ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 101172

FEDERAL AID PROJECT NO. CPFCDS-NHPP-0011(66)

CORNING BYPASS (FUTURE I-57) (S)

STATE HIGHWAY 657 **SECTION** 3

IN CLAY **COUNTY**

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GEOTECHNICAL REPORT
CORNING BYPASS (FUTURE I-57) (S)
CLAY COUNTY, ARKANSAS

ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT NO. 101172

Prepared for:

ARKANSAS DEPARTMENT OF TRANSPORTATION (ARDOT)
LITTLE ROCK, ARKANSAS

Prepared by:

GEOTECHNOLOGY, LLC
MEMPHIS, TENNESSEE

Date:

JUNE 26, 2024

Geotechnology Project No.:

J045279.01

SAFETY
QUALITY
INTEGRITY
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GEOTECHNOLOGY
A UES Company

June 26, 2024

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

Re: Geotechnical Report
Corning Bypass (Future I-57) (S)
Clay County, Arkansas
ArDOT Project No. 101172
Geotechnology Project No. J045279.01

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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CLAY COUNTY, ARKANSAS

June 26, 2024 | Geotechnology Project No. J045279.01

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 new bridge along County Road 139 (CR139) over the proposed future Interstate 57 (I-57) alignment in Clay County, Arkansas. The proposed bridge will be part of the overall Corning Bypass project. It is our understanding the anticipated foundation type for support of the new bridge will be 16- or 18-inch, closed-ended driven pipe piles at the exterior (abutment) bents and 18- or 20-inch, closed-ended driven pipe piles at the interior (pile-supported footing) bents as provided by ArDOT. The project location is shown on Figure 1 included in Appendix B.

The project site was relocated from the original location on County Road 143 to its current location on County Road 139. Additional geotechnical exploration was performed at the relocated site; the results of the relocated geotechnical exploration are presented in this report. 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 three borings and two CPT soundings (one a 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 plots, 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 proposed CR139 over I-57 bridge will be a two-lane, four-span structure approximately 413½-foot-long and 30½-foot-wide. The proposed bridge will be constructed in one phase and is part of the overall construction of the proposed Arkansas Job 101172 Corning Bypass (Future I-57).

Riprap is planned along the abutment slopes based on the provided preliminary plans[1]; abutment slopes and side slopes are anticipated to be three horizontal units for every vertical unit (3H:1V). Up to 24 feet of fill will be required to reach design grades.

Topography

The proposed CR139 over I-57 bridge is located in Clay County, Arkansas. According to the provided plans, existing elevations at the south and north abutments of the proposed bridge location are approximately El 292.

Drainage

The drainage system in the project area consists of the Murray Creek Ditch-Black River Watershed, a division of the Upper Black Watershed. The Upper Black 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, a division of the overall Mississippi Alluvial Plain. The Mississippi Embayment is a trough-like depression dipping southward along an axis approximately following the Mississippi River. The site is located near the western extent of the Mississippi Embayment in Arkansas.

The Geologic Map of Arkansas[2] indicates the upper geology of the site generally consists of Pleistocene-age Dune Sand and Terrace deposits. Dune Sand deposits are deposited mostly on the Terrace Deposits of the second level and are generally younger than some Terrace deposits but older than others. The Terrace deposits are alluvial deposits on one or more terrace levels.

3.0 GEOTECHNICAL EXPLORATION

Cone Penetration Testing

Two cone penetration testing (CPT) soundings were performed in the proposed bridge alignment for continuous soil data collection. One seismic CPT sounding (CPT-4) was performed to measure the average shear wave velocity of the soils at the site. The CPT soundings were performed to depths of approximately 100 feet.

The CPT soundings were advanced using a 20-ton, track-mounted Vertek direct-push rig on April 10, 2024. 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. Plots of the CPT measurements are presented in Appendix D along

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[1] *"A Fully Controlled Access Facility", Arkansas Department of Transportation, Construction Plans for State Highway, Corning Bypass (Future I-57) (S), Clay County, Route 139, Section 7, Job 101172*. Prepared by Arkansas State Highway Commission, dated February 8, 2024.

[2] Geologic Map of Arkansas, Boyd R. Haley, U.S. Geological Survey assisted by Ernest E Glick, U.S. Geological Survey and William V. Bush, Benjamin F. Clardy, Charles G. Stone, and Doy L. Zachry, Arkansas Geological Commission, 1993.

with interpreted soil behavior types. Seismic CPT tests were performed in CPT-4 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 three borings were drilled to an approximate depth of 100 feet at select locations in the proposed bridge alignment. The borings were drilled on April 2 through 4, 2024 using a rotary drill rig (CME 550X), 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, 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
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

The borings and CPT soundings were performed in the alignment of the proposed Bridge. The ground surface at the exploration locations was covered with approximately 2 inches of topsoil. Below the surficial material, the soil stratigraphy at the exploration locations generally consisted of predominately fine-grained soils underlain by predominately coarse-grained soils that extended to the maximum depth of exploration. The boring logs and CPT sounding plots, with more detailed descriptions, are provided in Appendix C and Appendix D, respectively. Laboratory testing was used to determine the AASHTO classifications as presented in Appendix E.

The upper, fine-grained soils were interpreted and classified as high plasticity “fat” clay (CH), A-7-5, A-7-6; low plasticity “lean” clay (CL), A-7-6; and silt (ML), A-4. The upper, fine-grained soils ranged from medium stiff to stiff in consistency.

The lower, coarse-grained soils were interpreted and classified as poorly graded sand (SP), A-3, A-1-b, A-3; poorly graded sand with clay (SP-SC), A-2-6; clayey sand (SC); silty sand (SM); and poorly graded sand with silt (SP-SM). The lower, coarse-grained soils ranged from loose to very dense conditions.

Groundwater

Groundwater was encountered in Borings B-1, -3, and -5 at depths of approximately 18.5 feet. Groundwater was interpreted in CPT-2 and -4 at depths of approximately 20 feet. Groundwater levels encountered in the borings and interpreted in the CPT soundings correlate to groundwater elevations ranging from approximately El 272 to El 273.5. Groundwater levels could vary significantly over time due to the effects of seasonal variations in precipitation 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 3H: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 cut sections will not be required at the proposed bridge abutments. Based on the stratigraphy, excavations for pile-supported footings at interior bents will terminate in fat clay at Bents 2 and 3 and sand at Bent 4. 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[3], and should meet the minimum requirements set forth in ARDOT's Special Provision[4] (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

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

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

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.

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), and “AASHTO Guide Specifications for LRFD Seismic Bridge Design”, second edition (2011) with current interims.

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 36.431220°N/Longitude 90.622202°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	0.595g	Ground motion parameters obtained from a computer program supplied with the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition (2011), with 2022 Interims, using the indicated latitude and coordinates of the project site and the seismic site class based on boring data.
S_1	0.154g	
F_a	1.324	
F_v	2.183	
F_{PGA}	1.191	
Seismic Design Category (SDC)	C	
S_{DS}	0.788g	
S_{D1}	0.337g	
PGA	0.309g	
As	0.368g	

Site-Specific Ground Motion Response Analysis

A site-specific study was performed for the original project site location (CR-143) 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 744 feet per second at the original project site. An additional shear wave velocity was performed at CPT-4 of the relocated bridge; the measured $V_{S,100}$ at the relocated bridge was 702 feet per second. A plot of the shear wave velocity profiles and $V_{S,100}$ measurements at the original and relocated bridge locations is 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 profile. 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}	0.575g
S_{D1}	0.388g
MCE_G	0.295g

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-2 and -4. 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 0.295g was utilized as obtained from the site-specific seismic study.

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

Table 5. Results of Liquefaction Analyses.

Location	Approximate Groundwater Depth (feet below ground surface)	Liquefiable Zones[a] (feet below existing ground surface)	Estimated Dynamic Settlement in Upper 50 Feet[b] (inches)
B-1	18.5	18.5 – 50[b]	13½
CPT-2	20	21 – 23.5 | 31 – 36.5 | 37.5 – 50[b]	6
B-3	18.5	18.5 – 23.5 | 28.5 – 33.5 | 38.5 – 48.5	11
CPT-4	20	22.5 – 26 | 31.5 – 37 | 39 – 44 | 45 – 50[b]	5½
B-5	18.5	33.5 – 48.5	6½

[a] Defined as zones with a factor of safety against liquefaction in the design earthquake of less than 1.0, as computed using the deterministic method by Idriss and Boulanger, 2014.

[b] The occurrence of substantial liquefaction in relatively deep sand deposits below 50 feet is considered unlikely as discussed below; liquefaction and dynamic settlement deeper than 50 feet below existing ground surface not considered.

Please note that the current state of practice for liquefaction hazard assessment is based on what is known as “The Simplified Method” as introduced by Seed (1971) and subsequent modifications/revisions by others (Seed 1982, Idriss 1999, Youd 2001, and Idriss and Boulanger 2008, among others). The Simplified Method was based on observations and assessments of soil zones that either liquefied or did not liquefy in the upper 50 feet. Because of reported uncertainties in the literature, the occurrence of substantial liquefaction in relatively deep sand deposits below 50 feet is considered unlikely.

Lateral Spreading. Lateral spreading is triggered and sustained by earthquake ground motions. Based on our seismic slope stability analyses, it is our professional opinion the potential for lateral spreading is low at the site. However, in the event of an earthquake, the soils that liquefy will have residual strengths which can have potentially destabilizing effects on overlying slopes. Post-liquefaction residual shear strength parameters are presented in Appendix I.

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 24 and 23 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 6. 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 6. Summary of Estimated Settlement.

Southern Abutment (Exterior Bent No. 1)				Northern Abutment (Exterior Bent No. 5)			
Max. Fill (feet)	Estimated Settlement (inches)			Max. Fill (feet)	Estimated Settlement (inches)		
	Immediate	Long-Term (Consolidation)	Total		Immediate	Long-Term (Consolidation)	Total
24	4½	3	7½	23	5	3	8

The bent numbers presented in Table 6 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 5 such that Bent No. 1 is at the southern abutment. The bents are numbered in succession from south to north along the proposed bridge alignment with exterior Bent No. 5 at the northern abutment of the bridge.

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

It should be noted the estimated time to achieve practical completion of consolidation settlement is based on one-dimensional consolidation tests performed in our laboratory on samples recovered in the in-situ clay soils. The consolidation test confines the drainage path during sample loading to one dimension; in the field, drainage may take place in three dimensions. Therefore, it is our professional opinion the estimated settlement will occur in a shorter period of time; however, we are not able to accurately estimate the shorter time. Proposed methods of ground improvement and expedition of consolidation settlement are presented subsequently in this report.

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 0.1475g, 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; an average groundwater elevation of approximately El 273.5 feet, as encountered in the borings and interpreted in the CPT soundings, was utilized for the 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 7. Results of Slope Stability Analyses.

Location	**Description**	**Slope Height (ft.)**	**Calculated Factor of Safety**		
			Short-Term Static[a,c]	**Long-Term Static**[a,c]	**Seismic**[b,c]
Southern Abutment STA 210.08+80	3H:1V Fill Slope	17	3.09	1.99	1.97
Northern Abutment STA 214+22.20	3H:1V Fill Slope	15.5	3.18	2.05	2.08
South Abutment South Side Slope	3H:1V Fill Slope	25	3.11	1.98	2.03
Northern Abutment South Side Slope	3H:1V Fill Slope	25	3.54	2.02	2.28

[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 273.5 as interpreted in the CPT soundings and encountered in the borings.

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; closed-ended pipe pile sizes of 16- and 18-inch diameter were considered for exterior Bent Nos. 1 and 5 and closed-ended pipe pile sizes of 18- and 20-inch diameter were considered for interior Bent Nos. 2 through 4 as provided by Garver. Pile wall thicknesses of 1/2-inch for the 16- and 18- inch diameter piles and 5/8-inch for the 20-inch diameter piles were considered. Geotechnology should be notified if other foundation types or sizes are to be considered. 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 resistance (skin friction + end bearing) for Bent Nos. 1 through 5 are presented in Appendix J. Nominal resistances for each bent location are presented in Table 8 through Table 10. Uplift (tension) resistances may be calculated using the resistance provided by skin friction. Embedment lengths presented for piles driven at exterior Bent Nos. 1 and 5 are in reference to bottom of pile cap elevations of El 307.7 and El 306.6, respectively as provided by Garver. Embedment lengths presented for piles driven at interior Bent Nos. 2 through 4 are in reference to bottom of pile-supported footing elevations ranging from El 278.3 to El 283.6 as provided by Garver.

Static Settlement-Induced Downdrag at Exterior Bent Nos. 1 and 5. The resistance values presented for exterior Bent Nos. 1 and 5 are subjected to downdrag loads imposed by fill placement at the abutments if essential completion of settlement due to fill placement is not allowed prior to driving of piles. Presented in Table 8 are nominal (unfactored) drag loads due to consolidation settlement at exterior Bent Nos. 1 and 5 based on the cumulative side resistance above the depth below the bottom of pile caps where approximately 0.4 inch of consolidation settlement is predicted to occur. Piles placed at exterior Bent Nos.1 and 5 should be designed to account for drag loads imposed on the piles due to the downward movement of soils if essential completion of consolidation settlement is not allowed.

Table 8. Nominal Axial Resistance of Driven Closed-Ended Pipe Piles – Exterior Bent Nos. 1 and 5.

Location	Pile Diameter (inches)	Embedment Length[a] (feet)	Nominal Static Resistance (tons)			Nominal Post-Consolidation Resistance (tons)	
			Skin Friction	End Bearing	Compression Total	Compression Total[b]	Nominal Drag Load
Southern Abutment (Exterior Bent No. 1; Boring B-1)	16	60	256	126	382	352	30
		70	334	140	474	444	
		80	420	140	560	530	
	18	60	288	157	445	411	34
		70	376	177	553	519	
		80	473	177	650	616	
Northern Abutment (Exterior Bent No. 5; Boring B-5)	16	60	266	92	358	331	27
		70	334	90	424	397	
		80	410	119	529	502	
	18	60	299	118	417	387	30
		70	376	114	490	460	
		80	461	149	610	580	

[a] Embedment length referenced from bottom of pile cap elevations at Exterior Bents 1 and 5 of El 307.7 and El 306.6, respectively. Piles anticipated to extend through approximately 16 and 15 feet of fill material, respectively, placed at the exterior bent locations.

[b] Nominal post-consolidation resistance has not been reduced by the drag load. Nominal drag load provide in right-most column.

Liquefaction-Induced Downdrag. Post-liquefaction nominal resistances and nominal drag loads on piles due to liquefaction settlement are presented in Table 9 and Table 10 for the exterior and interior bents, respectively. For analysis of post-liquefaction pile resistance, liquefaction was limited to the upper 50 feet of the natural soil profile. Post-liquefaction nominal resistance curves are presented in Appendix J.

Table 9. Nominal Axial Resistance of Driven Closed-Ended Pipe Piles – Exterior Bent Nos. 1 and 5.

Location	Pile Diameter (inches)	Embedment Length[a] (feet)	Nominal Static Resistance (tons)			Nominal Post-Liquefaction Resistance[b] (tons)	
			Skin Friction	End Bearing	Compression Total	Compression Total[c]	Nominal Drag Load
Southern Abutment (Exterior Bent No. 1; Boring B-1)	16	60	256	126	382	160	80
		70	334	140	474	252	
		80	420	140	560	337	
	18	60	288	157	445	195	90
		70	376	177	553	303	
		80	473	177	650	399	
Northern Abutment (Exterior Bent No. 5; Boring B-5)	16	60	266	92	358	133	147
		70	334	90	424	200	
		80	410	119	529	304	
	18	60	299	118	417	164	166
		70	376	114	490	238	
		80	461	149	610	357	

[a] Embedment length referenced from bottom of pile cap elevations at Exterior Bents 1 and 5 of El 307.7 and El 306.6, respectively. Piles anticipated to extend through approximately 16 and 15 feet of fill material, respectively, placed at the exterior bent locations.

[b] For the purposes of downdrag analyses, liquefaction-induced downdrag has been limited to the upper 50 feet of the soil profile below existing ground surface elevation. Liquefiable layers below 50 feet not considered for downdrag analyses.

[c] Nominal post-liquefaction resistance has not been reduced by the drag load. Nominal drag load provide in right-most column.

Table 10. Nominal Static Axial Resistance of Driven Closed-Ended Pipe Piles – Interior Bents 2 through 4.

Location	Pile Diameter (inches)	Embedment Length (feet)	Nominal Static Resistance (tons)			Nominal Post-Liquefaction Resistance[d] (tons)	
			Skin Friction	End Bearing	Compression Total	Compression Total[e]	Nominal Drag Load
Interior Bent No. 2[a] (CPT-2)	18	60	189	112	301	209	37
		70	251	96	347	255	
		80	320	186	506	414	
	20	60	210	138	348	246	41
		70	279	118	397	295	
		80	356	229	585	483	
Interior Bent No. 3[b] (Boring B-3)	18	60	185	75	260	168	44
		70	242	104	346	254	
		80	314	169	483	391	
	20	60	206	92	298	195	49
		70	269	130	399	296	
		80	348	209	557	455	
Interior Bent No. 4[c] (CPT-4)	18	60	186	94	280	205	32
		70	247	159	406	330	
		80	316	167	483	408	
	20	60	206	118	324	241	36
		70	274	196	470	386	
		80	351	207	558	474	

[a] Embedment length referenced from bottom of pile-supported footing elevation of El 282.6.

[b] Embedment length referenced from bottom of pile-supported footing elevation of El 283.6.

[c] Embedment length referenced from bottom of pile-supported footing elevation of El 278.3.

[d] For the purposes of downdrag analyses, liquefaction-induced downdrag has been limited to the upper 50 feet of the soil profile below existing ground surface elevation. Liquefiable layers below 50 feet not considered for downdrag analyses.

[e] Nominal post-liquefaction resistance has not been reduced by the drag load. Nominal drag load provide in right-most column.

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 11 may be applied.

Table 11. 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 12.

Table 12. 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 pile depends on the final foundation configuration.

Downdrag Due to Dynamic Settlement. Based on the results of the liquefaction analyses, dynamic settlement in the range of 5½ to 13½ inches is anticipated in the upper 50 feet of soil during the design earthquake event (7% probability of exceedance in 75 years) at the exploration locations. Liquefaction-induced downdrag can be reduced by performing ground improvement on the potentially liquefiable layers. It is our professional opinion that driving the closed-ended pipe piles through the liquefiable layers will densify the susceptible layers in the vicinity of the piles; however, the amount and extent of densification cannot be ascertained during the design phase of the project. However, the extent of densification can be determined by advancing CPT soundings in the vicinity of the piles after they are driven. Pre-drilling or applying viscous coatings to pile are not recommended to reduce liquefaction-induced downdrag, because such methods will also reduce the nominal static compressive resistance of the piles. Please contact Geotechnology if more information is desired.

<u>Downdrag Due to Fill-Induced Settlement</u>. Based on settlement analyses performed for the maximum fill placement at the abutments, up to 8 inches of settlement is predicted. We anticipate consolidation settlement to take up to approximately 4 months to achieve essential completion.

Piles driven through the fill embankment at exterior Bents 1 and 5 could be subject to downdrag as the soil consolidates under the fill load. Nominal (unfactored) drag loads from consolidation settlement at exterior Bents 1 and 5 are presented in Table 8 based on the cumulative skin friction resistance above the depth where approximately 0.4 inches of consolidation settlement is predicted to occur. Piles driven at exterior Bents 1 and 5 should be designed to account for drag loads imposed on the piles due to the downward movement of soils.

The following options are presented as methods for accommodating for the fill-induced settlement and downdrag loads on piles placed at exterior Bents 1 and 5. Options 1, 2, and 3 are presented if piles at the exterior bent locations are to be driven after essential completion of consolidation settlement is achieved; in this case, downdrag will not mobilize and will have minimal effect on piles placed at exterior Bents 1 and 5. Option 4 is presented if piles at exterior Bents 1 and 5 are to be driven immediately after fill placement; in this case, downdrag will be exerted on the pile.

1. Driving of piles and continued construction of the abutments can commence as soon as fill-induced settlement at exterior Bents 1 and 5 is essentially complete (less than 0.4 inches of settlement remaining). We recommend a settlement monitoring system be implemented and survey data be sent to Geotechnology to estimate when settlement is essentially complete. The recommended settlement monitoring program is discussed subsequently in this report.

2. To accelerate consolidation settlement, the upper 5 feet of clayey soils in the areas of proposed fill placement can be undercut and replaced with coarse-grained engineered fill. Recommendations for undercut of in-situ soils and replacement with coarse-grained engineered fill, referred to as a clean sand blanket, are provided subsequently in this report.

3. To accelerate consolidation settlement further, the in-situ, clayey soils can be undercut fully to the lower, sandy soils and replaced with engineered fill. Further recommendations for full-depth undercut and replacement of the in-situ, clayey soils are provided subsequently in this report.

4. In lieu of ground improvement alternatives, piles can be driven immediately after fill placement if pile lengths and configurations account for the drag loads imposed by settlement due to fill placement. Drag loads imposed on piles driven at exterior Bent Nos. 1 and 5 prior to essential completion of fill-induced settlement are presented in Table 8.

Settlement Monitoring

The duration of subgrade settlement at the bridge approaches is difficult to predict due to variations in permeability and drainage of the different soil layers. Settlement monitoring can be utilized at the bridge abutments to determine when pile foundations and roadway components can be constructed.

Settlement monitoring devices may include settlement plates and riser pipes, hydrostatic level cells referenced to a fixed fluid reservoir, or horizontal inclinometers installed at the base of the fill prior to fill placement. Settlement plates are the simplest method but are prone to disturbance or destruction of grading equipment. Therefore, redundant settlement plate locations should be installed.

Settlement plates can be installed approximately 1-foot below the existing ground surface and extend in 5-foot calibrated increments as the height of embankment fill increases. To protect the riser pipes, fill should be hand-compacted within a 4-foot-radius of each plate. A typical settlement plate detail is presented in Figure 4 in Appendix B. We recommended settlement plates be placed no further than 50 feet apart, with at least one in the deepest area of fill at the abutments. The project surveyor should be retained to monitor the settlement plate riser pipe. Settlement at the site should be measured twice weekly during fill placement and weekly after filling is completed.

Ground Improvement – 5-Foot Undercut and Backfill with Clean Sand Blanket

Described subsequently are recommendations for undercut of in-situ soils at the abutment locations and replacement with coarse-grained engineered fill, referred to as a clean sand blanket, as described in ARDOT's Special Provision[5]. Replacement of the in-situ clay soils with a clean sand blanket will facilitate drainage of excess pore water pressure at the top of the clayey soils generated by embankment fill placement, as well as reduce the amount of predicted consolidation settlement. Settlement analyses were performed assuming a minimum of 5 feet of in-situ soil is undercut and replaced with a clean sand blanket. Presented in Table 13 are the reduced predicted consolidation settlements at exterior Bents 1 and 5.

Table 13. Reduction of Consolidation Settlement – 5-Foot Clean Sand Blanket Undercut.

Location	Estimated Consolidation Settlement (inches)	
	In-Situ Clayey Soils (No Undercut)	5 Feet of Undercut Clean Sand Blanket
Southern Abutment (Exterior Bent 1)	3	2
Northern Abutment (Exterior Bent 5)	3	2

At exterior Bents 1 and 5, it is recommended the undercut and backfilled clean sand blanket extend a minimum of 5 feet past the toe of the abutment slope and 5 feet past the toe of side slopes of the abutment. The clean sand blanket should also extend a minimum of 100 feet behind the crest of the abutment slope. The granular backfill should be drained to a deeper swale or storm drain.

Based on the analyses of estimated consolidation settlement with 5 feet of undercut and replacement of the in-situ clayey soils with a clean sand blanket, the estimated amount of consolidation settlement

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[5] Special Provision "Sand Drainage Blanket", developed by ARDOT, dated January 10, 2022.

is reduced at the exterior bent locations. The estimated consolidation settlement at exterior Bents 1 and 5 is anticipated to take approximately 30 days to be essentially complete after placement of fill.

Ground Improvement – Full-Depth Undercut and Replacement of Clayey Soils

Described subsequently are recommendations for full-depth undercut of in-situ clayey soils at the abutment locations and replacement with engineered fill. Based on the depth of in-situ clayey soils encountered in Borings B-1 and -5, undercut depths ranging from 18.5 to 13.5 feet will be required to fully remove the in-situ clayey soils. The depth of undercut will vary across the proposed area of embankment fill and could be deeper than the depths encountered in the borings. Engineered fill used as backfill of the undercut soils should consist of "sandy" soils as described in the Fill Materials section of this report and placed as described in the Fill and Backfill Placement section. The undercut and "sandy" soil backfill should extend a minimum of 5 feet beyond the toe of the spile slope and side slopes of the abutment, and at least 100 feet back from the abutments. The "sandy" soil backfill should be drained to a deeper swale or storm drain.

If full-depth undercut and replacement of the in-situ clayey soil is performed, it is anticipated consolidation settlement will not occur at the site under embankment fill placement site. Based on analyses performed for full-depth undercut and replacement, up to 5 and 6 inches of immediate settlement is anticipated at the southern and northern approaches, respectively. It is anticipated settlement will be immediate and will occur shortly after completion of fill placement.

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 14 and are included in Appendix E.

Table 14. Results of pH and Soil Resistivity Testing.

Boring	Sample No.	Sample Depth (feet)	pH	Soil Resistivity (ohm-cm)
B-1	SS-7	18.5	8.60	3,648
	SS-10	33.5	8.40	4,845
B-3	SS-9	28.5	7.84	6,555
	SS-12	43.5	8.34	8,151
B-5	SS-8	23.5	8.89	5,301
	SS-13	48.5	8.78	8,322

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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

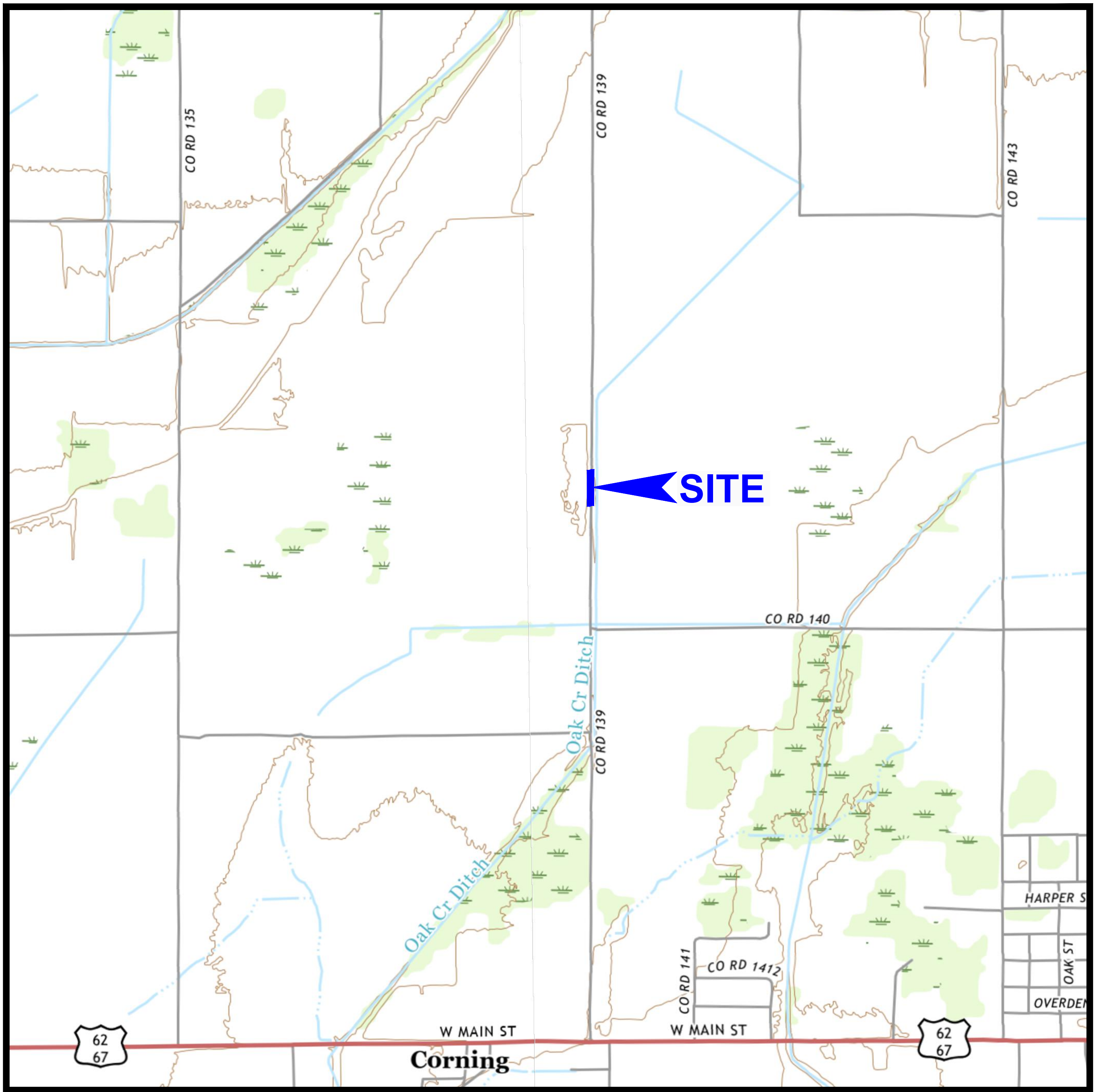
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B – FIGURES



NOTES

1. Plan adapted from 7.5 minute U.S.G.S. maps for Corning and Datto, Arkansas quadrangles, 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: 4-23-24	Date: 5-16-24	Date: 5-16-24

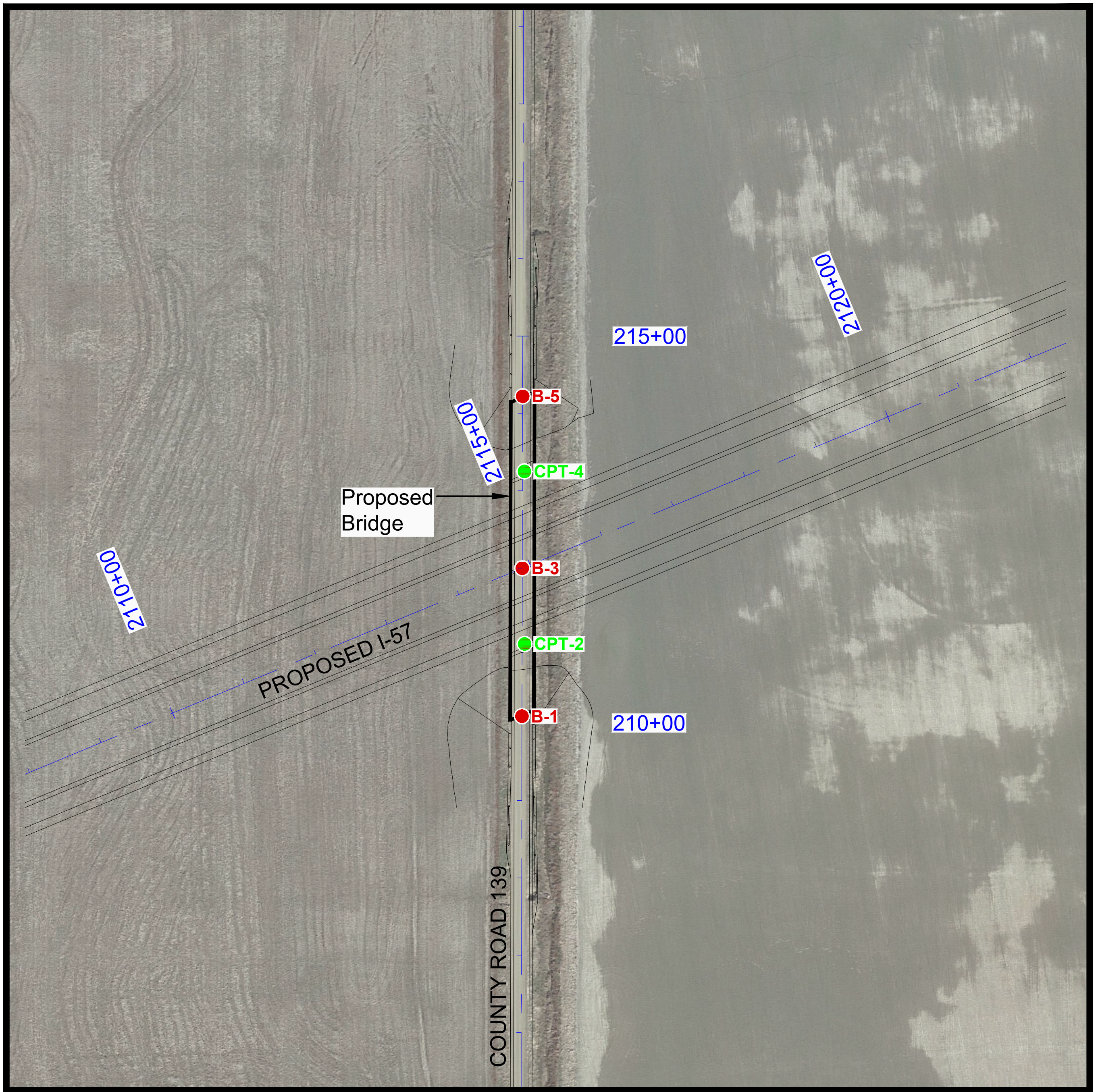
GEOTECHNOLOGY
A UES Company

ARDOT 101172 Corning Bypass
(Future I-57) (S)
Clay County, Arkansas

SITE LOCATION AND TOPOGRAPHY

Project Number
J045279.01

FIGURE 1



NOTES

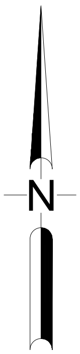
1. Plan adapted from a November 9, 2022 aerial photograph courtesy of Google Earth and a drawing dated March 20, 2024, titled "Sheet 1 of 3 Layout of Bridge County Road 139 Over I-57", prepared by Arkansas Site Highway Commission.

2. Borings 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: 4-23-24	Date: 5-16-24	Date: 5-16-24

GEOTECHNOLOGY
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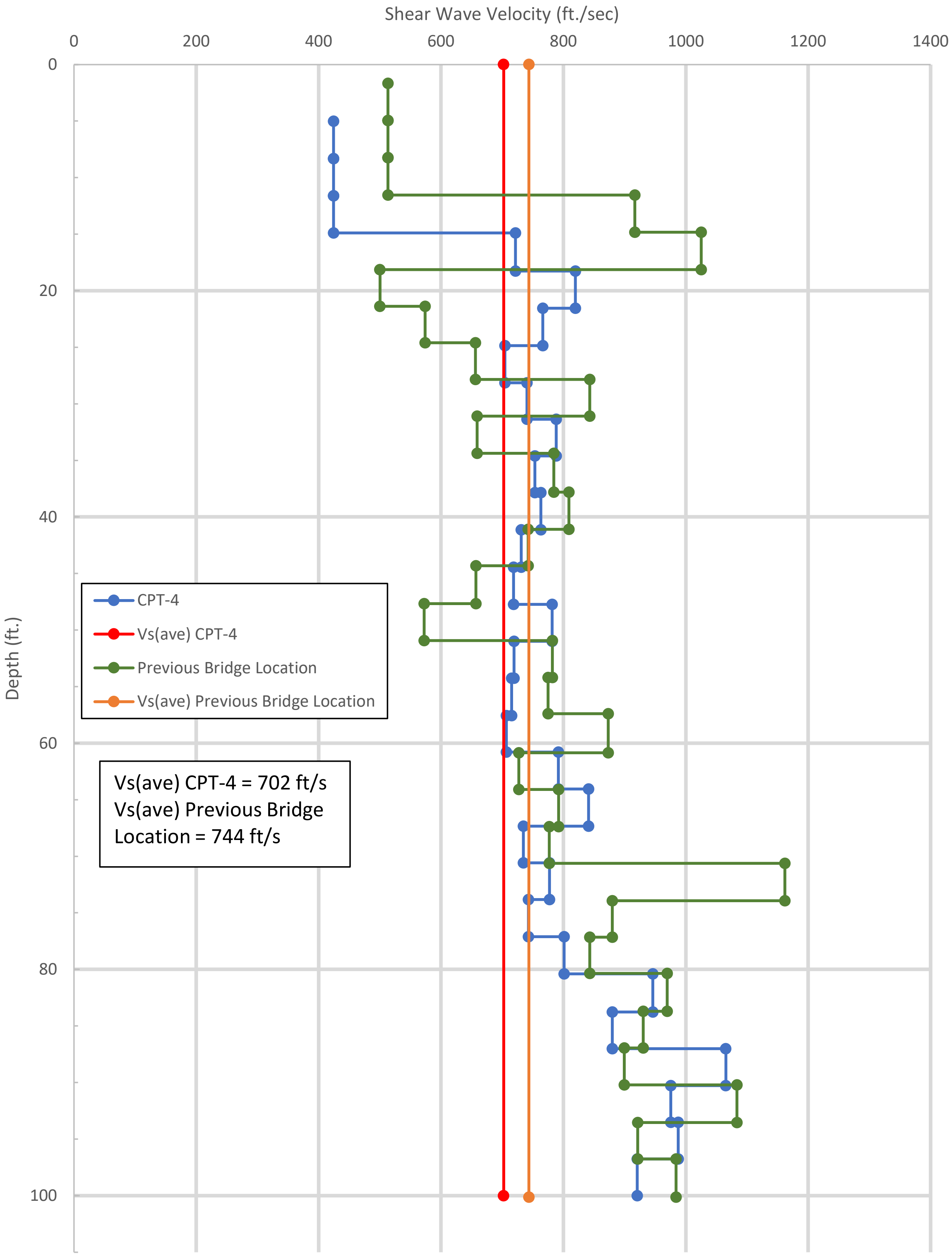
ARDOT 101172 Corning Bypass
(Future I-57) (S)
Clay County, Arkansas

AERIAL PHOTOGRAPH OF SITE AND BORING LOCATIONS

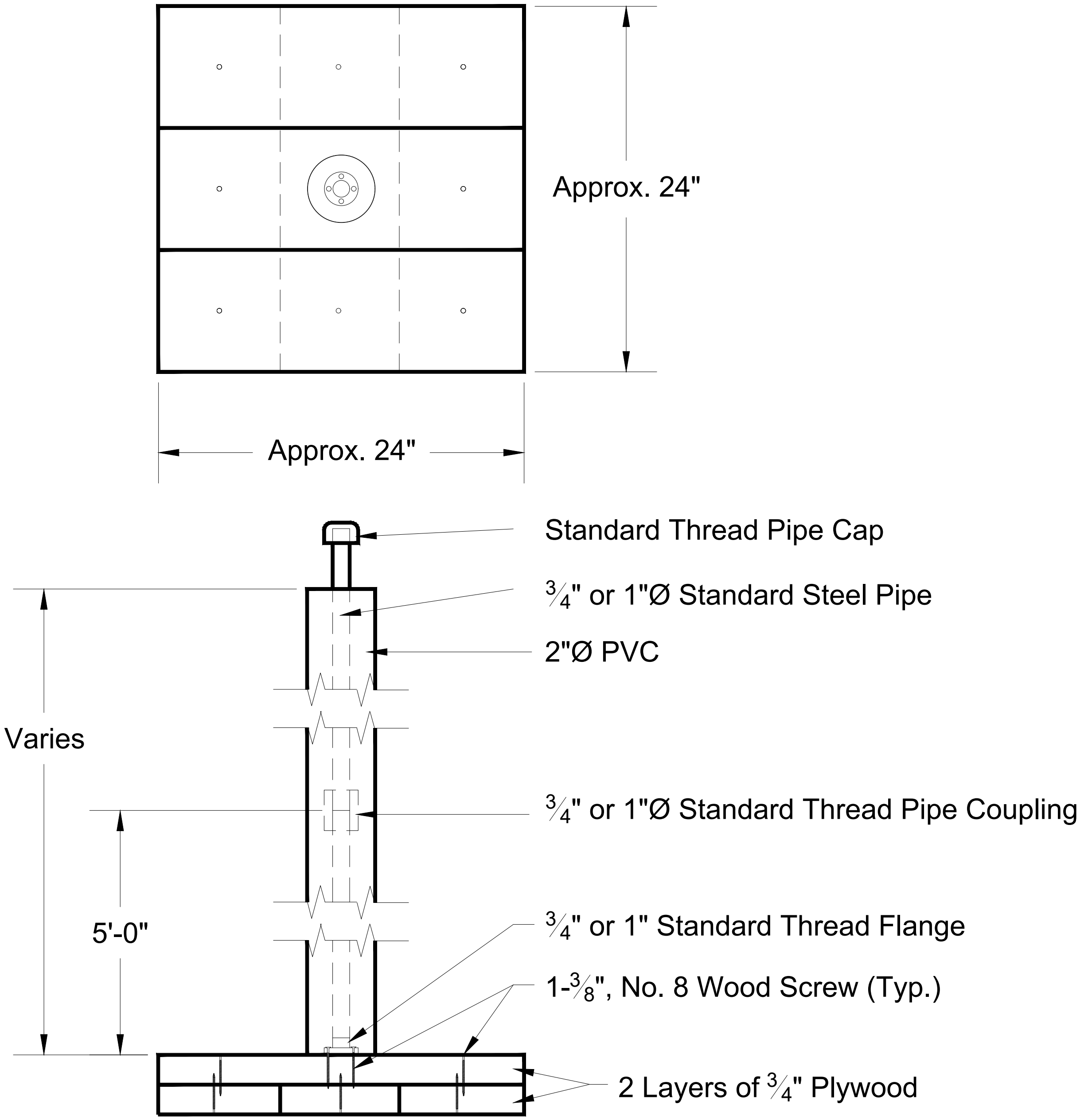
Project Number
J045279.01

FIGURE 2

Figure 3 - Shear Wave Velocity Profile



Vs(ave) CPT-4 = 702 ft/s
Vs(ave) Previous Bridge Location = 744 ft/s



NOTES

1. Place plate on level surface, a minimum of 2 feet below ground level and hand compact backfill adjacent to PVC.

Drawn By: WAH	Ck'd By: JDM	App'vd By: ASE
Date: 11-1-23	Date: 5-14-24	Date: 5-14-24

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Corning Bypass (Future I-57)(S)
Clay County, Arkansas

SETTLEMENT PLATE DETAIL

Project Number
J045279.01

FIGURE 4

APPENDIX C – BORING INFORMATION

Boring Logs

Boring Log Terms and Symbols

Surface Elevation: **291.52**

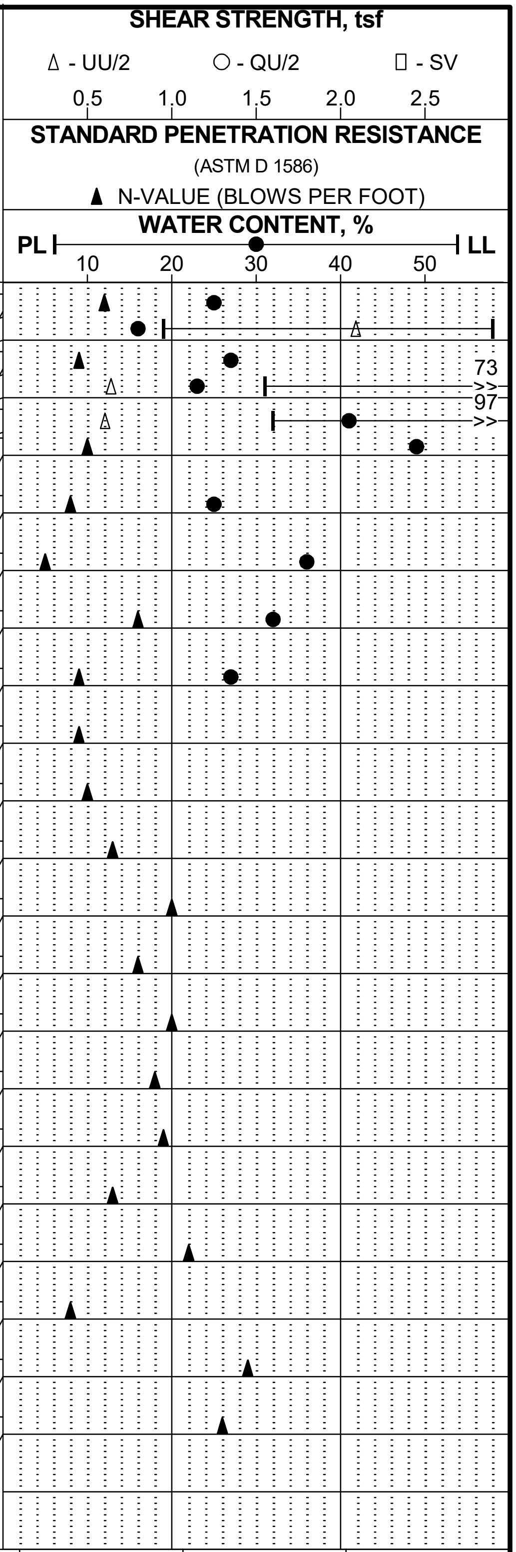
Datum **NAVD88**

Completion Date: **4/2/24**

Lat: **36.430704**

Long: **-90.622163**

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Topsoil: 2 inches			
				2-4-8	SS1
5	287	Stiff, brown and gray, FAT CLAY - (CH) little sand, trace gravel		95	ST2
				3-4-5	SS3
10	282	Stiff, gray, sandy, FAT CLAY - (CH) 71.8% passing No. 200 sieve		92	ST4
				71	ST5
15	277			0-3-7	SS6
			▽		
20	272	Loose, gray SAND with clay - SP-SC		3-3-5	SS7
25	267			2-2-3	SS8
30	262	Medium dense, gray and brown SAND - SP		4-5-11	SS9
35	257			4-4-5	SS10
40	252	Medium dense, gray and brown SAND, trace clay and lignite - SP-SC 5.3% passing No. 200 sieve		4-4-5	SS11
45	247	Loose to medium dense, gray and brown to gray and black SAND, trace gravel - SP		4-4-6	SS12
50	242	little gravel		6-8-5	SS13
55	237	2.5% passing No. 200 sieve some gravel		9-9-11	SS14
60	232			9-9-7	SS15
65	227			8-10-10	SS16
70	222	little gravel		8-8-10	SS17
75	217	some gravel		9-9-10	SS18
80	212	2.4% passing No. 200 sieve trace gravel		8-7-6	SS19
85	207			8-9-13	SS20
90	202	1.6% passing No. 200 sieve trace gravel		5-5-3	SS21
95	197	trace lignite		2-18-11	SS22
100	192	little gravel 2-inch silty lean clay seam		9-12-14	SS23
105	187	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

ENCOUNTERED AT 18.5 FEET ▽

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 20 FEET

JCG DRILLER BJH LOGGER

CME 550X DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 80 %

REMARKS: Boring elevation and coordinates based on survey performed at site.

Drawn by: SAS	Checked by: JDM	App'vd. by: ASE
Date: 4/8/24	Date: 5/14/24	Date: 5/14/24

UES™

Corning Bypass (Future I-57) (S)
Clay County, Arkansas

LOG OF BORING: B-1

Project No. J045279.01

Surface Elevation: **291.29**

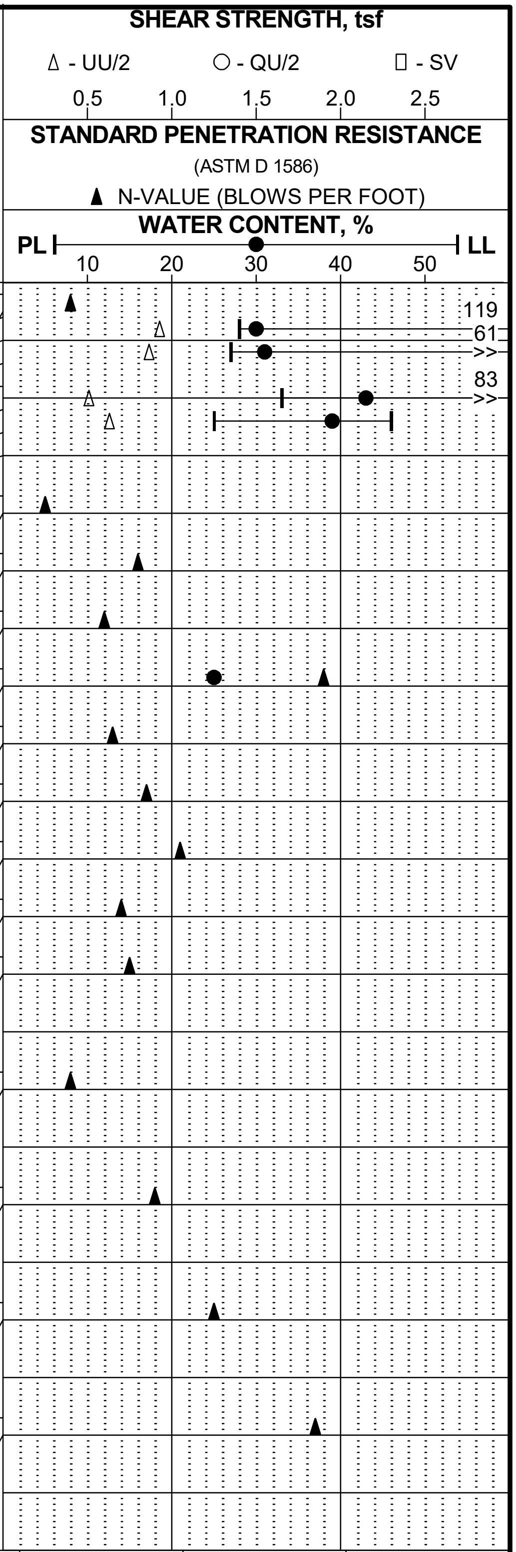
Datum **NAVD88**

Completion Date: **4/3/24**

Lat: **36.431209**

Long: **-90.622155**

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Topsoil: 2 inches			
		Medium stiff, brown and gray, FAT CLAY - (CH)		4-4-4	SS1
5	286	little roots		81	ST2
		93.9% passing No. 200 sieve		93	ST3
10	281	trace sand		76	ST4
		Brown and gray, LEAN CLAY - (CL)		87	ST5
		Brown and gray, FAT CLAY - CH			ST6
15	276	81.9% passing No. 200 sieve			
			∇		
20	271	little sand		1-2-3	SS7
		Loose to dense, gray and brown SAND, trace clay - SP			
25	266	2.5% passing No. 200 sieve		5-6-10	SS8
30	261	trace lignite		3-5-7	SS9
35	256			14-17-21	SS10
40	251	3.2% passing No. 200 sieve		4-4-9	SS11
45	246			5-8-9	SS12
50	241	little lignite		7-10-11	SS13
55	236			6-6-8	SS14
60	231	2.6% passing No. 200 sieve		7-8-7	SS15
65	226				
70	221	1.5% passing No. 200 sieve trace gravel		4-3-5	SS16
75	216				
80	211	some gravel		10-9-9	SS17
85	206				
90	201			7-13-12	SS18
95	196				
100	191	Boring terminated at 100 feet.		11-18-19	SS19
105	186				



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

GROUNDWATER DATA

ENCOUNTERED AT 18.5 FEET ∇

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 20 FEET

JCG DRILLER BJH LOGGER

CME 550X DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 80 %

REMARKS: Boring elevation and coordinates based on survey performed at site.

Drawn by: SAS	Checked by: JDM	App'vd. by: ASE
Date: 4/8/24	Date: 5/14/24	Date: 5/14/24

UES™

Corning Bypass (Future I-57) (S)
Clay County, Arkansas

LOG OF BORING: B-3

Project No. J045279.01

Surface Elevation: **291.46**

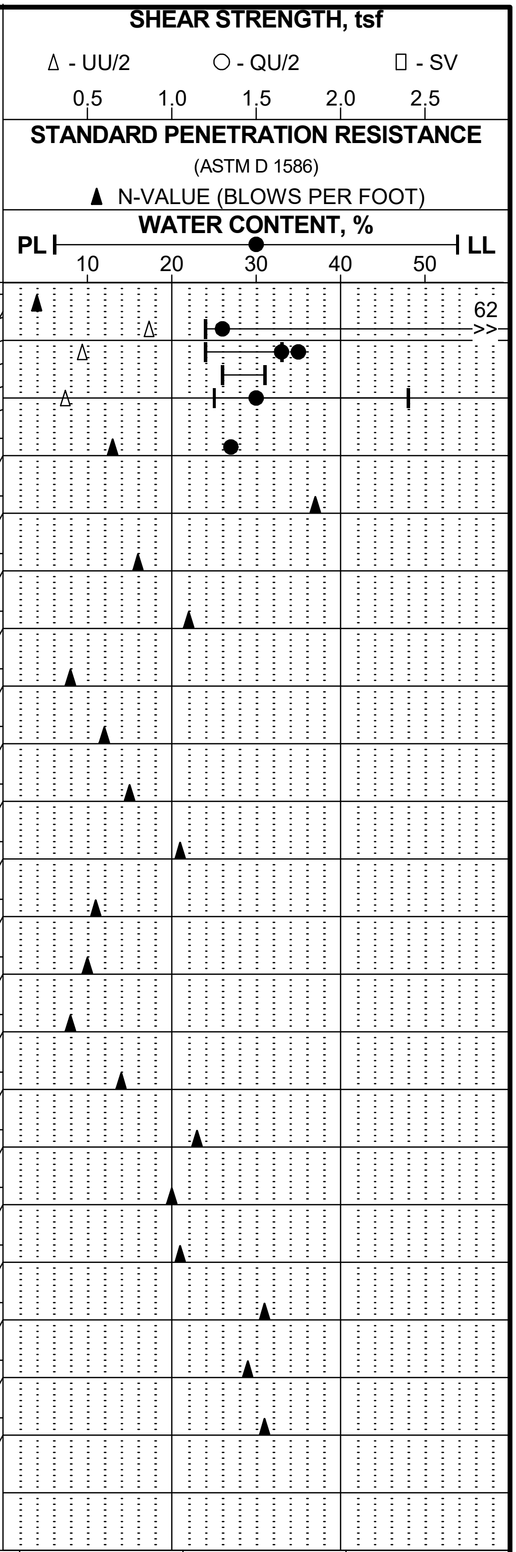
Datum **NAVD88**

Completion Date: **4/4/24**

Lat: **36.431837**

Long: **-90.622144**

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Topsoil: 2 inches	2-2-2	SS1
		Soft, brown to gray, FAT CLAY - (CH) trace sand and gravel	93	ST2
5	286		84	ST3
		Brown and gray, clayey SILT - (ML) 73% passing No. 200 sieve some sand		ST4
10	281		108	ST5
15	276	Brown and gray, silty, LEAN CLAY - (CL)	3-6-7	SS6
		Loose to dense, tan to gray and brown SAND, trace silt and gravel - SP ▽ 5-inch tan silty, sandy lean clay layer 3.9% passing No. 200 sieve		
20	271		7-15-22	SS7
25	266		8-9-7	SS8
30	261		9-11-11	SS9
35	256	3.7% passing No. 200 sieve trace lignite	5-4-4	SS10
40	251		3-5-7	SS11
45	246		4-6-9	SS12
50	241		8-9-12	SS13
55	236		5-6-5	SS14
60	231	Loose, gray, WELL-GRADED SAND with gravel - SW 1.7% passing No. 200 sieve little gravel	5-5-5	SS15
65	226		4-4-4	SS16
		Loose to dense, gray SAND - SP		
70	221		7-7-7	SS17
75	216		9-9-14	SS18
80	211	2.5% passing No. 200 sieve little gravel	10-9-11	SS19
85	206	3.5% passing No. 200 sieve trace gravel	9-10-11	SS20
90	201		12-14-17	SS21
95	196	trace lignite	10-12-17	SS22
100	191	Boring terminated at 100 feet.	10-16-15	SS23
105	186			



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

GROUNDWATER DATA

ENCOUNTERED AT 18.5 FEET ▽

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 20 FEET
JCG DRILLER BJH LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 80 %

REMARKS: Boring elevation and coordinates based on survey performed at site.

Drawn by: SAS	Checked by: JDM	App'vd. by: ASE
Date: 4/8/24	Date: 5/14/24	Date: 5/14/24

Corning Bypass (Future I-57) (S)
Clay County, Arkansas

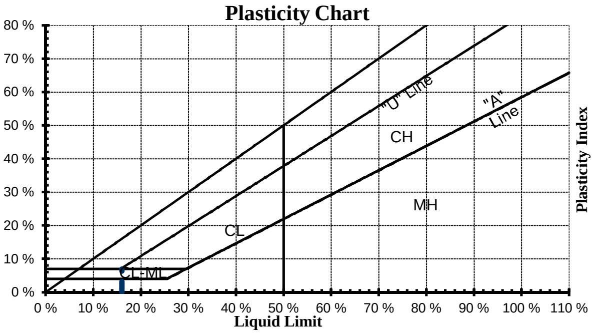
LOG OF BORING: B-5

Project No. J045279.01

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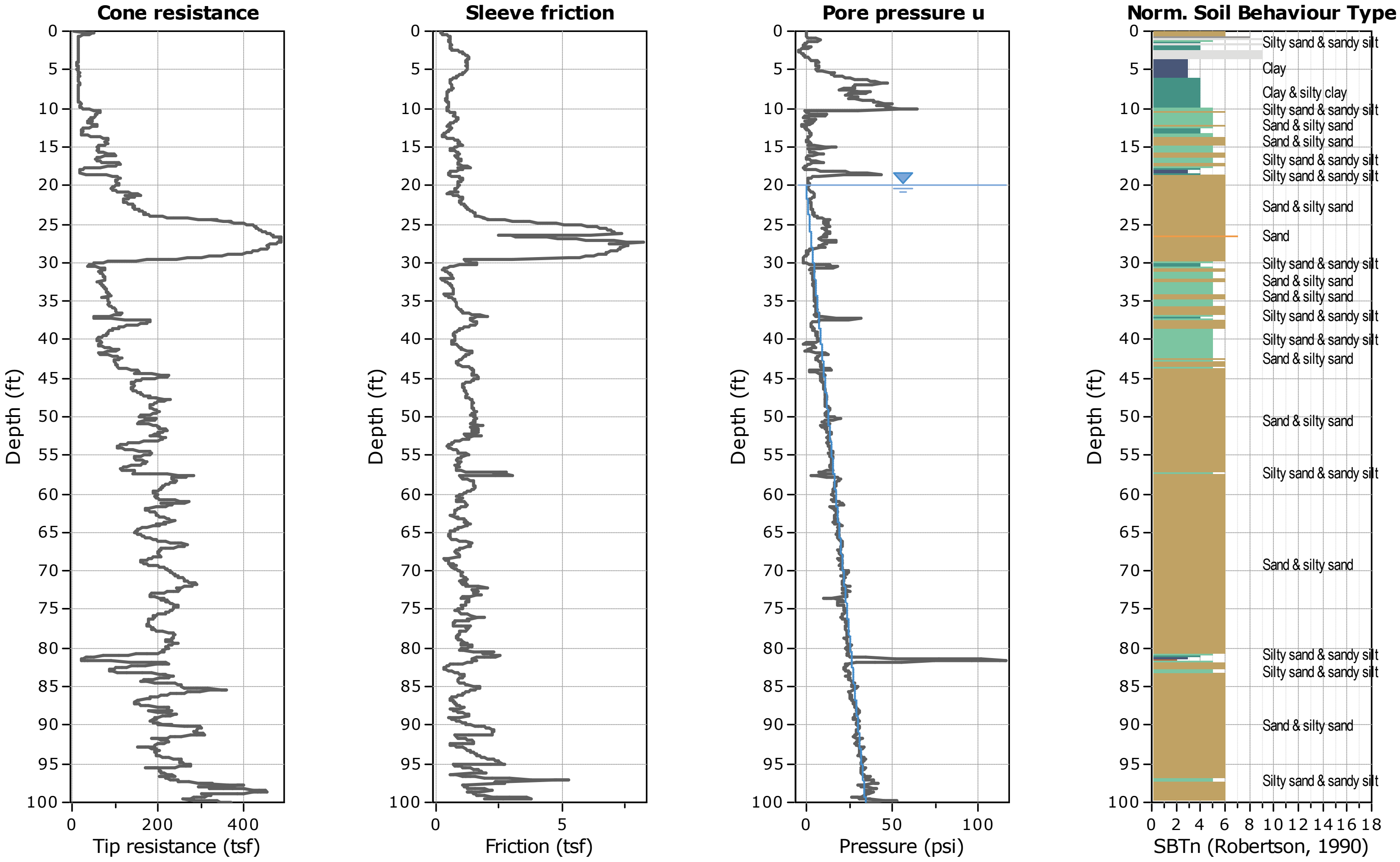
Relative composition and Unified Soil Classification System (USCS) designations are based on visual descriptions and are approximate only. If laboratory tests were performed to classify the soil, the USCS designation is shown in parenthesis.

APPENDIX D – CPT SOUNDING PLOTS

Geotechnology, LLC
3312 Winbrook Drive
Memphis, Tennessee
geotechnology.com

CPT: CPT-2
Total depth: 100.02 ft, Date: 4/10/2024
Coords: lat 36.430936° lon -90.622156°
Cone Type: 15-Square-Centimeter
Cone Operator: DWJ

Project: **ARDOT 101172 Corning Bypass (S)**
Location: **Clay County, Arkansas**



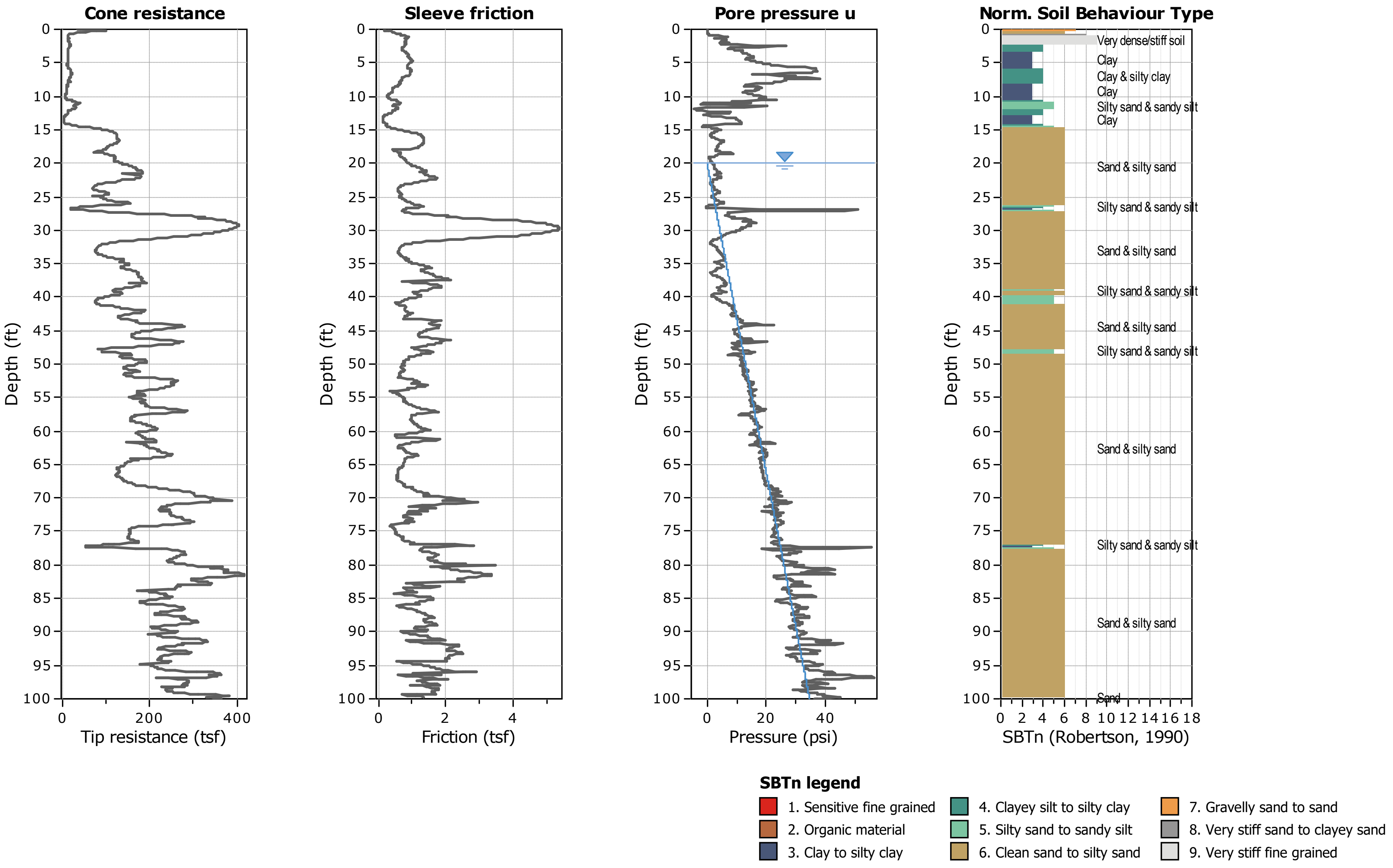
SBTn legend

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

Geotechnology, LLC
3312 Winbrook Drive
Memphis, Tennessee
geotechnology.com

CPT: CPT-4
Total depth: 100.07 ft, Date: 4/10/2024
Coords: lat 36.431543° lon -90.622144°
Cone Type: 15-Square-Centimeter
Cone Operator: DWJ

Project: **ARDOT 101172 Corning Bypass (S)**
Location: **Clay County, Arkansas**



APPENDIX E – LABORATORY TEST DATA

Atterberg Limits

Grain Size Distributions

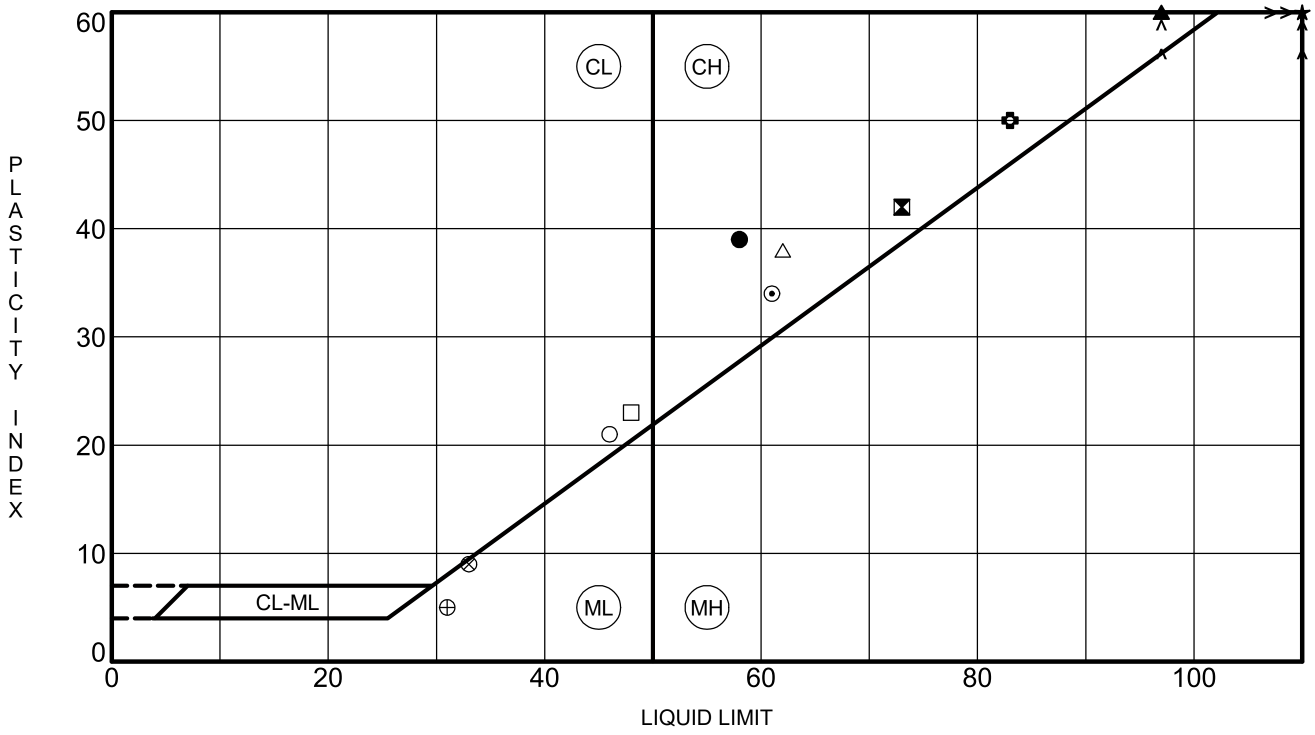
Unconsolidated-Undrained Triaxial Compression

One-Dimensional Consolidation

Direct Shear

Resistivity

pH



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**3.0**	**58**	**19**	**39**		**FAT CLAY(CH)**
⧗	**B-1**	**8.0**	**73**	**31**	**42**	**72**	**FAT CLAY with SAND(CH)**
▲	**B-1**	**11.0**	**97**	**32**	**65**		**FAT CLAY(CH)**
★	**B-3**	**3.0**	**119**	**28**	**91**		**FAT CLAY(CH)**
⊙	**B-3**	**5.0**	**61**	**27**	**34**	**94**	**FAT CLAY(CH)**
✿	**B-3**	**9.0**	**83**	**33**	**50**		**FAT CLAY(CH)**
○	**B-3**	**11.0**	**46**	**25**	**21**		**LEAN CLAY(CH)**
△	**B-5**	**3.0**	**62**	**24**	**38**		**FAT CLAY(CH)**
⊗	**B-5**	**5.0**	**33**	**24**	**9**		**SILT(ML)**
⊕	**B-5**	**7.0**	**31**	**26**	**5**	**73**	**SILT(ML)**
□	**B-5**	**9.0**	**48**	**25**	**23**		**LEAN CLAY(CL)**

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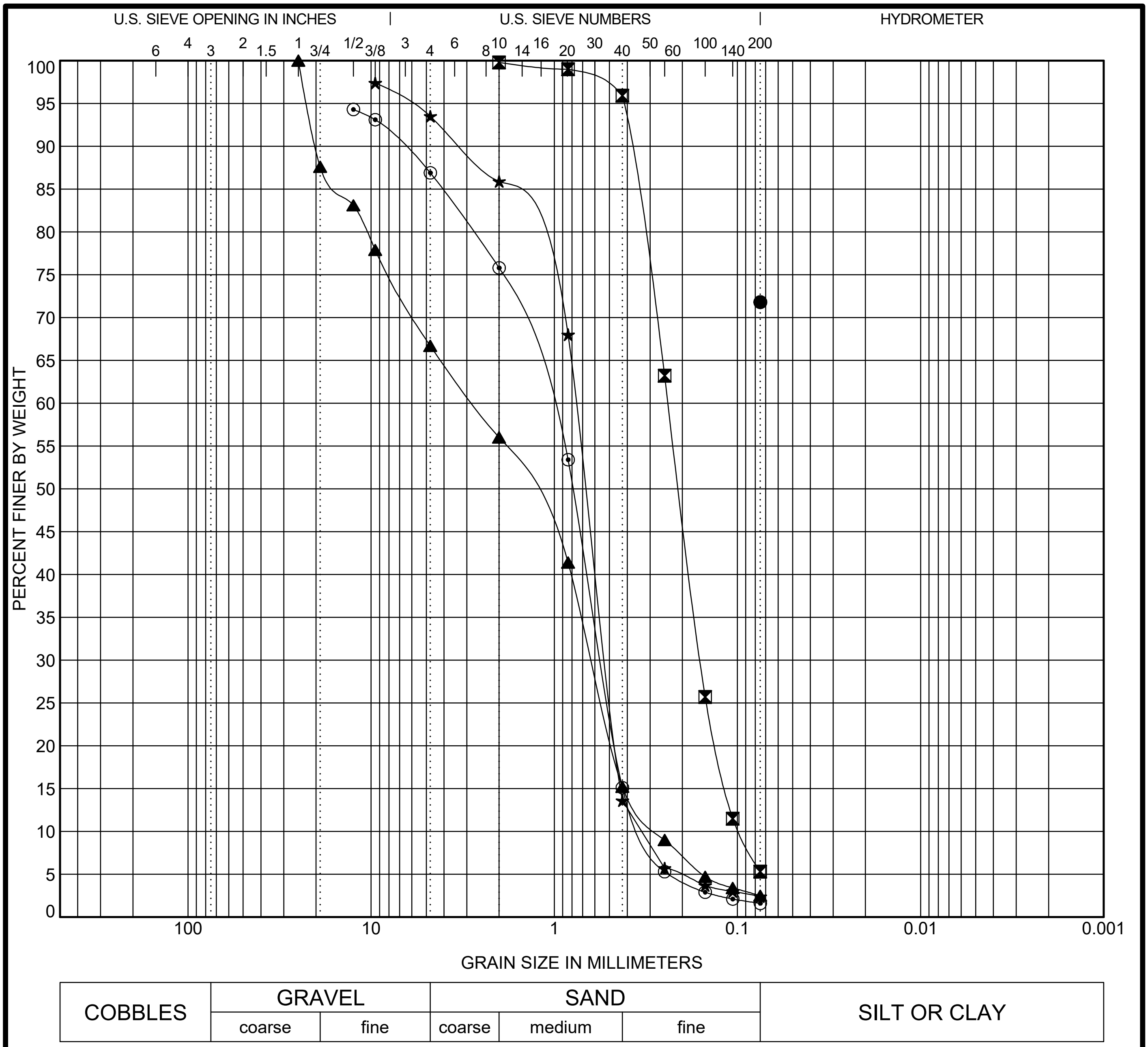
ATTERBERG LIMITS RESULTS

Corning Bypass (Future I-57) (S)
Clay County, Arkansas
J045279.01

US_ATTERBERG_LIMITS J045279.01.GPJ US LAB.GDT 5/15/24



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	B-1	8.0	**FAT CLAY with SAND(CH)**	73	31	42		
⊠	B-1	38.5	**POORLY GRADED SAND with CLAY(SP-SC)**				1.08	2.46
▲	B-1	53.5	**POORLY GRADED SAND with GRAVEL(SP)**				0.52	10.15
★	B-1	78.5	**POORLY GRADED SAND(SP)**				1.07	2.28
⊙	B-1	88.5	**POORLY GRADED SAND(SP)**				0.88	3.36

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	8.0	0.075				0.0	0.0	71.8	
⊠	B-1	38.5	2	0.239	0.159	0.097	0.0	94.5	5.3	
▲	B-1	53.5	25	2.764	0.625	0.272	33.3	64.2	2.5	
★	B-1	78.5	9.5	0.76	0.522	0.334	3.9	91.1	2.4	
⊙	B-1	88.5	12.5	1.085	0.554	0.322	7.4	85.3	1.6	

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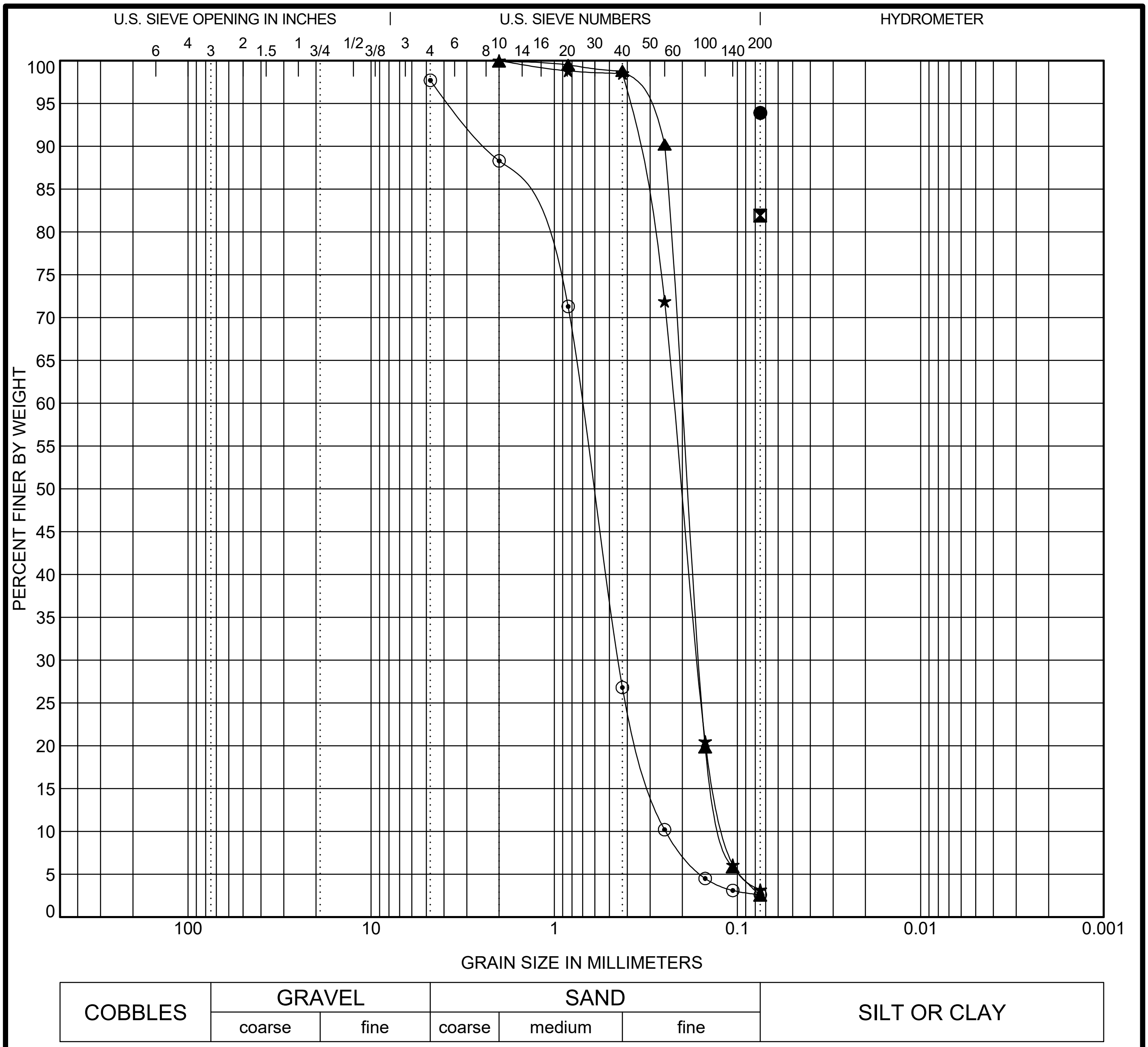
GRAIN SIZE DISTRIBUTION

Corning Bypass (Future I-57) (S)
Clay County, Arkansas
J045279.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	B-3	5.0	**FAT CLAY(CH)**	**61**	**27**	**34**		
⊠	B-3	13.0	**FAT CLAY(CH)**					
▲	B-3	18.5	**POORLY GRADED SAND(SP)**				**1.10**	**1.71**
★	B-3	38.5	**POORLY GRADED SAND(SP)**				**1.05**	**1.91**
⊙	B-3	58.5	**POORLY GRADED SAND(SP)**				**1.15**	**2.88**

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-3	5.0	**0.075**				**0.0**	**0.0**	**93.9**	
⊠	B-3	13.0	**0.075**				**0.0**	**0.0**	**81.9**	
▲	B-3	18.5	**2**	**0.201**	**0.162**	**0.118**	**0.0**	**97.4**	**2.5**	
★	B-3	38.5	**2**	**0.222**	**0.165**	**0.116**	**0.0**	**96.8**	**3.2**	
⊙	B-3	58.5	**4.75**	**0.707**	**0.446**	**0.246**	**0.0**	**95.1**	**2.6**	

GEOTECHNOLOGY
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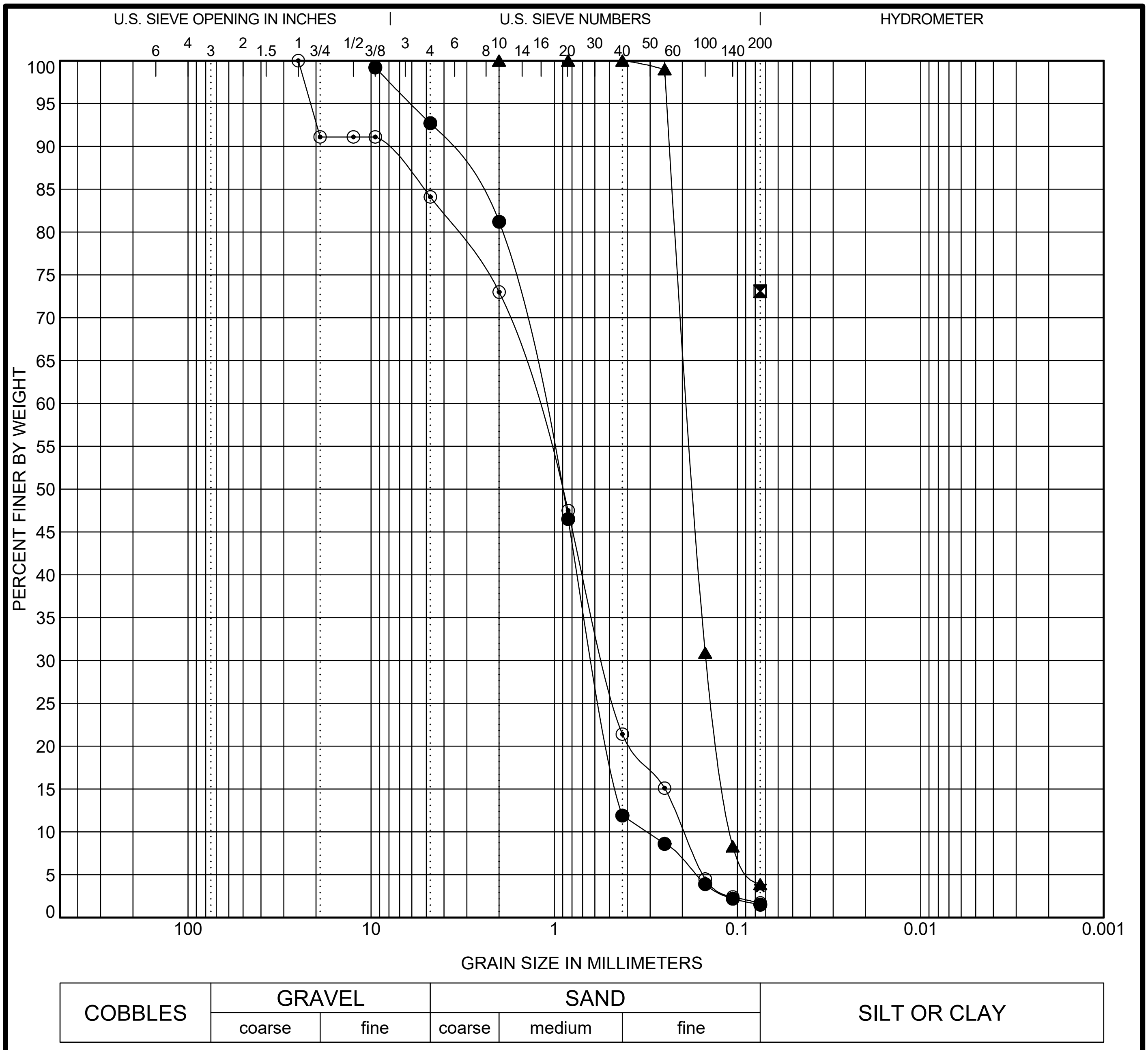
GRAIN SIZE DISTRIBUTION

Corning Bypass (Future I-57) (S)
Clay County, Arkansas
J045279.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**B-3**	**68.5**	**POORLY GRADED SAND(SP)**				**1.00**	**3.76**
⧗	**B-5**	**7.0**	**SILT(ML)**	**31**	**26**	**5**		
▲	**B-5**	**18.5**	**POORLY GRADED SAND(SP)**				**1.08**	**1.71**
★	**B-5**	**33.5**	**POORLY GRADED SAND(SP)**					
⊙	**B-5**	**58.5**	**WELL-GRADED SAND with GRAVEL(SW)**				**1.13**	**6.57**

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-3**	**68.5**	**9.5**	**1.177**	**0.607**	**0.313**	**6.5**	**91.2**	**1.5**	
⧗	**B-5**	**7.0**	**0.075**				**0.0**	**0.0**	**73.1**	
▲	**B-5**	**18.5**	**2**	**0.187**	**0.148**	**0.109**	**0.0**	**96.1**	**3.9**	
★	**B-5**	**33.5**	**0.075**				**0.0**	**0.0**	**3.7**	
⊙	**B-5**	**58.5**	**25**	**1.285**	**0.532**	**0.196**	**15.9**	**82.4**	**1.7**	

GEOTECHNOLOGY
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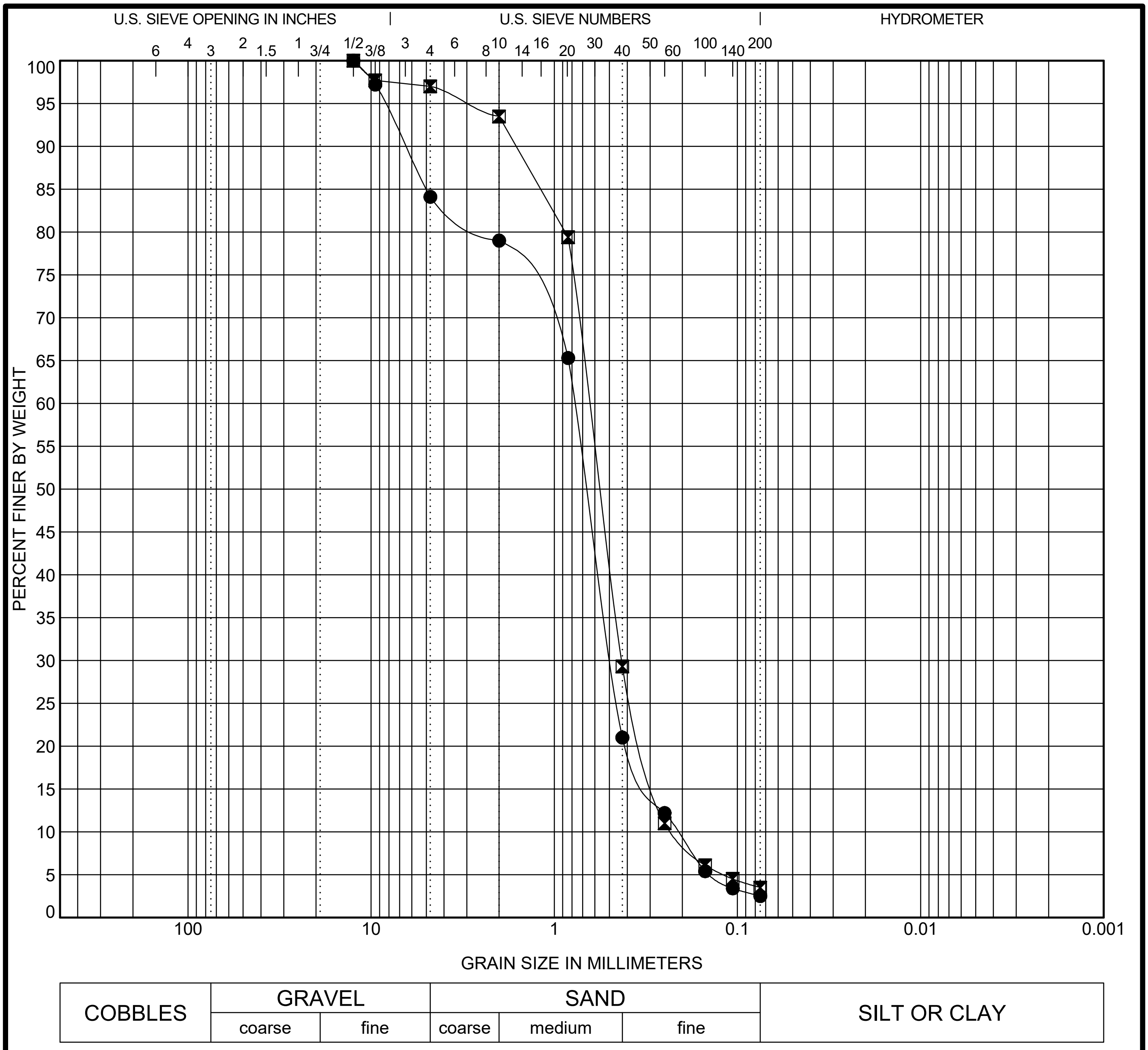
GRAIN SIZE DISTRIBUTION

Corning Bypass (Future I-57) (S)
Clay County, Arkansas
J045279.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**B-5**	**78.5**	**POORLY GRADED SAND with GRAVEL(SP)**				**1.45**	**3.65**
☒	**B-5**	**83.5**	**POORLY GRADED SAND(SP)**				**1.27**	**2.86**

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-5**	**78.5**	**12.5**	**0.774**	**0.488**	**0.212**	**15.9**	**81.6**	**2.5**	
☒	**B-5**	**83.5**	**12.5**	**0.645**	**0.429**	**0.225**	**3.0**	**93.5**	**3.5**	

GEOTECHNOLOGY

A UES Company

GRAIN SIZE DISTRIBUTION

Corning Bypass (Future I-57) (S)
Clay County, Arkansas
J045279.01

US GRAIN SIZE J045279.01.GPJ US LAB.GDT 5/15/24

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department
DATE: 4/19/2024
PROJECT NO.: J045279.01
PROJECT: ARDOT 2023-2027 Contract TO G017 Job 101172 Corning Bypass (Future I-57), Clay County
LOCATION: Corning, AR

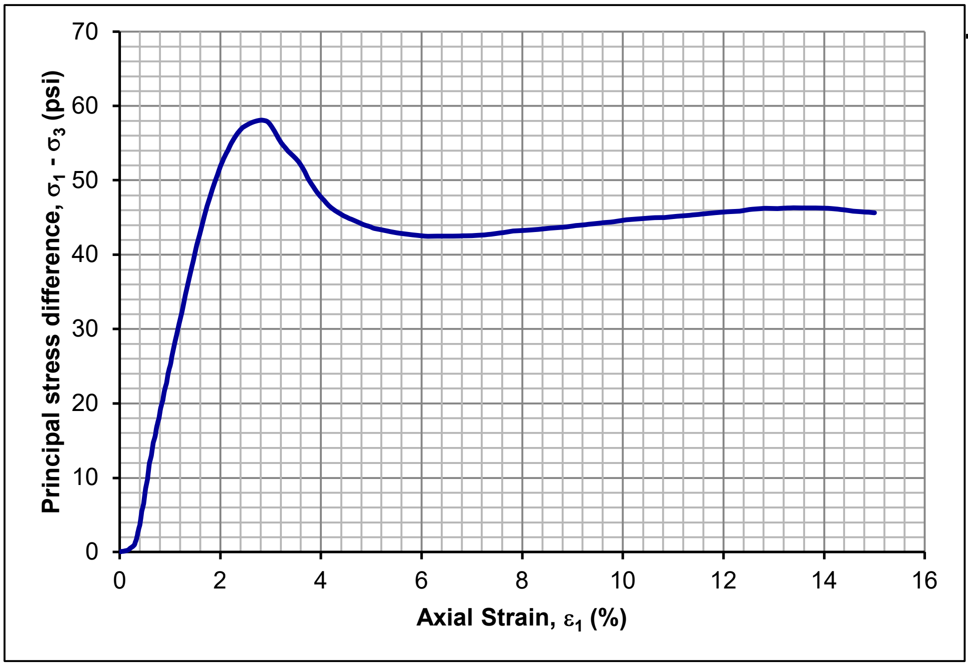
BORING NO.: B1A
SAMPLE NO.: ST2
DEPTH (ft.): 3.0-5.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Dark Brown Fat Clay

LIQUID LIMIT (%): 58
PLASTIC LIMIT (%): 19
PLASTICITY INDEX (%): 39
USCS: CH

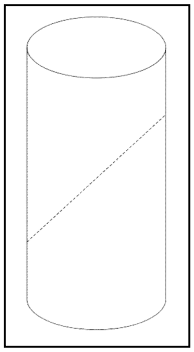
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.89
HEIGHT (in.):	5.76
HEIGHT TO DIAMETER RATIO:	1.99
WET UNIT WEIGHT (pcf):	113.8
DRY UNIT WEIGHT (pcf):	94.7
VOID RATIO:	0.81
MOISTURE CONTENT (%)*:	20.1
DEGREE OF SATURATION (%):	68.2

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**16.2**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**2.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**58.1**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**2.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**60.4**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**8,370**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**4,185**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**N/A**



FAILURE SHAPES



REMARKS :

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* 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: 4/17/2024
PROJECT NO.: J045279.01
PROJECT: ARDOT 2023-2027 Contract TO G017 Job 101172 Corning Bypass (Future I-57), Clay County
LOCATION: Corning, AR

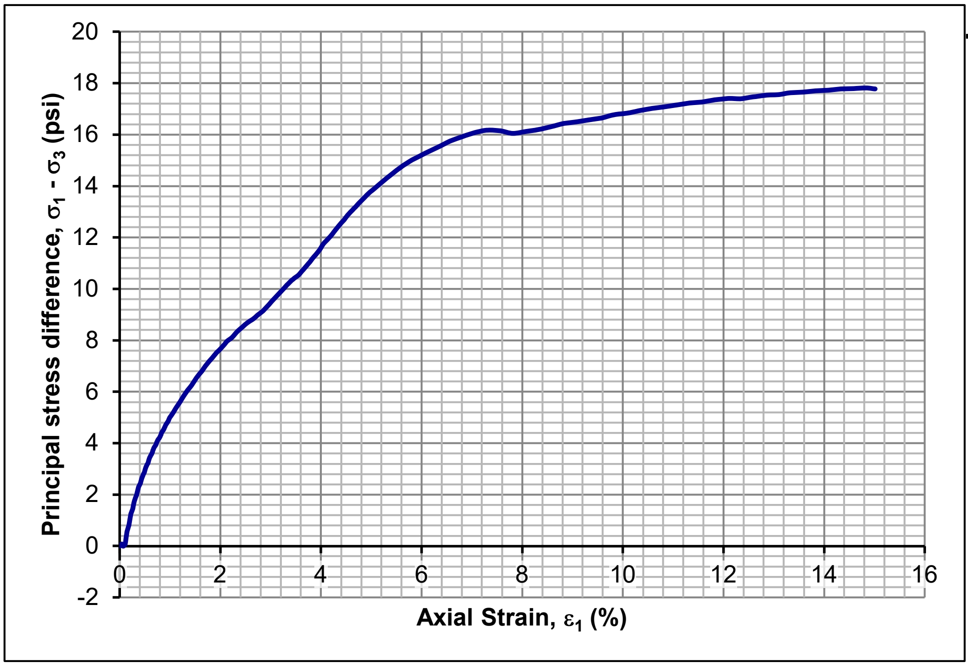
BORING NO.: B1 SAMPLE NO.: ST-4 DEPTH (ft.): 8.0-10.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Gray Fat Clay with Sand

LIQUID LIMIT (%): 73 PLASTIC LIMIT (%): 31 PLASTICITY INDEX (%): 42 USCS: CH

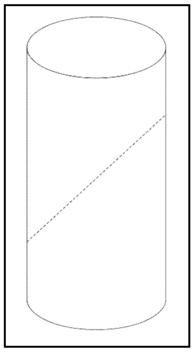
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.85
HEIGHT (in.):	5.86
HEIGHT TO DIAMETER RATIO:	2.06
WET UNIT WEIGHT (pcf):	120.3
DRY UNIT WEIGHT (pcf):	92.3
VOID RATIO:	0.86
MOISTURE CONTENT (%)*:	30.4
DEGREE OF SATURATION (%):	97.1

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**23.1**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**17.8**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**5.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**23.1**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,560**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,280**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**2,420**



FAILURE SHAPES



REMARKS :

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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department
DATE: 4/18/2024
PROJECT NO.: J045279.01
PROJECT: ARDOT 2023-2027 Contract TO G017 Job 101172 Corning Bypass (Future I-57), Clay County
LOCATION: Corning, AR

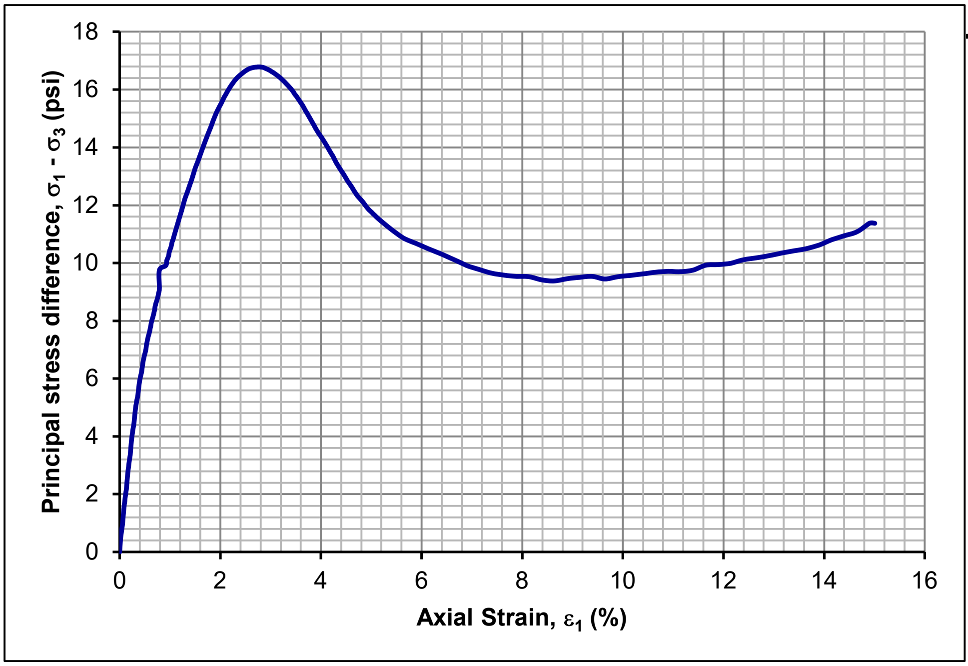
BORING NO.: B1
SAMPLE NO.: ST5
DEPTH (ft.): 11.0-13.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Brown Fat Clay

LIQUID LIMIT (%): 97
PLASTIC LIMIT (%): 32
PLASTICITY INDEX (%): 65
USCS: CH

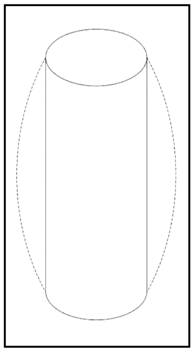
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.85
HEIGHT (in.):	5.74
HEIGHT TO DIAMETER RATIO:	2.01
WET UNIT WEIGHT (pcf):	106.6
DRY UNIT WEIGHT (pcf):	71.0
VOID RATIO:	1.42
MOISTURE CONTENT (%)*:	50.1
DEGREE OF SATURATION (%):	97.3

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**41.3**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**2.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**16.8**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**7.0**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**23.8**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,420**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,210**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**N/A**



FAILURE SHAPES



REMARKS :

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** Final moisture content determined from entire sample.
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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department DATE: 4/17/2024
PROJECT NO.: J045279.01
PROJECT: ARDOT 2023-2027 Contract TO G017 Job 101172 Corning Bypass (Future I-57), Clay County
LOCATION: Corning, AR

BORING NO.: B3 SAMPLE NO.: ST2 DEPTH (ft.): 3.0-5.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Brown Fat Clay

LIQUID LIMIT (%): 119 PLASTIC LIMIT (%): 28 PLASTICITY INDEX (%): 91 USCS: CH

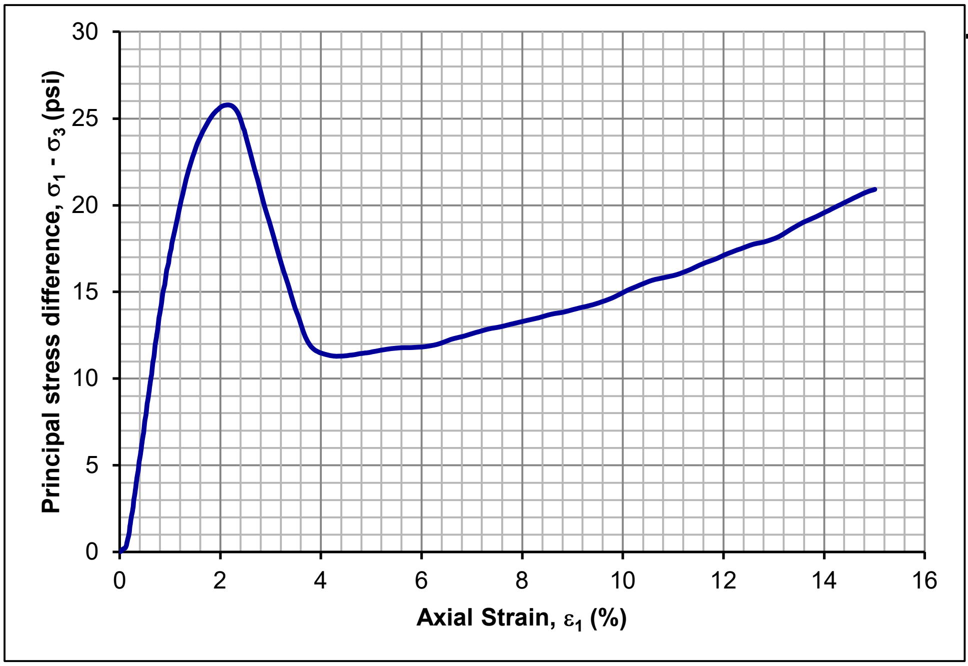
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA

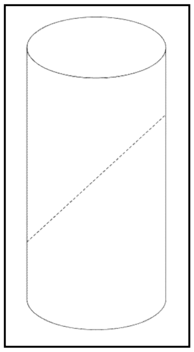
AVERAGE DIAMETER (in.):	2.85
HEIGHT (in.):	5.72
HEIGHT TO DIAMETER RATIO:	2.01
WET UNIT WEIGHT (pcf):	110.6
DRY UNIT WEIGHT (pcf):	80.9
VOID RATIO:	1.12
MOISTURE CONTENT (%)*:	36.7
DEGREE OF SATURATION (%):	90.1

FAILURE DATA***

MOISTURE CONTENT AFTER FAILURE (%)**:	**29.5**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**2.2**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**25.8**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**2.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**28.1**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**3,710**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,855**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**N/A**



FAILURE SHAPES



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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department
DATE: 4/17/2024
PROJECT NO.: J045279.01
PROJECT: ARDOT 2023-2027 Contract TO G017 Job 101172 Corning Bypass (Future I-57), Clay County
LOCATION: Corning, AR

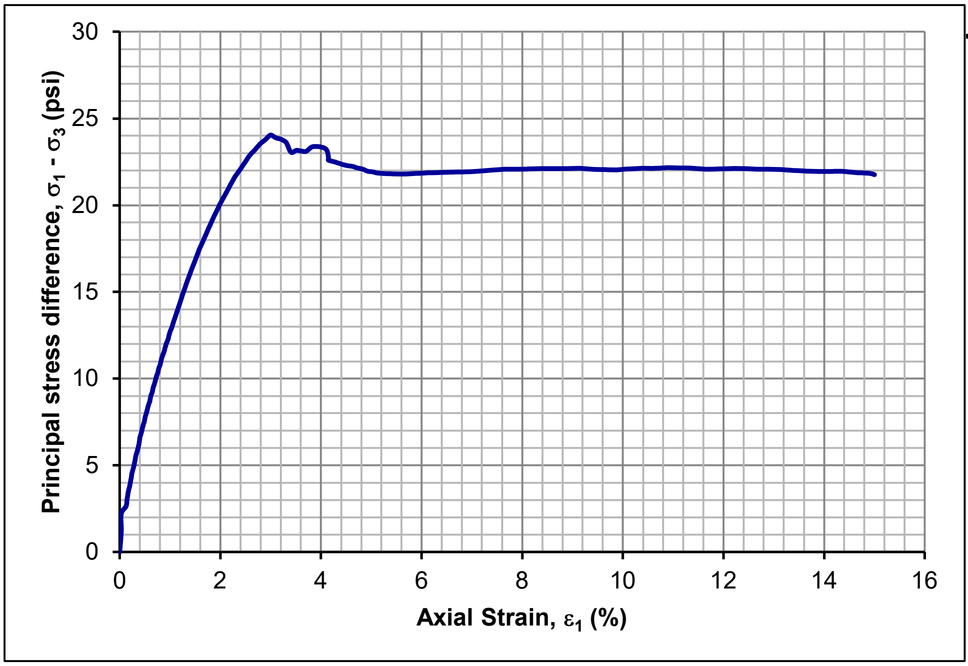
BORING NO.: B-3
SAMPLE NO.: ST-3
DEPTH (ft.): 5.0-7.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Gray Fat Clay

LIQUID LIMIT (%): 61
PLASTIC LIMIT (%): 27
PLASTICITY INDEX (%): 34
USCS: CH

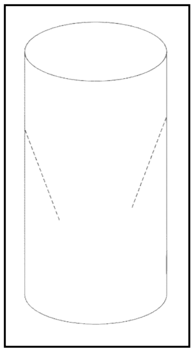
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.85
HEIGHT (in.):	5.78
HEIGHT TO DIAMETER RATIO:	2.03
WET UNIT WEIGHT (pcf):	118.1
DRY UNIT WEIGHT (pcf):	92.5
VOID RATIO:	0.86
MOISTURE CONTENT (%)*:	27.8
DEGREE OF SATURATION (%):	89.2

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**31.3**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**3.0**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**24.0**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**3.5**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**27.5**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**3,460**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,730**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**N/A**



FAILURE SHAPES



REMARKS :

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** Final moisture content determined from entire sample.
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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department DATE: 4/23/2024
PROJECT NO.: J045279.01
PROJECT: ARDOT 2023-2027 Contract TO G017 Job 101172 Corning Bypass (Future I-57), Clay County
LOCATION: Corning, AR

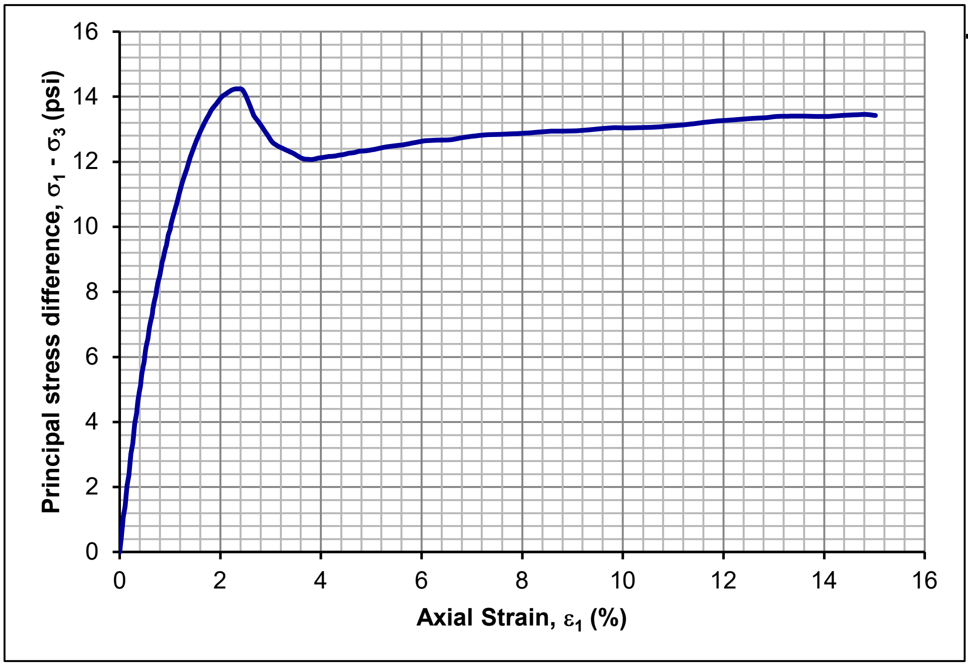
BORING NO.: B-3 SAMPLE NO.: ST-4 DEPTH (ft.): 9.0-11.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Brown Lean Clay, Trace Organics, Little Sand

LIQUID LIMIT (%): 83 PLASTIC LIMIT (%): 33 PLASTICITY INDEX (%): 50 USCS: CH

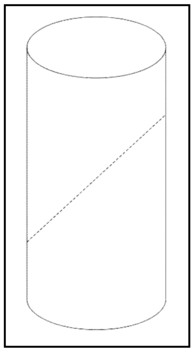
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.84
HEIGHT (in.):	5.77
HEIGHT TO DIAMETER RATIO:	2.03
WET UNIT WEIGHT (pcf):	109.6
DRY UNIT WEIGHT (pcf):	76.1
VOID RATIO:	1.25
MOISTURE CONTENT (%)*:	43.9
DEGREE OF SATURATION (%):	96.4

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**42.6**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**2.3**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**14.2**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**5.8**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**20.1**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,050**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,025**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**N/A**



FAILURE SHAPES



REMARKS :

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** Final moisture content determined from entire sample.
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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department DATE: 4/19/2024
PROJECT NO.: J045279.01
PROJECT: ARDOT 2023-2027 Contract TO G017 Job 101172 Corning Bypass (Future I-57), Clay County
LOCATION: Corning, AR

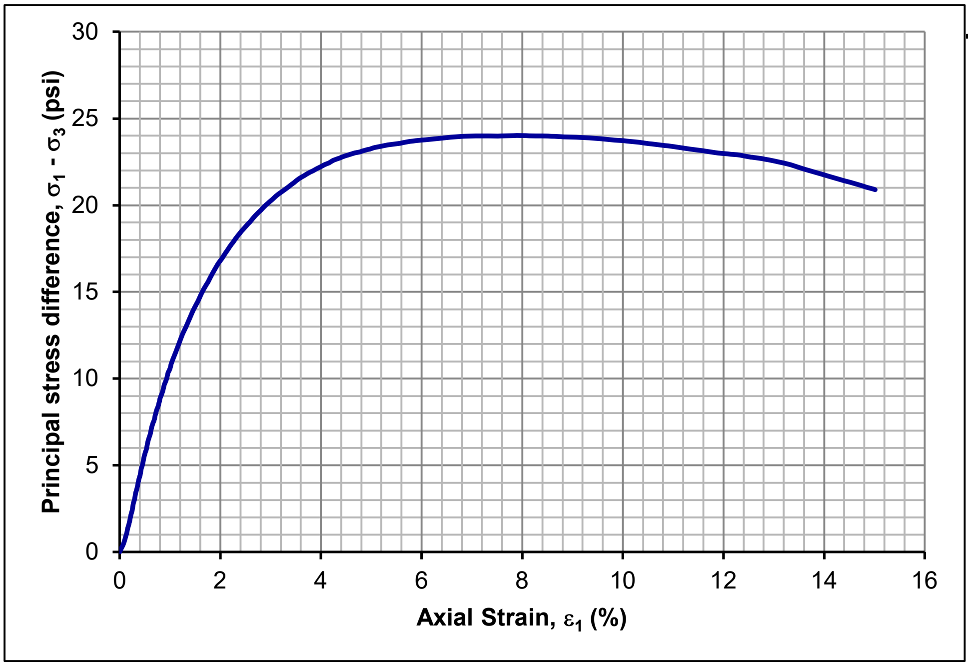
BORING NO.: B5 SAMPLE NO.: ST2 DEPTH (ft.): 3.0-5.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Dark Brown Fat Clay

LIQUID LIMIT (%): 62 PLASTIC LIMIT (%): 24 PLASTICITY INDEX (%): 38 USCS: CH

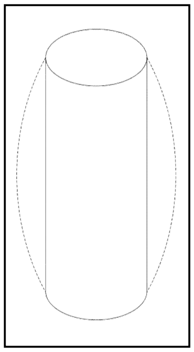
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.85
HEIGHT (in.):	5.70
HEIGHT TO DIAMETER RATIO:	2.00
WET UNIT WEIGHT (pcf):	119.9
DRY UNIT WEIGHT (pcf):	93.1
VOID RATIO:	0.84
MOISTURE CONTENT (%)*:	28.8
DEGREE OF SATURATION (%):	93.9

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**26.4**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**7.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**24.0**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**2.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**26.3**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**3,460**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,730**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**N/A**



FAILURE SHAPES



REMARKS :

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GEOTECHNOLOGY
A UES Company

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department
DATE: 4/26/2024
PROJECT NO.: J045279.01
PROJECT: ARDOT 2023-2027 Contract TO G017 Job 101172 Corning Bypass (Future I-57), Clay County
LOCATION: Corning, AR

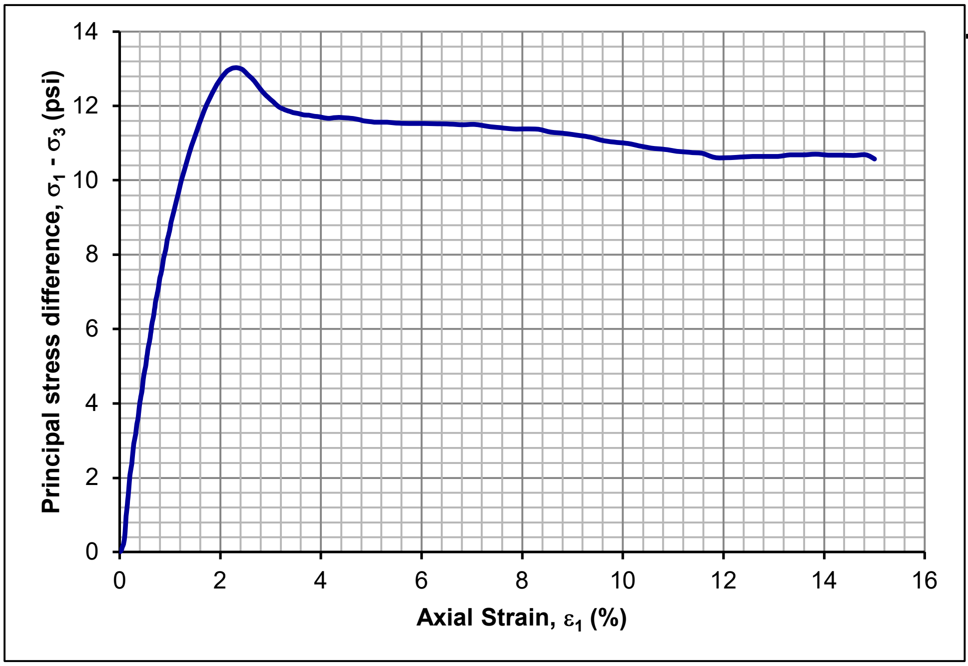
BORING NO.: B5
SAMPLE NO.: ST3
DEPTH (ft.): 5.0-7.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Gray Silt

LIQUID LIMIT (%): 33
PLASTIC LIMIT (%): 24
PLASTICITY INDEX (%): 9
USCS: ML

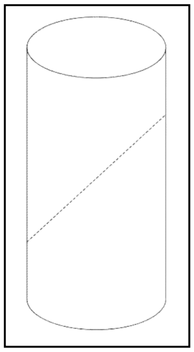
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.83
HEIGHT (in.):	5.74
HEIGHT TO DIAMETER RATIO:	2.03
WET UNIT WEIGHT (pcf):	112.8
DRY UNIT WEIGHT (pcf):	83.9
VOID RATIO:	1.05
MOISTURE CONTENT (%)*:	34.5
DEGREE OF SATURATION (%):	90.7

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**34.5**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**2.3**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**13.0**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**3.5**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**16.5**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,880**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**940**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**N/A**



FAILURE SHAPES



REMARKS :

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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department
DATE: 4/19/2024
PROJECT NO.: J045279.01
PROJECT: ARDOT 2023-2027 Contract TO G017 Job 101172 Corning Bypass (Future I-57), Clay County
LOCATION: Corning, AR

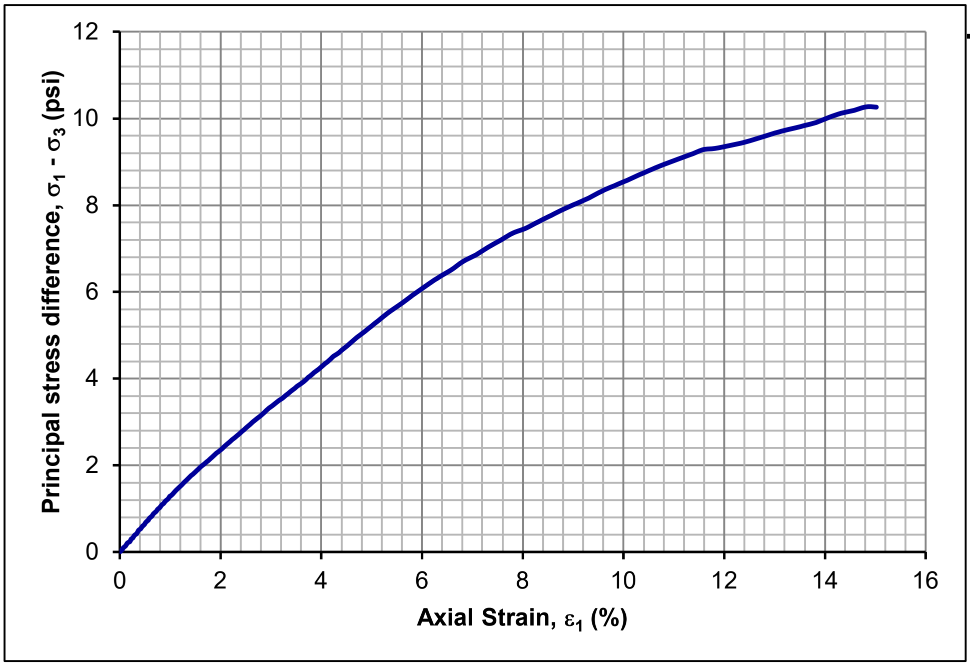
BORING NO.: B5
SAMPLE NO.: ST5
DEPTH (ft.): 9.0-11.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Gray Lean Clay

LIQUID LIMIT (%): 48
PLASTIC LIMIT (%): 25
PLASTICITY INDEX (%): 23
USCS: CL

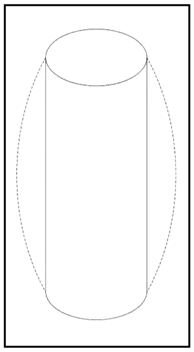
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.64
HEIGHT (in.):	5.84
HEIGHT TO DIAMETER RATIO:	2.21
WET UNIT WEIGHT (pcf):	138.9
DRY UNIT WEIGHT (pcf):	108.0
VOID RATIO:	0.59
MOISTURE CONTENT (%)*:	28.5
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**29.9**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**10.3**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**5.8**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**16.1**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,480**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**740**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**1,230**



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
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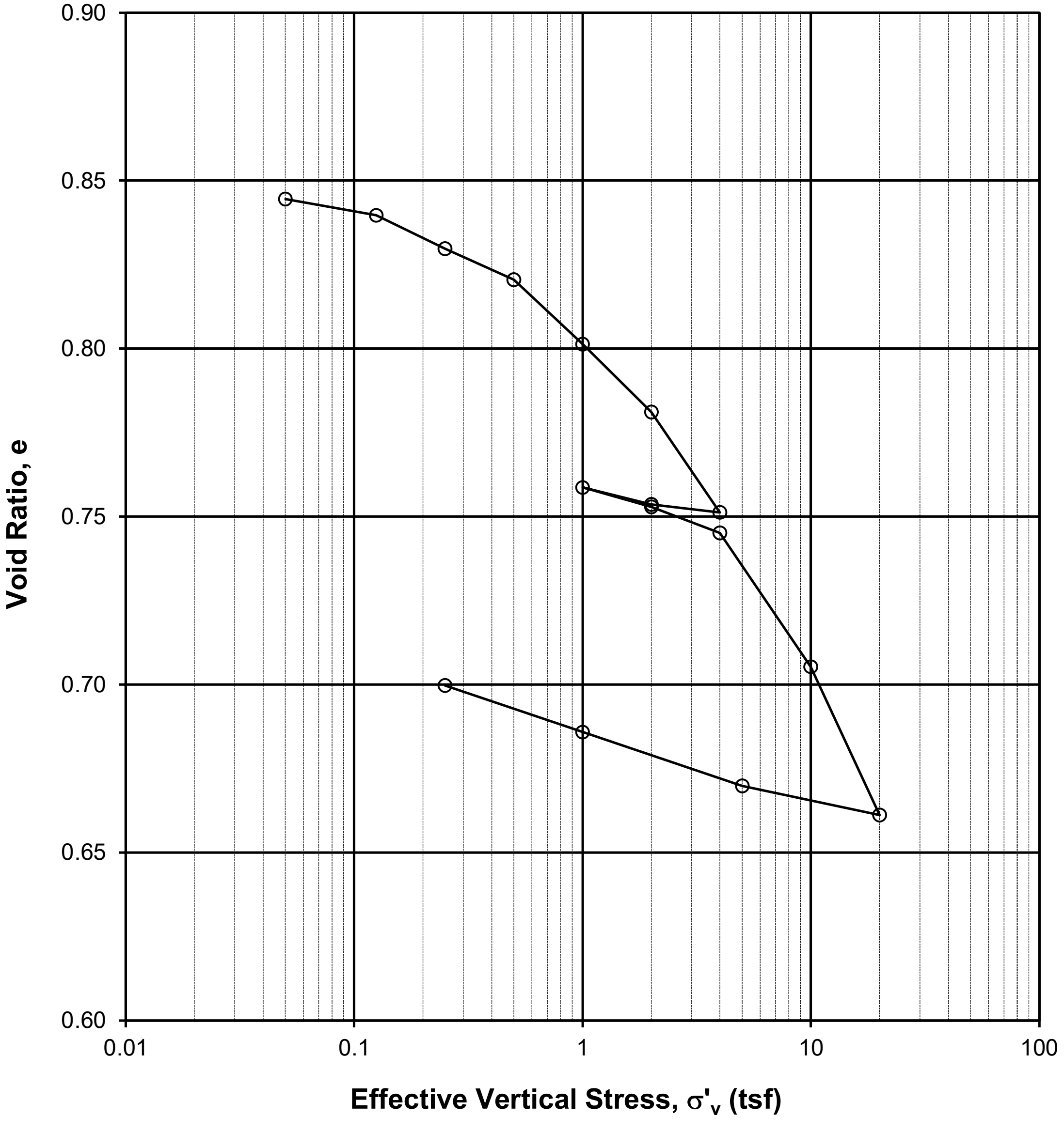
Liquid Limit= 73 Plastic Limit= 31 Plasticity Index = 42 USCS: CH

Compression Index, C_c = 0.12

Recompression Index, C_r = 0.03

Void Ratio, e_o = 0.844

Preconsolidation Pressure = 1.500 tsf



1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435
Project No.: J045279.01
Boring: B-1
Sample: ST-4 - Depth: 8

GEOTECHNOLOGY
A Universal Engineering Sciences Company

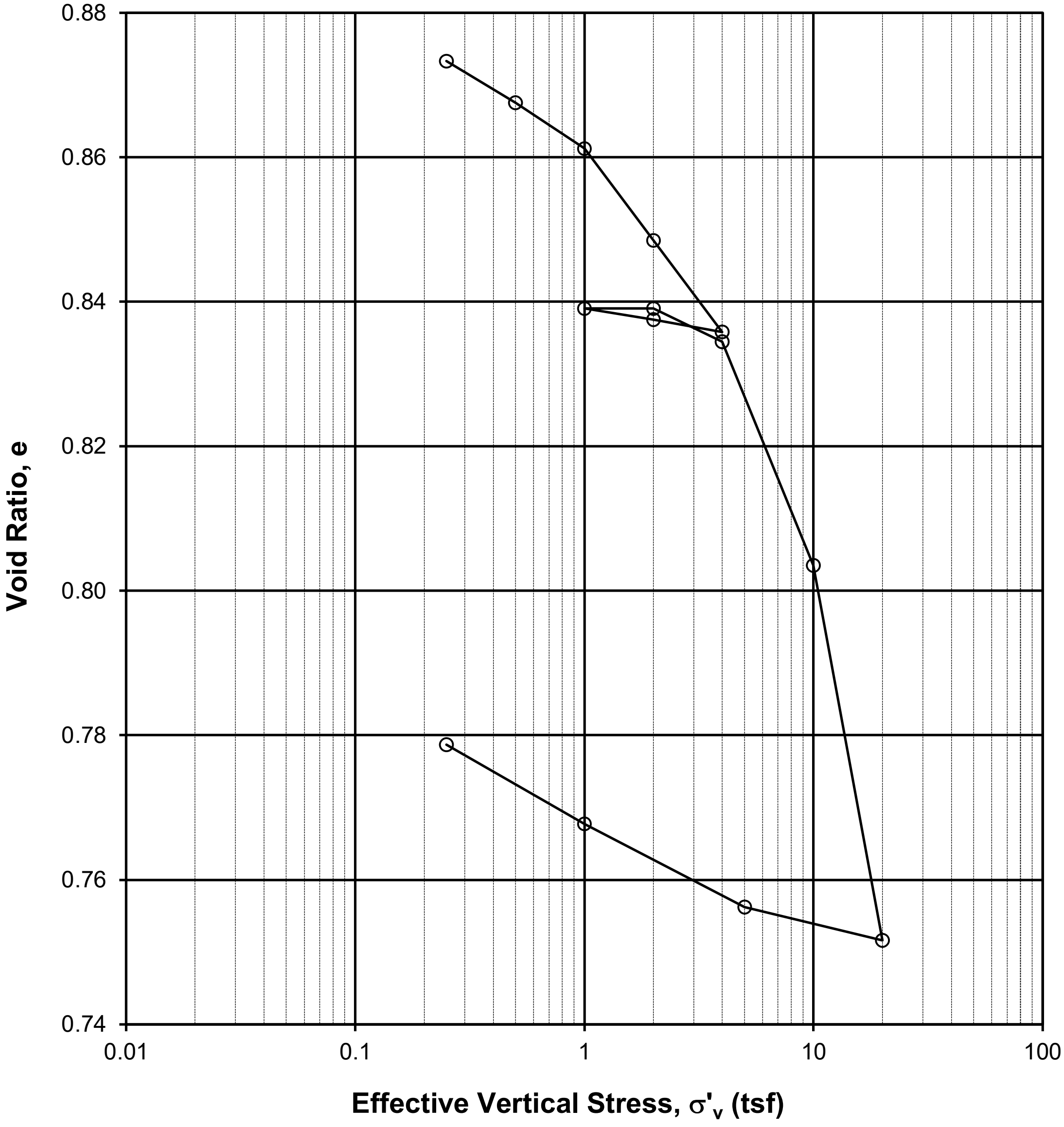
Liquid Limit= 31 Plastic Limit= 26 Plasticity Index = 5 USCS: ML

Compression Index, C_c = 0.12

Void Ratio, e_o = 0.873

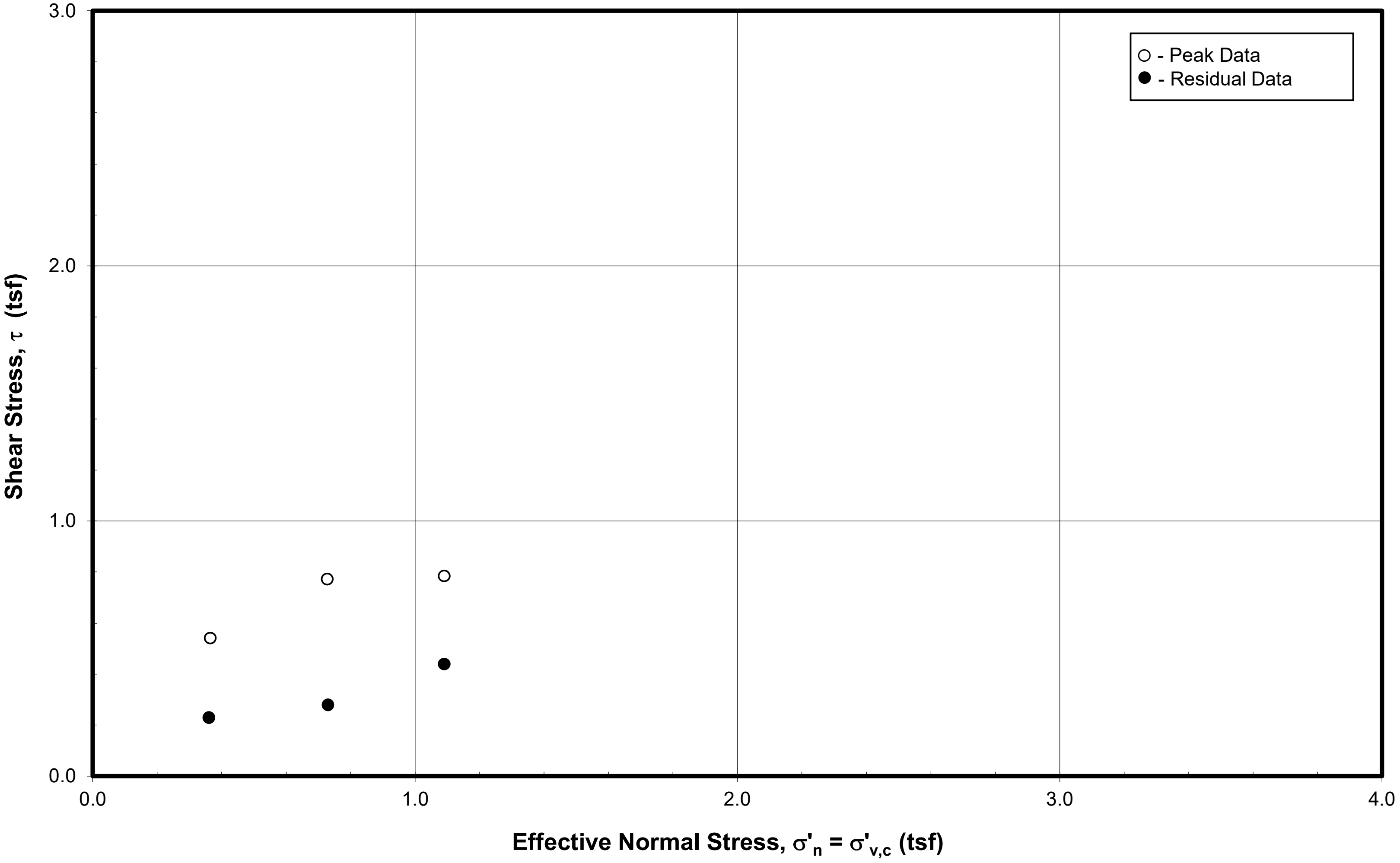
Recompression Index, C_r = 0.02

Preconsolidation Pressure = 2.500 tsf



1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435
Project No.: J045279.01
Boring: B-5
Sample: ST-4 - Depth: 7.0



DRAINED DIRECT SHEAR TEST

ASTM D 3080

Boring: B-3 Sample: ST-2 -Depth: 3ft

SOIL RESISTIVITY TEST REPORT

Project No.:	**J045279.01**	**April 17, 2024**
Project Name:	**ARDOT Corning**	**Page 1 of 1**
Boring Number:	**B-1**	
Sample ID:	**SS-7**	
Depth (ft):	**18.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	38,200	0.57	21,774.00	11.1
#2	14,500	0.57	8,265.00	18.4
#3	6,400	0.57	3,648.00	26.9
#4	6,800	0.57	3,876.00	32.4

Minimum Soil Resistivity **3,648.00**

SOIL RESISTIVITY TEST REPORT

Project No.:	**J045279.01**	**April 17, 2024**
Project Name:	**ARDOT Corning**	**Page 1 of 1**
Boring Number:	**B-1**	
Sample ID:	**SS-10**	
Depth (ft):	**33.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	35,500	0.57	20,235.00	10.6
#2	16,600	0.57	9,462.00	18.0
#3	8,500	0.57	4,845.00	27.8
#4	9,700	0.57	5,529.00	28.2

Minimum Soil Resistivity **4,845.00**

SOIL RESISTIVITY TEST REPORT

Project No.:	**J045279.01**	**April 17,2024**
Project Name:	**ARDOT Corning**	**Page 1 of 1**
Boring Number:	**B-3**	
Sample ID:	**SS-9**	
Depth (ft):	**28.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	50,900	0.57	29,013.00	11.2
#2	22,100	0.57	12,597.00	18.2
#3	11,900	0.57	6,783.00	26.9
#4	11,500	0.57	6,555.00	31.2
#5	13,700	0.57	7,809.00	32.4

Minimum Soil Resistivity **6,555.00**

SOIL RESISTIVITY TEST REPORT

Project No.:	**J045279.01**	**April 17,2024**
Project Name:	**ARDOT Corning**	**Page 1 of 1**
Boring Number:	**B-3**	
Sample ID:	**SS12**	
Depth (ft):	**43.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	44,200	0.57	25,194.00	21.8
#2	20,900	0.57	11,913.00	18.1
#3	14,300	0.57	8,151.00	20.6
#4	15,300	0.57	8,721.00	25.7

Minimum Soil Resistivity **8,151.00**

SOIL RESISTIVITY TEST REPORT

Project No.:	**J045279.01**	**April 17, 2024**
Project Name:	**ARDOT Corning**	**Page 1 of 1**
Boring Number:	**B-5**	
Sample ID:	**SS-8**	
Depth (ft):	**23.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	34,800	0.57	19,836.00	11.0
#2	17,200	0.57	9,804.00	17.1
#3	10,400	0.57	5,928.00	24.9
#4	9,300	0.57	5,301.00	25.8
#5	11,000	0.57	6,270.00	25.0

Minimum Soil Resistivity **5,301.00**

SOIL RESISTIVITY TEST REPORT

Project No.:	**J045279.01**	**April 17, 2024**
Project Name:	**ARDOT Corning**	**Page 1 of 1**
Boring Number:	**B-5**	
Sample ID:	**SS-13**	
Depth (ft):	**48.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	37,600	0.57	21,432.00	10.6
#2	18,400	0.57	10,488.00	17.4
#3	14,600	0.57	8,322.00	22.8
#4	19,500	0.57	11,115.00	20.8

Minimum Soil Resistivity **8,322.00**

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

GEOTECHNOLOGY
A UES Company

DATE	PROJECT NAME	PROJECT NO.
4/17/2024	ARDOT 101172 Corning Bypass	J045279.01

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	SS7	18.50		1:1	8.60 ------------- 22.4			

B-1	SS10	33.50		1:1	8.40 ------------- 23.5			

B-3	SS9	28.50		1:1	7.84 ------------- 21.3			

B-3	SS12	43.50		1:1	8.34 ------------- 21.6			

B-5	SS13	48.50		1:1	8.89 ------------- 21.9			

B-5	SS8	23.50		1:1	8.784 ------------- 21.4			

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

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

Tested By: MS
Date: 04/17/24

Calculated By: HP
Date: 05/15/24

Checked By: JDM
Date: 05/15/24

APPENDIX F – SITE-SPECIFIC SEISMIC STUDY

Site-Specific Seismic Study
Corning Bypass (Future I-57)
Clay County, Arkansas

By

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

March 12, 2024

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Site-Specific Seismic Study
Corning Bypass (Future I-57)
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 36.437361°N and 90.604030°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 the 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 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 (A UES Company) 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. A copy of the boring used for this study is provided in Appendix B.

Table 1. Shear-Wave Velocities Measured.

Depth1 (ft)	**Depth2 (ft)**	**V_s (ft/sec)**
1.67	4.95	513.48
4.95	8.23	513.48
8.23	11.55	513.48
11.55	14.83	916.56
14.83	18.14	1025.36
18.14	21.39	500.13
21.39	24.60	574.00
24.60	27.85	656.20
27.85	31.09	842.93
31.09	34.37	658.98
34.37	37.82	784.35
37.82	41.10	808.88
41.10	44.31	742.30
44.31	47.66	657.15
47.66	50.94	572.43
50.94	54.19	782.08
54.19	57.40	774.87
57.40	60.84	873.20
60.84	64.09	727.34
64.09	67.37	792.19
67.37	70.62	777.06
70.62	73.93	1161.71
73.93	77.15	879.76
77.15	80.36	842.96
80.36	83.71	969.47
83.71	86.95	930.37
86.95	90.20	899.54
90.20	93.55	1083.48
93.55	96.76	921.48
96.76	100.14	984.00

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.

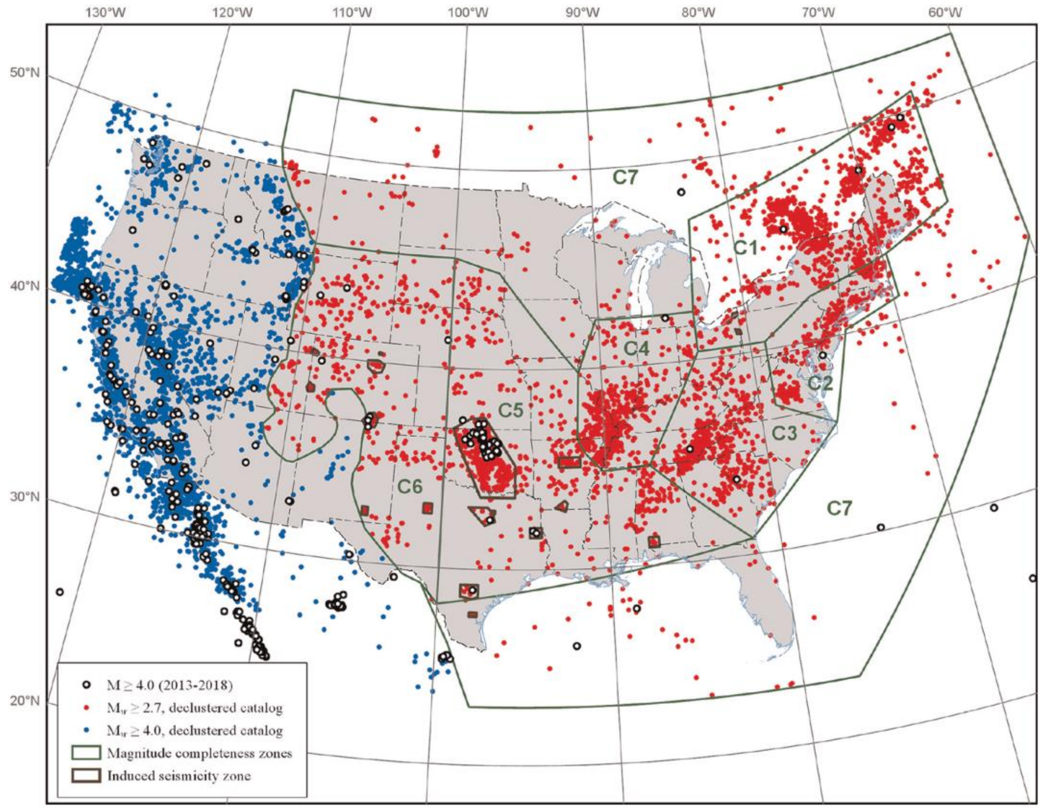


Figure 1. The 2018 NSHM Declustered Catalog for Central and Eastern United States (red) and Western United States (blue).

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

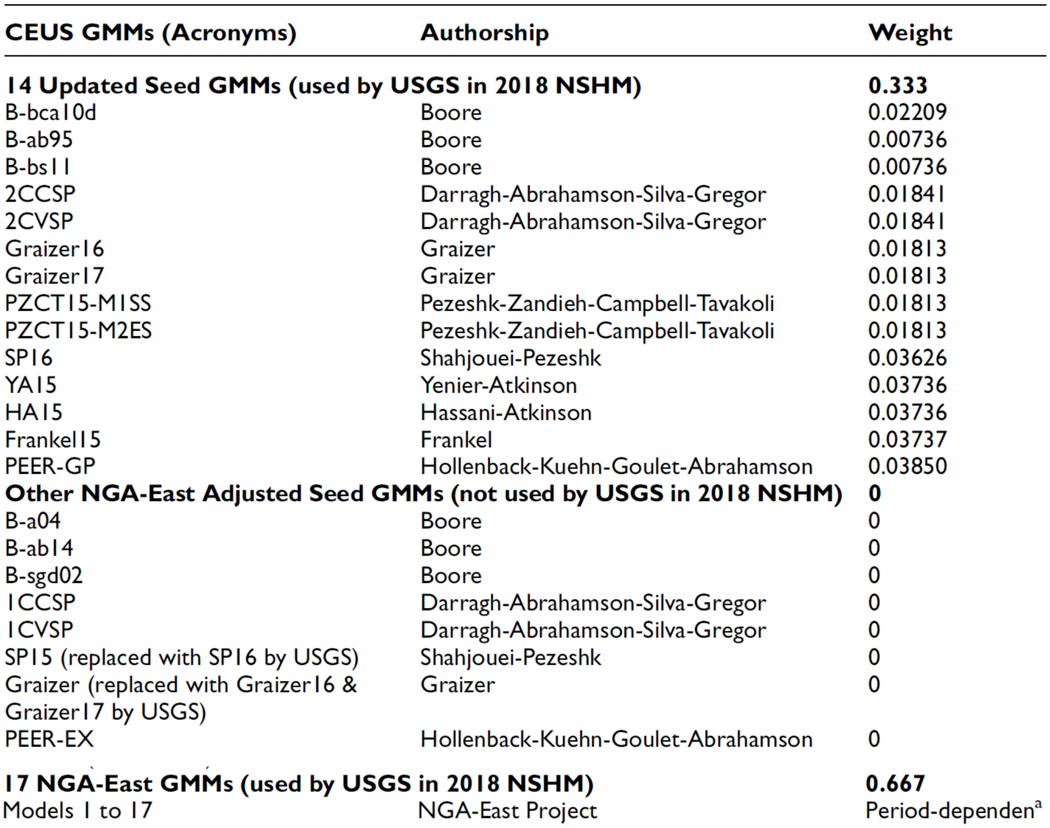
In general, the characteristics of the fault source, such as distance, type, magnitude, and site conditions, are used to estimate the magnitude of an earthquake parameter (spectral acceleration, peak ground acceleration, etc.) via ground-motion models (GMMs) or ground-motion prediction equations (GMPEs), 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).[*Source Rezaeian et al.* (2021)].



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
17 NGA-East GMMs (used by USGS in 2018 NSHM)		**0.667**
Models 1 to 17	NGA-East Project	Period-dependen[a]

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 with 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 744 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 with 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	1.320
F_v	2.180
F_{PGA}	1.189
S_S	0.599
S_1	0.155
S_{DS}	0.791
S_{D1}	0.338
PGA	0.311
A_s	0.370

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.

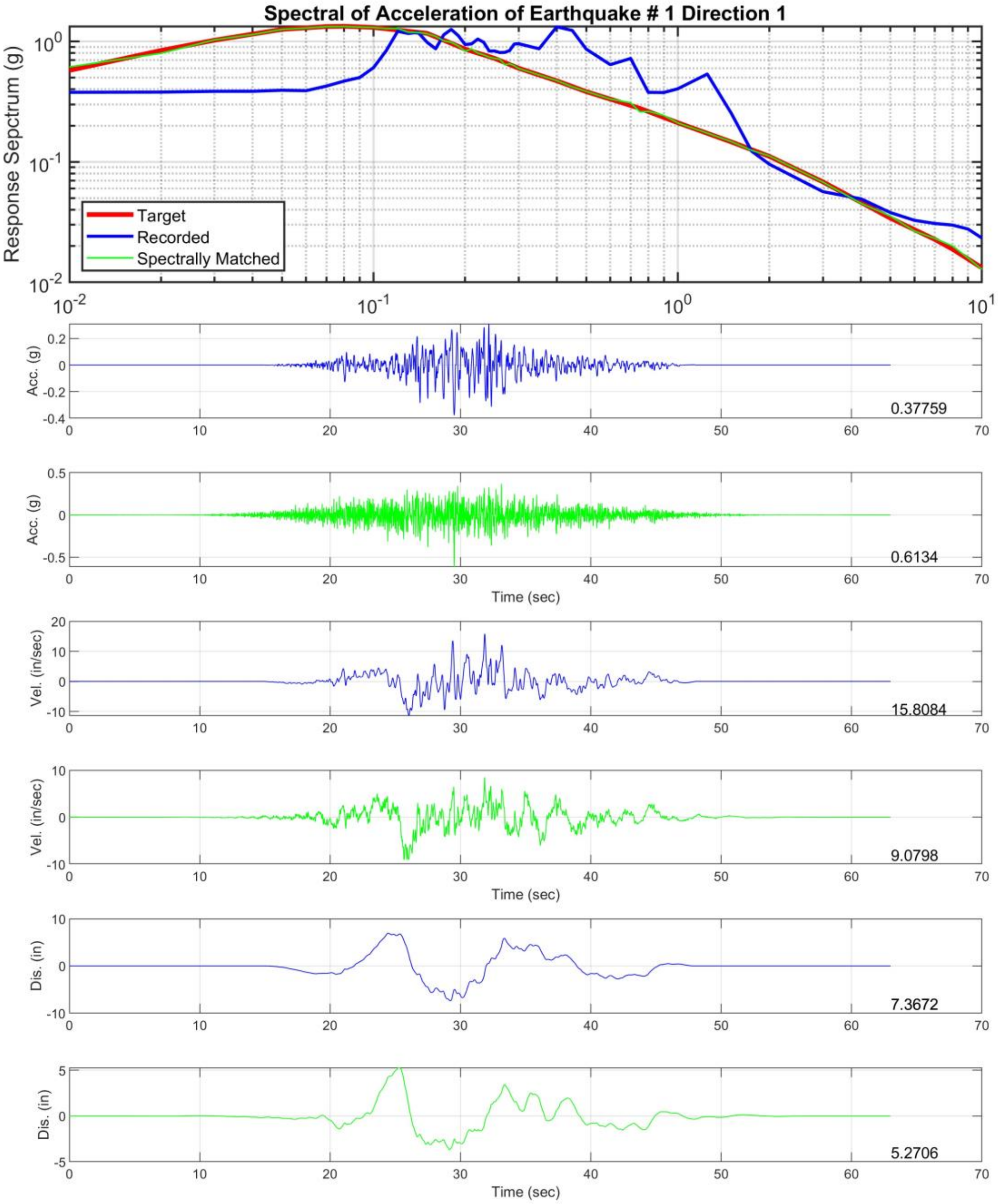


Figure 2. Time Histories Before and After the Spectral Matching Process for Earthquake #1. The numbers Shown in the Bottom right of Each Figure Represent the Absolute Maximum Value of the Graph.

Table 4. Deaggregation.

Mean and Mode Deaggregation Parameter at 1,033 Years					
Mean			**Mode**		
Period	**M**	**R (km)**	**Period**	**M**	**R (km)**
PGA	7.09	62.48	PGA	7.5	87.86
0.01	7.09	62.31	0.01	7.76	84.25
0.02	7.05	61.63	0.02	7.76	84.26
0.03	7.06	62.09	0.03	7.76	84.26
0.05	7.12	64.09	0.05	7.76	84.27
0.08	7.16	65.84	0.08	7.52	86.31
0.10	7.21	67.62	0.10	7.52	86.32
0.15	7.29	70.11	0.15	7.52	86.97
0.20	7.33	71.55	0.20	7.52	86.99
0.25	7.38	73.08	0.25	7.52	87.00
0.30	7.40	73.89	0.30	7.52	87.01
0.40	7.43	75.21	0.40	7.52	87.02
0.50	7.45	76.15	0.50	7.52	87.03
0.75	7.50	77.98	0.75	7.52	87.04
1.00	7.52	78.98	1.00	7.52	87.04
1.50	7.56	80.02	1.50	7.51	87.46
2.00	7.59	80.66	2.00	7.76	84.35
3.00	7.62	81.00	3.00	7.76	84.36
4.00	7.63	81.28	4.00	7.76	84.36
5.00	7.64	81.41	5.00	7.51	87.10
7.50	7.66	81.44	7.50	7.50	86.97
10.00	7.67	81.52	10.00	7.50	86.97

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 abovementioned procedure, 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 so that it is not 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	0.295
0.030	0.345
0.040	0.378
0.050	0.411
0.070	0.477
0.100	0.527
0.150	0.533
0.200	0.575
0.250	0.582
0.300	0.609
0.400	0.553
0.500	0.527
0.750	0.470
1.000	0.388
1.500	0.336
2.000	0.273
3.000	0.137
4.000	0.073
5.000	0.051
7.500	0.030
10.000	0.035

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

PARAMETER	DESIGN ACCELERATION PARAMETERS (g)
S_{DS}	0.575
S_{D1}	0.388
S_{MS}	0.575
S_{M1}	0.388
MCE_G	0.295

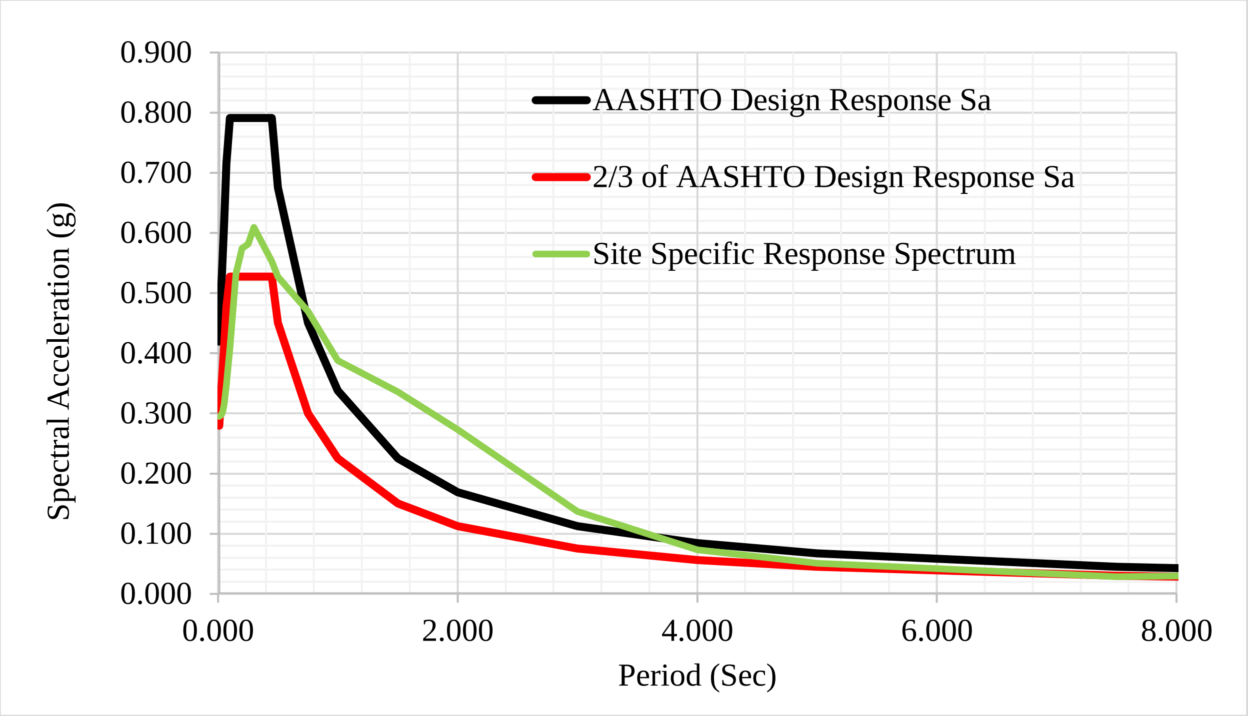


Figure 3. Site-Specific Design Response Spectrum, AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions Design Response Spectrum.

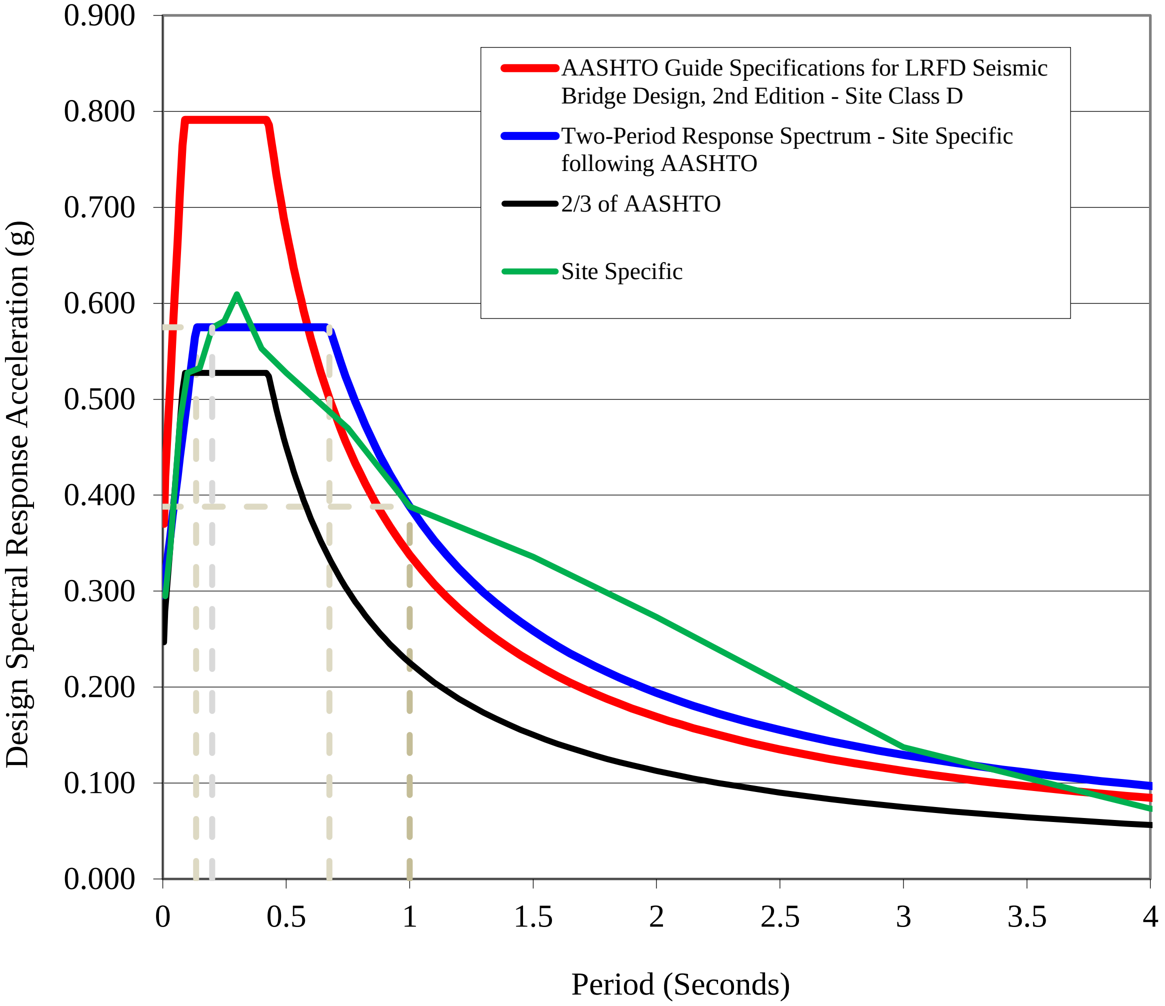


Figure 4. Design Response Spectrum based on AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions, 2/3 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions 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

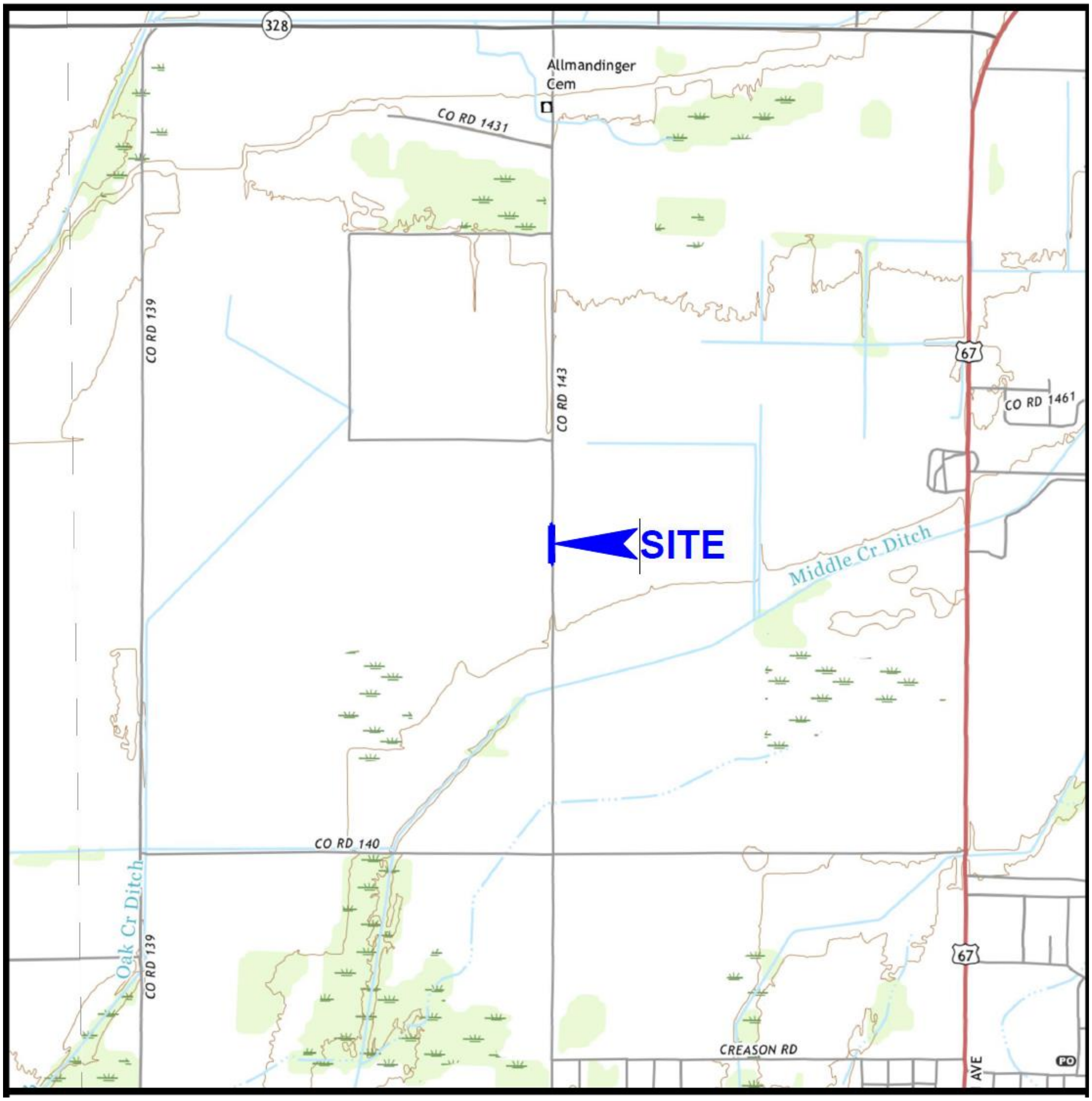


Figure A.1. The Location of the Study Site.

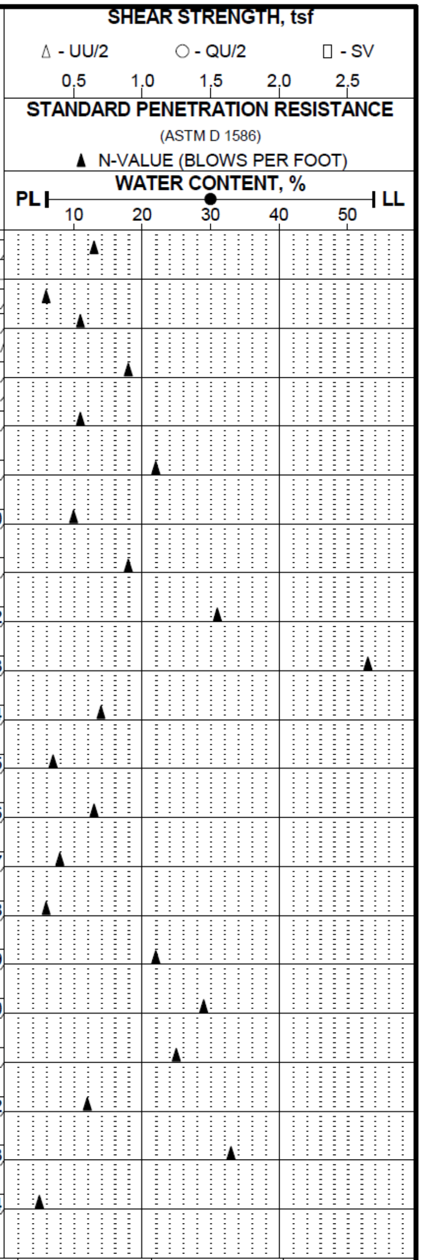
APPENDIX B. Boring Log

Surface Elevation: 292

Datum NAVD88

Completion Date: 2/22/24

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		9 inches of sand and gravel	3-6-7	SS1
		Medium stiff to stiff, gray and brown to gray, silty, FAT CLAY - CH		ST2
5	287	trace gravel	2-2-4	SS3
		trace sand	1-5-6	SS4
10	282	trace gravel and sand		ST5
		little sand		
15	277	3-inch gray clayey sand seam	4-7-11	SS6
				ST7
		Loose to very dense, gray and brown to gray SAND - SP ∇		
20	272		4-6-5	SS8
25	267		6-11-11	SS9
30	262		3-5-5	SS10
35	257		4-7-11	SS11
40	252		6-11-20	SS12
45	247		13-26-27	SS13
50	242	trace gravel	6-8-6	SS14
55	237	trace gravel	4-4-3	SS15
60	232	2.5-inch black lignite seam	8-7-6	SS16
		some gravel		
65	227	Medium dense, gray and brown SAND and gravel - SP	4-4-4	SS17
		trace lignite		
70	222	Loose to dense, gray SAND - SP	2-3-3	SS18
		trace gravel		
75	217	trace gravel and clay	6-12-10	SS19
80	212		9-14-15	SS20
85	207		6-11-14	SS21
90	202	little gravel, trace clay	7-7-5	SS22
95	197	trace gravel and lignite	6-17-16	SS23
100	192	trace gravel	4-3-2	SS24
		1-inch gravel and sand seam		
		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

ENCOUNTERED AT 18.5 FEET ∇

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 20 FEET
WEC DRILLER SAS LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 85 %

REMARKS:

Drawn by: SAS	Checked by:	App'vd. by:
Date: 2/26/24	Date:	Date:

GEOTECHNOLOGY
A UES Company

Corning Bypass (Future I-57) (S)
Clay County, Arkansas

LOG OF BORING: B-1

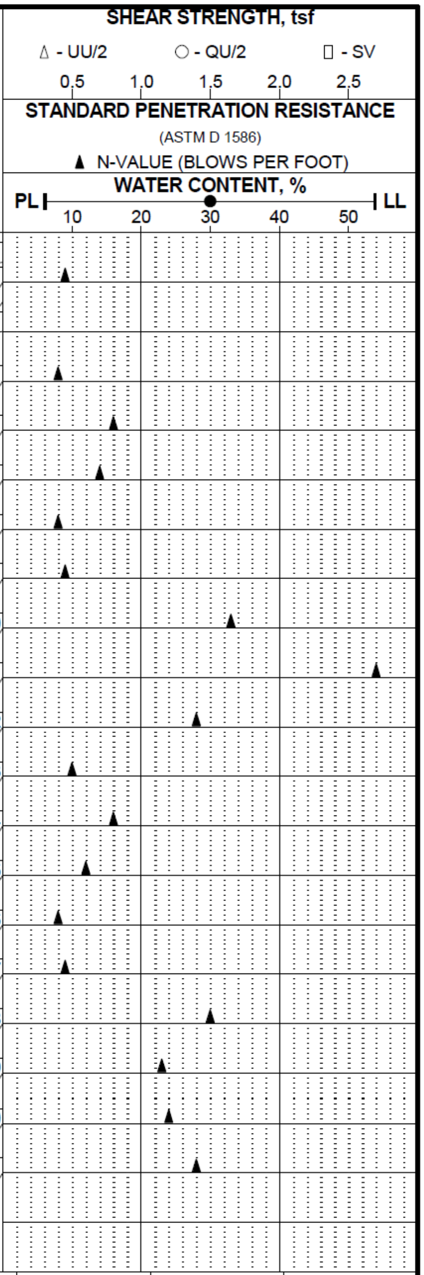
Project No. J045279.01

Surface Elevation: 292

Datum NAVD88

Completion Date: 2/23/24

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		9 inches of sand and gravel		ST1
5	287	Stiff, gray and brown to gray, silty, FAT CLAY - CH little organics, trace sand	2-4-5	SS2
				ST3
10	282			ST4
15	277	Medium dense, gray, CLAYEY SAND - SC	1-3-5	SS5
20	272	Medium dense to very dense, gray SAND - SP	3-8-8	SS6
25	267		3-8-6	SS7
30	262		2-4-4	SS8
35	257		3-4-5	SS9
40	252	trace lignite	4-11-22	SS10
45	247		9-24-30	SS11
50	242		8-17-11	SS12
55	237	some gravel	3-5-5	SS13
60	232	trace gravel	4-7-9	SS14
65	227	trace gravel	5-8-4	SS15
70	222	Medium dense, gray SAND and gravel - SP	3-4-4	SS16
75	217		3-4-5	SS17
80	212	Dense, gray SAND - SP trace gravel	6-12-18	SS18
85	207		5-9-14	SS19
90	202	trace gravel	5-10-14	SS20
95	197	3-inch lignite seam Boring terminated at 95 feet due to borehole collapse.	5-11-17	SS21
100	192			



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

LOG OF BORING 2020 JDM - ELEVATIONS J045279.01.GPJ GTINC 0638301.GPJ 2/27/24

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 10 FEET
WEC DRILLER SAS LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 85 %

REMARKS: Borehole collapsed at approxiamtely 95 feet.

Drawn by: SAS	Checked by:	App'vd. by:
Date: 2/26/24	Date:	Date:

GEOTECHNOLOGY
A UES Company

Corning Bypass (Future I-57) (S)
Clay County, Arkansas

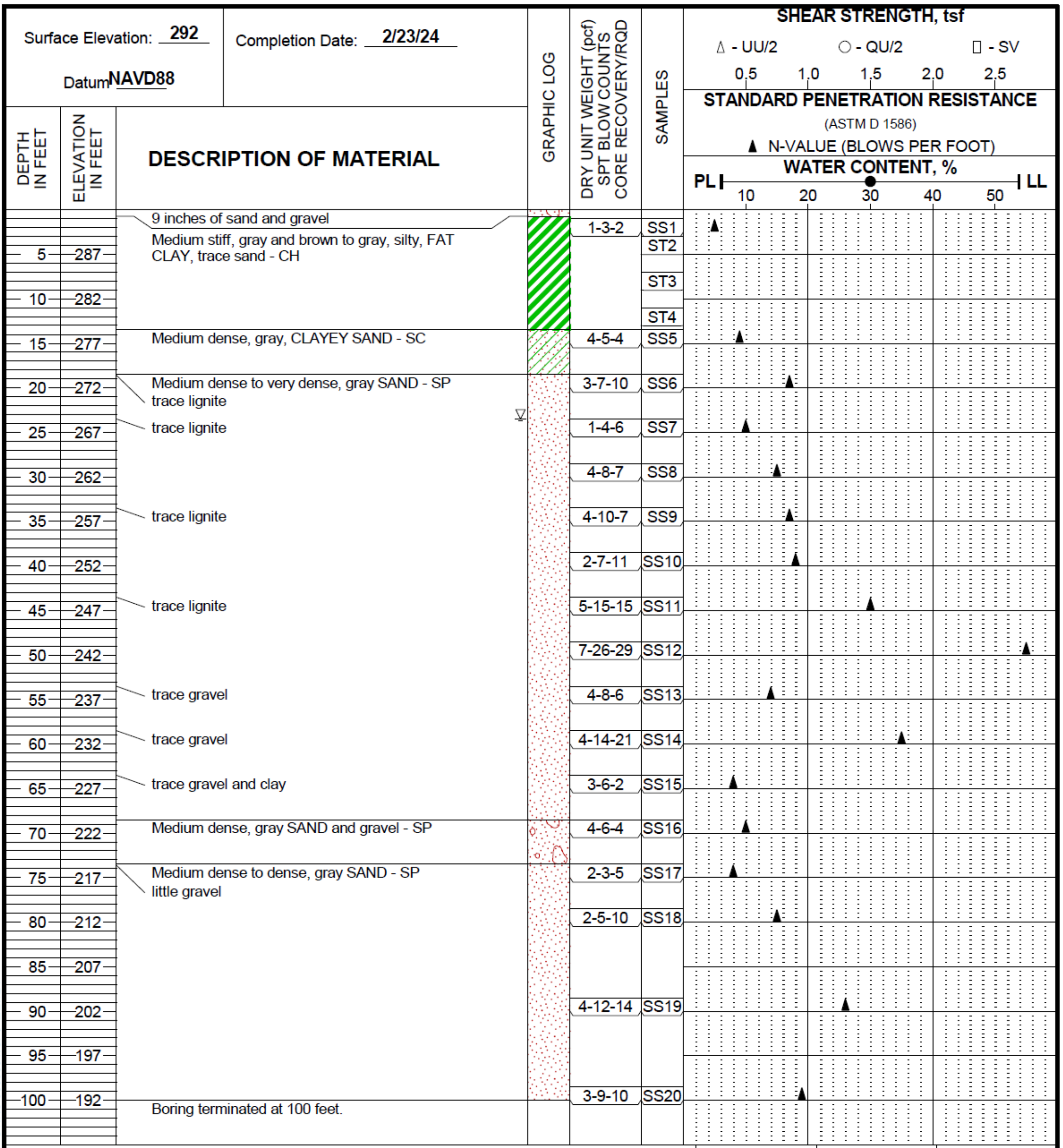
LOG OF BORING: B-3

Project No. J045279.01

Surface Elevation: 292

Datum NAVD88

Completion Date: 2/23/24



DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		9 inches of sand and gravel	1-3-2	SS1
		Medium stiff, gray and brown to gray, silty, FAT CLAY, trace sand - CH		ST2
5	287			
				ST3
10	282			
				ST4
		Medium dense, gray, CLAYEY SAND - SC	4-5-4	SS5
15	277			
		Medium dense to very dense, gray SAND - SP trace lignite	3-7-10	SS6
20	272			
		∇		
		trace lignite	1-4-6	SS7
25	267			
			4-8-7	SS8
30	262			
		trace lignite	4-10-7	SS9
35	257			
			2-7-11	SS10
40	252			
		trace lignite	5-15-15	SS11
45	247			
			7-26-29	SS12
50	242			
		trace gravel	4-8-6	SS13
55	237			
		trace gravel	4-14-21	SS14
60	232			
		trace gravel and clay	3-6-2	SS15
65	227			
		Medium dense, gray SAND and gravel - SP	4-6-4	SS16
70	222			
		Medium dense to dense, gray SAND - SP little gravel	2-3-5	SS17
75	217			
			2-5-10	SS18
80	212			
85	207			
			4-12-14	SS19
90	202			
95	197			
			3-9-10	SS20
100	192	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

ENCOUNTERED AT 23.5 FEET ∇

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
WEC DRILLER SAS LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 85 %

REMARKS:

Drawn by: SAS	Checked by:	App'vd. by:
Date: 2/26/24	Date:	Date:

GEOTECHNOLOGY
A UES Company

Corning Bypass (Future I-57) (S)
Clay County, Arkansas

LOG OF BORING: B-5

Project No. J045279.01

LOG OF BORING 2020 JDM - ELEVATIONS J045279.01.GPJ GTINC 0638301.GPJ 2/27/24

APPENDIX G – AASHTO AND USCS CLASSIFICATIONS

ARDOT 101172 Corning Bypass (Future I-57)(S)
J045279.01

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	58	19	39	--	--	--	--	A-7-6	CH
	8	73	31	42	100.0	100.0	71.8	32	A-7-5 (32)	CH
	11	97	32	65	--	--	--	--	A-7-5	CH
	38.5	--	--	--	99.8	95.9	5.3	--	A-2-6	SP-SC
	53.5	--	--	--	56.0	15.2	2.5	--	A-1-b	SP
	78.5	--	--	--	85.9	13.6	2.4	--	A-1-b	SP
	88.5	--	--	--	75.8	15.1	1.6	--	A-1-b	SP
B-3	3	119	28	91	--	--	--	--	A-7-6	CH
	5	61	27	34	100.0	100.0	93.9	37	A-7-6 (37)	CH
	9	83	33	50	--	--	--	--	A-7-5	CH
	11	46	25	21	--	--	--	--	A-7-6	CL
	13	--	--	--	100.0	100.0	81.9	--	A-7-6	CH
	18.5	--	--	--	99.9	98.8	2.5	--	A-3	SP
	38.5	--	--	--	100.0	98.5	3.2	--	A-3	SP
	58.5	--	--	--	88.3	26.8	2.6	--	A-1-b	SP
	68.5	--	--	--	81.2	11.9	1.5	--	A-1-b	SP
B-5	3	62	24	38	--	--	--	--	A-7-6	CH
	5	33	24	9	--	--	--	--	A-4	ML
	7	31	26	5	100.0	100.0	73.1	3	A-4 (3)	ML
	9	48	25	23	--	--	--	--	A-7-6	CL
	18.5	--	--	--	100.0	100.0	3.9	--	A-3	SP
	33.5	--	--	--	100.0	100.0	3.7	--	A-3	SP
	58.5	--	--	--	73.0	21.4	1.7	--	A-1-b	SP
	78.5	--	--	--	79.0	21.0	2.5	--	A-1-b	SP
	83.5	--	--	--	93.5	29.3	3.5	--	A-1-b	SP

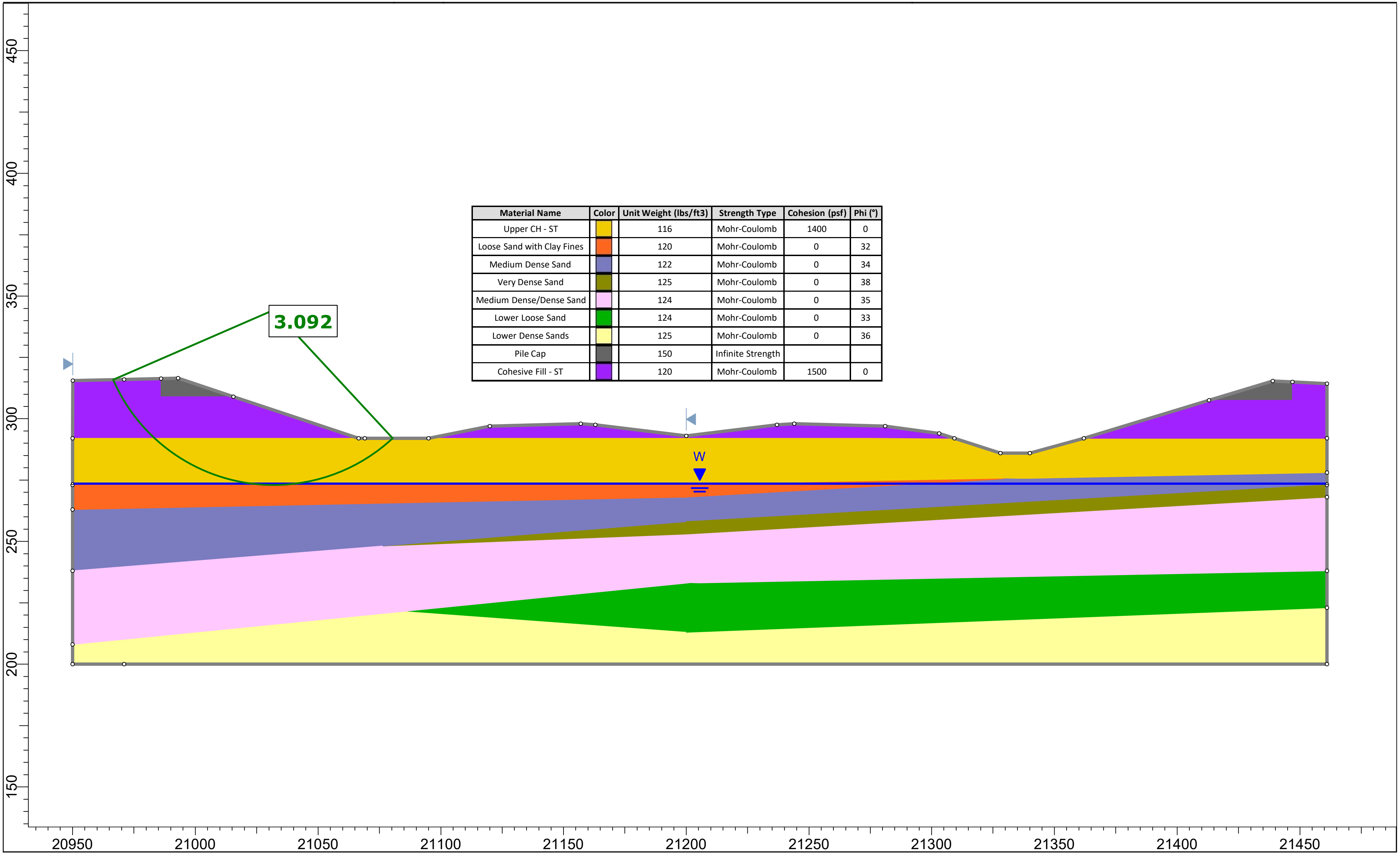
APPENDIX H – GLOBAL STABILITY ANALYSES

GEOTECHNOLOGY
A UES Company

File Name: ARDOT 101172 Clay County Bridge Abutments.slmd
Name: South Abutment
Description: Short Term Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.032



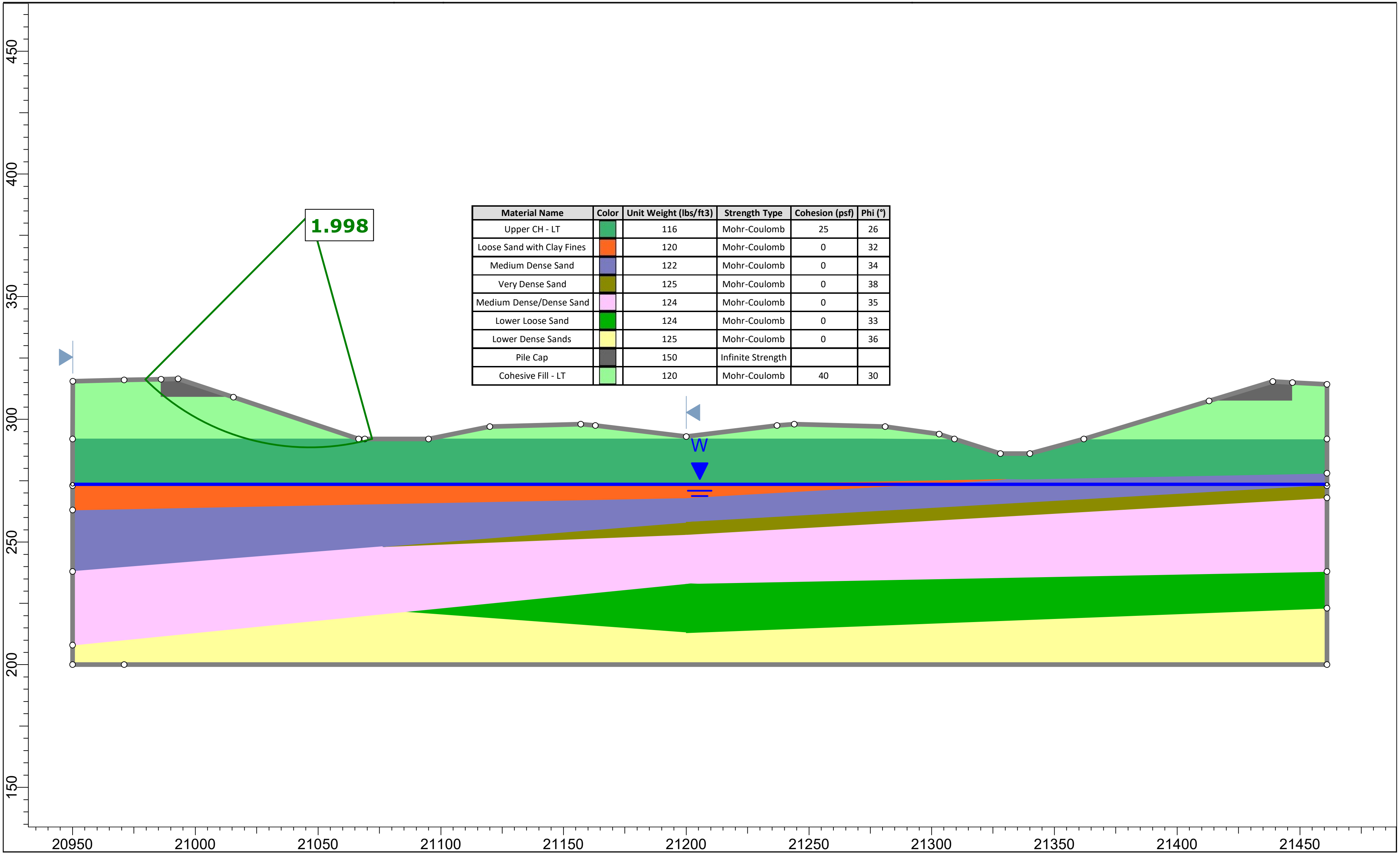
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - ST	🟨	116	Mohr-Coulomb	1400	0
Loose Sand with Clay Fines	🟧	120	Mohr-Coulomb	0	32
Medium Dense Sand	🟦	122	Mohr-Coulomb	0	34
Very Dense Sand	🟫	125	Mohr-Coulomb	0	38
Medium Dense/Dense Sand	⬜	124	Mohr-Coulomb	0	35
Lower Loose Sand	🟩	124	Mohr-Coulomb	0	33
Lower Dense Sands	🟨	125	Mohr-Coulomb	0	36
Pile Cap	⬛	150	Infinite Strength		
Cohesive Fill - ST	🟪	120	Mohr-Coulomb	1500	0

GEOTECHNOLOGY
A UES Company

File Name: ARDOT 101172 Clay County Bridge Abutments.slmd
Name: South Abutment
Description: Long Term Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.032



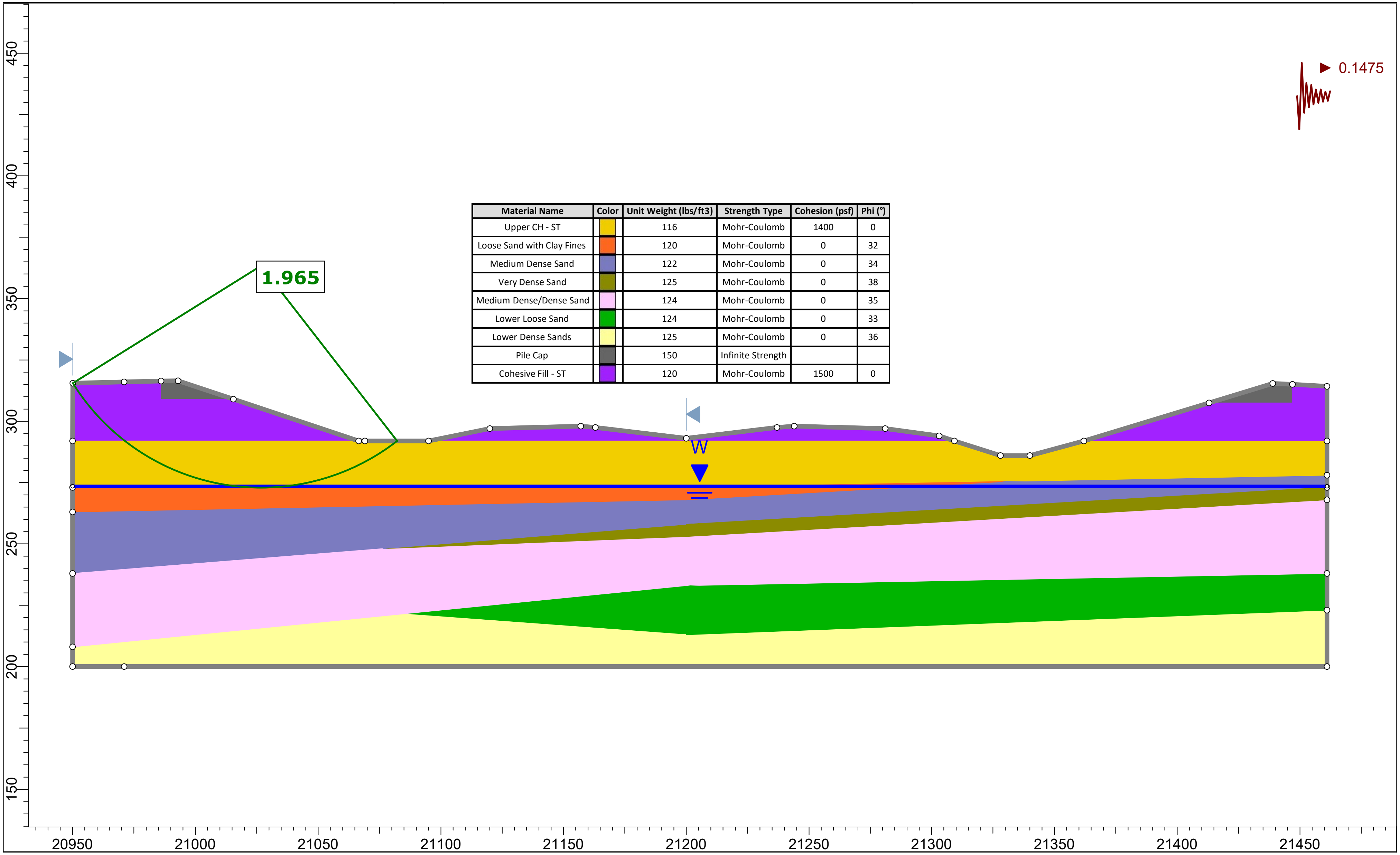
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - LT	🟩	116	Mohr-Coulomb	25	26
Loose Sand with Clay Fines	🟧	120	Mohr-Coulomb	0	32
Medium Dense Sand	🟪	122	Mohr-Coulomb	0	34
Very Dense Sand	🟫	125	Mohr-Coulomb	0	38
Medium Dense/Dense Sand	🟪	124	Mohr-Coulomb	0	35
Lower Loose Sand	🟩	124	Mohr-Coulomb	0	33
Lower Dense Sands	🟨	125	Mohr-Coulomb	0	36
Pile Cap	⬛	150	Infinite Strength		
Cohesive Fill - LT	🟩	120	Mohr-Coulomb	40	30

GEOTECHNOLOGY
A UES Company

File Name: ARDOT 101172 Clay County Bridge Abutments.slmd
Name: South Abutment
Description: Seismic Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.032



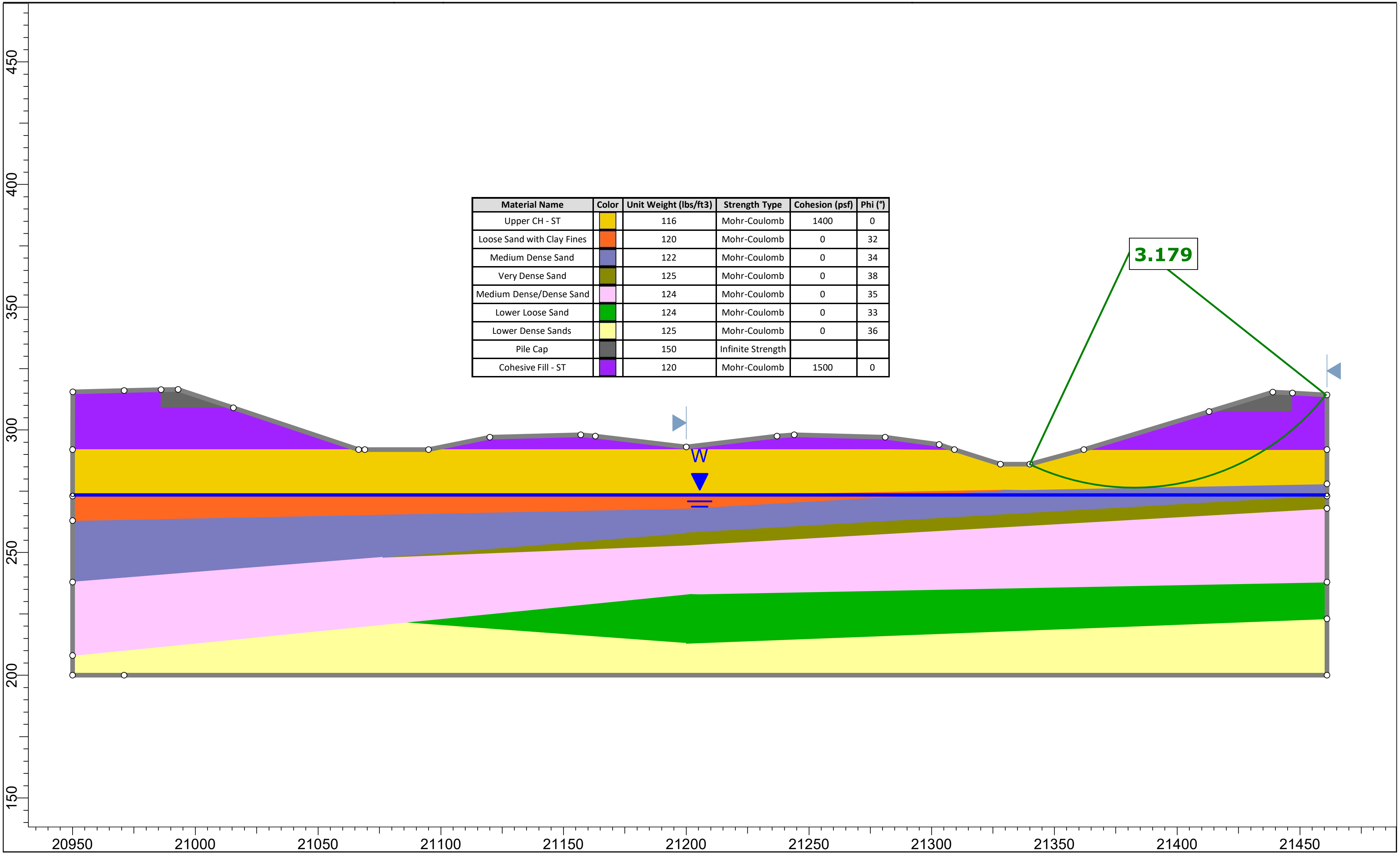
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - ST	🟨	116	Mohr-Coulomb	1400	0
Loose Sand with Clay Fines	🟧	120	Mohr-Coulomb	0	32
Medium Dense Sand	🟦	122	Mohr-Coulomb	0	34
Very Dense Sand	🟫	125	Mohr-Coulomb	0	38
Medium Dense/Dense Sand	▨	124	Mohr-Coulomb	0	35
Lower Loose Sand	🟩	124	Mohr-Coulomb	0	33
Lower Dense Sands	🟨	125	Mohr-Coulomb	0	36
Pile Cap	⬛	150	Infinite Strength		
Cohesive Fill - ST	🟪	120	Mohr-Coulomb	1500	0

GEOTECHNOLOGY
A UES Company

File Name: ARDOT 101172 Clay County Bridge Abutments.slmd
Name: North Abutment
Description: Short Term Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.032



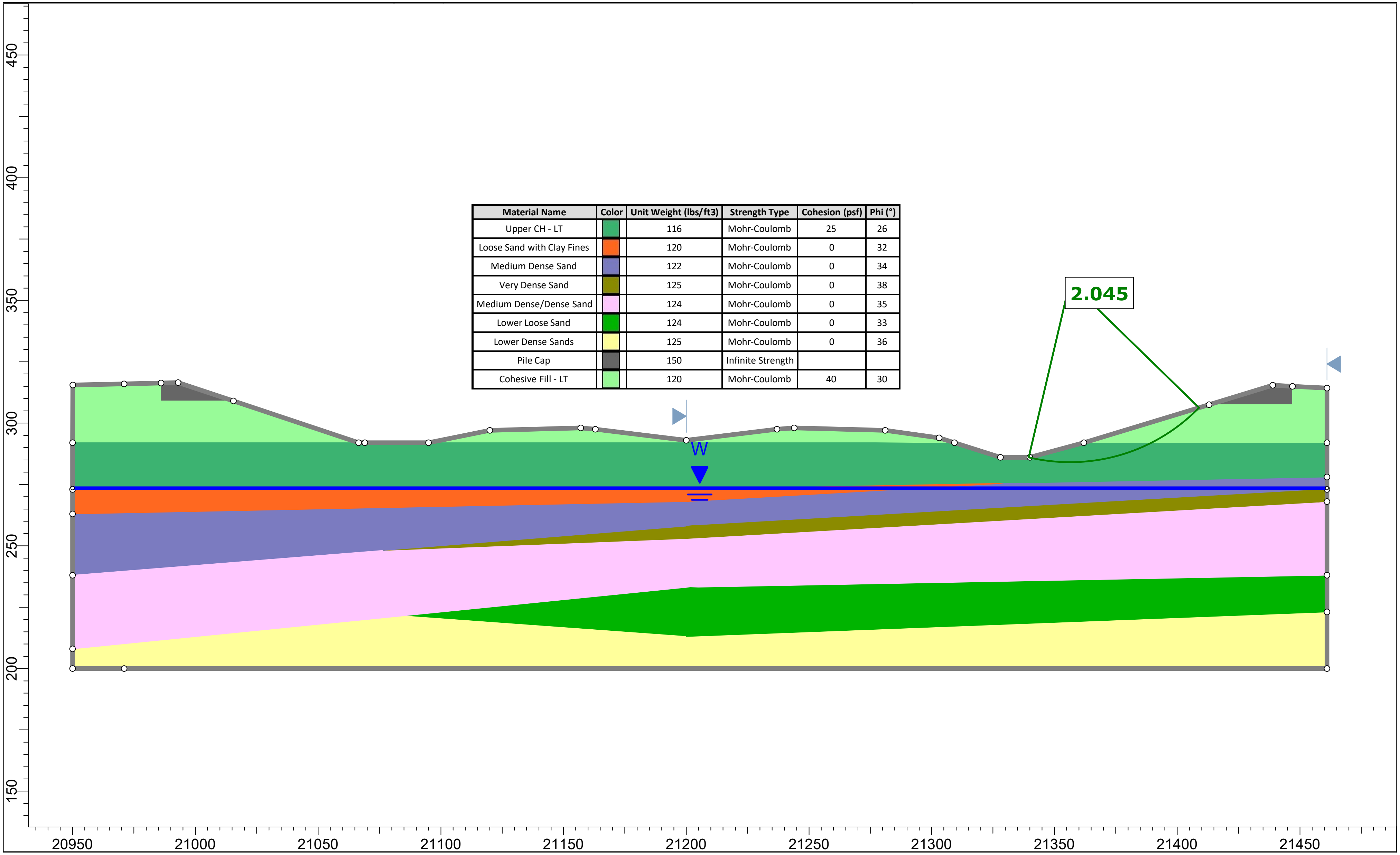
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - ST	🟨	116	Mohr-Coulomb	1400	0
Loose Sand with Clay Fines	🟧	120	Mohr-Coulomb	0	32
Medium Dense Sand	🟦	122	Mohr-Coulomb	0	34
Very Dense Sand	🟫	125	Mohr-Coulomb	0	38
Medium Dense/Dense Sand	⬜	124	Mohr-Coulomb	0	35
Lower Loose Sand	🟩	124	Mohr-Coulomb	0	33
Lower Dense Sands	🟨	125	Mohr-Coulomb	0	36
Pile Cap	⬛	150	Infinite Strength		
Cohesive Fill - ST	🟪	120	Mohr-Coulomb	1500	0

GEOTECHNOLOGY
A UES Company

File Name: ARDOT 101172 Clay County Bridge Abutments.slmd
Name: North Abutment
Description: Long Term Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.032



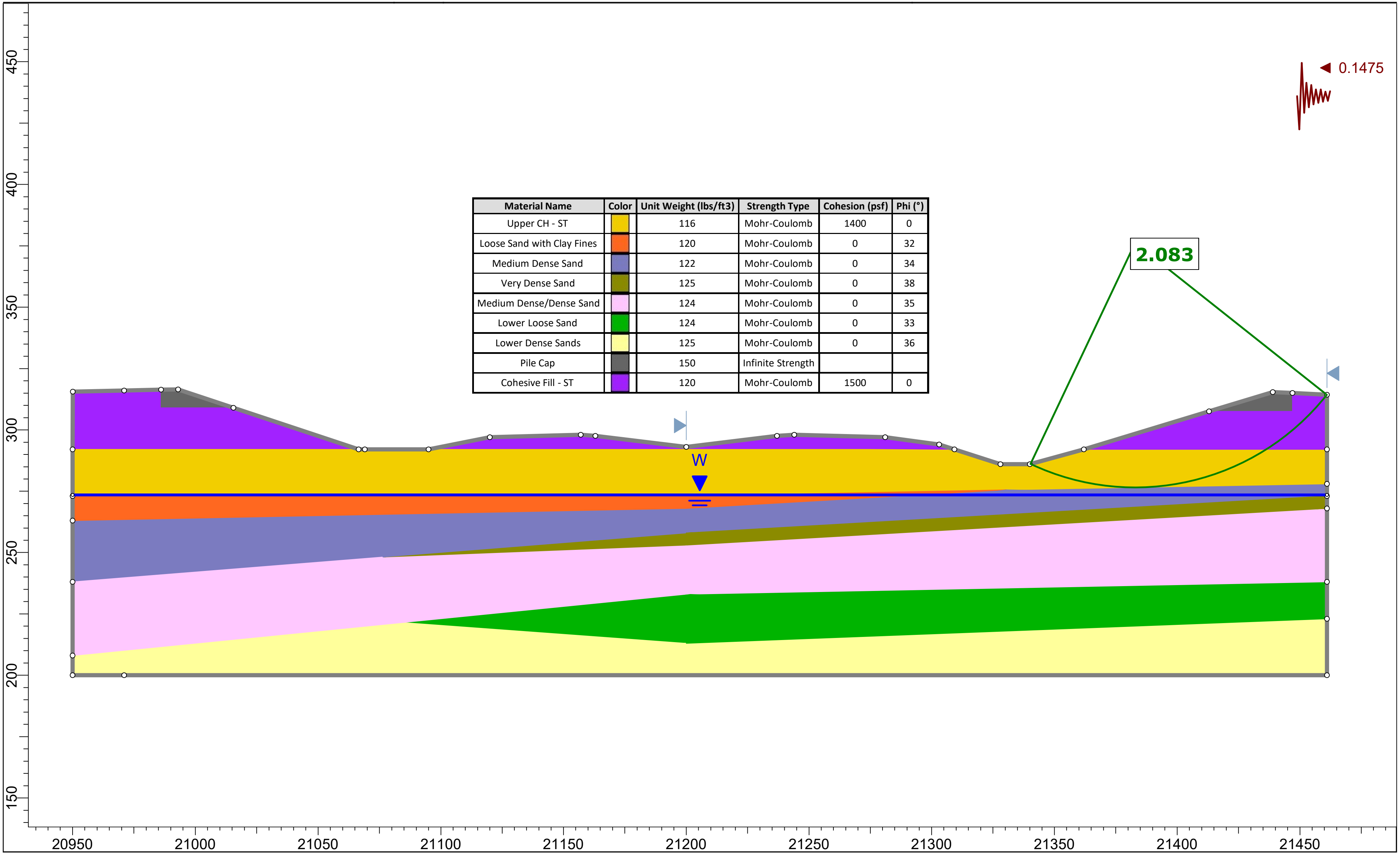
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - LT	🟩	116	Mohr-Coulomb	25	26
Loose Sand with Clay Fines	🟧	120	Mohr-Coulomb	0	32
Medium Dense Sand	🟦	122	Mohr-Coulomb	0	34
Very Dense Sand	🟫	125	Mohr-Coulomb	0	38
Medium Dense/Dense Sand	🟪	124	Mohr-Coulomb	0	35
Lower Loose Sand	🟩	124	Mohr-Coulomb	0	33
Lower Dense Sands	🟨	125	Mohr-Coulomb	0	36
Pile Cap	⬛	150	Infinite Strength		
Cohesive Fill - LT	🟩	120	Mohr-Coulomb	40	30

GEOTECHNOLOGY
A UES Company

File Name: ARDOT 101172 Clay County Bridge Abutments.slmd
Name: North Abutment
Description: Seismic Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.032



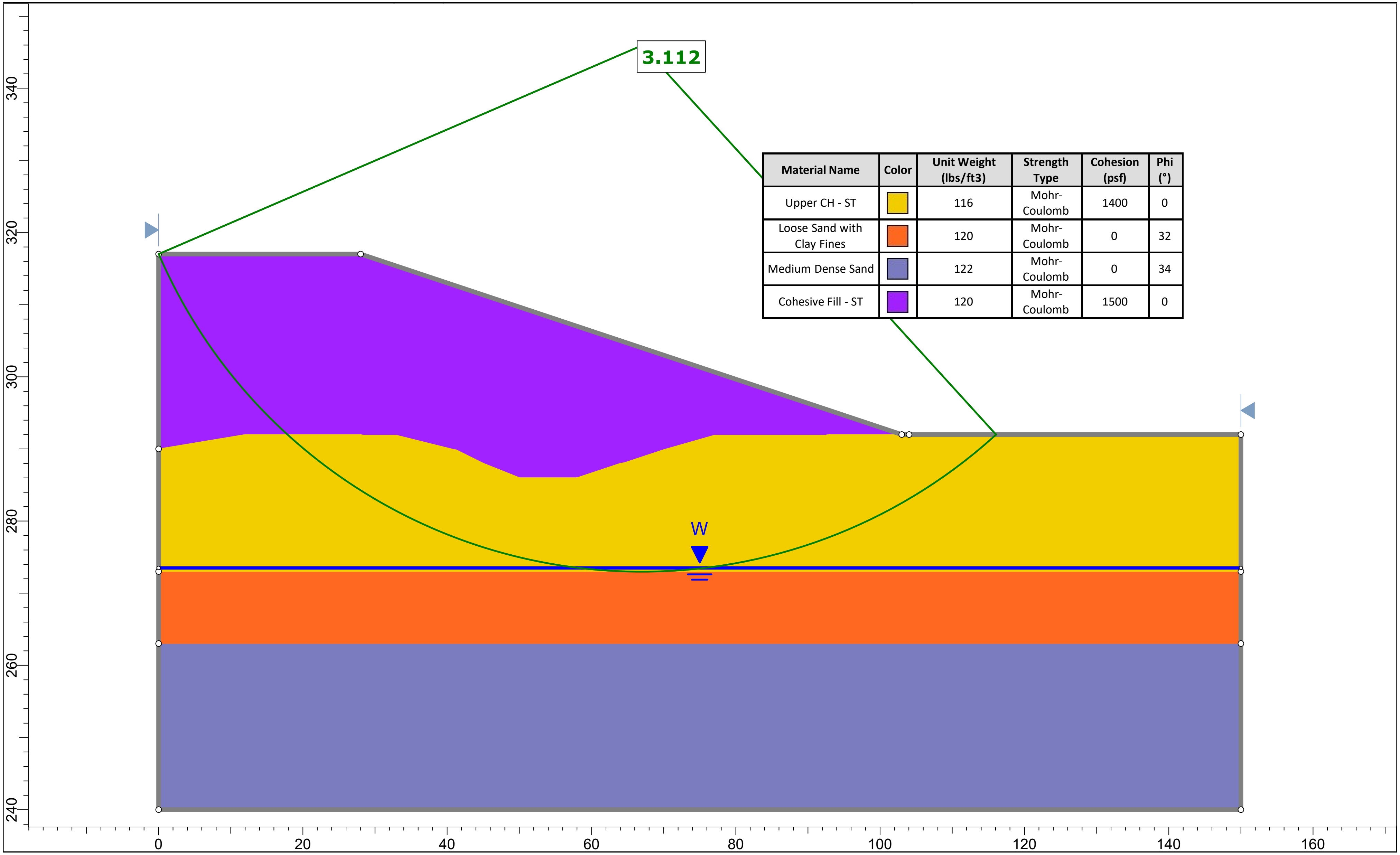
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - ST	🟨	116	Mohr-Coulomb	1400	0
Loose Sand with Clay Fines	🟧	120	Mohr-Coulomb	0	32
Medium Dense Sand	🟦	122	Mohr-Coulomb	0	34
Very Dense Sand	🟫	125	Mohr-Coulomb	0	38
Medium Dense/Dense Sand	■	124	Mohr-Coulomb	0	35
Lower Loose Sand	🟩	124	Mohr-Coulomb	0	33
Lower Dense Sands	🟨	125	Mohr-Coulomb	0	36
Pile Cap	⬛	150	Infinite Strength		
Cohesive Fill - ST	🟪	120	Mohr-Coulomb	1500	0

GEOTECHNOLOGY
A UES Company

File Name: South Abutment South Side Slope.slmd
Name: STA 210+00.00
Description: Short Term Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.034



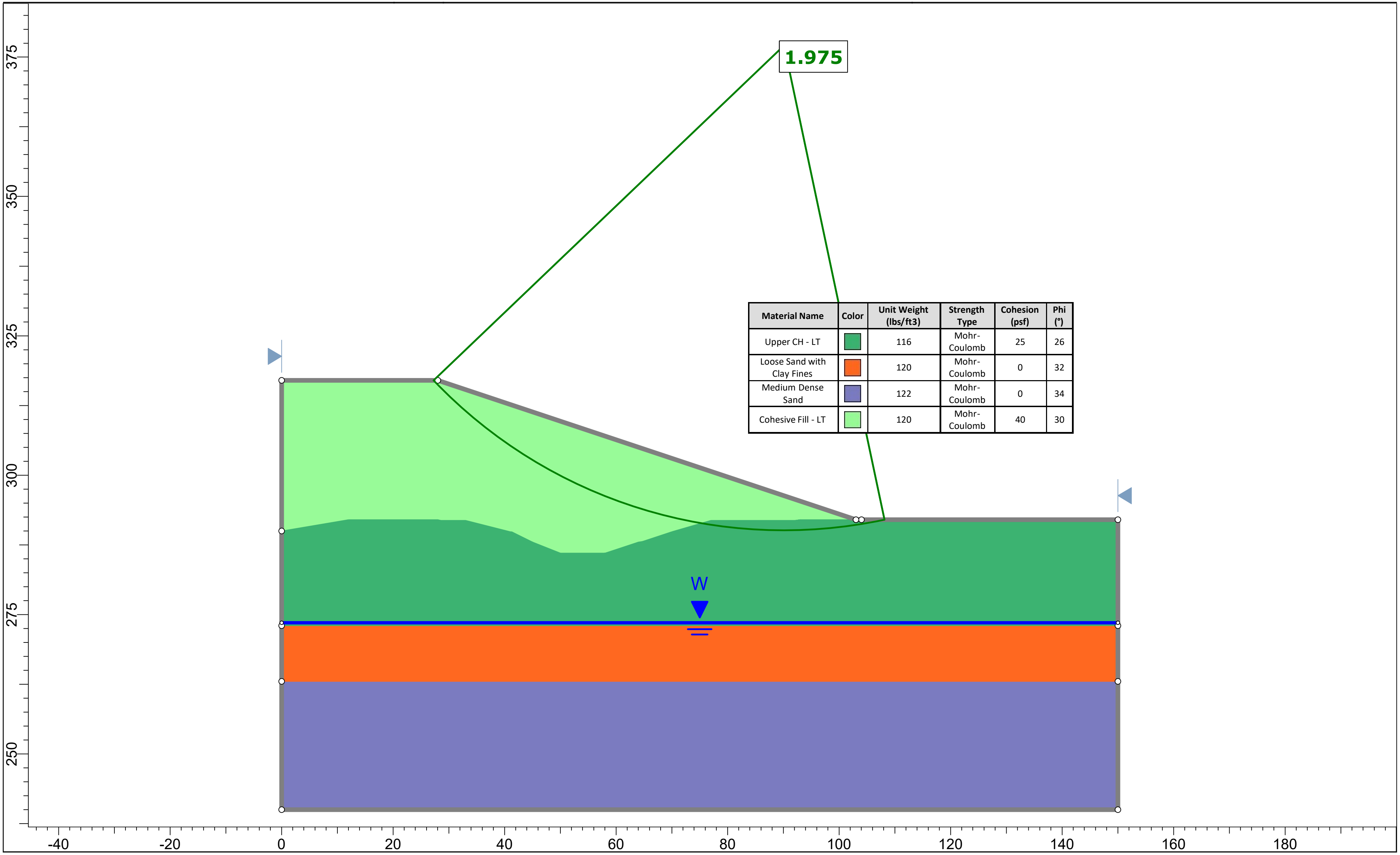
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - ST	🟨	116	Mohr-Coulomb	1400	0
Loose Sand with Clay Fines	🟧	120	Mohr-Coulomb	0	32
Medium Dense Sand	🟦	122	Mohr-Coulomb	0	34
Cohesive Fill - ST	🟪	120	Mohr-Coulomb	1500	0

GEOTECHNOLOGY
A UES Company

File Name: South Abutment South Side Slope.slmd
Name: STA 210+00.00
Description: Long Term Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.034



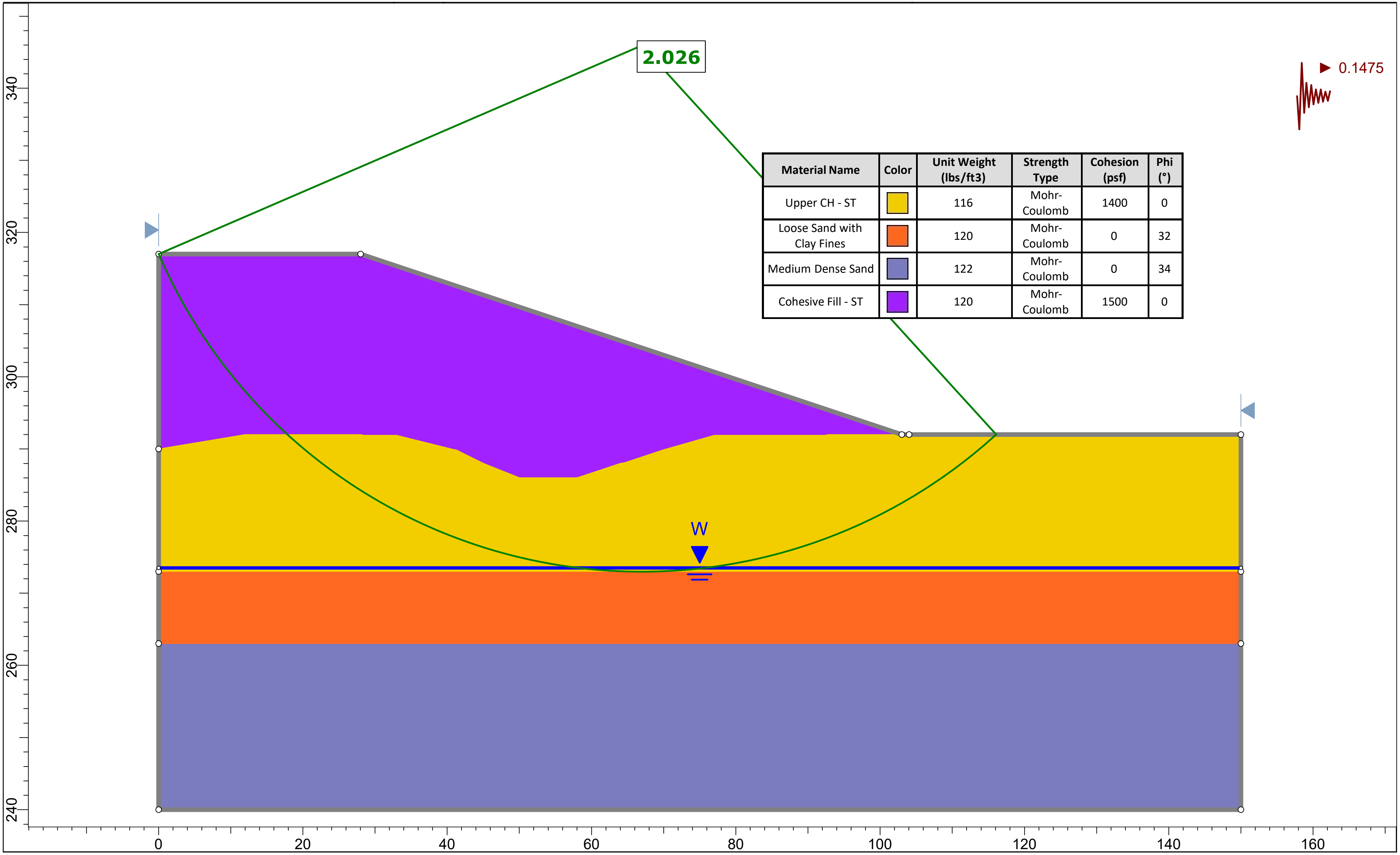
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - LT	🟩	116	Mohr-Coulomb	25	26
Loose Sand with Clay Fines	🟧	120	Mohr-Coulomb	0	32
Medium Dense Sand	🟪	122	Mohr-Coulomb	0	34
Cohesive Fill - LT	🟩	120	Mohr-Coulomb	40	30

GEOTECHNOLOGY
A UES Company

File Name: South Abutment South Side Slope.slmd
Name: STA 210+00.00
Description: Seismic Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.034



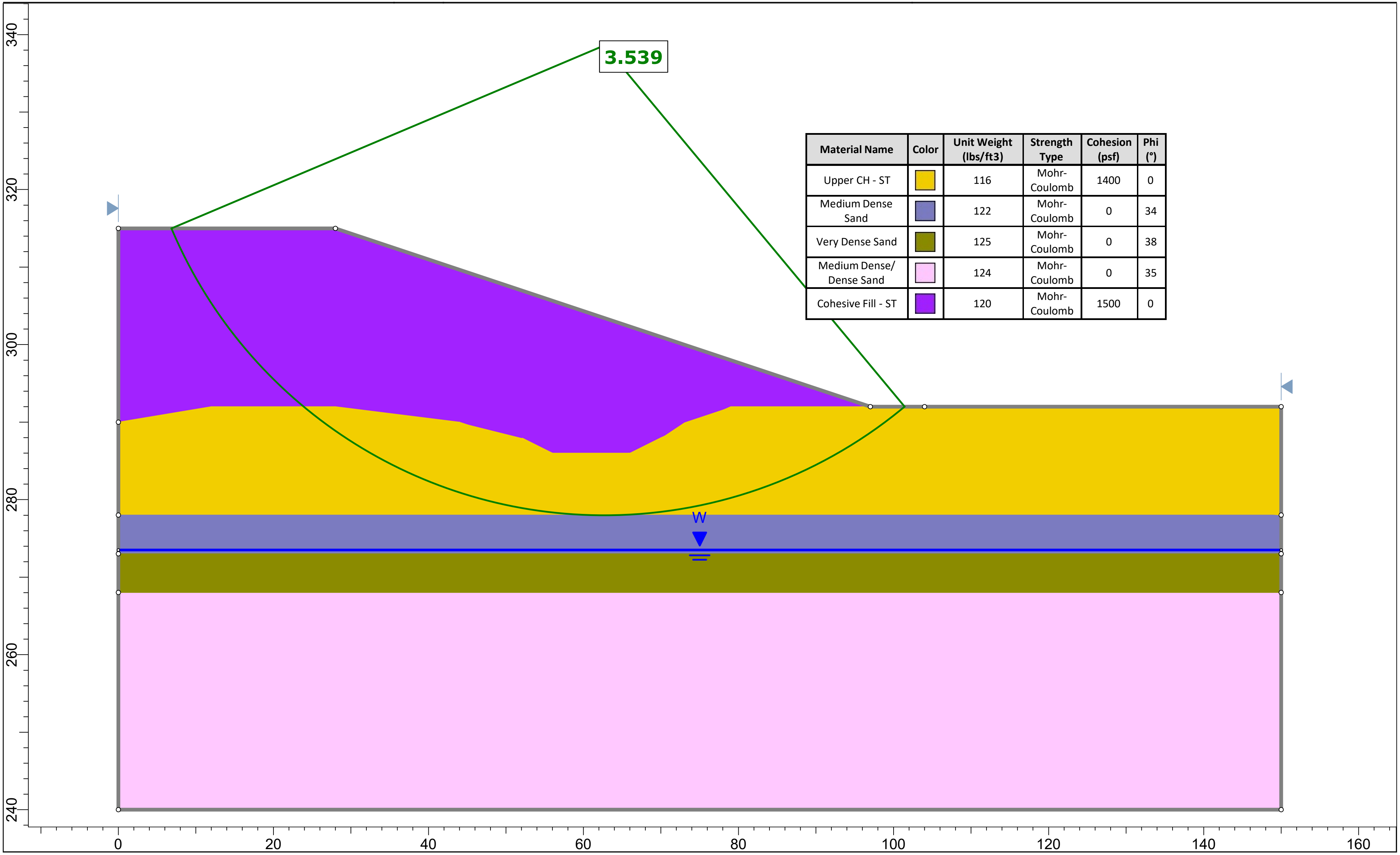
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - ST	🟨	116	Mohr-Coulomb	1400	0
Loose Sand with Clay Fines	🟧	120	Mohr-Coulomb	0	32
Medium Dense Sand	🟦	122	Mohr-Coulomb	0	34
Cohesive Fill - ST	🟪	120	Mohr-Coulomb	1500	0

GEOTECHNOLOGY
A UES Company

File Name: North Abutment South Side Slope.slmd
Name: STA 214+50.00
Description: Short Term Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.034



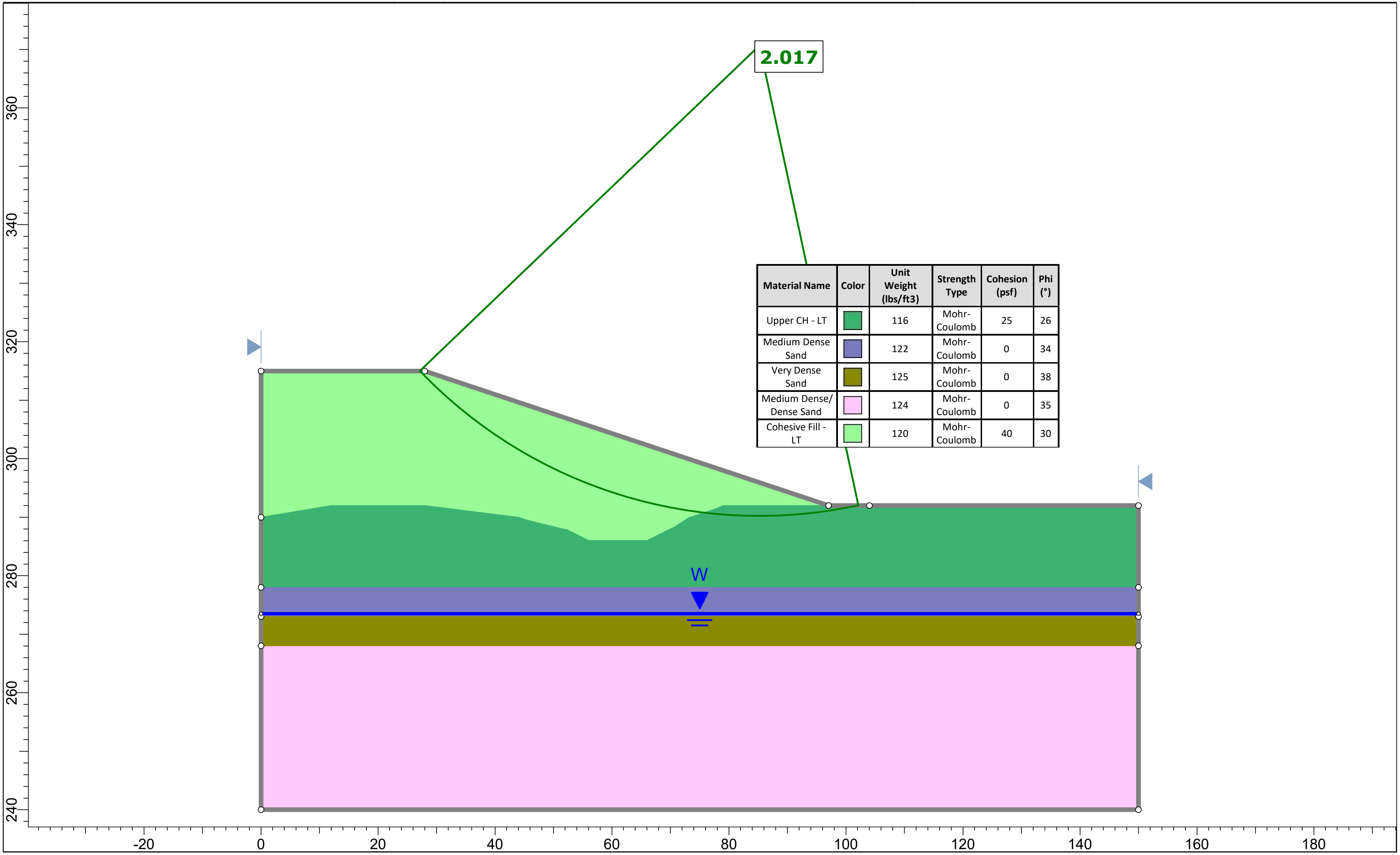
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - ST	🟨	116	Mohr-Coulomb	1400	0
Medium Dense Sand	🟦	122	Mohr-Coulomb	0	34
Very Dense Sand	🟫	125	Mohr-Coulomb	0	38
Medium Dense/ Dense Sand	■	124	Mohr-Coulomb	0	35
Cohesive Fill - ST	🟪	120	Mohr-Coulomb	1500	0

GEOTECHNOLOGY
A UES Company

File Name: North Abutment South Side Slope.slmd
Name: STA 214+50.00
Description: Long Term Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.034

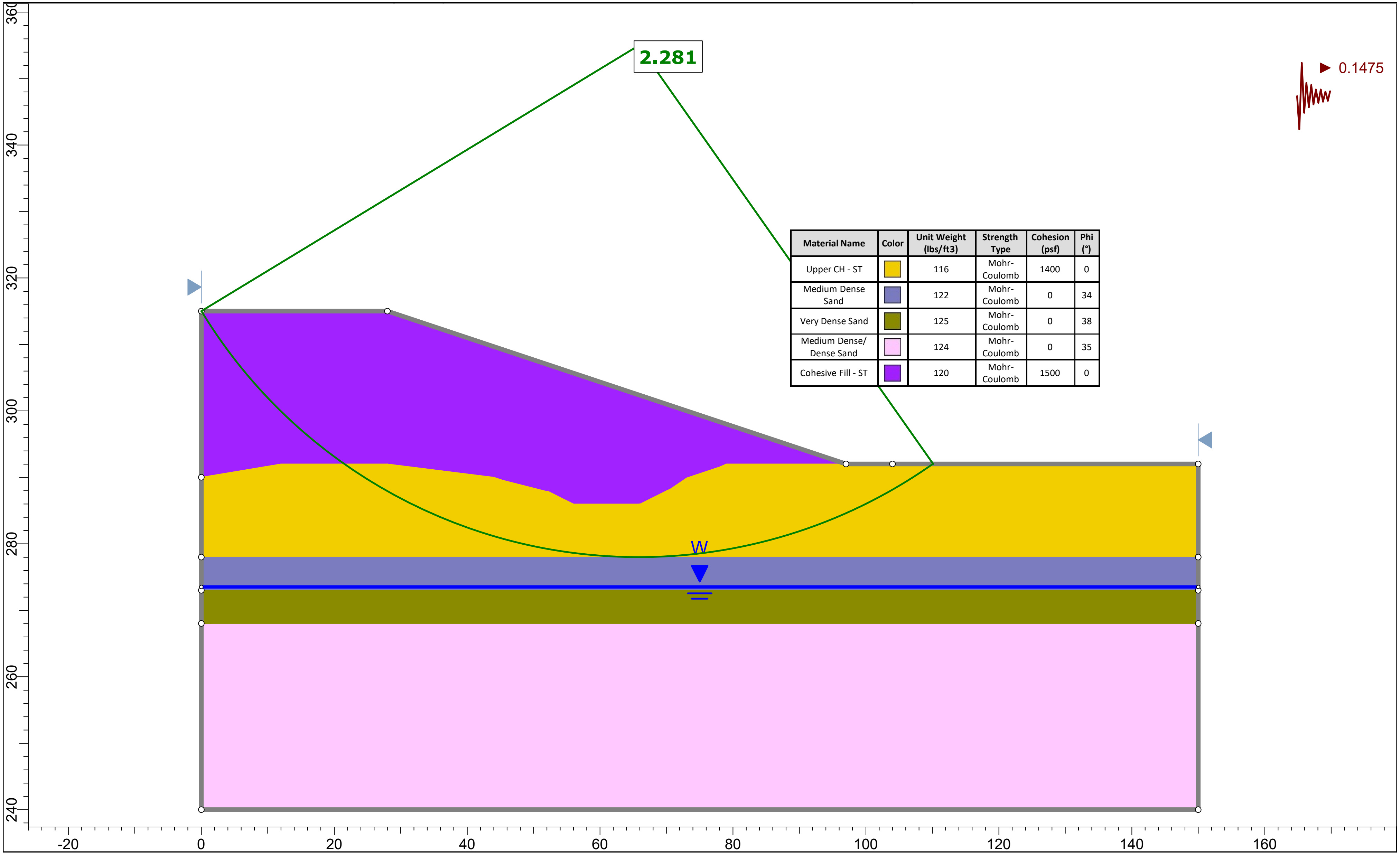


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - LT	🟩	116	Mohr-Coulomb	25	26
Medium Dense Sand	🟦	122	Mohr-Coulomb	0	34
Very Dense Sand	🟫	125	Mohr-Coulomb	0	38
Medium Dense/ Dense Sand	🟪	124	Mohr-Coulomb	0	35
Cohesive Fill - LT	🟩	120	Mohr-Coulomb	40	30

File Name: North Abutment South Side Slope.slmd
Name: STA 214+50.00
Description: Seismic Conditions
Method: Spencer
Date: 5/7/2024

Project Number: J045279.01
Client: ARDOT
Project: ARDOT G017 101172
Corning Bypass (Future I-57) (S)
Clay County, Arkansas

SLIDEINTERPRET 9.034



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Upper CH - ST	🟨	116	Mohr-Coulomb	1400	0
Medium Dense Sand	🟦	122	Mohr-Coulomb	0	34
Very Dense Sand	🟫	125	Mohr-Coulomb	0	38
Medium Dense/ Dense Sand	▨	124	Mohr-Coulomb	0	35
Cohesive Fill - ST	🟪	120	Mohr-Coulomb	1500	0

APPENDIX I – SOIL PARAMETERS FOR SYNTHETIC PROFILES

BENT 1 (SOUTH ABUTMENT; BORING B-1)

APPROXIMATE GROUND SURFACE ELEVATION = EL 292; BOTTOM OF PILE CAP ELEVATION = EL 307.66

ZONE	SOIL TYPES	DEPTH[a] (feet below pile cap)		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]			LIQUEFACTION SHEAR STRENGTH PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	LPILE SOIL MODEL	RESIDUAL COHESION (PSF)	RESIDUAL Φ (DEGREE)
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)					
1	Engineered Fill	0[b]	15.5	120	1,500	--	--	28	0.007	500	Stiff Clay w/o Free Water	1,200	--
2	Stiff Fat Clay	15.5	34	116	1,400	--	--	26	0.007	400	Stiff Clay w/o Free Water	1,100	--
3	Loose Sand with Clay	34	44	122	--	32	--	32	--	20	Sand	--	7
4	Medium Dense Sands	44	65.5	122	--	33	--	33	--	60		--	8
5	Loose Sand	65.5	69	122	--	33	--	33	--	40		N/A[e]	N/A[e]
6	Medium Dense Sand	69	99	122	--	35	--	35	--	60		--	8
7	Medium Dense Sand with Gravel	99	115.5	125	--	36	--	36	--	80		N/A[e]	N/A[e]

Note: Groundwater elevation assumed at El. 273.5, approximately 18.5 feet below ground surface, 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 below pile cap.

[b] Zero depth as measured at top of soil profile below pile cap.

[c] Pounds per cubic inch.

[d] For lateral load analysis only.

[e] Liquefaction not anticipated for these layers.

BENT 2 (CPT-2)

APPROXIMATE AVERAGE GROUND SURFACE ELEVATION = EL 292; PROPOSED BOTTOM OF PILE-SUPPORTED FOOTING ELEVATION = EL 282.56

ZONE	SOIL TYPES	DEPTH[a] (feet from ground surface)		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]			LIQUEFACTION SHEAR STRENGTH PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	LPILE SOIL MODEL	RESIDUAL COHESION (PSF)	RESIDUAL Φ (DEGREE)
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)					
1	Stiff Clay with Sand	0[b]	11.5	116	1,400	--	--	26	0.01	100	Stiff Clay w/o Free Water	1,100	--
2	Loose Sand	11.5	14	120	--	32	--		--	20	Sand		7
3	Very Dense Sand	14	21.5	125	--	38	--		--	125		N/A[e]	N/A[a]
4	Medium Dense Silty Sand	21.5	27	122	--	33	--		--	60		--	7
5	Medium Dense Silty Sand	27	28	122	--	34	--		--	50		N/A[e]	N/A[a]
6	Medium Dense Sand	28	32.5	122	--	34	--		--	60		--	8
7	Medium Dense Sand	32.5	40.5	124	--	35	--		--	60		--	8
8	Dense Sand	40.5	44.5	124	--	35			--	90		N/A[e]	N/A[a]
9	Dense Sand	44.5	70.5	124	--	35			--	90		N/A[e]	N/A[a]
10	Loose Silty Sand	70.5	73.5	122	--	32			--	30		N/A[e]	N/A[a]
11	Dense Sand	73.5	90.5	125	--	36			--	90		N/A[e]	N/A[a]

Note: Groundwater elevation assumed at El. 272, approximately 20 feet below ground surface, 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 from proposed bottom of pile-supported footing at El 282.56, approximately 9.5 feet below top of boring.

[c] Pounds per cubic inch.

[d] For lateral load analysis only.

[e] Liquefaction not anticipated in these layers.

BENT 3 (Boring B-3)

APPROXIMATE GROUND SURFACE ELEVATION = EL 292; PROPOSED BOTTOM OF PILE-SUPPORTED FOOTING ELEVATION = EL 283.57

ZONE	SOIL TYPES	DEPTH[a] (feet from ground surface)		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]			LIQUEFACTION SHEAR STRENGTH PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	LPILE SOIL MODEL	RESIDUAL COHESION (PSF)	RESIDUAL Φ (DEGREE)
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)					
1	Stiff Fat Clay	0[b]	10.5	116	1,400	--	--	26	0.007	400	Stiff Clay w/o Free Water	1,100	--
2	Loose Sand	10.5	15.5	122	--	32	--	32	--	20	Sand	--	7
3	Medium Dense Sand	15.5	20.5	122	--	34	--	34	--	60		N/A[e]	N/A[e]
4	Medium Dense Sand	20.5	25.5	122	--	34	--	34	--	60		--	8
5	Dense Sand	25.5	30.5	125	--	38	--	38	--	125		N/A[e]	N/A[e]
6	Medium Dense Sand	30.5	40.5	124		35		35	--	90		--	8
7	Medium Dense Sand	40.5	50.5	124	--	35	--	35	--	90		N/A[e]	N/A[e]
8	Loose Sand	50.5	70.5	124	--	33	--	33	--	40		N/A[e]	N/A[e]
9	Dense Sands	70.5	92	125	--	36	--	36	--	100		N/A[e]	N/A[e]

Note: Groundwater elevation assumed at El. 273.5, approximately 18.5 feet below ground surface, 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. Proposed bottom of pile-supported footing at El 283.57, approximately 8 feet below top of boring.

[c] Pounds per cubic inch.

[d] For lateral load analysis only.

[e] Liquefaction not anticipated in these layers.

BENT 4 (CPT-4)

APPROXIMATE GROUND SURFACE ELEVATION = EL 292; PROPOSED BOTTOM OF PILE-SUPPORTED FOOTING ELEVATION = EL 278.31

ZONE	SOIL TYPES	DEPTH[a] (feet from ground surface)		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]			LIQUEFACTION SHEAR STRENGTH PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	LPILE SOIL MODEL	RESIDUAL COHESION (PSF)	RESIDUAL Φ (DEGREE)
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)					
1	Dense Sands	0[b]	8.5	118	--	35	--	35	--	90	Sand	N/A[e]	N/A[e]
2	Medium Dense Sands	8.5	12	122	--	34	--	34	--	60		--	7
3	Very Dense Sands	12	17.5	125	--	38	--	38	--	125		N/A[e]	N/A[e]
4	Medium Dense Sands	17.5	23	122	--	34	--	34	--	40		--	7
5	Medium Dense Sands	23	25	122	--	34	--	34	--	50		N/A[e]	N/A[e]
6	Medium Dense Silty Sands	25	30	122	--	34	--	34	--	40		--	8
7	Dense Sands	30	31	124	--	35	--	35	--	70		N/A[e]	N/A[e]
8	Medium Dense Sands	31	36	122	--	35	--	35	--	60		--	8
9	Dense Sands	36	50	122	--	35	--	35	--	70		N/A[e]	N/A[e]
10	Medium Dense Sands	50	58	124	--	34	--	34	--	40		N/A[e]	N/A[e]
11	Loose Sands	58	63	124	--	33	--	33	--	30		N/A[e]	N/A[e]
12	Dense Sands	63	70	124	--	36	--	36	--	100		N/A[e]	N/A[e]
13	Dense Sands	70	86	124	--	35	--	35	--	80		N/A[e]	N/A[e]

Note: Groundwater elevation assumed at El. 272, approximately 20 feet below ground surface, 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. Proposed bottom of pile-supported footing at El 278.31, approximately 14 feet below top of boring.

[c] Pounds per cubic inch.

[d] For lateral load analysis only.

[e] Liquefaction not anticipated below 50 feet.

BENT 5 (NORTH ABUTMENT; BORING B-5)

APPROXIMATE GROUND SURFACE ELEVATION = EL 292; BOTTOM OF PILE CAP ELEVATION = EL 306.57

ZONE	SOIL TYPES	DEPTH[a] (feet below pile cap)		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]			LIQUEFACTION SHEAR STRENGTH PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	LPILE SOIL MODEL	RESIDUAL COHESION (PSF)	RESIDUAL Φ (DEGREE)
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)					
1	Engineered Fill	0[b]	14.5	120	1,500	--	--	28	0.007	500	Stiff Clay w/o Free Water	1,200	--
2	Stiff Fat Clay	14.5	28	116	1,100	--	--	26	0.007	400	Stiff Clay w/o Free Water	900	--
3	Medium Dense Sands	28	33	122	--	34	--	34	--	60	Sand	N/A[e]	N/A[e]
4	Dense Sands	33	38	125	--	38	--	38	--	125		N/A[e]	N/A[e]
5	Medium Dense Sands	38	48	124	--	34	--	34	--	80		N/A[e]	N/A[e]
6	Loose Sands	48	63	124	--	33	--	33	--	30		--	8
7	Medium Dense Sands	63	68	124	--	35	--	35	--	90		N/A[e]	N/A[e]
8	Medium Dense Sands	68	83	124	--	34	--	34	--	50		N/A[e]	N/A[e]
9	Dense Sands	83	114.5	125	--	36	--	36	--	125		N/A[e]	N/A[e]

Note: Groundwater elevation assumed at El. 273.5, approximately 18.5 feet below ground surface, 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 below pile cap.

[b] Zero depth as measured at top of soil profile below pile cap.

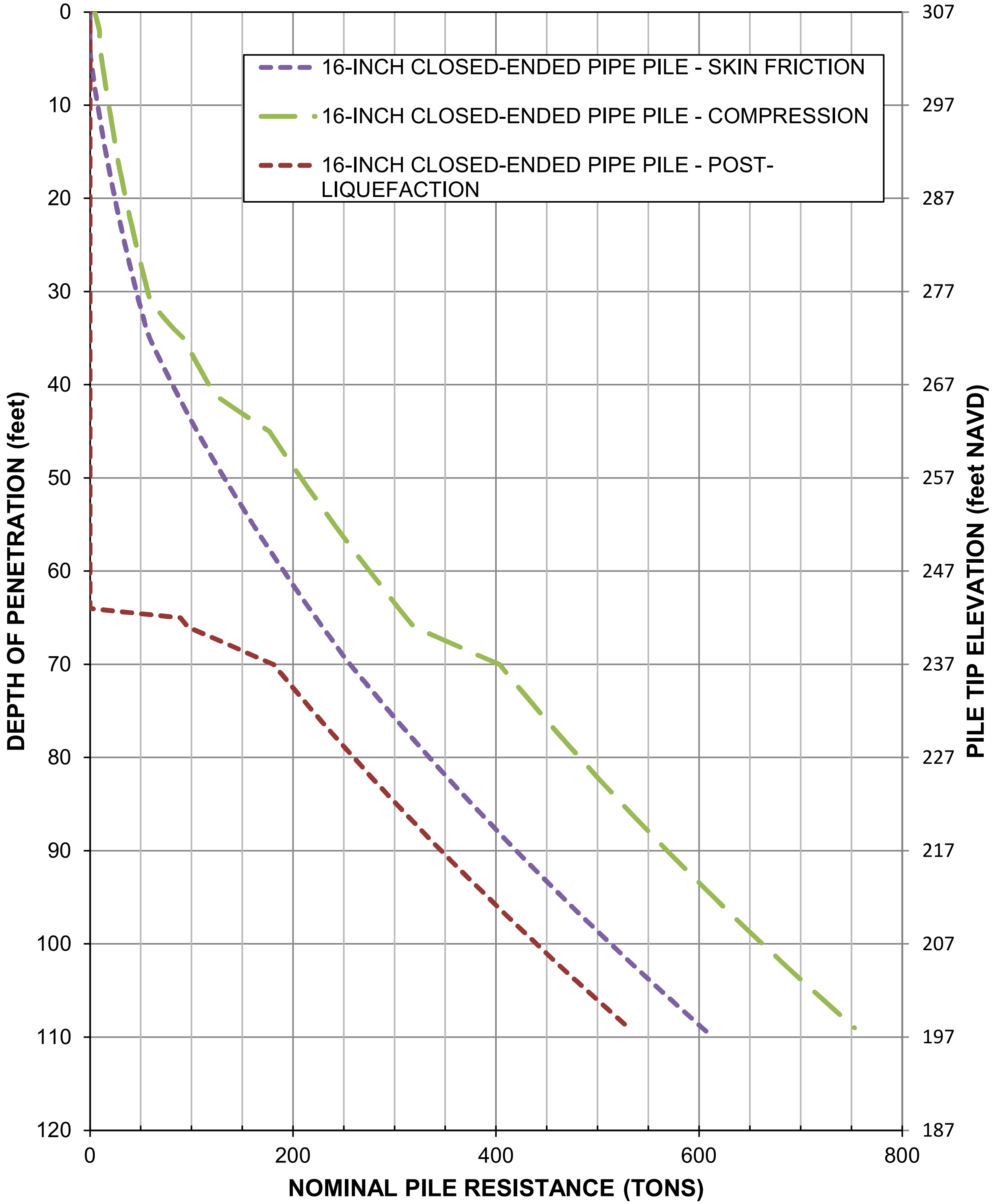
[c] Pounds per cubic inch.

[d] For lateral load analysis only.

[e] Liquefaction not anticipated below 50 feet.

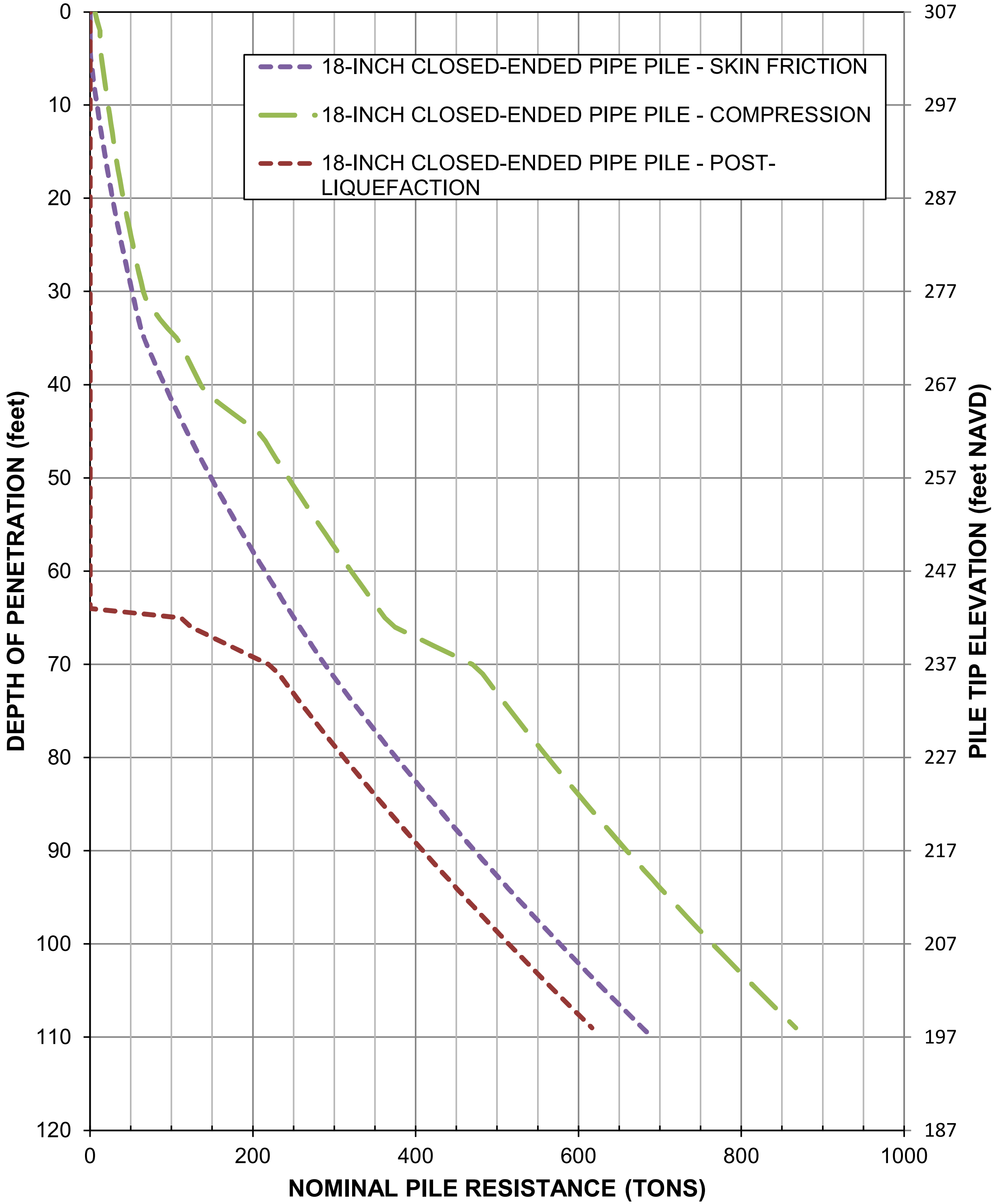
APPENDIX J – NOMINAL RESISTANCE CURVES

NOMINAL RESISTANCE CURVES
16-INCH CLOSED-ENDED PIPE PILE
ARDOT 101172 CORNING BYPASS
(FUTURE I-57) (S), CLAY COUNTY - BENT 1

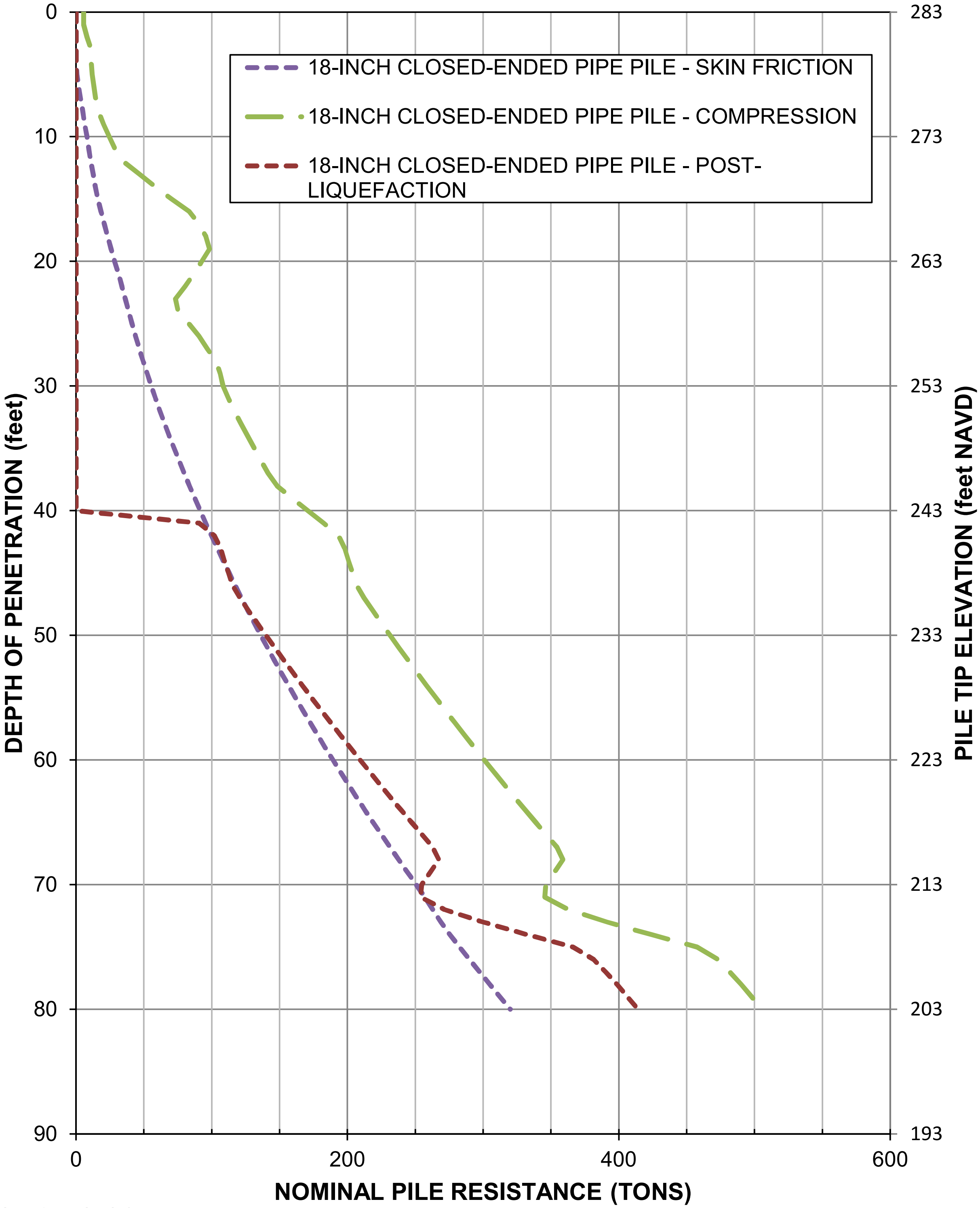


GEOTECHNOLOGY
PROJECT NUMBER J045279.01

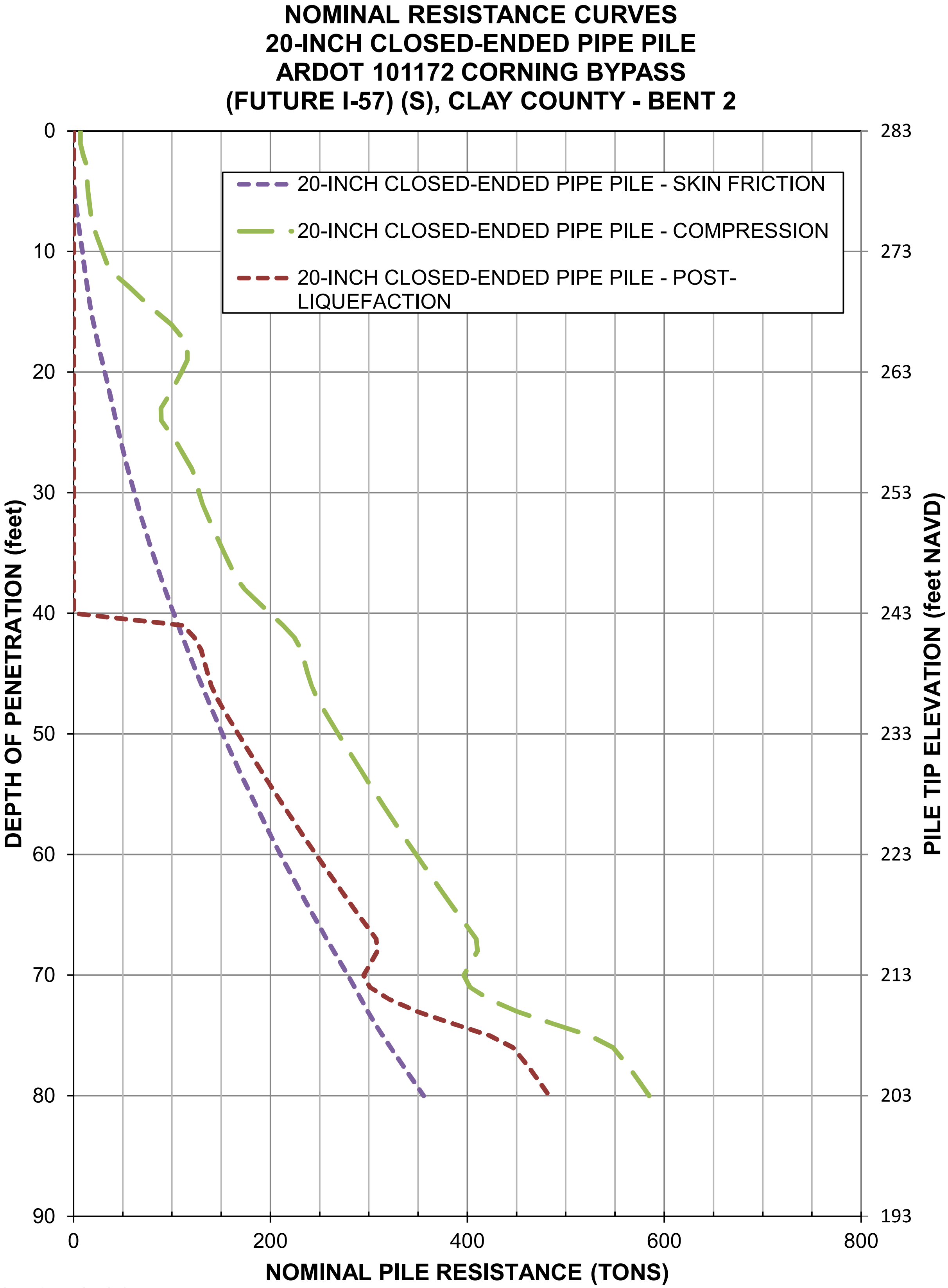
NOMINAL RESISTANCE CURVES
18-INCH CLOSED-ENDED PIPE PILE
ARDOT 101172 CORNING BYPASS
(FUTURE I-57) (S), CLAY COUNTY - BENT 1



NOMINAL RESISTANCE CURVES
18-INCH CLOSED-ENDED PIPE PILE
ARDOT 101172 CORNING BYPASS
(FUTURE I-57) (S), CLAY COUNTY - BENT 2

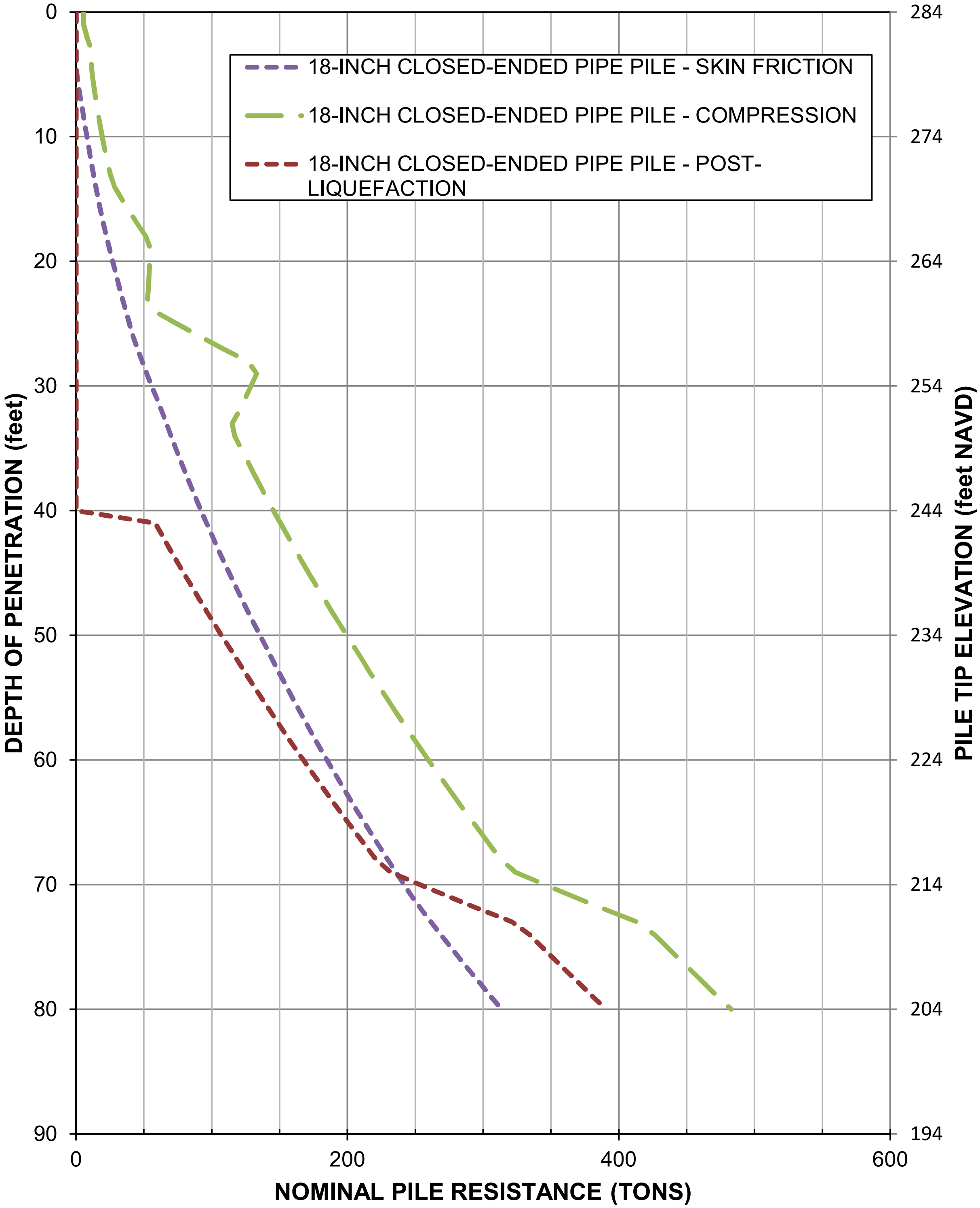


GEOTECHNOLOGY
PROJECT NUMBER J045279.01



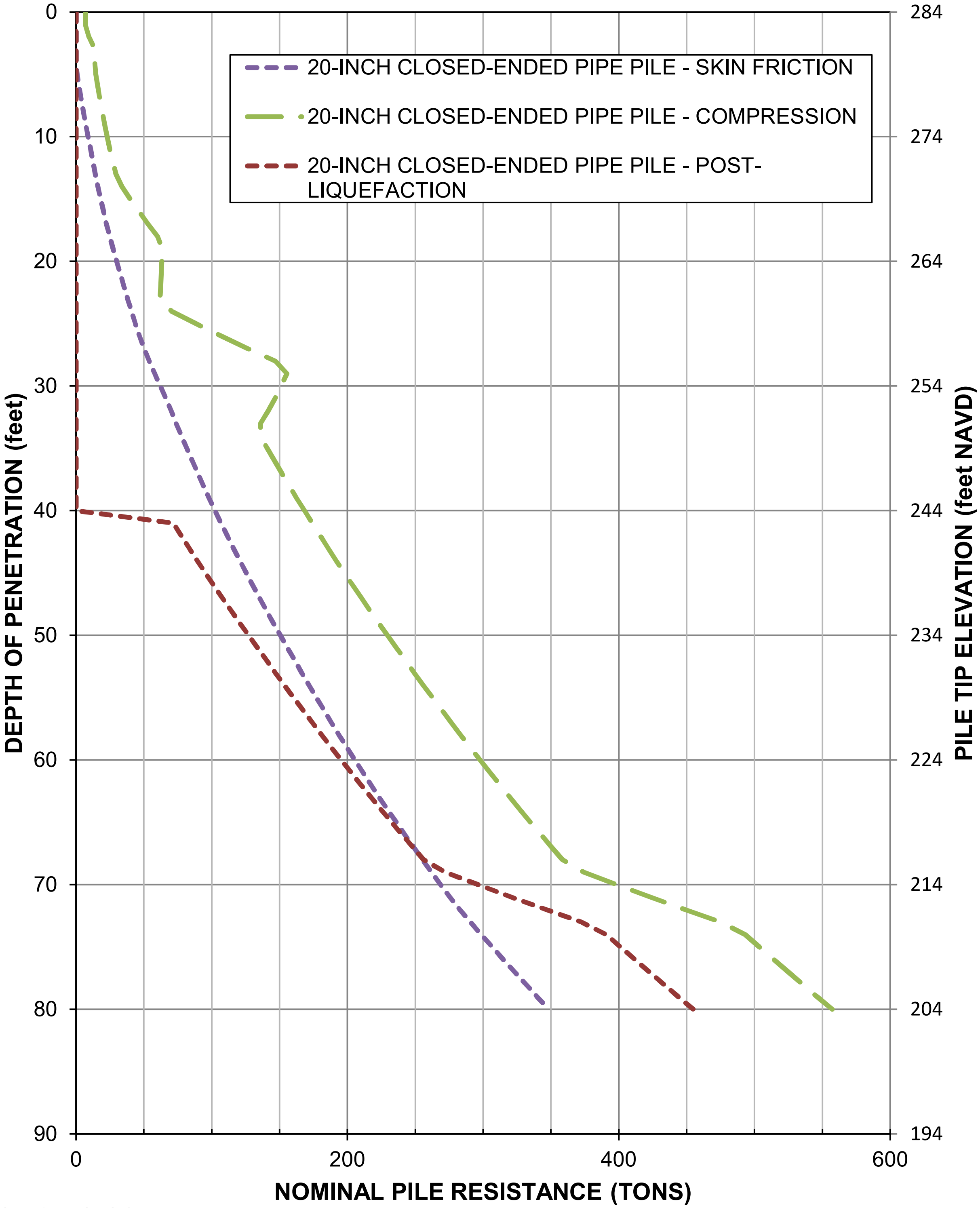
GEOTECHNOLOGY
PROJECT NUMBER J045279.01

NOMINAL RESISTANCE CURVES 18-INCH CLOSED-ENDED PIPE PILE ARDOT 101172 CORNING BYPASS (FUTURE I-57) (S), CLAY COUNTY - BENT 3



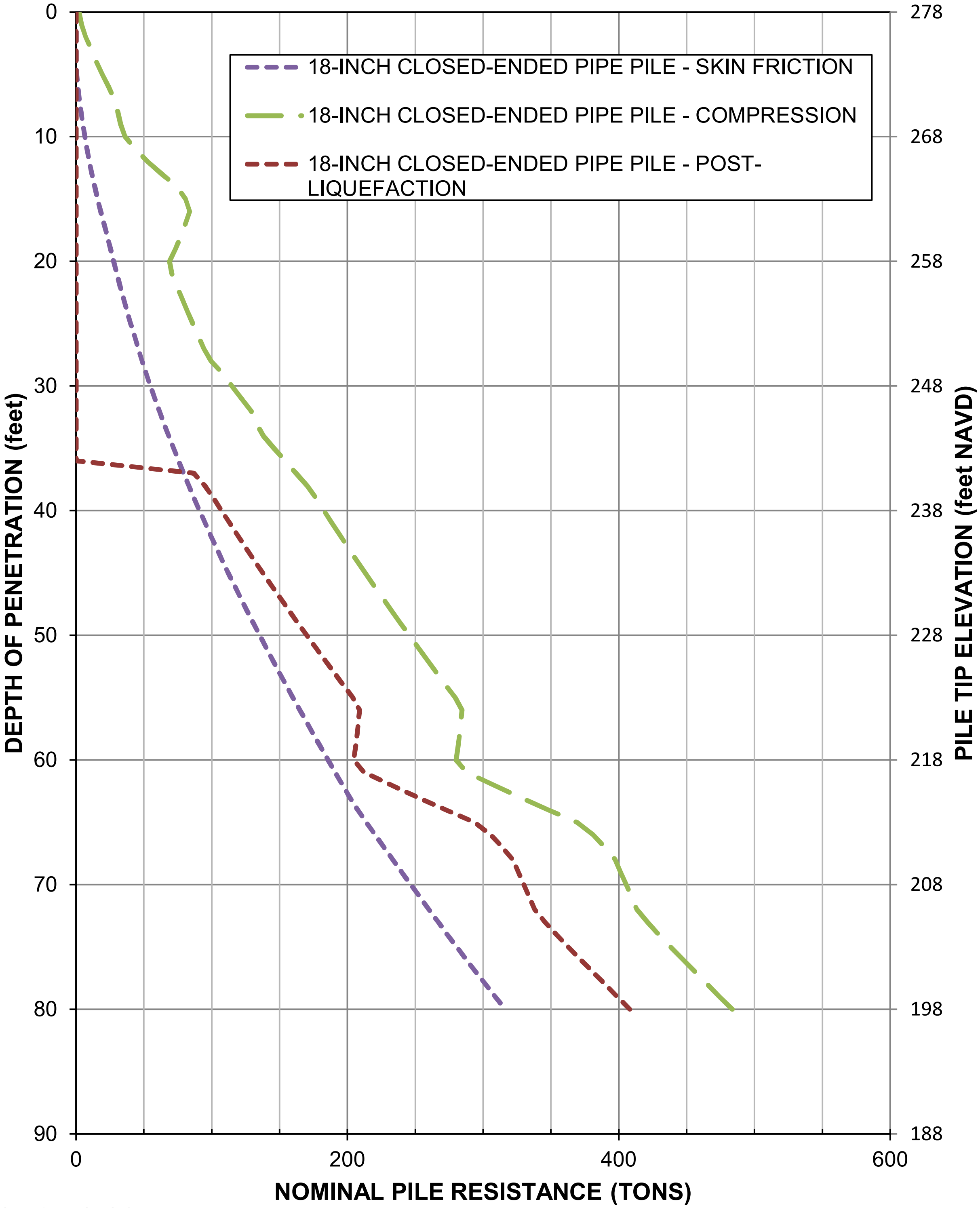
GEOTECHNOLOGY
PROJECT NUMBER J045279.01

NOMINAL RESISTANCE CURVES
20-INCH CLOSED-ENDED PIPE PILE
ARDOT 101172 CORNING BYPASS
(FUTURE I-57) (S), CLAY COUNTY - BENT 3



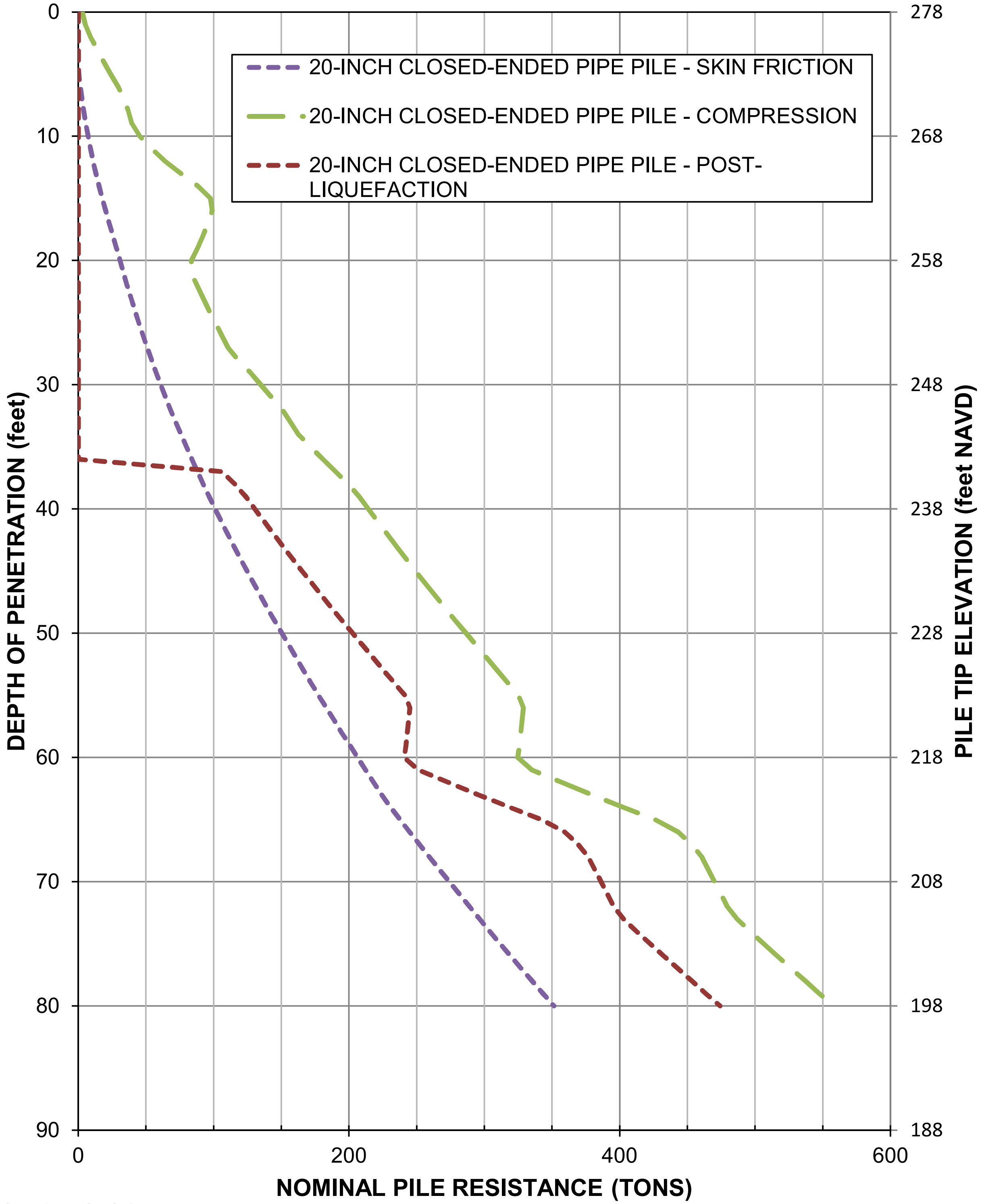
GEOTECHNOLOGY
PROJECT NUMBER J045279.01

NOMINAL RESISTANCE CURVES
18-INCH CLOSED-ENDED PIPE PILE
ARDOT 101172 CORNING BYPASS
(FUTURE I-57) (S), CLAY COUNTY - BENT 4



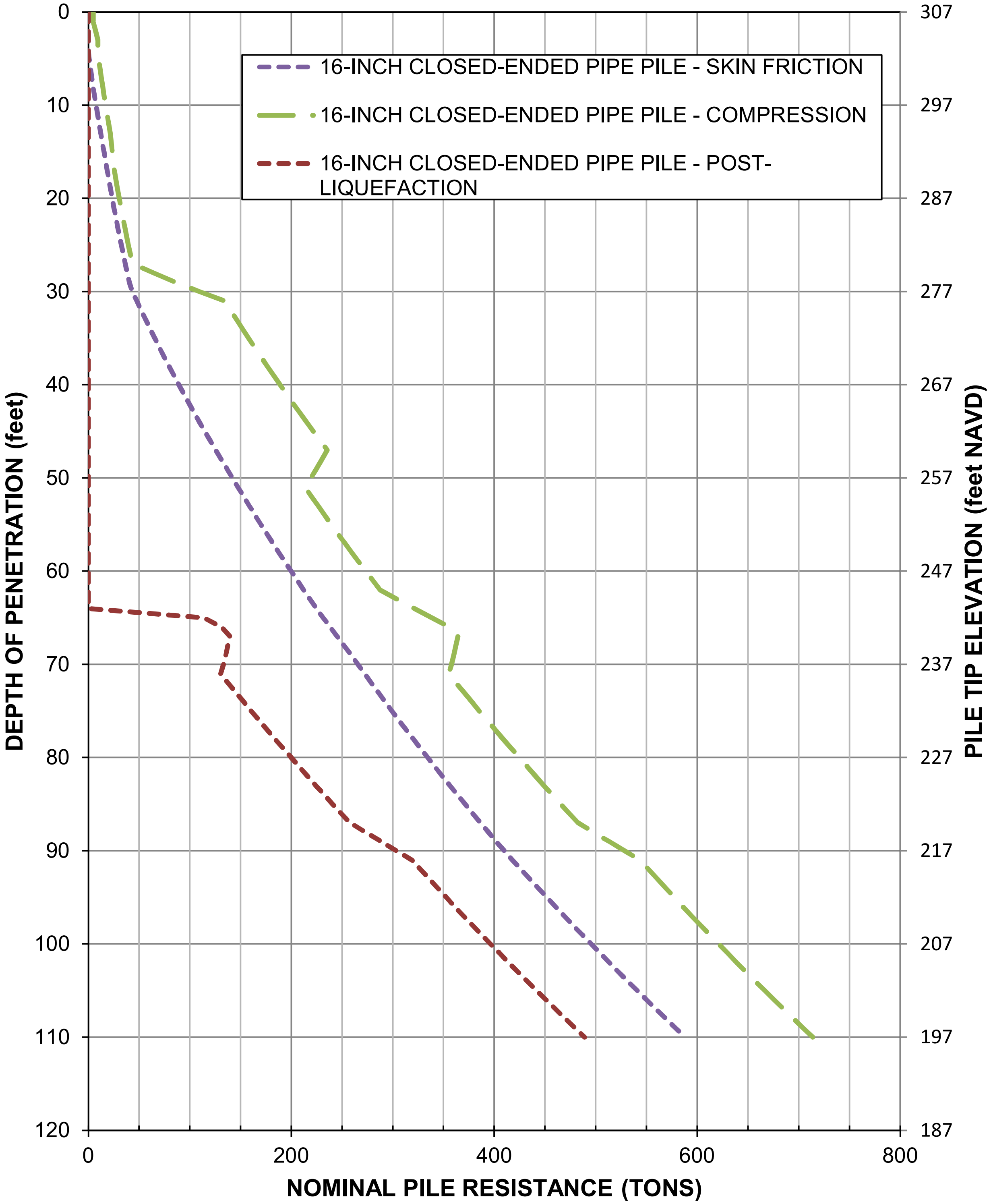
GEOTECHNOLOGY
PROJECT NUMBER J045279.01

NOMINAL RESISTANCE CURVES
20-INCH CLOSED-ENDED PIPE PILE
ARDOT 101172 CORNING BYPASS
(FUTURE I-57) (S), CLAY COUNTY - BENT 4



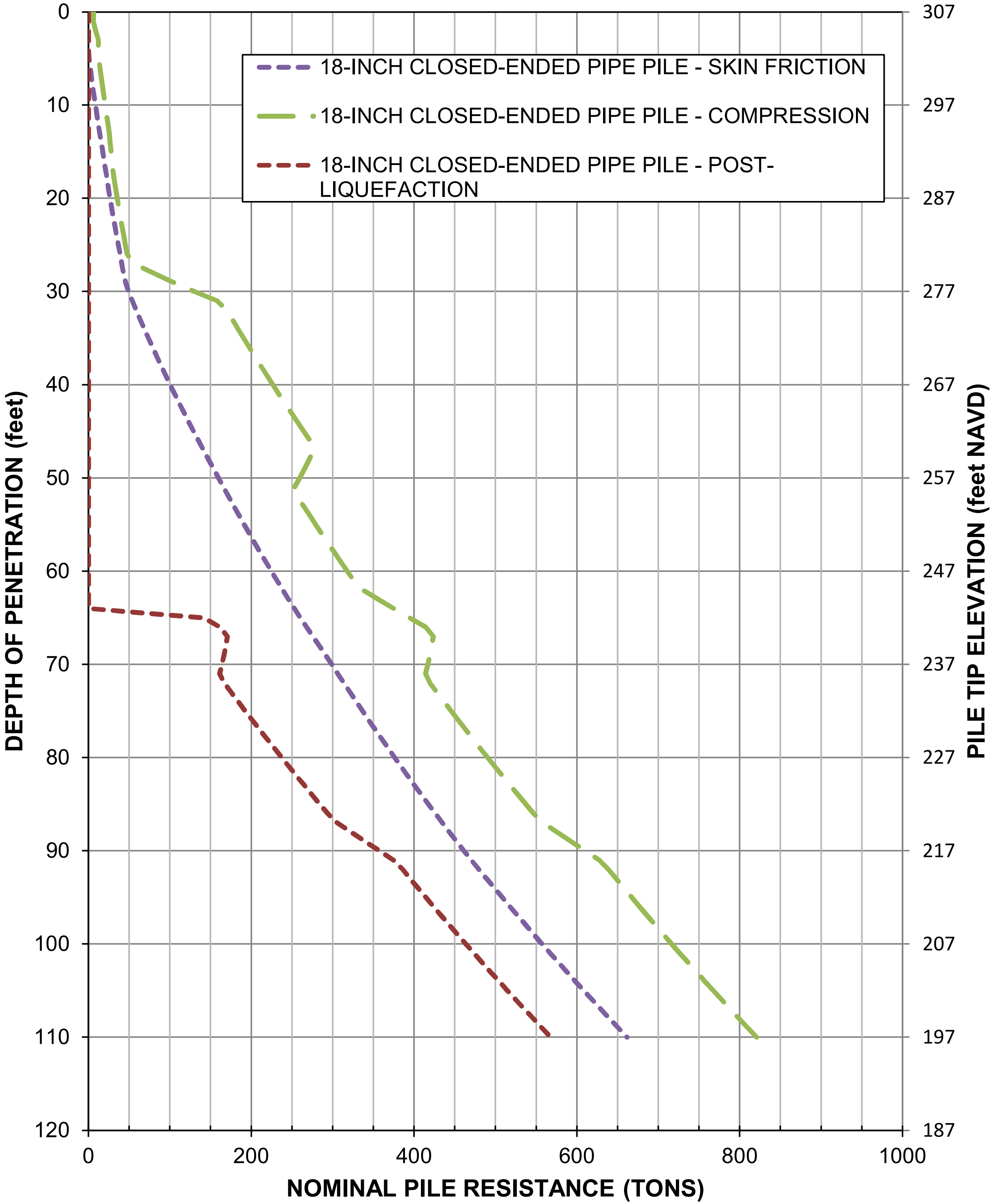
GEOTECHNOLOGY
PROJECT NUMBER J045279.01

NOMINAL RESISTANCE CURVES
16-INCH CLOSED-ENDED PIPE PILE
ARDOT 101172 CORNING BYPASS
(FUTURE I-57) (S), CLAY COUNTY - BENT 5



GEOTECHNOLOGY
PROJECT NUMBER J045279.01

NOMINAL RESISTANCE CURVES
18-INCH CLOSED-ENDED PIPE PILE
ARDOT 101172 CORNING BYPASS
(FUTURE I-57) (S), CLAY COUNTY - BENT 5



GEOTECHNOLOGY
PROJECT NUMBER J045279.01

Site-Specific Seismic Study
Corning Bypass (Future I-57)
Clay County, Arkansas

By

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

March 12, 2024

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Site-Specific Seismic Study
Corning Bypass (Future I-57)
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 36.437361°N and 90.604030°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 the 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 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 (A UES Company) 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. A copy of the boring used for this study is provided in Appendix B.

Table 1. Shear-Wave Velocities Measured.

Depth1 (ft)	**Depth2 (ft)**	**V_s (ft/sec)**
1.67	4.95	513.48
4.95	8.23	513.48
8.23	11.55	513.48
11.55	14.83	916.56
14.83	18.14	1025.36
18.14	21.39	500.13
21.39	24.60	574.00
24.60	27.85	656.20
27.85	31.09	842.93
31.09	34.37	658.98
34.37	37.82	784.35
37.82	41.10	808.88
41.10	44.31	742.30
44.31	47.66	657.15
47.66	50.94	572.43
50.94	54.19	782.08
54.19	57.40	774.87
57.40	60.84	873.20
60.84	64.09	727.34
64.09	67.37	792.19
67.37	70.62	777.06
70.62	73.93	1161.71
73.93	77.15	879.76
77.15	80.36	842.96
80.36	83.71	969.47
83.71	86.95	930.37
86.95	90.20	899.54
90.20	93.55	1083.48
93.55	96.76	921.48
96.76	100.14	984.00

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.

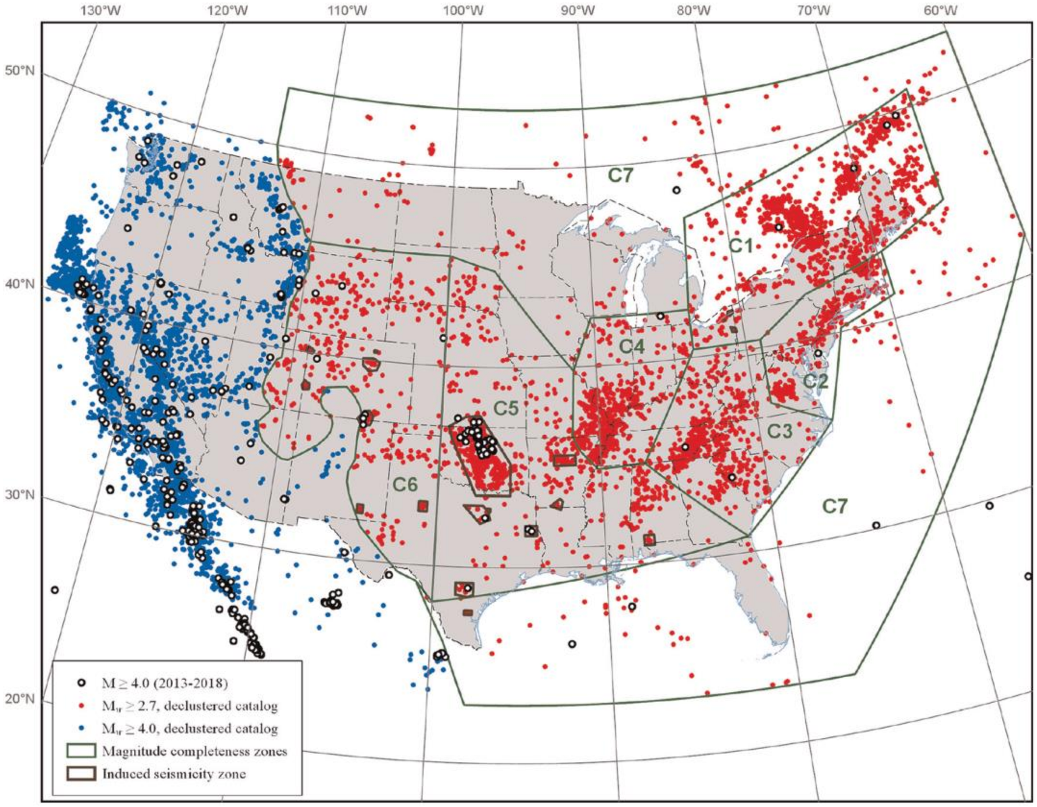


Figure 1. The 2018 NSHM Declustered Catalog for Central and Eastern United States (red) and Western United States (blue).

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

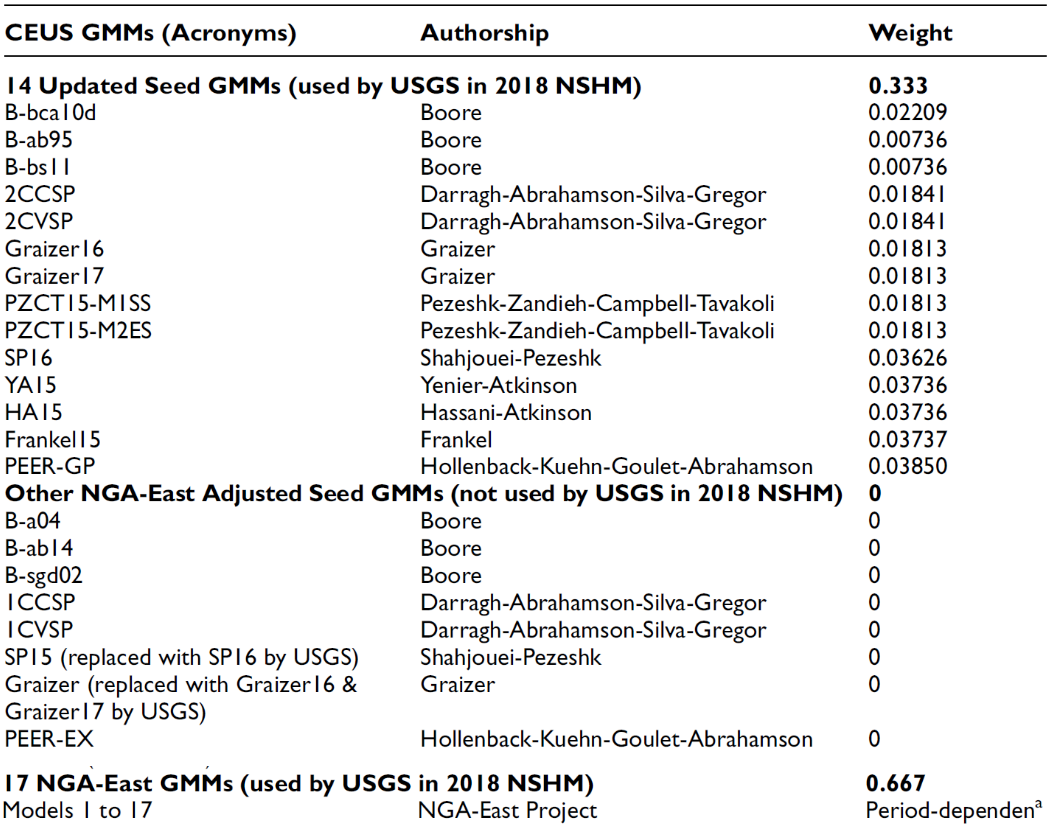
In general, the characteristics of the fault source, such as distance, type, magnitude, and site conditions, are used to estimate the magnitude of an earthquake parameter (spectral acceleration, peak ground acceleration, etc.) via ground-motion models (GMMs) or ground-motion prediction equations (GMPEs), 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).[*Source Rezaeian et al.* (2021)].



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
17 NGA-East GMMs (used by USGS in 2018 NSHM)		**0.667**
Models 1 to 17	NGA-East Project	Period-dependen[a]

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 with 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 744 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 with 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	1.320
F_v	2.180
F_{PGA}	1.189
S_S	0.599
S_1	0.155
S_{DS}	0.791
S_{D1}	0.338
PGA	0.311
A_s	0.370

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.

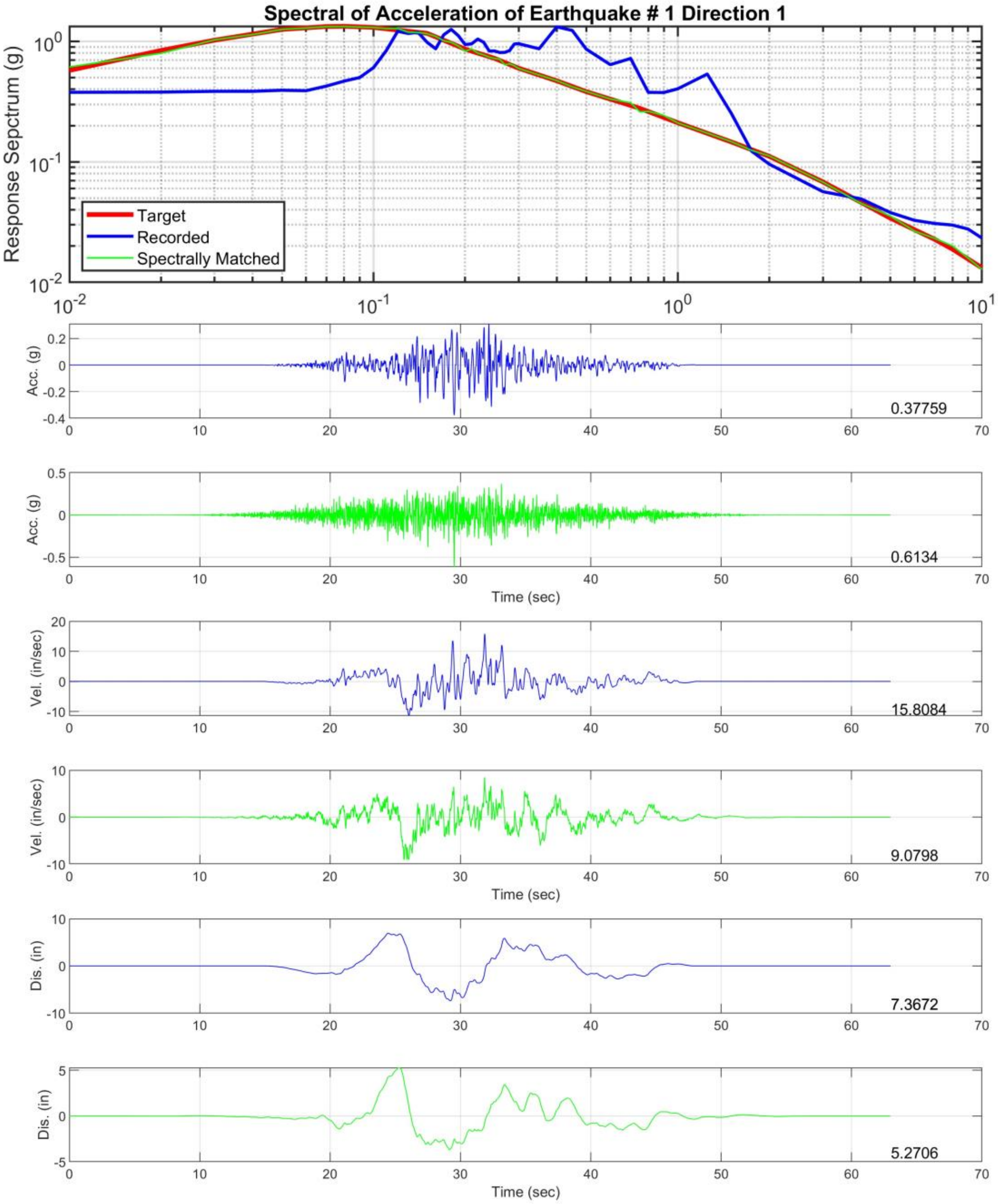


Figure 2. Time Histories Before and After the Spectral Matching Process for Earthquake #1. The numbers Shown in the Bottom right of Each Figure Represent the Absolute Maximum Value of the Graph.

Table 4. Deaggregation.

Mean and Mode Deaggregation Parameter at 1,033 Years					
Mean			Mode		
Period	M	R (km)	Period	M	R (km)
PGA	7.09	62.48	PGA	7.5	87.86
0.01	7.09	62.31	0.01	7.76	84.25
0.02	7.05	61.63	0.02	7.76	84.26
0.03	7.06	62.09	0.03	7.76	84.26
0.05	7.12	64.09	0.05	7.76	84.27
0.08	7.16	65.84	0.08	7.52	86.31
0.10	7.21	67.62	0.10	7.52	86.32
0.15	7.29	70.11	0.15	7.52	86.97
0.20	7.33	71.55	0.20	7.52	86.99
0.25	7.38	73.08	0.25	7.52	87.00
0.30	7.40	73.89	0.30	7.52	87.01
0.40	7.43	75.21	0.40	7.52	87.02
0.50	7.45	76.15	0.50	7.52	87.03
0.75	7.50	77.98	0.75	7.52	87.04
1.00	7.52	78.98	1.00	7.52	87.04
1.50	7.56	80.02	1.50	7.51	87.46
2.00	7.59	80.66	2.00	7.76	84.35
3.00	7.62	81.00	3.00	7.76	84.36
4.00	7.63	81.28	4.00	7.76	84.36
5.00	7.64	81.41	5.00	7.51	87.10
7.50	7.66	81.44	7.50	7.50	86.97
10.00	7.67	81.52	10.00	7.50	86.97

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 abovementioned procedure, 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 so that it is not 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	0.295
0.030	0.345
0.040	0.378
0.050	0.411
0.070	0.477
0.100	0.527
0.150	0.533
0.200	0.575
0.250	0.582
0.300	0.609
0.400	0.553
0.500	0.527
0.750	0.470
1.000	0.388
1.500	0.336
2.000	0.273
3.000	0.137
4.000	0.073
5.000	0.051
7.500	0.030
10.000	0.035

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

PARAMETER	DESIGN ACCELERATION PARAMETERS (g)
S_{DS}	0.575
S_{D1}	0.388
S_{MS}	0.575
S_{M1}	0.388
MCE_G	0.295

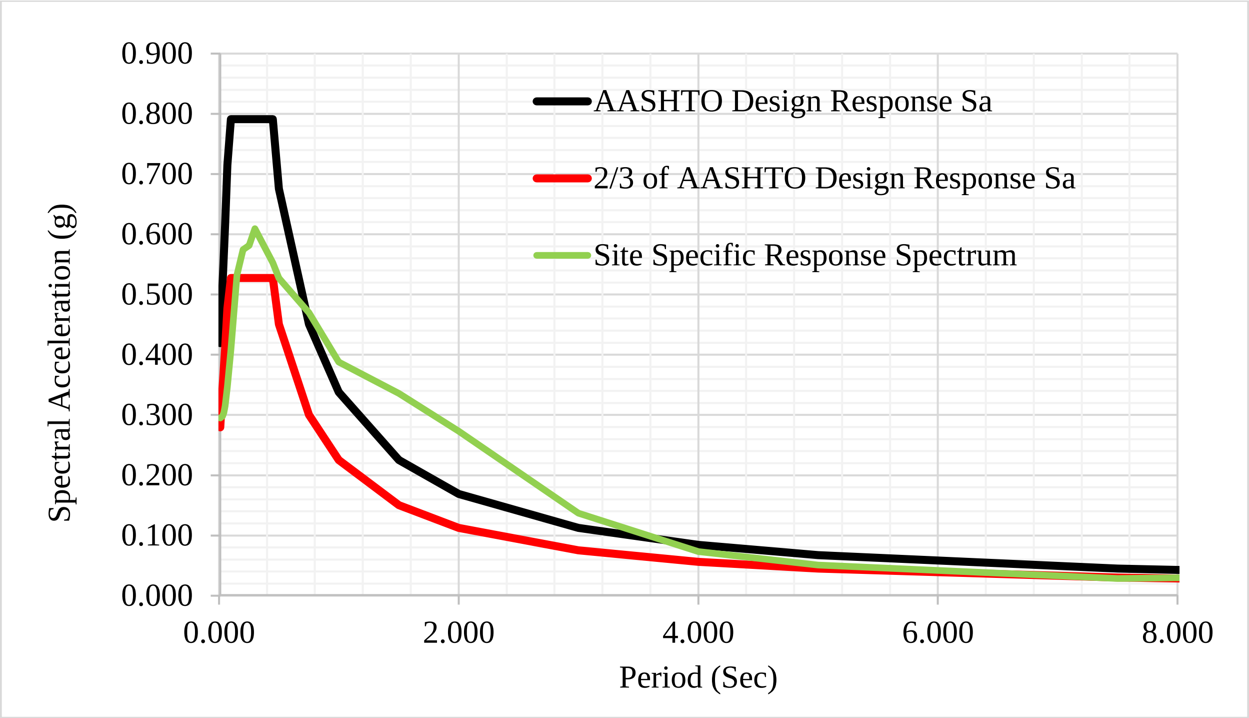


Figure 3. Site-Specific Design Response Spectrum, AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions Design Response Spectrum.

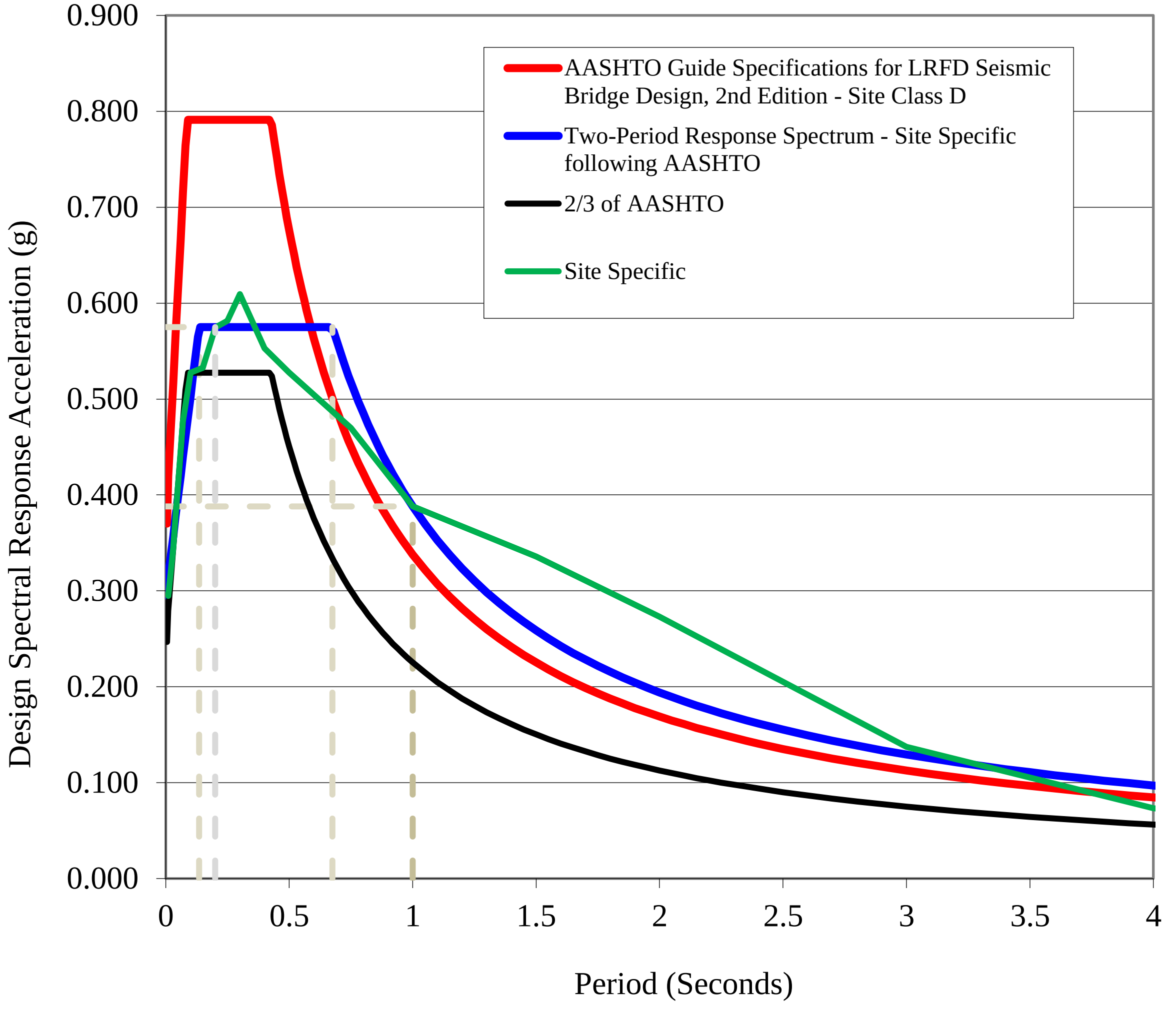


Figure 4. Design Response Spectrum based on AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions, 2/3 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions 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

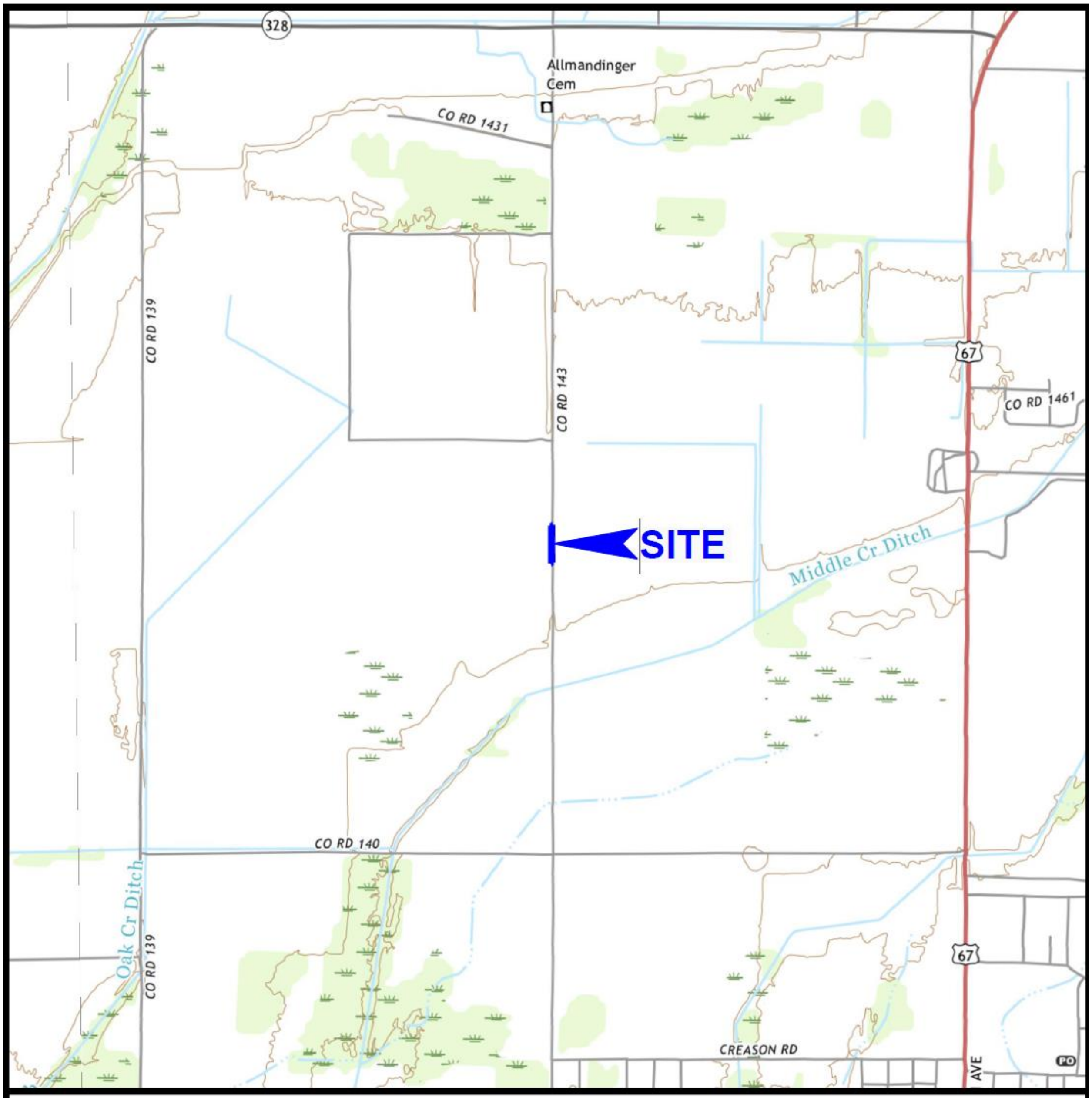


Figure A.1. The Location of the Study Site.

APPENDIX B. Boring Log

Surface Elevation: 292

Datum NAVD88

Completion Date: 2/22/24

DESCRIPTION OF MATERIAL

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

9 inches of sand and gravel

Medium stiff to stiff, gray and brown to gray, silty, FAT CLAY - CH
- trace gravel
- trace sand
- trace gravel and sand
- little sand
- 3-inch gray clayey sand seam

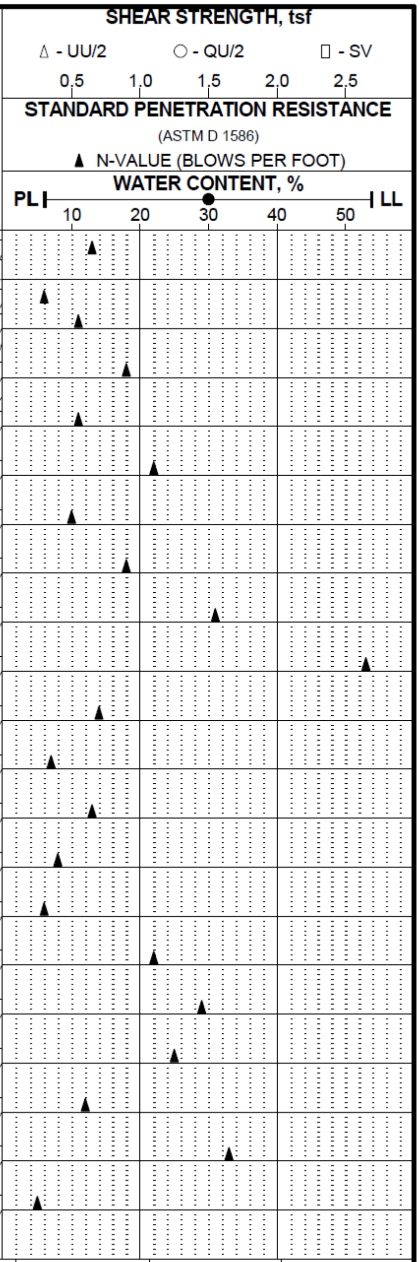
Loose to very dense, gray and brown to gray SAND - SP ∇
- trace gravel
- trace gravel
- 2.5-inch black lignite seam
- some gravel

Medium dense, gray and brown SAND and gravel - SP
- trace lignite

Loose to dense, gray SAND - SP
- trace gravel
- trace gravel and clay
- little gravel, trace clay
- trace gravel and lignite
- trace gravel
- 1-inch gravel and sand seam

Boring terminated at 100 feet.

DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
3-6-7	SS1
	ST2
2-2-4	SS3
1-5-6	SS4
	ST5
4-7-11	SS6
	ST7
4-6-5	SS8
6-11-11	SS9
3-5-5	SS10
4-7-11	SS11
6-11-20	SS12
13-26-27	SS13
6-8-6	SS14
4-4-3	SS15
8-7-6	SS16
4-4-4	SS17
2-3-3	SS18
6-12-10	SS19
9-14-15	SS20
6-11-14	SS21
7-7-5	SS22
6-17-16	SS23
4-3-2	SS24



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

LOG OF BORING 2020 JDM - ELEVATIONS J045279.01.GPJ GTINC 0638301.GPJ 2/27/24

GROUNDWATER DATA

ENCOUNTERED AT 18.5 FEET ∇

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 20 FEET
WEC DRILLER SAS LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 85 %

REMARKS:

Drawn by: SAS	Checked by:	App'vd. by:
Date: 2/26/24	Date:	Date:

GEOTECHNOLOGY
A UES Company

Corning Bypass (Future I-57) (S)
Clay County, Arkansas

LOG OF BORING: B-1

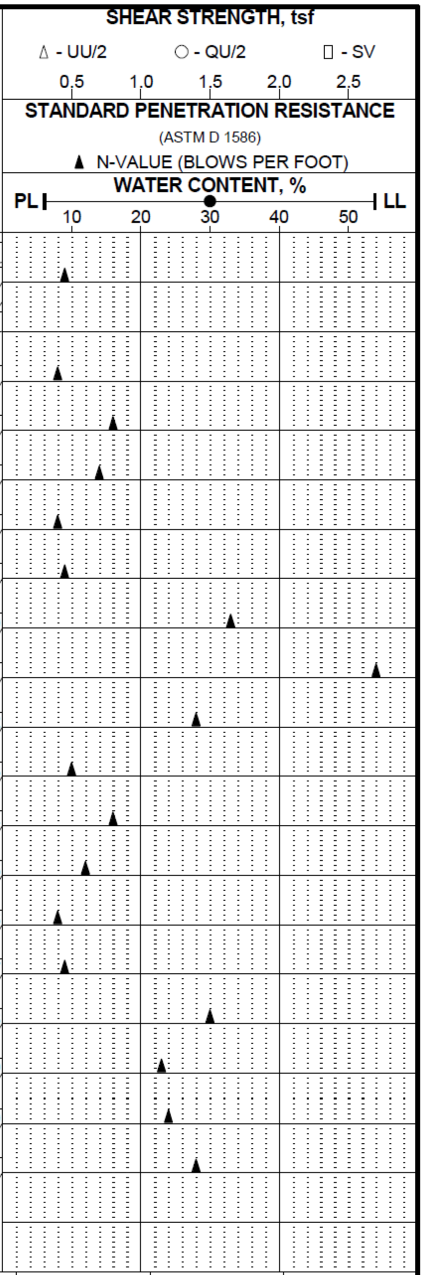
Project No. J045279.01

Surface Elevation: 292

Datum NAVD88

Completion Date: 2/23/24

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		9 inches of sand and gravel		ST1
5	287	Stiff, gray and brown to gray, silty, FAT CLAY - CH little organics, trace sand	2-4-5	SS2 ST3 ST4
10	282			
15	277	Medium dense, gray, CLAYEY SAND - SC	1-3-5	SS5
20	272	Medium dense to very dense, gray SAND - SP	3-8-8	SS6
25	267		3-8-6	SS7
30	262		2-4-4	SS8
35	257		3-4-5	SS9
40	252	trace lignite	4-11-22	SS10
45	247		9-24-30	SS11
50	242		8-17-11	SS12
55	237	some gravel	3-5-5	SS13
60	232	trace gravel	4-7-9	SS14
65	227	trace gravel	5-8-4	SS15
70	222	Medium dense, gray SAND and gravel - SP	3-4-4	SS16
75	217		3-4-5	SS17
80	212	Dense, gray SAND - SP trace gravel	6-12-18	SS18
85	207		5-9-14	SS19
90	202	trace gravel	5-10-14	SS20
95	197	3-inch lignite seam Boring terminated at 95 feet due to borehole collapse.	5-11-17	SS21
100	192			



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

LOG OF BORING 2020 JDM - ELEVATIONS J045279.01.GPJ GTINC 0638301.GPJ 2/27/24

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 10 FEET

WEC DRILLER SAS LOGGER

CME750X DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 85 %

REMARKS: Borehole collapsed at approxiamtely 95 feet.

Drawn by: SAS	Checked by:	App'vd. by:
Date: 2/26/24	Date:	Date:

GEOTECHNOLOGY
A UES Company

Corning Bypass (Future I-57) (S)
Clay County, Arkansas

LOG OF BORING: B-3

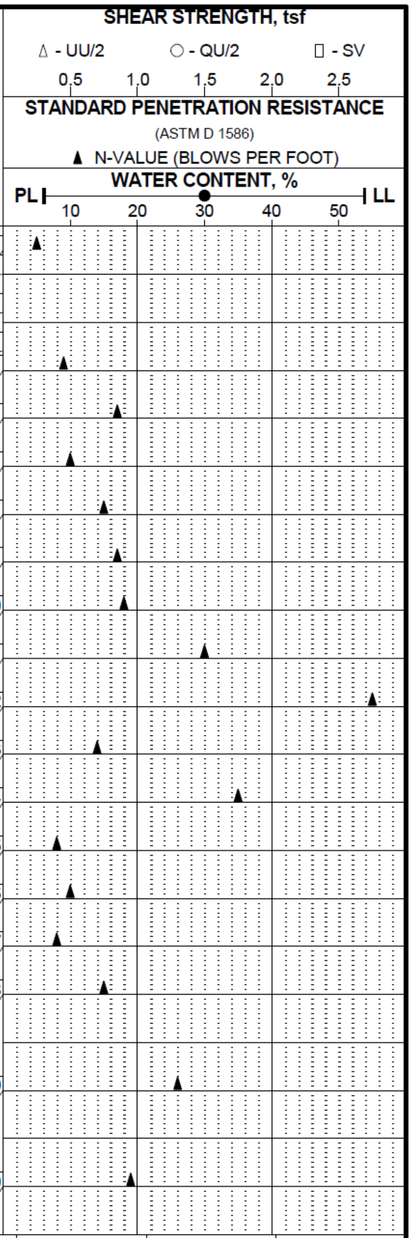
Project No. J045279.01

Surface Elevation: 292

Datum NAVD88

Completion Date: 2/23/24

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		9 inches of sand and gravel		
		Medium stiff, gray and brown to gray, silty, FAT CLAY, trace sand - CH	1-3-2	SS1
				ST2
5	287			
				ST3
10	282			
				ST4
		Medium dense, gray, CLAYEY SAND - SC	4-5-4	SS5
15	277			
		Medium dense to very dense, gray SAND - SP trace lignite	3-7-10	SS6
20	272			
		⊻		
		trace lignite	1-4-6	SS7
25	267			
			4-8-7	SS8
30	262			
		trace lignite	4-10-7	SS9
35	257			
			2-7-11	SS10
40	252			
		trace lignite	5-15-15	SS11
45	247			
			7-26-29	SS12
50	242			
		trace gravel	4-8-6	SS13
55	237			
		trace gravel	4-14-21	SS14
60	232			
		trace gravel and clay	3-6-2	SS15
65	227			
		Medium dense, gray SAND and gravel - SP	4-6-4	SS16
70	222			
		Medium dense to dense, gray SAND - SP little gravel	2-3-5	SS17
75	217			
			2-5-10	SS18
80	212			
85	207			
			4-12-14	SS19
90	202			
95	197			
			3-9-10	SS20
100	192	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

ENCOUNTERED AT 23.5 FEET ⊻

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
WEC DRILLER SAS LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 85 %

REMARKS:

Drawn by: SAS	Checked by:	App'vd. by:
Date: 2/26/24	Date:	Date:

GEOTECHNOLOGY
A UES Company

Corning Bypass (Future I-57) (S)
Clay County, Arkansas

LOG OF BORING: B-5

Project No. J045279.01

LOG OF BORING 2020 JDM - ELEVATIONS J045279.01.GPJ GTINC 0638301.GPJ 2/27/24