ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 101120

FEDERAL AID PROJECT NO. NHPP-0011(60)

HWY. 141 STRS. & APPRS. (S)

STATE HIGHWAY 141 **SECTION** 5

IN CLAY **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.

GEOTECHNICAL REPORT
HWY. 141 OVER DITCH NO. 8
STR. AND APPRS. (S)
CLAY COUNTY, ARKANSAS

ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT NO. 101120

Prepared for:

ARKANSAS DEPARTMENT OF TRANSPORTATION (ARDOT)
LITTLE ROCK, ARKANSAS

Prepared by:

GEOTECHNOLOGY, LLC
MEMPHIS, TENNESSEE

Date:

NOVEMBER 20, 2023

Geotechnology Project No.:

J042991.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

GEOTECHNOLOGY
A UES Company

November 20, 2023

Mr. Paul Tierney
Geotechnical Engineer
Arkansas Department of Transportation (ARDOT)
PO Box 2261
Little Rock, Arkansas 72203

Re: Geotechnical Report
Hwy. 141 Over Ditch No. 8 Str. and Apprs. (S)
Clay County, Arkansas
ARDOT Project No. 101120
Geotechnology Project No. J042991.01

Dear Mr. Tierney:

Presented in this report are the results of the geotechnical exploration performed by Geotechnology, LLC 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, LLC

No. 10815
ASHRAF S. ELSAYED

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Copies submitted: Client (email)

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GEOTECHNOLOGY
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GEOTECHNICAL REPORT
HWY. 141 OVER DITCH NO. 8 STR. AND APPRS. (S)
CLAY COUNTY, ARKANSAS

November 20, 2023 | Geotechnology Project No. J042991.01

1.0 SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design and construction of the proposed replacement of the existing Bridge No. M3566 over Ditch No. 8 along Highway 141 (Hwy. 141) in Clay County, Arkansas. The referenced project includes the construction of a new bridge to replace the existing Bridge No. M3566. It is our understanding the anticipated foundation type for support of the new bridge will be 16-inch, closed-ended driven pipe piles at the external (abutment) bents and 18- or 20-inch, closed-ended driven pipe piles at the interior bents as provided by ARDOT. The project location is shown on Figure 1 included in Appendix B.

The recommendations presented in this report are based on the geology, provided plans and project information, and the results of the geotechnical exploration. Results of the borings, Cone Penetration Test (CPT) sounding, in-situ testing, sampling, and laboratory testing are included in the report. A total of four borings and one seismic CPT sounding were performed in the vicinity of the site as shown on Figure 2 included in Appendix B. The boring logs and CPT sounding plot, 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.

2.0 GENERAL INFORMATION

Planned Modifications

The existing two-lane, approximately 93-foot-long, 28-foot-wide, two-span Hwy. 141 Bridge No. M3566 over Ditch No. 8 will be replaced by a two-lane, 106-foot-long, 30-foot-wide, three-span bridge. The centerline of the bridge will be shifted approximately 50 feet east of the existing bridge. It is our understand the bridge will be constructed in two phases: in Phase 1 the existing Bridge No. M3566 will remain in use during construction of the replacement bridge; in Phase 2 the existing bridge will be demolished, and traffic will be shifted to the new bridge.

Riprap is planned along the abutment slopes based on the provided preliminary plans; however, the thickness of proposed riprap was not detailed. The abutment slopes are anticipated to be two horizontal units for every vertical unit (2H:1V) and side slopes are anticipated to be 3H:1V. Up to 9 feet of fill will be required to reach design grades.

Topography

The proposed Hwy. 141 bridge replacement is located in Clay County, Arkansas. According to the provided plans[1], existing elevations at the south and north abutments of the proposed bridge location are approximately El 294 and El 292, respectively, which slope down to the bottom of Ditch No. 8 at approximately El 277.

Drainage

Drainage at the site consists of Ditch No. 8 which is aligned east-west. The drainage system in the project area consists of the Cache Watershed. The Cache Watershed, in turn, is part of the overall drainage system of the White River Basin.

Geology

Clay County is located in northeast Arkansas in the Mississippi Embayment. The Mississippi embayment is a trough-like depression dipping southward along an axis approximately following the Mississippi River. The site is located on the western extent of a linear upland within the embayment known as Crowley's Ridge that trends south-southwest. The site geology generally consists of alluvial deposits of clay and silt underlain by fine-grained sand.

3.0 GEOTECHNICAL EXPLORATION

Cone Penetration Testing

One seismic cone penetration testing (CPT) sounding was performed in the proposed bridge northern approach for continuous soil data collection and to measure the average shear wave velocity. The CPT sounding was performed to a depth of approximately 100 feet.

CPT-1 was advanced using a 20-ton, track-mounted Vertek direct-push rig on May 3, 2023. The data were collected using a Vertek 15 square-centimeter end area, seismic piezometric cone with a u_2 pore pressure location (behind the cone) following the procedures outlined in ASTM D3441 and D5778. A plot of the CPT measurements is presented in Appendix D along with interpreted soil behavior types. Seismic CPT tests were performed in CPT-1 at approximately 1-meter intervals to collect shear wave velocity data. A plot of the shear wave velocity profile is presented in Figure 3 in Appendix B.

Drilling and Sampling

A total of four borings were drilled to an approximate depth of 100 feet at select locations in the proposed bridge approaches and through the existing bridge deck. The borings were drilled on May

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[1] *Arkansas Department of Transportation Soil Boring Request for Proposed Bridge, Highway 141 Over Ditch No. 8, Hwy. 141 Strs. & Apprs (S), Clay County Route 151 Section 5, Job 101120*. Provided by Arkansas Department of Transportation, dated August 2022.

2 through 5, 2023 using a rotary drill rig (CME 750X), hollow-stem augers, and wet rotary methods. Sampling procedures included Standard Penetration Test (SPT) and thin-wall (Shelby) tube methods. SPT's were conducted at 2.5, 5, and 10-foot depth intervals using an automatic hammer. 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 along with an explanation of the terms and symbols used on the boring logs. Included on each boring log are elevation 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 and CPT sounding plot 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 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 CPT sounding plot 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.

4.0 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), one-dimension consolidation, direct shear, consolidated-undrained triaxial compression (CU), pH, and soil resistivity test results are also provided in Appendix E. 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
One-Dimensional Consolidation	D 2435	T 216
Direct Shear	D 3080	T 236
Consolidated-Undrained Triaxial Compression	D 4767	T 297
pH of Soil	D 4972	T 289
Soil Electrical Resistivity	G 57	T 288

The boring logs and CPT plot 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.

5.0 SUBSURFACE CONDITIONS

Subgrade Materials

Borings B-1 and -4 and the CPT-1 sounding were performed in the alignment of the proposed Bridge. Borings B-2 and -3 were drilled through the existing bridge deck and mudline of Ditch No. 8. Underlying the topsoil in Borings B-1 and -4, the soils generally consisted of an upper layer of predominately fine-grained soils. Below the fine-grained soils and below the mudline of Ditch No. 8, the soils generally consisted of predominately coarse-grained soils that extended to the 100-foot maximum depth of exploration in the borings and CPT-1. The boring logs and CPT sounding plot, with more detailed descriptions, are included in Appendix C and D, respectively. Laboratory testing was used to determine the AASHTO classifications as presented in Appendix G.

The upper, fine-grained soils were classified as low plasticity “lean” clay (CL), A-4, A-6, and A-7-6; and silt (ML), A-4. The fine-grained soils ranged from very soft to stiff in consistency.

The lower, coarse-grained soils were classified as poorly graded sand (SP), A-3; poorly graded sand with clay (SP-SC), A-2-6; and poorly graded sand with silt (SP-SM), A-3. Other coarse-grained soils were visually classified as poorly graded gravel (GP) and clayey sand (SC). The coarse-grained soils ranged from loose to very dense conditions.

Groundwater

Groundwater was not encountered in the upper 50 feet of the borings during drilling operations; groundwater levels may have been masked due to the use of wet rotary methods. Definitive groundwater levels were not interpreted in the CPT sounding locations; however, we have assumed a groundwater depth of approximately 35 feet in CPT-1 based on pore pressure data recorded in the sounding. Groundwater levels could vary significantly over time due to the effects of seasonal variations in precipitation, water level and recharge rate of Ditch No. 8, or other factors not evident at the time of exploration.

6.0 ENGINEERING EVALUATION, ANALYSIS, AND RECOMMENDATIONS

Site Preparation and Earthwork

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

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

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

Side Slopes. Existing slopes steeper than 4H:1V should be benched prior to placing new fill. Slope ratios of 2H:1V are proposed for abutment slopes. Slope ratios of 3H:1V or flatter are recommended for all cut and fill side slopes along the proposed alignment.

Cut Areas. It is our understanding up to 6 feet of cut will be required at the abutments. Based on the stratigraphy, excavations for pile cap foundations will terminate in lean clay. 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 D698/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[2], and should meet the minimum requirements set forth in ARDOT's Special Provision[3] (SP) dated March 1, 2022. Soils classifying as AASHTO A-4 or better are considered to be select fill. Fine-grained "silt-clay" soils (A-4 through A-6) should have a maximum LL of 45 and a PI between 8 and 20 percent. Coarse-grained "sandy" soils used for embankment fills should have a minimum PI of 5 to lower potential for erosion. Fill materials should be free from organic matter, debris, or other deleterious materials, and have a maximum particle size of 2 inches.

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[2] A-6 soils or better as determined by ARDOT.

[3] Special Provision "Compacted Embankment", developed by ARDOT, dated March 1, 2022.

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 3 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 a 70% of the minimum relative density as evaluated from the maximum and minimum index densities measured by ASTM D4253 and D4254, respectively. The upper 3 feet of soil beneath the base of pavement should be compacted to 75% of the minimum relative density.

Fill Placement on Slopes. In areas that require fill to be placed on slopes, benching of existing slopes should be performed during placement of new fill. Fill on the sloped areas should begin from the toe of the slope and proceed upward, placing new fill on horizontal benches. Bench shelves should be 8 to 10 feet wide, and bench faces should be 1 to 2 feet in height. Fill lifts should be keyed into the slope to reduce the potential of a slip place between the new fill and existing soils. Fill slopes should be constructed by extending the compacted fill beyond the planned profile of the slope and then trimming the slope to the desired configuration.

Moisture Considerations. Maintaining the moisture content of bearing and subgrade soils within the acceptable range is important during and after construction. Silty and clayey 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. Positive drainage should be established to promote drainage of surface water away from the roadway.

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 earthquake 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”, ninth edition (2020).

AASHTO LRFD 2020 Seismic Site Classification and Seismic Design Parameters

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 3.

Table 3. Seismic Design Parameters (7% Probability of Exceedance in 75 years).

Latitude 36.333161°N/Longitude 90.444211°W		
Category/ Parameter	Designation/ Value	Reference
Seismic Zone	3	AASHTO LRFD 2020 Table 3.10.6-1
Seismic Site Class	D	AASHTO LRFD 2020 Table 3.10.3.1-1
S_S	0.726g	Ground motion parameters obtained from a computer program supplied with the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions using the indicated latitude and coordinates of the project site and the seismic site class based on boring data.
S_1	0.183g	
F_a	1.219	
F_v	2.066	
F_{PGA}	1.109	
t_s	0.428	
t_0	0.086	
S_{DS}	0.885g	
S_{D1}	0.379g	
PGA	0.391g	
As	0.434g	

Site-Specific Ground Motion Assessment

A site-specific study was performed for the project site to develop a site-specific seismic design response spectrum. The process included seismic cone testing to measure the shear wave velocity of the soil profile, performing probabilistic seismic hazard analyses to determine probabilistic consistent magnitudes and epicentral distances, generation of time histories, and evaluation of the near-surface soil effects. Data measured using the seismic cone resulted in an average shear wave velocity in the upper 100 feet ($V_{S,100}$) of CPT-1 of 783 feet per second as shown on Figure 3 (Shear Wave Velocity Profile) in Appendix B.

The results of the shear wave velocity measurements indicate that the site is a Site Class D, "stiff soil", profile based on a $V_{S,100}$ of 783 feet per second. According to the results of the site-specific seismic study, the recommended site-specific design accelerations are presented in Table 4.

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

Parameter	Value
S_{DS}	0.757g
S_{D1}	0.532g
MCE_G	0.360g

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 depth of the water table, SPT N-values, and information collected in CPT-1. The laboratory data included USCS classification and soil unit weight. An earthquake magnitude (M_W) of 7.7 with a probability of exceedance of 7% in 75 years was considered. A site peak ground acceleration of 0.360g was utilized as obtained from the site-specific seismic study. Groundwater was set at a depth of approximately 35 feet as indicated on the CPT-1 plot in Appendix D.

Subsurface conditions (as characterized by field and laboratory data) and earthquake characteristics were used to estimate the safety factors against liquefaction in each layer, as well as the associated dynamic settlement during the design seismic event. Based on the analysis, the potential for liquefaction at the site is relatively low in the upper 50 feet.

Lateral Spreading. Lateral spreading is triggered and sustained by earthquake ground motions. Based on our seismic slope stability analyses, it is our professional opinion that the potential for lateral spreading is low at the site.

Approach Embankment Settlement

Settlement analyses of natural soils were performed to assess fill-induced settlement for the approaches. Based on the provided preliminary plans, up to approximately 9 feet and 7 feet of fill will be required at the southern and northern approaches, respectively, to bring the site to design grade. For settlement analyses, we have assumed cohesive, engineered fill will be used for the fill material. The results of the settlement due to fill placement are shown in Table 5. If grade changes will require the placement of additional fill, Geotechnology should be contacted to perform additional settlement analyses for fill-induced settlement at the approaches.

Table 5. Summary of Estimated Settlement.

Southern Abutment (Exterior Bent No. 1)				Northern Abutment (Exterior Bent No. 4)			
Max Fill (feet)	Estimated Settlement (inches)			Max Fill (feet)	Estimated Settlement (inches)		
	Immediate	Long-Term (Consolidation)	Total		Immediate	Long-Term (Consolidation)	Total
9	1½	½	2	7	1½	½	2

The bent numbers presented in Table 5 are in reference to the bent number designations presented on the provided preliminary plans. Based on review of the preliminary plans, the bents are numbered from 1 to 4 such that Bent No. 1 is at the southern abutment.

Discussion of Fill-Induced Settlement. The results of the settlement analyses indicate immediate and long-term (primary) consolidation settlement at the approaches. We anticipate the immediate settlement to occur shortly after fill placement and practical completion of consolidation to occur within 2 to 4 weeks after fill placement.

Global Stability

Geotechnology performed stability analyses for deep-seated, global failure of bridge abutment slopes using the computer program SLIDE2. Short-term, long-term, and seismic conditions were considered using the Spencer method to compute factors of safety for the proposed slopes.

Calculated minimum factors of safety are summarized in the following table. Minimum required factors of safety for the proposed bridge were based on the ARDOT Minimum Acceptable Factors of Safety as provided by ARDOT using a seismic operational class of “Other”. A pseudo-static seismic acceleration of 0.180g, corresponding to one-half the peak ground acceleration (per FHWA Publication HI-99-012) was utilized.

Fill material consists of engineered fill as described in the Fill Materials section of this report; a groundwater elevation of approximately El 257 feet, as noted in CPT-1, was utilized for the short-term and seismic condition analyses and a design highwater elevation of El 290.5, as obtained from the preliminary plans, was used for the long-term condition analyses. Section profiles with critical slip surfaces and utilized soil parameters are presented in Appendix H for the selected analyses. The analysis models did not consider the effect of foundation piles driven at the abutments or riprap placed on the abutment slopes that would provide additional restraining force to stabilize the slopes.

Table 6. Results of Slope Stability Analyses.

Location	Description	Slope Height (ft.)	Calculated Factor of Safety		
			Short-Term Static[a,c]	Long-Term Static[a,d]	Seismic[b,c]
Southern Abutment STA 217+45.20	2H:1V	18.5	4.03	1.37	2.63
	Cut Slope				
Northern Abutment STA 218+51.20	2H:1V	18.5	4.10	1.37	2.61
	Cut Slope				

[a] Target factor of safety = 1.3, approximately equivalent to a global stability resistance factor = 0.75, as provided by ARDOT.

[b] Target factor of safety = 1.1, approximately equivalent to a global stability resistance factor = 0.9, as provided by ARDOT.

[c] Based on a groundwater elevation of approximately El 257; approximately 35 feet below existing ground surface at the abutments.

[d] Based on a Ditch No. 8 design highwater elevation of El 290.5 as obtained from the preliminary plans provided by ARDOT.

Fill material used for construction of the embankments will be required to meet the criteria established in the Special Provision dated March 1, 2022 provided by ARDOT.

Deep Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2020).

It is our understanding the proposed foundation type for the bridge will be driven, closed-ended pipe piles; a closed-ended pipe pile size of 16-inch diameter was considered for the end bents (Bents 1 and 4) and a closed-ended pipe pile size of 18- and 20-inch diameter was considered for the interior bents (Bents 2 and 3) as provided by ARDOT. Geotechnology should be notified if other foundation types or sizes are to be considered. Based on the provided information, we have assumed a pile cap elevation of approximately El 289 for the exterior bents and an existing ground surface elevation of El 278 for the interior bents. Soil parameters, including LPILE lateral load analysis parameters, for each bent are included in Appendix I.

Nominal resistance curves showing axial resistance from skin friction and total axial capacity (skin friction + end bearing) for Bents 1 through 4 are presented in Appendix J. Nominal static resistances at each bent for the driven, closed-ended pipe piles are presented in Table 7. Uplift (tension) capacities may be calculated using the resistance provided by skin friction.

Table 7. Nominal Axial Resistance of Driven Closed-Ended Pipe Piles.

Location	Pile Diameter (inches)	Embedment Length (feet)	Nominal Static Resistance (tons)		
			Skin Friction	End Bearing	Compression Total
Bent 1[a] (South Abutment)	16	60	252	147	399
		70	340	147	486
		80	435	147	581
Bent 4[a] (North Abutment)		60	245	127	372
		70	332	147	479
		80	427	147	574
Bents 2 & 3[b] (Interior Bents)	18	60	273	186	458
		70	362	186	548
		80	460	186	646
	20	60	303	229	532
		70	402	229	631
		80	511	229	740

[a] Embedment length referenced from a pile cap elevation of El 289.
[b] Embedment length referenced from approximate ground surface elevation of El 278.

Resistance Factors. Resistance factors should be applied to the nominal resistances provided. Based solely on the static analysis methods used to calculate nominal pile resistances, the factors presented in Table 8 may be applied.

Table 8. Resistance Factors Based on Static Analysis Methods.

Deep Foundation and Condition	Clay		Sand	
	Side Resistance	End-Bearing	Side Resistance	End-Bearing
Nominal Compressive Resistance of Single Pile	0.35	0.35	0.45	0.45
Uplift Resistance of Single Pile	0.25	--	0.35	--

Based on the AASHTO LRFD (2020) Table 10.5.5.2.3-1, a higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 9.

Table 9. 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 (2020) 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 (2020) 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 (2020) sections 10.7.3.9 and 10.7.3.11.

Driven Pile Construction Considerations. Minimum hammer energies required to drive the piles were not evaluated for the proposed foundations. If minimum hammer energy evaluations are required, Geotechnology should be contacted to perform analyses for the required minimum hammer energies for driving piles.

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 (2020) 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 (2020) 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 (2020) Table 10.5.5.2.3-1 or 2 percent of the production piles, whichever results in a larger number of tests. We recommend that the identified piles be tested at the end of initial drive (EOID) and a restrike performed at a minimum seven days after EOID.

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

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

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

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

Lateral Resistance. The lateral resistance of pile foundations depends on the lengths and dimensions of the foundations 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 I 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 (2020) 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 pile or pile group is 1.0.

Downdrag

The AASHTO LRFD (2020) suggests that soil settlement relative to a pile of 0.4-inch or greater could produce downdrag on pile foundations. Downdrag occurs as the soil strata move downward relative to 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 analyses performed for the 9-foot maximum fill placement at the abutments and approaches, up to 2 inches of settlement is predicted. The settlement due to fill placement is estimated to occur during fill placement and be practically complete within 2 to 4 weeks after placement of fill. Pile driving should not begin for at least 4 weeks following completion of fill placement to allow time for settlement to be essentially complete. Piles driven after fill placement prior to completion of settlement may be subject to drag loads as the soil below the fill consolidates due to the weight of the fill.

Downdrag Due to Dynamic Settlement. Based on the low liquefaction potential in the upper 50 feet at the site, liquefaction-induced drag loads and settlement were not considered.

Corrosion Potential

In addition to laboratory soil classification and strength testing, soil resistivity testing was also conducted. The purpose of soil resistivity testing is to provide soil data for use by a structural engineer for analysis of any necessary protection of the piling, concrete, reinforcing steel, etc. Corrosion and deterioration protection requirements and guidelines for piling are set forth in Section 10.7.5 of the AASHTO LRFD Bridge Design Specifications. The corrosion and deterioration testing results are summarized in Table 10 and are included in Appendix E.

Table 10. Results of pH and Soil Resistivity Testing.

Boring	Sample No.	Sample Depth (feet)	pH	Soil Resistivity (ohm-cm)
B-1	SS-6	13.5	7.39	518.70
	SS-12	38.5	9.16	4,220.85
B-2	SS-7	23.5	7.65	5,248.56
	SS-11	43.5	8.73	5,061.60
B-3	SS-6	18.5	8.49	5,751.30
	SS-12	48.5	8.61	5,985.00
B-4	SS-15	53.5	8.71	7,250.40
	SS-16	58.5	8.66	4,670.01

The following soil conditions should be considered as indicative of a potential for steel pile deterioration or corrosion:

- Resistivity values less than 2,000 ohms-cm; or
- pH less than 5.5.

The following soil conditions should be considered as indicative of a potential for steel reinforcement corrosion or deterioration:

- Resistivity values less than 3,000 ohms-cm; or
- pH less than 5.5.

Interpretation of the data and corrosion protection of the bridge structural components should be performed by the design team.

7.0 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.

8.0 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.*

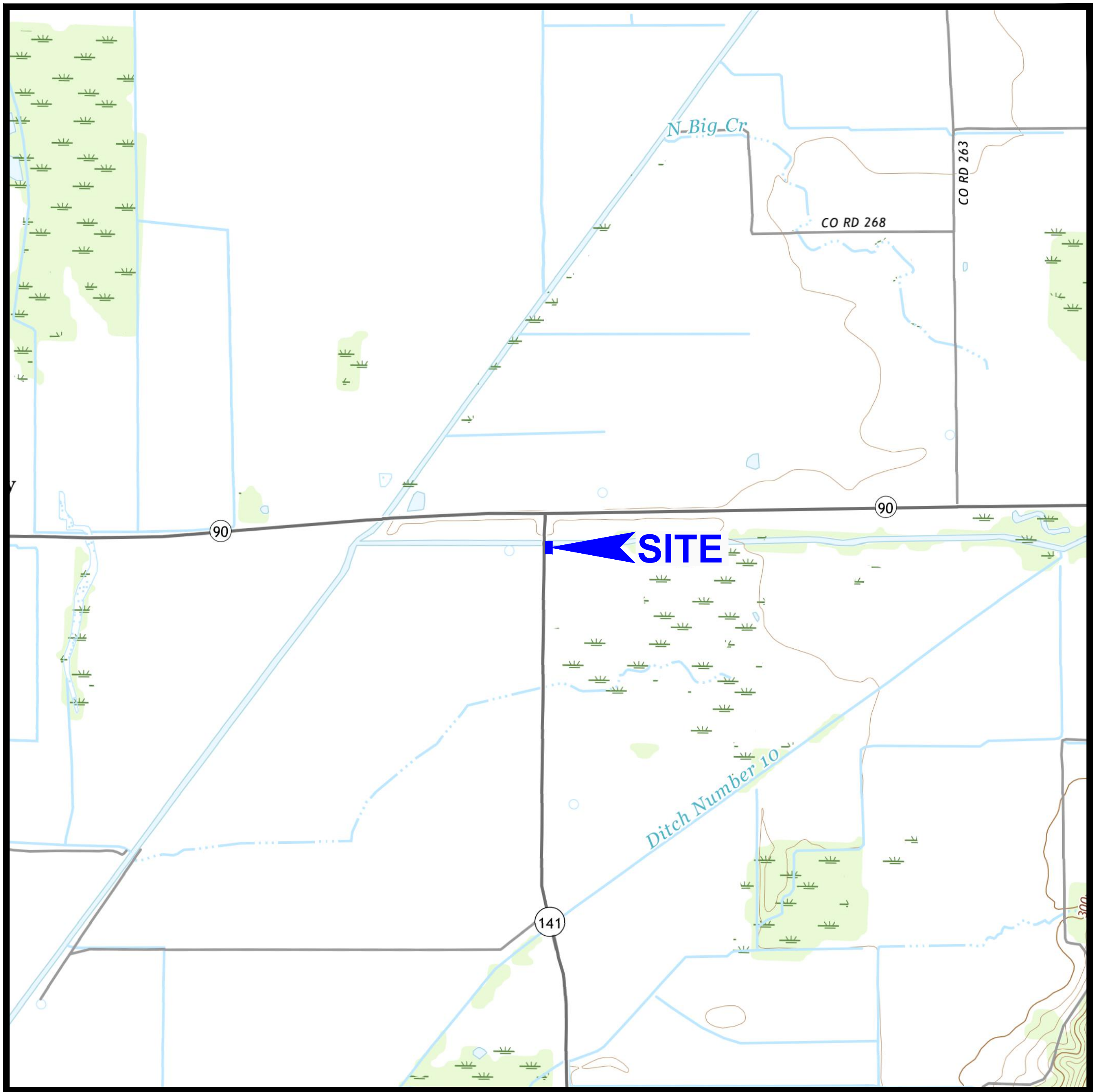
Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

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 a 7.5 minute U.S.G.S. map for Boydsville, Arkansas quadrangle, last revised in 2020.

0 2,000 4,000

SCALE IN FEET

N

Drawn By: WAH	Ck'd By: JDM	App'vd By: ASE
Date: 5-4-23	Date: 8-10-23	Date: 8-10-23

GEOTECHNOLOGY
A UES Company

ARDOT 101120 Hwy. 141
Over Ditch No. 8 Str. & Apprs. (S)
Clay County, Arkansas

SITE LOCATION AND TOPOGRAPHY

Project Number
J042991.01

FIGURE 1

NOTES

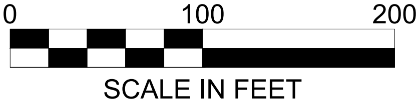
1. Plan adapted from a November 9, 2021 aerial photograph courtesy of Google Earth and a drawing dated August 2022, titled "Soil Boring Request for Proposed Bridge", prepared by Arkansas State Highway Commission.

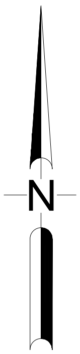
2. Borings and CPT sounding were located in the field with reference to site features and are shown approximate only.

LEGEND

● Boring Location

⊙ CPT Sounding Location





Drawn By: WAH	Ck'd By: JDM	App'vd By: ASE
Date: 5-4-23	Date: 8-10-23	Date: 8-10-23

GEOTECHNOLOGY
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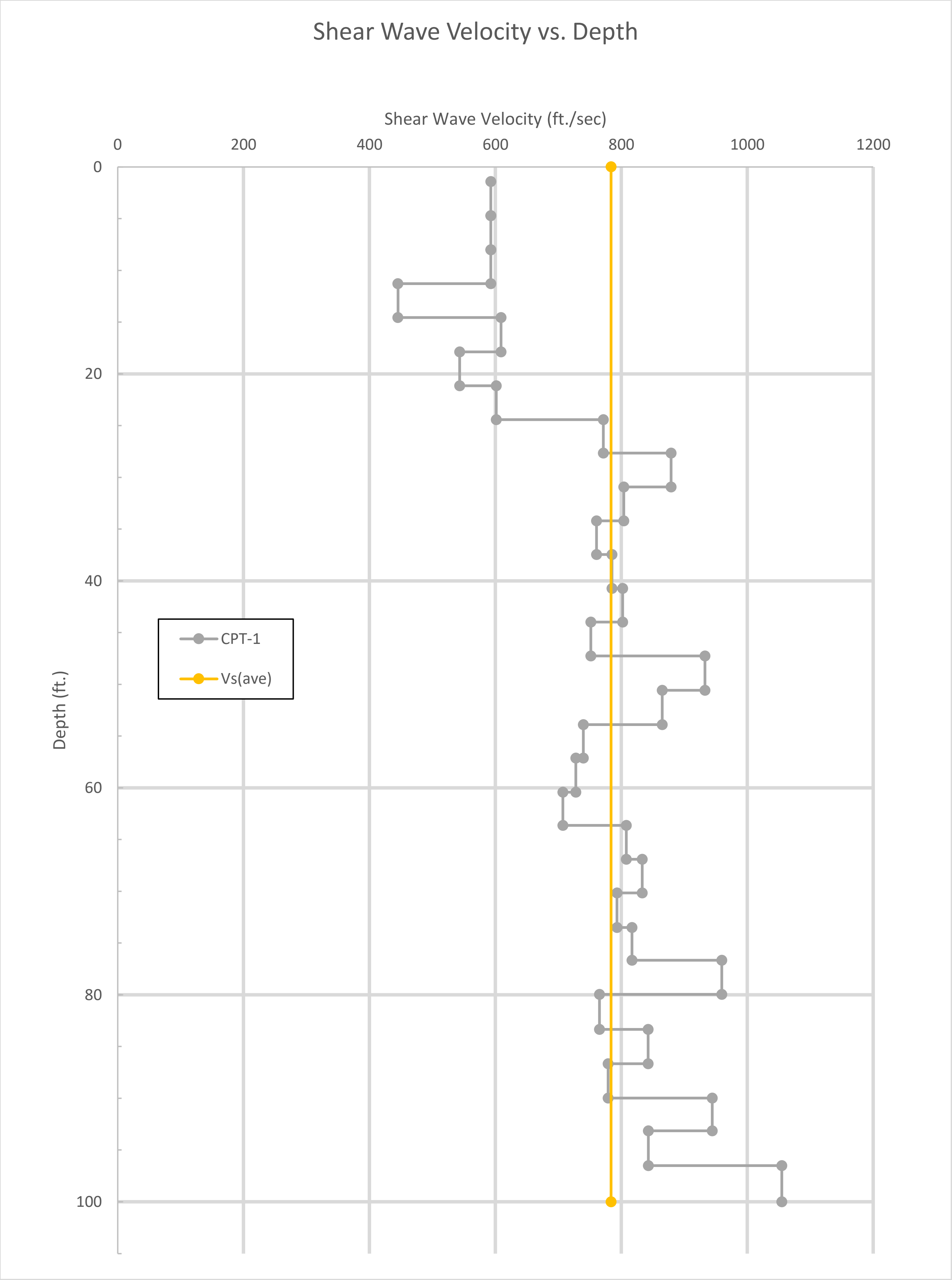
ARDOT 101120 Hwy. 141
Over Ditch No. 8 Str. & Apprs. (S)
Clay County, Arkansas

AERIAL PHOTOGRAPH OF SITE AND BORING LOCATIONS

Project Number
J042991.01

FIGURE 2

Figure 3 - Shear Wave Velocity Profile



APPENDIX C – BORING INFORMATION

Boring Logs

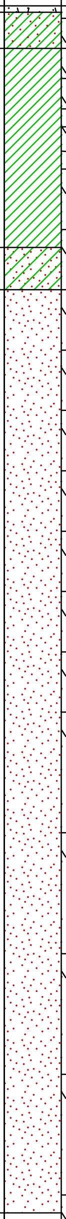
Boring Log Terms and Symbols

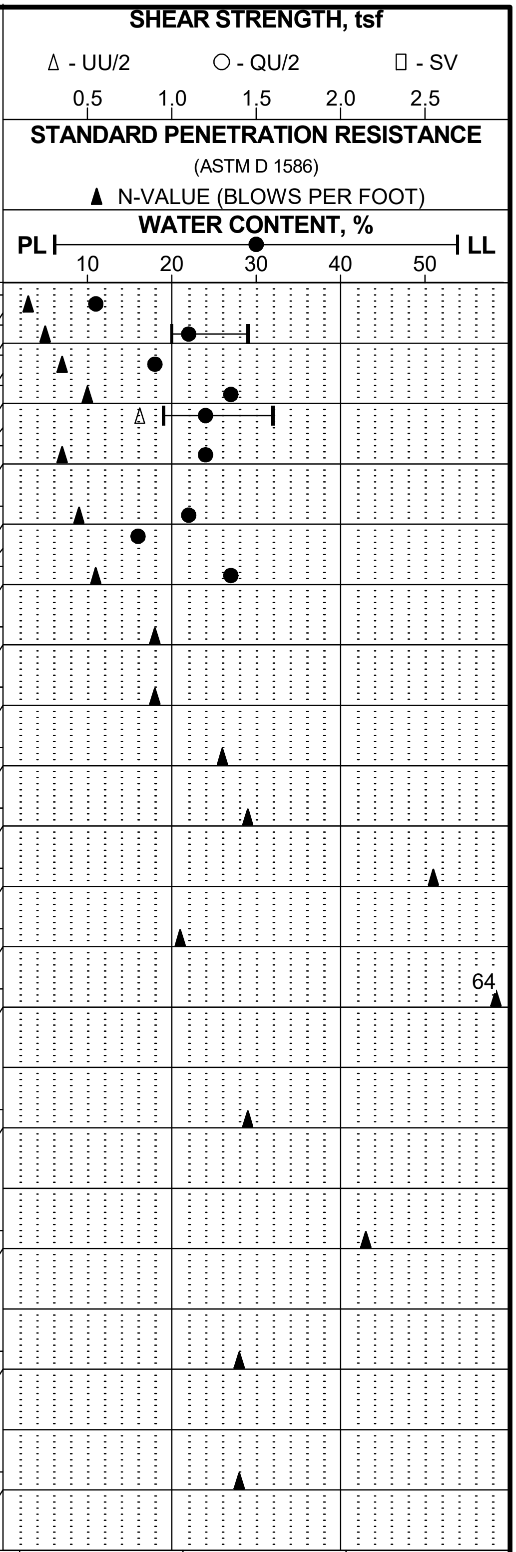
Surface Elevation: 294

Datum NAVD88

Completion Date: 5/5/23

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			
		Soft, brown, sandy, LEAN CLAY, trace gravel and roots - CL		2-2-1	SS1
5	289			1-2-3	SS2
		Medium stiff to stiff, brown to gray, LEAN CLAY - (CL)		2-3-4	SS3
10	284	trace roots		1-3-7	SS4
		trace roots and sand		98	ST5
15	279			3-3-4	SS6
				3-3-6	SS7
20	274	Loose, gray, SAND with clay - SP-SC 8.3% passing No. 200 sieve			ST8
25	269	Medium dense to very dense, gray SAND - SP 4.2% passing No. 200 sieve		3-4-7	SS9
30	264			8-8-10	SS10
35	259	3.2% passing No. 200 sieve		7-8-10	SS11
40	254			9-12-14	SS12
45	249			8-13-16	SS13
50	244			18-25-26	SS14
55	239			9-10-11	SS15
60	234			17-31-33	SS16
65	229				
70	224			8-14-15	SS17
75	219				
80	214			16-19-24	SS18
85	209				
90	204			13-13-15	SS19
95	199				
100	194	trace gravel		13-14-14	SS20
		Boring terminated at 100 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 25 FEET
KJB DRILLER WEC LOGGER
CME 750X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 81 %

REMARKS: Shelby Tube ST-8 too sandy for strength testing.

Drawn by: JSH	Checked by: JDM	App'vd. by: ASE
Date: 5/8/23	Date: 8/10/23	Date: 8/10/23

GEOTECHNOLOGY
A UES Company

ARDOT G002 101120 Hwy. 141 Over Ditch No. 8 Str. and Apprs(S)
Clay County, Arkansas

LOG OF BORING: B-1

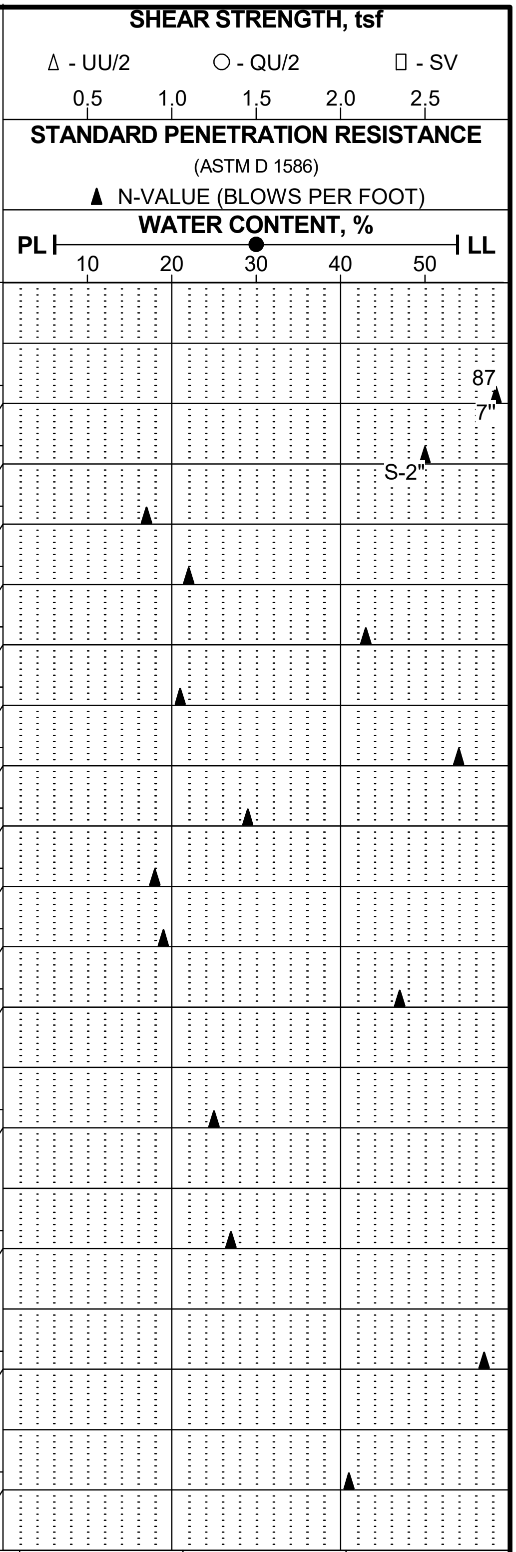
Project No. J042991.01

Surface Elevation: 278

Datum NAVD88

Completion Date: 5/4/23

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Casing set in creek bed to 8.5 feet.			
5	273				
10	268	Very dense to medium dense, brown to gray SAND - SP		22-37-50/1"	SS4
15	263			50/2"	SS5
20	258	2.5% passing No. 200 sieve		5-7-10	SS6
25	253			9-8-14	SS7
30	248			15-20-23	SS8
35	243			9-9-12	SS9
40	238			24-27-27	SS10
45	233			20-14-15	SS11
50	228	3.8% passing No. 200 sieve		11-9-9	SS12
55	223			8-7-12	SS13
60	218			21-23-24	SS14
65	213				
70	208			8-11-14	SS15
75	203				
80	198			14-12-15	SS16
85	193				
90	188			15-25-32	SS17
95	183				
100	178			22-22-19	SS18
		Boring terminated at 100 feet.			



GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM __ FEET
KJB DRILLER JWD LOGGER
CME 750X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 81 %

REMARKS: Approximately 22 feet from bridge deck to creek mudline. Sampling started at 8.5 feet to set casing.

Drawn by: JSH	Checked by: JDM	App'vd. by: ASE
Date: 5/10/23	Date: 8/10/23	Date: 8/10/23

GEOTECHNOLOGY
A UES Company

ARDOT G002 101120 Hwy. 141 Over Ditch No. 8 Str. and Apprs(S)
Clay County, Arkansas

LOG OF BORING: B-2

Project No. J042991.01

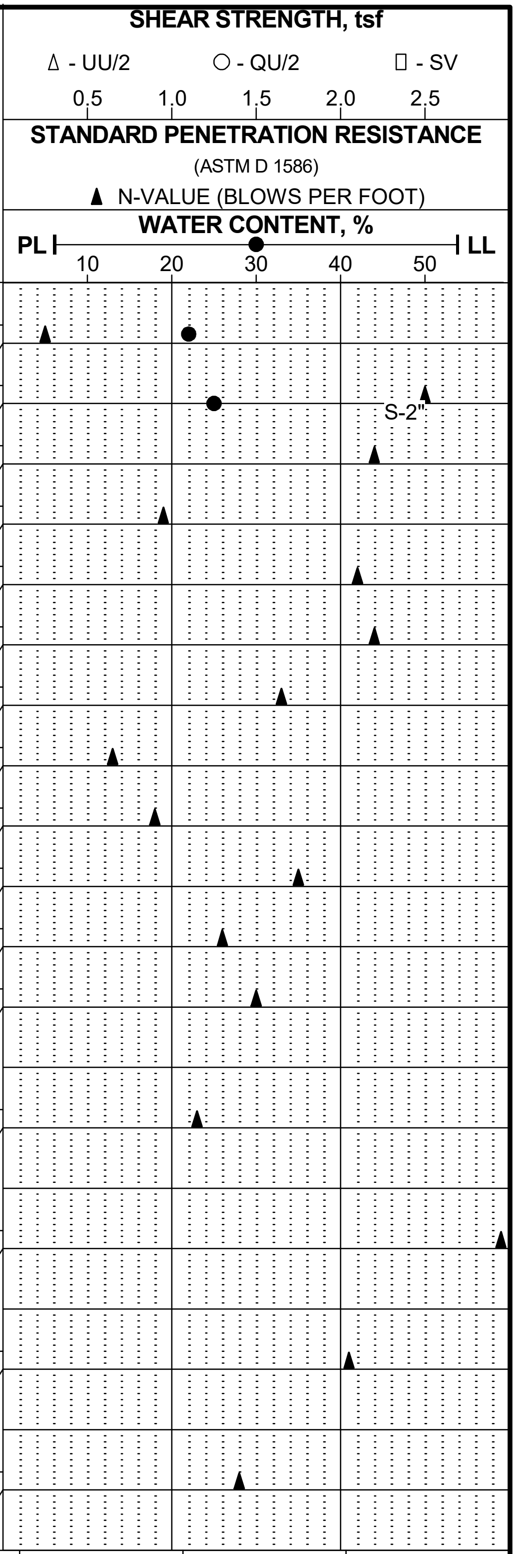
NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

Surface Elevation: 278

Datum NAVD88

Completion Date: 5/3/23

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Casing set in creek bed to 3.5 feet.			
5	273	Loose, brown SAND, some organics - SP		7-4-1	SS2
10	268	Very dense, brown and gray GRAVEL - GP		50/2"	SS4
15	263	Dense to medium dense, brown to gray SAND - SP		14-20-24	SS5
20	258			6-9-10	SS6
25	253			12-22-20	SS7
30	248			11-22-22	SS8
35	243			17-20-13	SS9
40	238	Medium dense, gray SAND with silt - SP-SM 5.5% passing No. 200 sieve		5-6-7	SS10
45	233	Medium dense to very dense, gray SAND - SP 3.0% passing No. 200 sieve		9-8-10	SS11
50	228			8-14-21	SS12
55	223			10-12-14	SS13
60	218			13-14-16	SS14
65	213				
70	208			8-10-13	SS15
75	203				
80	198			12-19-40	SS16
85	193				
90	188			12-17-24	SS17
95	183				
100	178	Boring terminated at 100 feet.		9-12-16	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

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM __ FEET

KJB DRILLER JWD LOGGER

CME 750X DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 81 %

REMARKS: Approximately 22 feet from bridge deck to creek mudline. Sampling started at 3.5 feet to set casing.

Drawn by: JSH	Checked by: JDM	App'vd. by: ASE
Date: 5/10/23	Date: 8/10/23	Date: 8/10/23

GEOTECHNOLOGY
A UES Company

ARDOT G002 101120 Hwy. 141 Over Ditch No. 8 Str. and Apprs(S)
Clay County, Arkansas

LOG OF BORING: B-3

Project No. J042991.01

Surface Elevation: 292

Datum NAVD88

Completion Date: 5/2/23

DEPTH IN FEET	ELEVATION IN FEET
5	287
10	282
15	277
20	272
25	267
30	262
35	257
40	252
45	247
50	242
55	237
60	232
65	227
70	222
75	217
80	212
85	207
90	202
95	197
100	192

DESCRIPTION OF MATERIAL

Topsoil: 6 inches

Soft to very soft, brown, LEAN CLAY - (CL)
- trace roots
- little gravel and trace roots

Medium stiff, brown SILT - (ML)

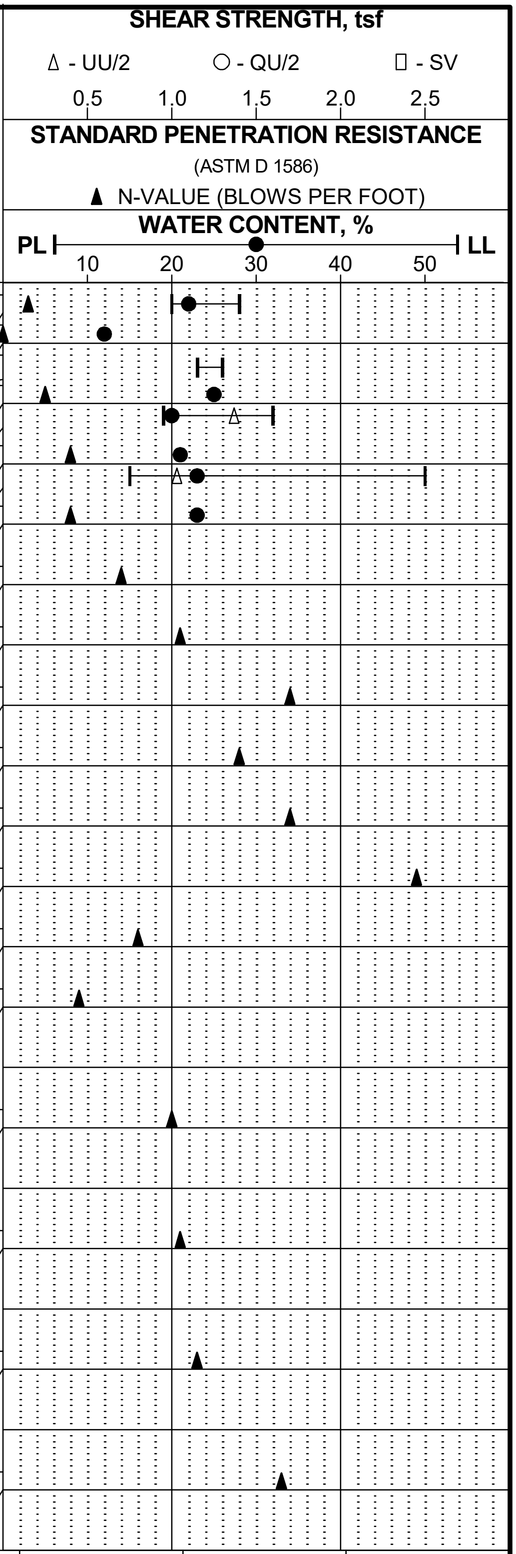
Very stiff to medium stiff, brown to brown and gray, silty, LEAN CLAY - (CL)
- 73.8% passing No. 200 sieve

Medium dense, gray SAND with clay - SP-SC
- 5.4% passing No. 200 sieve

Dense to loose, gray SAND - SP

Boring terminated at 100 feet.

DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
3-1-2	SS1
1-0-0	SS2
	ST3
1-1-4	SS4
107	ST5
3-3-5	SS6
119	ST7
2-3-5	SS8
5-6-8	SS9
4-9-12	SS10
6-13-21	SS11
5-10-18	SS12
4-15-19	SS13
8-21-28	SS14
4-7-9	SS15
4-5-4	SS16
6-7-13	SS17
8-11-10	SS18
5-8-15	SS19
19-19-14	SS20



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
KJB DRILLER JWD LOGGER
CME 750X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 81 %

REMARKS: Shelby Tube ST-3 used for CU test.

Drawn by: JSH	Checked by: JDM	App'vd. by: ASE
Date: 5/10/23	Date: 8/10/23	Date: 8/10/23

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A UES Company

ARDOT G002 101120 Hwy. 141 Over Ditch No. 8 Str. and Apprs(S)
Clay County, Arkansas

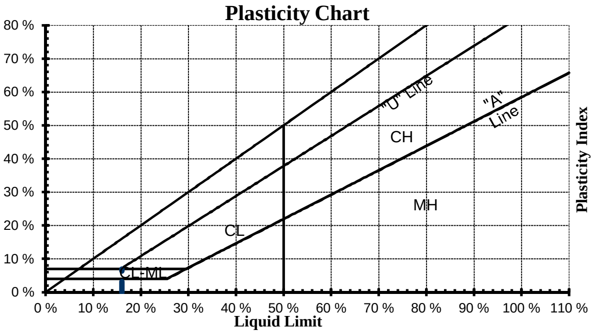
LOG OF BORING: B-4

Project No. J042991.01

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

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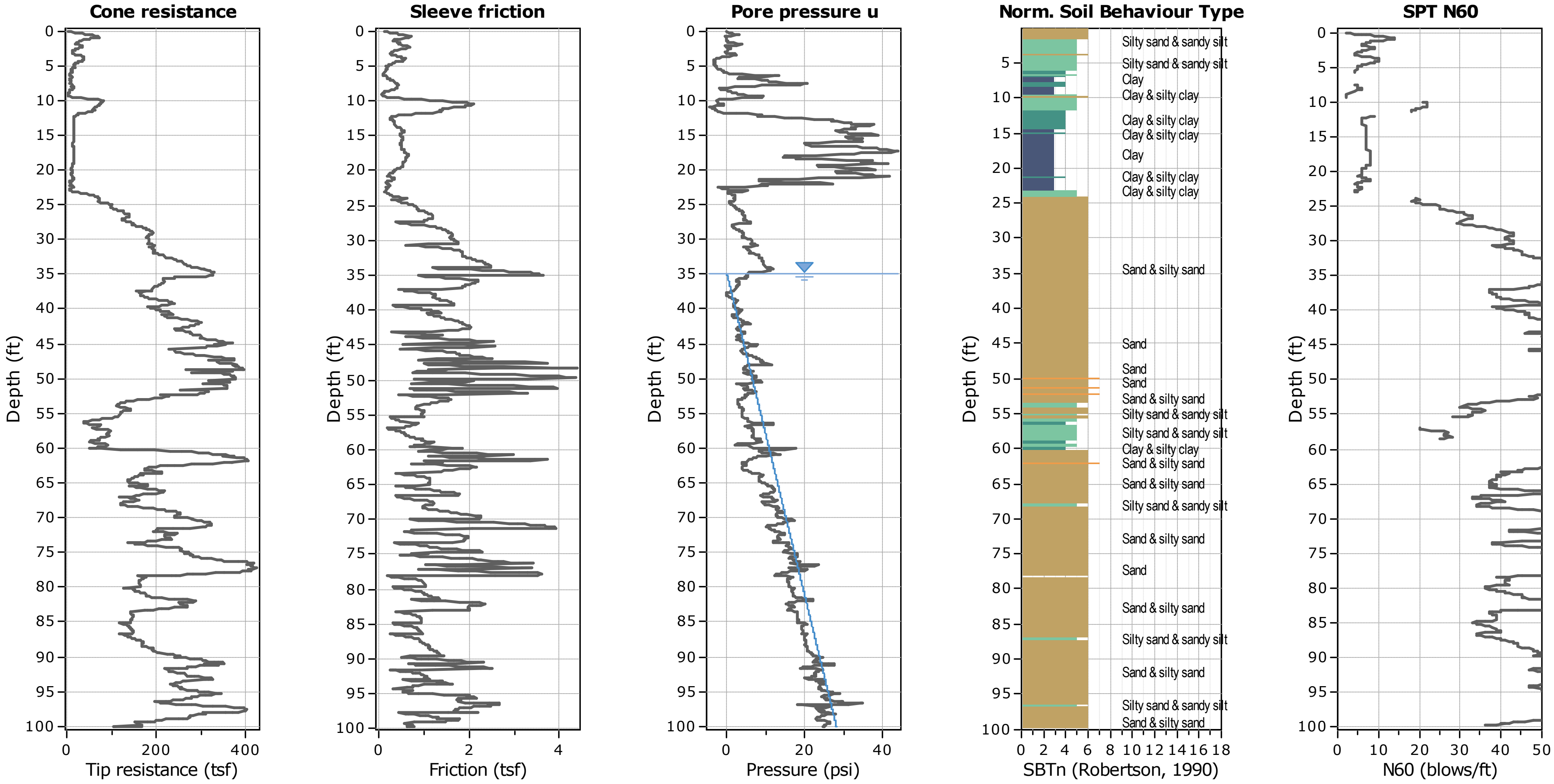
Relative composition and Unified Soil Classification System (USCS) designations are based on visual descriptions and are approximate only. If laboratory tests were performed to classify the soil, the USCS designation is shown in parenthesis.

APPENDIX D – CPT-1 SOUNDING PLOT

Geotechnology, LLC
3312 Winbrook Dr
Memphis, TN 38116
(901) 353-1981

CPT: CPT-1
Total depth: 100.07 ft, Date: 5/3/2023
Coords: lat 36.333161° lon -90.444211°
Cone Type:
Cone Operator: DWJ

Project: **ARDOT G002 101120 Hwy. 141 Over Ditch No. 8 Str. & Apprs. (S)**
Location: **Clay County, Arkansas**



SBTn legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬜ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

APPENDIX E – LABORATORY TEST DATA

Atterberg Limits

Grain Size Distributions

Unconsolidated-Undrained Triaxial Compression

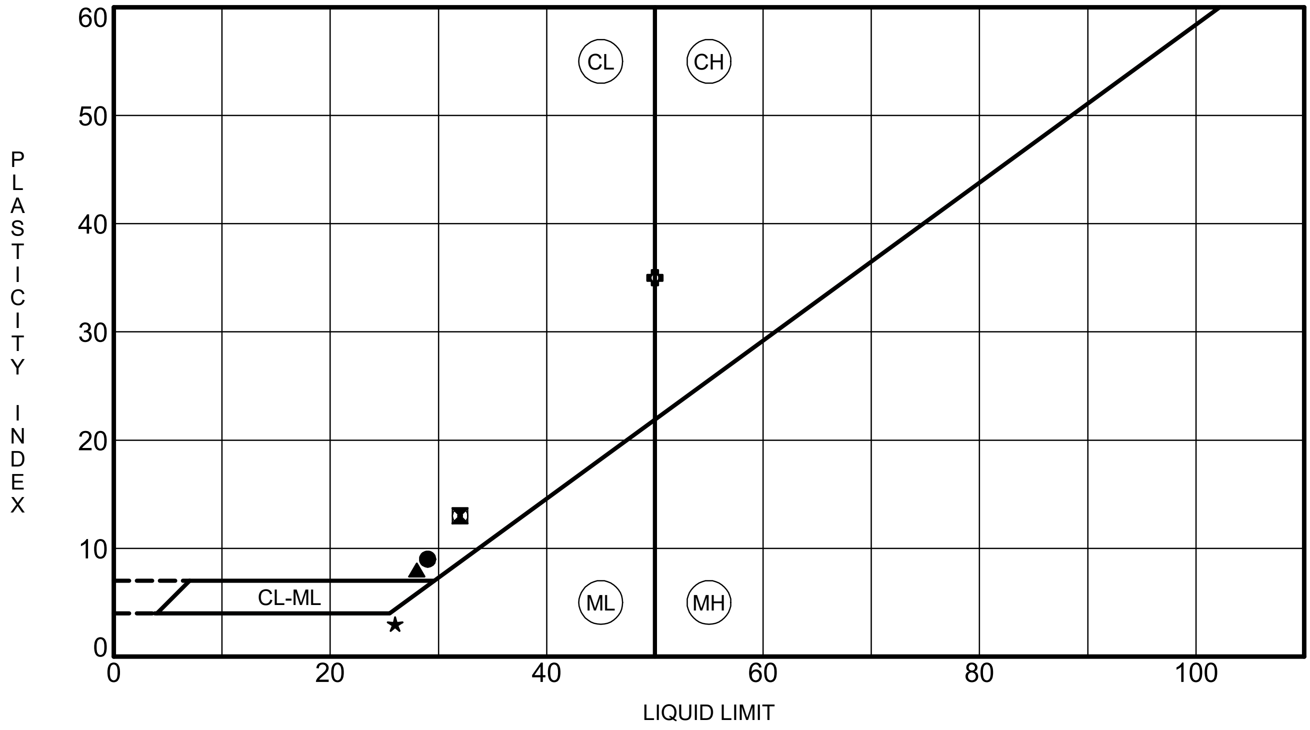
One-Dimensional Consolidation

Direct Shear

Consolidated-Undrained Triaxial Compression

Resistivity

pH



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**3.5**	**29**	**20**	**9**		**LEAN CLAY(CL)**
⧗	**B-1**	**10.0**	**32**	**19**	**13**		**LEAN CLAY(CL)**
▲	**B-4**	**1.0**	**28**	**20**	**8**		**LEAN CLAY(CL)**
★	**B-4**	**6.0**	**26**	**23**	**3**		**SILT(ML)**
⊙	**B-4**	**10.0**	**32**	**19**	**13**		**LEAN CLAY(CL)**
✿	**B-4**	**15.0**	**50**	**15**	**35**		**LEAN CLAY(CL)**

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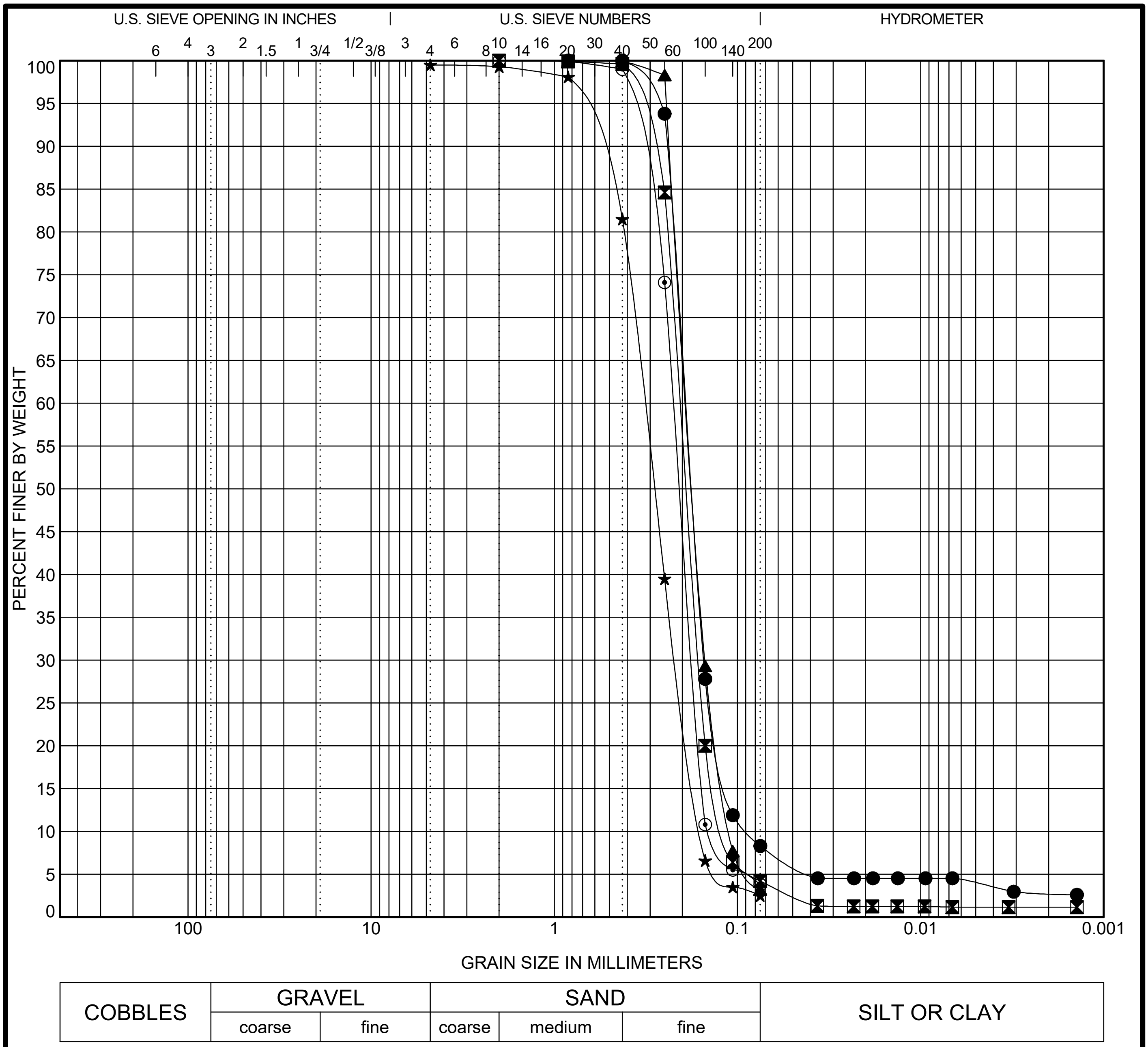
A UES Company

ATTERBERG LIMITS RESULTS

ARDOT G002 101120 Hwy. 141 Over
Ditch No. 8 Str. and Apprs(S)
Clay County, Arkansas
J042991.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**20.0**	**POORLY GRADED SAND with CLAY(SP-SC)**				**1.37**	**2.18**
☒	**B-1**	**23.5**	**POORLY GRADED SAND(SP)**				**1.10**	**1.77**
▲	**B-1**	**33.5**	**POORLY GRADED SAND(SP)**				**1.10**	**1.71**
★	**B-2**	**18.5**	**POORLY GRADED SAND(SP)**				**0.91**	**2.05**
⊙	**B-2**	**48.5**	**POORLY GRADED SAND(SP)**				**0.97**	**1.57**

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**20.0**	**0.84**	**0.192**	**0.153**	**0.088**	**0.0**	**91.7**	**4.4**	**3.9**
☒	**B-1**	**23.5**	**2**	**0.206**	**0.162**	**0.116**	**0.0**	**95.8**	**3.0**	**1.2**
▲	**B-1**	**33.5**	**0.84**	**0.188**	**0.151**	**0.11**	**0.0**	**96.8**	**3.2**	
★	**B-2**	**18.5**	**4.75**	**0.324**	**0.216**	**0.158**	**0.0**	**97.0**	**2.5**	
⊙	**B-2**	**48.5**	**2**	**0.223**	**0.175**	**0.142**	**0.0**	**96.2**	**3.8**	

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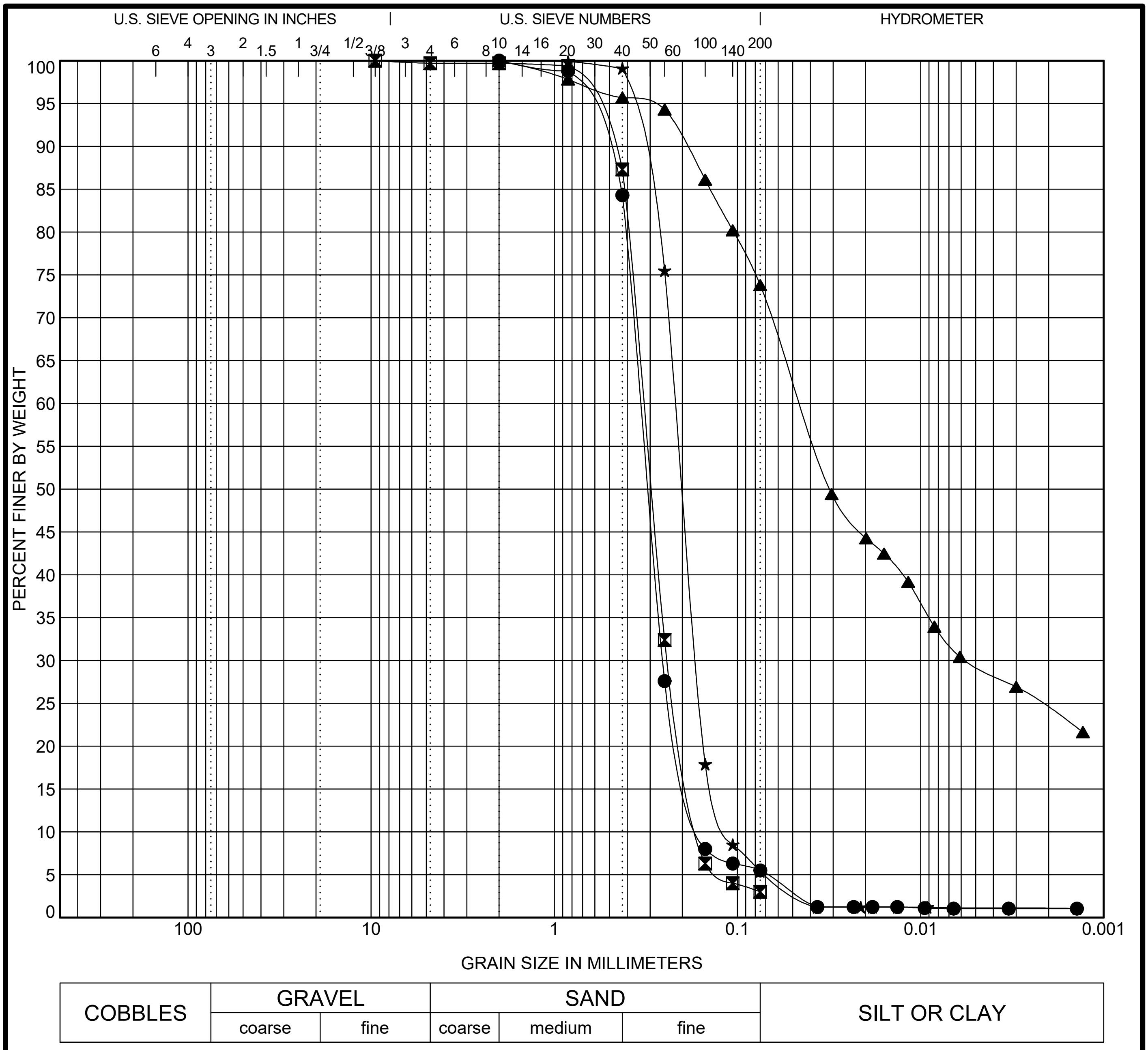
A UES Company

GRAIN SIZE DISTRIBUTION

ARDOT G002 101120 Hwy. 141 Over
Ditch No. 8 Str. and Apprs(S)
Clay County, Arkansas
J042991.01

US GRAIN SIZE J042991.01.GPJ US LAB.GDT 8/10/23



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**B-3**	**38.5**	**POORLY GRADED SAND with SILT(SP-SM)**				**1.22**	**2.14**
☒	**B-3**	**43.5**	**POORLY GRADED SAND(SP)**				**1.08**	**2.02**
▲	**B-4**	**18.5**	**LEAN CLAY(CL)**					
★	**B-4**	**28.5**	**POORLY GRADED SAND with CLAY(SP-SC)**				**1.14**	**1.94**

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-3**	**38.5**	**2**	**0.339**	**0.256**	**0.158**	**0.0**	**94.5**	**4.5**	**1.0**
☒	**B-3**	**43.5**	**9.5**	**0.326**	**0.239**	**0.161**	**0.3**	**96.7**	**3.0**	
▲	**B-4**	**18.5**	**2**	**0.045**	**0.006**		**0.0**	**26.2**	**44.4**	**29.4**
★	**B-4**	**28.5**	**2**	**0.218**	**0.167**	**0.112**	**0.0**	**94.6**	**4.3**	**1.1**

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A UES Company

GRAIN SIZE DISTRIBUTION

ARDOT G002 101120 Hwy. 141 Over
Ditch No. 8 Str. and Apprs(S)
Clay County, Arkansas
J042991.01

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department
DATE: 5/24/2023
PROJECT NO.: J042991.01
PROJECT: ARDOT 2023-2027 Contract TO G002 Highway 141 Bridge over Ditch No. 8, Clay County
LOCATION: Clay County, Arkansas

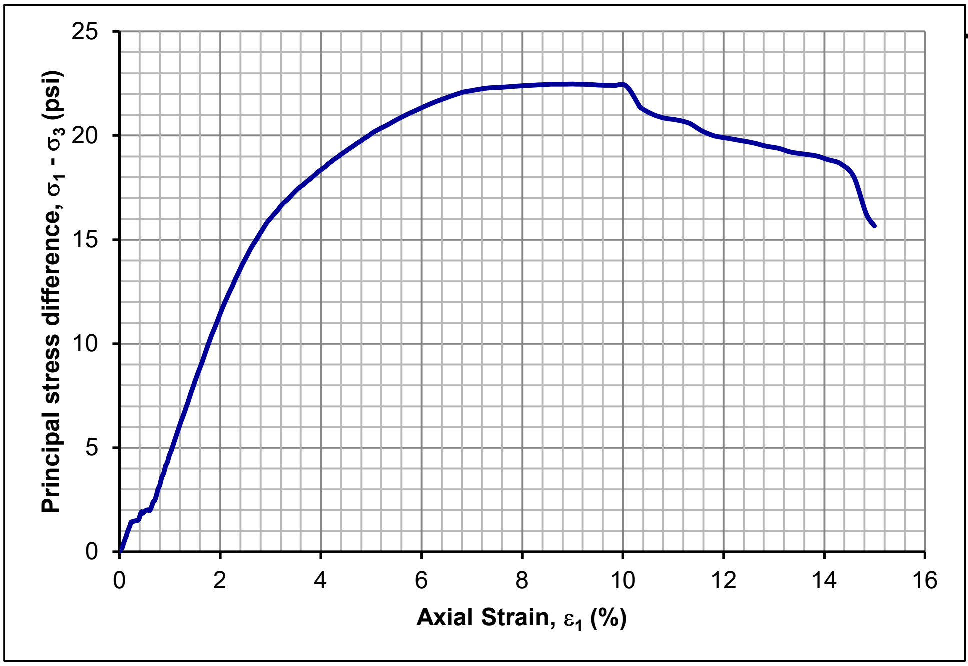
BORING NO.: B-1
SAMPLE NO.: ST-5
DEPTH (ft.): 10.0-12.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Brown Lean Clay

LIQUID LIMIT (%): 32
PLASTIC LIMIT (%): 19
PLASTICITY INDEX (%): 13
USCS: CL

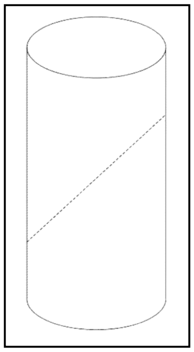
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.85
HEIGHT (in.):	5.98
HEIGHT TO DIAMETER RATIO:	2.10
WET UNIT WEIGHT (pcf):	120.9
DRY UNIT WEIGHT (pcf):	97.5
VOID RATIO:	0.76
MOISTURE CONTENT (%)*:	24.1
DEGREE OF SATURATION (%):	87.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**24.2**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**9.1**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**22.5**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**6.4**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**28.9**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**3,240**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,620**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**N/A**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

GEOTECHNOLOGY
A UES Company

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department
DATE: 5/24/2023
PROJECT NO.: J042991.01
PROJECT: ARDOT 2023-2027 Contract TO G002 Highway 141 Bridge over Ditch No. 8, Clay County
LOCATION: Clay County, Arkansas

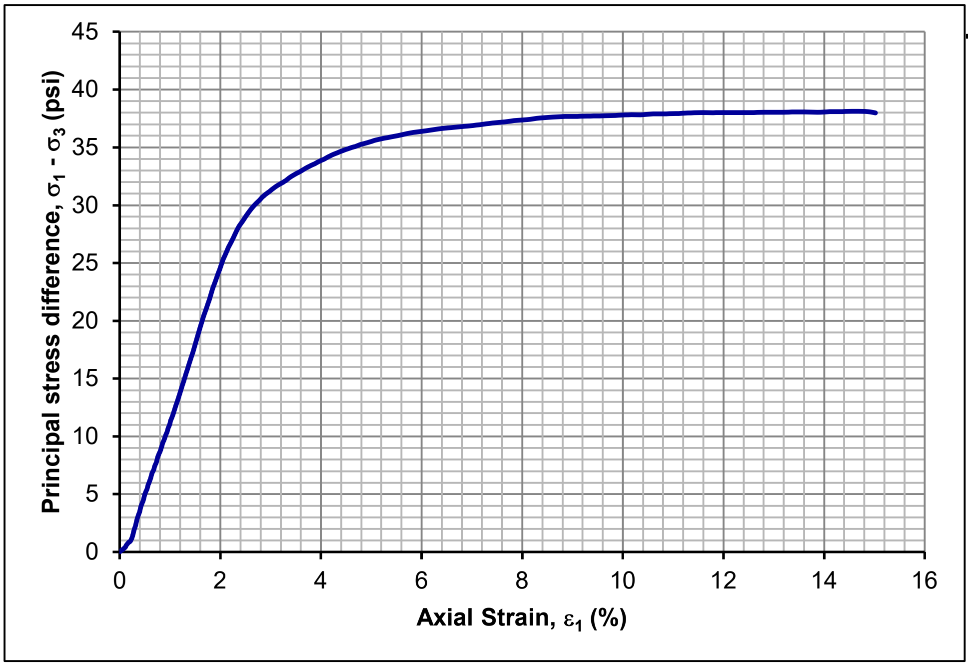
BORING NO.: B-4
SAMPLE NO.: ST-5
DEPTH (ft.): 10.0-12.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Gray Lean Clay

LIQUID LIMIT (%): 32
PLASTIC LIMIT (%): 19
PLASTICITY INDEX (%): 13
USCS: CL

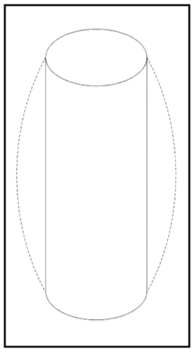
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.83
HEIGHT (in.):	6.02
HEIGHT TO DIAMETER RATIO:	2.13
WET UNIT WEIGHT (pcf):	126.5
DRY UNIT WEIGHT (pcf):	106.7
VOID RATIO:	0.61
MOISTURE CONTENT (%)*:	18.5
DEGREE OF SATURATION (%):	83.8

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**20.4**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.6**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**38.1**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**6.4**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**44.5**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**5,490**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**2,745**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**5,445**



FAILURE SHAPES



REMARKS :

---

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department
DATE: 5/24/2023
PROJECT NO.: J042991.01
PROJECT: ARDOT 2023-2027 Contract TO G002 Highway 141 Bridge over Ditch No. 8, Clay County
LOCATION: Clay County, Arkansas

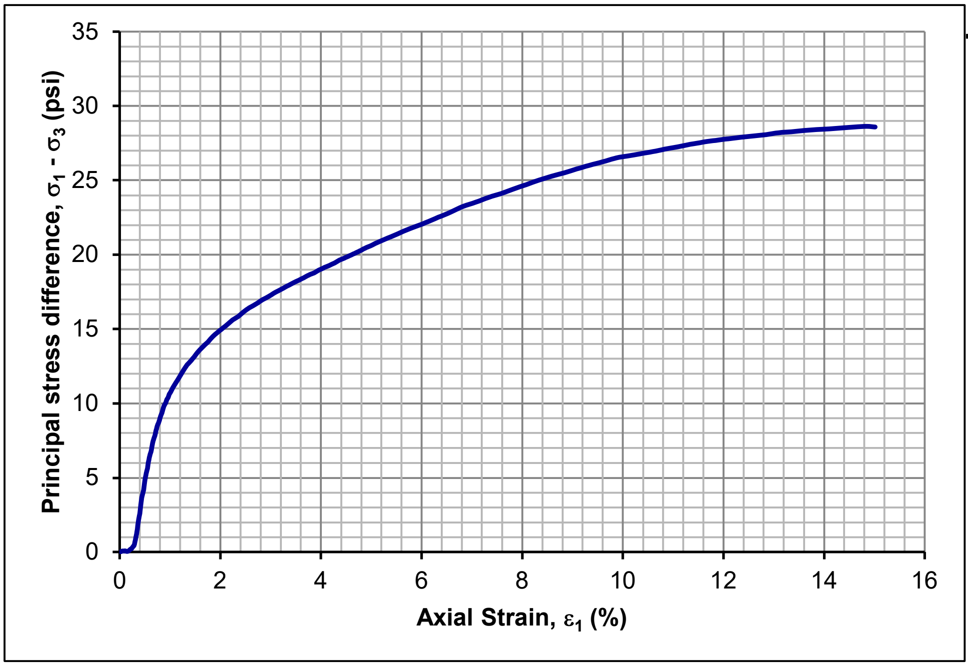
BORING NO.: B-4
SAMPLE NO.: ST-7
DEPTH (ft.): 15.0-17.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Gray Lean Clay

LIQUID LIMIT (%): 50
PLASTIC LIMIT (%): 15
PLASTICITY INDEX (%): 35
USCS: CL

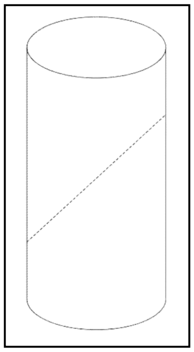
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.84
HEIGHT (in.):	6.00
HEIGHT TO DIAMETER RATIO:	2.12
WET UNIT WEIGHT (pcf):	119.3
DRY UNIT WEIGHT (pcf):	97.2
VOID RATIO:	0.77
MOISTURE CONTENT (%)*:	22.7
DEGREE OF SATURATION (%):	81.7

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**22.8**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.9**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**28.6**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**9.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**37.9**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**4,120**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**2,060**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**3,830**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

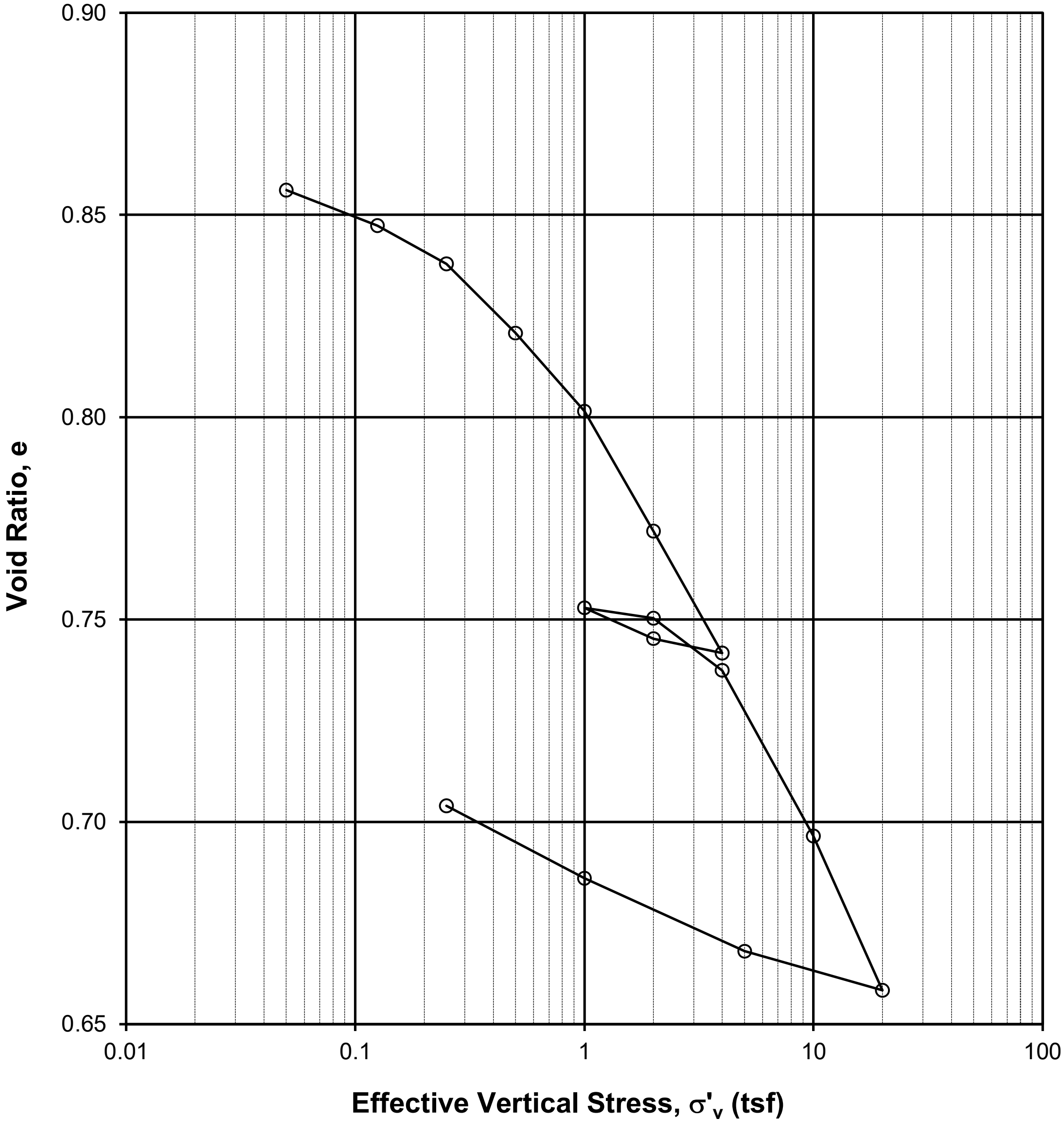
Liquid Limit= 32 Plastic Limit= 19 Plasticity Index = 13 USCS: CL

Compression Index, C_c = 0.18

Recompression Index, C_r = 0.04

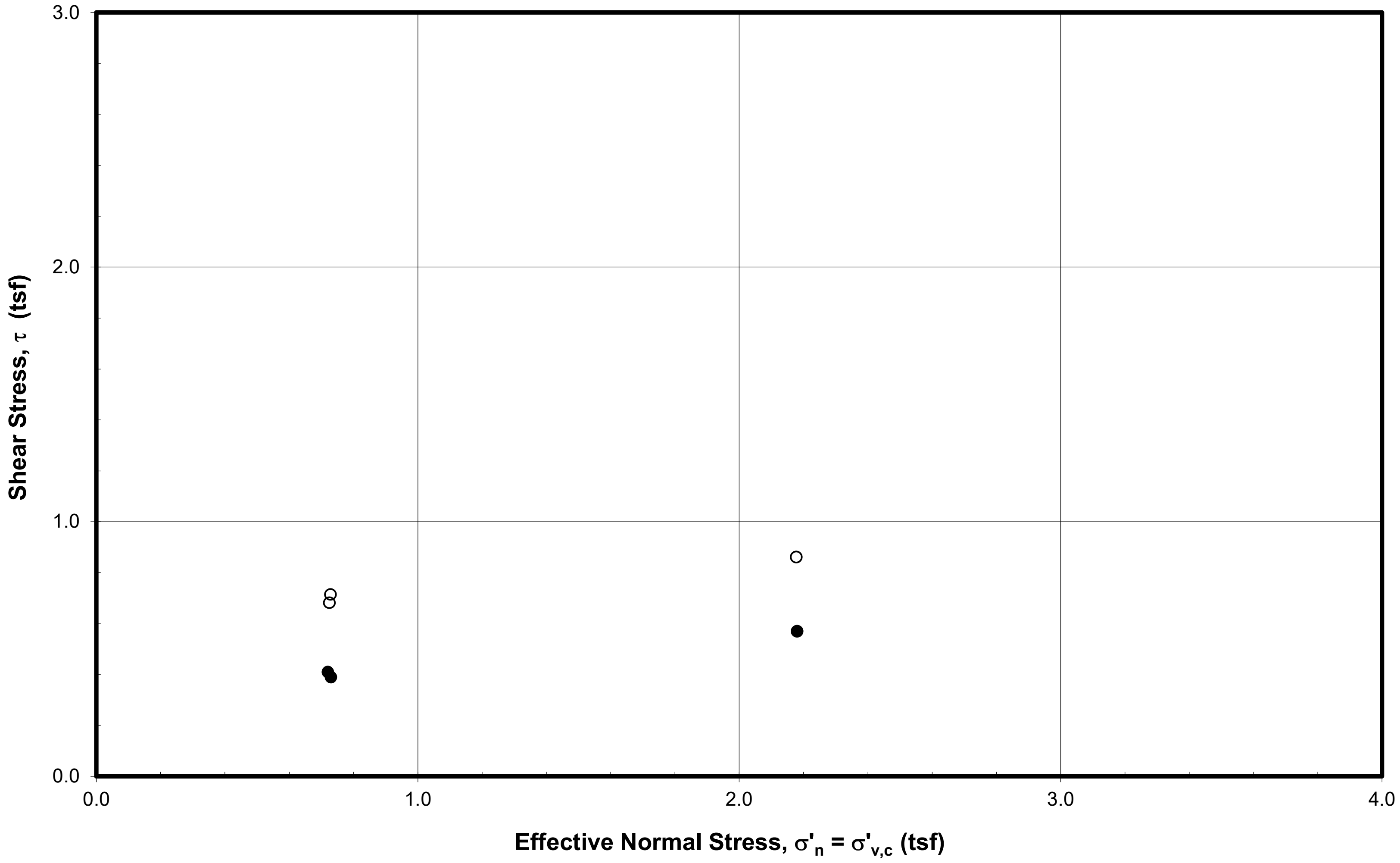
Void Ratio, e_o = 0.856

Preconsolidation Pressure = 0.65 tsf



1-D CONSOLIDATION TEST: INCREMENTAL

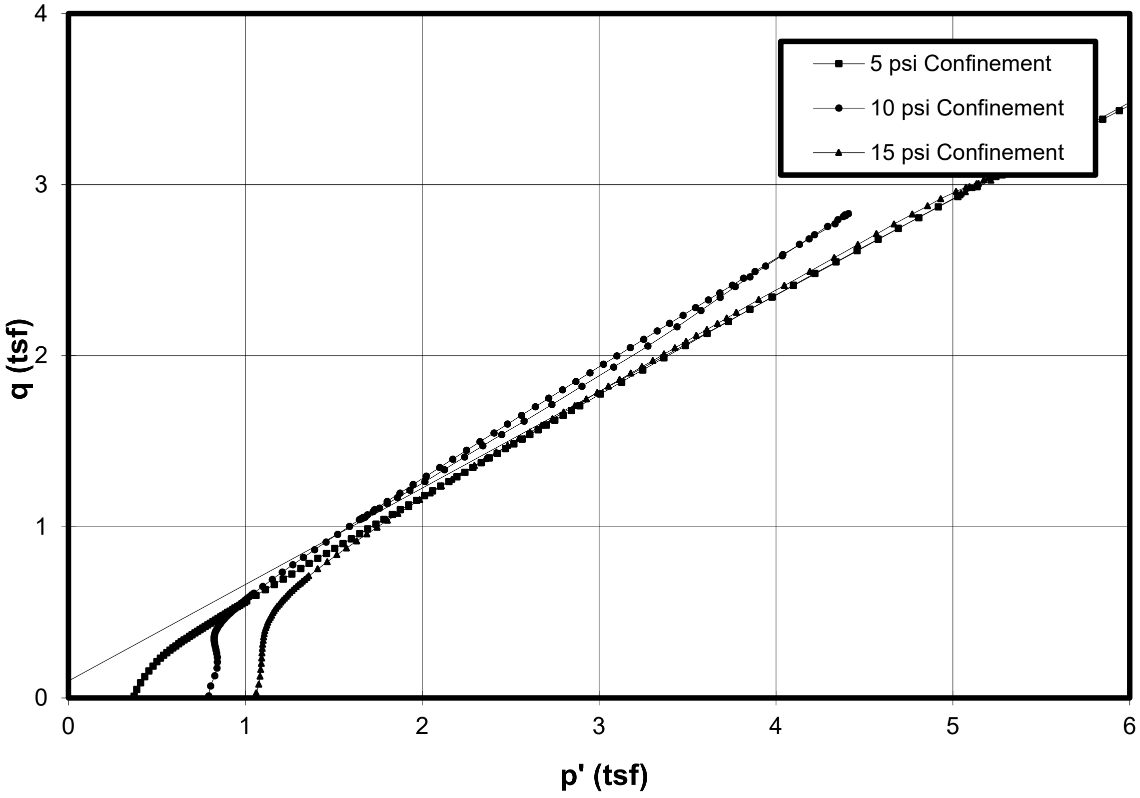
ASTM D 2435
Project No.: J042991.01
Boring: B-1
Sample: ST-5 - Depth: 10.0

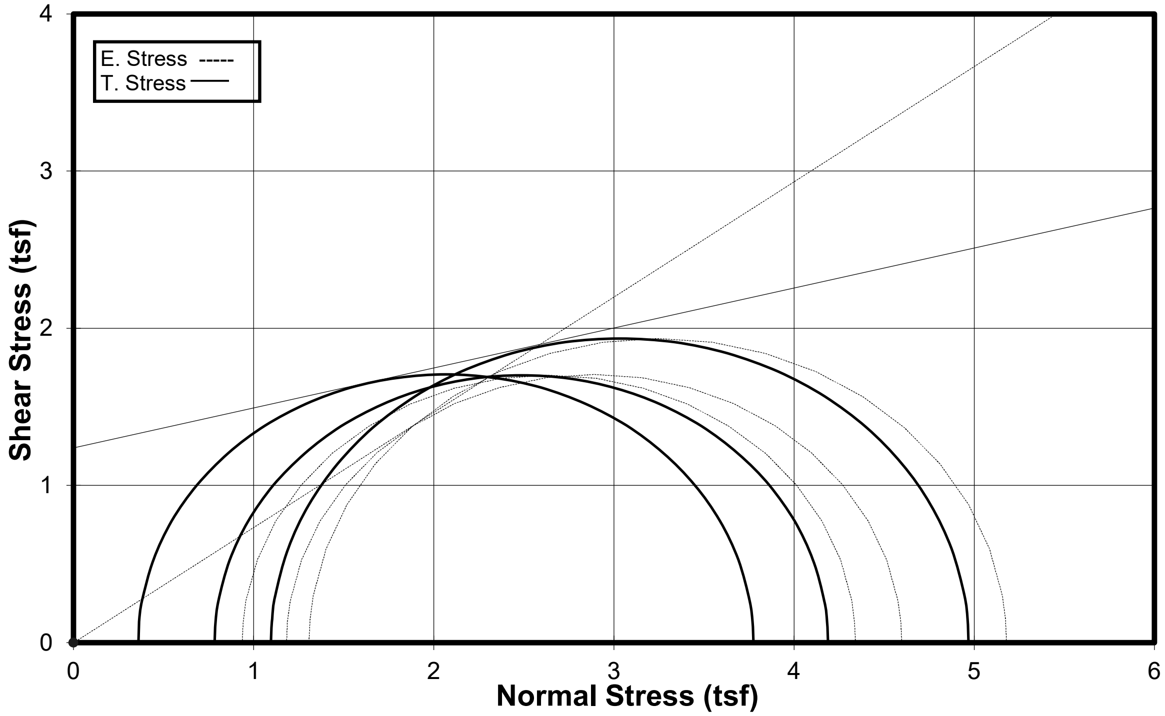


DRAINED DIRECT SHEAR TEST

ASTM D 3080

Boring: B-4 Sample: ST-7 -Depth: 15.0ft





CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 4767
Project No.: J042991.01
Boring: B-4
Sample: ST-3 - Depth: 6

SOIL RESISTIVITY TEST REPORT

Project No.:	**J042991.01**	**May 30, 2023**
Project Name:	**ARDOT 101120 Clay Co.**	**Page 1 of 1**
Boring Number:	**B-1**	
Sample ID:	**SS-6**	
Depth (ft):	**13.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	1,570	0.57	894.90	15.7
#2	910	0.57	518.70	21.0
#3	975	0.57	555.75	26.5

Minimum Soil Resistivity **518.70**

SOIL RESISTIVITY TEST REPORT

Project No.: J042991.01 May 30, 2023
Project Name: ARDOT 101120 Clay Co. Page 1 of 1
Boring Number: B-1
Sample ID: SS-12
Depth (ft): 38.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	12,960	0.57	7,387.20	9.1
#2	8,392	0.57	4,783.44	15.9
#3	7,405	0.57	4,220.85	45.7
#4	8,768	0.57	4,997.76	31.7

Minimum Soil Resistivity **4,220.85**

SOIL RESISTIVITY TEST REPORT

Project No.:	**J042991.01**	**May 30, 2023**
Project Name:	**ARDOT 101120 Clay Co.**	**Page 1 of 1**
Boring Number:	**B-2**	
Sample ID:	**SS-7**	
Depth (ft):	**23.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	13,170	0.57	7,506.90	9.5
#2	9,208	0.57	5,248.56	15.8
#3	9,330	0.57	5,318.10	22.8

Minimum Soil Resistivity **5,248.56**

SOIL RESISTIVITY TEST REPORT

Project No.: J042991.01 **May 30, 2023**
Project Name: ARDOT 101120 Clay Co. **Page 1 of 1**
Boring Number: B-2
Sample ID: SS-11
Depth (ft): 43.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	28,790	0.57	16,410.30	10.1
#2	16,930	0.57	9,650.10	17.0
#3	12,070	0.57	6,879.90	28.9
#4	9,788	0.57	5,579.16	28.6
#5	8,880	0.57	5,061.60	32.9
#6	17,060	0.57	9,724.20	35.9

Minimum Soil Resistivity **5,061.60**

SOIL RESISTIVITY TEST REPORT

Project No.:	**J042991.01**	**May 30, 2023**
Project Name:	**ARDOT 101120 Clay Co.**	**Page 1 of 1**
Boring Number:	**B-3**	
Sample ID:	**SS-6**	
Depth (ft):	**18.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	17,390	0.57	9,912.30	9.6
#2	10,090	0.57	5,751.30	15.8
#3	11,950	0.57	6,811.50	22.9

Minimum Soil Resistivity **5,751.30**

SOIL RESISTIVITY TEST REPORT

Project No.:	**J042991.01**	**May 30, 2023**
Project Name:	**ARDOT 101120 Clay Co.**	**Page 1 of 1**
Boring Number:	**B-3**	
Sample ID:	**SS-12**	
Depth (ft):	**48.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	22,730	0.57	12,956.10	9.8
#2	12,950	0.57	7,381.50	16.1
#3	10,500	0.57	5,985.00	21.2
#4	11,170	0.57	6,366.90	29.2

Minimum Soil Resistivity **5,985.00**

SOIL RESISTIVITY TEST REPORT

Project No.:	**J042991.01**	**May 30, 2023**
Project Name:	**ARDOT 101120 Clay Co.**	**Page 1 of 1**
Boring Number:	**B-4**	
Sample ID:	**SS-15**	
Depth (ft):	**53.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	25,440	0.57	14,500.80	9.5
#2	15,900	0.57	9,063.00	15.8
#3	12,720	0.57	7,250.40	23.4
#4	13,430	0.57	7,655.10	30.7

Minimum Soil Resistivity **7,250.40**

SOIL RESISTIVITY TEST REPORT

Project No.:	**J042991.01**	**May 30, 2023**
Project Name:	**ARDOT 101120 Clay Co.**	**Page 1 of 1**
Boring Number:	**B-4**	
Sample ID:	**SS-16**	
Depth (ft):	**58.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	18,560	0.57	10,579.20	10.7
#2	13,640	0.57	7,774.80	17.4
#3	10,410	0.57	5,933.70	24.9
#4	8,193	0.57	4,670.01	28.5
#5	9,583	0.57	5,462.31	35.1

Minimum Soil Resistivity **4,670.01**

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

GEOTECHNOLOGY
A UES Company

DATE	PROJECT NAME	PROJECT NO.
May 22, 2023	ARDOT 101120 Clay County	J042991.01

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/ Temp.)	Tare No. Air Drying	Jar Number	Remarks
B-1	SS-6	13.50			7.39 ------------- 21.9			
B-1	SS-12	38.50			9.16 ------------- 22.0			
B-2	SS-7	23.50			7.65 ------------- 22.2			
B-2	SS-11	43.50			8.73 ------------- 22.1			
B-3	SS-6	18.50			8.49 ------------- 22.1			
B-3	SS-12	48.50			8.61 ------------- 21.8			
B-4	SS-15	53.50			8.71 ------------- 22.3			
B-4	SS-16	58.50			8.66 ------------- 21.9			

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

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

Tested By: CY
Date: 05/22/23

Calculated By: MM
Date: 06/19/23

Checked By: JDM
Date: 06/21/23

APPENDIX F – SITE-SPECIFIC SEISMIC STUDY

Site-Specific Seismic Study ARDOT G002 101120 Hwy. 141 Over Ditch No. 8 Str. and Apprs(S) Clay County, Arkansas

By

Shahram Pezeshk, Ph.D., P.E.
Email: s.pezeshk@aol.com
901-606-6934

June 17, 2023

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Site-Specific Seismic Study
ARDOT G002 101120 Hwy. 141 Over Ditch No. 8 Str. and Apprs(S)
Clay County, Arkansas

1.0. EXECUTIVE SUMMARY

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

- The location of the study site is 36.333161°N and 90.444211°W (See Appendix A).
- Based on the recommendations of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions, A_S (zero-period), S_{DS} (short period), and S_{D1} (long period) are provided in Table 3.
- Site-specific recommendations following the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions are provided in Table 5 and Table 6.

2.0. SCOPE OF WORK

The purpose of our study is to estimate the design spectra following the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions. The structural design of new buildings allows two procedures for determining design ground motions:

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

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

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

1. Perform probabilistic seismic hazard analysis (PSHA) to estimate ground motions in the rock underlying the site;
2. Determine Uniform Hazard Response Spectrum (UHRS) at the rock level;
3. Determine probabilistic consistent magnitude and distances from deaggregation;
4. Select ground motions consistent with magnitude and distances obtained in step 3;
5. Perform spectral matching to match the selected ground motions to the UHRS of Step 2;
6. Perform one-dimensional equivalent linear site-specific ground response analysis using the site-specific earthquake time histories by using the computer program SHAKE91 (Idriss and Sun, 1992) and considering the uncertainties associated with the shear-wave velocity and layer thicknesses for the soil profile; and
7. Develop site-specific response spectra for the existing subsurface conditions using the procedure outlined in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, with 2022 Interim Revisions, based on 7 percent probability of exceedance in 75 years and 5 percent damping for a single degree of freedom (SDOF) structure.

3.0. SUBSURFACE CONDITIONS

This study is based on the available information on the soil stratigraphy provided by Geotechnology and the shear-wave velocity profile obtained using Seismic Cone Penetration Testing (SCPT).

4.0. SHEAR-WAVE VELOCITY PROFILE

Seismic Cone Penetration Testing (SCPT) was performed by Geotechnology (a UES Company). Table 1 provides the shear-wave velocity obtained from SCPT.

Table 1. Shear-Wave Velocities Measured.

Depth1 (ft)	**Depth2 (ft)**	**V_s (ft/sec)**
1.41	4.72	592.86
4.72	8.00	592.86
8.00	11.28	592.86
11.28	14.56	445.26
14.56	17.88	608.96
17.88	21.16	543.36
21.16	24.44	601.42
24.44	27.65	771.46
27.65	30.93	879.24
30.93	34.21	803.83
34.21	37.46	760.47
37.46	40.74	785.13
40.74	43.98	802.22
43.98	47.26	751.51
47.26	50.58	932.77
50.58	53.89	865.00
53.89	57.14	739.61
57.14	60.42	727.64
60.42	63.63	707.20
63.63	66.91	807.80
66.91	70.16	833.09
70.16	73.50	793.27
73.50	76.65	816.88
76.65	79.97	959.66
79.97	83.34	765.36
83.34	86.66	842.60
86.66	89.97	779.16
89.97	93.15	944.48
93.15	96.50	842.83
96.50	100.01	1054.75

5.0 GENERAL INFORMATION

For this project, we have been requested to perform a site-specific seismic study to produce the ground surface response spectrum and a set of time series based on the seismic parameters used in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions, which include: seismic hazards related to 7 percent probability of exceedance in 75 years and 5 percent damping for SDOF structure.

6.0. REGIONAL SEISMICITY

Petersen et al. (2019) used fault models from the 2014 NSHM to model large earthquakes and apply gridded, smoothed seismicity models from an earthquake catalog to account for smaller earthquakes on and off the faults. They developed new seismicity catalogs for the CEUS and WUS, including earthquakes from 2013 through 2017 that occurred since the last model was constructed. Between 2013, when the catalog was last updated, and 2018, strongly felt earthquakes (magnitude 4+) occurred in almost half of the states in the United States. Figure 1 shows the USGS 2018 declustered catalog for CEUS.

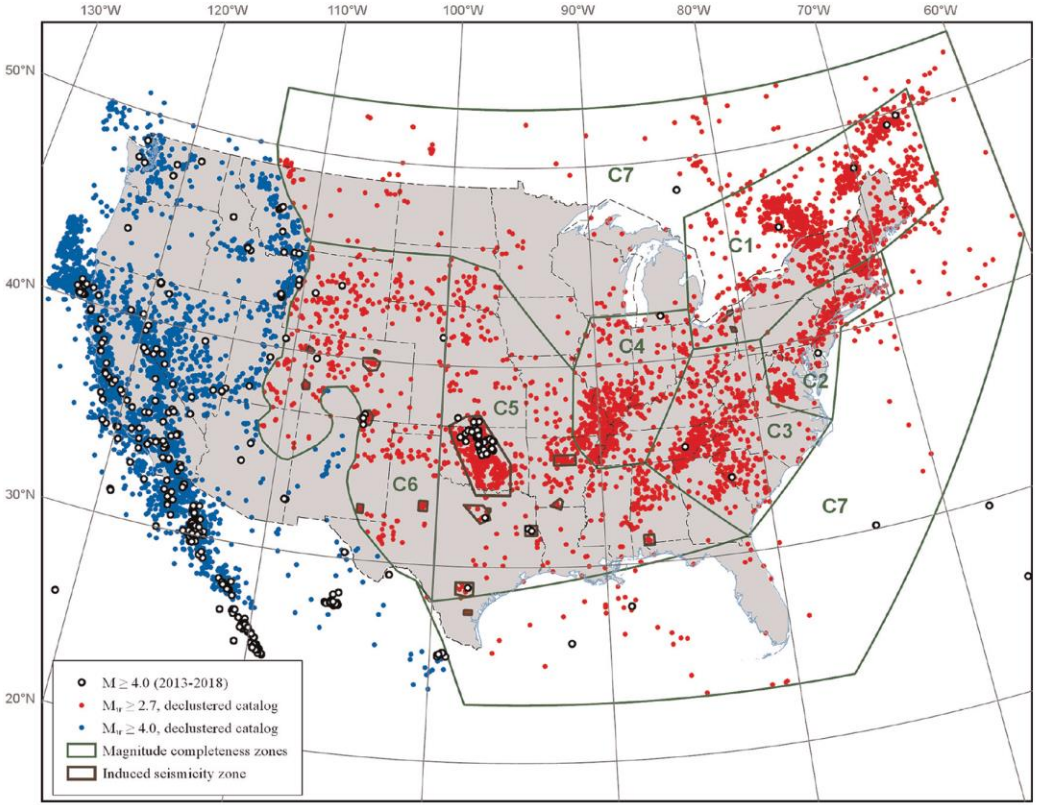


Figure 1. The 2018 NSHM Declustered Catalog for Central and Eastern United States (red) and Western United States (blue).

7.0. SEISMIC HAZARD ANALYSIS

A PSHA was performed to estimate the seismic ground motions for a rock site condition. The analytical model used for the PSHA is based on models developed initially by Cornell (1968). These models' underlying assumption is that earthquakes occur in space and time within a particular seismic zone is entirely random (i.e., a Poisson process). This type of probabilistic model is commonly used for seismic hazard analyses of essential facilities throughout the world.

The two primary components of the probabilistic model are:

1. The seismic source models specify the spatial, temporal, and magnitude distribution of earthquake occurrences expected in each of the seismic sources, and

2. The ground-motion attenuation models which determine the distribution of ground motions expected at the site for a potential earthquake occurrence (characterized by magnitude and location, and usually by other factors) on a seismic source.

The above two components comprise the inputs to the PSHA. In the PSHA, probability-of-exceedance rates (hazard curves) are computed for a range of horizontal ground motions. These ground motions are expressed in terms of peak ground acceleration (PGA) and 5 percent-damped pseudo absolute spectral accelerations (S_a) at various single-degree-of-freedom oscillator periods. From the probability-of-exceedance rates, the Uniform Hazard Response Spectrum (UHRS) corresponding to average return periods of 7% probability of exceedance in 75 years is computed.

7.1. SEISMIC SOURCE MODELS

The USGS seismic source models have been used for this project. The USGS addressed the causes of earthquakes in the Central and Eastern United States in two ways: (1) earthquake fault; and (2) background or smoothed seismicity models, which forecast the occurrence rates and magnitudes of potential seismic events.

7.2. GROUND MOTION MODELS

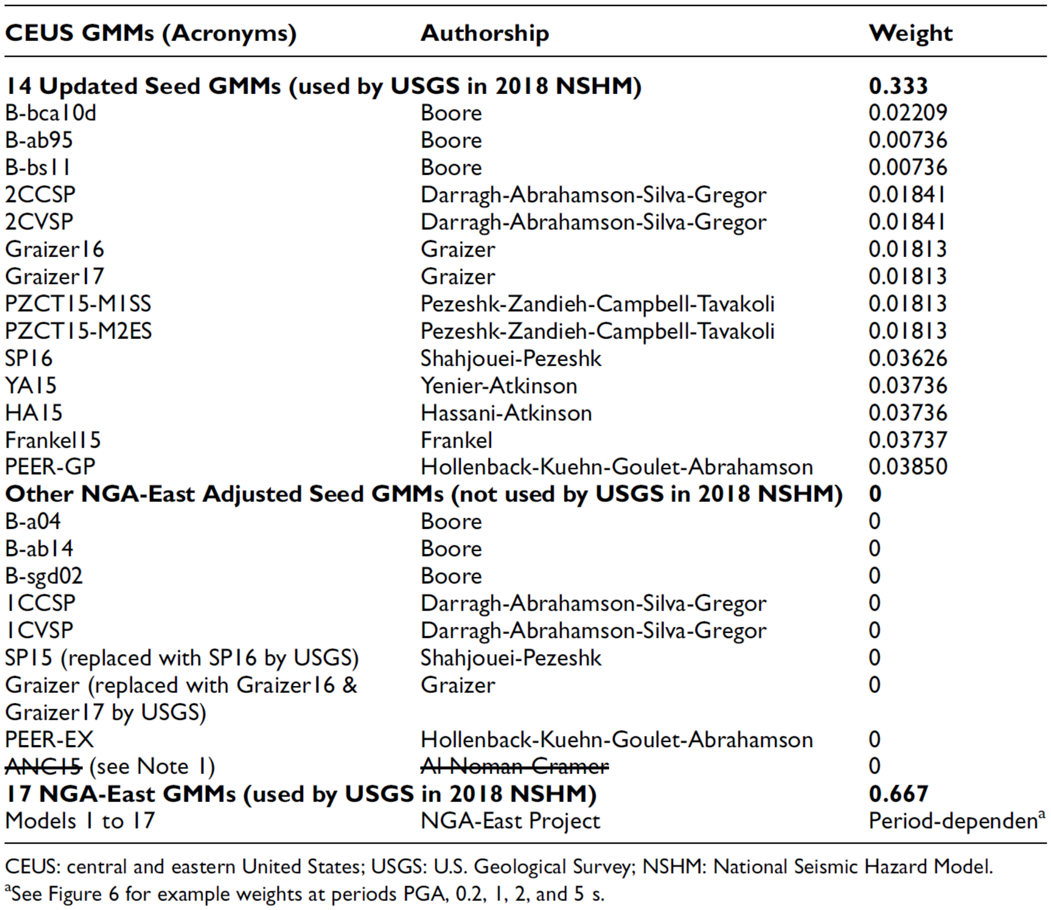
In general, the characteristics of the fault source, such as distance, type, magnitude, and site conditions, are used to estimate the magnitude of an earthquake parameter (spectral acceleration, peak ground acceleration, etc.) via ground-motion models (GMMs) or ground-motion prediction equations (GMPEs), also known as attenuation relationships. Various attenuation relationships have developed for specific regions using a database of appropriate ground motion records.

Petersen et al. (2020a) presented only a summary of the CEUS GMM updates, which included comparisons of the 2018 weighted median GMMs to the 2014 National Seismic Hazard Model (NSHM) and an overview of the aleatory variability (GMM standard deviation) and site-effect models. Rezaeian et al. (2021) discuss the CEUS GMM updates and implementation in the 2018 NSHM in detail. These updates consist of (1) 31 new GMMs, including the state-of-the-art Next Generation Attenuation relationships for central and eastern North America (NGA-East) (Goulet et al., 2018, 2017, 2021; Pacific Earthquake Engineering Research Center (PEER), 2015a), (2) an associated model of aleatory variability (based on Al Atik, 2015; Goulet et al., 2017; Stewart et al., 2019), and (3) a new site-effect model (for amplification or deamplification) specific to the CEUS (Hashash et al., 2020; Stewart et al., 2020). In the following, we discuss the individual GMMs in terms of their medians, assigned weights, weighted averages, attenuations with distance, and epistemic uncertainty.

According to Rezaeian et al. (2021), NSHM 2018 was updated to generate national seismic hazard maps for the Central and Eastern United States. The logic tree weights are based on the distance and the geometric spreading term used by each model. The models with a faster geometric spreading term are given more weight. The New Madrid seismic zone is the most likely seismic source that could affect the considered site. NSHM removed the attenuation relationships not applicable beyond 500 km, and weights were renormalized.

Table 2 lists the selected GMMs from the NSHM 2018 models with their associated weights. Three of the models were developed by Pezeshk and his colleagues [Pezeshk et al. 2015; 2018 (PZCT15-M1SS, PZCT15-M2ES), Shajouei and Pezeshk (2016) (SP16)].

Table 2. Ground Motion Models (GMMs).



CEUS GMMs (Acronyms)	**Authorship**	**Weight**
14 Updated Seed GMMs (used by USGS in 2018 NSHM)		**0.333**
B-bca10d	Boore	0.02209
B-ab95	Boore	0.00736
B-bs11	Boore	0.00736
2CCSP	Darragh-Abrahamson-Silva-Gregor	0.01841
2CVSP	Darragh-Abrahamson-Silva-Gregor	0.01841
Graizer16	Graizer	0.01813
Graizer17	Graizer	0.01813
PZCT15-M1SS	Pezeshk-Zandieh-Campbell-Tavakoli	0.01813
PZCT15-M2ES	Pezeshk-Zandieh-Campbell-Tavakoli	0.01813
SP16	Shahjouei-Pezeshk	0.03626
YA15	Yenier-Atkinson	0.03736
HA15	Hassani-Atkinson	0.03736
Frankel15	Frankel	0.03737
PEER-GP	Hollenback-Kuehn-Goulet-Abrahamson	0.03850
Other NGA-East Adjusted Seed GMMs (not used by USGS in 2018 NSHM)		**0**
B-a04	Boore	0
B-ab14	Boore	0
B-sgd02	Boore	0
1CCSP	Darragh-Abrahamson-Silva-Gregor	0
1CVSP	Darragh-Abrahamson-Silva-Gregor	0
SP15 (replaced with SP16 by USGS)	Shahjouei-Pezeshk	0
Graizer (replaced with Graizer16 & Graizer17 by USGS)	Graizer	0
PEER-EX	Hollenback-Kuehn-Goulet-Abrahamson	0
~~ANC15~~ (see Note 1)	~~Al Noman-Cramer~~	0
17 NGA-East GMMs (used by USGS in 2018 NSHM)		**0.667**
Models 1 to 17	NGA-East Project	Period-dependen[a]

CEUS: central and eastern United States; USGS: U.S. Geological Survey; NSHM: National Seismic Hazard Model.

[a] See Figure 6 for example weights at periods PGA, 0.2, 1, 2, and 5 s.

7.3. TREATMENT OF UNCERTAINTIES

Seismic-hazard studies distinguish between two types of uncertainty, namely epistemic and aleatory. Aleatory uncertainty is probabilistic variability that results from a natural physical process. For example, the size, location, and time of the next earthquake on a fault and the details of the ground motion are considered aleatory uncertainties. In advanced seismic hazard studies, integration is performed over aleatory uncertainties to get a single hazard curve—the epistemic uncertainty results from a lack of knowledge about earthquakes and their effects. In principle, epistemic uncertainties are addressed by multiple models and parameters. The most well-known epistemic uncertainties associated with the input parameters in seismic hazard analysis include the uncertainties in seismic source models (i.e., tectonic stresses, geological features, geometries, etc.), seismicity (i.e., activity rate, slip rate, etc.), and attenuation relationships (source, path, and site effects). The USGS 2014 procedure (Petersen *et al.*, 2014) is followed in this project to address the uncertainty in seismic-source characterization, which is quantified by considering alternative geometries, multiple magnitude-recurrence parameters, and multiple maximum magnitudes.

8.0. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2022 Interim Revisions

Time-averaged shear-wave velocity in the top 100 ft (30 m) is defined as V_{S30}. The V_{S30} for the study site is determined to be 783 ft/sec, which according to the Guide Specifications, the study site is determined to be a Site Class "D" (Table 3.4.2.1-1, Site Class Definitions). Site coefficients F_{pga}, F_a, and F_v for the study site following Tables 3.4.2.3-1 and 3.4.2.302 mapped spectral acceleration are summarized in Table 3.

8.1. Dynamic Soil Properties

Low-strain soil shear modulus and damping are the required dynamic soil properties for seismic ground response analysis. A brief discussion of these properties is given below.

8.1.1. Low Strain Soil Shear Modulus

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

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

8.1.2. Damping

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

8.1.3. Effect of Strain on Dynamic Soil Properties

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

9.0. CODE-BASED DESIGN APPROACH

9.1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2022 Interim Revisions

Using the United States Geological Survey (USGS) Hazard Maps and the project location, the mapped 0.2-second spectral response acceleration (S_s) and the mapped 1.0-second spectral response acceleration (S_1) are provided in Table 3. Based on the average shear-wave velocities of the top 100 ft of soil, the site class has been determined to be site class “D.” Based on the mapped spectral acceleration and site class D, the site coefficients F_{PGA}, $F_{a,}$ and F_v are provided in Table 3. provides a summary of these parameters.

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

Parameter	Value
F_a	1.219
F_v	2.066
F_{PGA}	1.109
S_S	0.726
S_1	0.183
S_{DS}	0.885
S_{D1}	0.379
PGA	0.391
A_s	0.434

10.0. SITE-SPECIFIC PROCEDURE

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

The PSHA was performed to obtain a uniform hazard response spectrum (UHRS). The PSHA and de-aggregation results were used to select earthquakes for the site response analyses. Eleven horizontal components (total of 11) of previously recorded earthquakes within the range of de-aggregation magnitudes and distances were selected.

Table *4* provides the mean and the modal deaggregation magnitude and distances for various periods. The UHRS was selected as the target spectrum, and the chosen time histories were matched with the target spectrum. As an example, acceleration, velocity, and displacement time histories for a typically selected earthquake are illustrated in Figure 2. The same process was repeated for all eleven earthquakes for both components.

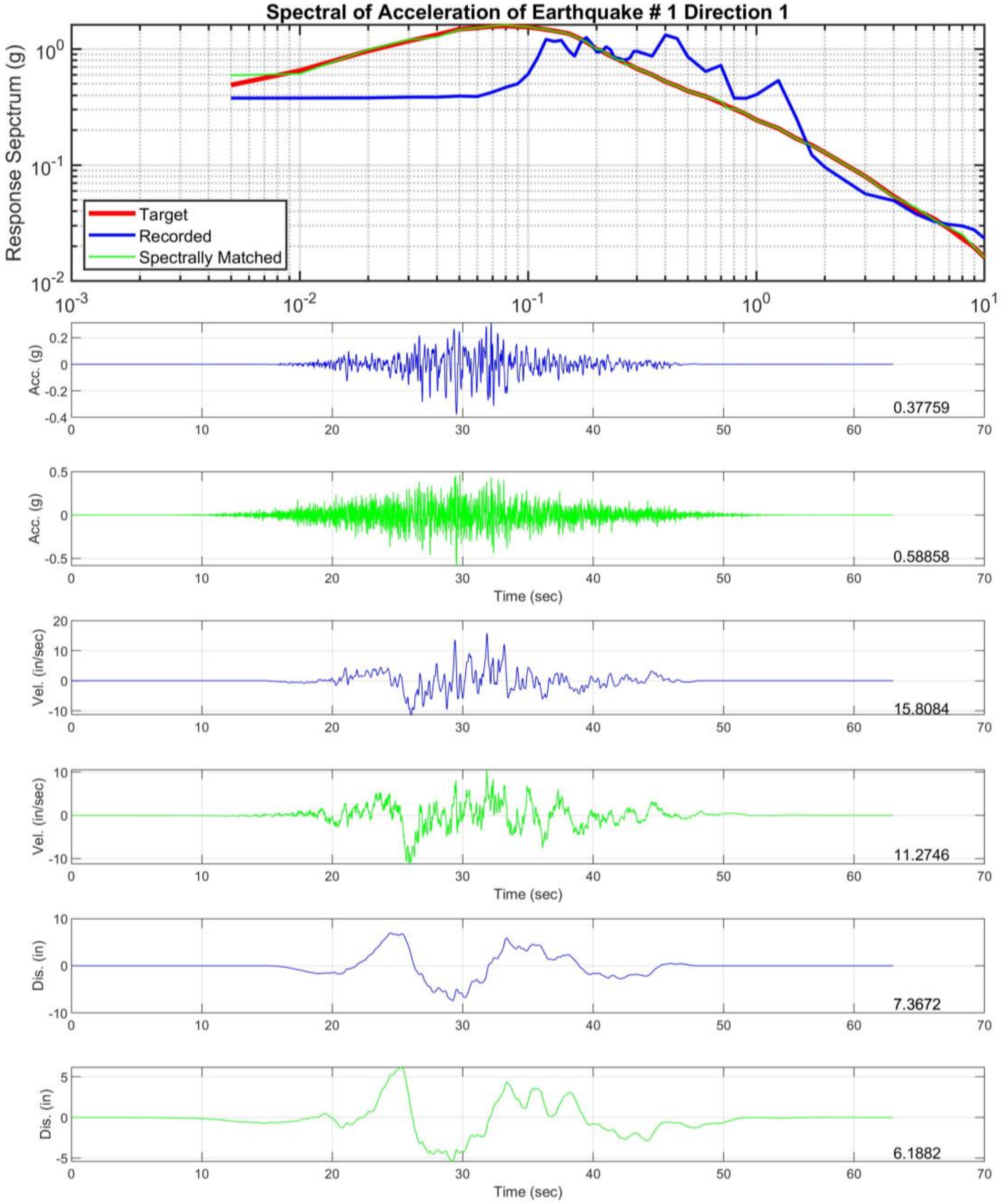


Figure 2. Time Histories Before and After the Spectral Matching Process for Earthquake #1. The numbers Shown in the Bottom right of Each Figure Represent the Absolute Maximum Value of the Graph.

Table 4. Deaggregation.

Mean and Mode Deaggregation Parameter at 1,033 Years					
Mean			**Mode**		
Period	**M**	**R (km)**	**Period**	**M**	**R (km)**
PGA	7.18	53.53	PGA	7.76	69.96
0.01	7.17	53.33	0.01	7.75	70.89
0.02	7.15	53.23	0.02	7.75	70.91
0.03	7.16	53.60	0.03	7.75	70.92
0.05	7.19	54.87	0.05	7.75	70.90
0.08	7.23	56.09	0.08	7.76	70.03
0.10	7.27	57.35	0.10	7.52	72.28
0.15	7.33	58.98	0.15	7.52	72.34
0.20	7.37	60.04	0.20	7.52	72.34
0.25	7.41	61.14	0.25	7.52	72.35
0.30	7.43	61.69	0.30	7.52	72.36
0.40	7.46	62.64	0.40	7.52	72.38
0.50	7.47	63.30	0.50	7.52	72.13
0.75	7.51	64.63	0.75	7.52	72.16
1.00	7.53	65.36	1.00	7.52	72.17
1.50	7.57	66.04	1.50	7.51	73.24
2.00	7.59	66.53	2.00	7.51	73.27
3.00	7.62	66.70	3.00	7.51	72.99
4.00	7.64	66.87	4.00	7.52	72.98
5.00	7.65	66.95	5.00	7.52	73.25
7.50	7.67	66.85	7.50	7.52	73.07
10.00	7.68	66.90	10.00	7.50	72.01

10.1. Seismic Hazard Analysis

The uniform hazard response spectrum (UHRS) and the magnitude and distance deaggregation for a 7 percent probability of exceedance in 75 years (equivalent to a return period of about 1033 years) are calculated from the PSHA. The seismic hazard is calculated for the uniform firm site condition with 760 m/s shear-wave velocity in the upper 30 m (V_{s30}), representing the boundary between NEHRP site classes B and C.

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

A probabilistic characterization of the soil shear-wave velocity profile was used to simulate shear-wave profiles. Two separate components; one for the thickness of each layer called the layering model that captures the variability in the thickness of soil layers, and one for the shear-wave velocity associated with each layer called the velocity model to account for the variability in the shear-wave velocity of each layer are used. A non-homogeneous Poisson model is used with a depth-dependent rate to account for the fact that the soil thickness of layers increases with depth.

In this project, the variability in the shear-wave velocity are considered. The model used statistically captures the soil layer shear-wave velocity and thickness uncertainties and their correlation with depth. A total of 60 cases were generated. These 60 soil profiles are used to capture the soil layer shear-wave velocity and thickness uncertainties and their correlation with depth.

10.3. Site-Specific Results

Following the procedure outlined above, the site-specific response spectra were obtained, analyzing sixty profiles for each matched ground motion with the UHRS.

The site-specific results were obtained by performing PSHA using all seismic sources and faults and appropriate and recent ground motion prediction equations for Central and Eastern United States following the provisions of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions. All uncertainties associated with each aspect of the site-specific analysis were carefully considered. Figure 3 shows the design response spectra, Guide Specifications, and 2/3 of Guide Specifications design spectra. In this figure, the site-specific spectrum is not limited to 2/3 of the Guide Specifications response spectrum for illustration.

Site-specific seismic design recommendations following the Guide Specifications provisions are provided in Table 5 and Table 6. The recommendation is to use the design Sa values provided in Table 5. Figure 4 shows the design response spectra, Guide Specifications, 2/3 of Guide Specifications design spectra, and the site-specific design spectrum constructed based on three periods of PGA, 0.2 sec and 1 sec. In Figure 4, the site-specific response spectrum is adjusted not to be less than 2/3 of the Guide Specifications design response spectrum.

11.0. DESIGN RESPONSE SPECTRAL PARAMETERS

The design spectral response acceleration parameters listed in Table 5 were developed following Guide Specifications.

Table 5. Site-Specific Spectral Acceleration Considering 5% Damping following the Guide Specifications.

Period	**Site-Specific Response Spectra**
(s)	**(g)**
0.010	0.360
0.030	0.361
0.040	0.395
0.050	0.430
0.070	0.500
0.100	0.590
0.150	0.670
0.200	0.757
0.250	0.740
0.300	0.726
0.400	0.810
0.500	0.800
0.750	0.720
1.000	0.532
1.500	0.466
2.000	0.355
3.000	0.272
4.000	0.204
5.000	0.107
7.500	0.043
10.000	0.040

Table 6. Site-Specific Response Accelerations Considering 5% Damping.

PARAMETER	**DESIGN ACCELERATION PARAMETERS (g)**
S_{DS}	0.757
S_{D1}	0.532
S_{MS}	0.757
S_{M1}	0.532
MCE_G	0.360

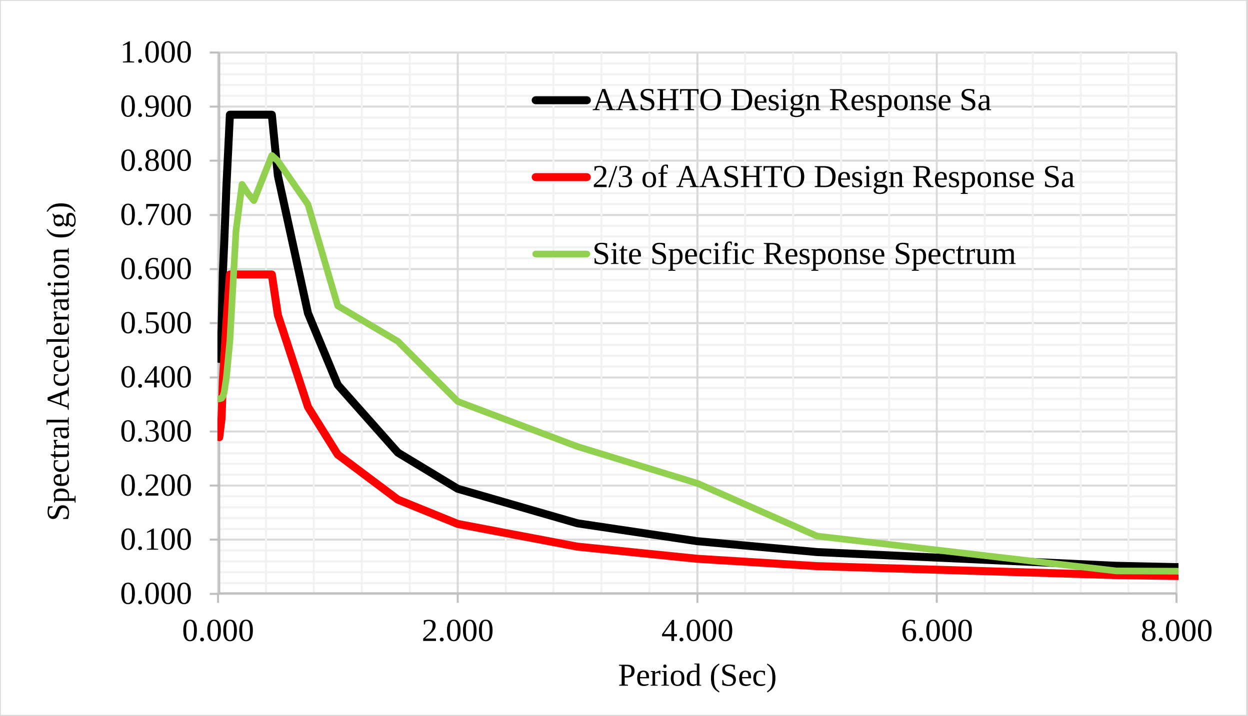


Figure 3. Site-Specific Design Response Spectrum, AASHTO Guide Specifications Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications Design Response Spectrum.

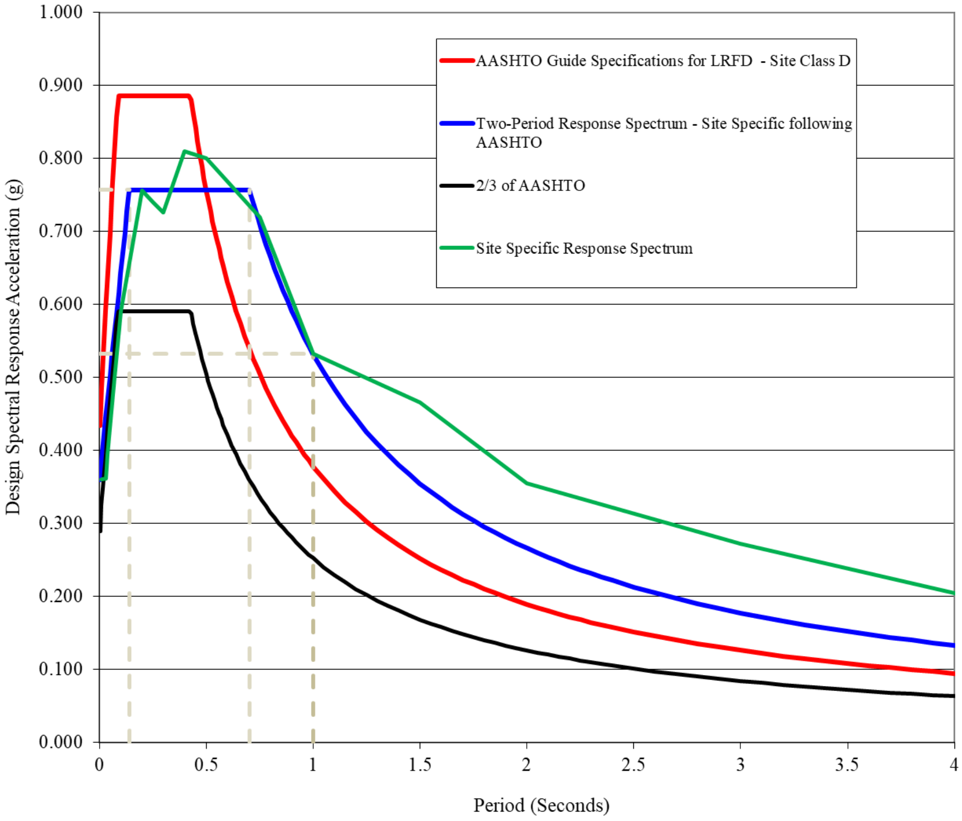


Figure 4. Design Response Spectrum based on AASHTO Guide Specifications, 2/3 of the AASHTO Guide Specifications Site-Specific, and Design Response Spectrum Based on PGA, 0.2, and 1 Second.

12.0 LIMITATIONS OF THE REPORT

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

This Report is copy-righted and was prepared for the exclusive use of the owner, architect, and engineer to evaluate the project's design related to the ground response discussed in this Report.

13.0 REFERENCES

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APPENDIX A. Site Location

Figure A.1. The Location of the Study Site.

APPENDIX B. Boring Log

Surface Elevation: 292

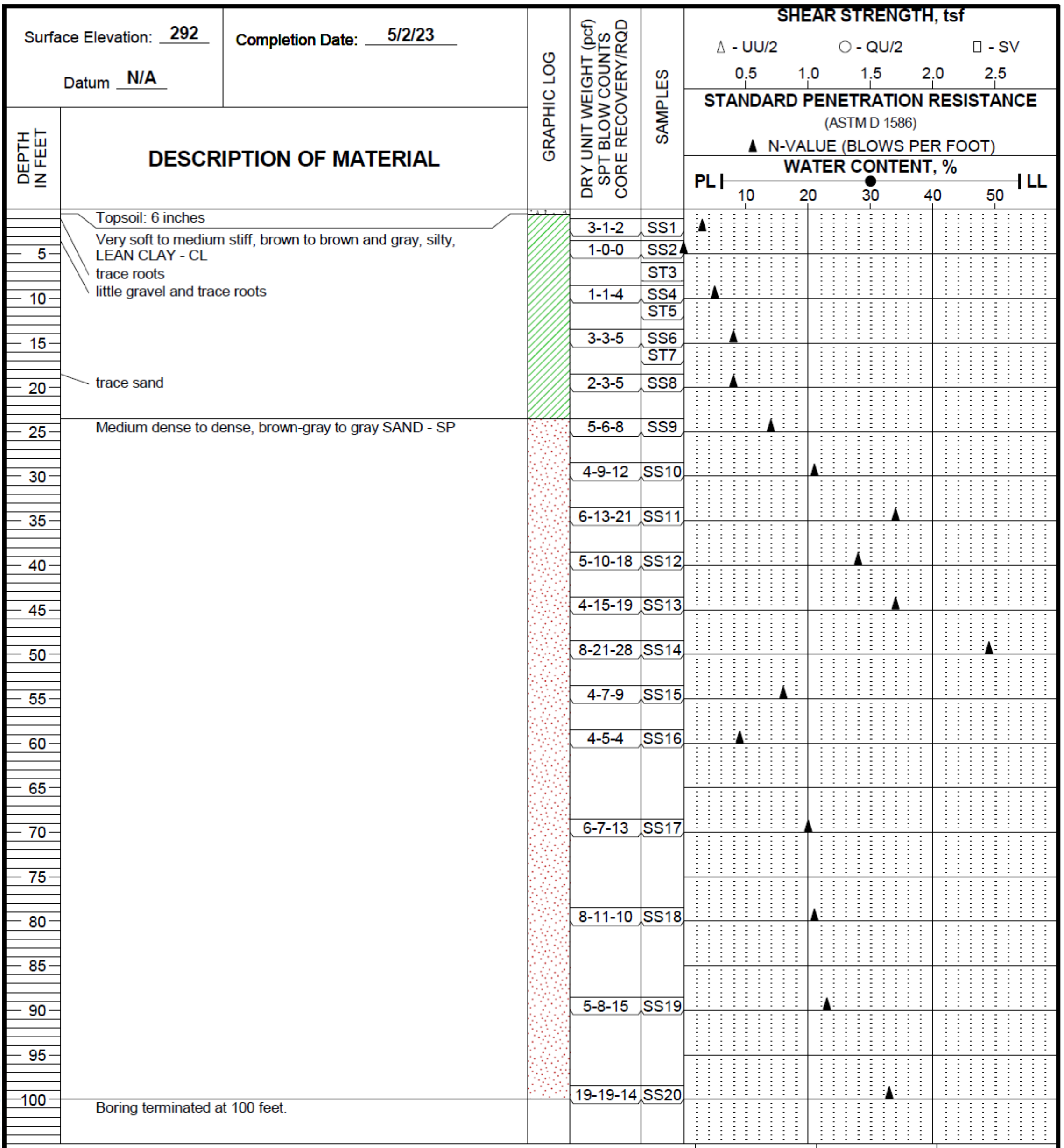
Datum N/A

Completion Date: 5/2/23

LOG OF BORING: B-4

DEPTH IN FEET	DESCRIPTION OF MATERIAL
	Topsoil: 6 inches
5	Very soft to medium stiff, brown to brown and gray, silty, LEAN CLAY - CL trace roots little gravel and trace roots
20	trace sand
25	Medium dense to dense, brown-gray to gray SAND - SP
100	Boring terminated at 100 feet.

DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
3-1-2	SS1
1-0-0	SS2
	ST3
1-1-4	SS4
	ST5
3-3-5	SS6
	ST7
2-3-5	SS8
5-6-8	SS9
4-9-12	SS10
6-13-21	SS11
5-10-18	SS12
4-15-19	SS13
8-21-28	SS14
4-7-9	SS15
4-5-4	SS16
6-7-13	SS17
8-11-10	SS18
5-8-15	SS19
19-19-14	SS20



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
KJB DRILLER JWD LOGGER
CME 750X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 81 %

REMARKS:

Drawn by: JSH	Checked by:	App'vd. by:
Date: 5/10/23	Date:	Date:

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ARDOT G002 101120 Hwy. 141 Over Ditch No. 8 Str. and Apprs(S)
Clay County, Arkansas

LOG OF BORING: B-4

Project No. J042991.01

LOG OF BORING 2020 JDM J042991.01.GPJ GTINC 0638301.GPJ 5/12/23

APPENDIX G – AASHTO AND USCS CLASSIFICATIONS

Project: ARDOT 101120
Hwy. 141 Over Ditch No. 8
Number: J042991.01

Borehole	Depth	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	%<#10 Sieve	%<#40 Sieve	%<#200 Sieve	GI	AASHTO CLASS.	USCS CLASS.
B-1	3.5	29	20	9	-	-	-	N/A	A-4	CL
	10	32	19	13	-	-	-	N/A	A-6	CL
	20	-	-	-	100.0	100.0	8.3	N/A	A-2-6	SP-SC
	23.5	-	-	-	100.0	99.6	4.2	N/A	A-3	SP
	33.5	-	-	-	100.0	99.9	3.2	N/A	A-3	SP
B-2	18.5	-	-	-	99.3	81.5	2.5	N/A	A-3	SP
	48.5	-	-	-	100.0	99.0	3.8	N/A	A-3	SP
B-3	38.5	-	-	-	100.0	84.3	5.5	N/A	A-3	SP-SM
	43.5	-	-	-	99.7	87.3	3.0	N/A	A-3	SP
B-4	1	28	20	8	-	-	-	N/A	A-4	CL
	6	26	23	3	-	-	-	N/A	A-4	ML
	10	32	19	13	-	-	-	N/A	A-6	CL
	15	50	15	35	-	-	-	N/A	A-7-6	CL
	18.5	-	-	-	100.0	95.7	73.8	N/A	A-6	CL
	28.5	-	-	-	100.0	99.1	5.4	N/A	A-2-6	SP-SC

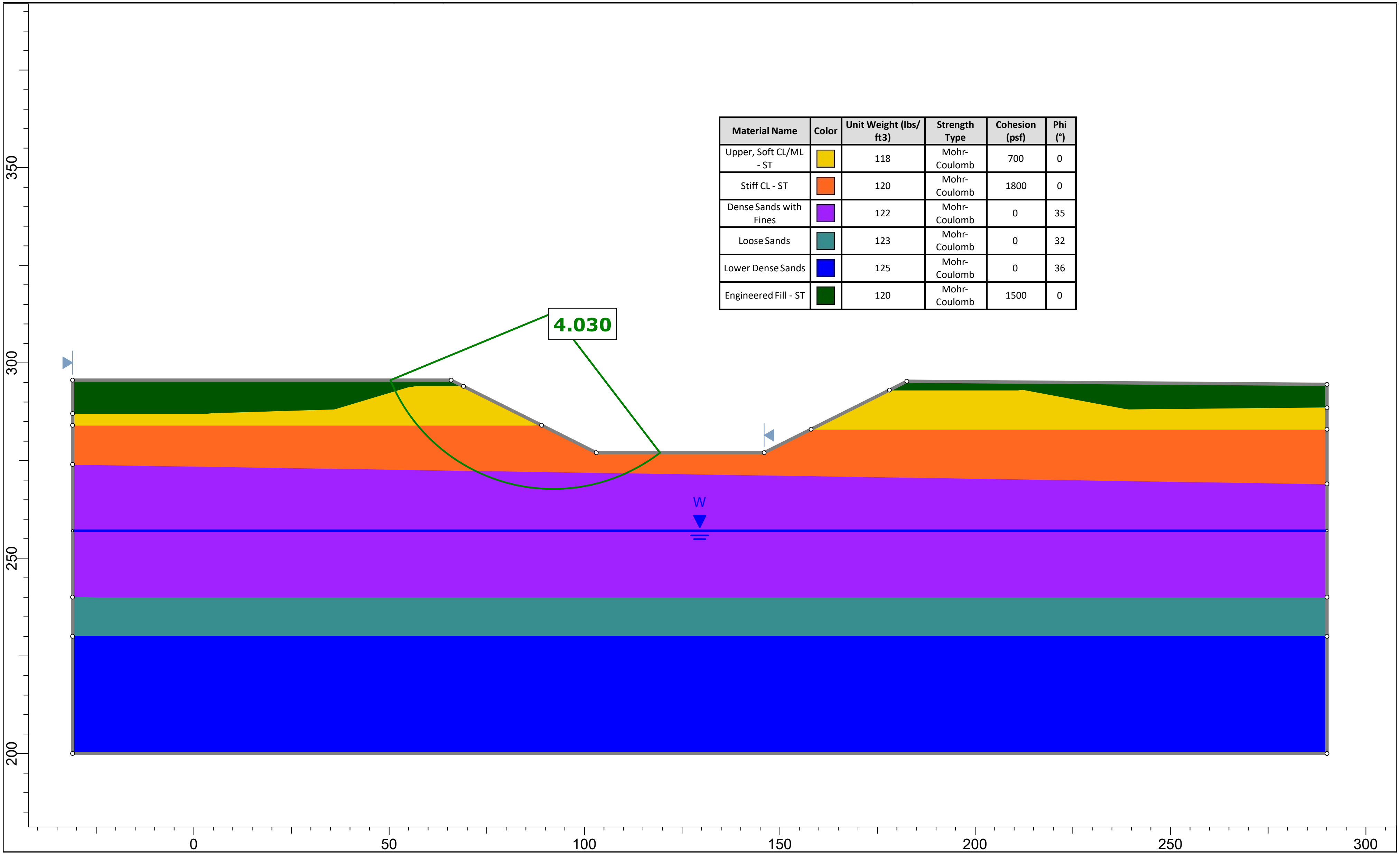
APPENDIX H – GLOBAL STABILITY ANALYSES

GEOTECHNOLOGY
A UES Company

File Name: ARDOT 101120 Abutment Slope Stability.slmd
Name: Southern Abutment
Description: Short Term Conditions
Method: Spencer
Date: 8/10/2023

Project Number: J042991.01
Client: ARDOT
Project: ARDOT 101120 Str. & Apprs (S)
Clay County, Arkansas

SLIDEINTERPRET 9.028

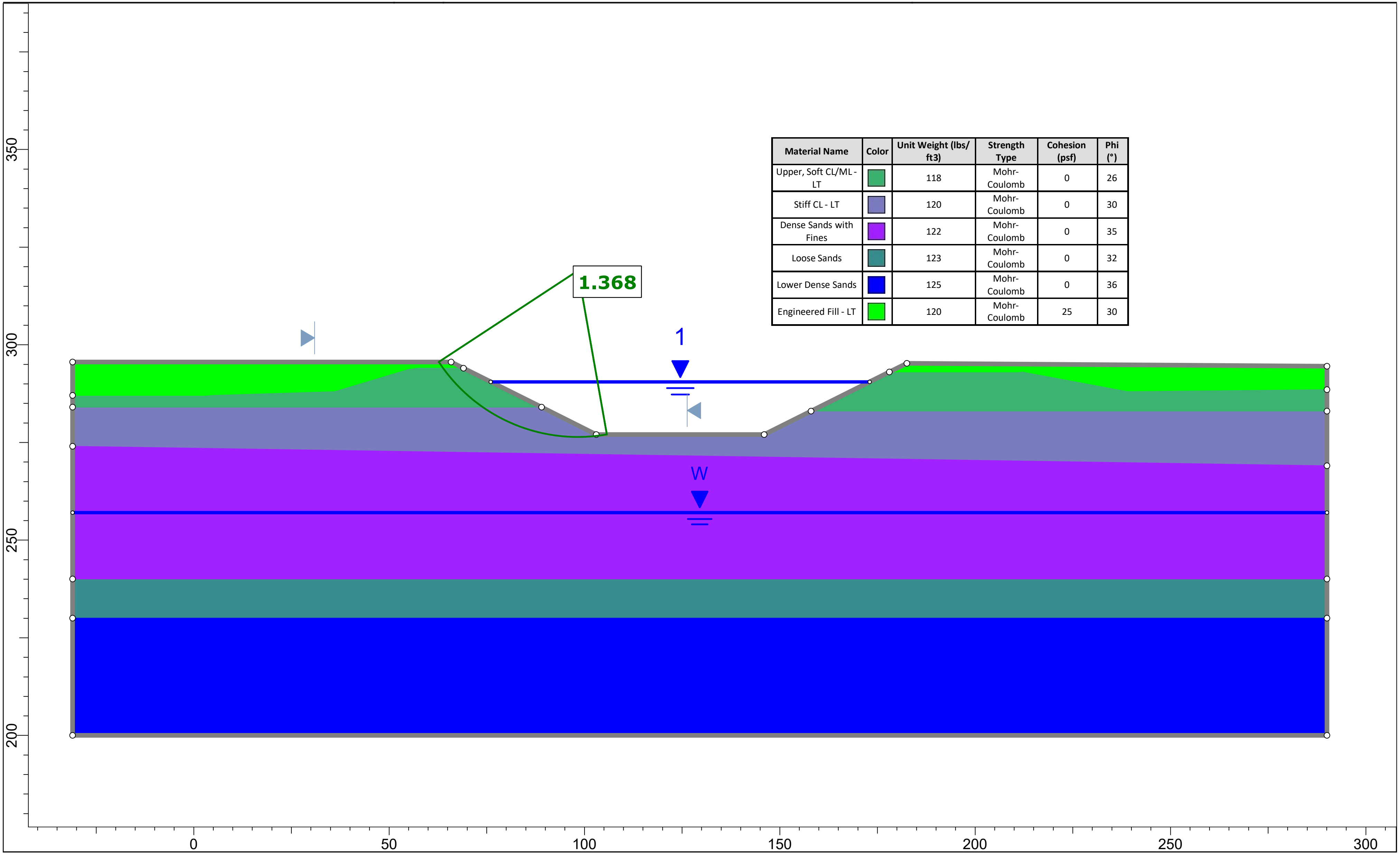


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper, Soft CL/ML - ST	🟨	118	Mohr-Coulomb	700	0
Stiff CL - ST	🟧	120	Mohr-Coulomb	1800	0
Dense Sands with Fines	🟪	122	Mohr-Coulomb	0	35
Loose Sands	🟦	123	Mohr-Coulomb	0	32
Lower Dense Sands	🟦	125	Mohr-Coulomb	0	36
Engineered Fill - ST	🟩	120	Mohr-Coulomb	1500	0

File Name: ARDOT 101120 Abutment Slope Stability.slmd
Name: Southern Abutment
Description: Long Term Conditions
Method: Spencer
Date: 8/10/2023

Project Number: J042991.01
Client: ARDOT
Project: ARDOT 101120 Str. & Apprs (S)
Clay County, Arkansas

SLIDEINTERPRET 9.028



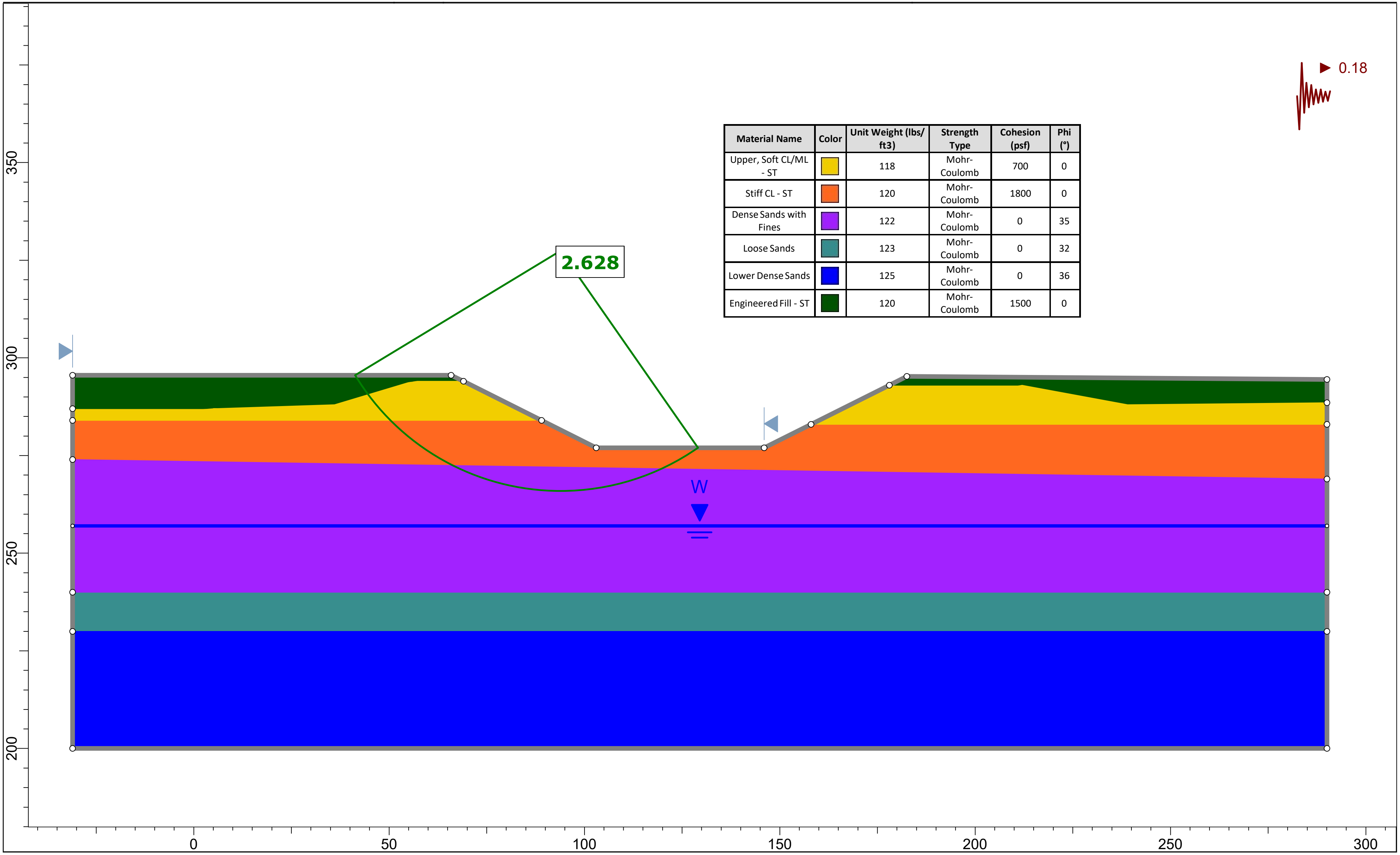
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper, Soft CL/ML - LT	🟩	118	Mohr-Coulomb	0	26
Stiff CL - LT	🟪	120	Mohr-Coulomb	0	30
Dense Sands with Fines	🟪	122	Mohr-Coulomb	0	35
Loose Sands	🟦	123	Mohr-Coulomb	0	32
Lower Dense Sands	🟦	125	Mohr-Coulomb	0	36
Engineered Fill - LT	🟩	120	Mohr-Coulomb	25	30

GEOTECHNOLOGY
A UES Company

File Name: ARDOT 101120 Abutment Slope Stability.slmd
Name: Southern Abutment
Description: Seismic Conditions
Method: Spencer
Date: 8/10/2023

Project Number: J042991.01
Client: ARDOT
Project: ARDOT 101120 Str. & Apprs (S)
Clay County, Arkansas

SLIDEINTERPRET 9.028



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper, Soft CL/ML - ST	🟨	118	Mohr-Coulomb	700	0
Stiff CL - ST	🟧	120	Mohr-Coulomb	1800	0
Dense Sands with Fines	🟪	122	Mohr-Coulomb	0	35
Loose Sands	▨	123	Mohr-Coulomb	0	32
Lower Dense Sands	🟦	125	Mohr-Coulomb	0	36
Engineered Fill - ST	🟩	120	Mohr-Coulomb	1500	0

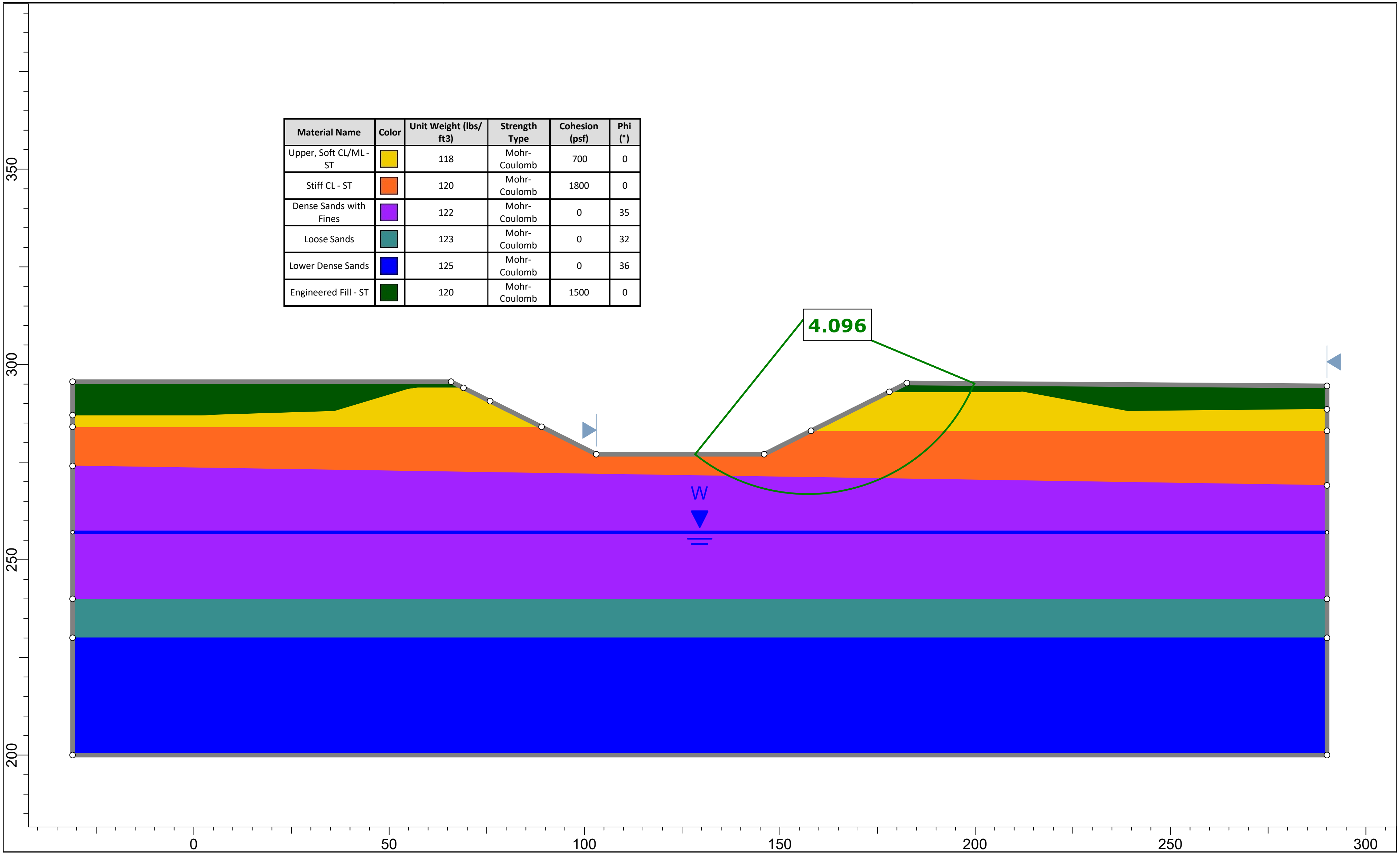
GEOTECHNOLOGY
A UES Company

File Name: ARDOT 101120 Abutment Slope Stability.slmd
Name: Northern Abutment
Description: Short Term Conditions
Method: Spencer
Date: 8/10/2023

Project Number: J042991.01
Client: ARDOT
Project: ARDOT 101120 Str. & Apprs (S)
Clay County, Arkansas

SLIDEINTERPRET 9.028

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper, Soft CL/ML - ST	🟨	118	Mohr-Coulomb	700	0
Stiff CL - ST	🟧	120	Mohr-Coulomb	1800	0
Dense Sands with Fines	🟪	122	Mohr-Coulomb	0	35
Loose Sands	🟦	123	Mohr-Coulomb	0	32
Lower Dense Sands	🟦	125	Mohr-Coulomb	0	36
Engineered Fill - ST	🟩	120	Mohr-Coulomb	1500	0



GEOTECHNOLOGY
A UES Company

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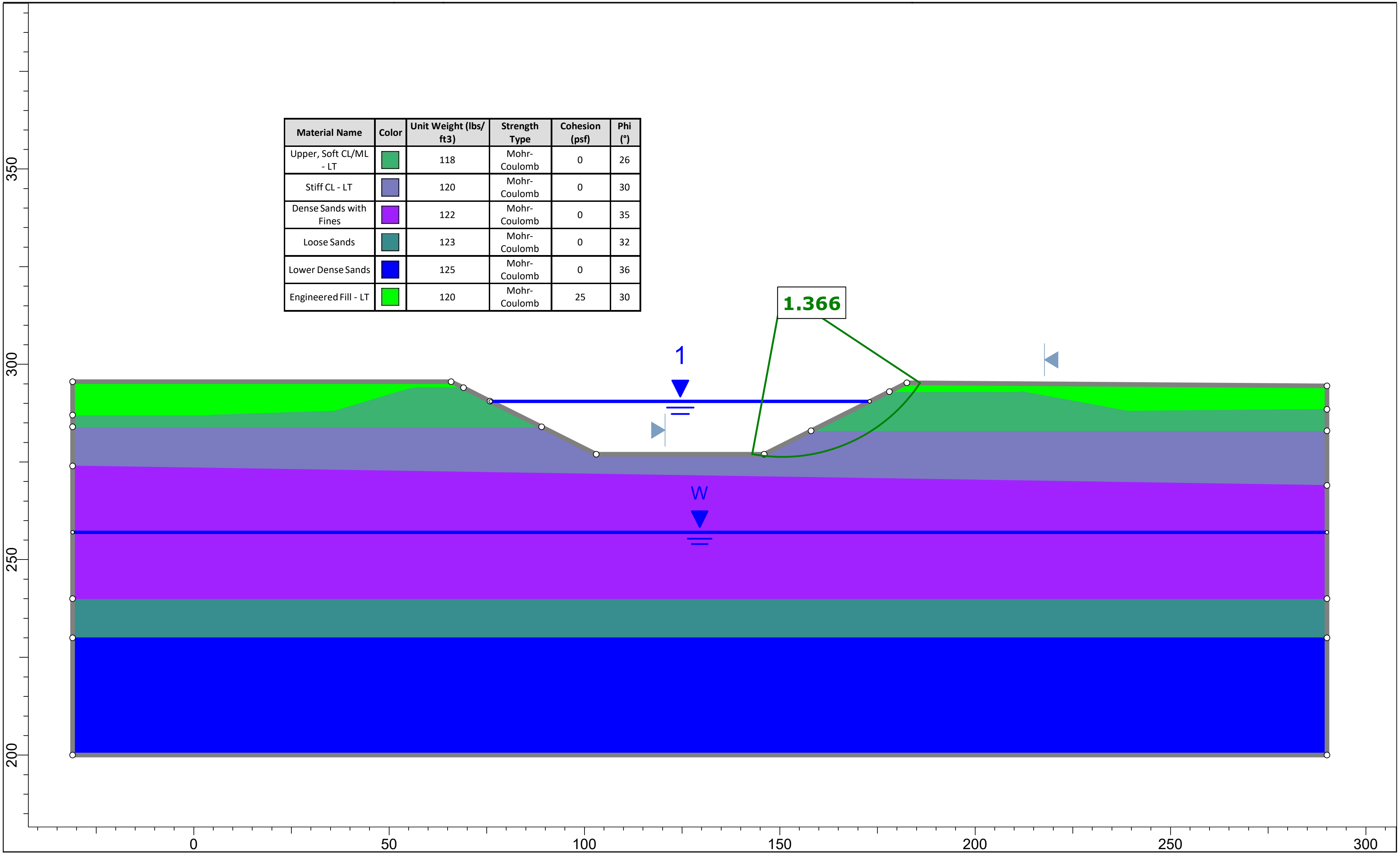
Clay County, Arkansas

SLIDEINTERPRET 9.028



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper, Soft CL/ML - LT	🟩	118	Mohr-Coulomb	0	26
Stiff CL - LT	🟪	120	Mohr-Coulomb	0	30
Dense Sands with Fines	🟪	122	Mohr-Coulomb	0	35
Loose Sands	🟦	123	Mohr-Coulomb	0	32
Lower Dense Sands	🟦	125	Mohr-Coulomb	0	36
Engineered Fill - LT	🟩	120	Mohr-Coulomb	25	30

GEOTECHNOLOGY
A UES Company

File Name: ARDOT 101120 Abutment Slope Stability.slmd
Name: Northern Abutment
Description: Seismic Conditions
Method: Spencer
Date: 8/10/2023

Project Number: J042991.01
Client: ARDOT
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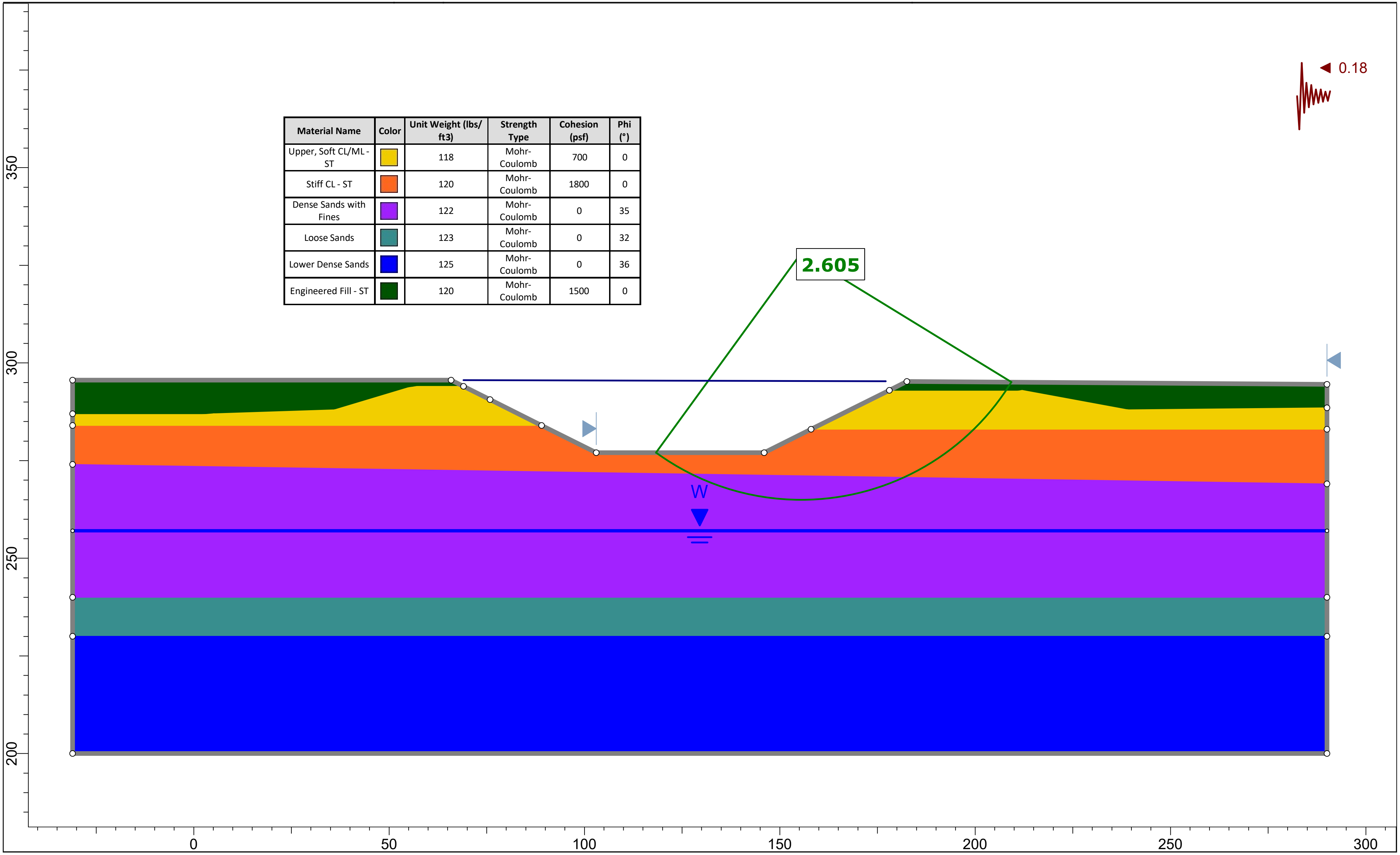
Clay County, Arkansas

SLIDEINTERPRET 9.028



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper, Soft CL/ML - ST	🟨	118	Mohr-Coulomb	700	0
Stiff CL - ST	🟧	120	Mohr-Coulomb	1800	0
Dense Sands with Fines	🟪	122	Mohr-Coulomb	0	35
Loose Sands	🟦	123	Mohr-Coulomb	0	32
Lower Dense Sands	🟦	125	Mohr-Coulomb	0	36
Engineered Fill - ST	🟩	120	Mohr-Coulomb	1500	0

APPENDIX I – SOIL PARAMETERS FOR SYNTHETIC PROFILES

HWY. 141 BRIDGE OVER DITCH NO. 8 – BENT 1 (BORING B-1)

APPROXIMATE GROUND SURFACE ELEVATION = EL 294

ZONE	SOIL TYPES	DEPTH[a] (feet from ground surface)		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]		
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	LPILE SOIL MODEL
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)			
1	Soft Lean Clay	0[b]	10	118	700	--	--	26	0.01	100	Soft Clay
2	Stiff Lean Clay / Silt	10	20	120	1,800	--	--	28	0.007	500	Stiff Clay without Free Water
3	Medium Dense Sands	20	53.5	122	--	35	--	35	--	60	Sand
4	Loose Sands	53.5	63.5	123	--	32	--	32	--	20	
5	Dense Sands	63.5	100	125	--	36	--	36	--	125	

Note: Groundwater elevation assumed at El. 257 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level. Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

[a] Depth in reference to ground surface at boring locations.

[b] Zero depth as measured at top of boring.

[c] Pounds per cubic inch.

[d] For lateral load analysis only.

HWY. 141 BRIDGE OVER DITCH NO. 8 – BENTS 2 & 3 (BORINGS B-2 & -3)

APPROXIMATE AVERAGE GROUND SURFACE ELEVATION = EL 278

ZONE	SOIL TYPES	DEPTH[a] (feet from ground surface)		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]		
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	LPILE SOIL MODEL
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)			
1	Medium Dense Sands	0[b]	33.5	122	--	35	--	35	--	60	Sand
2	Loose Sands	33.5	43.5	123	--	32	--	32	--	20	
3	Dense Sands	43.5	100	125	--	36	--	36	--	125	

Note: Groundwater elevation assumed at El. 257 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level. Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

[a] Depth in reference to ground surface at boring locations.

[b] Zero depth as measured at top of boring.

[c] Pounds per cubic inch.

[d] For lateral load analysis only.

HWY. 141 BRIDGE OVER DITCH NO. 8 – BENT 4 (BORING B-4)

APPROXIMATE GROUND SURFACE ELEVATION = EL 292

ZONE	SOIL TYPES	DEPTH[a] (feet from ground surface)		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]		
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	LPILE SOIL MODEL
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)			
1	Soft Lean Clay	0[b]	10	118	700	--	--	26	0.01	100	Soft Clay
2	Stiff Lean Clay / Silt	10	23.5	120	1,800	--	--	28	0.007	500	Stiff Clay without Free Water
3	Medium Dense Sands	23.5	52	122	--	35	--	35	--	60	Sand
4	Loose Sands	52	62	123	--	32	--	32	--	20	
5	Dense Sands	62	100	125	--	36	--	36	--	125	

Note: Groundwater elevation assumed at El. 257 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level. Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

[a] Depth in reference to ground surface at boring locations.

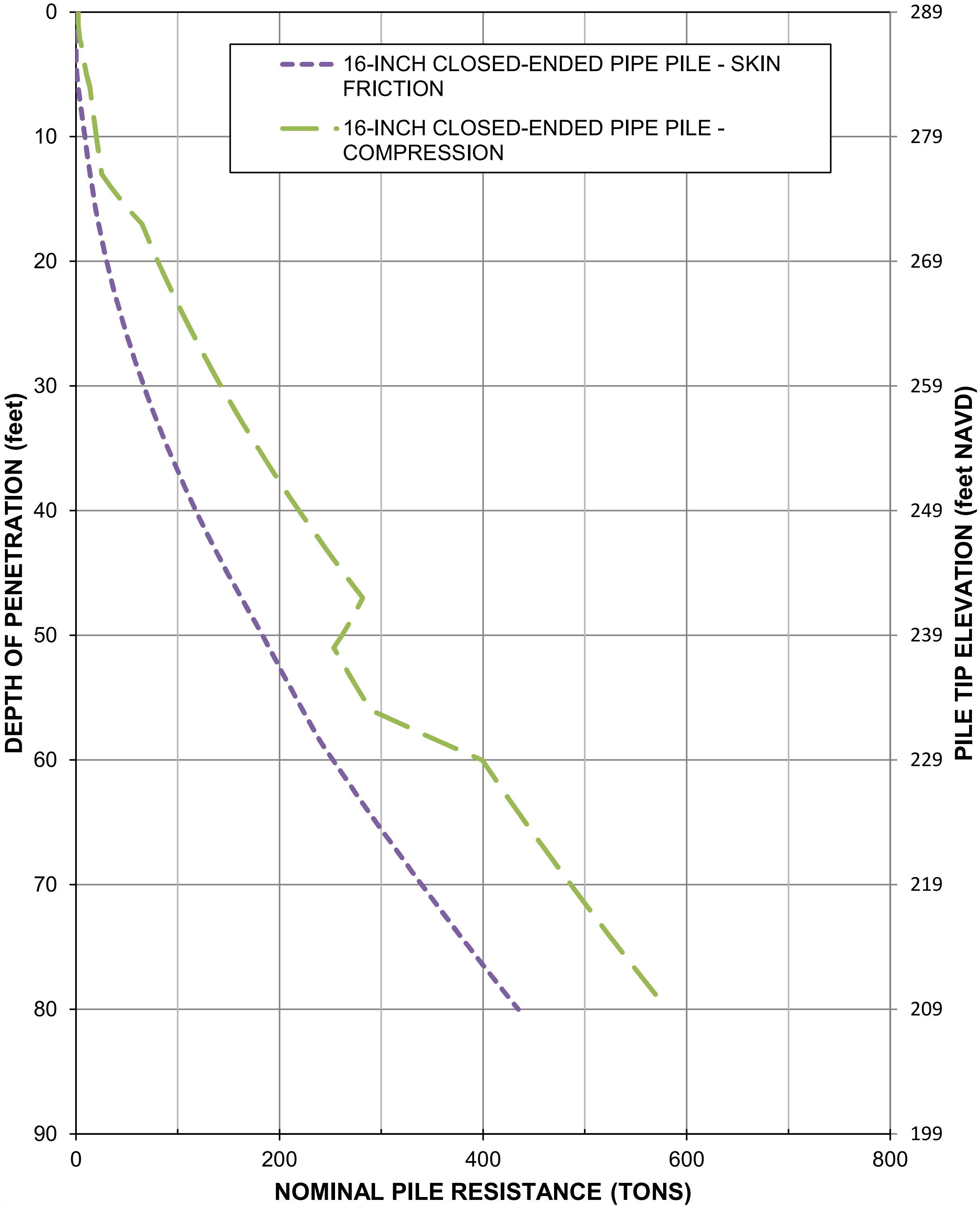
[b] Zero depth as measured at top of boring.

[c] Pounds per cubic inch.

[d] For lateral load analysis only.

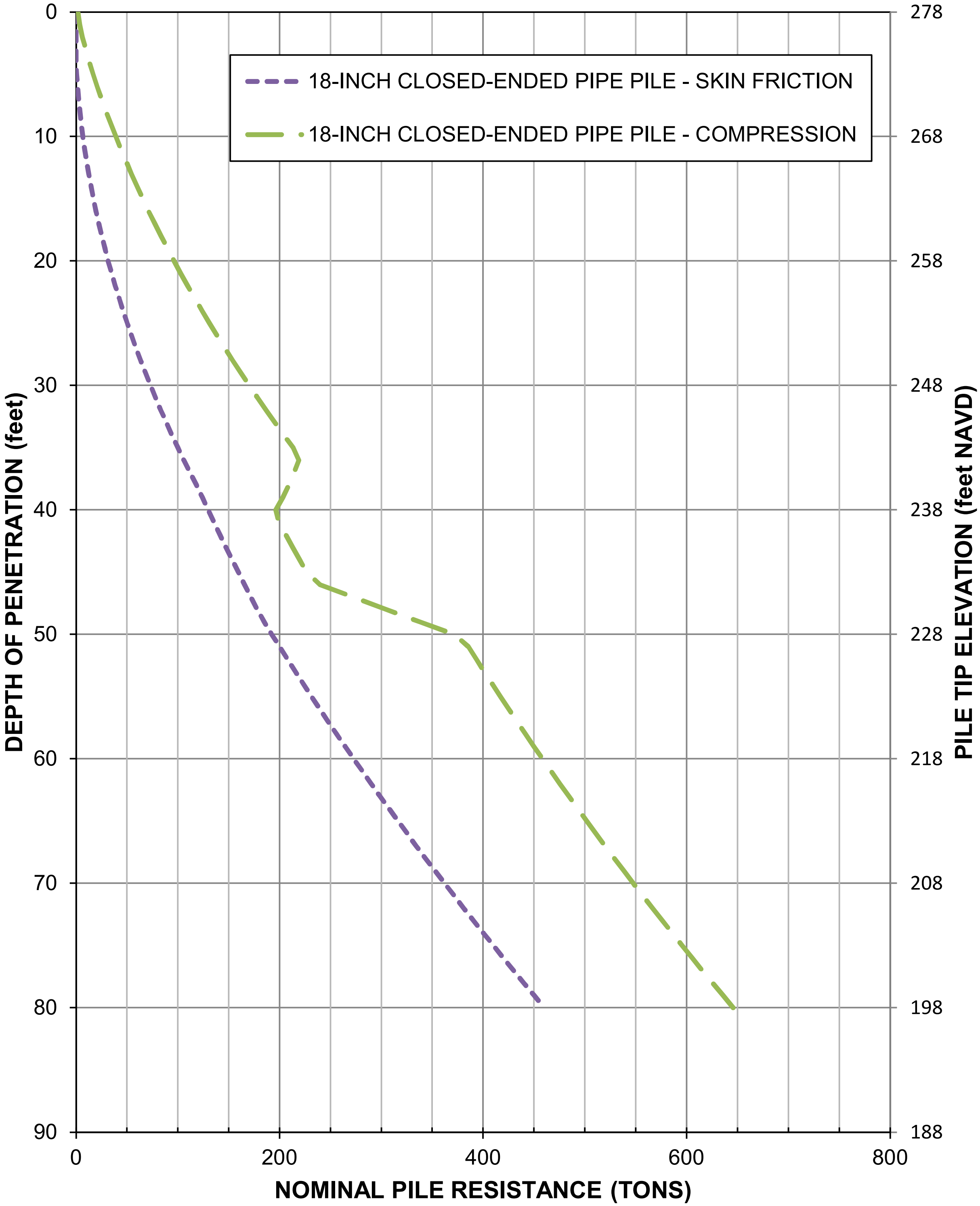
APPENDIX J – NOMINAL RESISTANCE CURVES

NOMINAL RESISTANCE CURVES
16-INCH CLOSED-ENDED PIPE PILE
ARDOT HWY. 141 OVER DITCH NO. 8
STR. & APPRS. (S), CLAY COUNTY - BENT 1



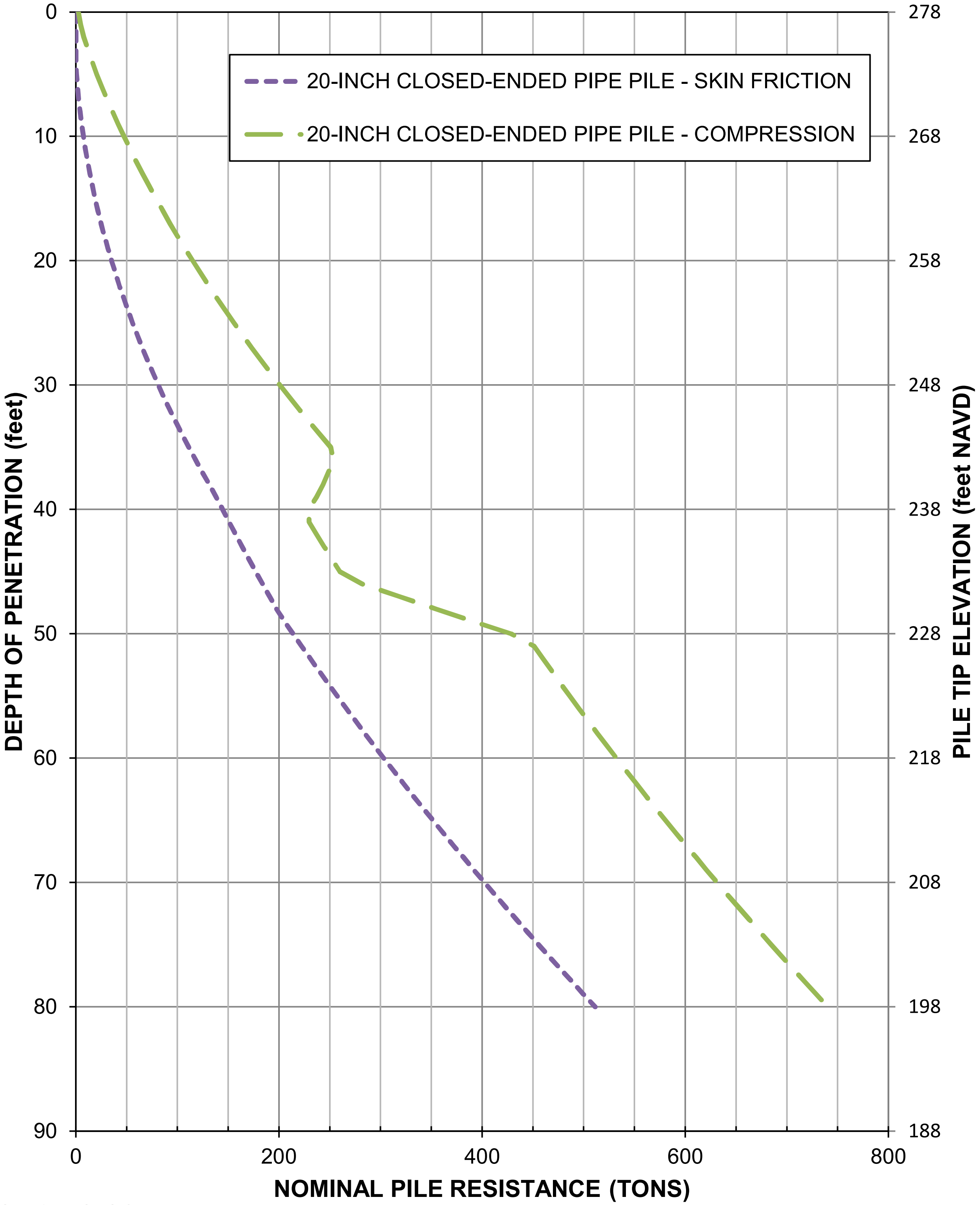
GEOTECHNOLOGY
PROJECT NUMBER J042991.01

NOMINAL RESISTANCE CURVES
18-INCH CLOSED-ENDED PIPE PILE
ARDOT HWY. 141 OVER DITCH NO. 8
STR. & APPRS. (S), CLAY COUNTY - BENTS 2 & 3



GEOTECHNOLOGY
PROJECT NUMBER J042991.01

NOMINAL RESISTANCE CURVES
20-INCH CLOSED-ENDED PIPE PILE
ARDOT HWY. 141 OVER DITCH NO. 8
STR. & APPRS. (S), CLAY COUNTY - BENTS 2 & 3



GEOTECHNOLOGY
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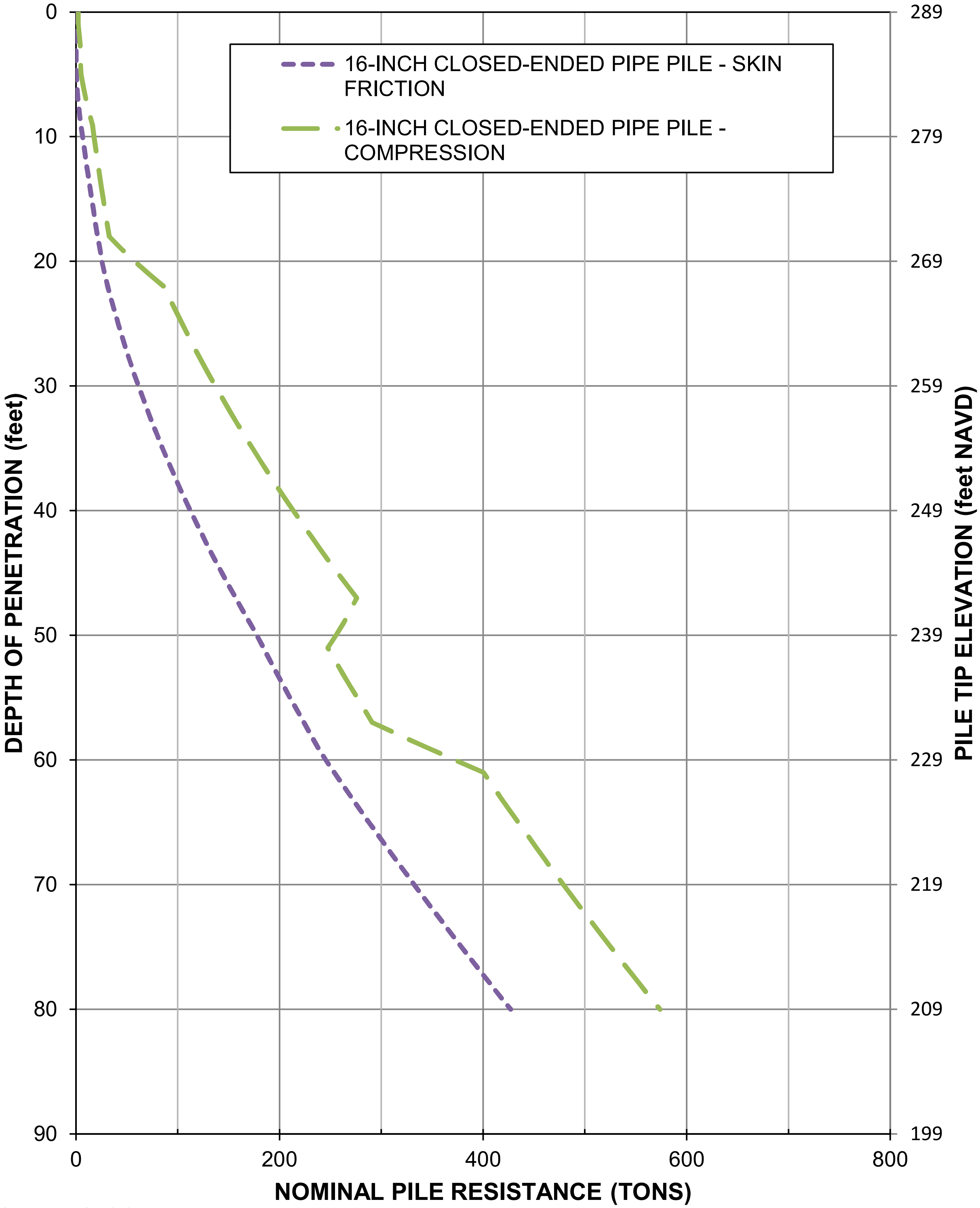
NOMINAL RESISTANCE CURVES
16-INCH CLOSED-ENDED PIPE PILE
ARDOT HWY. 141 OVER DITCH NO. 8
STR. & APPRS. (S), CLAY COUNTY - BENT 4



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