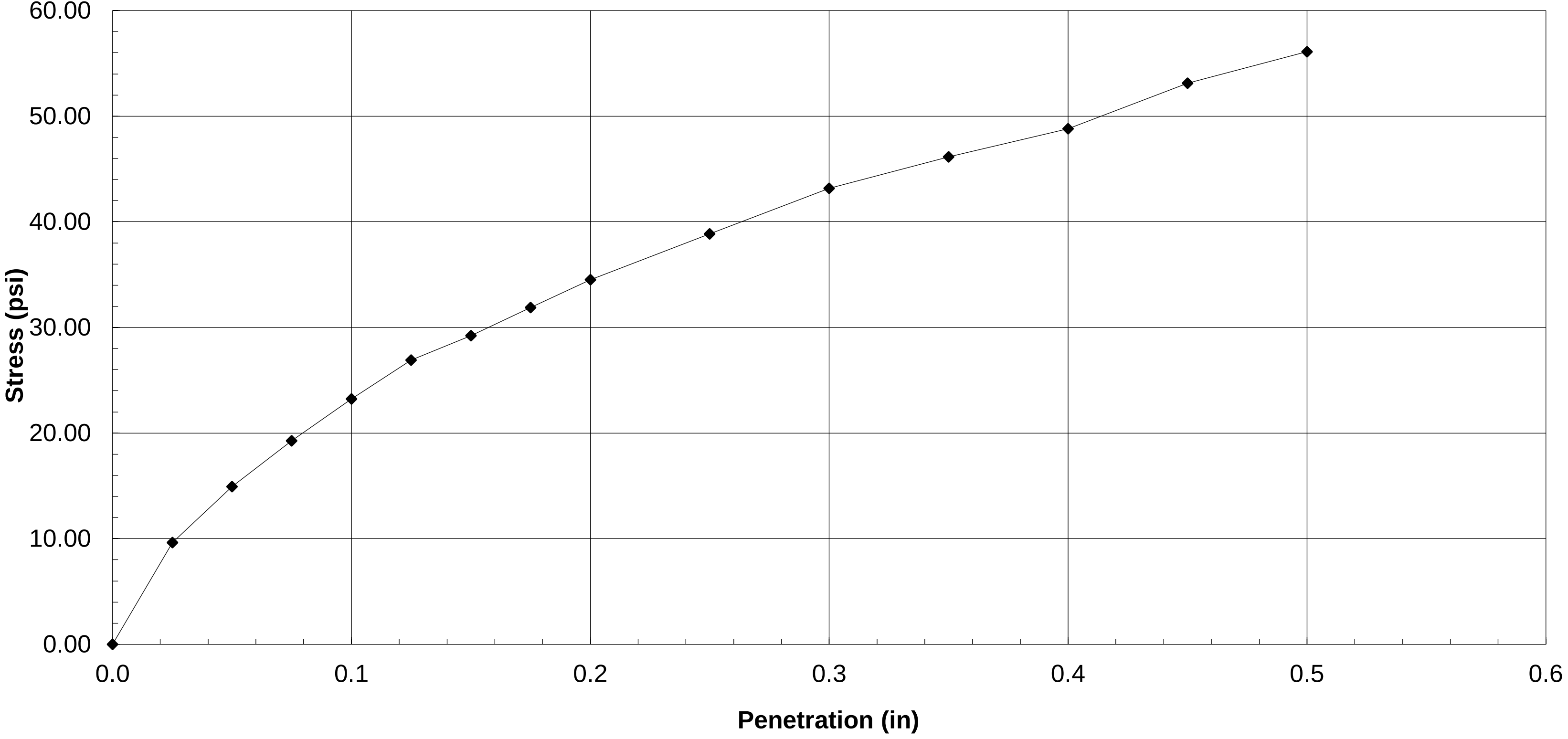
ASTM D 1883

Project No.: J033659.01

Boring: B-12

Sample: 25 Blows - Depth: 0 ft.

GEOTECHNOLOGY INC
FROM THE GROUND UP



CALIFORNIA BEARING RATIO (CBR) TEST
ASTM D 1883
Project No.: J033659.01
Boring: B-12
Sample: 56 Blows - Depth: 0 ft.

APPENDIX E – AASHTO AND USCS CLASSIFICATIONS

SUMMARY OF CLASSIFICATION TEST RESULTS
Highway 278 Improvements Bridge over Union Pacific Railroad
Desha County, Arkansas
ARDOT 020590

Boring No.	Depth	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Sieve Analysis Percent Passing								AASHTO CLASS.	USCS CLASS.
					2 in.	1 in.	3/4 in.	3/8 in.	#4	#10	#40	#200		
B-1	3.5	85	34	51	--	--	--	--	--	--	--	--	A-7-5	CH
B-2	10	74	25	49	--	--	--	--	--	--	--	--	A-7-6	CH
B-2	0	42	20	22	100.0	100.0	100.0	100.0	100.0	93.2	85.7	74.5	A-7-6	CL
B-2	33.5	111	40	71	--	--	--	--	--	--	--	--	A-7-5	CH
B-3	10	82	31	51	--	--	--	--	--	--	--	--	A-7-5	CH
B-3	53.5	--	--	--	100.0	100.0	100.0	100.0	99.9	99.8	79.2	3.3	A-2-6	SP
B-4	8	105	32	73	100.0	100.0	83.0	76.4	73.0	71.1	68.9	65.1	A-7-5	CH
B-4	33.5	105	32	73	--	--	--	--	--	--	--	--	A-7-5	CH
B-4	58.5	--	--	--	100.0	100.0	100.0	74.7	69.0	63.5	25.9	4.1	A-2-6	SP
B-4	68.5	--	--	--	100.0	100.0	100.0	100.0	100.0	99.6	85.9	15.7	A-2-6	SC
B-5	6	58	18	40	--	--	--	--	--	--	--	--	A-7-6	CH
B-5	23.5	74	27	47	--	--	--	--	--	--	--	--	A-7-6	CH
B-5	53.5	--	--	--	100.0	100.0	100.0	100.0	99.2	95.4	58.5	3.7	A-2-6	SP
B-6	3	38	17	21	--	--	--	--	--	--	--	--	A-6	CL
B-6	15	105	42	63	--	--	--	--	--	--	--	--	A-7-5	CH
B-6	33.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.4	75.1	A-4	ML
B-6	88.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.8	7.4	A-2-6	SP-SC
B-7	20	104	32	72	--	--	--	--	--	--	--	--	A-7-5	CH
B-7	38.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	100.0	32.7	A-2-6	SC
B-7	68.5	--	--	--	100.0	100.0	100.0	98.3	97.3	97.0	22.5	4.0	A-2-6	SP
B-8	8	43	17	26	--	--	--	--	--	--	--	--	A-7-6	CL
B-8	33.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.9	19.5	A-2-4	SM
B-8	48.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	95.3	11.7	A-2-4	SP-SM
B-9	10	96	32	64	--	--	--	--	--	--	--	--	A-7-5	CH
B-9	15	104	37	67	--	--	--	--	--	--	--	--	A-7-5	CH
B-9	33.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.0	59.4	A-7-5	CH
B-9	78.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	71.0	8.7	A-3	SP-SM
B-10	6	46	21	25	--	--	--	--	--	--	--	--	A-7-6	CL
B-10	15	106	33	73	--	--	--	--	--	--	--	--	A-7-5	CH
B-10	43.5	--	--	--	100.0	100.0	100.0	100.0	100.0	99.9	89.3	5.0	A-3	SP-SM

SUMMARY OF CLASSIFICATION TEST RESULTS

Highway 278 Improvements Bridge over Union Pacific Railroad
Desha County, Arkansas
ARDOT 020590

Boring No.	Depth	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Sieve Analysis Percent Passing								AASHTO CLASS.	USCS CLASS.
					2 in.	1 in.	3/4 in.	3/8 in.	#4	#10	#40	#200		
B-11	20	99	29	70	--	--	--	--	--	--	--	--	A-7-6	CH
B-11	33.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	94.8	16.0	A-2-4	SM
B-11	58.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	97.0	10.6	A-2-4	SP-SM
B-12	0	49	21	28	100.0	100.0	100.0	100.0	100.0	100.0	99.5	95.0	A-7-6	CL
B-12	6	95	30	65	--	--	--	--	--	--	--	--	A-7-5	CH
B-12	12	29	17	12	--	--	--	--	--	--	--	--	A-6	CL
B-12	48.5	--	--	--	100.0	100.0	100.0	100.0	99.8	99.8	93.0	4.2	A-2-6	SP
B-12	78.5	--	--	--	100.0	100.0	100.0	100.0	93.9	84.9	44.4	5.9	A-1-b	SP-SM
B-13	8.5	89	30	59	--	--	--	--	--	--	--	--	A-7-5	CH

APPENDIX F - GLOBAL STABILITY ANALYSES

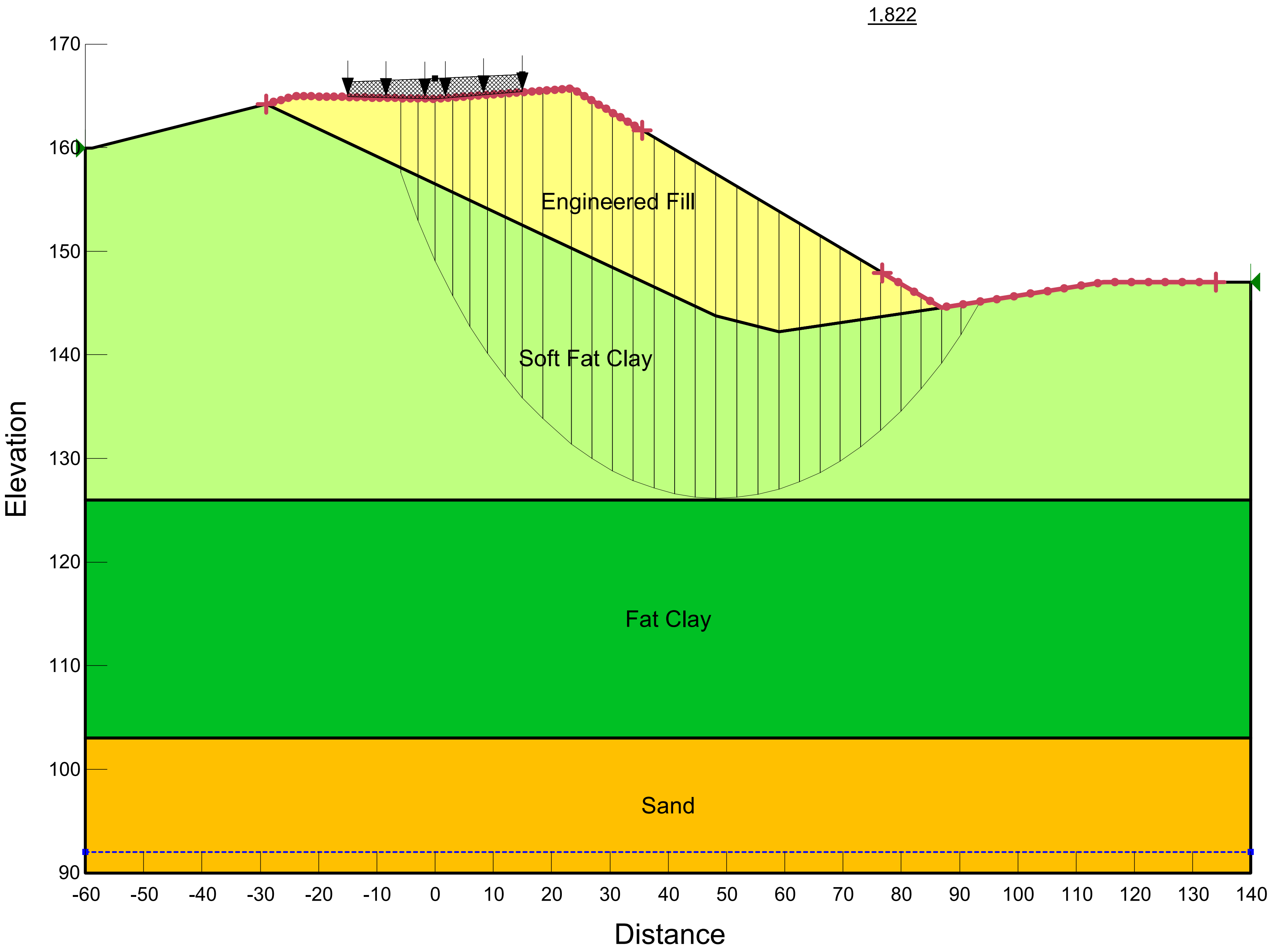
Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
Hwy 278 Station 110+18
Spencer's Method - Short Term Analysis

Engineered Fill
Model: Undrained (Phi=0)
Unit Weight: 120 pcf
Cohesion': 1,500 psf

Soft Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 800 psf

Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 1,200 psf

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36°



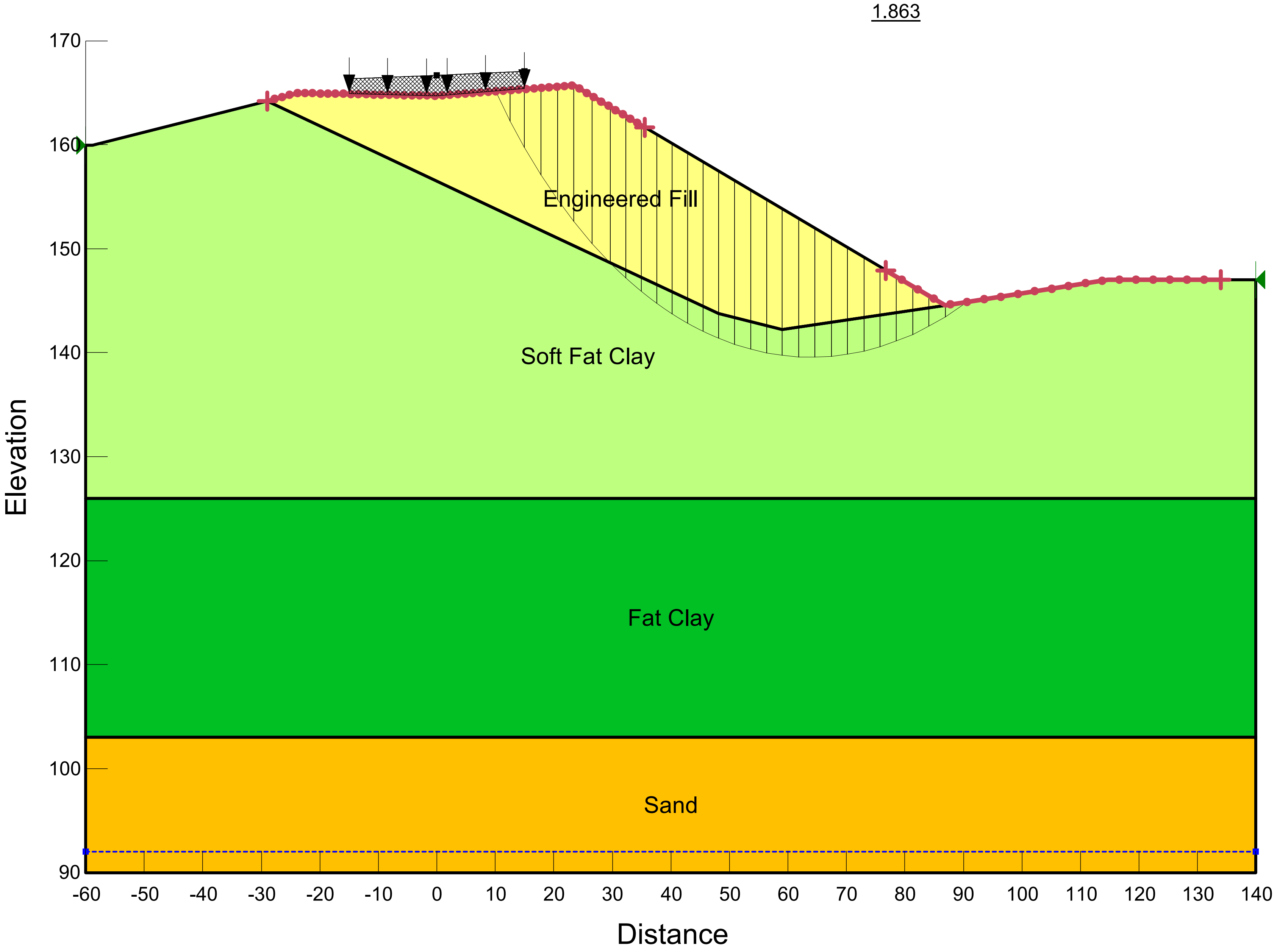
Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
Hwy 278 Station 110+18
Spencer's Method - Long Term Analysis

Engineered Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 50 psf
Phi': 30°

Soft Fat Clay
Model: MOhr-Coulomb
Unit Weight: 116 pcf
Cohesion': 0 psf
Phi': 24°

Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 0 psf
Phi': 24°

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36°



Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
Hwy 278 Station 110+18
Spencer's Method - Seismic Analysis

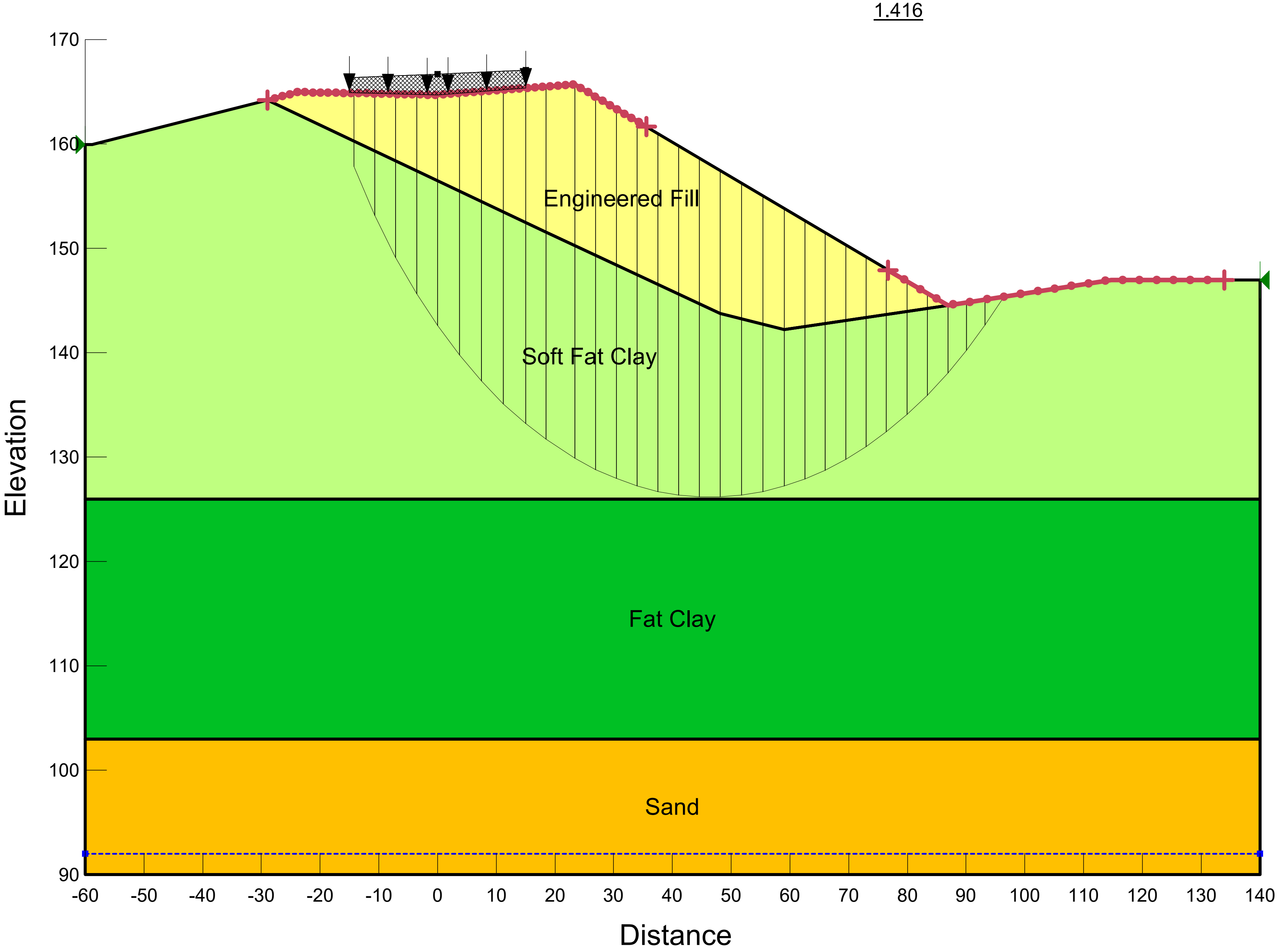
Ks = 0.073

Engineered Fill
Model: Undrained (Phi=0)
Unit Weight: 120 pcf
Cohesion': 1,500 psf

Soft Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 800 psf

Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 1,200 psf

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36º



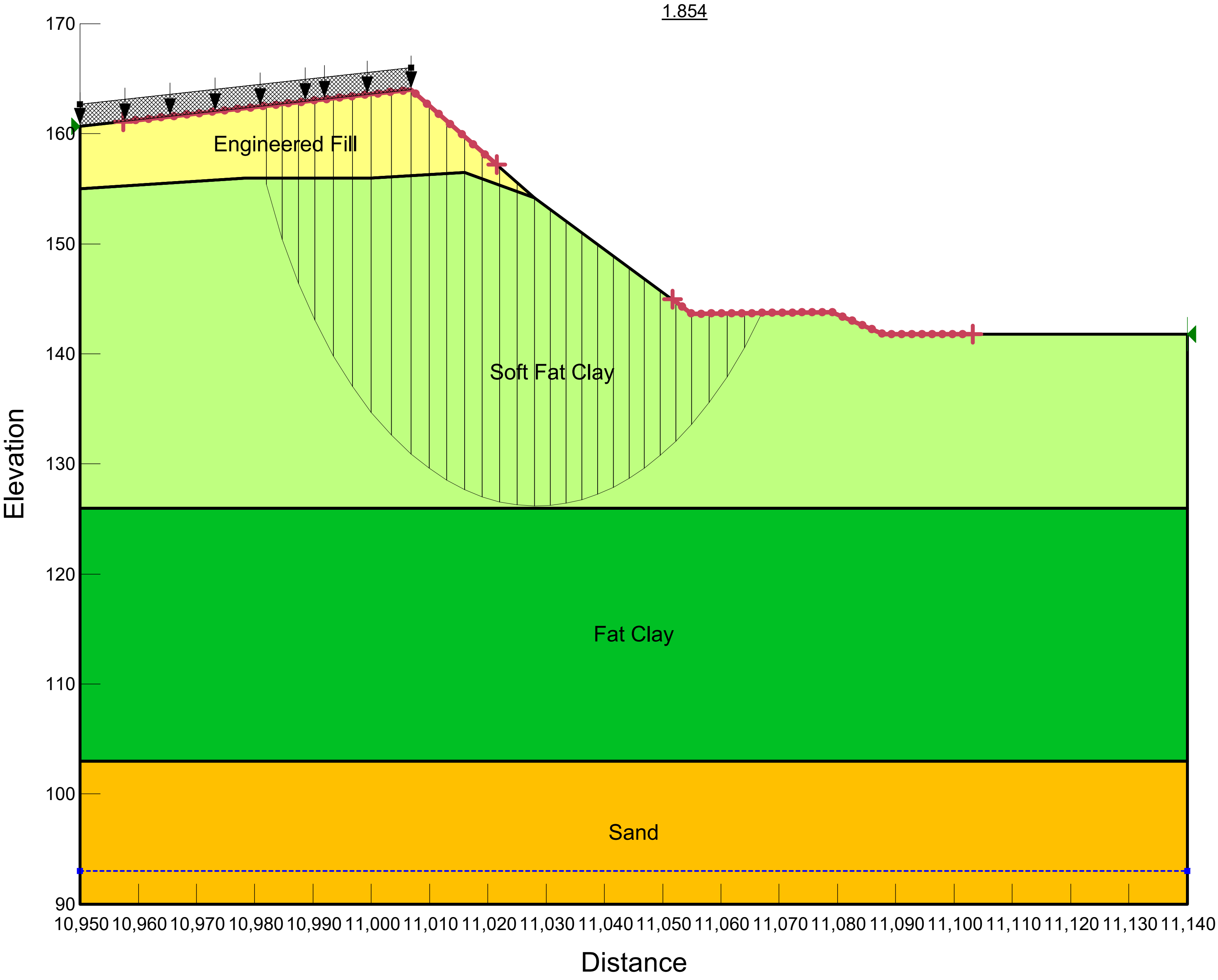
Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
West Abutment
Spencer's Method - Short Term Analysis

Engineered Fill
Model: Undrained (Phi=0)
Unit Weight: 120 pcf
Cohesion': 1,500 psf

Soft Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 800 psf

Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 1200 psf

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36º



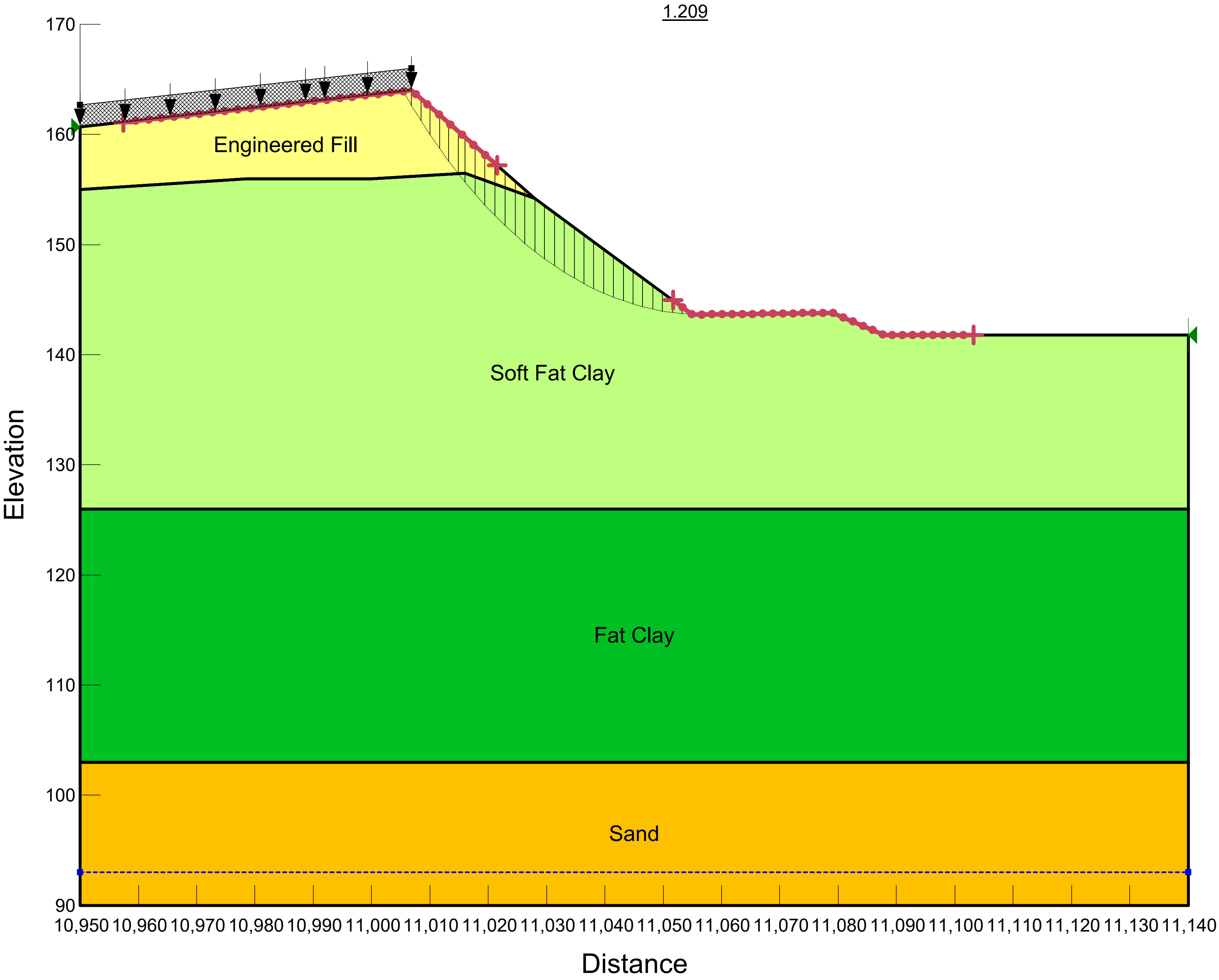
Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
West Abutment
Spencer's Method - Long Term Analysis

Engineered Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 50 psf
Phi': 30°

Soft Fat Clay
Model: Mohr-Coulomb
Unit Weight: 116 pcf
Cohesion': 0 psf
Phi': 24°

Fat Clay
Model: Mohr-Coulomb
Unit Weight: 116 pcf
Cohesion': 0 psf
Phi': 24°

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36°



Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
West Abutment
Spencer's Method - Seismic Analysis

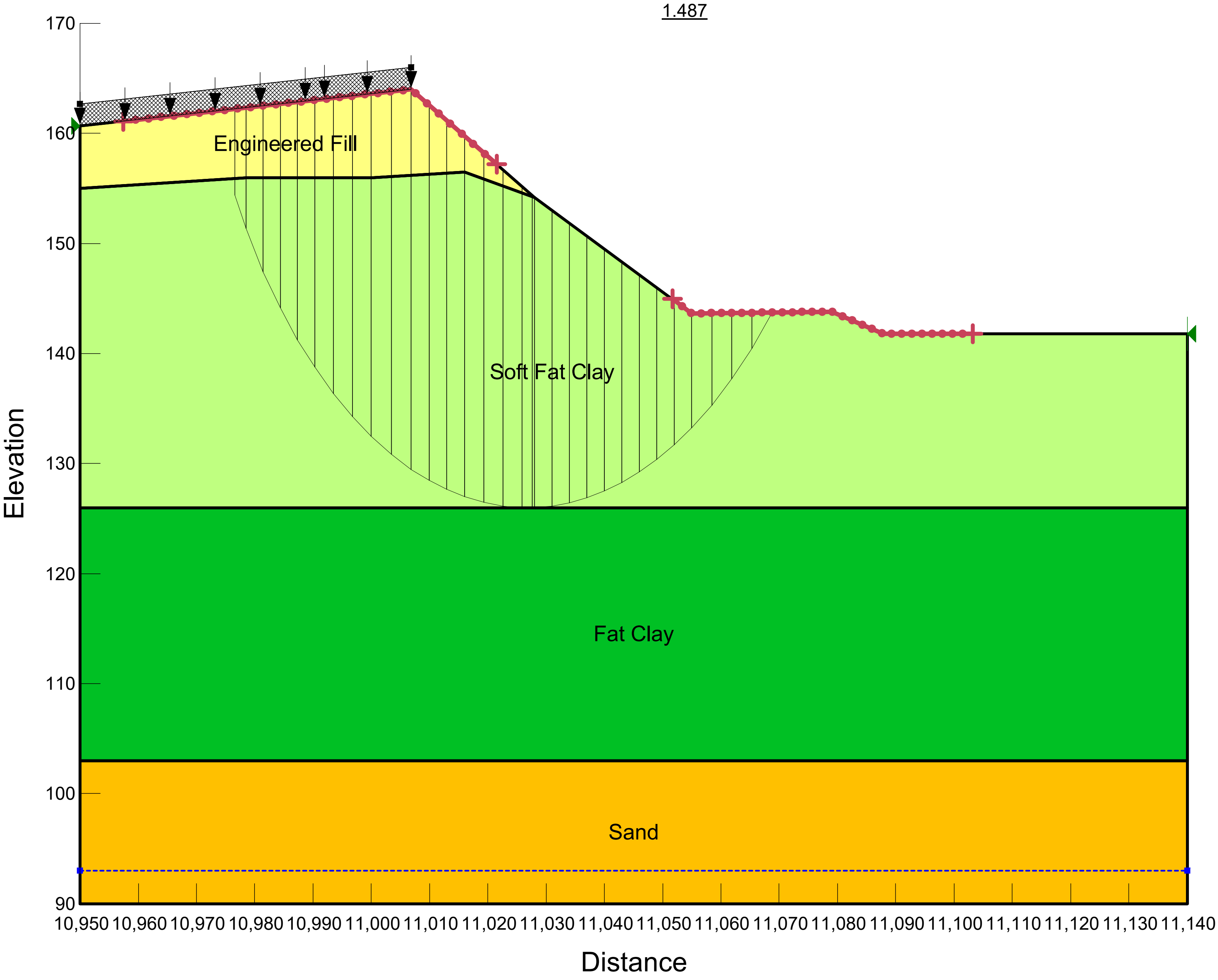
Ks = 0.073

Engineered Fill
Model: Undrained (Phi=0)
Unit Weight: 120 pcf
Cohesion': 1,500 psf

Soft Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 800 psf

Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 1200 psf

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36º



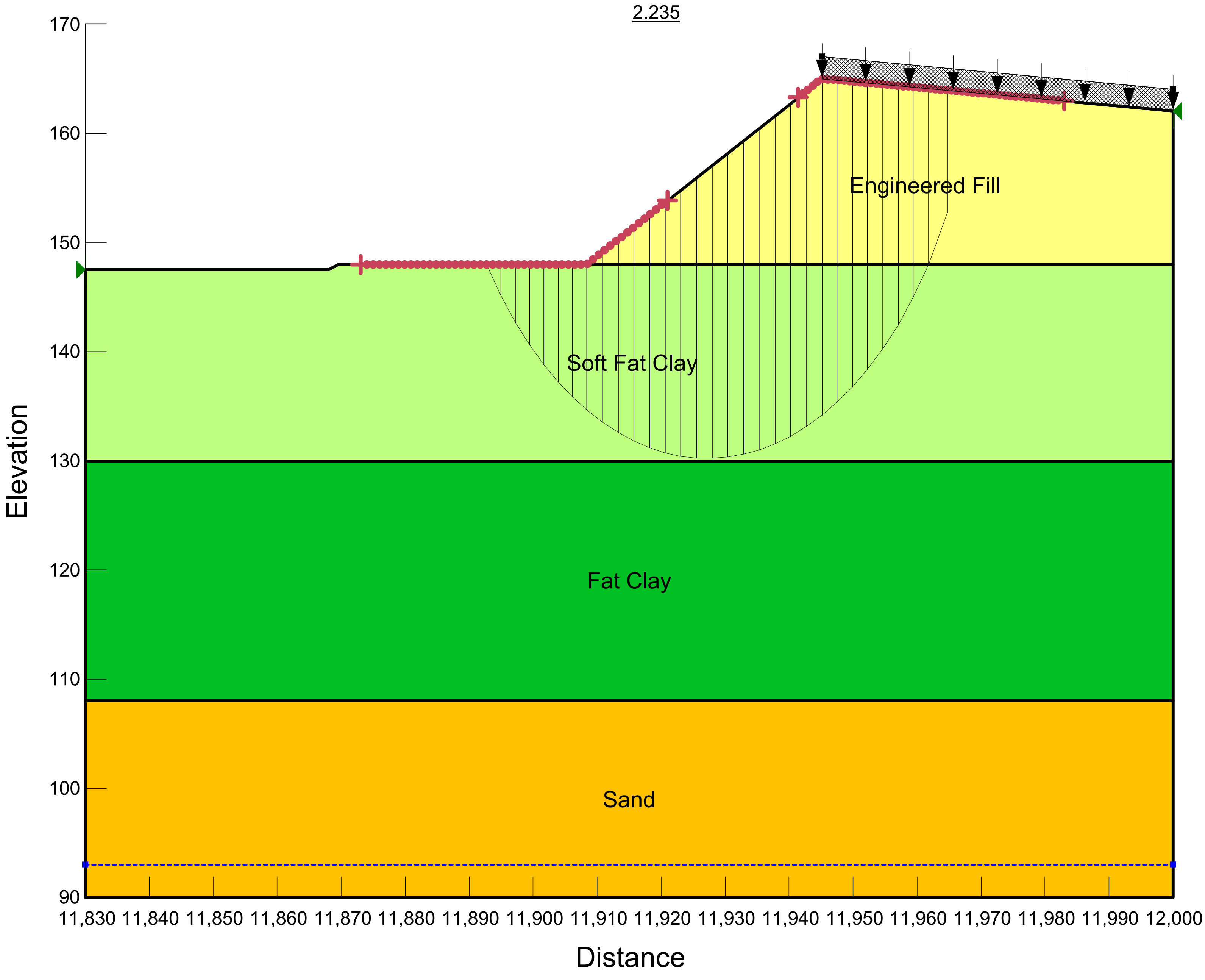
Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
East Abutment
Spencer's Method - Short Term Analysis

Engineered Fill
Model: Undrained (Phi=0)
Unit Weight: 120 pcf
Cohesion': 1,500 psf

Soft Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 800 psf

Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 1,200 psf

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36º



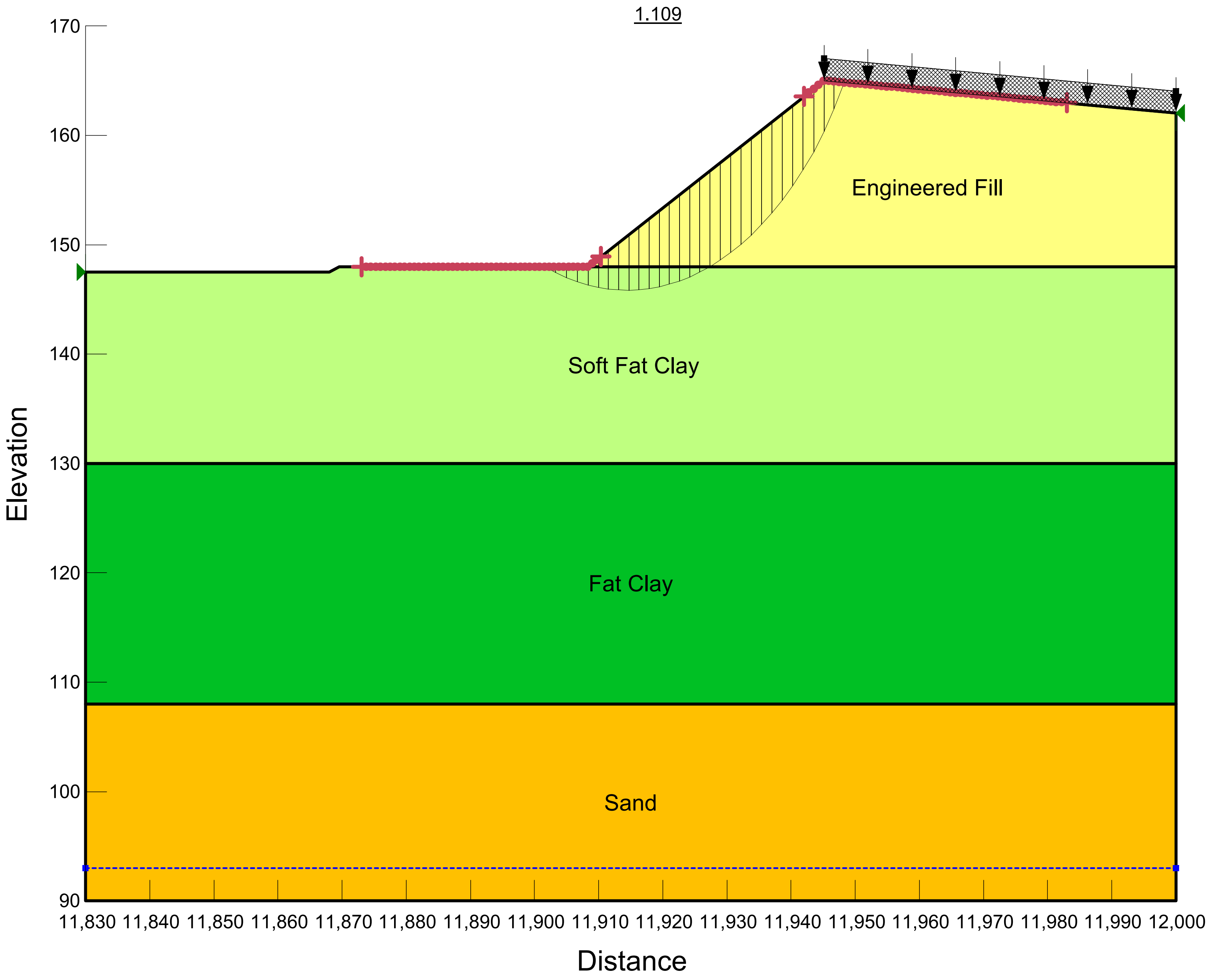
Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
East Abutment
Spencer's Method - Long Term Analysis

Engineered Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 50 psf
Phi': 30°

Soft Fat Clay
Model: Mohr-Coulomb
Unit Weight: 116 pcf
Cohesion': 0 psf
Phi': 24°

Fat Clay
Model: Mohr-Coulomb
Unit Weight: 116 pcf
Cohesion': 0 psf
Phi': 24°

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36°



Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
East Abutment
Spencer's Method - Seismic Analysis

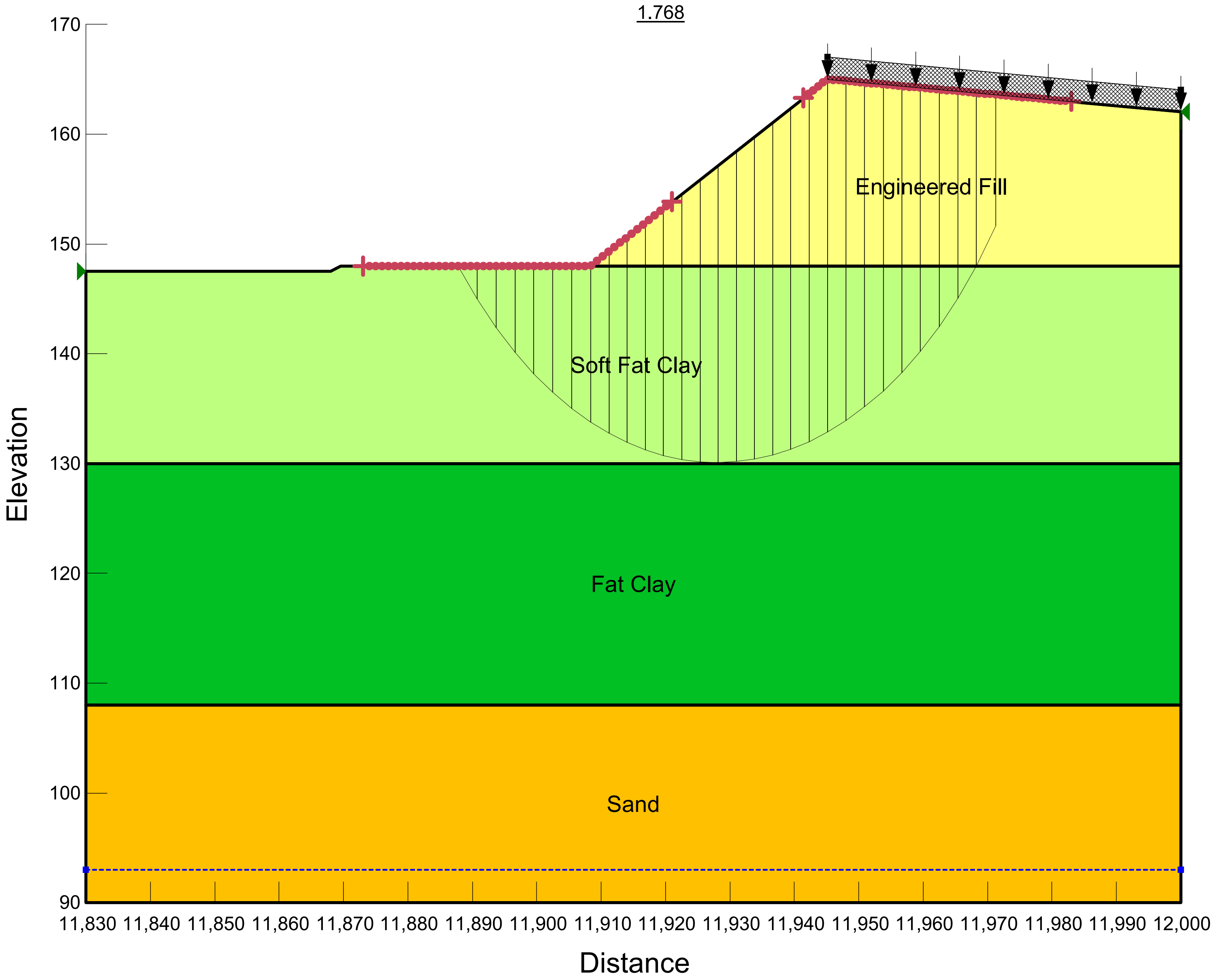
Ks = 0.073

Engineered Fill
Model: Undrained (Phi=0)
Unit Weight: 120 pcf
Cohesion': 1,500 psf

Soft Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 800 psf

Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 1,200 psf

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36°



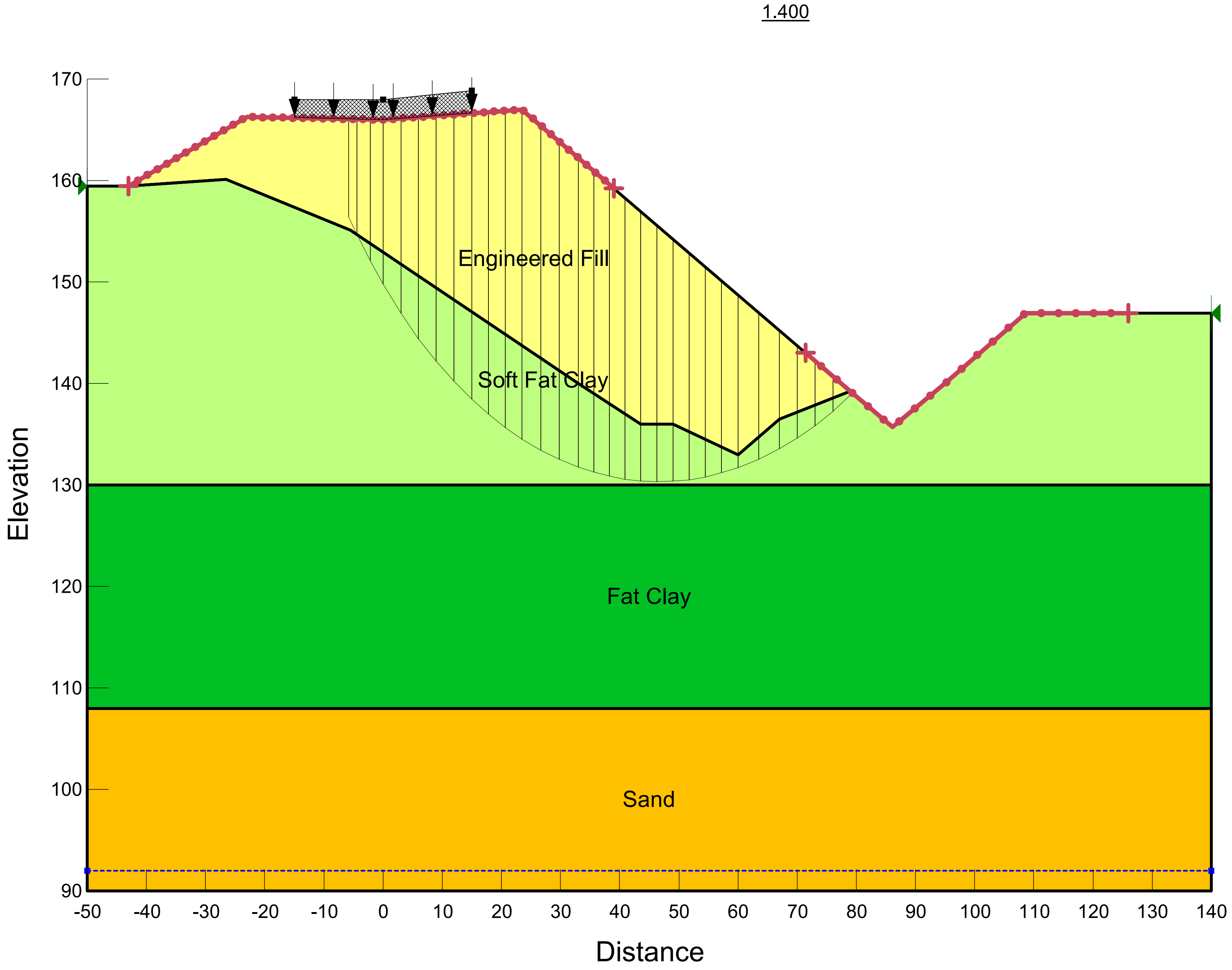
Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
Hwy 278 Station 119+20
Spencer's Method - Short Term Analysis

Engineered Fill
Model: Undrained (Phi=0)
Unit Weight: 120 pcf
Cohesion': 1,500 psf

Soft Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 800 psf

Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 1,200 psf

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36º



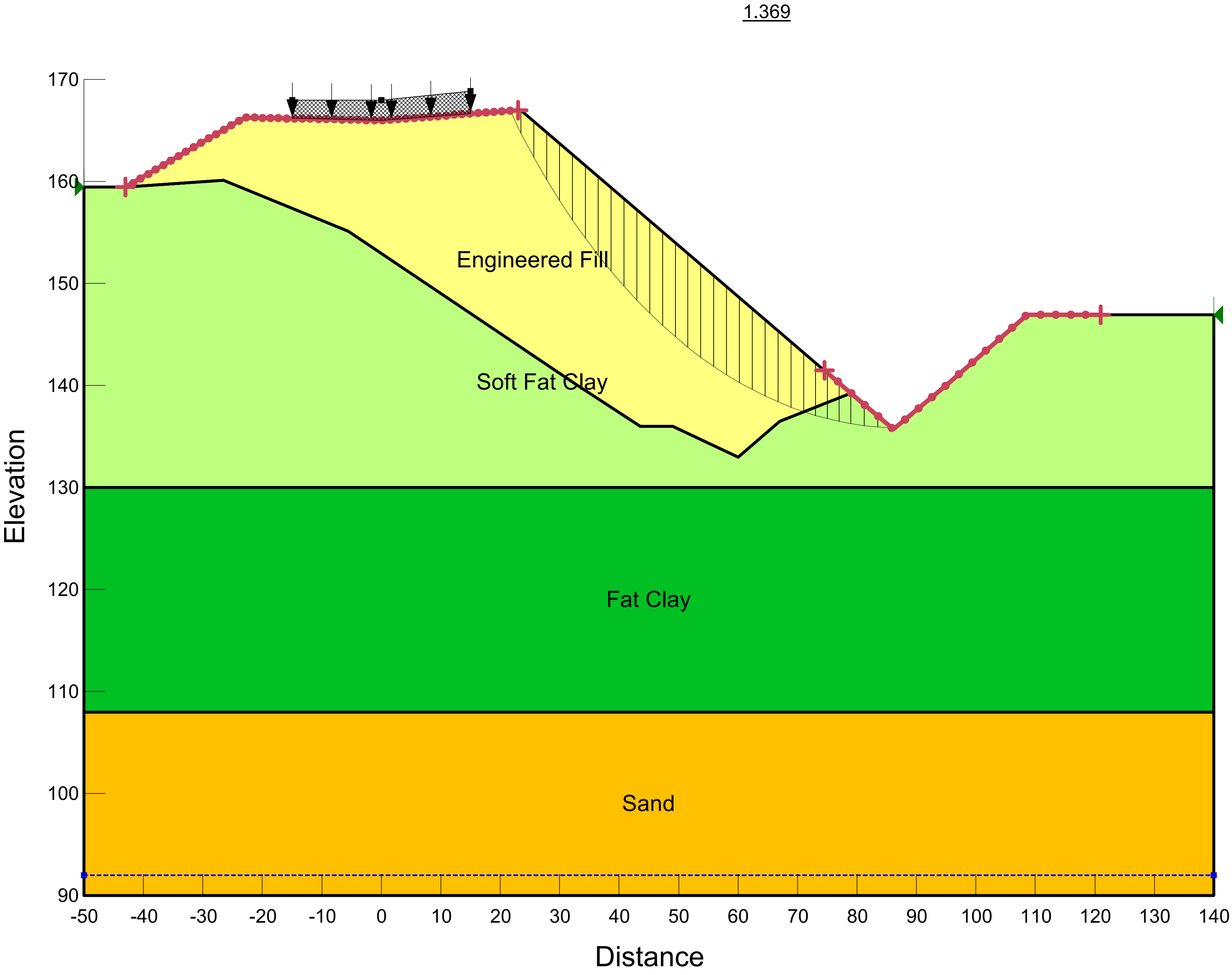
Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
Hwy 278 Station 119+20
Spencer's Method - Long Term Analysis

Engineered Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 50 psf
Phi': 30°

Soft Fat Clay
Model: Mohr-Coulomb
Unit Weight: 116 pcf
Cohesion': 0 psf
Phi': 24°

Fat Clay
Model: Mohr-Coulomb
Unit Weight: 116 pcf
Cohesion': 0 psf
Phi': 24°

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36°



Hwy 278 Over Union Pacific Railroad
Desha County, Arkansas
Hwy 278 Station 119+20
Spencer's Method - Seismic Analysis

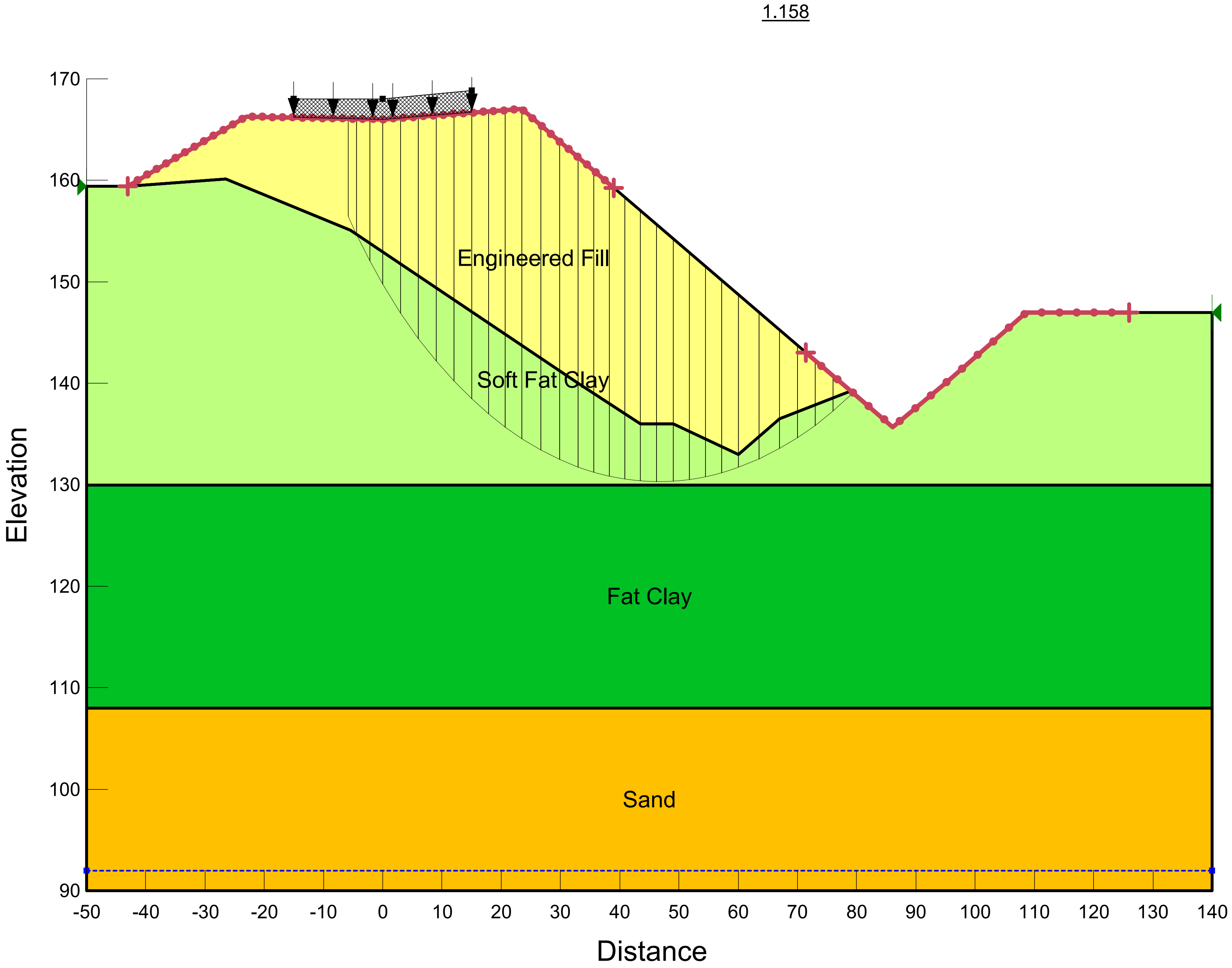
Ks = 0.073

Engineered Fill
Model: Undrained (Phi=0)
Unit Weight: 120 pcf
Cohesion': 1,500 psf

Soft Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 800 psf

Fat Clay
Model: Undrained (Phi=0)
Unit Weight: 116 pcf
Cohesion': 1,200 psf

Sand
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion': 0 psf
Phi': 36º



APPENDIX G - SOIL PARAMETERS

WEST ABUTMENT – BORINGS B-1 THROUGH -5										
ZONE	SOIL TYPES	ELEVATION (FEET)		WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)**
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)		
1	Engineered Fill	159*	156	120	1,500	--	50	30	0.007	200
2	Fat Clay	156	126	116	800	--	--	24	0.01	100
3	Fat Clay	126	103	116	1,200	--	--	24	0.007	500
4	Sand	103	92	123	--	36	--	36	--	90
5	Sand	92	73	123	--	34	--	34	--	60
6	Sand	73	45	122	--	36	--	36	--	60

* Based on Base of Footings at Abutment

**Pounds per cubic inch

Groundwater assumed at approximately El. 92

BENTS – BORINGS B-4 THROUGH -12

ZONE	SOIL TYPES	ELEVATION (FEET)		WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)**
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	FROM		
1	Fat Clay	140*	134	116	700	--	--	24	0.01	100
2	Fat Clay	134	130	116	700	--	--	24	0.01	100
3	Fat Clay	130	109	116	1,000	--	--	24	0.01	100
4	Sand	109	99	121	--	36	--	36	--	60
5	Sand	99	84	122	--	34	--	34	--	60
6	Sand	84	46	123	--	36	--	36	--	60

* Based on Base of Footing at Bent 9.

**Pounds per cubic inch

Groundwater assumed at approximately El. 134

EAST ABUTMENT – BORINGS B-11 THROUGH -14										
ZONE	SOIL TYPES	ELEVATION* (FEET)		WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)**
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	FROM		
1	Engineered Fill	159*	153	120	1,500	--	50	30	0.007	200
2	Fat Clay	153	130	116	800	--	--	24	0.01	100
3	Fat Clay	130	108	116	1,200	--	--	24	0.007	500
4	Sand	108	92	122	--	36	--	36	--	90
5	Sand	92	46	122	--	36	--	36	--	60

*. Based on Base of Footings at Abutment

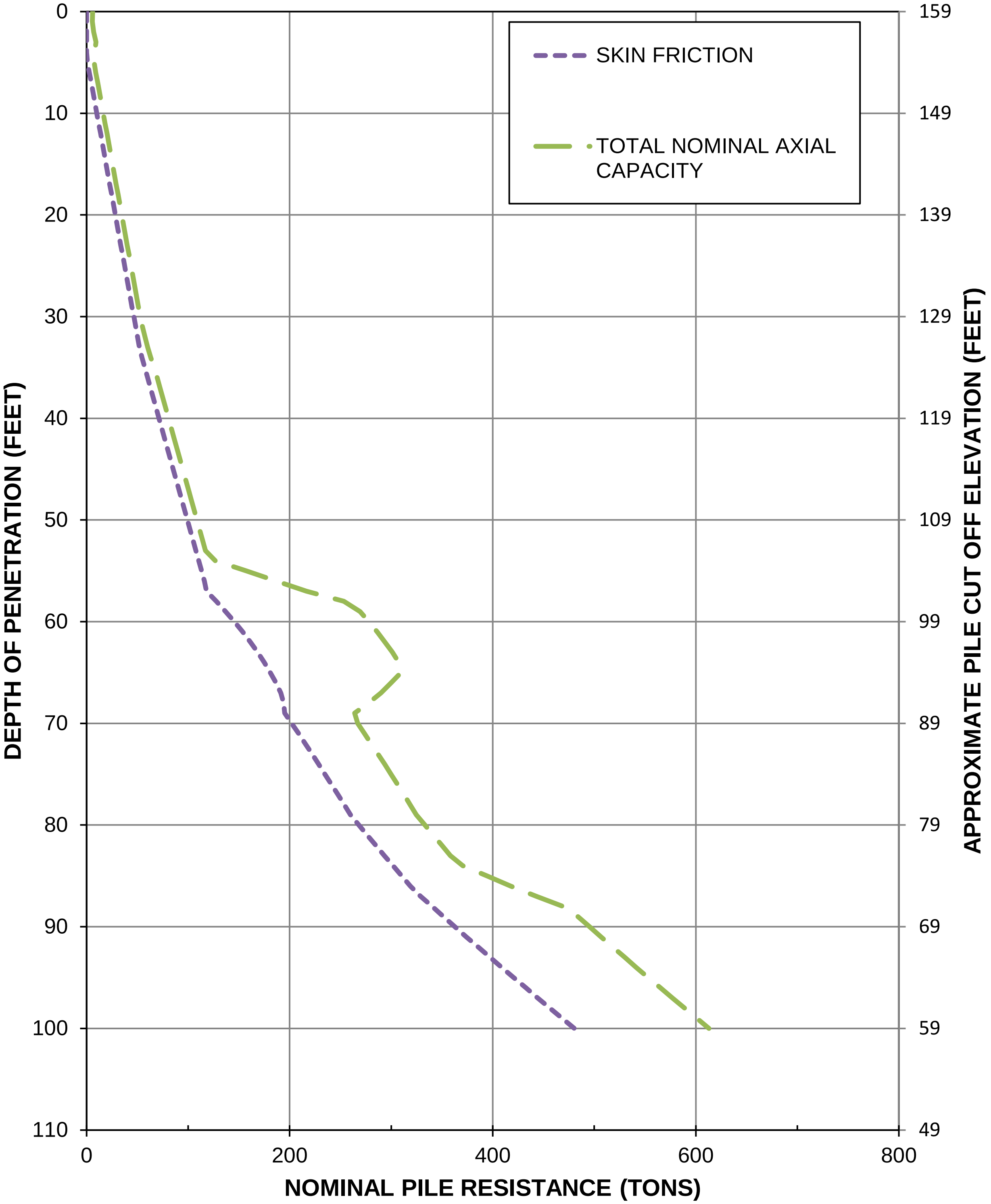
**Pounds per cubic inch

Groundwater assumed at approximately El. 92

APPENDIX H - NOMINAL RESISTANCE CURVES

WEST ABUTMENT
HWY 278 OVER UNION PACIFIC RAILROAD

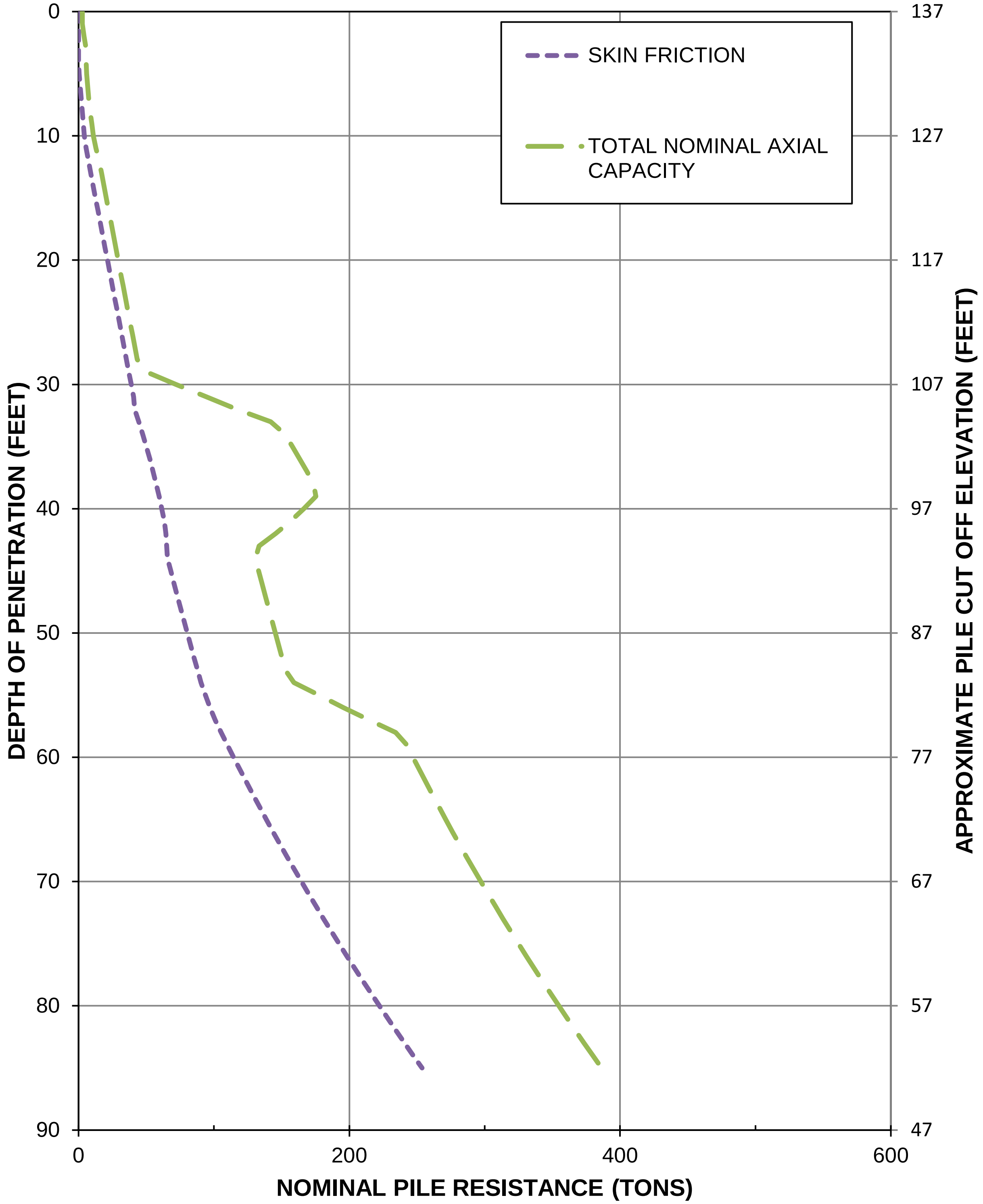
NOMINAL RESISTANCE CURVES
DRIVEN 18 INCH, CLOSED-ENDED, PIPE PILES



GEOTECHNOLOGY PROJECT NUMBER J033659.01

BENT LOCATIONS
HWY 278 OVER UNION PACIFIC RAILROAD

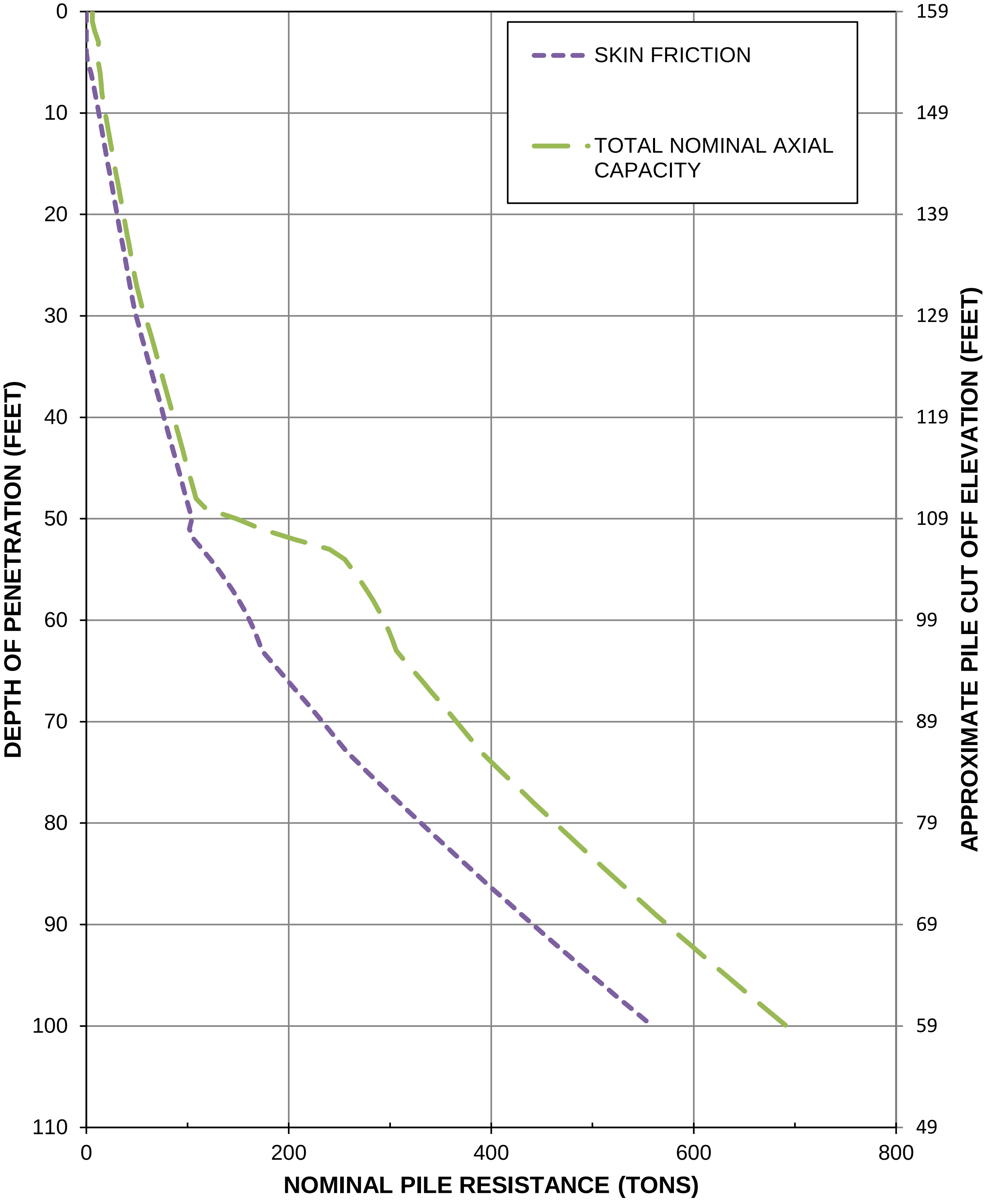
NOMINAL RESISTANCE CURVES
DRIVEN 18 INCH, CLOSED-ENDED, PIPE PILES



GEOTECHNOLOGY PROJECT NUMBER J033659.01

EAST ABUTMENT
HWY 278 OVER UNION PACIFIC RAILROAD

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



GEOTECHNOLOGY PROJECT NUMBER J033659.01