ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 061615

FEDERAL AID PROJECT NO. NHPP-0059(16)

LA GRUE BAYOU, WOLF ISLAND SLASH & HONEY CREEK STRS & APPRS

STATE HIGHWAY 63 & 33 **SECTION** 11 & 5

IN PRAIRIE **COUNTY**

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GEOTECHNICAL EXPLORATION

ROUTE 63 SECTION 11 STRUCTURES AND APPROACHES
(S) – BRIDGE NO. 01858
PRAIRIE COUNTY, ARKANSAS

ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT NO. 061615
FEDERAL AID PROJECT NO. 9990

Prepared for:

GARVER USA
NORTH LITTLE ROCK, ARKANSAS

Prepared by:

GEOTECHNOLOGY, LLC
MEMPHIS, TENNESSEE

Date:

MARCH 23, 2023

Geotechnology Project No.:

J034561.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

St. Louis, MO | Erlanger, KY | Memphis, TN | Overland Park, KS | Cincinnati, OH | Fairview Heights, IL
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GEOTECHNOLOGY
A Universal Engineering Sciences Company

March 23, 2023

Mr. John Ruddell, P.E., S.E.
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Re: Geotechnical Exploration
ARDOT 061615
Route 63 Section 11 Structures and Approaches (S) – Bridge No. 01858
Prairie County, Arkansas
Geotechnology Project No. J034561.01

Dear Mr. Ruddell:

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

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ROUTE 63 SECTION 11 STRUCTURES AND APPROACHES (S) – BRIDGE NO. 01858

Prairie County, Arkansas

March 23, 2023 | Geotechnology Project No. J034561.01

1.0 SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design, construction, and other related features for the proposed approach improvements and bridge replacement over Wolf Island Slash along Highway 63 in Prairie County, Arkansas. The referenced features include demolition of the existing bridge and construction of a new bridge (Structure No. 01858). It is our understanding the anticipated foundation type for support of the new bridge is driven, closed-ended, pipe piles. The existing bridge approaches will be modified to facilitate traffic flow over the new bridge. A general overview of the project extents is shown on Figure 1 included in Appendix B.

The recommendations presented in this report are based on the geology, topography, and the results of the geotechnical exploration. Results of borings, in-situ testing, sampling, and laboratory testing are included in the report. A total of 2 borings were drilled in the vicinity of the site as shown on Figure 2 included in Appendix B. The boring logs, along with field and laboratory test results, are enclosed. The collected data have been analyzed and the physical properties of the in-situ soils summarized. General site conditions are discussed, along with recommendations for foundations and subgrade preparation. Important information prepared by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association for studies of this type is presented in Appendix A for your review.

2.0 GENERAL INFORMATION

Planned Modifications

The existing 2-lane, 76.5-foot long, 27.4-foot wide, 3-span bridge supported on precast concrete piles over Wolf Island Slash will be replaced with a 4-lane, 100-foot long, 77.5-foot wide, single-span bridge. The new bridge will be constructed east of the existing bridge. Riprap is planned along the abutment slopes based on the provided plans[1]; abutment slopes are anticipated to be two horizontal units for every vertical unit (2H:1V) and side slopes at the approaches are anticipated to be 3H:1V. Up to 7 and 10 feet of cut and fill, respectively will be required to achieve design grades.

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[1] *Arkansas Department of Transportation Construction Plans for State Highway, La Grue Bayou, Wolf Island Slash & Honey Creek STRS. & APPRS. (S), Prairie County, Route 63 Section 11, Route 33 Section 5, Job 061615*, provided by ARDOT on September 28, 2020.

Drainage

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

Physiographic Setting & Geology

Prairie County is located in east-central Arkansas, in the Mississippi Embayment. The Mississippi Embayment is a trough-like depression containing thousands of feet of sediment and plunging southward along an axis approximating the present course of the Mississippi River. The deposits in the area consist of Holocene epoch alluvial gravel and sand. These materials are typically white to brown or gray, poorly to well sorted, fine- to coarse-quartz sand and gravel with minor silts and clays. These deposits form a broad terrace among the west side of the Mississippi River flood plan, and include both glacial outwash and non-glacial alluvium. Thickness can vary from 3 to 40 meters and may include loessal colluvium from nearby Crowley’s Ridge.

3.0 GEOTECHNICAL EXPLORATION

The borings were drilled between September 17th and 18th, 2020 with a rotary drill rig (CME 75) using hollow-stem auger and wash rotary drilling methods. The borings were drilled to an approximate depth of 100 feet. 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 automatic hammers. Thin-walled Shelby tube samples were collected in cohesive soils at selected depths. Groundwater observations were made during drilling operations.

The collected samples were visually examined by field staff and transported to our laboratory for further evaluation and testing. The samples were examined in the laboratory by a geotechnical professional who prepared descriptive logs of the materials encountered. The boring logs are presented in Appendix C. An explanation of the terms and symbols used on the boring logs is also provided in Appendix C. Included on each boring log are ground surface elevation, station and offset provided by representatives of ARDOT. 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

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, resistivity, consolidated-drained direct shear, and unconsolidated-undrained triaxial compression (UU) test results are also provided in Appendix D. The laboratory tests and corresponding test method standards are presented in Table 2.

Table 2. Summary of Laboratory Tests and Methods.

Laboratory Test	ASTM	AASHTO
Moisture Content	D 2216	T 265
Atterberg Limits	D 4318	T 98
Grain Size Analysis by Sieving	D 6913	T 88
Unconsolidated-Undrained Triaxial Compression	D 2850	T 296
Consolidated-Drained Direct Shear	D 3080	T 236
Soil Electrical Resistivity	G 57	T 288
Soil pH	D 4972	T 289

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

5.0 SUBSURFACE CONDITIONS

General Subsurface Conditions

The borings at this site include Borings W-1 and -2 drilled in the northbound shoulder of the existing southern and northern approaches, respectively. Beneath the approximately 9- and 10-inch thick layer of asphalt and 12- and 5-inch thick cement-treated base material in Borings W-1 and -2, respectively, the soils generally consisted of fine-grained soil underlain by interlayered fine- and coarse-grained soil layers to the 100-foot maximum depth of exploration. More detailed descriptions of the stratigraphy encountered at each bridge are included below and on the boring logs in Appendix C.

Fine-Grained Soil Layers. Underlying the pavement materials, the stratigraphy generally consisted of predominantly fine-grained soils underlain by interlayered fine- and coarse-grained soil layers to the 100-foot maximum depth of exploration in both borings. Fine-grained layers were encountered at depths of approximately 1 to 18 feet and 38 to 68 feet in Boring W-1 and at depths of approximately 1 to 23 feet and 38 to 73 feet in Boring W-2. The predominantly coarse-grained soil layer encountered at depths of 68 and 73 feet in Borings W-1 and -2, respectively, extended to the boring termination depth (100 feet).

The fine-grained soils were classified as lean clay (CL), fat clay (CH), and silty clay (CL-ML) by the Unified Soil Classification System (USCS) and A-6, A-7-6, A-7-5, or A-4 by the AASHTO classification method. The fine-grained soils were very soft to stiff based on SPT N-values, and medium stiff to stiff based on UU tests. The laboratory testing used to determine USCS and AASHTO classifications are presented in Appendix D.

Coarse-Grained Soil Layers. The coarse-grained soil layers encountered at approximate depths of 18 to 38 feet and 68 to 100 feet in boring W-1 and 23 to 38 feet and 73 to 100 feet in Boring W-2 were classified as intermixed, poorly-graded sand (SP-SM by USCS; A-1-b, A-3, or A-2-4 by AASHTO), silty sand (SM by USCS, A-1-a, A-1-b, A-2-4, A-2-6, or A-4 by AASHTO), and clayey sand (SC by USCS, A-4, A-6, and A-7-6 by AASHTO). Based on field test results, the coarse-grained soils were very loose to very dense. Very loose to loose sand (N < 10 bpf) was encountered in Boring W-1 from approximately 23 to 38 feet (approximately El 188 to El 173) and in Boring W-2 from approximately 23 to 33 feet (approximately El 188 to El 178). Very dense sand (N > 50 bpf) was encountered in Boring W-1 from approximate depths of 78 to 100 feet (approximately El 133 to El 112) and in Boring W-2 from approximate depths of 73 to 100 feet (approximately El 138 to El 112).

Soil Resistivity

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 to 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 below and are included in Appendix D.

Table 3. Results of Soil Resistivity Testing.

Boring	Sample No.	Sample Depth (ft.)	pH	Soil Resistivity (ohm-cm)
W-1	SS5	13.5	4.22	5,985
	SS7	23.5	6.00	9,120
	SS12	43.5	7.99	741
	SS15	68.5	6.86	1,596
W-2	SS6	18.5	5.78	5,700
	SS9	28.5	6.59	9,120
	ST-14	48	8.12	684
	SS17	63	8.10	798

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.
- pH less than 5.5.

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

- Resistivity less than 3,000 ohm-cm.

- pH less than 5.5

Results of the corrosion and deterioration testing indicate the site has potential for steel pile or steel reinforcement deterioration. Interpretation of the data and corrosion protection of the bridge structural components should be performed by the design team.

Groundwater

Groundwater was encountered at an approximate depth of 16½ feet during drilling operations in Boring W-2. The presence of higher groundwater levels in Boring W-1 could have been obscured by the use of mud rotary drilling methods, which introduces fluid to the borehole. Groundwater levels could vary significantly over time due to water levels in Wolf Island Slash and seasonal variation in precipitation, recharge, 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. Slopes steeper than 1V:4H must be benched prior to placing new fill. Slope ratios of 1V:3H or flatter are recommended for all cut and fill slopes along the proposed alignment, based on the results of global stability analyses (discussed in a subsequent section).

Cut Areas. After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 95% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D 698/AASHTO T 99). Areas supporting pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

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

Fill and Backfill Placement. Fill and backfill should be placed in level lifts up to 8 inches in loose thickness. For fill and backfill exhibiting a well-defined moisture-density relationship, each lift should be moisture-conditioned to within ±2% of the optimum moisture content and compacted with a sheepsfoot roller or self-propelled compactor to a minimum of 98% of the maximum dry unit weight as determined by the standard Proctor test. Moisture-conditioning can include: aeration and drying of wetter soils, wetting drier soils, and/or mixing wetter and drier soils into a uniform blend. The upper three feet of fill and backfill 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 at least 70% of the relatively density as evaluated from the maximum and minimum index densities measured by ASTM D4253 and D4254, respectively. The upper 3 feet of fill and backfill beneath the base of pavement should be compacted to at least 75% of the relatively density.

Moisture Considerations. The soils encountered in the borings are relatively wet and will most likely require drying. The time for drying will depend on the weather conditions during grading activities. We recommend construction take place during dry weather conditions. Wet weather conditions can cause rutting of the surficial soils which will require drying and recompacting.

Maintaining the moisture content of bearing and subgrade soils within the acceptable range is important during and after construction for the proposed structure. 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.

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

Seismic Considerations

Earthquake Risk. The project area is located within the New Madrid Seismic Zone (NMSZ). The NMSZ is located in the northern part of the Mississippi Embayment and trends in a northeast to southwest direction from southern Illinois to northeast Arkansas. In December 1811, a series of large magnitude earthquakes occurred, which were centered near New Madrid, Missouri. Three strong earthquakes occurred over a 3-month period 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”, eighth edition (2017) or latest edition. A site modified peak ground acceleration (A_s) of 0.252g was obtained from published values.

Seismic Design Parameters. Presented in Table 4 are seismic design parameters based on a seismic hazard with 7% probability of exceedance in 75 years and the field and laboratory testing results.

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

Latitude 34.676693°N/Longitude 91.555218°W		
Category/ Parameter	Designation/ Value	Reference
Seismic Zone	2	AASHTO LRFD 2017 Table 3.10.6-1
Seismic Site Class	D	AASHTO LRFD 2017 Table 3.10.3.1-1
S_S	0.377g	Ground motion parameters obtained from the computer program supplied with the AASHTO Guidelines for the Seismic Design of Highway Bridges (2009) using the indicated latitude and longitude coordinates of the project site and the seismic site class based on boring data.
S_1	0.111g	
F_a	1.499	
F_v	2.355	
F_{PGA}	1.454	
t_s	0.464	
t_0	0.093	
S_{DS}	0.565g	
S_{D1}	0.262g	
PGA	0.173g	
As	0.252g	

Liquefaction and Dynamic Settlement. A study was performed to evaluate the liquefaction and dynamic settlement potential at the site. Both field and laboratory data were used to perform the analysis. The field measurements included the assumed depth of the water table and the SPT N-values. The laboratory data included USCS/AASHTO classification and soil unit weight. An earthquake magnitude (Mw) of 7.5 was considered based on earthquake deaggregation data for the site. An A_s value of 0.252g was utilized as obtained from the referenced Seismic Design Maps. Groundwater was set at a depth of approximately 16 feet measured from the approximate ground surface at the location of Boring W-2.

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

Table 5. Results of Liquefaction Analyses.

Boring No.	Depth of Boring (ft.)	Depth Intervals with Liquefaction Factor of Safety Less than 1.0		Estimated Dynamic Settlement (in.)
		Depth (ft.)	Elevation	
W-1	100	23 - 38	189-174	6
W-2	100	23 - 38	189-174	7

The current state of practice for liquefaction hazard assessment is based on what is known as “the Simplified Method” as introduced by Seed (1971) and subsequent modifications/revisions by many researchers (Seed 1982, Idriss 1999, Youd 2001, and Idriss and Boulanger 2014, among others). The simplified method was based on observations and assessments of soil zones that either liquefied or did not liquefy in the upper 50 to 60 feet. A discussion of the downdrag potential due to dynamic settlement is included in a subsequent section.

Liquefaction hazard mitigation can be accomplished using compaction piles (large displacement piles) or proprietary ground improvement techniques such as earthquake drains or stone columns. Proprietary ground improvement techniques are typically performed by specialty firms on a design/build basis.

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. Geotechnology evaluated this condition, and more information is provided in the Global Stability section of this report.

Approach Embankment Settlement

Settlement analyses were performed to assess fill-induced settlement for the approaches. Based on the plans provided, approximately 10 feet of fill will be required at the proposed abutments to bring the sites to grade. For settlement analyses, we assumed cohesive, engineered fill will be used for the fill material. The results of the settlement analyses 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.

South Abutment			North Abutment		
Max Fill (feet)	Estimated Settlement (inches)		Max Fill (feet)	Estimated Settlement (inches)	
	Immediate	Consolidation		Immediate	Consolidation
10	2¾	¼	10	2½	½

Discussion of Fill-Induced Settlement. The results of the settlement analyses indicate immediate and primary consolidation settlement at the approaches. We anticipate the immediate settlement to occur within a week of fill placement. Based on the one-dimensional consolidation test performed on a sample collected from Boring L-1 at the adjacent bridge (No. 01859), practical completion of consolidation induced settlement will occur within 3 to 6 weeks following fill placement.

It should be noted the one-dimensional consolidation test confines the drainage pathway to one dimension while in the field, drainage takes place in three dimensions; therefore, it is our professional opinion the estimated settlement will occur in a shorter time period. If the anticipated waiting period adversely impacts project schedule, a settlement monitoring program may be implemented and survey data be forwarded to Geotechnology so that construction can commence as soon as the remaining settlement is tolerable.

Global Stability

Based on the provided plans, abutment fill will be placed at a 2H:1V slope and side slopes will be constructed at 3H:1V and 6H:1V slopes on top of the varying existing grades. Geotechnology performed stability analyses for deep-seated, global failure of bridge abutment slopes using the computer program Slide. 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 Table 7. A pseudo-static seismic acceleration of 0.126g, corresponding to one-half the peak ground acceleration (per FHWA Publication NHI-11-032) was utilized for the seismic condition. Section profiles with calculated critical failure surfaces and utilized soil parameters are presented in Appendix E for selected analyses.

Table 7. Slope Stability Analyses Results.

Location	Slope Ratio	Approximate Berm | Slope Height (ft.)	Calculated Factor of Safety		
			Short-Term Static[a]	Long-Term Static[a]	Seismic[b]
South Abutment Side Slope	3:1	-- | 10	3.09	2.20	2.05
South Abutment Spill Slope	2:1	12 | 15	2.74	1.61	2.04
North Abutment Side Slope	3:1 and 6:1	-- | 10	2.90	2.22	1.80
North Abutment Spill Slope	2:1	12 I 15	2.73	1.63	1.85

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

The models used in this computation did not consider the relative stabilizing effect of foundation piles driven to support the abutments or armoring of abutments with rip rap or concrete. In general, foundation piles may provide additional stabilizing force to the abutment slopes, resulting in a factor of safety higher than those presented in Table 7. Existing slopes should be benched prior to placing new fill to reduce the potential for development of slip planes between new and existing fill.

Deep Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2017). It is our understanding 16- and 18-inch diameter, closed-end, steel, pipe piles are being considered for support of the proposed bridge. Geotechnology should be notified if a different foundation type is being considered. Synthetic profiles have been compiled for each abutment location based upon the soil profile encountered in the borings, approximate boring elevations, and the proposed final grade. Based on the provided information, the pile cap elevation at both abutments is approximately El 207 and the upper 10 feet beneath the pile cap at each pile location will be pre-bored.

Nominal resistance curves showing axial resistance from skin friction and total axial capacity (skin friction + end bearing) for the abutments are presented in Appendix F and tabulated values are presented in Table 8; nominal post-liquefaction, resistance curves considering downdrag forces due to liquefaction are also included in Appendix F. Uplift (tension) capacities may be calculated using the resistance provided by skin friction. Soil parameters, including LPILE parameters, for each abutment are included in Appendix G.

Table 8. Nominal Axial Pile Resistance – Static and Post Liquefaction.

Location	Pile Diameter (inches)	Embedment Length (feet)	Nominal Static Resistance (tons)			Nominal Post-Liquefaction Resistance (tons)	
			Skin Friction	End Bearing	Compression Total	Compression Total[a,b]	Drag Load
South Abutment	16	65	102	105	208	200	8
		70	137	105	242	234	8
		75	175	105	280	273	8
North Abutment		65	100	7	107	90	17
		70	119	105	224	207	17
		75	158	105	263	246	17
South Abutment	18	65	121	127	248	239	9
		70	166	133	299	290	9
		75	215	133	348	339	9
North Abutment		65	118	9	127	108	19
		70	141	127	268	249	19
		75	190	133	323	304	19

[a] Nominal post-liquefaction resistance has not been reduced by the drag load.

[b] Based on estimated dynamic settlement due to liquefaction from El 189 to 174 presented in Table 5.

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 9 may be applied.

Table 9. Resistance Factors Based on Static Analysis Methods.

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

Based on AASHTO LRFD (2017) Table 10.5.5.2.3-1, a higher resistance factor can be used in accordance with the methods of pile testing performed as indicated in Table 10.

Table 10. 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 (2017) 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 (2017) 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 (2017) sections 10.7.3.9 and 10.7.3.11.

Driven Pile Construction Considerations. Minimum hammer energies required to drive the piles were evaluated using the computer software WEAP. The recommended minimum hammer energy is provided in Table 11.

Table 11. Minimum Hammer Energies.

Pile Diameter [a] (inches)	Location	Embedment Length (feet)	Required Capacity (tons)	Minimum Rated Hammer Energy (kip-feet)
16	South Abutment	75	269	52
	North Abutment	77		

[a] Closed-ended pipe piles with ½-inch thick walls.

Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. Alternatively, potential driving criteria can be developed using wave equation analyses after the pile hammer is selected.

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 (2017) 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 (2017) 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 (2017) Table 10.5.5.2.3-1 or 2 percent of the production piles, whichever results in a larger number of tests. We recommend that the identified piles be tested at the end of initial drive (EOID) and a restrike performed at a minimum seven days after EOID.

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

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

<u>Settlement</u>. 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.

<u>Uplift Resistance</u>. 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.

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

The effects of group interaction must be considered when evaluating pile/shaft group horizontal movement. The lateral resistance for individual piles calculated by LPILE must be reduced by the P-multipliers provided in Section 10.7.2.4 of the AASHTO LRFD (2017) to determine lateral resistance of a pile group. Alternatively, the GROUP software can be used to evaluate the lateral resistance of the pile/shaft groups. The resistance factor for lateral resistance of single piles or pile groups is 1.0.

Downdrag

The AASHTO LRFD (2017) suggests that settlement of 0.4-inch or greater could produce downdrag on pile foundations. Downdrag occurs as the soil strata move downward relative to the foundations due to settlement of the soil layers. The relative movement of the soil layers versus the shaft depends on the final foundation configuration.

Based on liquefaction analysis results, up to 6 and 7 inches of dynamic settlement was estimated at the southern and northern abutments, respectively, during the design earthquake event. Additional pile analysis was performed to evaluate post-liquefaction pile capacities. The post-liquefaction pile capacity curves are presented in Appendix F. Nominal capacities for the post-liquefaction case are presented in Table 8. Please note that densification of the potentially liquefiable sand layers is expected while driving the closed-ended pipe piles and may reduce the potential for liquefaction. The magnitude and extent of densification due to pile driving could be determined by performing SPT or CPT borings after pile driving.

Pre-drilling or applying bituminous or viscous coatings are not recommended to reduce liquefaction-induced downdrag because such methods will reduce the nominal static compressive resistance of the piles. If potential downdrag forces are not tolerable, consideration should be given to methods which mitigate dynamic settlement by reducing pore pressure. Such techniques are performed by specialty contractor; if more information is desired, please contact Geotechnology.

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 <u>not</u> 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 <u>Not</u> 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.

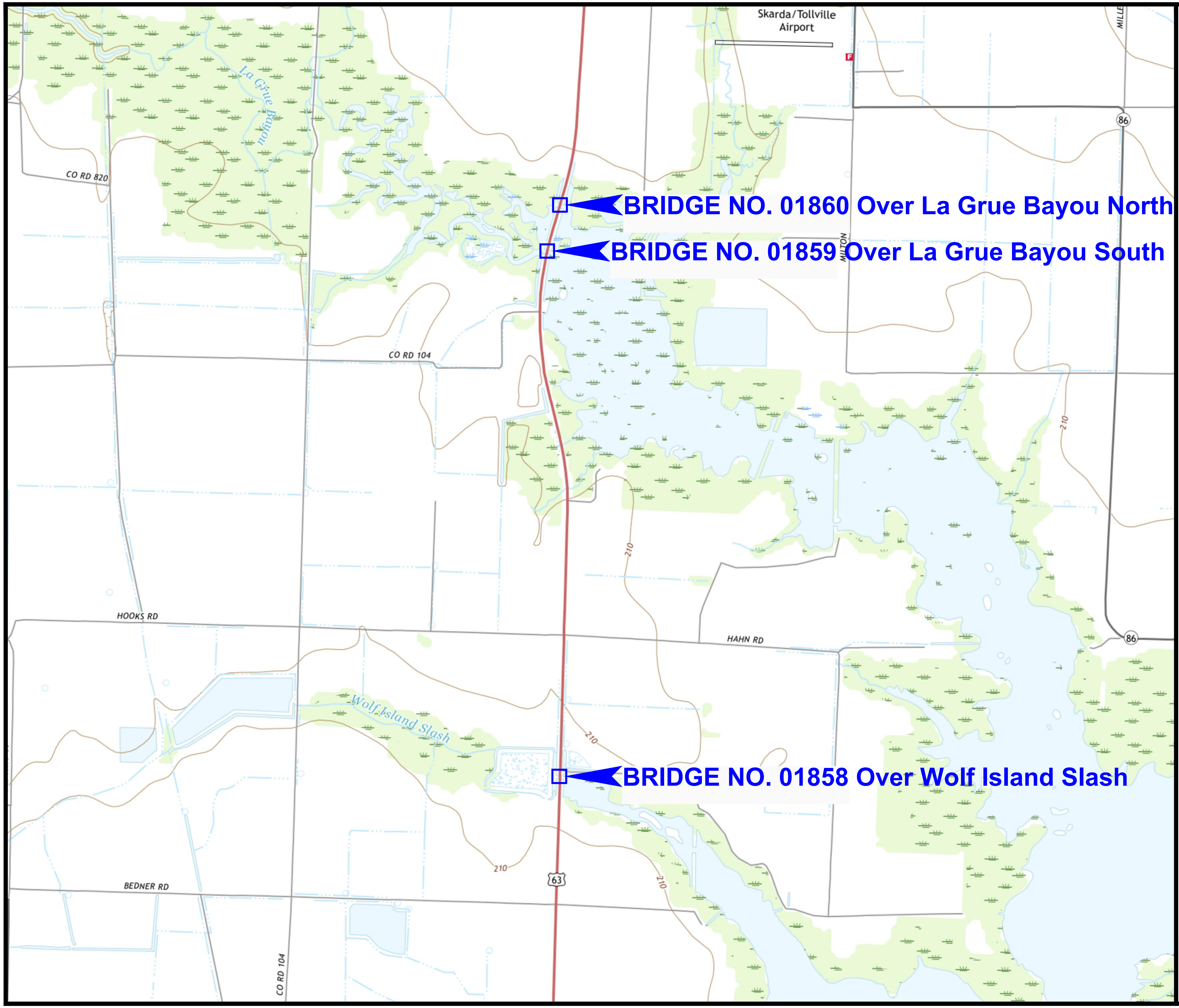
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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 <u>not</u> 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 <u>not</u> 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 a 7.5 minute U.S.G.S. map for Slovak, Arkansas quadrangle, last revised in 2017.

N

0 2,000 4,000

SCALE IN FEET

Drawn By: SDF	Ck'd By: ASM	App'vd By: DMS
Date: 12-8-20	Date: 12-28-20	Date: 12-28-20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Route 63 Section 11
Structures and Approaches
Bridge Nos. 01858, 01859 and 01860
Prairie County, Arkansas

BRIDGE LOCATIONS AND TOPOGRAPHY

Project Number J034561.01

FIGURE 1

NOTES

1. Plan adapted from an October 14, 2015 aerial photograph courtesy of Google Earth.
2. Borings were located in the field with reference to site features and are shown approximate only.

LEGEND

● Boring Location

0 20 40 80
SCALE IN FEET

N

Drawn By: SDF	Ck'd By: ASM	App'vd By: DMS
Date: 12-8-20	Date: 12-28-20	Date: 12-28-20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Route 63 Section 11
Structures and Approaches
Bridge No. 01858
Prairie County, Arkansas

AERIAL PHOTOGRAPH OF BRIDGE 01858 AND BORING LOCATIONS

Project Number
J034561.01

FIGURE 2

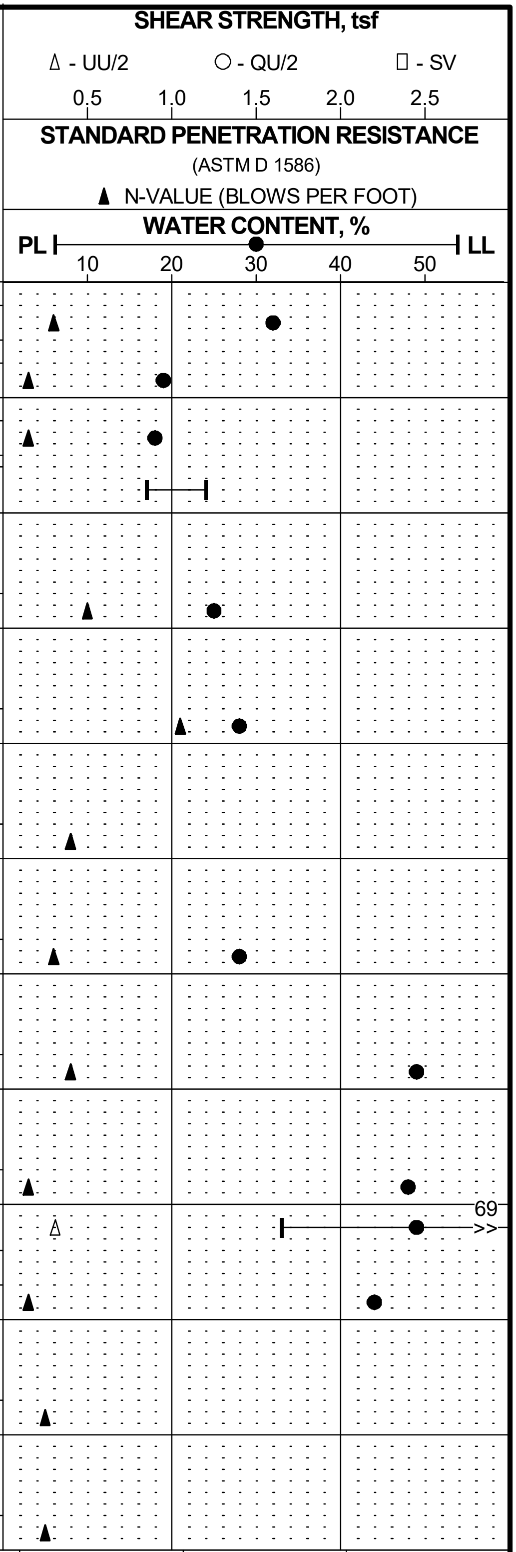
Appendix C

BORING INFORMATION

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

Surface Elevation: 212
Datum msl
Completion Date: 9/17/20
Station: 113+14
Offset: -6

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Asphalt: 9 inches		
		Base Materials: 12 inches of cement-treated soil, gravel and sand	2-2-4	SS1
		Brown, tan and red, FAT CLAY - CH	2-1-2	SS2
5	207	Soft, brown and gray, SILTY CLAY - (CL-ML)		
		trace sand	0-1-2	SS3
		82% passing No. 200 sieve		ST4
10	202	Stiff, brown and gray, LEAN CLAY - CL		
		little sand pH = 4.22 resistivity = 5,985 ohms-cm	3-5-5	SS5
15	197			
		Medium dense to loose, tan to red-brown, SILTY SAND - (SM) lean clay partings	9-9-12	SS6
20	192			
		20% passing No. 200 sieve pH = 6.00 resistivity = 9,120 ohms-cm	3-4-4	SS7
25	187			
		19% passing No. 200 sieve	3-3-3	SS8
30	182			
		18% passing No. 200 sieve	2-3-5	SS9
35	177			
		Soft to medium stiff, red-brown to gray, FAT CLAY - (CH)	0-1-2	SS10
40	172	trace organics	74	ST11
		pH = 7.99 resistivity = 741 ohms-cm	0-1-2	SS12
45	167			
			0-2-3	SS13
50	162			
			1-2-3	SS14



GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM
WASHBORING FROM 15 FEET
JCG DRILLER SWF LOGGER
CME 75 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 80 %

REMARKS:

Drawn by: ASM	Checked by: DMS	App'vd. by: ASE
Date: 9/24/20	Date: 12/17/20	Date: 12/17/20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ARDOT 061615
Highway 63 Bridge No. 01858
Over Wolf Island Slash
Prairie County, Arkansas

LOG OF BORING: W-1

Project No. J034561.01

Surface Elevation: 212

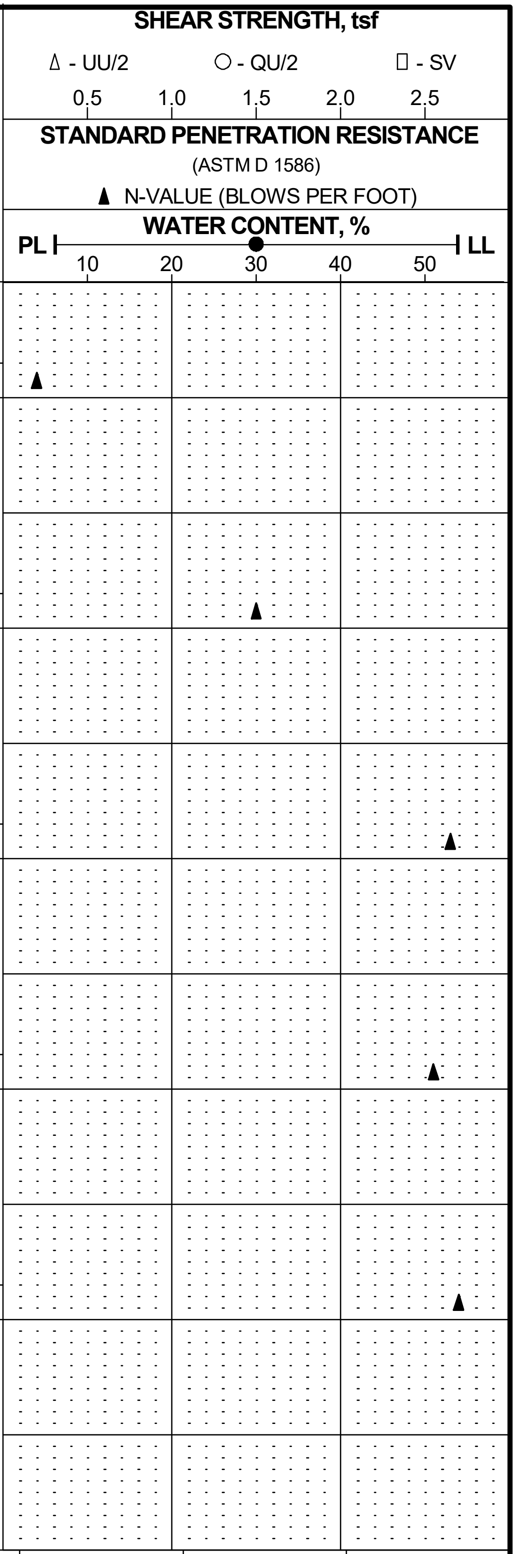
Datum msl

Completion Date: 9/17/20

Station: 113+14

Offset: -6

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Soft to medium stiff, red-brown to gray, FAT CLAY - (CH) *(continued)*			
				0-1-3	SS14
60	152				
65	147				
		Dense to very dense, gray SAND, trace silt - SP-SM trace organics pH = 6.86 resistivity = 1,596 ohms-cm		9-13-17	SS15
70	142				
75	137				
		trace gravel		18-21-32	SS16
80	132				
85	127				
		trace organics		13-16-35	SS17
90	122				
95	117				
		Very dense, brown and gray, CLAYEY SAND - SC		9-17-37	SS18
100	112	Boring terminated at 100 feet.			
105	107				



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 J034561.01.GPJ GTINC 0638301.GPJ 3/23/23

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM

WASHBORING FROM 15 FEET

JCG DRILLER SWF LOGGER

CME 75 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 80 %

REMARKS:

Drawn by: ASM	Checked by: DMS	App'vd. by: ASE
Date: 9/24/20	Date: 12/17/20	Date: 12/17/20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

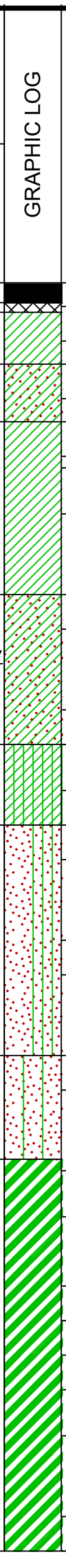
ARDOT 061615
Highway 63 Bridge No. 01858
Over Wolf Island Slash
Prairie County, Arkansas

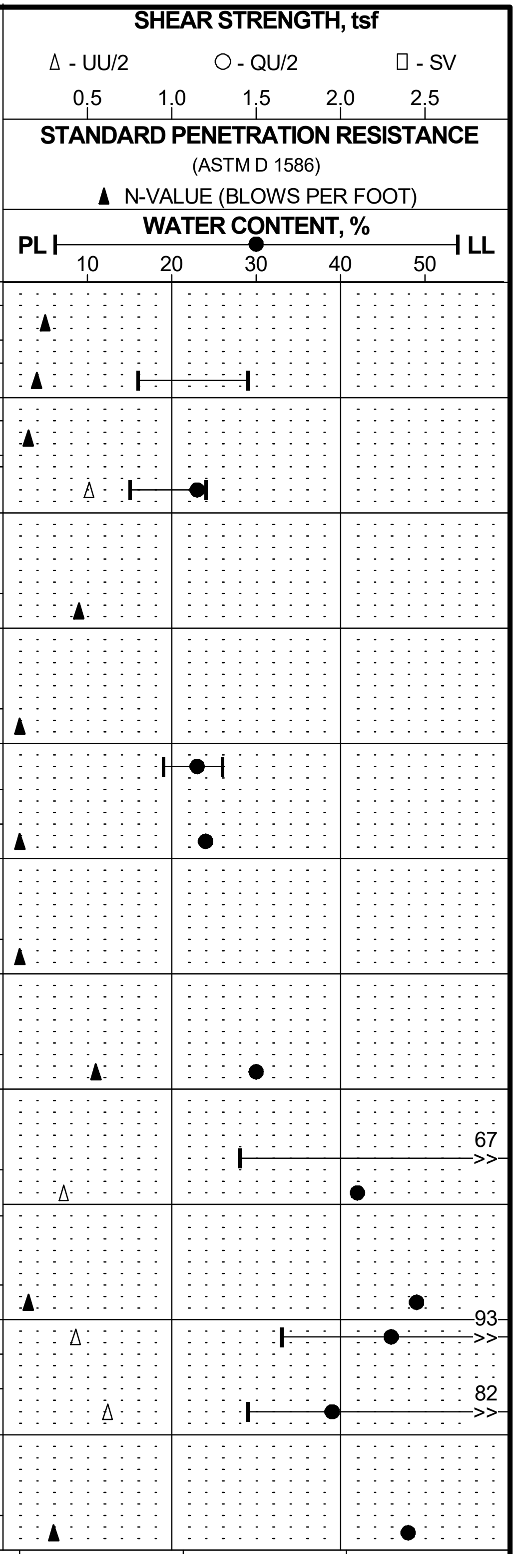
CONTINUATION OF LOG OF BORING: W-1

Project No. J034561.01

Surface Elevation: 212
Datum: msl
Completion Date: 9/18/20
Station: 114+09
Offset: -4

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Asphalt: 10 inches		
		Base Materials: 5 inches of cemen-treated soil	2-2-3	SS1
		Medium stiff, brown and gray, LEAN CLAY - (CL) trace organics	2-2-2	SS2
5	207	Soft, brown and gray, sandy, LEAN CLAY - (CL) 69% passing No. 200 sieve	1-2-1	SS3
		Soft to stiff, brown and gray, LEAN CLAY - (CL) trace sand 89% passing No. 200 sieve	109	ST4
10	202			
		Stiff, brown and gray, sandy, LEAN CLAY - CL	2-3-6	SS5
15	197			
		pH = 5.78 resistivity = 5,700 ohms-cm	0-1-1	SS6
20	192	Brown and gray, sandy, SILTY CLAY - (CL-ML) 66% passing No. 200 sieve	52	ST7
		Very loose, gray SAND with silt - (SP-SM) 8% passing No. 200 sieve	1-1-1	SS8
25	187			
		12% passing No. 200 sieve pH = 6.59 resistivity = 9,120 ohms-cm	1-1-1	SS9
30	182			
		Medium dense, gray to red-brown, SILTY SAND - (SM) 18% passing No. 200 sieve	3-5-6	SS10
35	177			
		Soft to stiff, brown, red and gray to gray, FAT CLAY - (CH) 98% passing No. 200 sieve	113	ST11
40	172			
			0-1-2	SS12
45	167		79	ST13
		pH = 8.12 resistivity = 684 ohms-cm	83	ST14
50	162			
			2-2-4	SS15





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 J034561.01.GPJ GTINC 0638301.GPJ 3/23/23

GROUNDWATER DATA

ENCOUNTERED AT 16.5 FEET ∇

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM
WASHBORING FROM 18.5 FEET
JCG DRILLER SWF LOGGER
CME 75 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 80 %

REMARKS: Shelby tube attempted at 6 feet.

Drawn by: ASM	Checked by: DMS	App'vd. by: ASE
Date: 9/25/20	Date: 12/17/20	Date: 12/17/20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ARDOT 061615
Highway 63 Bridge No. 01858
Over Wolf Island Slash
Prairie County, Arkansas

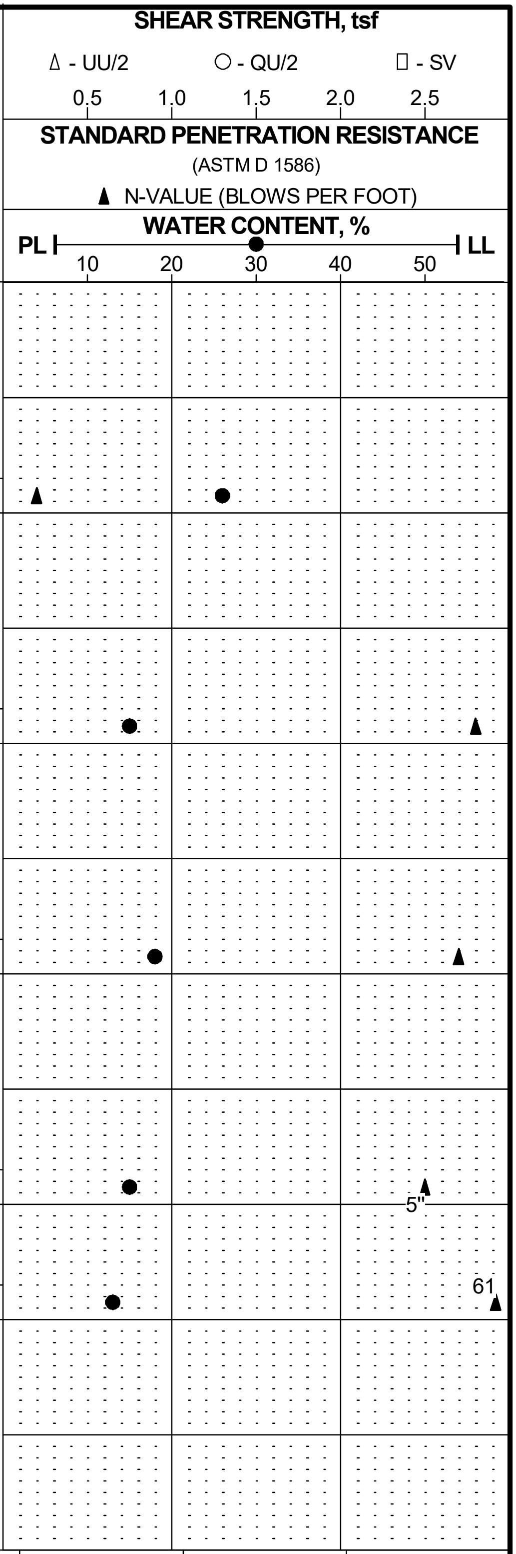
LOG OF BORING: W-2

Project No. J034561.01

Surface Elevation: 212
Datum msl

Completion Date: 9/18/20
Station: 114+09
Offset: -4

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Soft to stiff, brown, red and gray to gray, FAT CLAY - (CH) *(continued)*			
60	152				
		pH = 8.10 resistivity = 798 ohms-cm		0-1-3	SS17
65	147				
70	142				
		Very dense, gray, SILTY SAND, trace clay partings - SM		14-22-34	SS19
75	137				
80	132				
				13-24-30	SS21
85	127				
90	122				
				25-50/5"	SS23
95	117				
				13-27-34	SS24
100	112	Boring terminated at 100 feet.			
105	107				



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 J034561.01.GPJ GTINC 0638301.GPJ 3/23/23

GROUNDWATER DATA

ENCOUNTERED AT 16.5 FEET ⊻

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM
WASHBORING FROM 18.5 FEET
JCG DRILLER SWF LOGGER
CME 75 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 80 %

REMARKS: Shelby tube attempted at 6 feet.

Drawn by: ASM	Checked by: DMS	App'vd. by: ASE
Date: 9/25/20	Date: 12/17/20	Date: 12/17/20

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ARDOT 061615
Highway 63 Bridge No. 01858
Over Wolf Island Slash
Prairie County, Arkansas

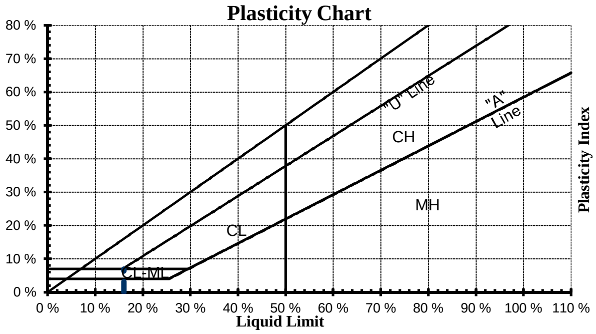
CONTINUATION OF
LOG OF BORING: W-2

Project No. J034561.01

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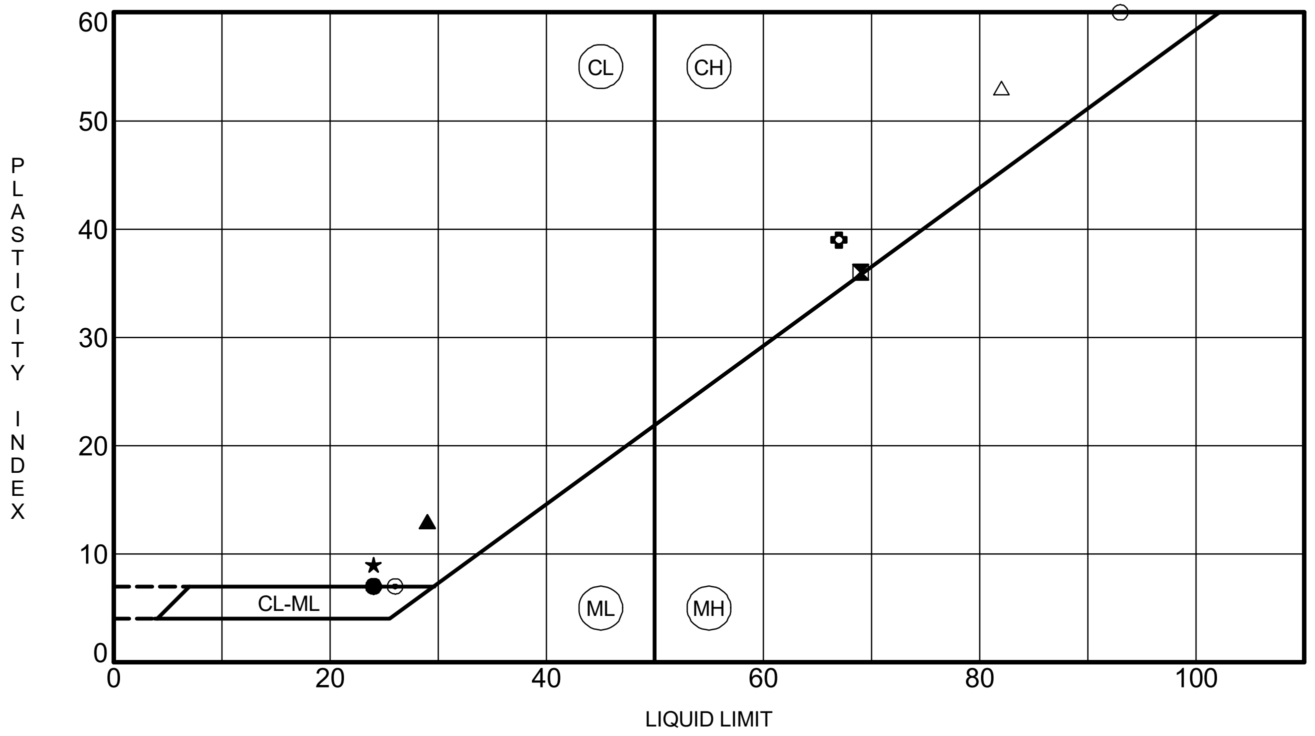
Relative composition and Unified Soil Classification System (USCS) designations are based on visual descriptions and are approximate only. If laboratory tests were performed to classify the soil, the USCS designation is shown in parenthesis.

Appendix D

LABORATORY TEST DATA



Specimen Identification			LL	PL	PI	Fines	Classification
●	**W-1**	**8.0**	**24**	**17**	**7**	**83**	**SILTY CLAY(CL-ML), A-4 (4)**
⧗	**W-1**	**40.0**	**69**	**33**	**36**		**FAT CLAY(CH), A-7-5**
▲	**W-2**	**3.5**	**29**	**16**	**13**	**69**	**SANDY LEAN CLAY(CL), A-6 (7)**
★	**W-2**	**8.0**	**24**	**15**	**9**	**89**	**LEAN CLAY(CL), A-4 (6)**
⊙	**W-2**	**20.0**	**26**	**19**	**7**	**66**	**SANDY SILTY CLAY(CL), A-4 (3)**
✤	**W-2**	**38.0**	**67**	**28**	**39**	**98**	**FAT CLAY(CH), A-7-6 (45)**
○	**W-2**	**45.0**	**93**	**33**	**60**		**FAT CLAY(CH), A-7-5**
△	**W-2**	**48.0**	**82**	**29**	**53**		**FAT CLAY(CH), A-7-5**

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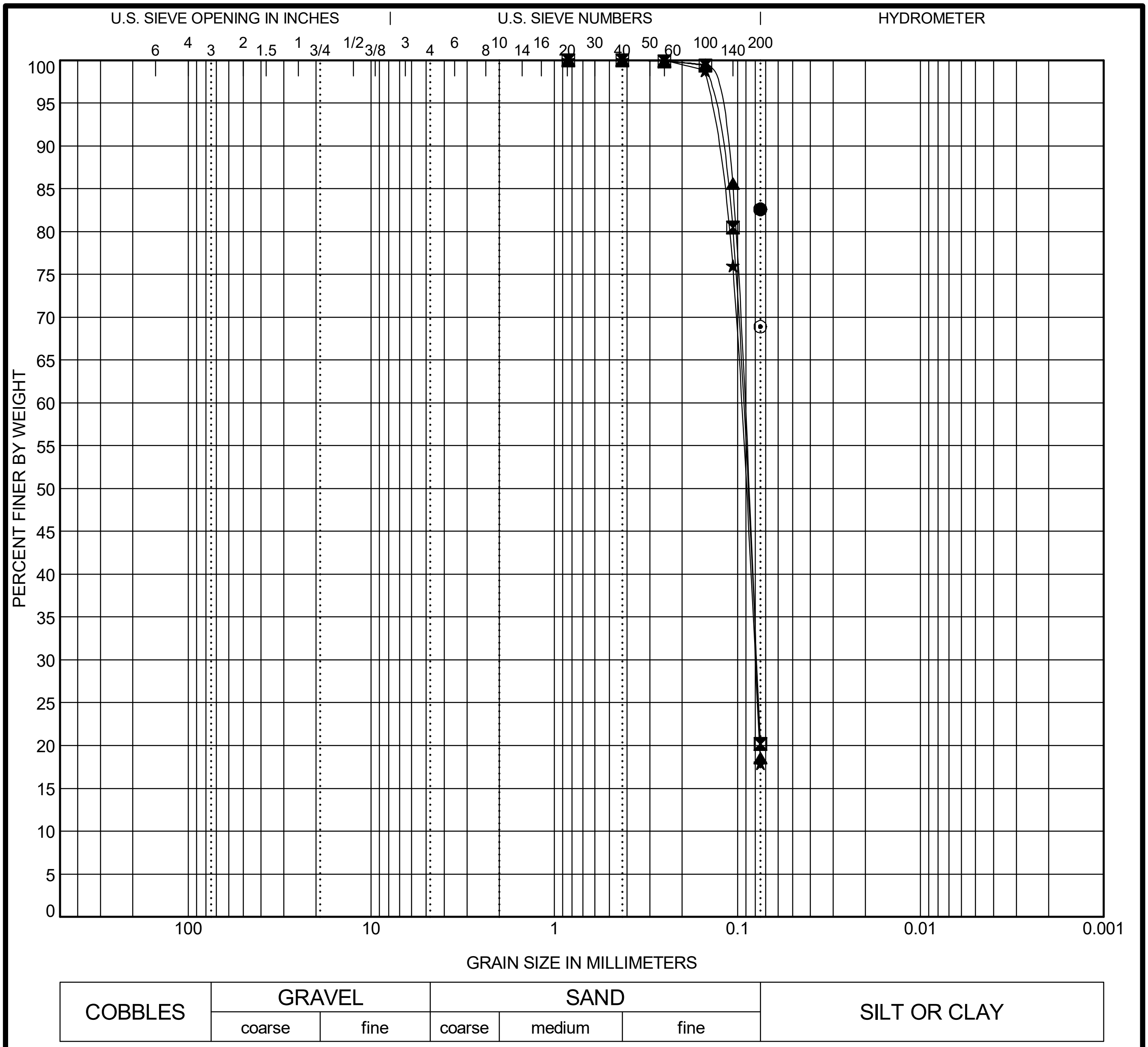
ATTERBERG LIMITS RESULTS

ARDOT 061615 Highway 63 Bridge No. 01858
Over Wolf Island Slash
Prairie County, Arkansas
J034561.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	W-1	8.0	**SILTY CLAY(CL-ML), A-4 (4)**	24	17	7		
⊠	W-1	23.5	**SILTY SAND(SM), A-2-4**					
▲	W-1	28.5	**SILTY SAND(SM), A-2-4**					
★	W-1	33.5	**SILTY SAND(SM), A-2-4**					
⊙	W-2	3.5	**SANDY LEAN CLAY(CL), A-6 (7)**	29	16	13		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	W-1	8.0	0.075				0.0	0.0	82.6	
⊠	W-1	23.5	0.84	0.094	0.079		0.0	79.8	20.2	
▲	W-1	28.5	0.84	0.093	0.08		0.0	81.5	18.5	
★	W-1	33.5	0.84	0.096	0.081		0.0	82.1	17.9	
⊙	W-2	3.5	0.075				0.0	0.0	68.9	

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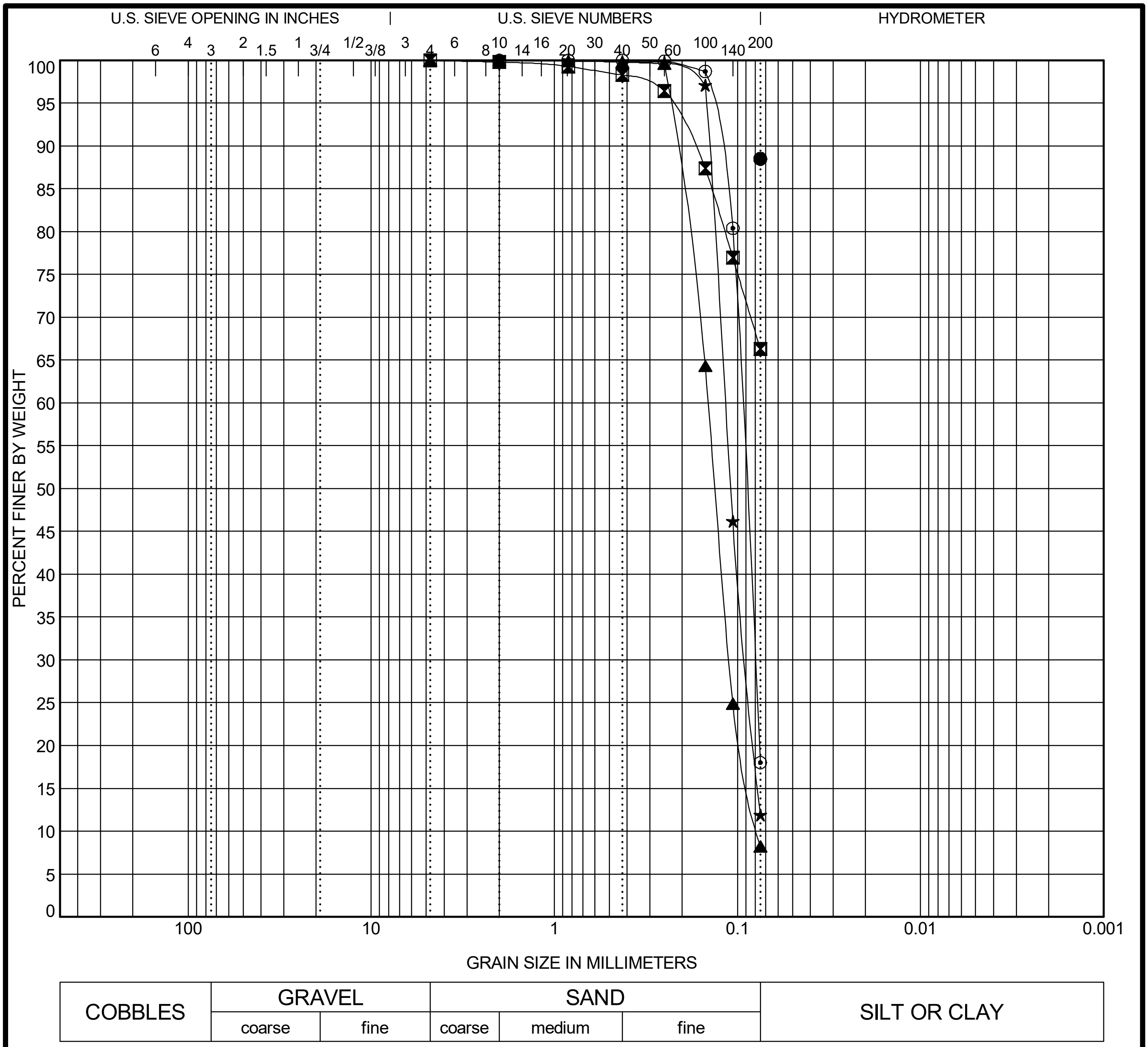
GRAIN SIZE DISTRIBUTION

ARDOT 061615 Highway 63 Bridge No. 01858
Over Wolf Island Slash
Prairie County, Arkansas
J034561.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**W-2**	**8.0**	**LEAN CLAY(CL), A-4 (6)**	**24**	**15**	**9**		
☒	**W-2**	**20.0**	**SANDY SILTY CLAY(CL), A-4 (3)**	**26**	**19**	**7**		
▲	**W-2**	**23.5**	**POORLY GRADED SAND with SILT(SP-SM), A-3**				**1.09**	**1.86**
★	**W-2**	**28.5**	**POORLY GRADED SAND with SILT(SP-SM), A-2-4**				**0.95**	**1.58**
⊙	**W-2**	**33.5**	**SILTY SAND(SM), A-2-4**					

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**W-2**	**8.0**	**0.075**				**0.0**	**0.0**	**88.5**	
☒	**W-2**	**20.0**	**4.75**				**0.0**	**33.7**	**66.3**	
▲	**W-2**	**23.5**	**4.75**	**0.144**	**0.111**	**0.078**	**0.0**	**91.8**	**8.2**	
★	**W-2**	**28.5**	**2**	**0.116**	**0.09**		**0.0**	**88.1**	**11.9**	
⊙	**W-2**	**33.5**	**2**	**0.095**	**0.08**		**0.0**	**82.0**	**18.0**	

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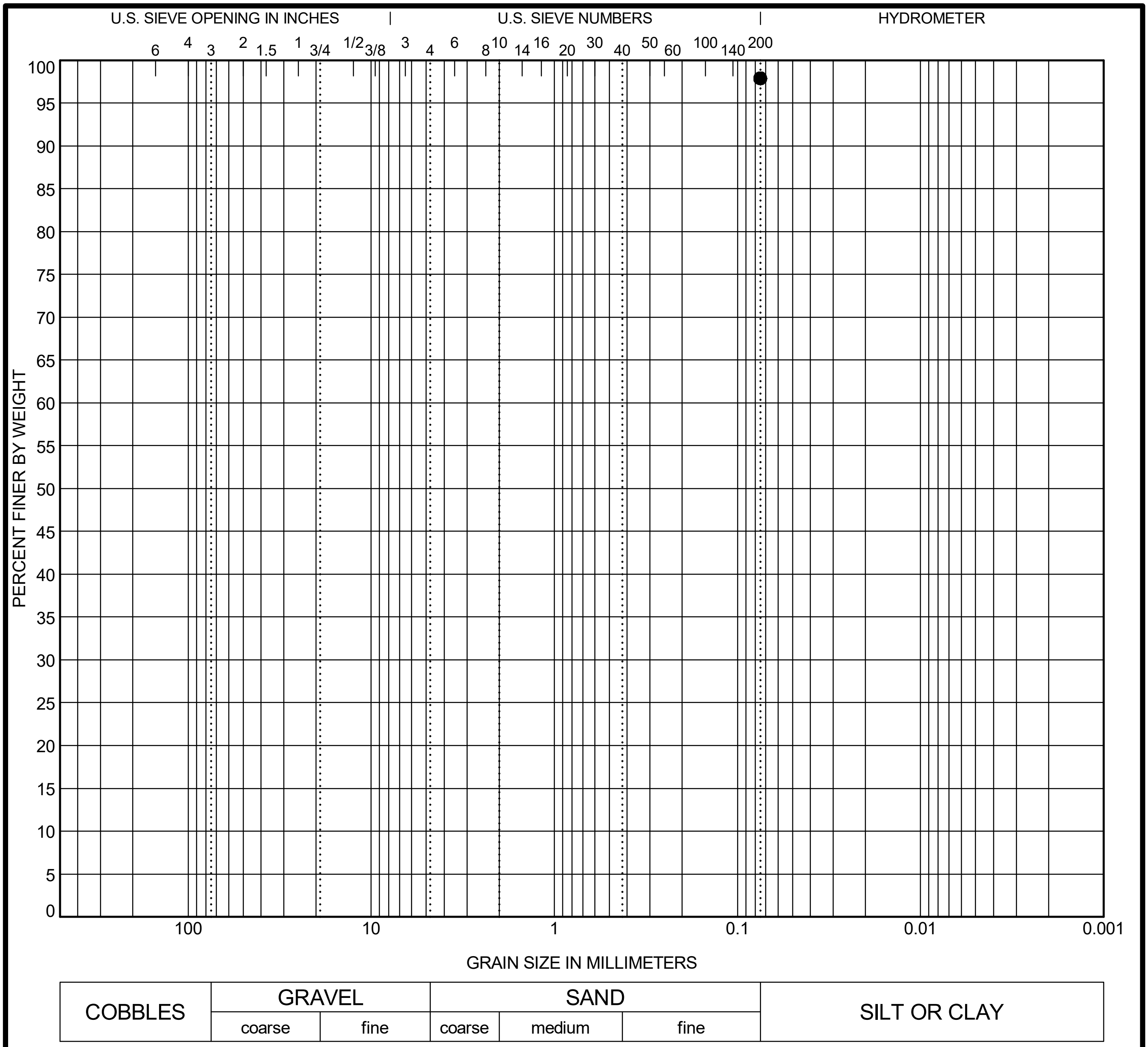
GRAIN SIZE DISTRIBUTION

ARDOT 061615 Highway 63 Bridge No. 01858
Over Wolf Island Slash
Prairie County, Arkansas
J034561.01



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● **W-2**	**38.0**	**FAT CLAY(CH), A-7-6 (45)**	**67**	**28**	**39**		

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **W-2**	**38.0**	**0.075**				**0.0**	**0.0**	**97.9**	

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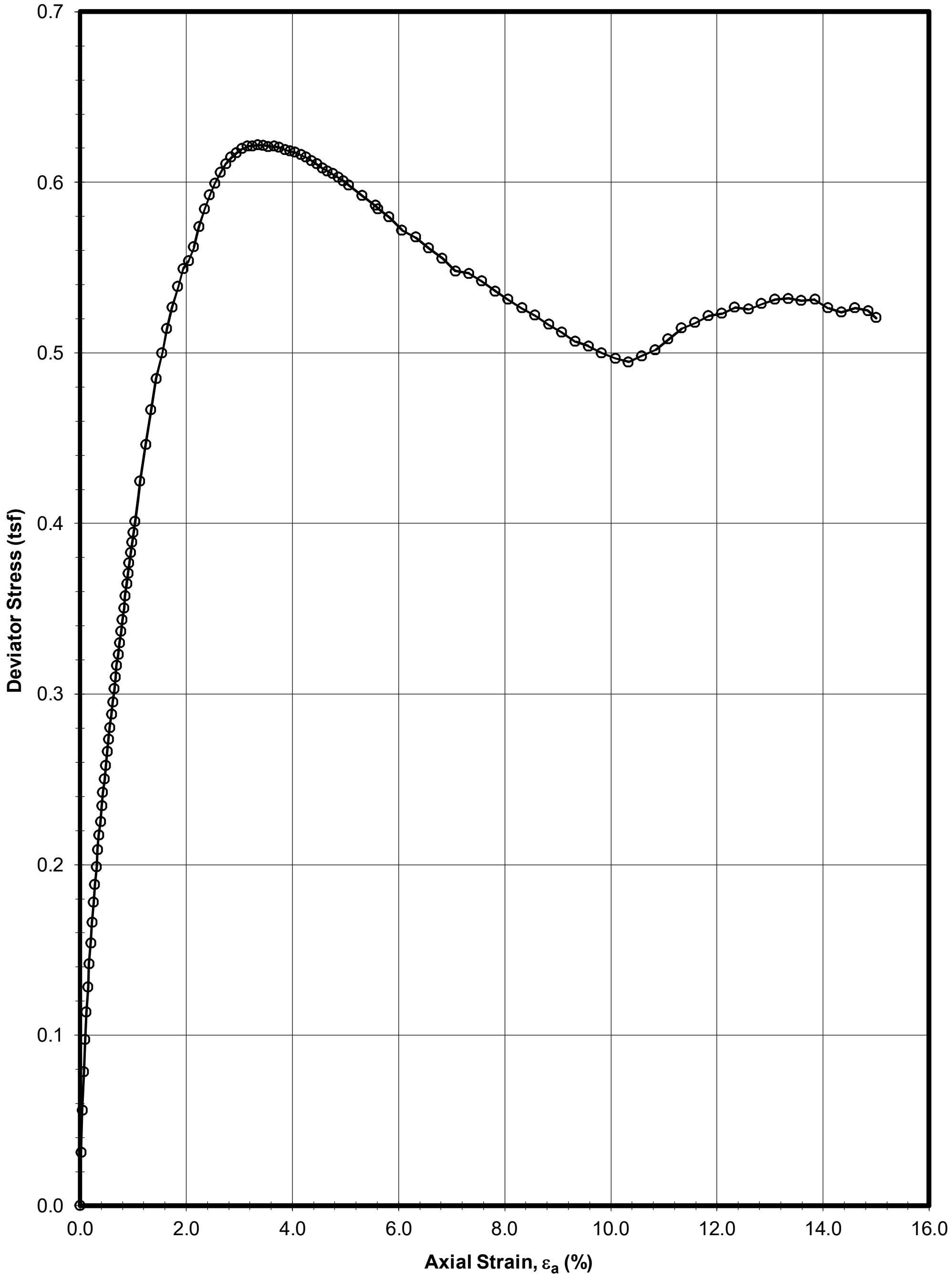
GRAIN SIZE DISTRIBUTION

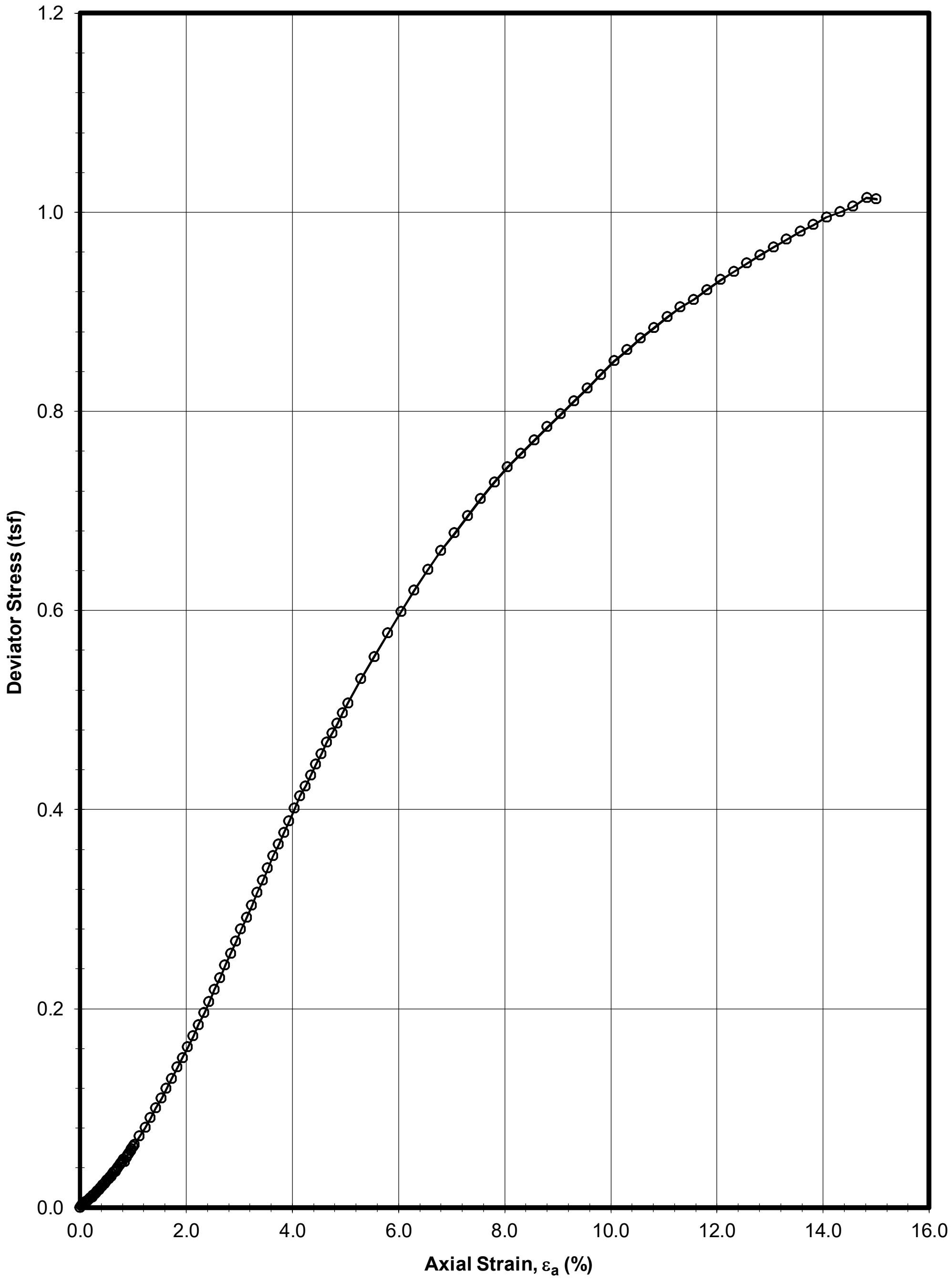
ARDOT 061615 Highway 63 Bridge No. 01858
Over Wolf Island Slash
Prairie County, Arkansas
J034561.01



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J034561.01
Boring: W-1
Sample: ST-11 - Depth: 40 ft.

GEOTECHNOLOGY
A Universal Engineering Sciences Company



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

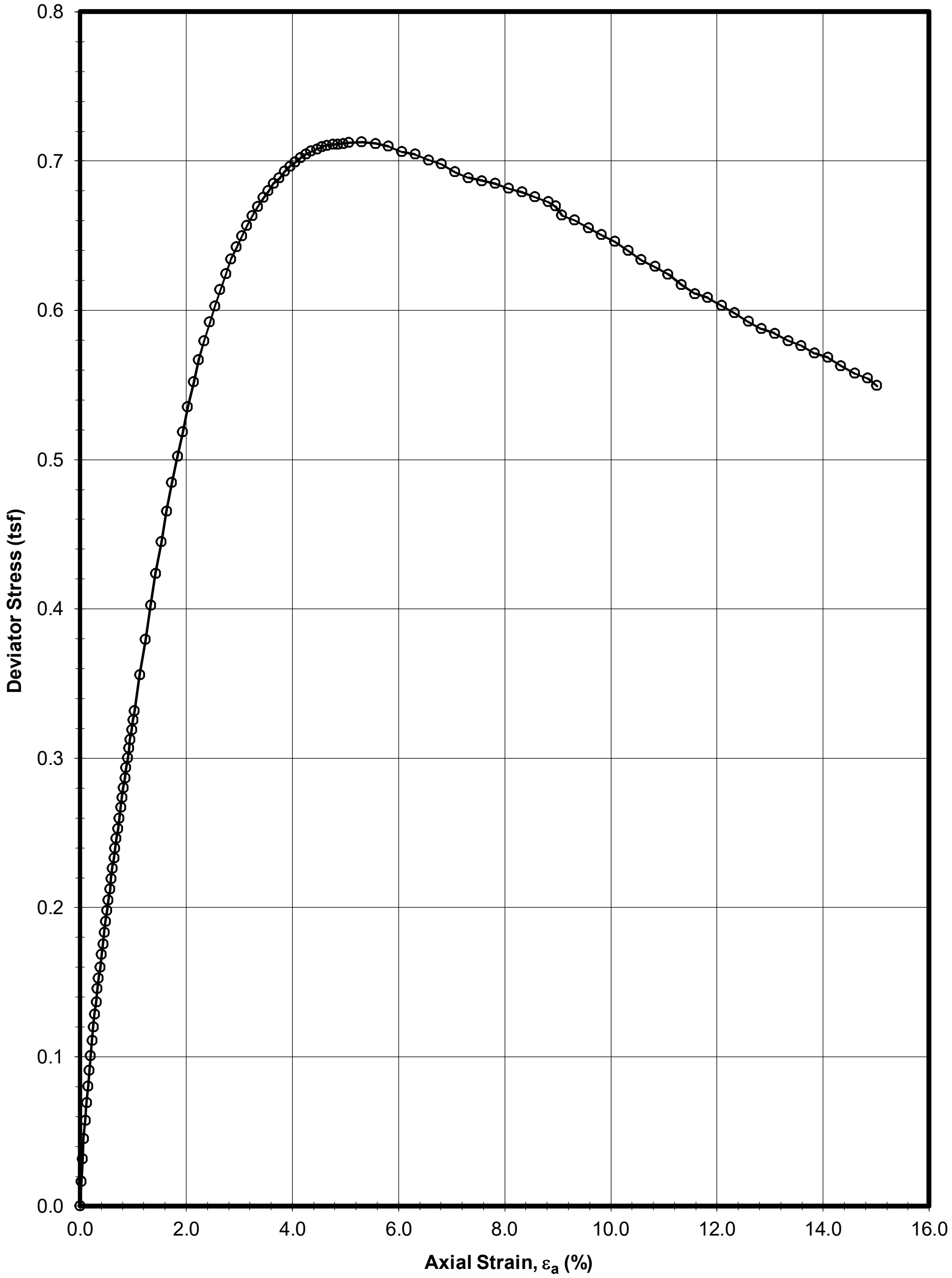
ASTM D 2850

Project No.: J034561.01

Boring: W-2

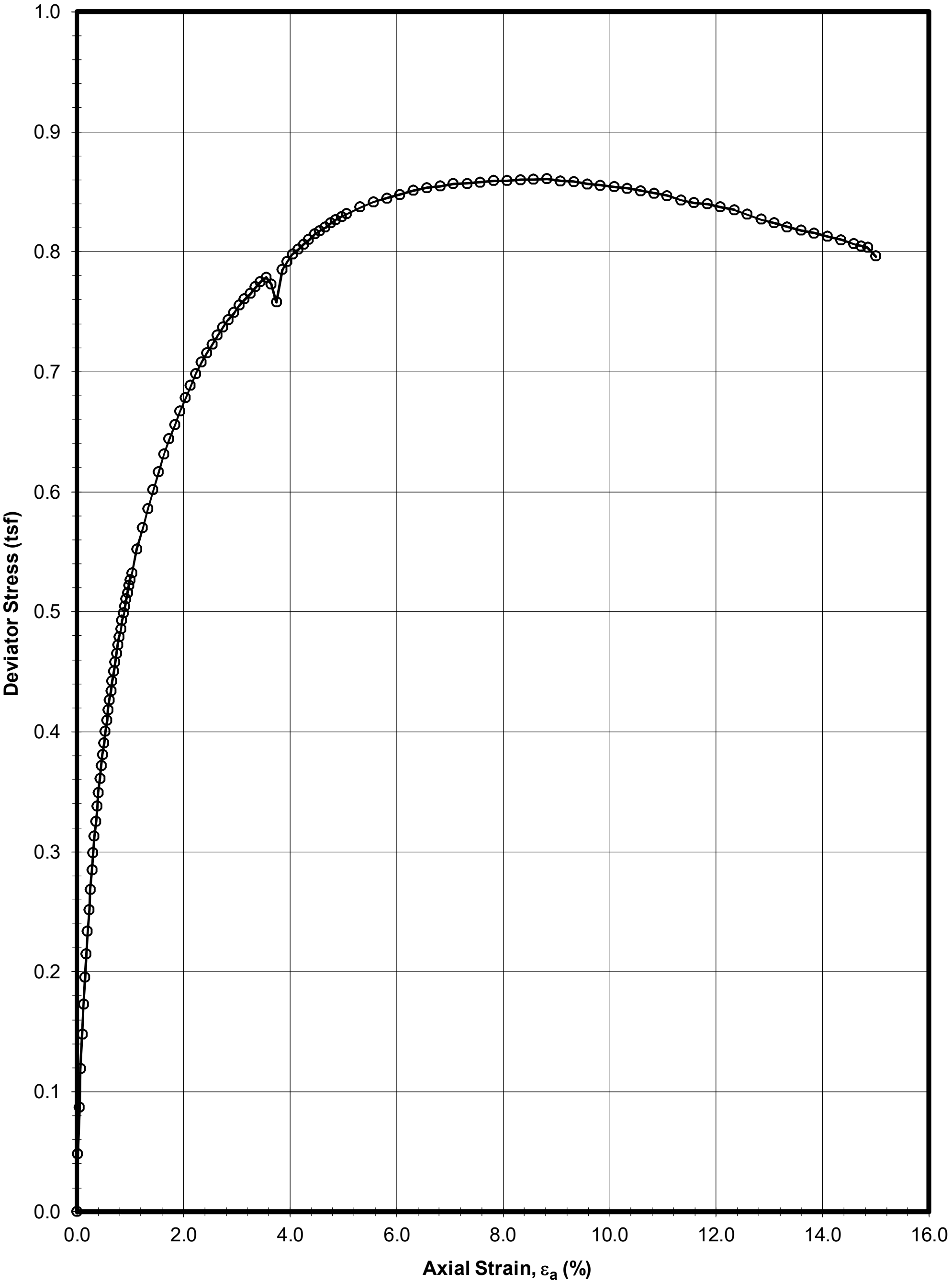
Sample: ST-4 - Depth: 8 ft.

GEOTECHNOLOGY
A Universal Engineering Sciences Company



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J034561.01
Boring: W-2
Sample: ST-11 - Depth: 38 ft.



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

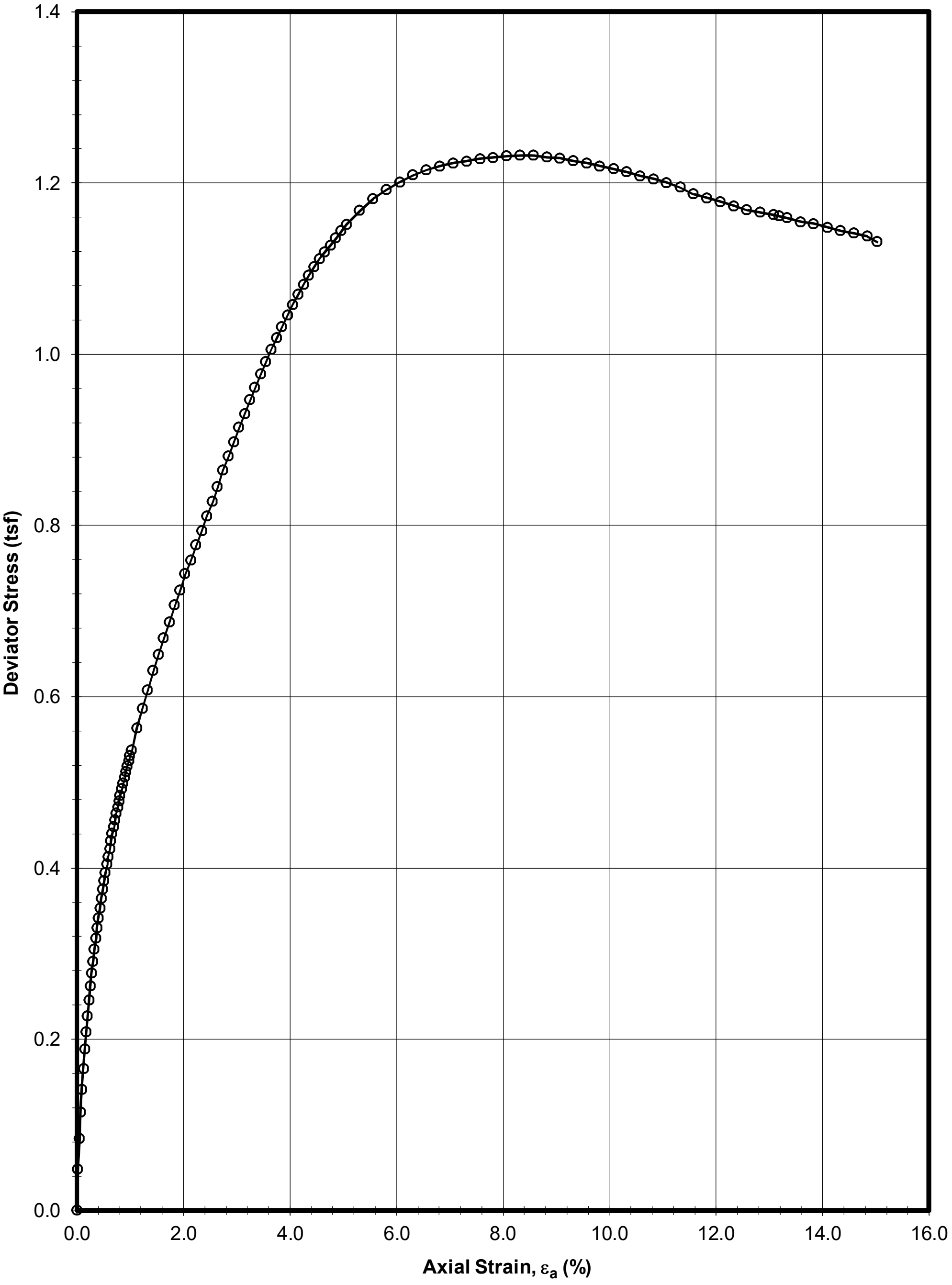
Project No.: J034561.01

Boring: W-2

Sample: ST-13 - Depth: 45 ft.

GEOTECHNOLOGY

A Universal Engineering Sciences Company



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

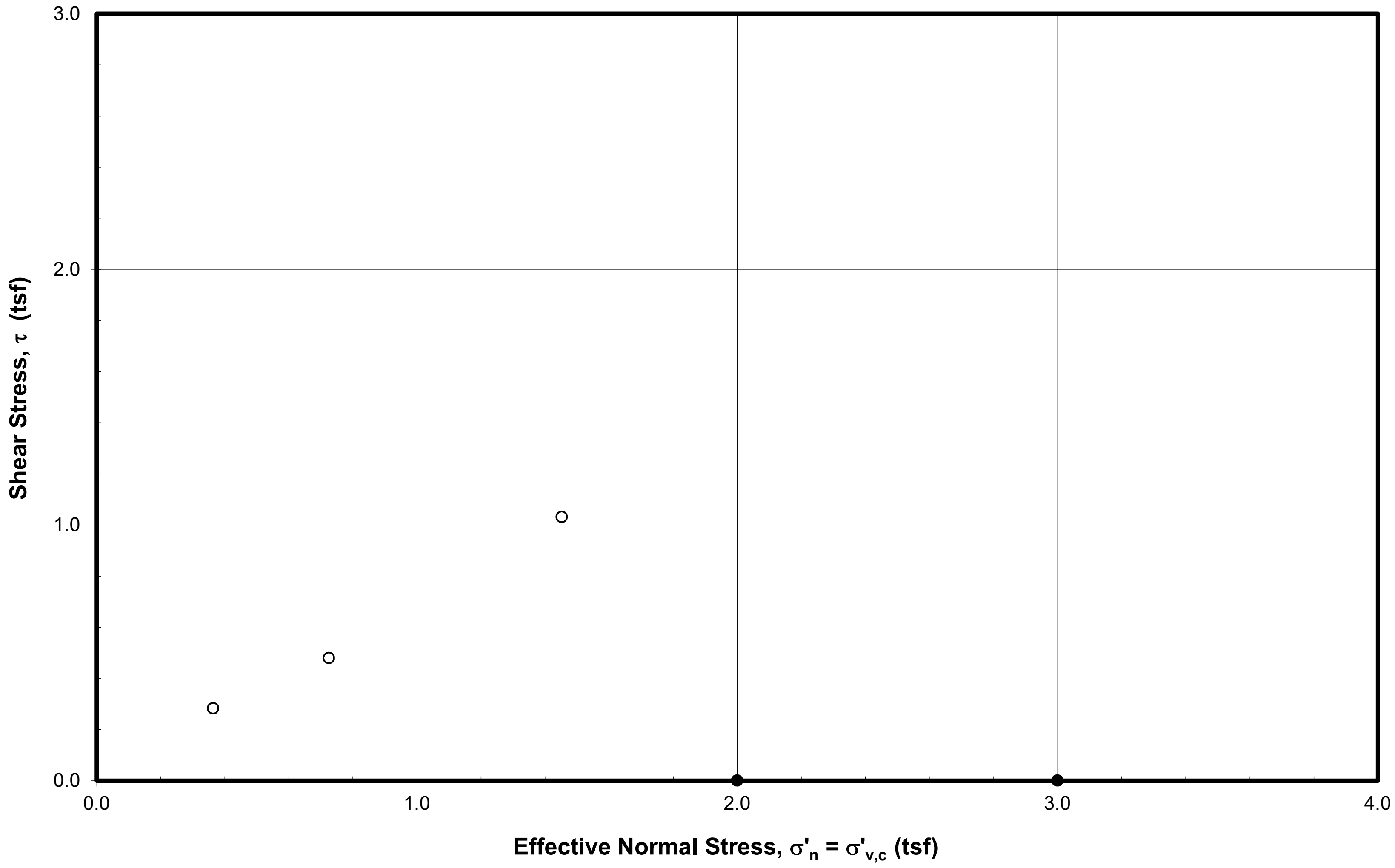
Project No.: J034561.01

Boring: W-2

Sample: ST-14 - Depth: 48 ft.

GEOTECHNOLOGY

A Universal Engineering Sciences Company



DRAINED DIRECT SHEAR TEST

ASTM D 3080

Boring: W-1 Sample: ST-4 -Depth: 8.0ft

TEST REPORT

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

Project No.: J034561.01 — **December 14, 2020**
Project Name: ARDOT 061615 — **Page 1 of 1**
Bridge No.: 01858
Boring Number: W-1
Sample ID: SS-5
Depth (ft): 13.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	24,500	0.57	13,965.00	11.9
#2	11,500	0.57	6,555.00	18.6
#3	10,500	0.57	5,985.00	25.2
#4	11,000	0.57	6,270.00	31.2

Minimum Soil Resistivity **5,985.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 14, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01858**	
Boring Number:	**W-1**	
Sample ID:	**SS-7**	
Depth (ft):	**23.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	23,500	0.57	13,395.00	10.5
#2	22,500	0.57	12,825.00	17.3
#3	16,000	0.57	9,120.00	24.2
#4	16,200	0.57	9,234.00	24.5

Minimum Soil Resistivity **9,120.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 14, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01858**	
Boring Number:	**W-1**	
Sample ID:	**SS-12**	
Depth (ft):	**43.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	22,000	0.57	12,540.00	18.0
#2	7,000	0.57	3,990.00	26.1
#3	1,700	0.57	969.00	32.6
#4	1,300	0.57	741.00	38.6
#5	1,320	0.57	752.40	48.7

Minimum Soil Resistivity **741.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 14, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01858**	
Boring Number:	**W-1**	
Sample ID:	**SS-16**	
Depth (ft):	**68.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	14,000	0.57	7,980.00	11.7
#2	7,500	0.57	4,275.00	18.9
#3	3,100	0.57	1,767.00	25.6
#4	2,800	0.57	1,596.00	27.8
#5	3,000	0.57	1,710.00	29.2

Minimum Soil Resistivity **1,596.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 14, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01858**	
Boring Number:	**W-2**	
Sample ID:	**SS-6**	
Depth (ft):	**18.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	12,000	0.57	6,840.00	11.2
#2	10,000	0.57	5,700.00	17.1
#3	11,000	0.57	6,270.00	24.1

Minimum Soil Resistivity **5,700.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 14, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01858**	
Boring Number:	**W-2**	
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	37,000	0.57	21,090.00	10.1
#2	23,000	0.57	13,110.00	17.6
#3	18,000	0.57	10,260.00	23.9
#4	16,000	0.57	9,120.00	26.1
#5	22,000	0.57	12,540.00	29.1

Minimum Soil Resistivity **9,120.00**

TEST REPORT

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

Project No.: J034561.01
Project Name: ARDOT 061615
Bridge No.: 01858
Boring Number: W-2
Sample ID: SS-16
Depth (ft): 63.5

December 16, 2020
Page 1 of 1

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	9,300	0.57	5,301.00	16.9
#2	2,800	0.57	1,596.00	22.3
#3	1,500	0.57	855.00	27.0
#4	1,400	0.57	798.00	33.8
#5	1,600	0.57	912.00	40.7

Minimum Soil Resistivity **798.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 16, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01858**	
Boring Number:	**W-2**	
Sample ID:	**ST-14**	
Depth (ft):	**48.0**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	8,000	0.57	4,560.00	17.4
#2	3,800	0.57	2,166.00	22.6
#3	1,850	0.57	1,054.50	29.1
#4	1,350	0.57	769.50	38.7
#5	1,200	0.57	684.00	44.6
#6	1,300	0.57	741.00	49.7

Minimum Soil Resistivity **684.00**

Appendix E

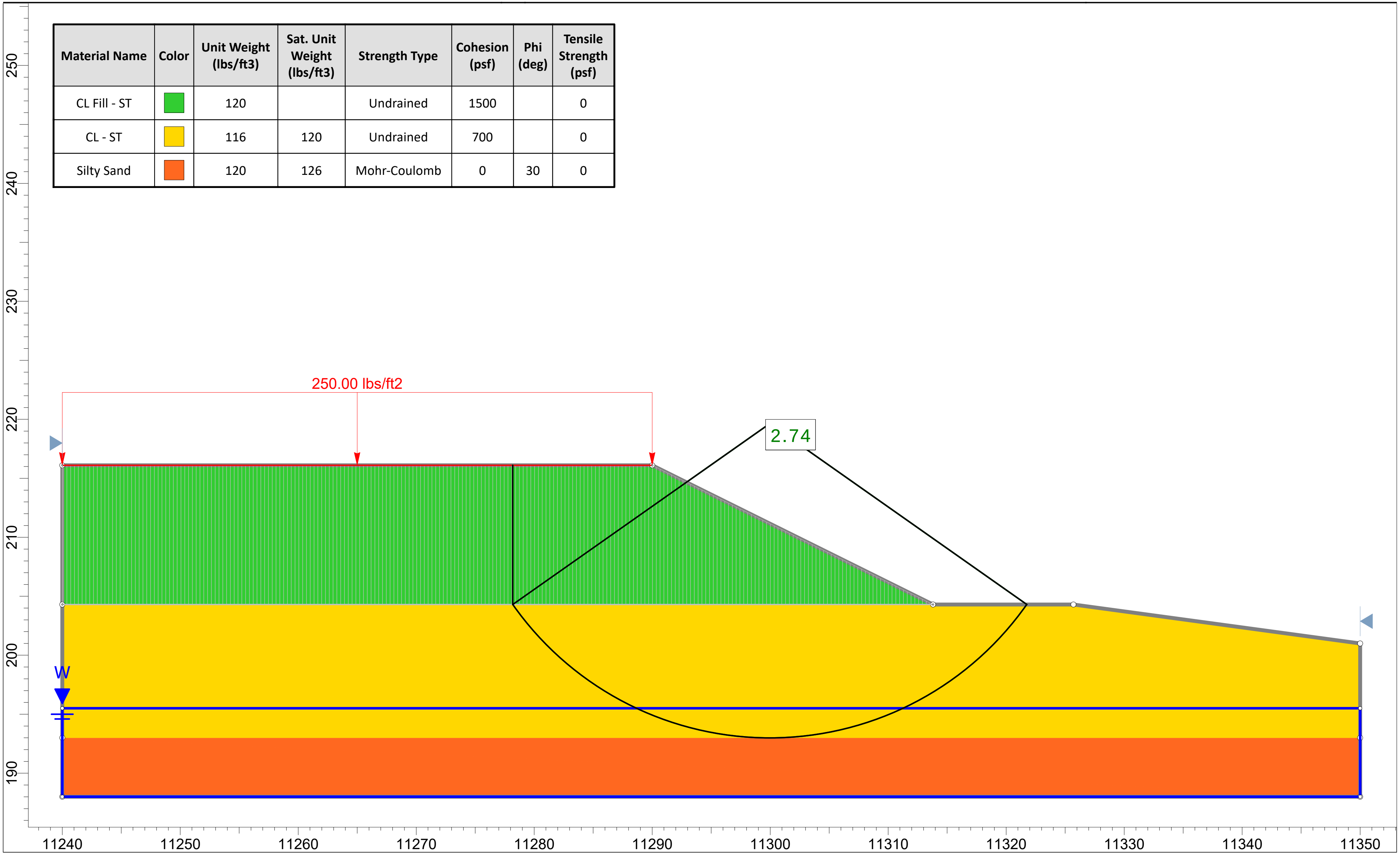
SELECTED GLOBAL STABILITY ANALYSES

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01858 Over Wolf Island Slash
Analysis Name: South Abutment Spill Slope
Analysis Description: Short Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500		0
CL - ST	🟨	116	120	Undrained	700		0
Silty Sand	🟧	120	126	Mohr-Coulomb	0	30	0

GEOTECHNOLOGY

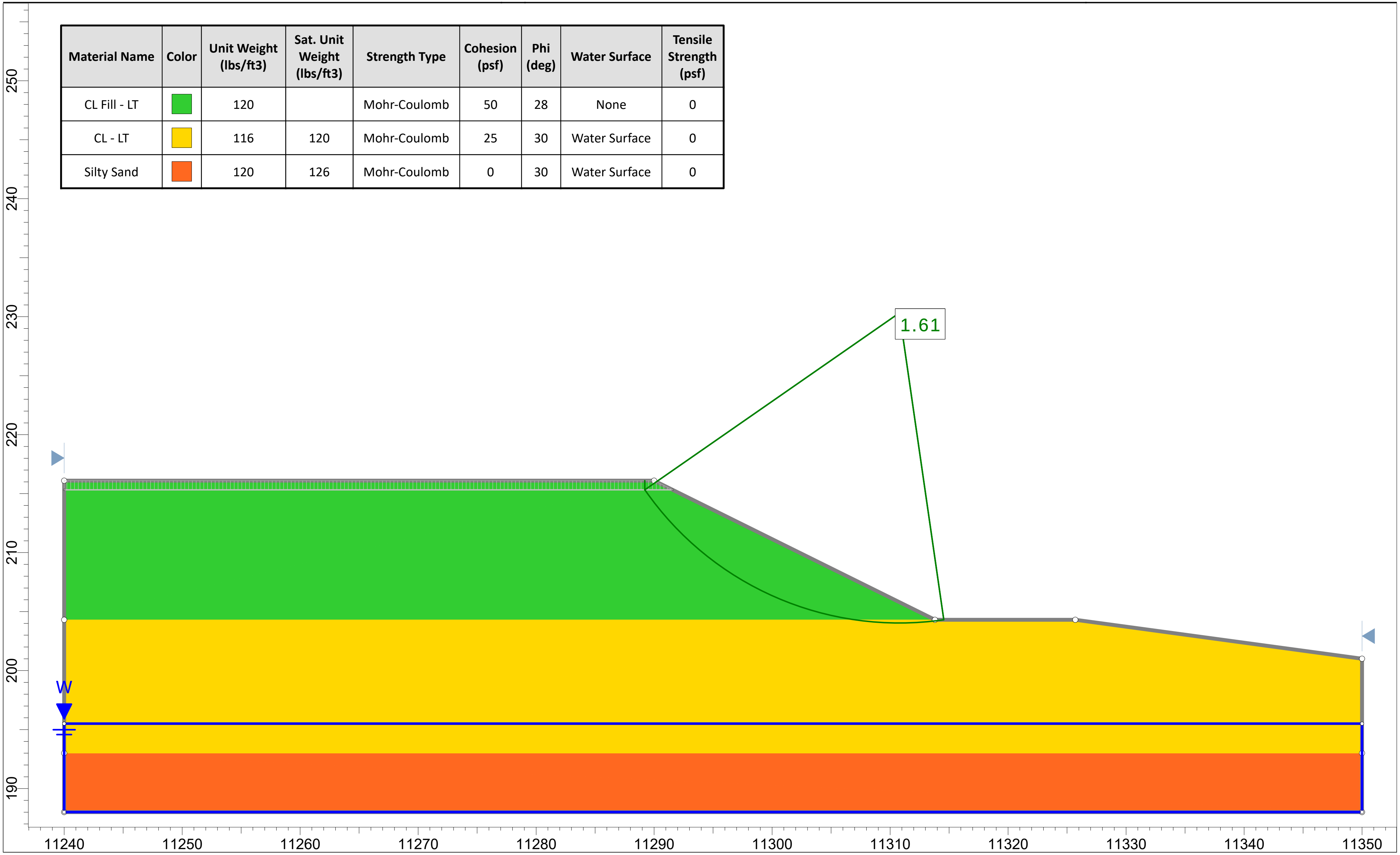
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01858 Over Wolf Island Slash
Analysis Name: South Abutment Spill Slope
Analysis Description: Long Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Tensile Strength (psf)
CL Fill - LT	🟩	120		Mohr-Coulomb	50	28	None	0
CL - LT	🟨	116	120	Mohr-Coulomb	25	30	Water Surface	0
Silty Sand	🟧	120	126	Mohr-Coulomb	0	30	Water Surface	0

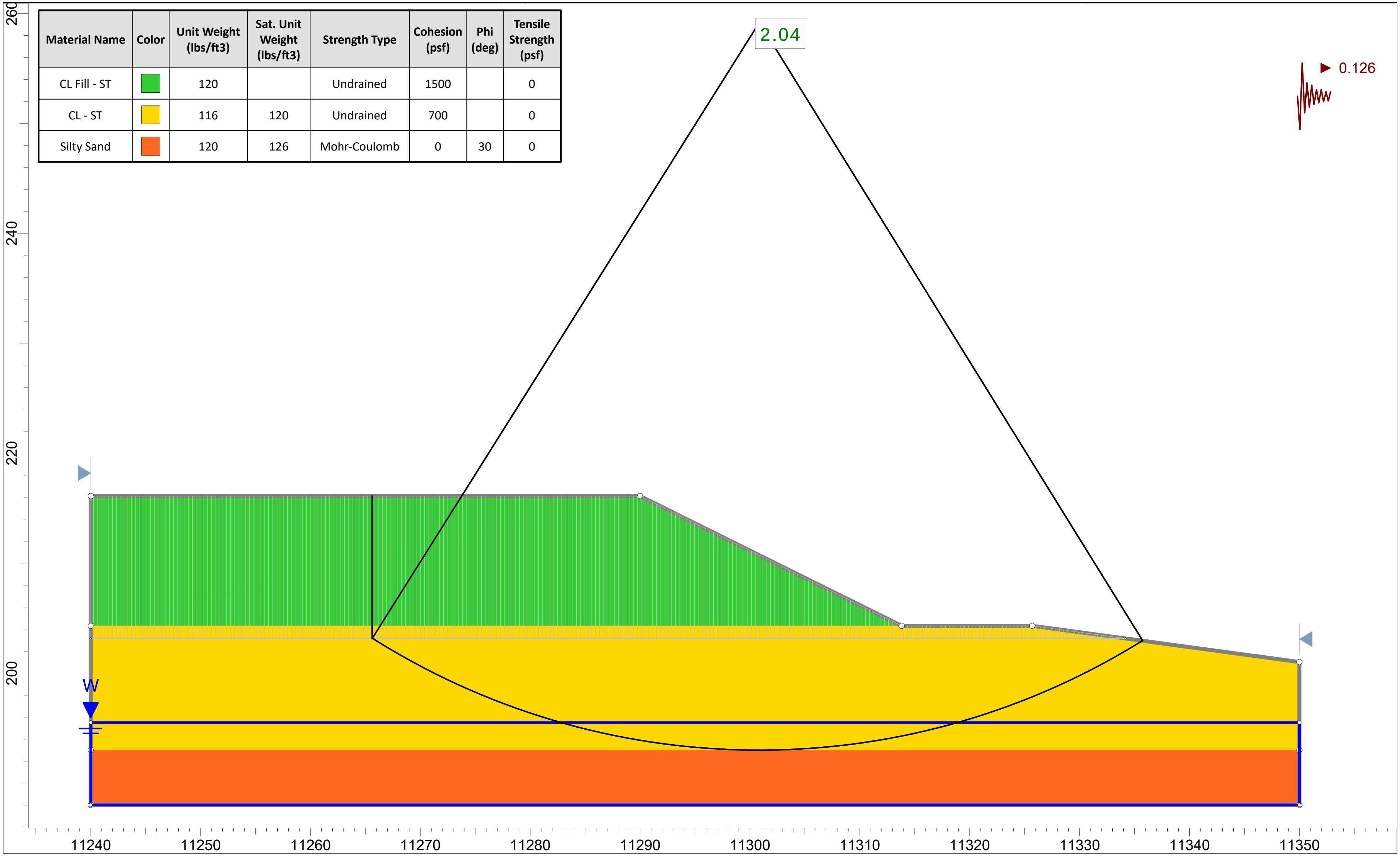


GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01858 Over Wolf Island Slash
Analysis Name: South Abutment Spill Slope
Analysis Description: Seismic
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032



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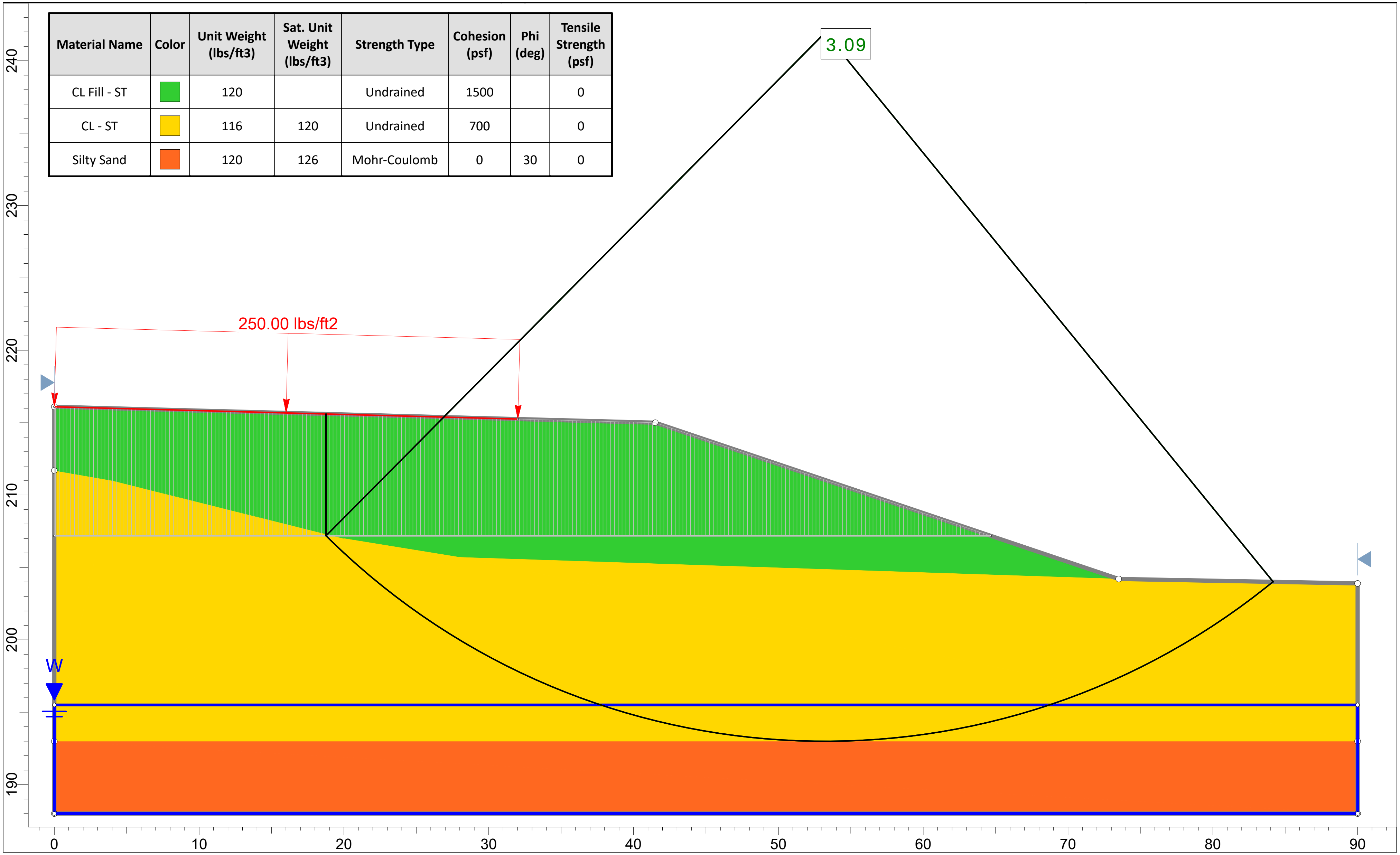
CL Fill - ST	🟩	120		Undrained	1500		0
CL - ST	🟨	116	120	Undrained	700		0
Silty Sand	🟧	120	126	Mohr-Coulomb	0	30	0

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01858 Over Wolf Island Slash
Analysis Name: South Abutment Side Slope Station 112+90
Analysis Description: Short Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032



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CL Fill - ST	🟩	120		Undrained	1500		0
CL - ST	🟨	116	120	Undrained	700		0
Silty Sand	🟧	120	126	Mohr-Coulomb	0	30	0

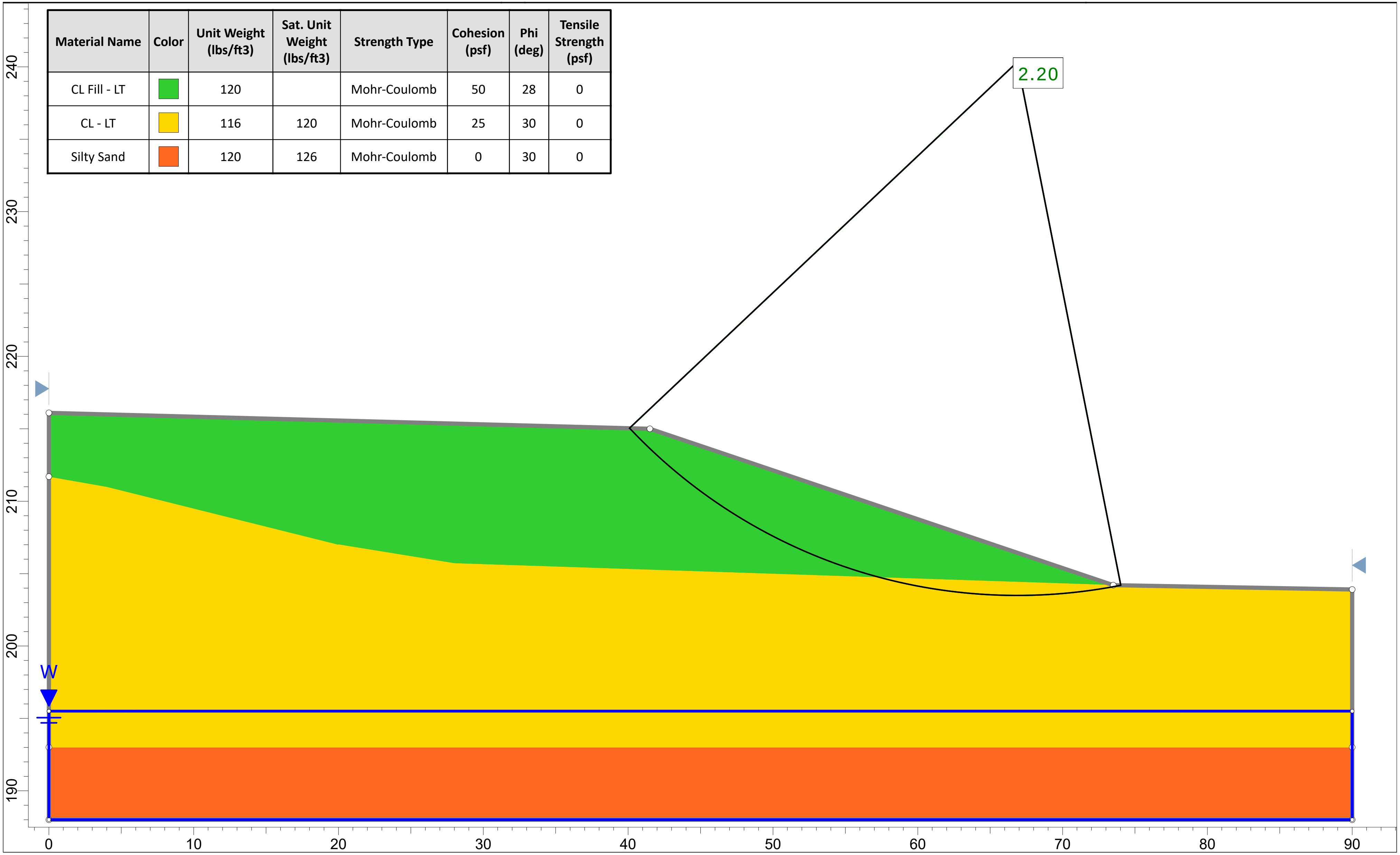
GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01858 Over Wolf Island Slash
Analysis Name: South Abutment Side Slope Station 112+90
Analysis Description: Long Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - LT	🟩	120		Mohr-Coulomb	50	28	0
CL - LT	🟨	116	120	Mohr-Coulomb	25	30	0
Silty Sand	🟧	120	126	Mohr-Coulomb	0	30	0



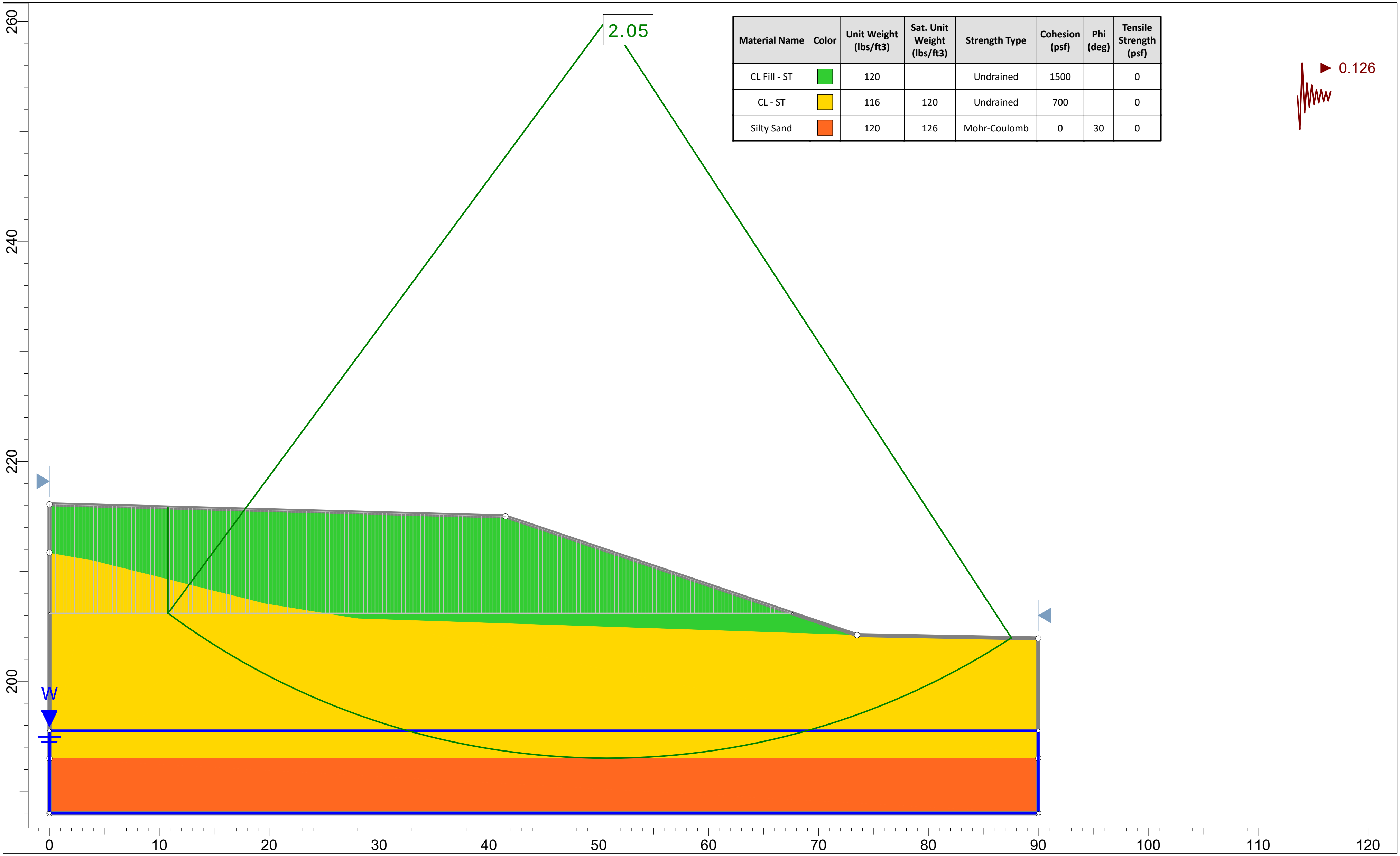
GEOTECHNOLOGY

A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01858 Over Wolf Island Slash
Analysis Name: South Abutment Side Slope Station 112+90
Analysis Description: Seismic
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500		0
CL - ST	🟨	116	120	Undrained	700		0
Silty Sand	🟧	120	126	Mohr-Coulomb	0	30	0

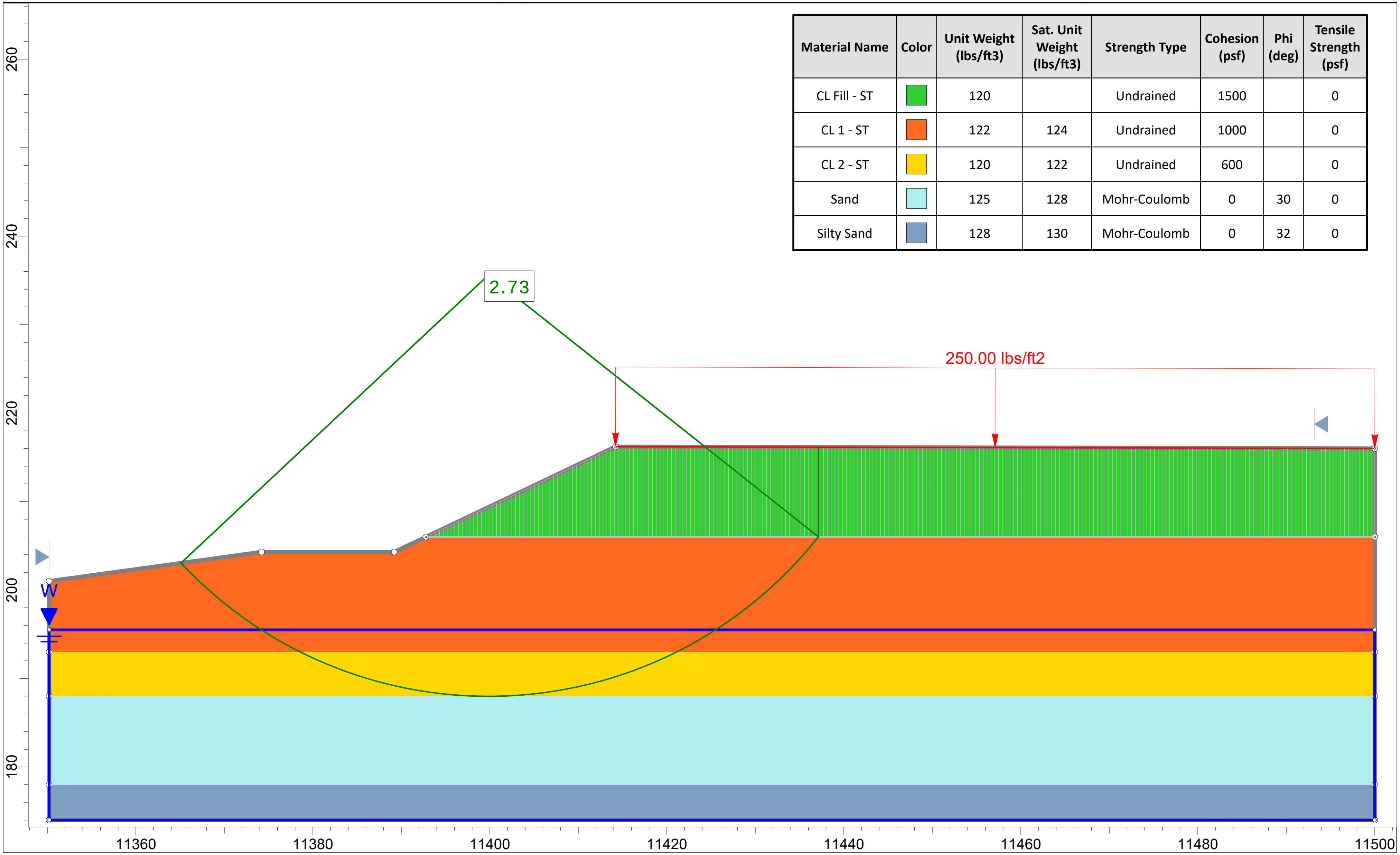
GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Birdge No. 01858 Over Wolf Island Slash
Analysis Name: North Abutment Spill Slope
Analysis Description: Short Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500		0
CL 1 - ST	🟧	122	124	Undrained	1000		0
CL 2 - ST	🟨	120	122	Undrained	600		0
Sand	🟦	125	128	Mohr-Coulomb	0	30	0
Silty Sand	🟦	128	130	Mohr-Coulomb	0	32	0

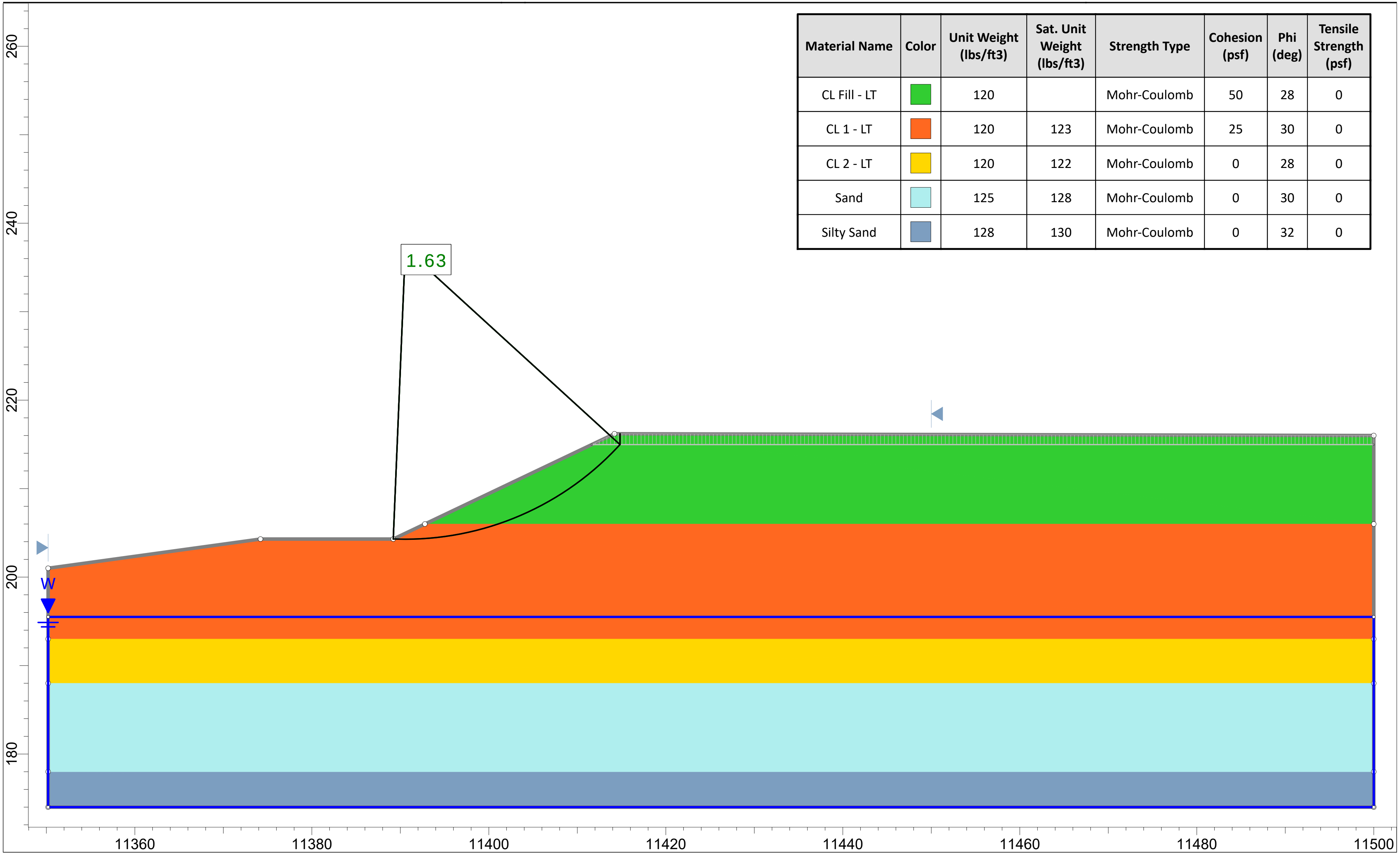


GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Birdge No. 01858 Over Wolf Island Slash
Analysis Name: North Abutment Spill Slope
Analysis Description: Long Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - LT	🟩	120		Mohr-Coulomb	50	28	0
CL 1 - LT	🟧	120	123	Mohr-Coulomb	25	30	0
CL 2 - LT	🟨	120	122	Mohr-Coulomb	0	28	0
Sand	🟦	125	128	Mohr-Coulomb	0	30	0
Silty Sand	🟦	128	130	Mohr-Coulomb	0	32	0

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Birdge No. 01858 Over Wolf Island Slash
Analysis Name: North Abutment Spill Slope
Analysis Description: Seismic
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

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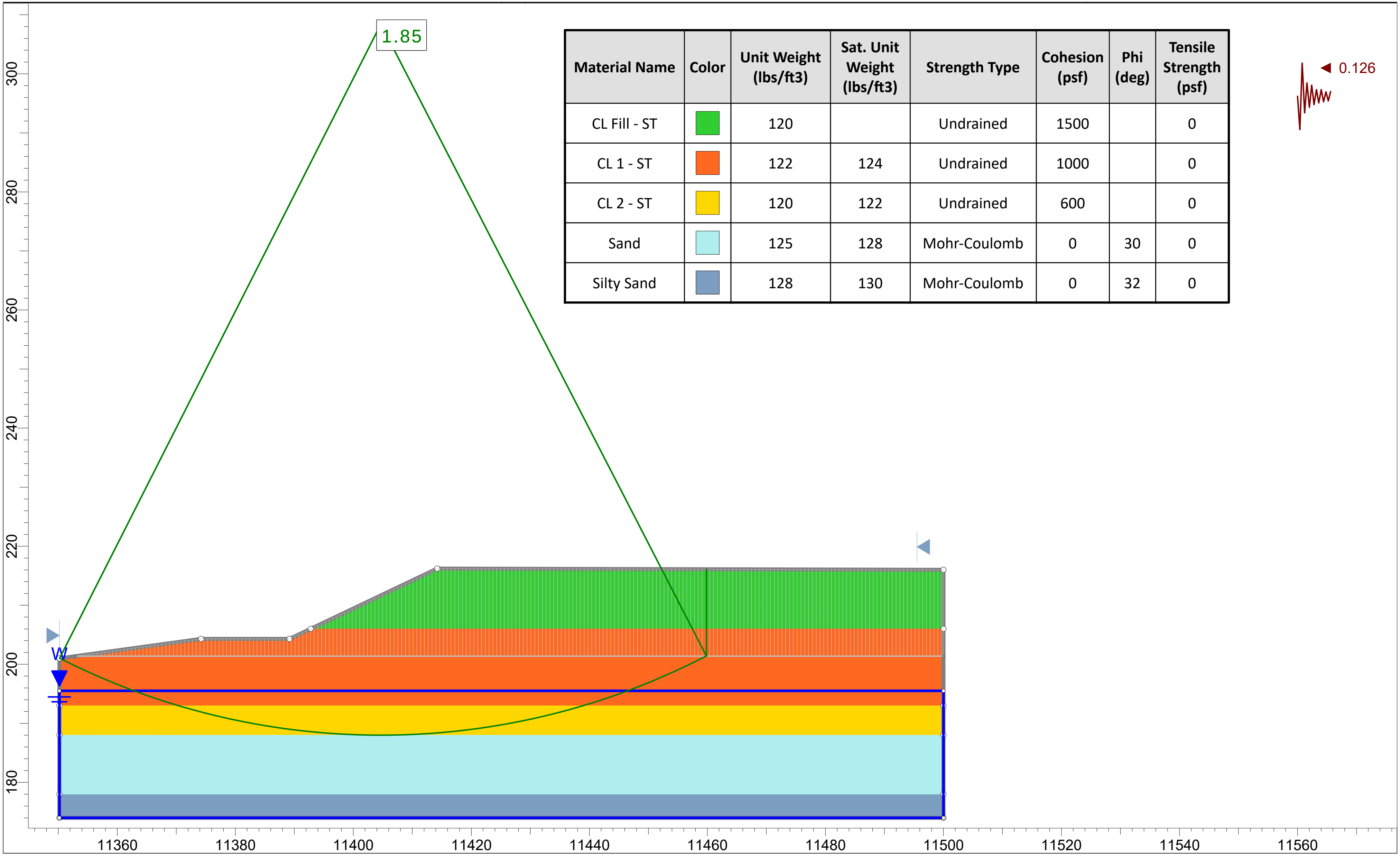


Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500		0
CL 1 - ST	🟧	122	124	Undrained	1000		0
CL 2 - ST	🟨	120	122	Undrained	600		0
Sand	🟦	125	128	Mohr-Coulomb	0	30	0
Silty Sand	🟦	128	130	Mohr-Coulomb	0	32	0

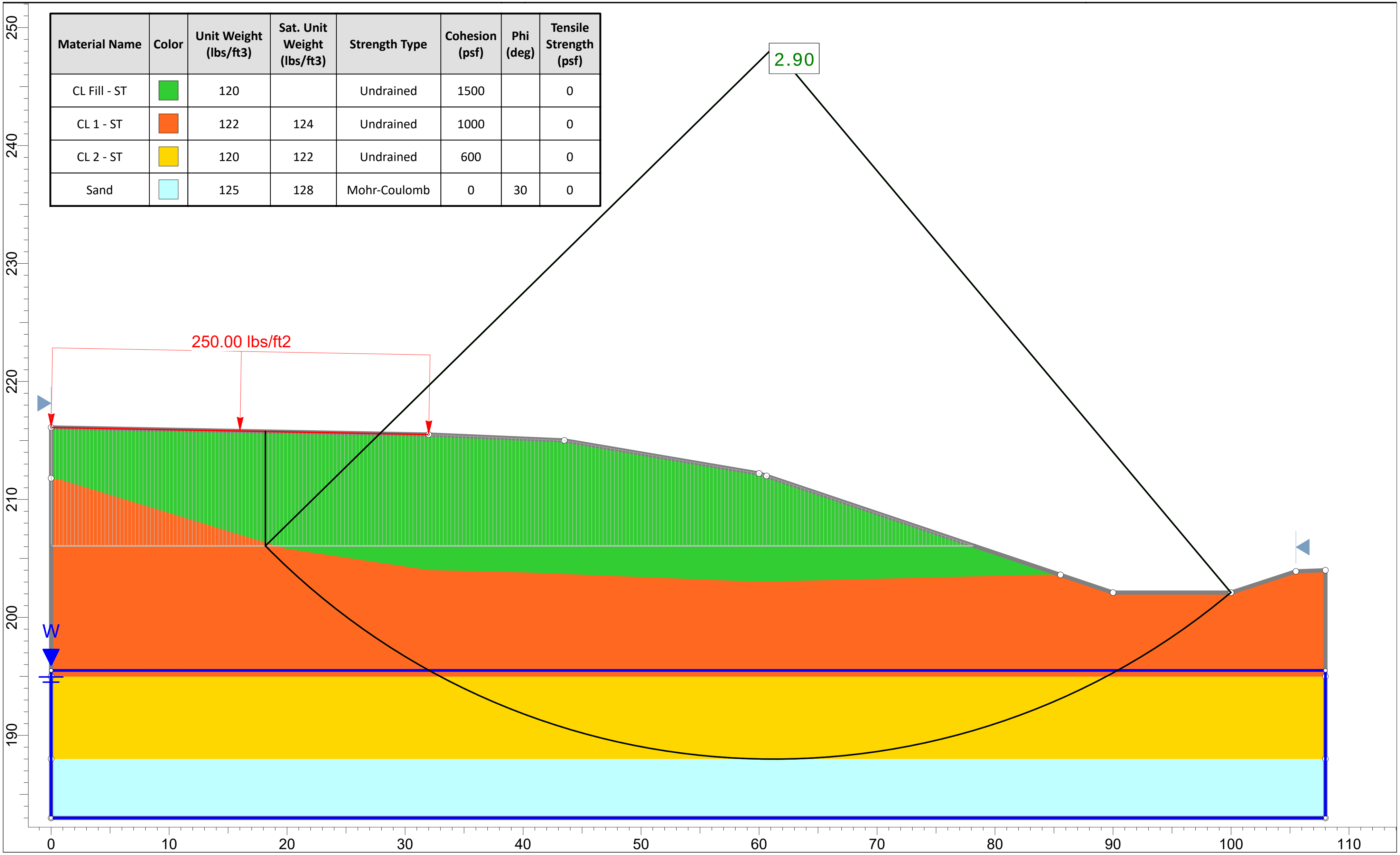
GEOTECHNOLOGY

A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01858 Over Wolf Island Slash
Analysis Name: North Abutment Side Slope Station 114+20
Analysis Description: Short Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500		0
CL 1 - ST	🟧	122	124	Undrained	1000		0
CL 2 - ST	🟨	120	122	Undrained	600		0
Sand	🟦	125	128	Mohr-Coulomb	0	30	0

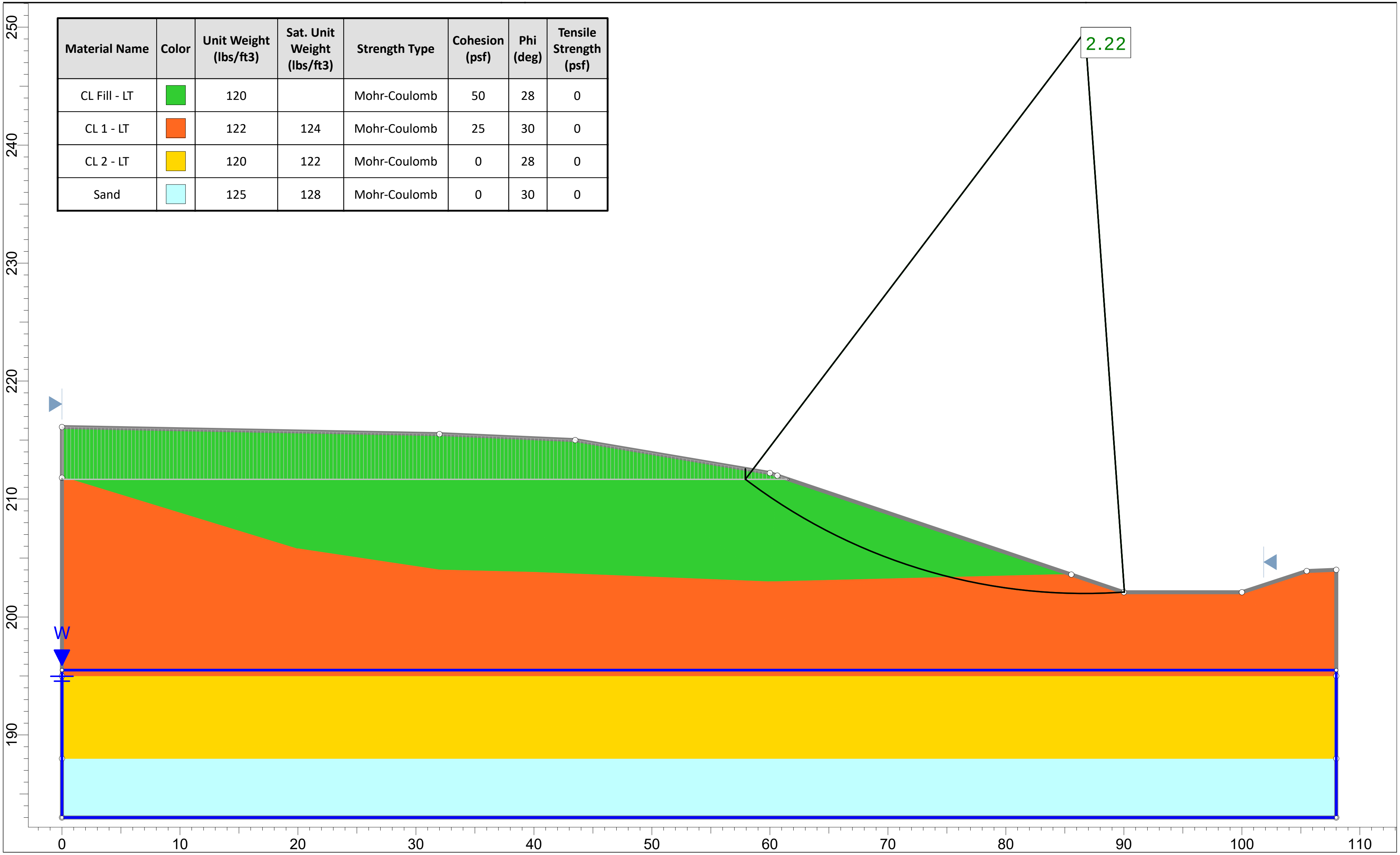
GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01858 Over Wolf Island Slash
Analysis Name: North Abutment Side Slope Station 114+20
Analysis Description: Long Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 3/1/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - LT	🟩	120		Mohr-Coulomb	50	28	0
CL 1 - LT	🟧	122	124	Mohr-Coulomb	25	30	0
CL 2 - LT	🟨	120	122	Mohr-Coulomb	0	28	0
Sand	🟦	125	128	Mohr-Coulomb	0	30	0



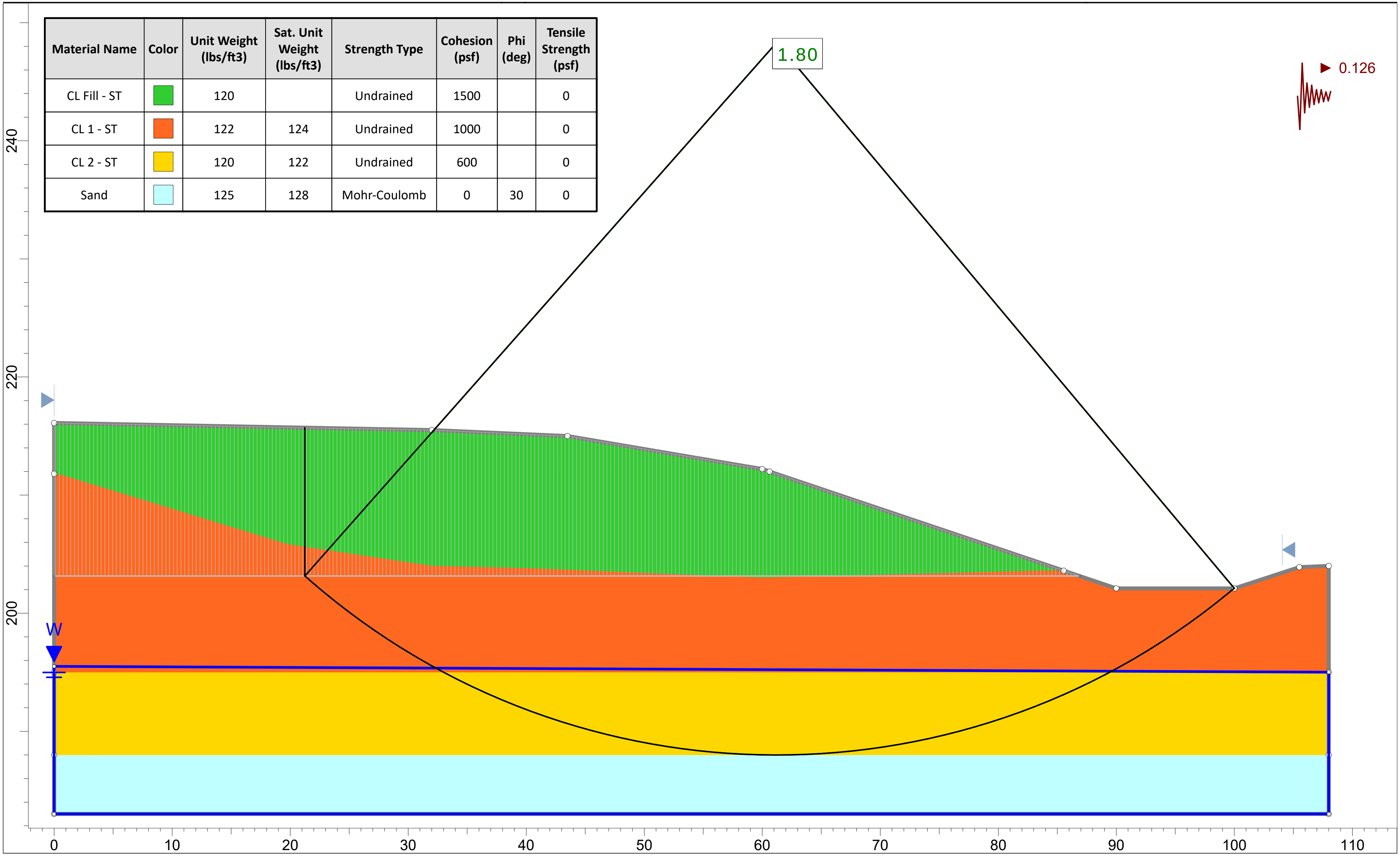
GEOTECHNOLOGY

A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01858 Over Wolf Island Slash
Analysis Name: North Abutment Side Slope Station 114+20
Analysis Description: Seismic
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032

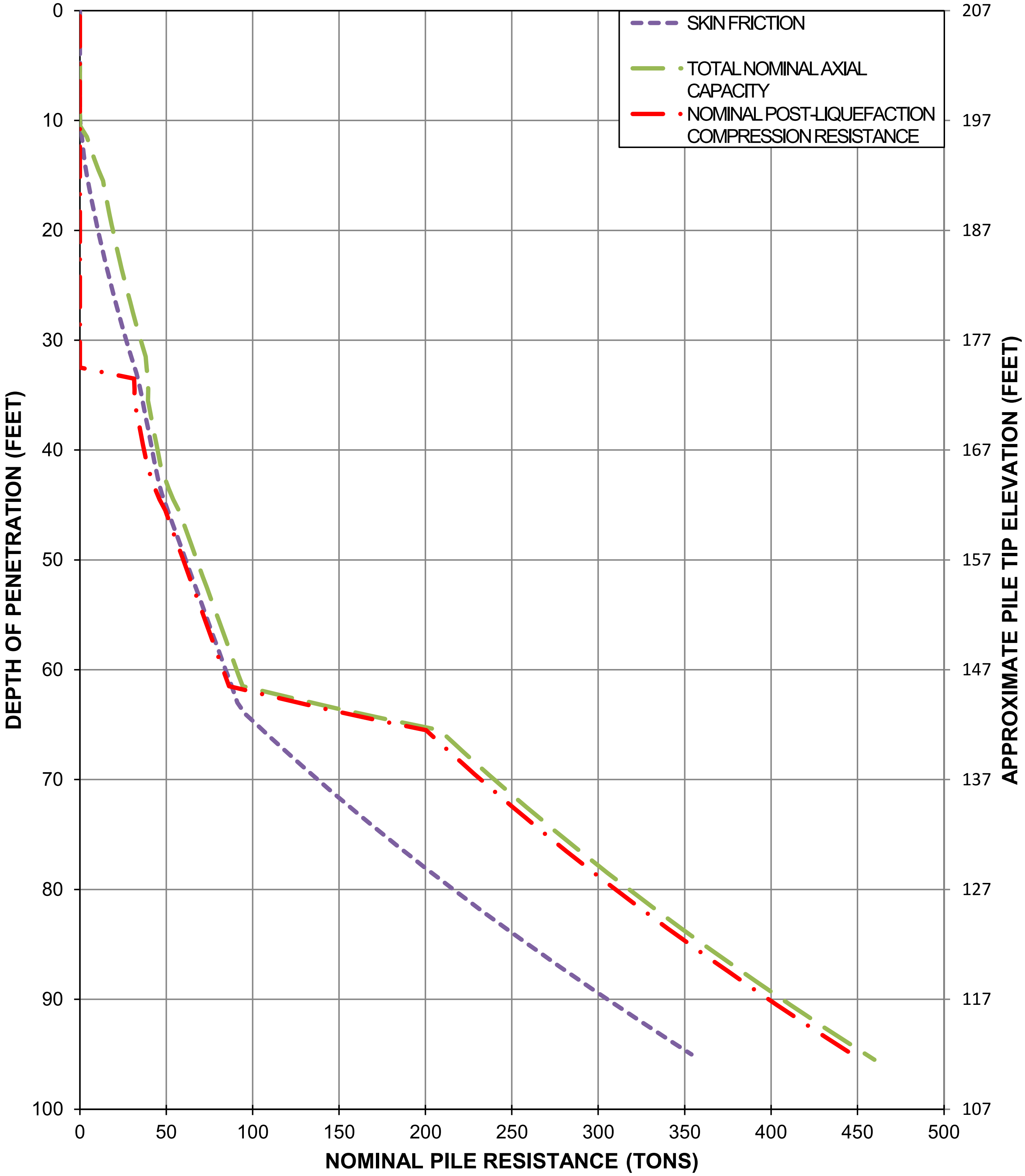


Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500		0
CL 1 - ST	🟧	122	124	Undrained	1000		0
CL 2 - ST	🟨	120	122	Undrained	600		0
Sand	🟦	125	128	Mohr-Coulomb	0	30	0

Appendix F
NOMINAL RESISTANCE CURVES FOR DRIVEN PILES

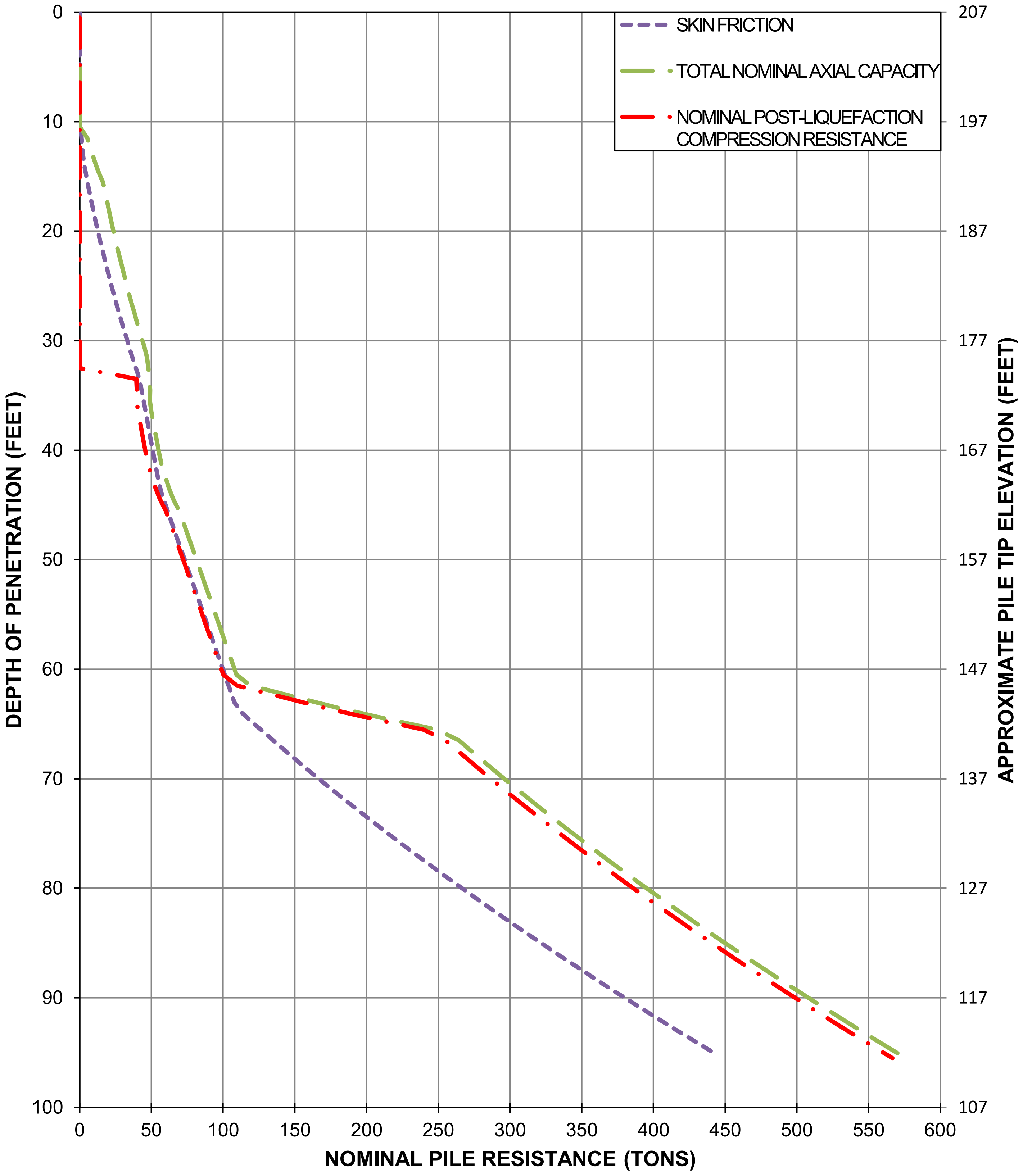
SOUTH ABUTMENT (BENT NO. 1)
HWY 63 BRIDGE NO. 01858 OVER WOLF ISLAND SLASH

NOMINAL RESISTANCE CURVES
DRIVEN 16-INCH, CLOSED-ENDED, PIPE PILES



SOUTH ABUTMENT (BENT NO. 1)
HWY 63 BRIDGE NO. 01858 OVER WOLF ISLAND SLASH

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

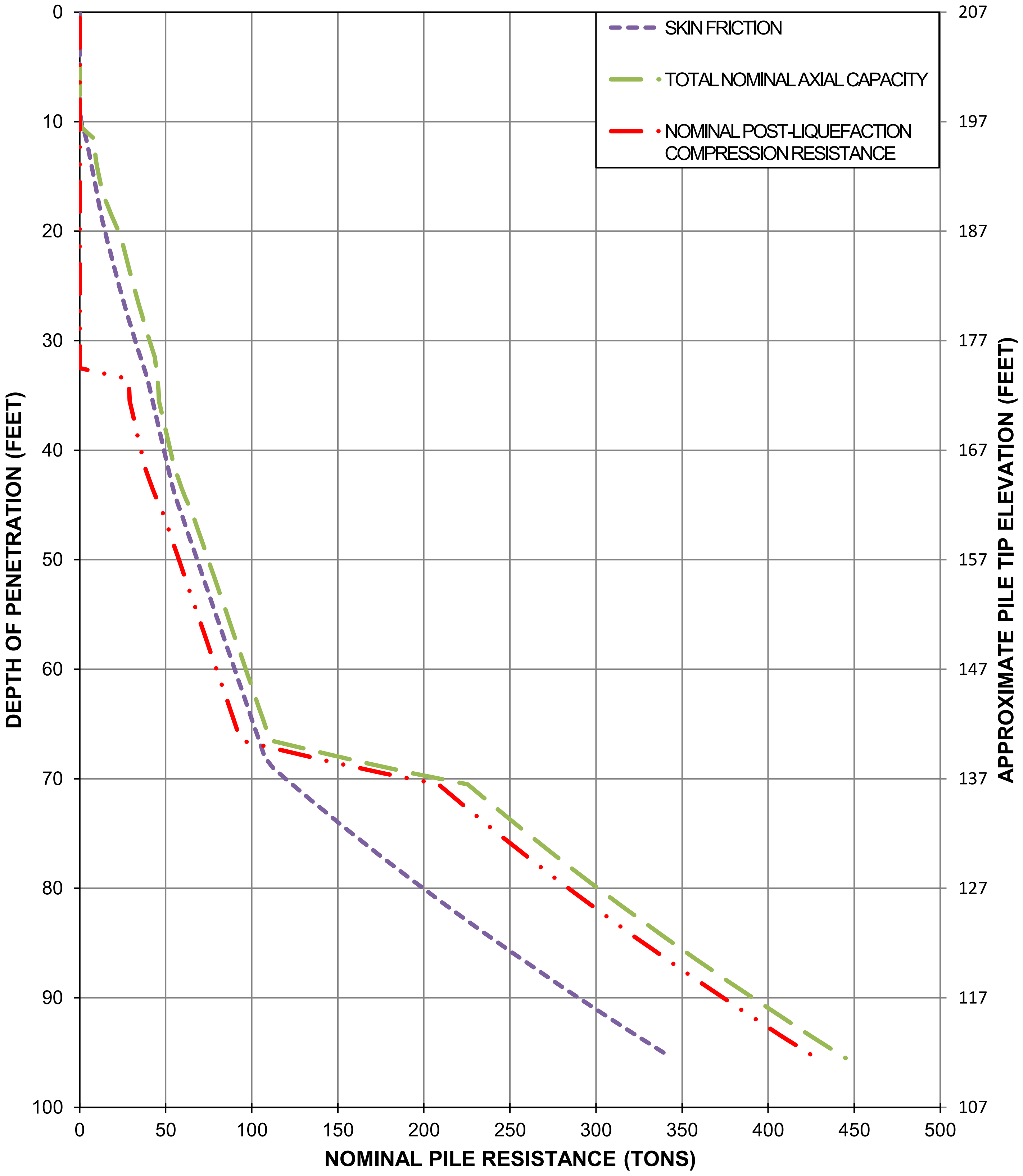


GEOTECHNOLOGY PROJECT NUMBER J034561.01

ARDOT 061615

NORTH ABUTMENT (BENT NO. 2)
HWY 63 BRIDGE NO. 01858 OVER WOLF ISLAND SLASH

NOMINAL RESISTANCE CURVES
DRIVEN 16-INCH, CLOSED-ENDED, PIPE PILES



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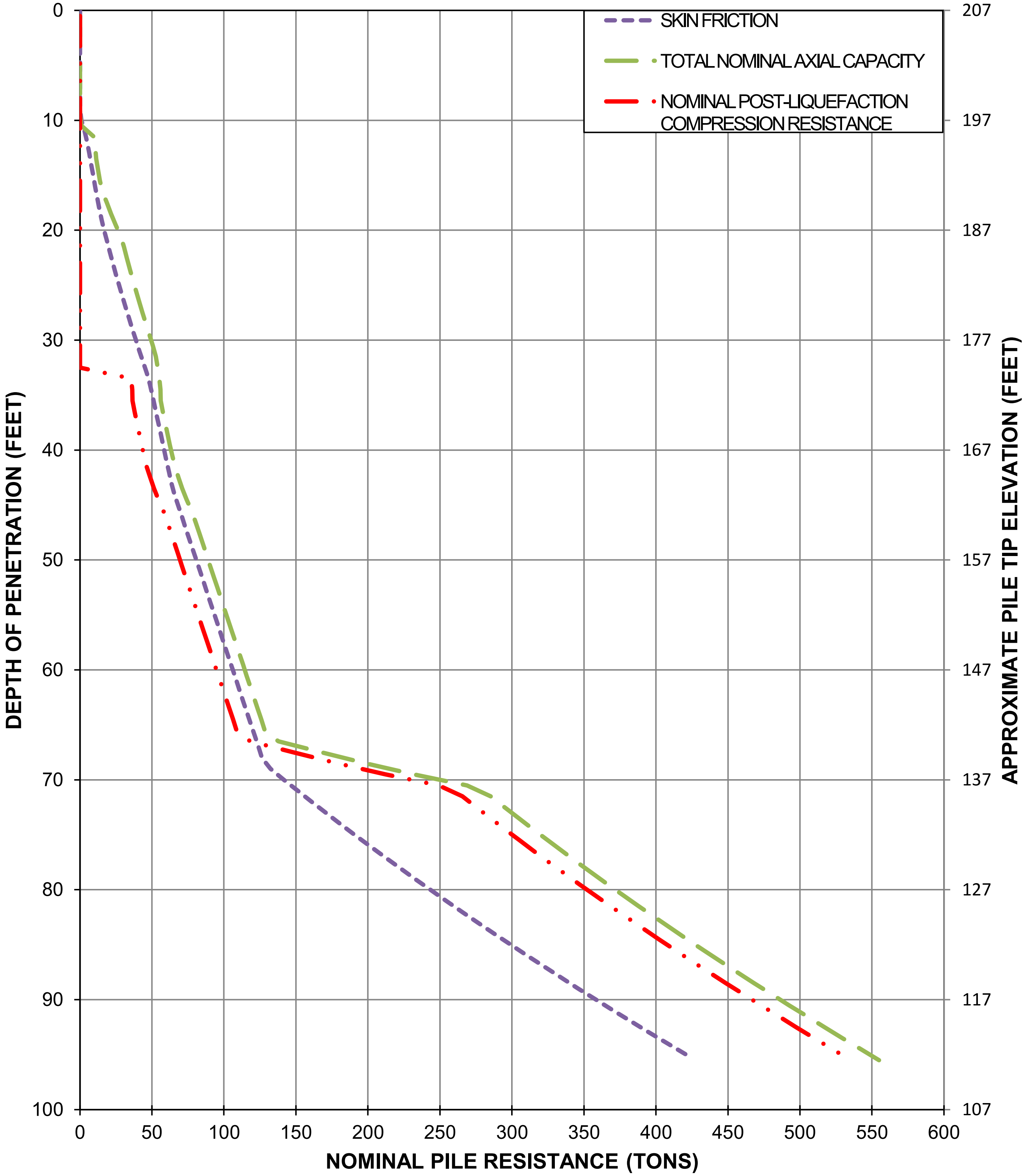
HWY 63 BRIDGE NO. 01858 OVER WOLF ISLAND SLASH

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



Appendix G

SOIL PARAMETERS FOR SYNTHETIC PROFILES

SOUTH ABUTMENT (BENT NO. 1) - BORING W-1

ZONE	SOIL TYPES	DEPTH[a] (ELEVATION)		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[D]		POST-LIQUEFACTION SHEAR STRENGTH PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)					
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)	SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	RESIDUAL COHESION (PSF)	RESIDUAL Φ’ (DEGREE)
1	Engineered Fill (Cohesive)	216[b]	206	120	1,200	--	50	28	0.007	500	1,200	--
2	Lean Clay	206	193	116	700	--	25	30	0.02	30	560	--
3	Silty Sand	193	174	120	--	30	--	30	--	20	--	7
4	Fat Clay	174	143	118	600	--	--	20	0.01	100	600	--
5	Sand with Silt	143	133	123	--	34	--	34	--	60	--	34
6	Sand with Silt	133	112	128	--	36	--	36	--	125	--	36

Note: Groundwater was not encountered in Boring W-1, but was encountered in Boring W-2 at approximate El 196. The effective unit weight should be used below the ground water level. Subtract 62.4 from the total unit weight to calculate the effective unit weight.

[a] Elevations are approximated from the provided drawing

[b] Approximate final grade at south abutment

[c] Pounds per cubic inch

[d] For lateral load analysis only

NORTH ABUTMENT (BENT NO. 2) - BORING W-2

ZONE	SOIL TYPES	DEPTH[a] (ELEVATION)		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[D]		POST-LIQUEFACTION SHEAR STRENGTH PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)					
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)	SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]	RESIDUAL COHESION (PSF)	RESIDUAL Φ' (DEGREE)
1	Engineered Fill (Cohesive)	216[b]	206	120	1,200	--	50	28	0.007	500	1,200	--
2	Lean Clay	206	193	122	1,000	--	25	30	0.01	100	800	--
3	Lean Clay	193	188	120	600	--	--	28	0.01	100	480	--
4	Sand with Silt	188	178	125	--	30	--	30	--	20	--	7
5	Silty Sand	178	174	128	--	32	--	32	--	20	--	7
6	Fat Clay	174	164	118	700	--	--	20	0.01	100	700	--
7	Fat Clay	164	138	122	1,200	--	--	20	0.007	500	1,200	--
8	Silty Sand	138	112	128	--	36	--	36	--	125	--	36

Note: Groundwater was encountered in Boring W-2 at approximate El 196. The effective unit weight should be used below the ground water level. Subtract 62.4 from the total unit weight to calculate the effective unit weight.

[a] Elevations are approximated from the provided drawing

[b] Approximate final grade at north abutment

[c] Pounds per cubic inch

[d] For lateral load analysis only

GEOTECHNICAL EXPLORATION

ROUTE 63 SECTION 11 STRUCTURES AND APPROACHES (S) – BRIDGE NO. 01859

PRAIRIE COUNTY, ARKANSAS

ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT NO. 061615
FEDERAL AID PROJECT NO. 9990

Prepared for:

GARVER USA
NORTH LITTLE ROCK, ARKANSAS

Prepared by:

GEOTECHNOLOGY, LLC
MEMPHIS, TENNESSEE

Date:

MARCH 23, 2023

Geotechnology Project No.:

J034561.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

GEOTECHNOLOGY
A Universal Engineering Sciences Company

March 23, 2023

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

Re: Geotechnical Exploration
ArDOT 061615
Route 63 Section 11 Structures and Approaches (S) – Bridge No. 01859
Prairie County, Arkansas
Geotechnology Project No. J034561.01

Dear Mr. Ruddell:

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

Amber Meadows

Amber Meadows
Project Engineer

Dale M. Smith

Dale M. Smith, P.E.
Senior Project Manager

ABM/DBA/DMS:abm/dms

Copies submitted: Client (email)

No. 17917
Dale M. Smith

Dale M. Smith

3/23/23

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

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Geotechnical Exploration
ROUTE 63 SECTION 11 STRUCTURES AND APPROACHES (S) – BRIDGE NO. 01859
Prairie County, Arkansas
March 23, 2023 | Geotechnology Project No. J034561.01

1.0 SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design, construction, and other related features for the proposed approach improvements and bridge replacement over La Grue Bayou along Highway 63 in Prairie County, Arkansas. The referenced features include demolition of the existing bridge and construction of a new bridge (Structure No. 01859). It is our understanding the anticipated foundation type for support of the new bridge is driven, closed-ended, pipe piles. The existing bridge approaches will be modified to facilitate traffic flow over the new bridge. A general overview of the project extents is shown on Figure 1 included in Appendix B.

The recommendations presented in this report are based on the geology, topography, and the results of the geotechnical exploration. Results of borings, in-situ testing, sampling, and laboratory testing are included in the report. A total of 2 borings were drilled in the vicinity of the site as shown on Figure 2 included in Appendix B. The boring logs, along with field and laboratory test results, are enclosed. The collected data have been analyzed and the physical properties of the in-situ soils summarized. General site conditions are discussed, along with recommendations for foundations and subgrade preparation. Important information prepared by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association for studies of this type is presented in Appendix A for your review.

2.0 GENERAL INFORMATION

Planned Modifications

The existing 2-lane, 101.5-foot long, 27.4-foot wide, 4-span bridge supported on precast concrete piles over La Grue Bayou (south) will be replaced with a 4-lane, 110-foot long, 77.5-foot wide, single-span bridge. The new bridge will be constructed east of the existing bridge. Riprap is planned along the abutment slopes based on the provided plans[1]; abutment slopes are anticipated to be two horizontal units for every vertical unit (2H:1V) and side slopes at the approaches are anticipated to be 3H:1V. Intersections of access drives will be modified to

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[1] *Arkansas Department of Transportation Construction Plans for State Highway, La Grue Bayou, Wolf Island Slash & Honey Creek STRS. & APPRS. (S), Prairie County, Route 63 Section 11, Route 33 Section 5, Job 061615*, provided by ARDOT on September 28, 2020.

accommodate the new alignment. Up to approximately 4 and 11 feet of cut and fill, respectively, will be required to achieve design grades.

Drainage

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

Physiographic Setting & Geology

Prairie County is located in east-central Arkansas, in the Mississippi Embayment. The Mississippi Embayment is a trough-like depression containing thousands of feet of sediment and plunging southward along an axis approximating the present course of the Mississippi River. The deposits in the area consist of Holocene epoch alluvial gravel and sand. These materials are typically white to brown or gray, poorly to well sorted, fine- to coarse-quartz sand and gravel with minor silts and clays. These deposits form a broad terrace among the west side of the Mississippi River flood plan, and include both glacial outwash and non-glacial alluvium. Thickness can vary from 3 to 40 meters and may include loessal colluvium from nearby Crowley's Ridge.

3.0 GEOTECHNICAL EXPLORATION

The borings were drilled between September 16th and 17th, 2020 with a rotary drill rig (CME 75) using hollow-stem auger and wash rotary drilling methods. The borings were drilled to an approximate depth of 100 feet. 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 automatic hammers. Thin-walled Shelby tube samples were collected in cohesive soils at selected depths. Groundwater observations were made during drilling operations.

The collected samples were visually examined by field staff and transported to our laboratory for further evaluation and testing. The samples were examined in the laboratory by a geotechnical professional who prepared descriptive logs of the materials encountered. The boring logs are presented in Appendix C. An explanation of the terms and symbols used on the boring logs is also provided in Appendix C. Included on each boring log are ground surface elevation, station and offset provided by representatives of ARDOT. 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

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, resistivity, consolidated-drained direct shear, unconsolidated-undrained triaxial compression (UU), and one-dimensional consolidation test results are also provided in Appendix D. The laboratory tests and corresponding test method standards are presented in Table 2.

Table 2. Summary of Laboratory Tests and Methods.

Laboratory Test	ASTM	AASHTO
Moisture Content	D 2216	T 265
Atterberg Limits	D 4318	T 98
Grain Size Analysis by Sieving	D 6913	T 88
Unconsolidated-Undrained Triaxial Compression	D 2850	T 296
Consolidated-Drained Direct Shear	D 3080	T 236
One-Dimensional Consolidation	D 2435	T 216
Soil Electrical Resistivity	G 57	T 288
Soil pH	D 4972	T 289

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

5.0 SUBSURFACE CONDITIONS

General Subsurface Conditions

The borings at this site include Borings L-1 and -2 drilled in the northbound shoulder of the existing, southern and northern approaches, respectively. Beneath the approximately 8- (Boring L-1) and 6-inch (Boring L-2) thick layer of asphalt and 4-inch thick cement-treated base material, the soils generally consisted of fine-grained soil underlain by coarse-grained soil layers to the 100-foot maximum depth of exploration. More detailed descriptions of the stratigraphy encountered at each bridge are included below and on the boring logs in Appendix C.

Fine-Grained Soil Layers. Underlying the pavement materials, the stratigraphy generally consisted of predominantly fine-grained soils underlain by predominantly coarse-grained soil at a depth of approximately 88 feet and extended to the 100-foot maximum depth of exploration in both borings. The fine-grained soils were classified as lean clay (CL), fat clay (CH), and silt (ML) by the Unified Soil Classification System (USCS) and A-6, A-7-5, A-7-6, or A-4 by the AASHTO classification method. The fine-grained soils were soft to very stiff based on SPT N-values, and medium stiff to very stiff based on UU tests. The laboratory testing used to determine USCS and AASHTO classifications are presented in Appendix D.

The underlying, coarse-grained soils were classified as silty sand (SM by USCS, A-1-a, A-1-b, A-2-4, A-2-6, or A-4 by AASHTO). Based on field test results, the coarse-grained soils were dense to very dense. Very dense sand (N > 50 bpf) was encountered in Boring L-1 from approximate depths of 88 to 100 feet (approximately El 124 to El 113) and in Boring L-2 from approximate depths of 98 to 100 feet (approximately El 111 to El 113).

Soil pH and Resistivity Test Results

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 to 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 below and are included in Appendix D.

Table 3. Results of Soil Resistivity Testing.

Boring	Sample No.	Sample Depth (ft.)	pH	Soil Resistivity (ohm-cm)
L-1	SS4	8.5	3.86	8,265
	ST-8	23	7.43	713
	SS12	43.5	8.22	684
	SS15	58.5	8.30	462
L-2	ST-4	8	4.53	4,047
	ST-6	18	4.57	3,135
	SS11	38.5	8.23	855
	ST-14	50	8.29	542
	SS17	63.5	8.31	855

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.
- pH less than 5.5.

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

- Resistivity less than 3,000 ohm-cm.
- pH less than 5.5.

Results of the corrosion and deterioration testing indicate the site has potential for steel pile or steel reinforcement deterioration. Interpretation of the data and corrosion protection of the bridge structural components should be performed by the design team.

Groundwater

Groundwater was encountered at an approximate depth of 19½ feet during drilling operations in Boring L-2. The presence of higher groundwater levels in Boring L-1 could have been obscured by the use of mud rotary drilling methods, which introduces fluid to the borehole. Groundwater levels could vary significantly over time due to water levels in La Grue Bayou and seasonal variation in precipitation, recharge, 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 1V:4H must be benched prior to placing new fill. Slope ratios of 1V:3H or flatter are recommended for all cut and fill slopes along the proposed alignment, based on the results of global stability analyses (discussed in a subsequent section).

Cut Areas. After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 95% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D 698/AASHTO T 99). Areas supporting pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

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

Fill and Backfill Placement. Fill and backfill should be placed in level lifts up to 8 inches in loose thickness. For fill and backfill exhibiting a well-defined moisture-density relationship, each lift should be moisture-conditioned to within ±2% of the optimum moisture content and compacted with a sheepsfoot roller or self-propelled compactor to a minimum of 98% of the maximum dry unit weight as determined by the standard Proctor test. Moisture-conditioning can include: aeration and drying of wetter soils, wetting drier soils, and/or mixing wetter and drier soils into a uniform blend. The upper three feet of fill and backfill 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 at least 70% of the relatively density as evaluated from the maximum and minimum index densities measured by ASTM D4253 and D4254, respectively. The upper 3 feet of fill and backfill beneath the base of pavement should be compacted to at least 75% of the relatively density.

Moisture Considerations. The soils encountered in the borings are relatively wet and will most likely require drying. The time for drying will depend on the weather conditions during grading activities. We recommend construction take place during dry weather conditions. Wet weather conditions can cause rutting of the surficial soils which will require drying and recompacting.

Maintaining the moisture content of bearing and subgrade soils within the acceptable range is important during and after construction for the proposed structure. 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.

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

Seismic Considerations

Earthquake Risk. The project area is located within the New Madrid Seismic Zone (NMSZ). The NMSZ is located in the northern part of the Mississippi Embayment and trends in a northeast to southwest direction from southern Illinois to northeast Arkansas. In December 1811, a series of large magnitude earthquakes occurred, which were centered near New Madrid, Missouri. Three strong earthquakes occurred over a 3-month period 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”, eighth edition (2017). A site modified peak ground acceleration (A_s) of 0.255g was obtained from published values.

Seismic Design Parameters. Presented in Table 4 are seismic design parameters based on a seismic hazard with 7% probability of exceedance in 75 years and the field and laboratory testing results.

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

Latitude 34.708138°N/Longitude 91.555041°W		
Category/ Parameter	Designation/ Value	Reference
Seismic Zone	2	AASHTO LRFD 2017 Table 3.10.6-1
Seismic Site Class	D	AASHTO LRFD 2017 Table 3.10.3.1-1
S_S	0.383g	Ground motion parameters obtained from the computer program supplied with the AASHTO Guidelines for the Seismic Design of Highway Bridges (2009) using the indicated latitude and longitude coordinates of the project site and the seismic site class based on boring data.
S_1	0.112g	
F_a	1.494	
F_v	2.351	
F_{PGA}	1.447	
t_s	0.462	
t_0	0.092	
S_{DS}	0.572g	
S_{D1}	0.264g	
PGA	0.176g	
As	0.255g	

Liquefaction and Dynamic Settlement. A study was performed to evaluate the liquefaction and dynamic settlement potential at the site. Both field and laboratory data were used to perform the analysis. The field measurements included the assumed depth of the water table and the SPT N-values. The laboratory data included USCS/AASHTO classification and soil unit weight. An earthquake magnitude (Mw) of 7.5 was considered. An A_s value of 0.255g was utilized as obtained from the referenced Seismic Design Maps. Groundwater was set at a depth of approximately 19 feet measured from the approximate ground surface at the location of Boring L-2.

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

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. Geotechnology evaluated this condition, and more information is provided in the Global Stability section of this report.

Approach Embankment Settlement

Settlement analyses were performed to assess fill-induced settlement for the approaches. Based on the plans provided, approximately 11 feet of fill will be required at the proposed abutments to bring the sites to grade. For settlement analyses, we assumed cohesive, engineered fill will be used for the fill material. The results of the settlement analyses are shown in Table 5. If grade changes will require the placement of additional fill, Geotechnology should be contacted to perform additional settlement analyses for fill-induced settlement at the approaches.

Table 5. Summary of Estimated Settlement.

South Abutment			North Abutment		
Max Fill (feet)	Estimated Settlement (inches)		Max Fill (feet)	Estimated Settlement (inches)	
	Immediate	Consolidation		Immediate	Consolidation
11	1	1	11	1	1

Discussion of Fill-Induced Settlement. The results of the settlement analyses indicate immediate and primary consolidation settlement at the approaches. We anticipate the immediate settlement to occur within a week of fill placement. Based on the one-dimensional consolidation test performed, practical completion of consolidation induced settlement will occur within 3 to 6 weeks following fill placement.

It should be noted the one-dimensional consolidation test confines the drainage pathway to one dimension while in the field, drainage takes place in three dimensions; therefore, it is our professional opinion the estimated settlement will occur in a shorter time period. If the anticipated waiting period adversely impacts project schedule, a settlement monitoring program may be implemented and survey data be forwarded to Geotechnology so that construction can commence as soon as the remaining settlement is tolerable.

Global Stability

Based on the provided plans, abutment fill will be placed at a 2H:1V slope and side slopes will be constructed at 3H:1V slopes on top of the varying existing grades. Geotechnology performed stability analyses for deep-seated, global failure of bridge abutment slopes using the computer program Slide. 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 Table 6. A pseudo-static seismic acceleration of 0.128g, corresponding to one-half the peak ground acceleration (per FHWA Publication NHI-11-032) was utilized for the seismic condition. Section profiles with calculated critical failure surfaces and utilized soil parameters are presented in Appendix E for selected analyses.

Table 6. Slope Stability Analyses Results.

Location	Slope Ratio	Approximate Berm | Embankment Height (ft.)	Calculated Factor of Safety		
			Short-Term Static[a]	Long-Term Static[a]	Seismic[b]
South Abutment Side Slope	3:1	-- | 11	2.66	2.09	1.79
South Abutment Spill Slope	2:1	14 | 17	2.57	1.56	1.52
North Abutment Side Slope	3:1	-- | 11	2.43	2.10	1.50
North Abutment Spill Slope	2:1	14 | 17	2.29	1.60	1.47

Note: Berm height is defined as height of the 2H:1V spill slope.

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

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

The models used in this computation did not consider the relative stabilizing effect of foundation piles driven to support the abutments or armoring of abutments with rip rap or concrete. In general, foundation piles may provide additional stabilizing force to the abutment slopes, resulting in a factor of safety higher than those presented in Table 6. Existing slopes should be benched prior to placing new fill to reduce the potential for development of slip planes between new and existing fill.

Deep Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2017). It is our understanding 16- and 18-inch diameter, closed-end, steel, pipe piles will be used for support of the proposed bridge. Geotechnology should be notified if a different foundation type is being considered.

Synthetic profiles have been compiled for each abutment based upon the soil profile encountered in the borings, approximate boring elevations, and the proposed final grade. Based on the provided information, the pile cap elevation at both abutments is approximately El 207 and the upper 10 feet beneath the pile cap at each pile location will be prebored. Soil parameters, including LPILE parameters, for each abutment are included in Appendix G.

Nominal resistance curves showing axial resistance from skin friction and total axial capacity (skin friction + end bearing) for the abutments are presented in Appendix F. Nominal resistances at each abutment are presented in Table 7. Uplift (tension) capacities may be calculated using the resistance provided by skin friction.

Table 7. Nominal Static Axial Resistance of Driven Piles.

Location	Pile Diameter (inches)	Embedment Length (feet)	Nominal Static Resistance (tons)			Nominal Post-Fill Resistance (tons)	
			Side Resistance	End Bearing	Compression Total	Compression Total[a,b]	Drag Load
South Abutment	16	65	86	7	93	88	5
		75	104	7	111	106	5
		85	133	105	238	233	5
North Abutment		65	101	8	109	105	5
		75	121	8	129	124	5
		85	152	105	257	252	5
South Abutment	18	65	96	9	105	99	6
		75	116	9	125	119	6
		85	151	127	278	272	6
North Abutment		65	112	11	123	116	6
		75	135	11	146	140	6
		85	172	127	299	293	6

[a] Nominal post-fill resistance has not been reduced by the drag load.

[b] Based on estimated settlement due to placement of fill.

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

Table 8. Resistance Factors Based on Static Analysis Methods.

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

Based on AASHTO LRFD (2017) Table 10.5.5.2.3-1, a higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 9.

Table 9. Resistance Factors for Driven Piles.

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

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

Pile Group Considerations. The settlement of pile groups should be evaluated as per AASHTO LRFD (2017) 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 (2017) 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 (2017) sections 10.7.3.9 and 10.7.3.11.

Driven Pile Construction Considerations. Minimum hammer energies required to drive the piles were evaluated using the computer software WEAP. The recommended minimum hammer energy is provided in Table 10.

Table 10. Minimum Hammer Energies.

Pile Diameter [a] (inches)	Location	Embedment Length (feet)	Required Capacity (tons)	Minimum Rated Hammer Energy (kip-feet)
16	South Abutment	91	283	67
	North Abutment	88		

[a] Closed-ended pipe piles with ½-inch thick walls.

Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. Alternatively, potential driving criteria can be developed using wave equation analyses after the pile hammer is selected.

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 (2017) 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 (2017) 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 (2017) Table 10.5.5.2.3-1 or 2 percent of the production piles, whichever results in a larger number of tests. We recommend that the identified piles be tested at the end of initial drive (EOID) and a restrike performed at a minimum seven days after EOID.

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

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

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

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

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

The effects of group interaction must be considered when evaluating pile/shaft group horizontal movement. The lateral resistance for individual piles calculated by LPILE must be reduced by the P-multipliers provided in Section 10.7.2.4 of the AASHTO LRFD (2017) to determine lateral resistance of a pile group. Alternatively, the GROUP software can be used to evaluate the lateral resistance of the pile/shaft groups. The resistance factor for lateral resistance of single piles or pile groups is 1.0.

Downdrag

The AASHTO LRFD (2017) suggests that settlement of 0.4-inch or greater could produce downdrag on pile foundations. Downdrag occurs as the soil strata move downward relative to the foundations due to settlement of the soil layers. The relative movement of the soil layers versus the shaft depends on the final foundation configuration.

Downdrag Due to Fill-Induced Settlement. Based on settlement analysis performed for the 11-foot maximum fill placement at the abutments, up to 2 inches of settlement is predicted. Approximately 1 inch of settlement is estimated to occur immediately after fill placement, and 1 inch of consolidation settlement is estimated to occur within approximately 3 to 6 weeks following fill placement. Pile driving should not begin until settlement is practically complete as determined by the settlement plate monitoring program. Piles driven after fill placement prior to completion of settlement will be subject to drag loads as the soil consolidates due to the weight of the fill. The settlement plate survey data recorded for the settlement monitoring program as recommended in the Approach Embankment Settlement Section should be forwarded to Geotechnology to determine if the settlement due to fill placement is practically complete prior to driving piles.

Downdrag due to Dynamic Settlement. Based on the low liquefaction potential at this site, liquefaction-induced drag loads were not be considered.

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.

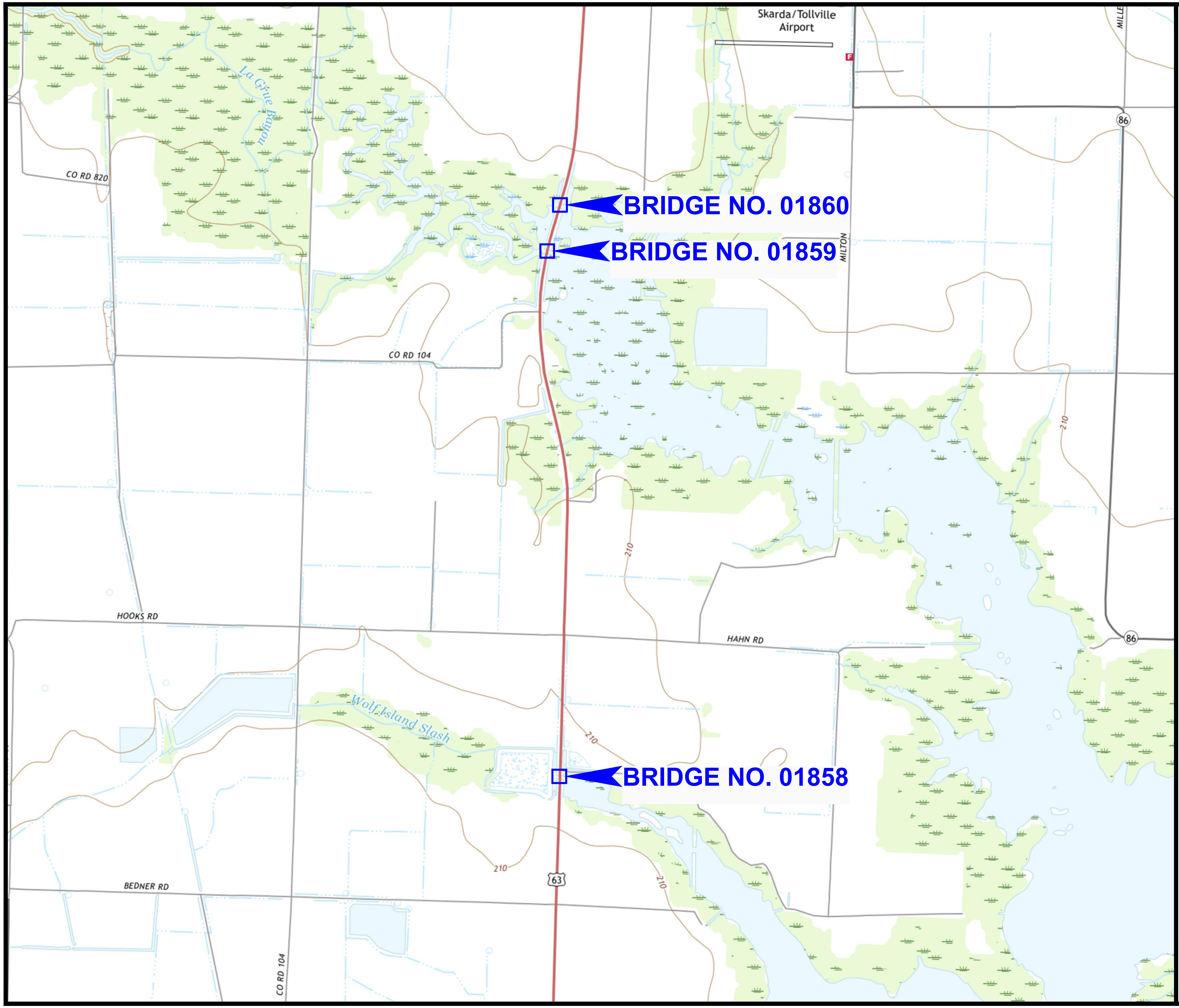
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

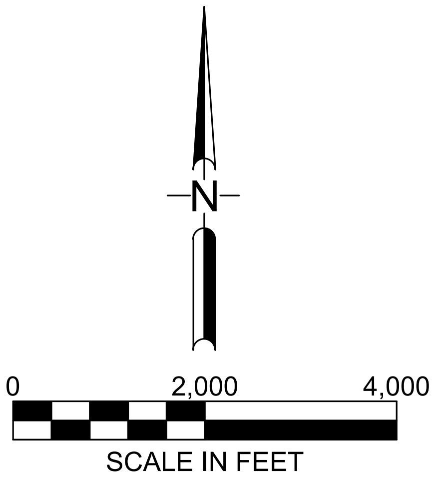
Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

Appendix B
FIGURES



NOTES

1. Plan adapted from a 7.5 minute U.S.G.S. map for Slovak, Arkansas quadrangle, last revised in 2017.



Drawn By: SDF	Ck'd By: ASM	App'vd By: DMS
Date: 12-8-20	Date: 12-29-20	Date: 12-29-20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Route 63 Section 11
Structures and Approaches
Bridge Nos. 01858, 01859 and 01860
Prairie County, Arkansas

BRIDGE LOCATIONS AND TOPOGRAPHY

Project Number
J034561.01

FIGURE 1

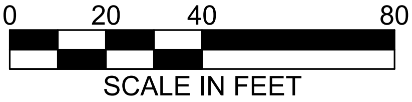
NOTES

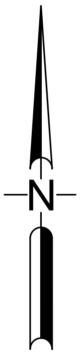
1. Plan adapted from an October 14, 2015 aerial photograph courtesy of Google Earth.

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

LEGEND

● Boring Location





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Date: 12-8-20	Date: 12-29-20	Date: 12-29-20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Route 63 Section 11
Structures and Approaches
Bridge No. 01859
Prairie County, Arkansas

AERIAL PHOTOGRAPH OF BRIDGE 01859 AND BORING LOCATIONS

Project Number
J034561.01

FIGURE 2

Appendix C

BORING INFORMATION

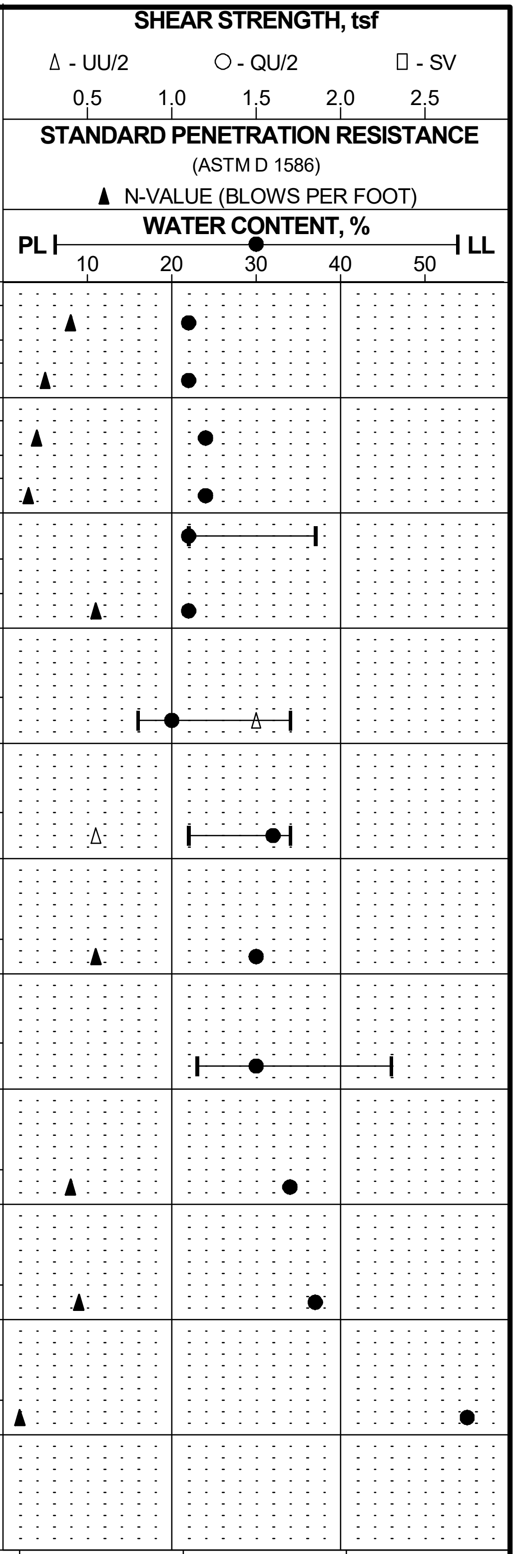
Surface Elevation: 213

Datum msl

Completion Date: 9/17/20
Station: 216+07
Offset: -7

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Asphalt: 8 inches		
		Base Materials: 4 inches	3-3-5	SS1
		Soft to very stiff, gray, LEAN CLAY - (CL) trace organics		
			1-2-3	SS2
5	208			
			1-2-2	SS3
		pH = 3.86	0-1-2	SS4
10	203	resistivity = 8,265 ohms-cm		
		with sand	94	ST5
		84% passing No. 200 sieve		
			2-7-4	SS6
15	198			
			108	ST7
20	193			
		pH = 7.43	93	ST8
25	188	resistivity = 712.5 ohms-cm		
			3-4-7	SS9
30	183			
			63	ST10
35	178			
		Stiff to soft, red to gray, FAT CLAY - CH	2-4-4	SS11
40	173			
		pH = 8.22	3-3-6	SS12
45	168	resistivity = 684 ohms-cm		
			0-1-1	SS14
50	163			

GRAPHIC LOG



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

___ AUGER 3 1/4" HOLLOW STEM
WASHBORING FROM 15 FEET
JCG DRILLER SWF LOGGER
CME 75 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 80 %

REMARKS:

Drawn by: ASM	Checked by: DMS	App'vd. by: ASE
Date: 9/25/20	Date: 12/17/20	Date: 12/17/20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ARDOT 061615
Highway 63 Bridge No. 01859
Over La Grue Bayou South
Prairie County, Arkansas

LOG OF BORING: L-1

Project No. J034561.01

Surface Elevation: 213

Datum msl

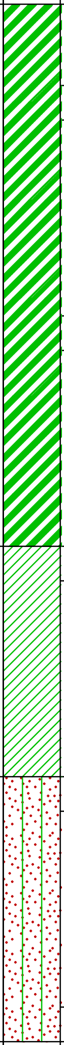
Completion Date: 9/17/20

Station: 216+07

Offset: -7

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Stiff to soft, red to gray, FAT CLAY - CH *(continued)*		
		pH = 8.3 resistivity = 462 ohms-cm	1-1-2	SS15
60	153			
65	148			
			2-4-6	SS16
70	143			
75	138			
		Soft, gray, LEAN CLAY - CL	2-1-3	SS17
80	133			
85	128			
		Very dense, gray, SILTY SAND - SM	19-29-29	SS18
90	123			
95	118			
		trace gravel	25-38-50	SS19
100	113	Boring terminated at 100 feet.		
105	108			

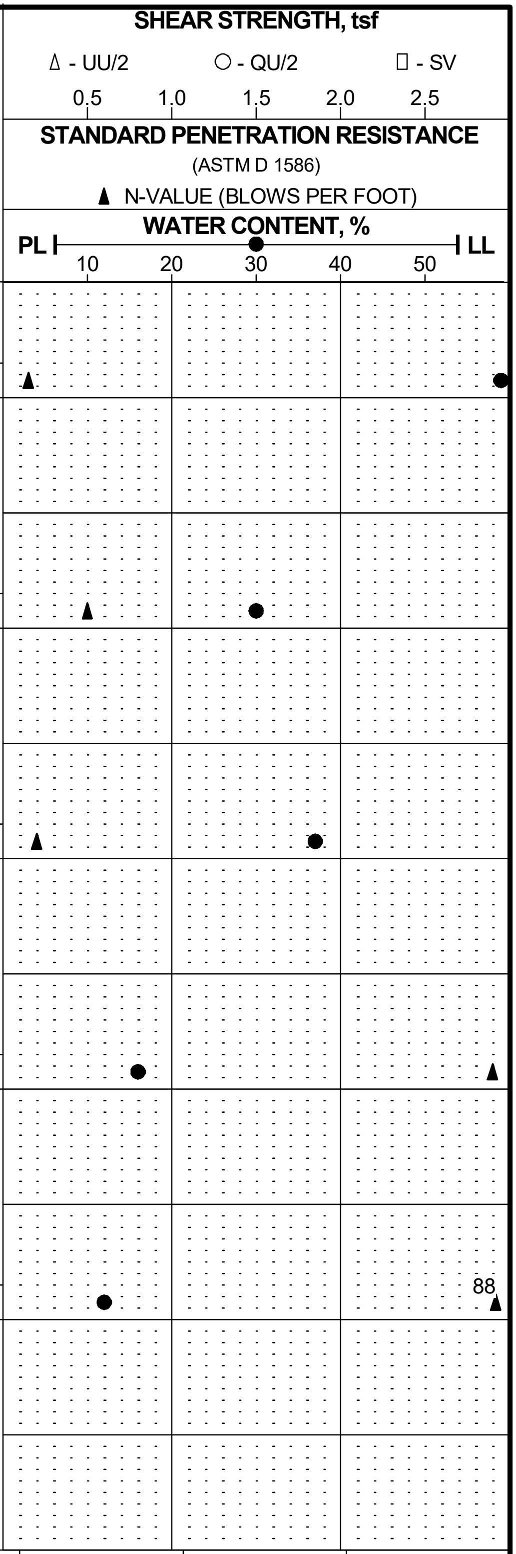
GRAPHIC LOG



SHEAR STRENGTH, tsf

STANDARD PENETRATION RESISTANCE (ASTM D 1586)

WATER CONTENT, %



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM

WASHBORING FROM 15 FEET

JCG DRILLER SWF LOGGER

CME 75 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 80 %

REMARKS:

Drawn by: ASM	Checked by: DMS	App'vd. by: ASE
Date: 9/25/20	Date: 12/17/20	Date: 12/17/20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ARDOT 061615
Highway 63 Bridge No. 01859
Over La Grue Bayou South
Prairie County, Arkansas

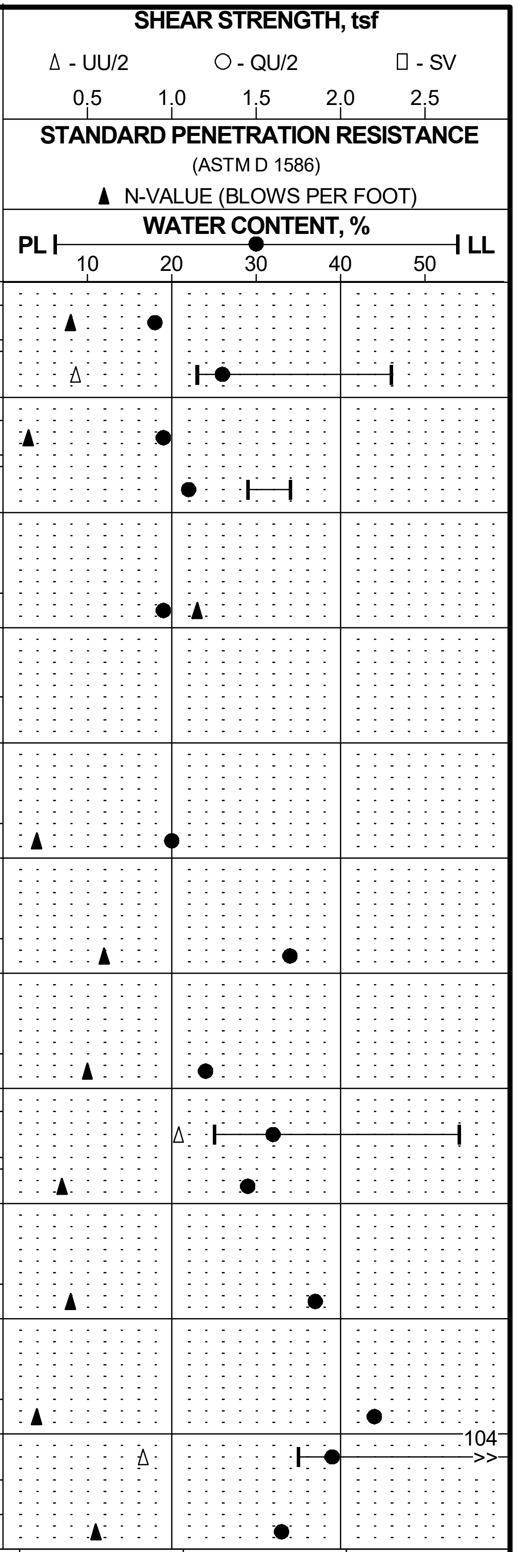
CONTINUATION OF LOG OF BORING: L-1

Project No. J034561.01

Surface Elevation: 213
Datum msl

Completion Date: 9/16/20
Station: 217+26
Offset: -7

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Asphalt: 6 inches		
		Base Materials: 4 inches of cement-treated soil	3-4-4	SS1
		Medium stiff, brown, LEAN CLAY, trace sand - (CL) trace gravel, sand and organics	95	ST2
5	208	with sand 82% passing No. 200 sieve	0-1-2	SS3
		Brown SILT - (ML)	106	ST4
10	203	pH = 4.53 resistivity = 4,047 ohms-cm		
		Very stiff to soft, brown, LEAN CLAY, trace sand - (CL)	4-9-14	SS5
15	198			
		pH = 4.57 resistivity = 3,135 ohms-cm ∇		ST6
20	193			
			2-3-1	SS7
25	188			
			2-5-7	SS8
30	183			
			3-5-5	SS9
35	178			
		Very stiff to soft, red and gray to gray, FAT CLAY - (CH)	92	ST10
		pH = 8.23 resistivity = 855 ohms-cm	2-3-4	SS11
40	173			
			3-4-4	SS12
45	168			
			2-2-2	SS13
50	163	pH = 8.29 resistivity = 542 ohms-cm	84	ST14
			3-5-6	SS15



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 19.5 FEET ∇

DRILLING DATA

___ AUGER 3 1/4" HOLLOW STEM
WASHBORING FROM 30 FEET
JCG DRILLER SWF LOGGER
CME 75 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 80 %

REMARKS:

Drawn by: ASM	Checked by: DMS	App'vd. by: ASE
Date: 9/25/20	Date: 12/17/20	Date: 12/17/20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ARDOT 061615
Highway 63 Bridge No. 01859
Over La Grue Bayou South
Prairie County, Arkansas

LOG OF BORING: L-2

Project No. J034561.01

Surface Elevation: 213

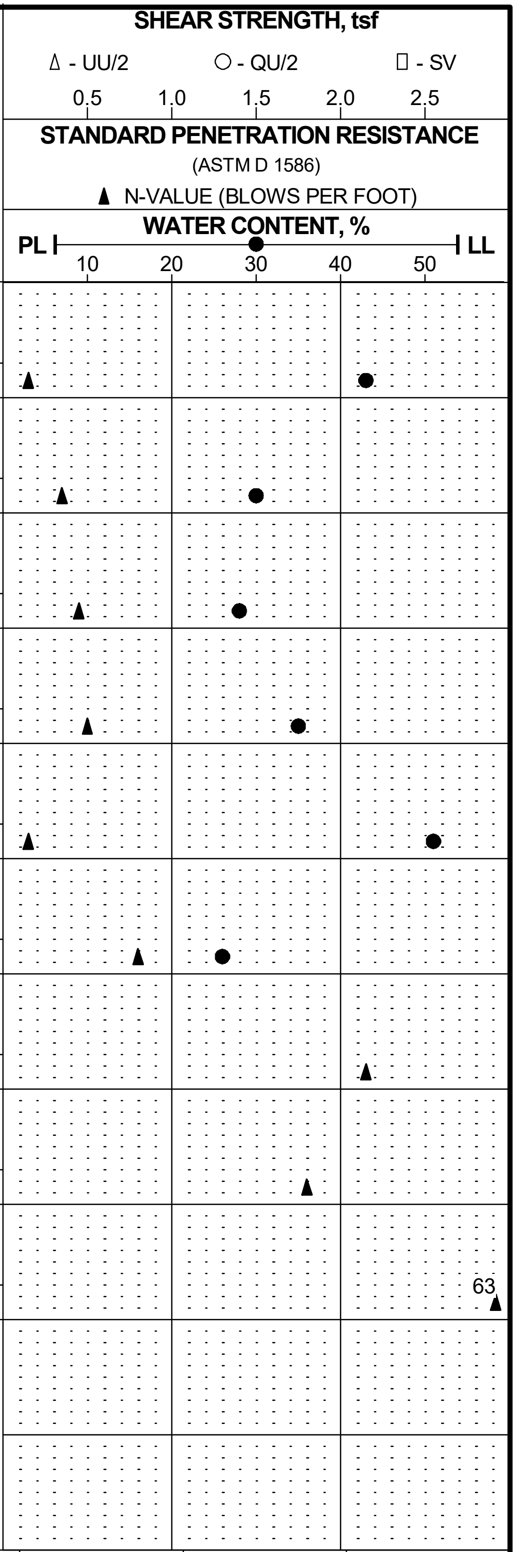
Datum msl

Completion Date: 9/16/20

Station: 217+26

Offset: -7

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Very stiff to soft, red and gray to gray, FAT CLAY - (CH) *(continued)*	▨		
				1-2-1	SS16
60	153				
		Medium stiff, gray, LEAN CLAY, trace sand - CL pH = 8.31 resistivity = 855 ohms-cm	▨	0-4-3	SS17
65	148				
		Stiff, gray, FAT CLAY - CH	▨	2-4-5	SS18
70	143				
		Stiff, gray, LEAN CLAY, trace sand seams - CL	▨	4-3-7	SS19
75	138				
		Soft, gray, FAT CLAY - CH	▨	2-1-2	SS20
80	133				
		Very stiff, gray SILT - ML	▥	2-4-12	SS21
85	128				
		Dense to very dense, gray, SILTY SAND - SM	░	20-22-21	SS22
90	123				
				15-18-18	SS23
95	118				
				25-28-35	SS24
100	113	Boring terminated at 100 feet.			
105	108				



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 J034561.01.GPJ GTINC 0638301.GPJ 3/23/23

GROUNDWATER DATA

ENCOUNTERED AT 19.5 FEET ∇

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM
WASHBORING FROM 30 FEET
JCG DRILLER SWF LOGGER
CME 75 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 80 %

REMARKS:

Drawn by: ASM	Checked by: DMS	App'vd. by: ASE
Date: 9/25/20	Date: 12/17/20	Date: 12/17/20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ARDOT 061615
Highway 63 Bridge No. 01859
Over La Grue Bayou South
Prairie County, Arkansas

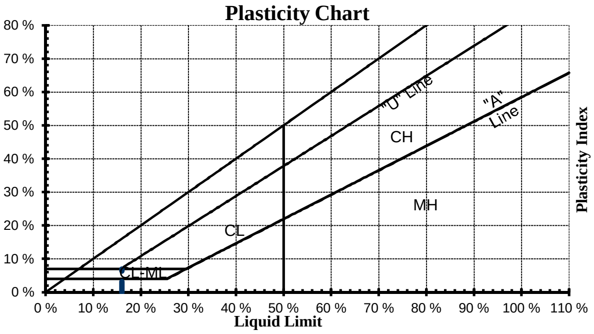
CONTINUATION OF
LOG OF BORING: L-2

Project No. J034561.01

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

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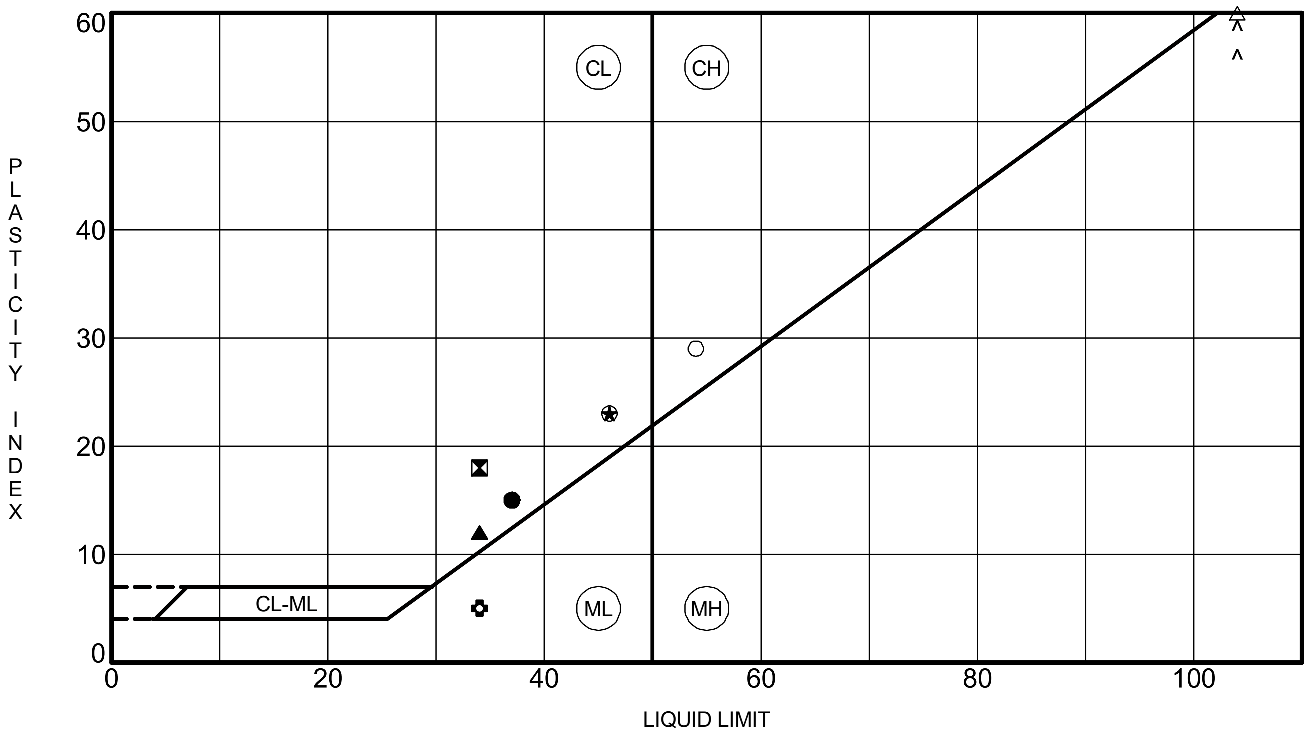
Relative composition and Unified Soil Classification System (USCS) designations are based on visual descriptions and are approximate only. If laboratory tests were performed to classify the soil, the USCS designation is shown in parenthesis.

Appendix D

LABORATORY TEST DATA

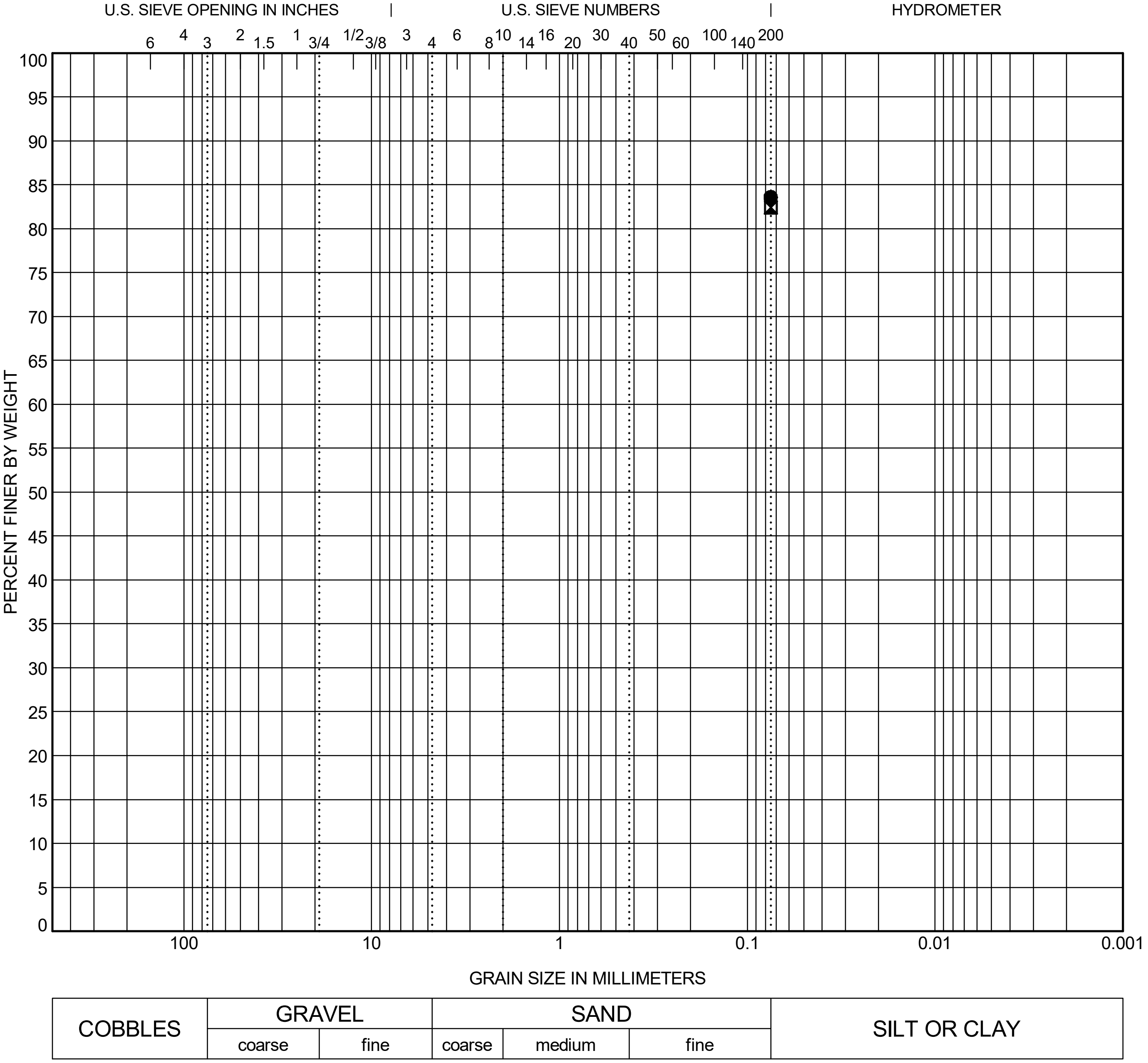


	Specimen Identification		LL	PL	PI	Fines	Classification
●	**L-1**	**10.0**	**37**	**22**	**15**	**84**	**LEAN CLAY with SAND(CL), A-6 (13)**
⧗	**L-1**	**18.0**	**34**	**16**	**18**		**LEAN CLAY(CL), A-6**
▲	**L-1**	**23.0**	**34**	**22**	**12**		**LEAN CLAY(CL), A-6**
★	**L-1**	**33.0**	**46**	**23**	**23**		**LEAN CLAY(CL), A-7-6**
⊙	**L-2**	**3.0**	**46**	**23**	**23**	**82**	**LEAN CLAY with SAND(CL), A-7-6 (20)**
✤	**L-2**	**8.0**	**34**	**29**	**5**		**SILT(ML), A-4**
○	**L-2**	**36.0**	**54**	**25**	**29**		**FAT CLAY(CH), A-7-6**
△	**L-2**	**50.0**	**104**	**35**	**69**		**FAT CLAY(CH), A-7-5**

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ATTERBERG LIMITS RESULTS

ARDOT 061615 Highway 63
Bridge No. 01859 Over La Grue Bayou South
Prairie County, Arkansas
J034561.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**L-1**	**10.0**	**LEAN CLAY with SAND(CL), A-6 (13)**	**37**	**22**	**15**		
☒	**L-2**	**3.0**	**LEAN CLAY with SAND(CL), A-7-6 (20)**	**46**	**23**	**23**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**L-1**	**10.0**	**0.075**				**0.0**	**0.0**	**83.6**	
☒	**L-2**	**3.0**	**0.075**				**0.0**	**0.0**	**82.4**	

GEOTECHNOLOGY
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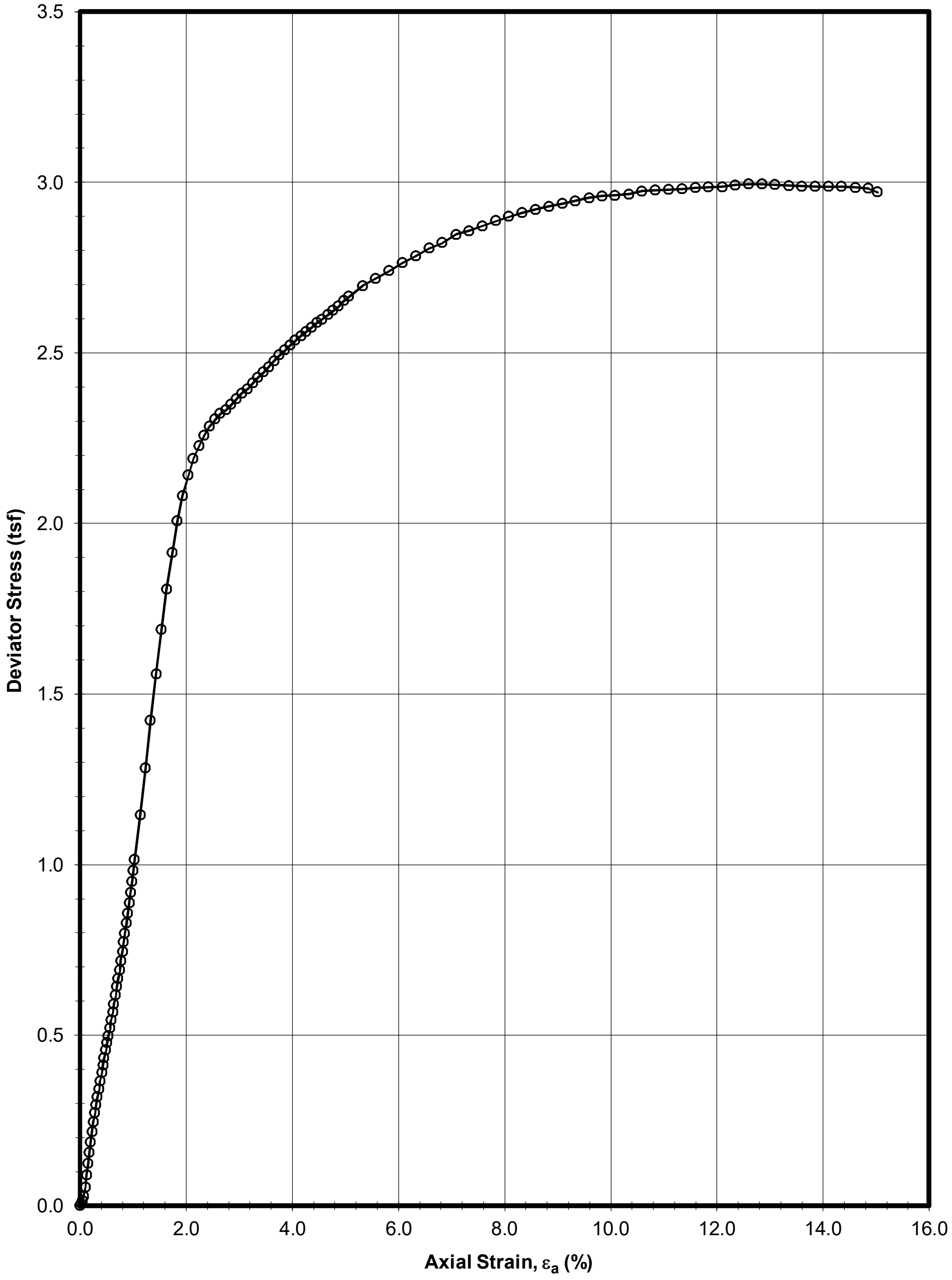
GRAIN SIZE DISTRIBUTION

ARDOT 061615 Highway 63
Bridge No. 01859 Over La Grue Bayou South
Prairie County, Arkansas
J034561.01

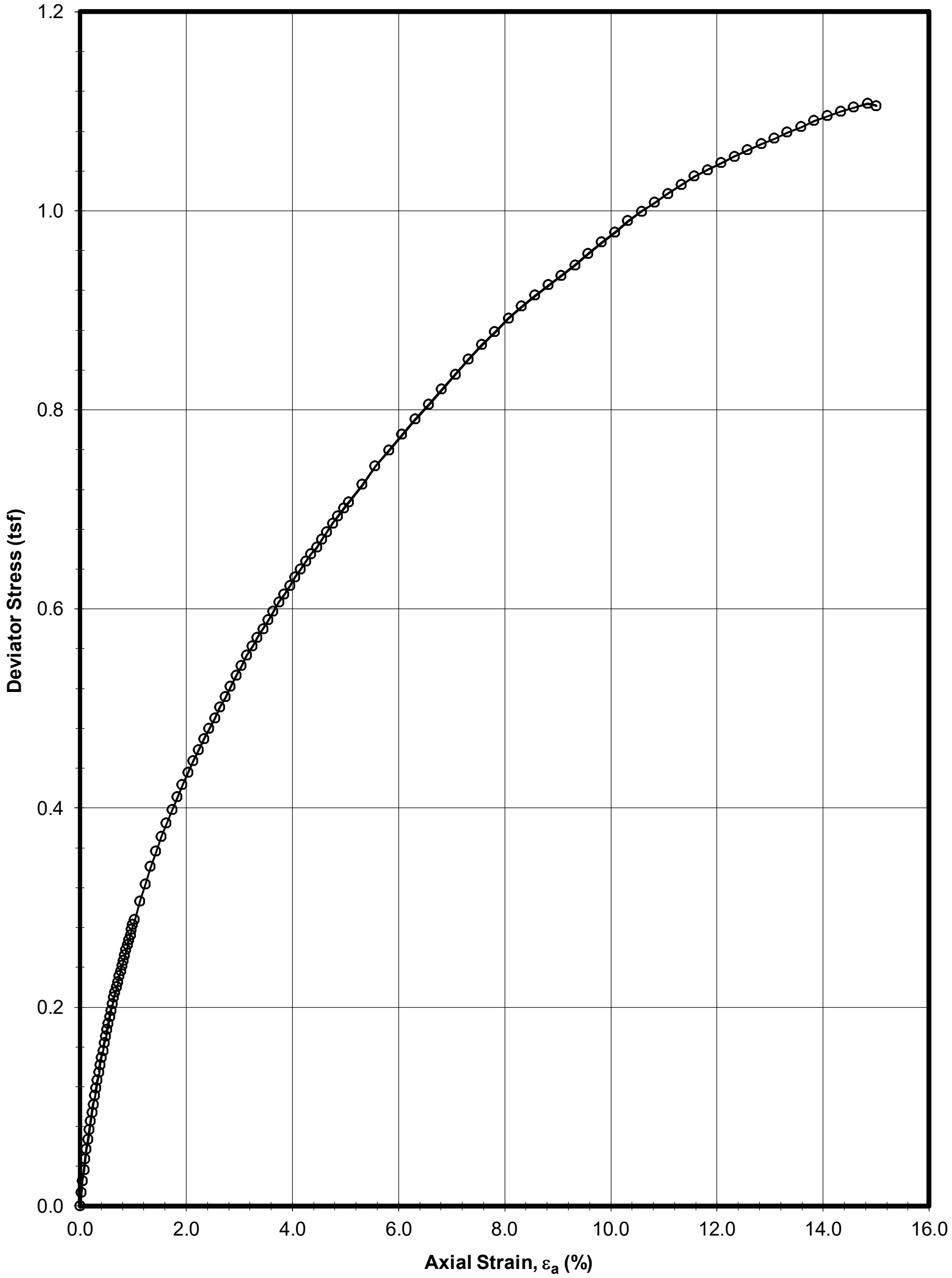


UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J034561.01
Boring: L-1
Sample: ST-7 - Depth: 18 ft.

GEOTECHNOLOGY

A Universal Engineering Sciences Company



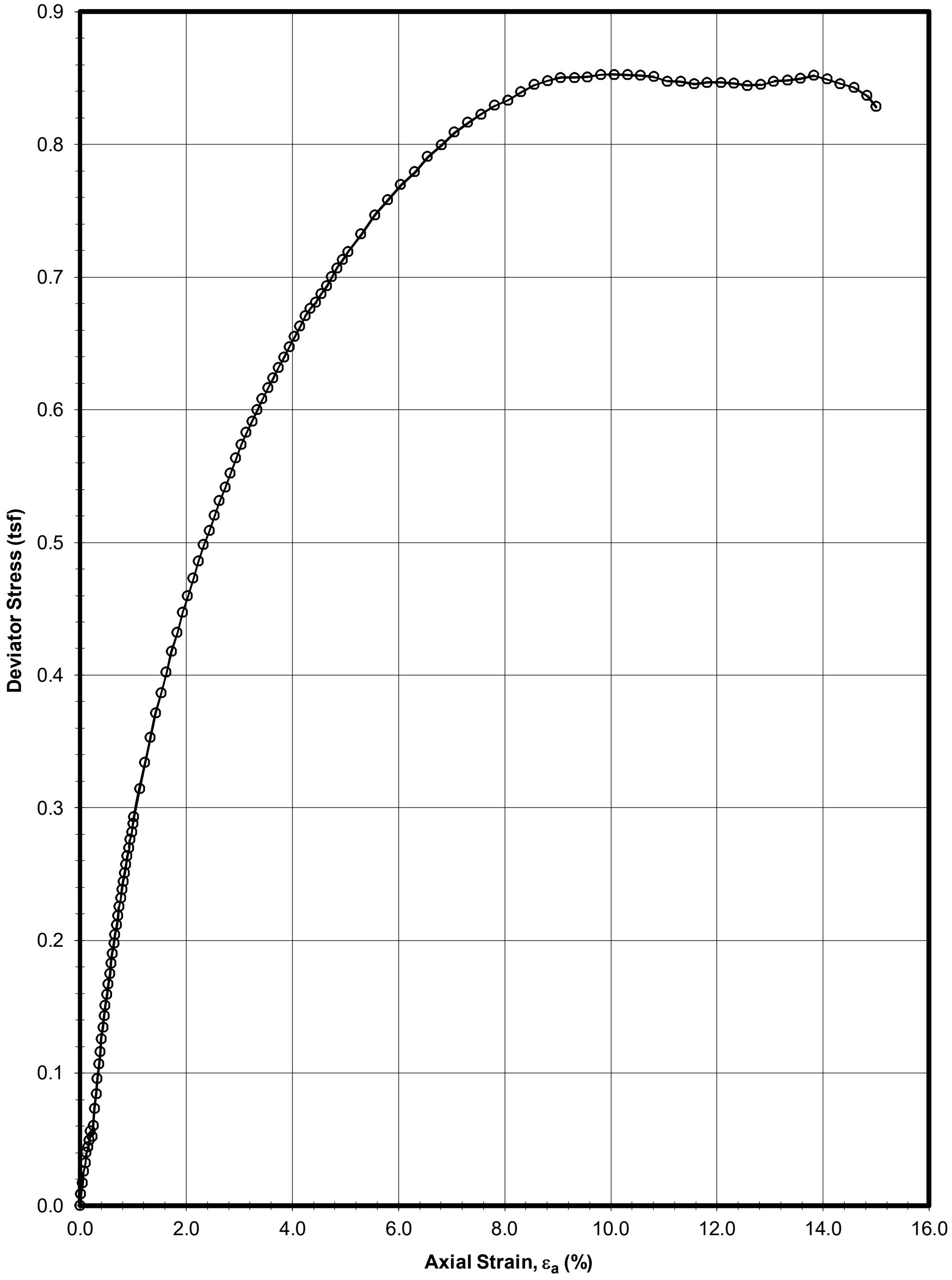
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034561.01

Boring: L-1

Sample: ST-8 - Depth: 23 ft.



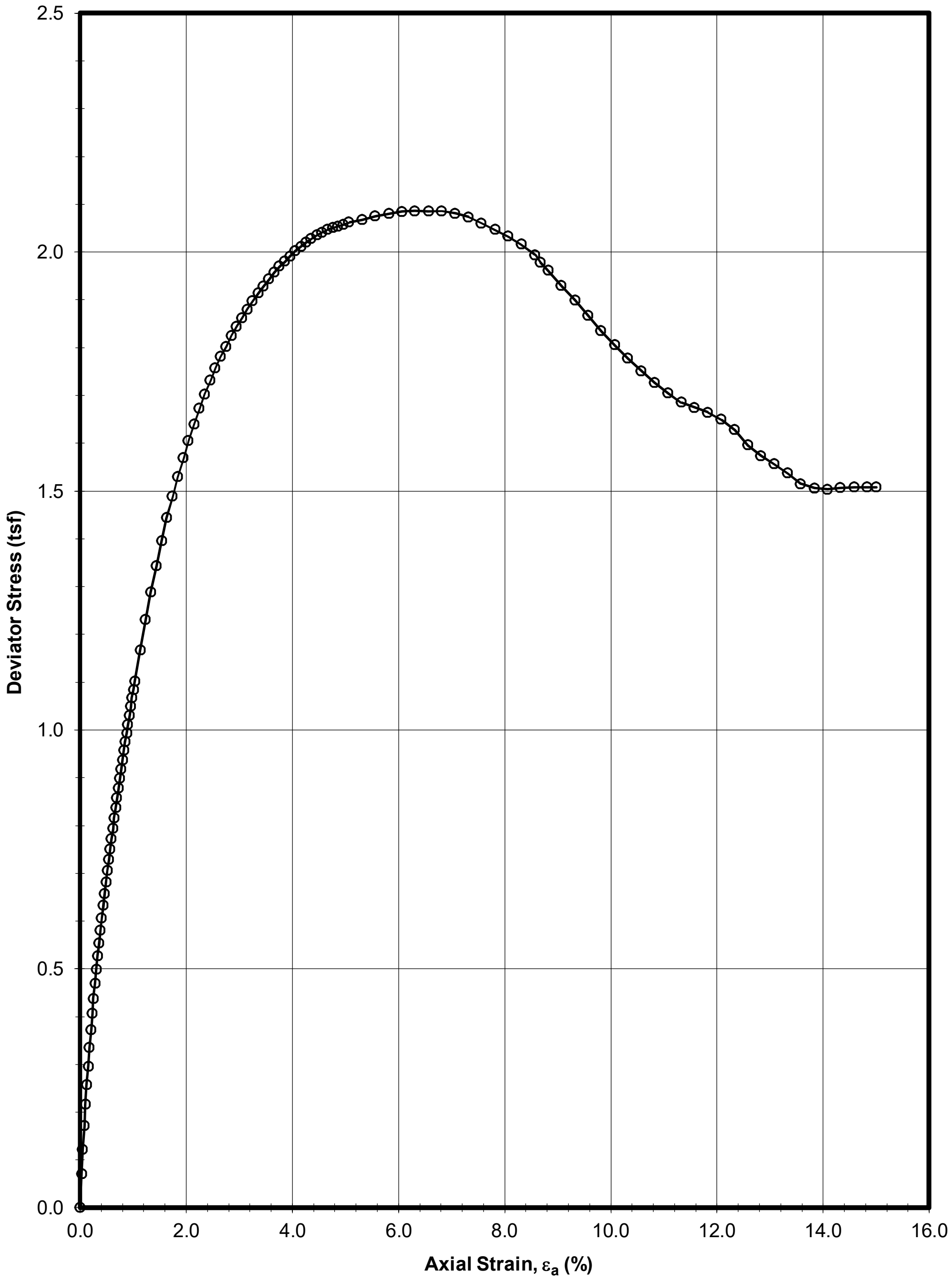
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034561.01

Boring: L-2

Sample: ST-2 - Depth: 3 ft.



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

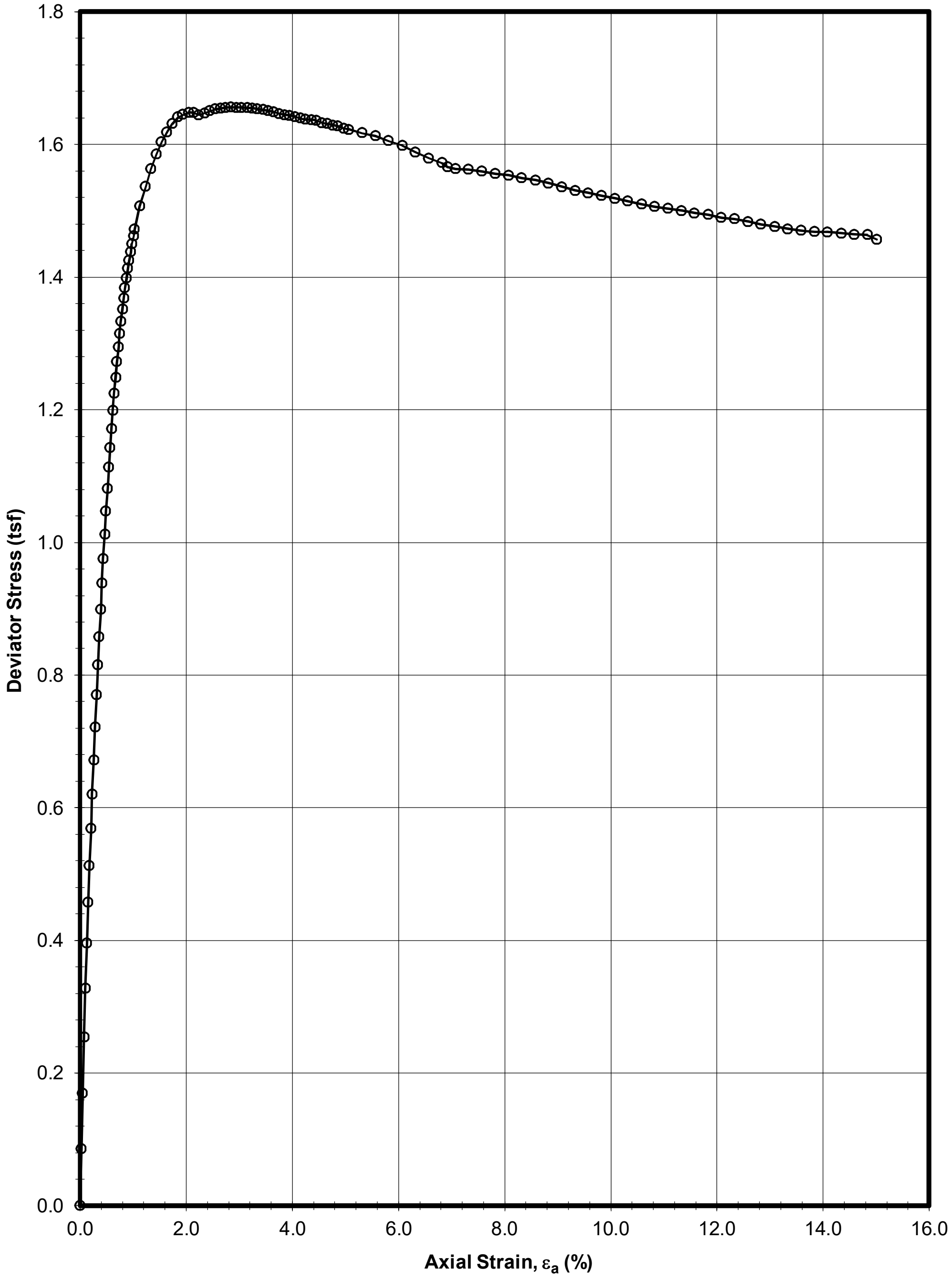
ASTM D 2850

Project No.: J034561.01

Boring: L-2

Sample: ST-10 - Depth: 36 ft.

GEOTECHNOLOGY
A Universal Engineering Sciences Company



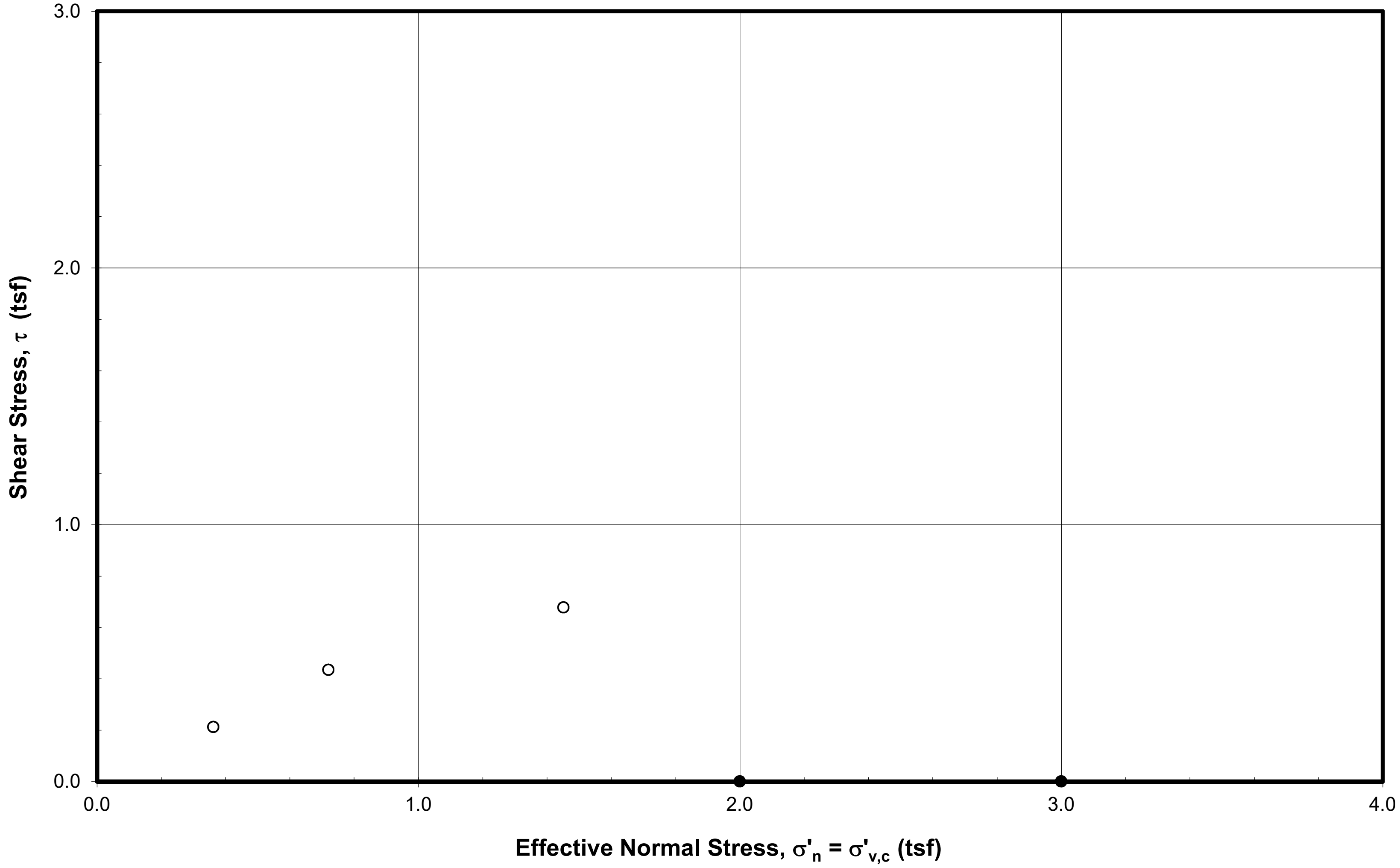
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034561.01

Boring: L-2

Sample: ST-14 - Depth: 50 ft.

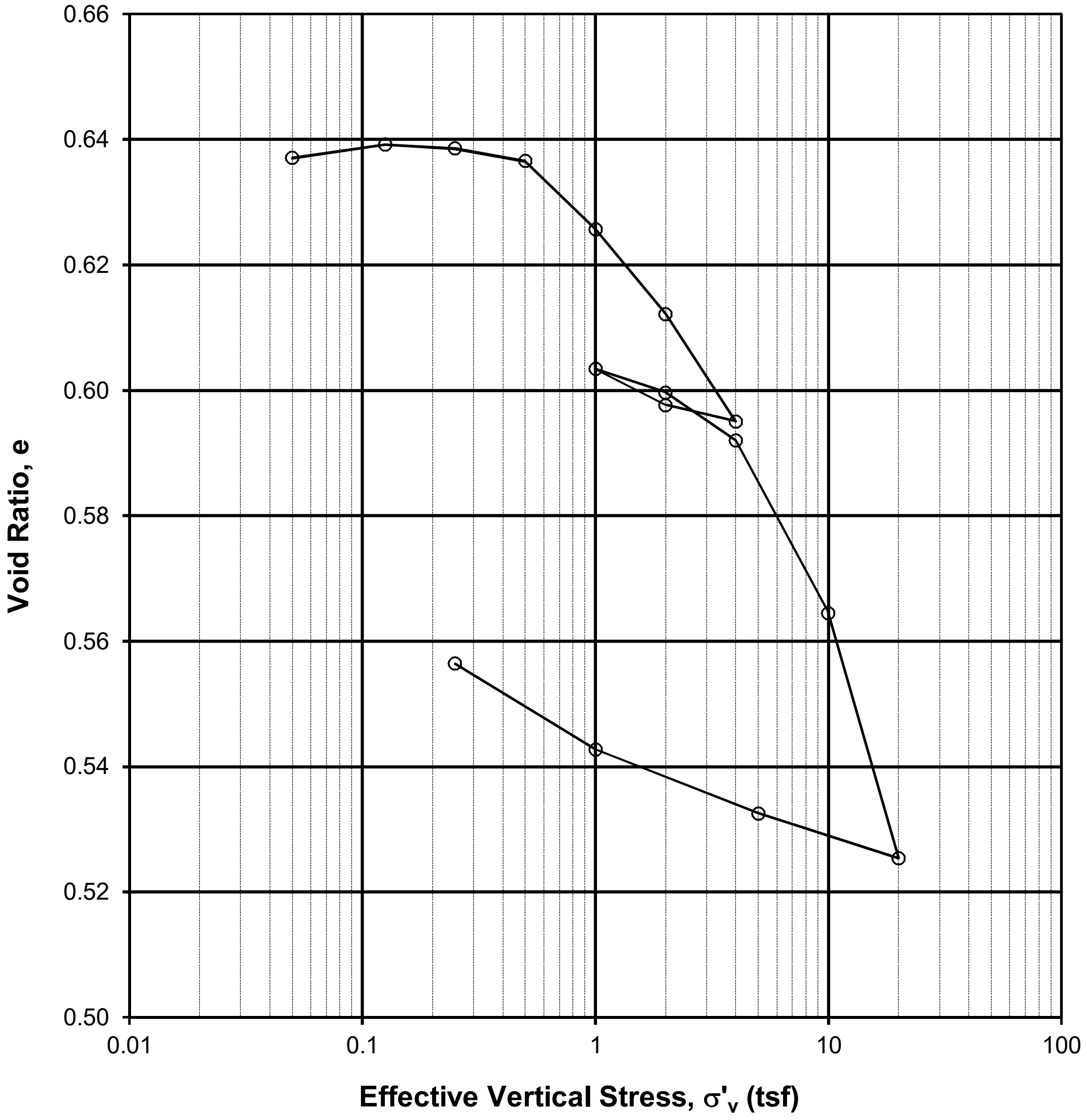


DRAINED DIRECT SHEAR TEST

ASTM D 3080
Boring: L-2 Sample: ST-4 -Depth: 8.0ft

Liquid Limit= 34 Plastic Limit= 22 Plasticity Index = 12 USCS: CL

Compression Index, C_c = 0.09
Void Ratio, e_o = 0.637
Recompression Index, C_r = 0.02
Preconsolidation Pressure = 1.7 tsf



1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435
Project No.: J034561.01
Boring: L-1
Sample: ST-8 - Depth: 23.0

TEST REPORT

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

Project No.:	**J034561.01**	**January 6, 2021**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01859**	
Boring Number:	**L-1**	
Sample ID:	**SS-11**	
Depth (ft):	**8.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	58,000	0.57	33,060.00	12.3
#2	17,000	0.57	9,690.00	12.1
#3	14,500	0.57	8,265.00	24.9
#4	17,500	0.57	9,975.00	31.8

Minimum Soil Resistivity **8,265.00**

TEST REPORT

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

Project No.: J034561.01 — **January 6, 2021**
Project Name: ARDOT 061615 — **Page 1 of 1**
Bridge No.: 01859
Boring Number: L-1
Sample ID: SS-12
Depth (ft): 43.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	20,000	0.57	11,400.00	17.4
#2	5,900	0.57	3,363.00	24.2
#3	1,900	0.57	1,083.00	32.4
#4	1,400	0.57	798.00	39.5
#5	1,200	0.57	684.00	23.6
#6	1,400	0.57	798.00	53.6

Minimum Soil Resistivity **684.00**

TEST REPORT

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

Project No.: J034561.01 **January 6, 2021**
Project Name: ARDOT 061615 **Page 1 of 1**
Bridge No.: 01859
Boring Number: L-1
Sample ID: SS-14
Depth (ft): 58.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	14,000	0.57	7,980.00	17.0
#2	5,400	0.57	3,078.00	23.6
#3	1,550	0.57	883.50	32.1
#4	1,000	0.57	570.00	38.7
#5	810	0.57	461.70	46.3
#6	1,200	0.57	684.00	51.4

Minimum Soil Resistivity **461.70**

TEST REPORT

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

Project No.:	**J034561.01**	**December 16, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01859**	
Boring Number:	**L-1**	
Sample ID:	**ST-8**	
Depth (ft):	**23.0**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	8,350	0.57	4,759.50	13.9
#2	2,500	0.57	1,425.00	20.2
#3	1,250	0.57	712.50	26.9
#4	1,350	0.57	769.50	31.5

Minimum Soil Resistivity **712.50**

TEST REPORT

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

Project No.: J034561.01 **January 6, 2021**
Project Name: ARDOT 061615 **Page 1 of 1**
Bridge No.: 01859
Boring Number: L-2
Sample ID: SS-11
Depth (ft): 38.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	19,000	0.57	10,830.00	15.5
#2	3,800	0.57	2,166.00	24.4
#3	1,800	0.57	1,026.00	31.4
#4	1,500	0.57	855.00	37.3
#5	1,500	0.57	855.00	43.1
#6	1,600	0.57	912.00	51.4

Minimum Soil Resistivity **855.00**

TEST REPORT

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

Project No.:	**J034561.01**	**January 6, 2021**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01859**	
Boring Number:	**L-2**	
Sample ID:	**SS-17**	
Depth (ft):	**63.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	4,850	0.57	2,764.50	14.6
#2	2,000	0.57	1,140.00	20.6
#3	1,500	0.57	855.00	26.5
#4	1,500	0.57	855.00	32.8
#5	1,600	0.57	912.00	39.1

Minimum Soil Resistivity **855.00**

TEST REPORT

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

Project No.:	**J034561.01**	**January 6, 2021**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01859**	
Boring Number:	**L-2**	
Sample ID:	**ST-4**	
Depth (ft):	**8**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	88,500	0.57	50,445.00	12.5
#2	16,000	0.57	9,120.00	19.4
#3	8,200	0.57	4,674.00	26.4
#4	7,100	0.57	4,047.00	32.9

Minimum Soil Resistivity **4,047.00**

TEST REPORT

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

Project No.:	**J034561.01**	**January 6, 2021**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01859**	
Boring Number:	**L-2**	
Sample ID:	**ST-6**	
Depth (ft):	**18**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	15,000	0.57	8,550.00	12.7
#2	5,500	0.57	3,135.00	19.9
#3	5,600	0.57	3,192.00	26.7

Minimum Soil Resistivity **3,135.00**

TEST REPORT

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

Project No.: J034561.01 — **December 16, 2020**
Project Name: ARDOT 061615 — **Page 1 of 1**
Bridge No.: 01859
Boring Number: L-2
Sample ID: ST-14
Depth (ft): 50.0

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	27,500	0.57	15,675.00	18.3
#2	4,600	0.57	2,622.00	22.5
#3	1,550	0.57	883.50	30.2
#4	1,000	0.57	570.00	35.4
#5	950	0.57	541.50	42.6
#6	1,000	0.57	570.00	44.6
	Minimum Soil Resistivity		**541.50**	

Appendix E

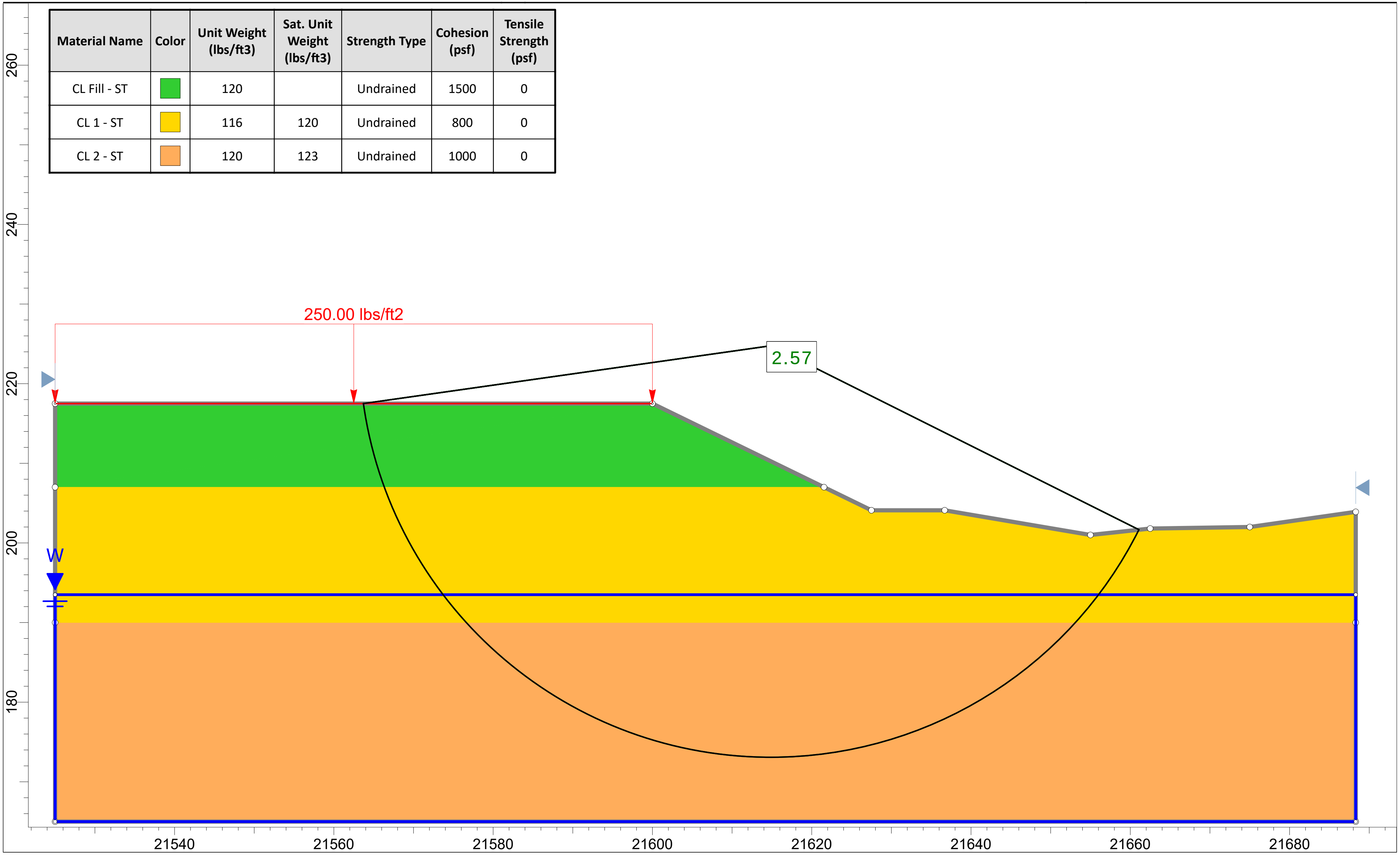
SELECTED GLOBAL STABILITY ANALYSES

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: South Abutment Spill Slope
Analysis Description: Short Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CL 1 - ST	🟨	116	120	Undrained	800	0
CL 2 - ST	🟧	120	123	Undrained	1000	0

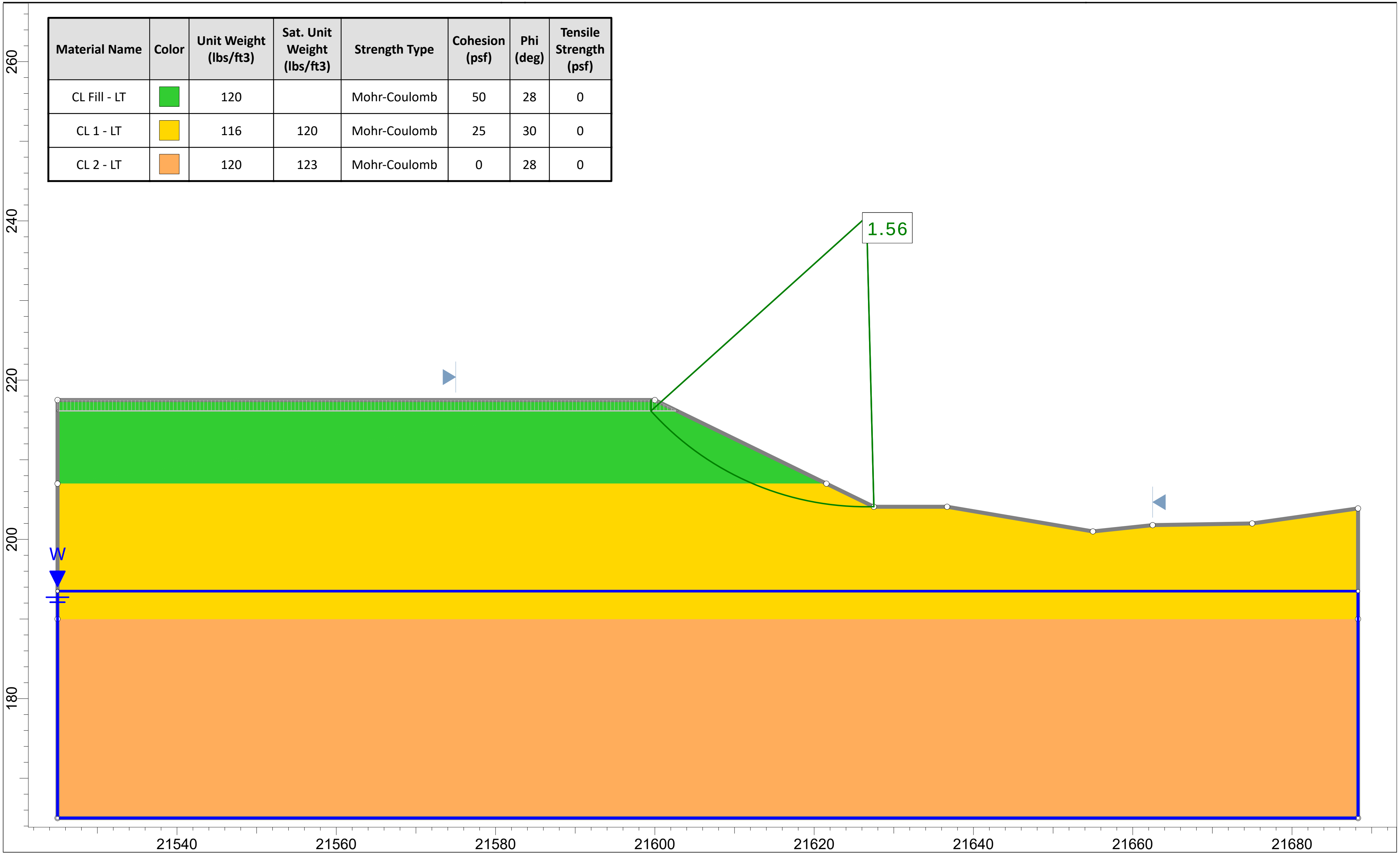


Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: South Abutment Spill Slope
Analysis Description: Long Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 3/1/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - LT	🟩	120		Mohr-Coulomb	50	28	0
CL 1 - LT	🟨	116	120	Mohr-Coulomb	25	30	0
CL 2 - LT	🟧	120	123	Mohr-Coulomb	0	28	0



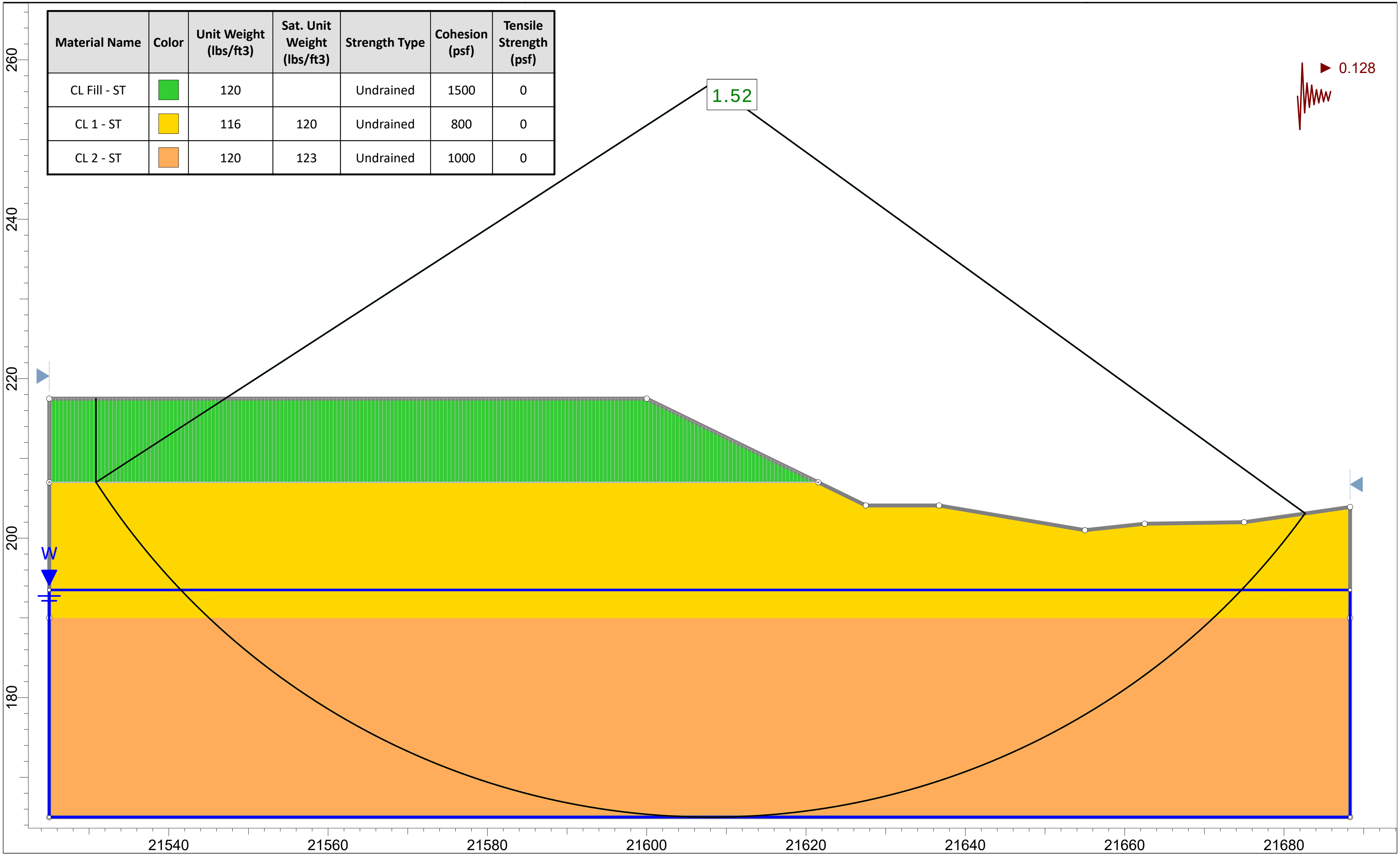
GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: South Abutment Spill Slope
Analysis Description: Seismic
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CL 1 - ST	🟨	116	120	Undrained	800	0
CL 2 - ST	🟧	120	123	Undrained	1000	0



GEOTECHNOLOGY

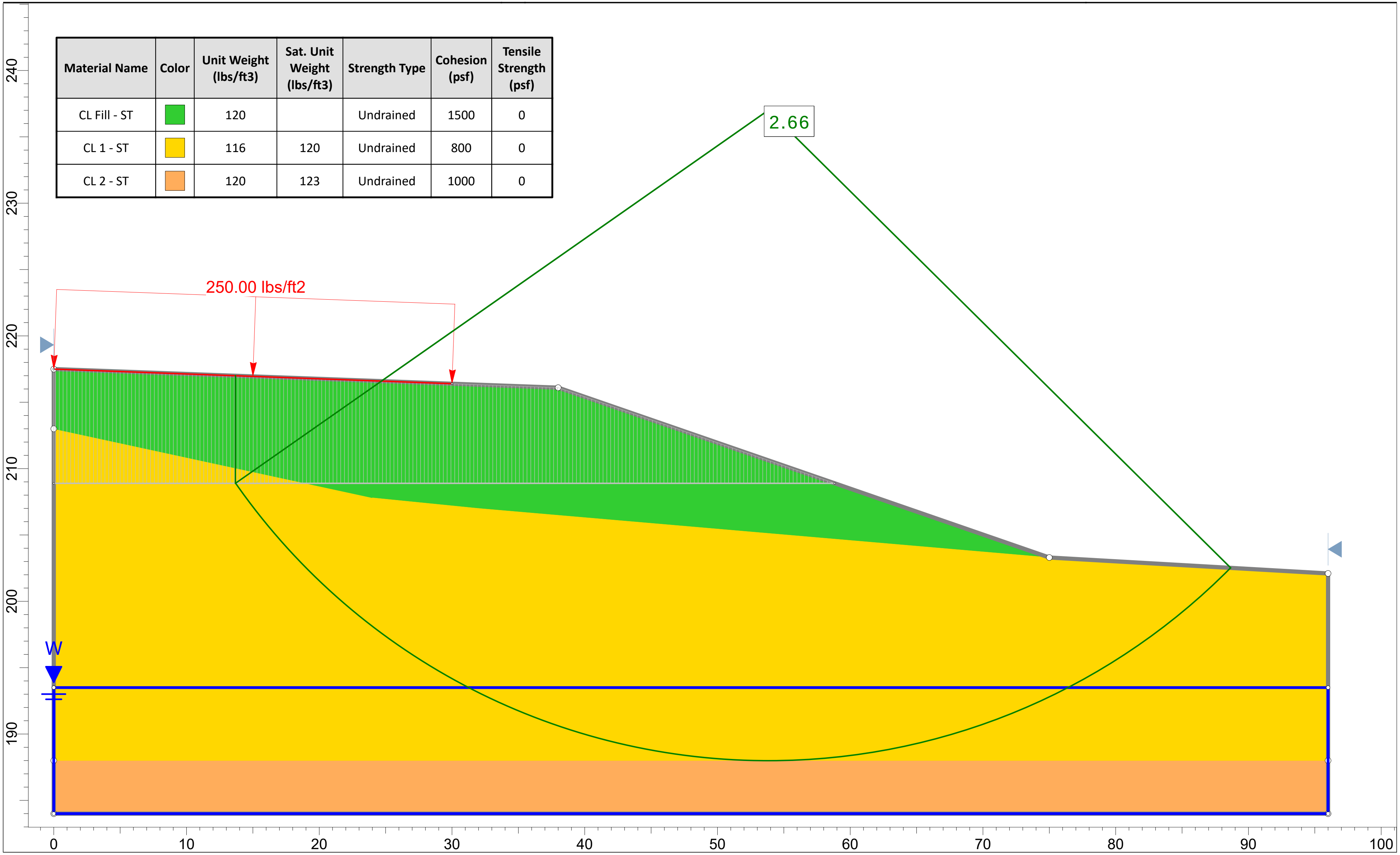
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: South Abutment Side Slope Station 216+00
Analysis Description: Short Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CL 1 - ST	🟨	116	120	Undrained	800	0
CL 2 - ST	🟧	120	123	Undrained	1000	0



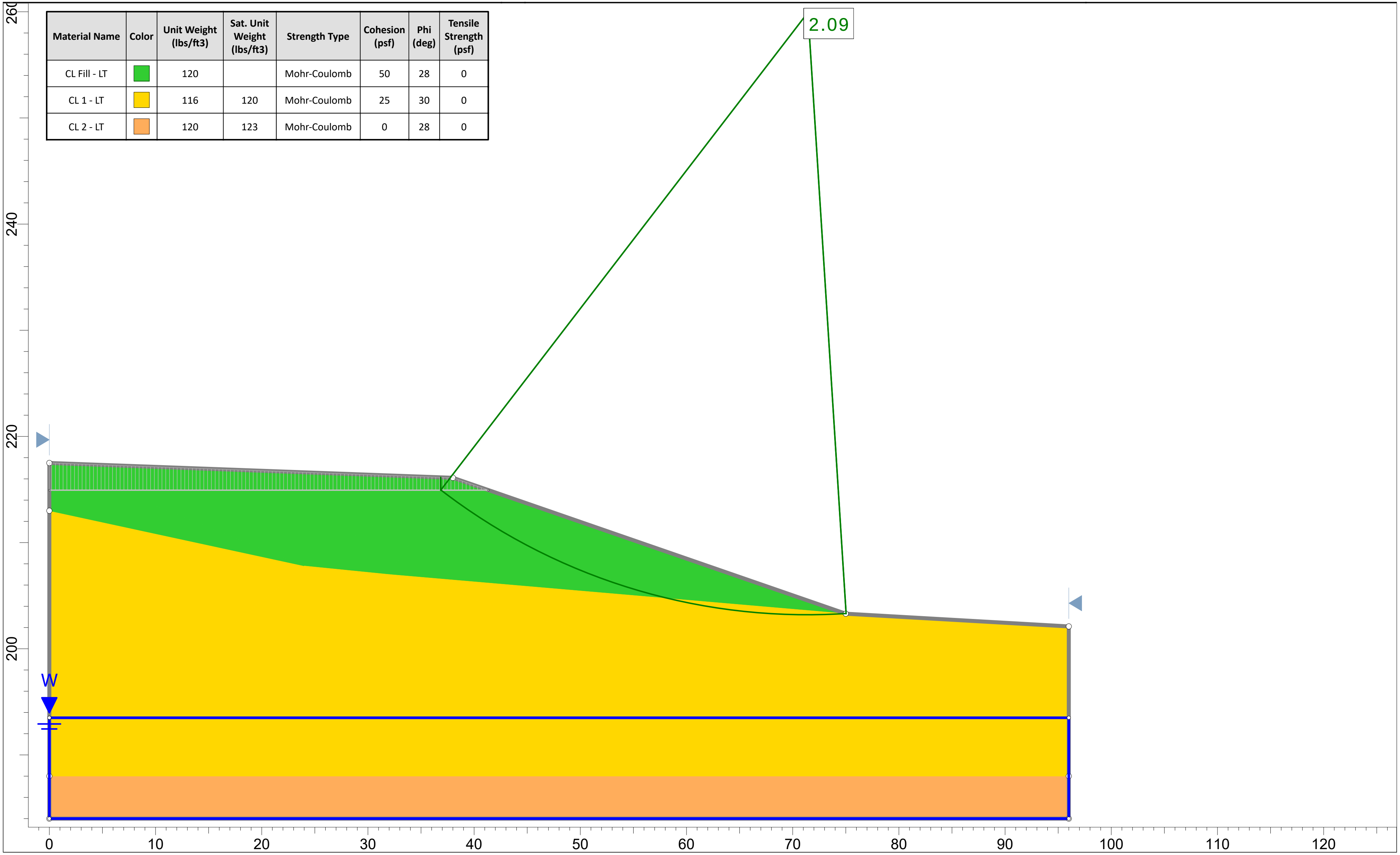
GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: South Abutment Side Slope Station 216+00
Analysis Description: Long Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 3/1/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - LT	🟩	120		Mohr-Coulomb	50	28	0
CL 1 - LT	🟨	116	120	Mohr-Coulomb	25	30	0
CL 2 - LT	🟧	120	123	Mohr-Coulomb	0	28	0

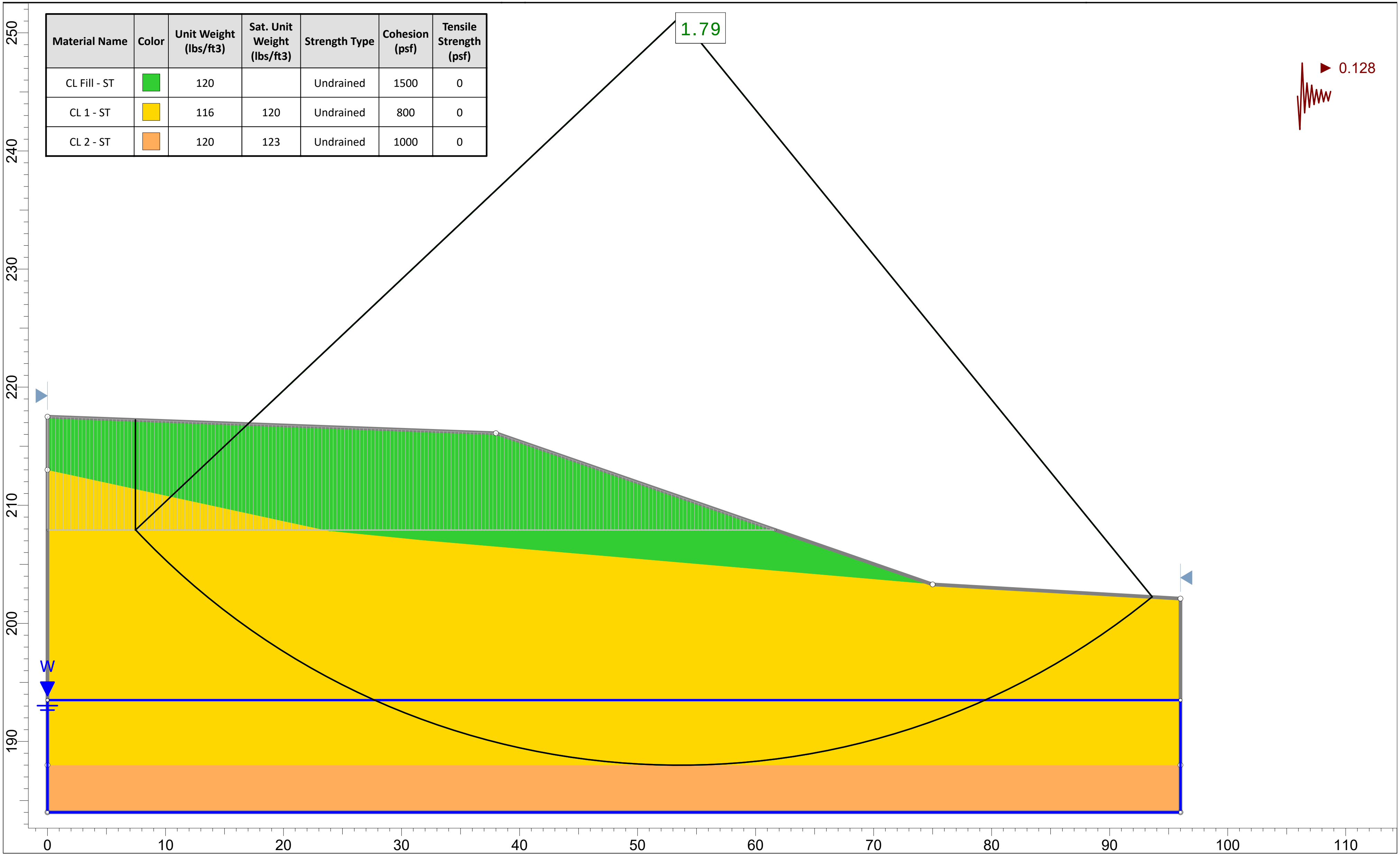


GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: South Abutment Side Slope Station 216+00
Analysis Description: Seismic
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CL 1 - ST	🟨	116	120	Undrained	800	0
CL 2 - ST	🟧	120	123	Undrained	1000	0

GEOTECHNOLOGY

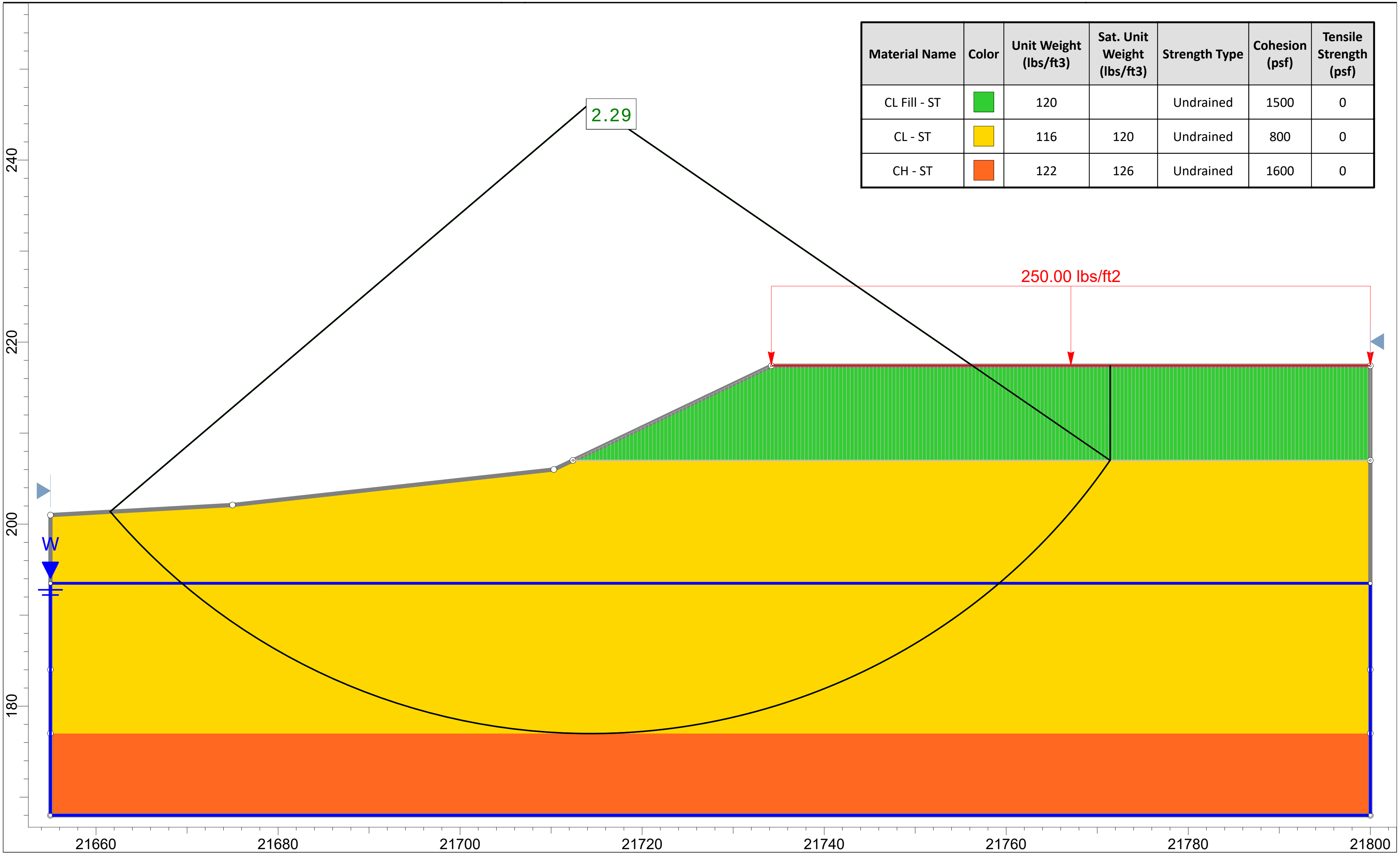
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: North Abutment Spill Slope
Analysis Description: Short Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CL - ST	🟨	116	120	Undrained	800	0
CH - ST	🟧	122	126	Undrained	1600	0



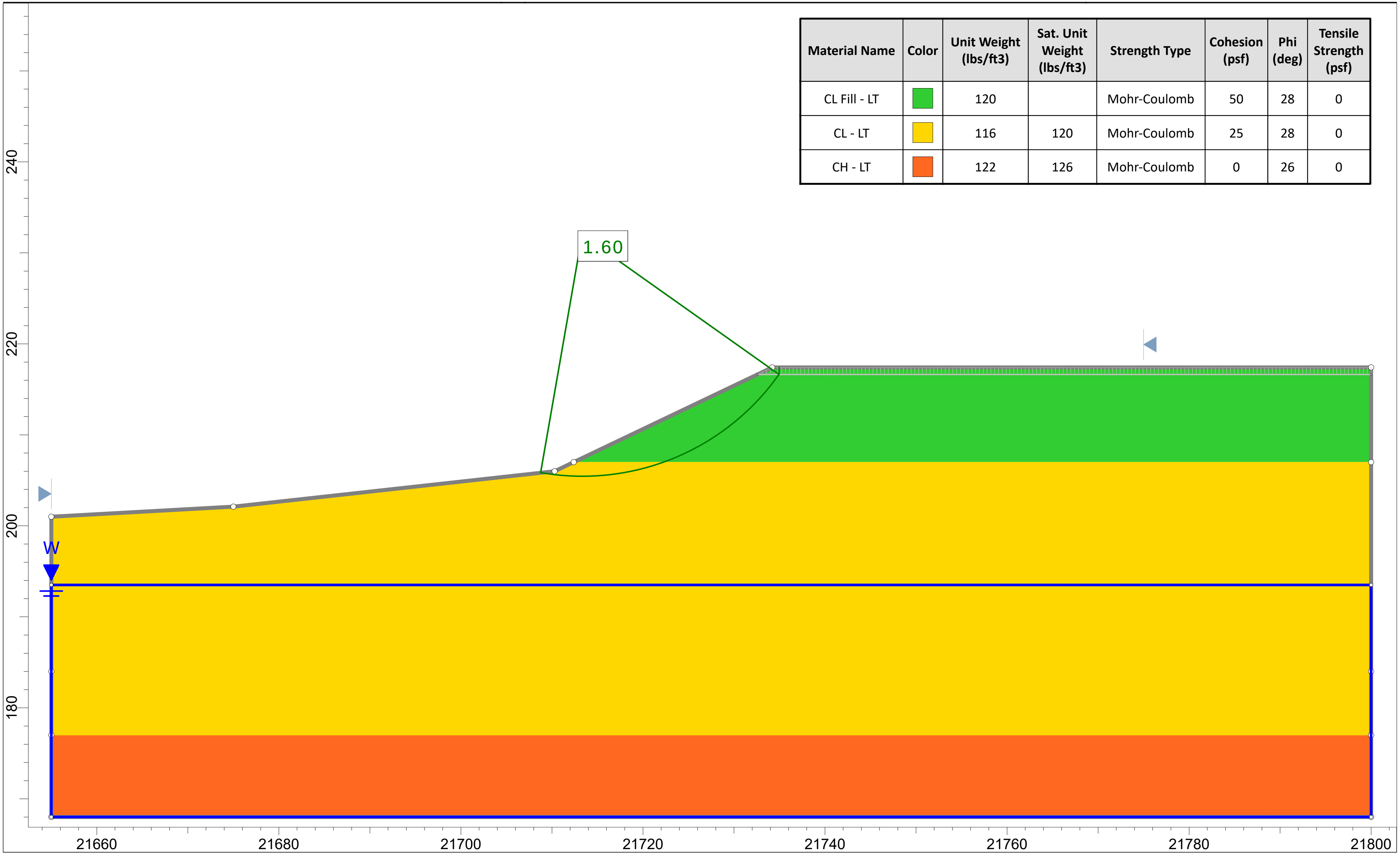
GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: North Abutment Spill Slope
Analysis Description: Long Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 3/1/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - LT	🟩	120		Mohr-Coulomb	50	28	0
CL - LT	🟨	116	120	Mohr-Coulomb	25	28	0
CH - LT	🟧	122	126	Mohr-Coulomb	0	26	0

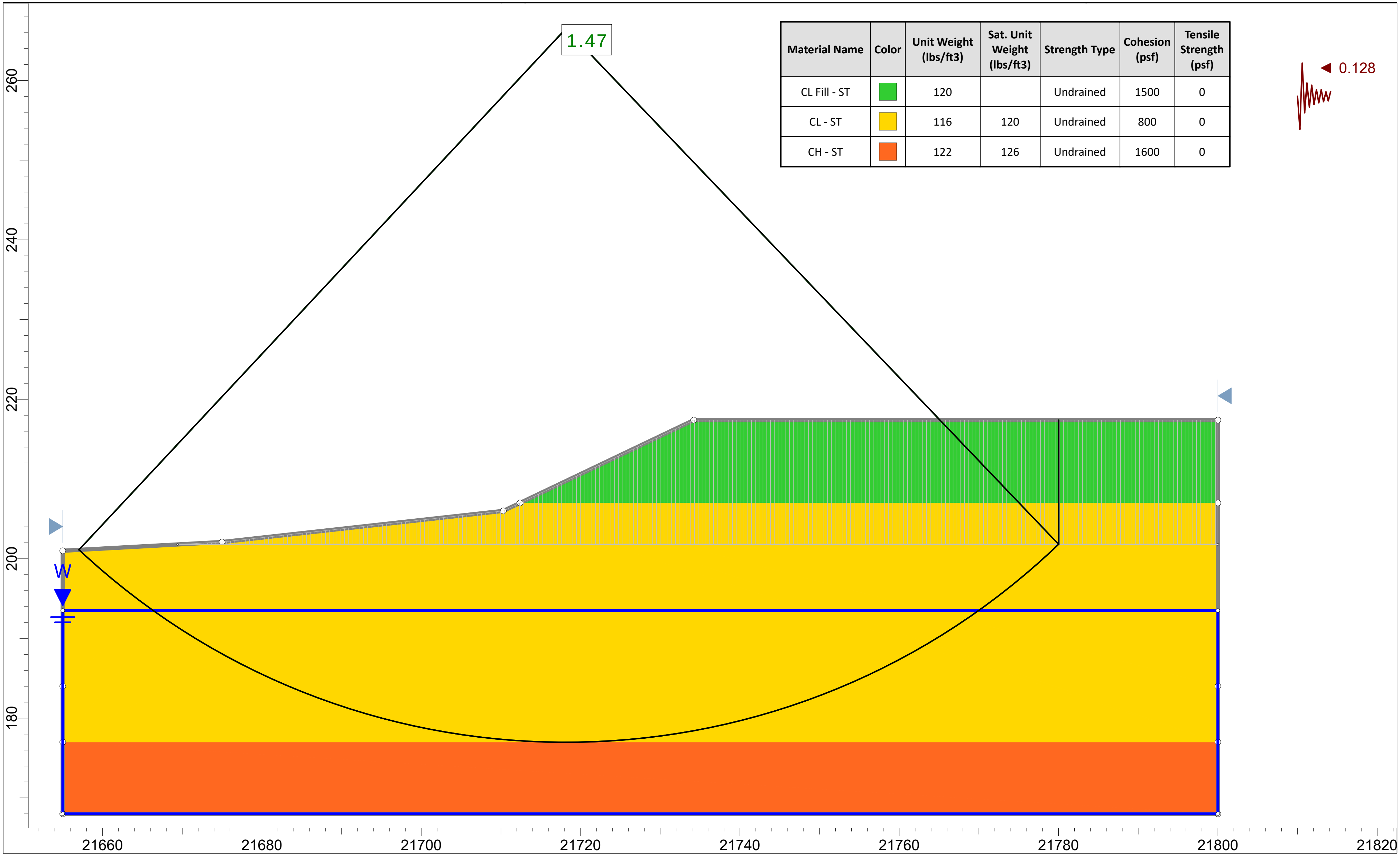


GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: North Abutment Spill Slope
Analysis Description: Seismic
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CL - ST	🟨	116	120	Undrained	800	0
CH - ST	🟧	122	126	Undrained	1600	0

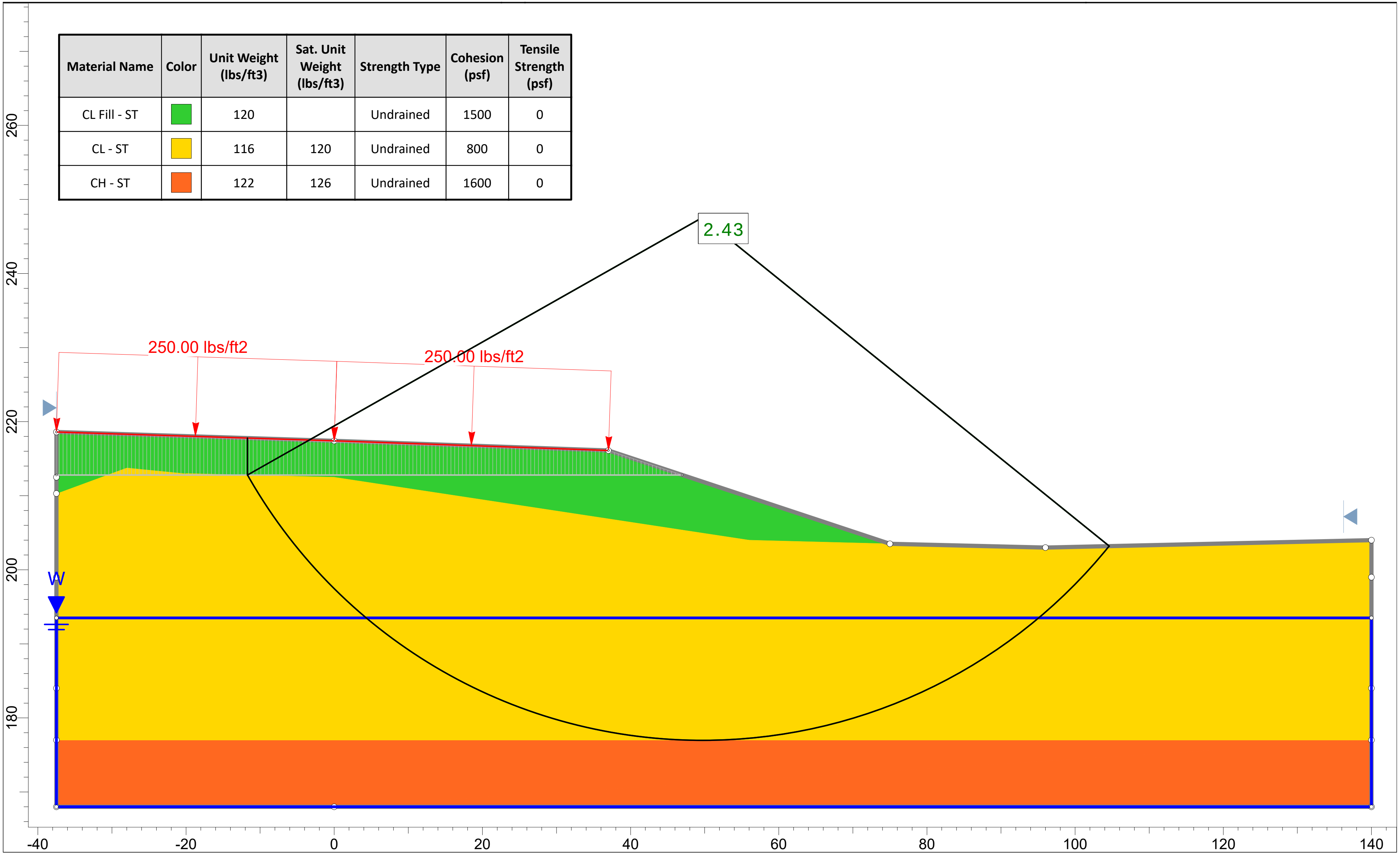
GEOTECHNOLOGY

A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: North Abutment Side Slope Station 217+30
Analysis Description: Short Term
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CL - ST	🟨	116	120	Undrained	800	0
CH - ST	🟧	122	126	Undrained	1600	0

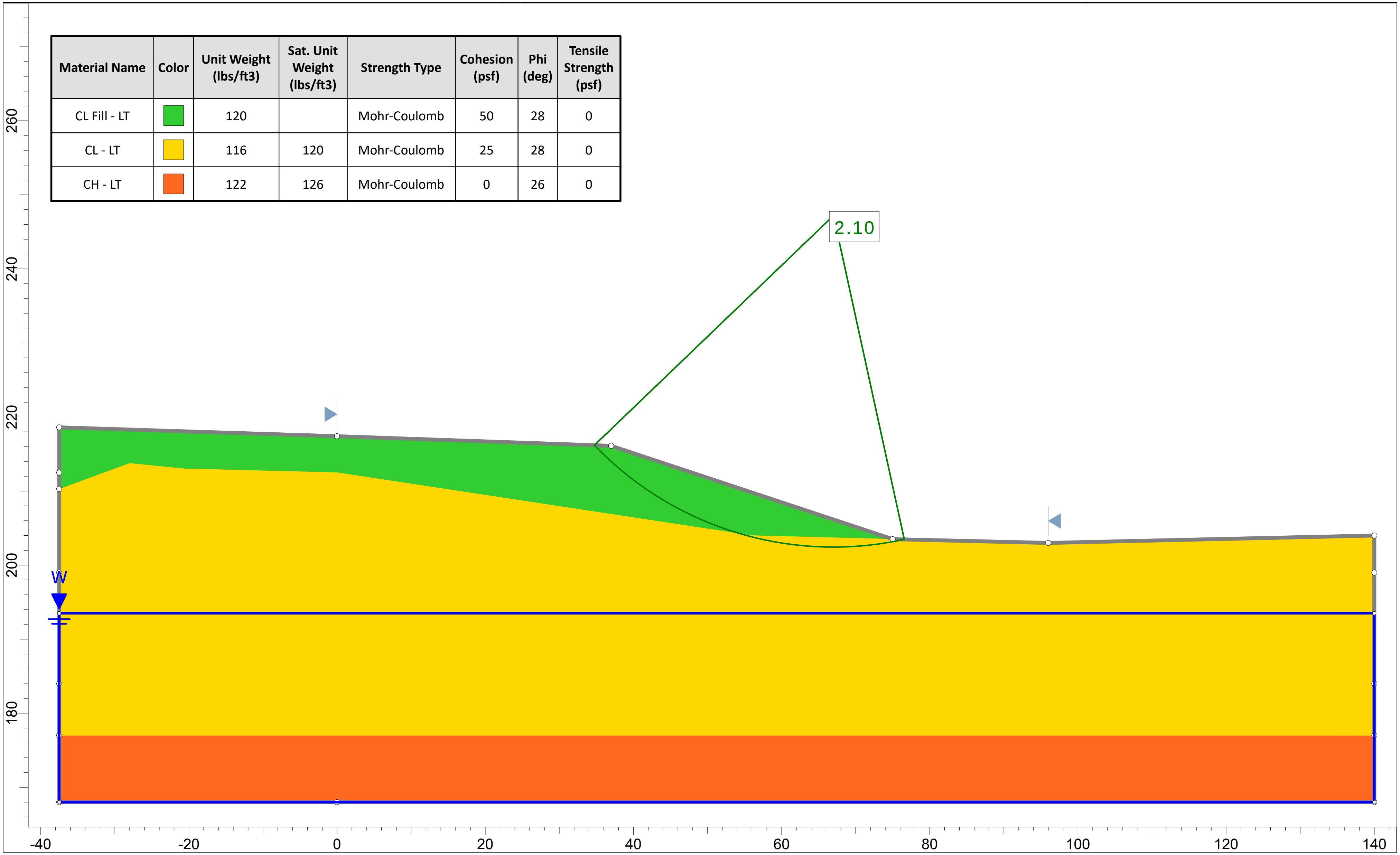
GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: North Abutment Side Slope Station 217+30
Analysis Description: Long Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 3/1/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - LT	🟩	120		Mohr-Coulomb	50	28	0
CL - LT	🟨	116	120	Mohr-Coulomb	25	28	0
CH - LT	🟧	122	126	Mohr-Coulomb	0	26	0

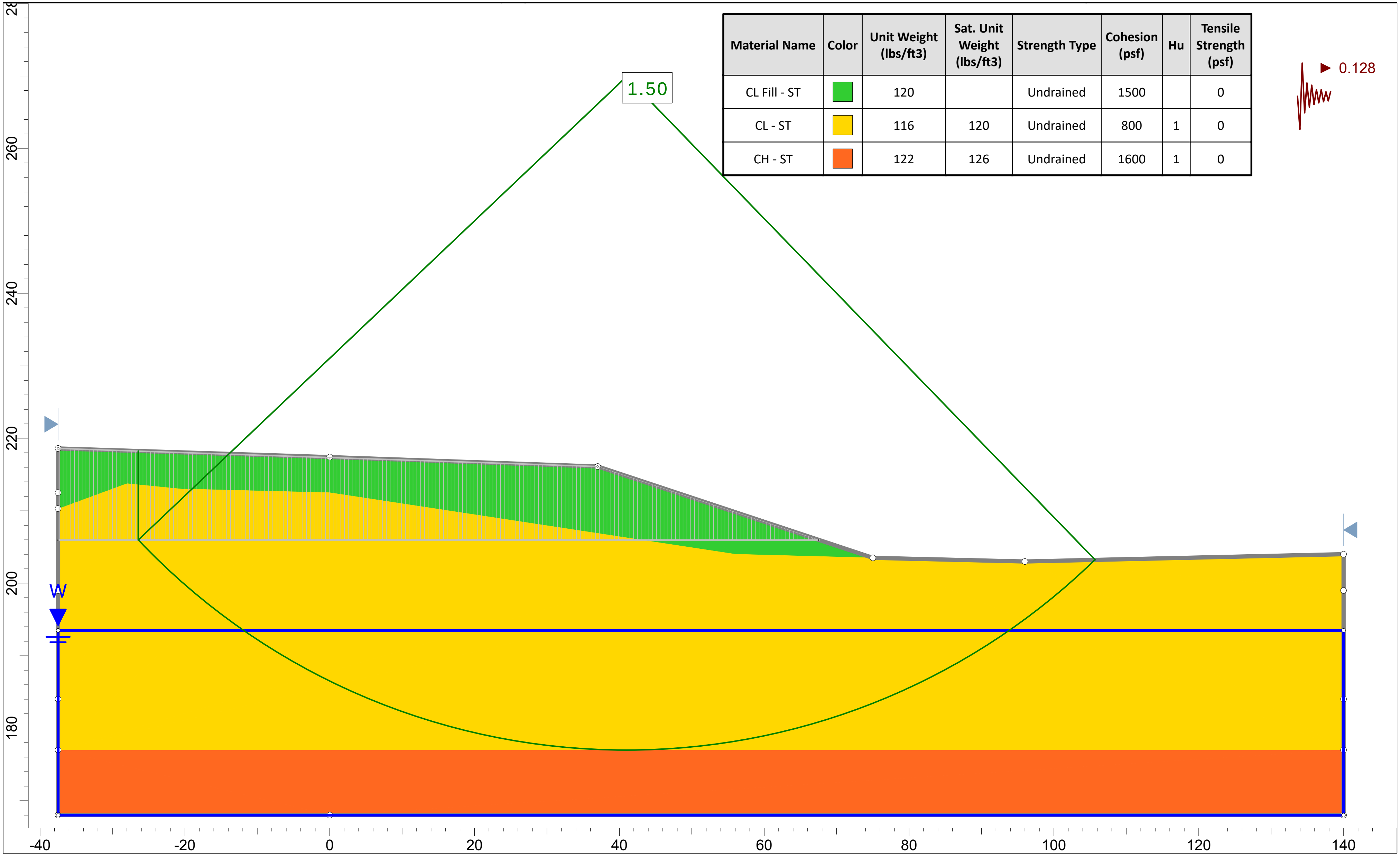


GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01859 Over La Grue Bayou South
Analysis Name: North Abutment Side Slope Station 217+30
Analysis Description: Seismic
Method: Spencer

ARDOT Project Number: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/11/2021

SLIDEINTERPRET 8.032

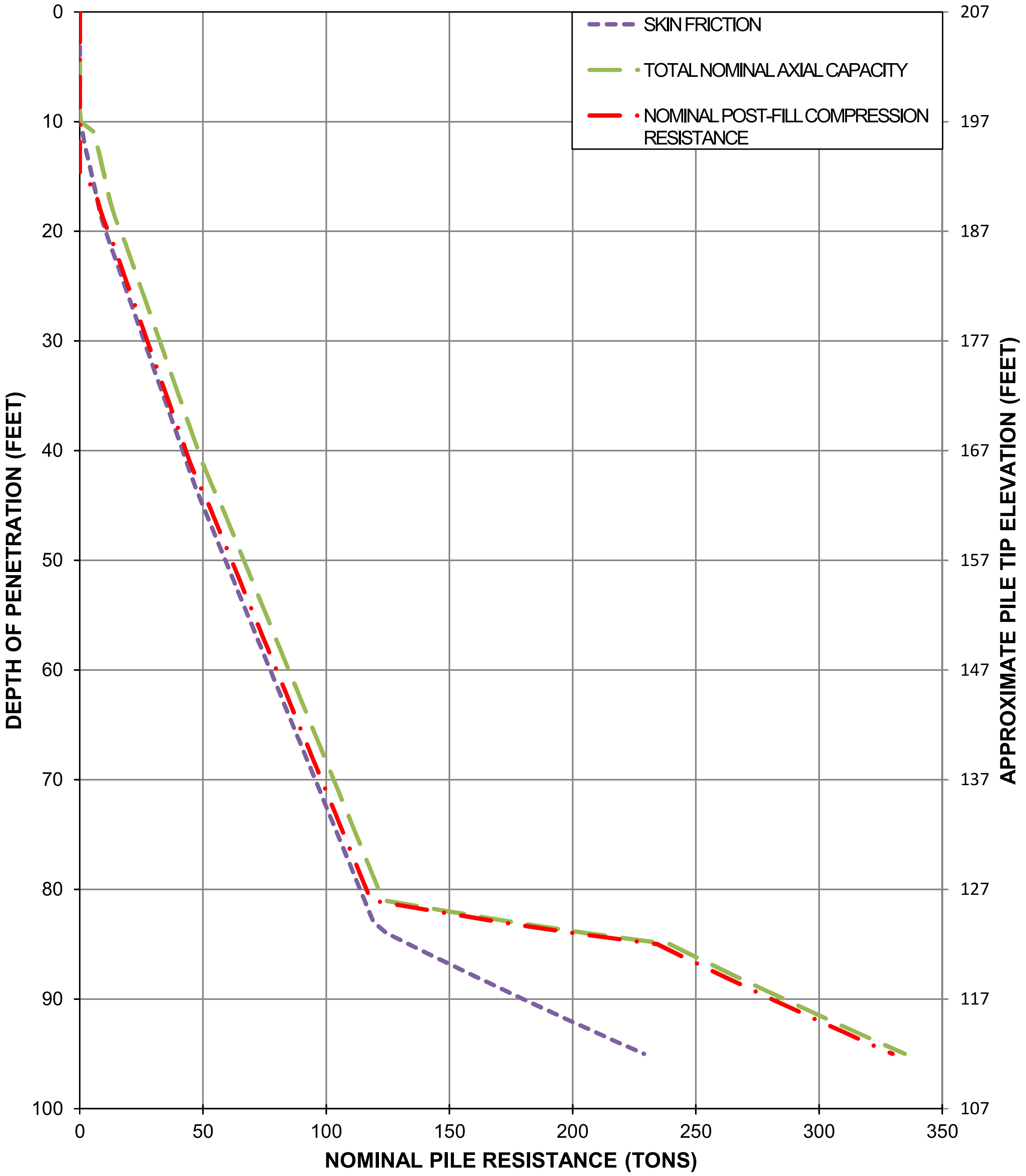


Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Hu	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500		0
CL - ST	🟨	116	120	Undrained	800	1	0
CH - ST	🟧	122	126	Undrained	1600	1	0

Appendix F
NOMINAL RESISTANCE CURVES FOR DRIVEN PILES

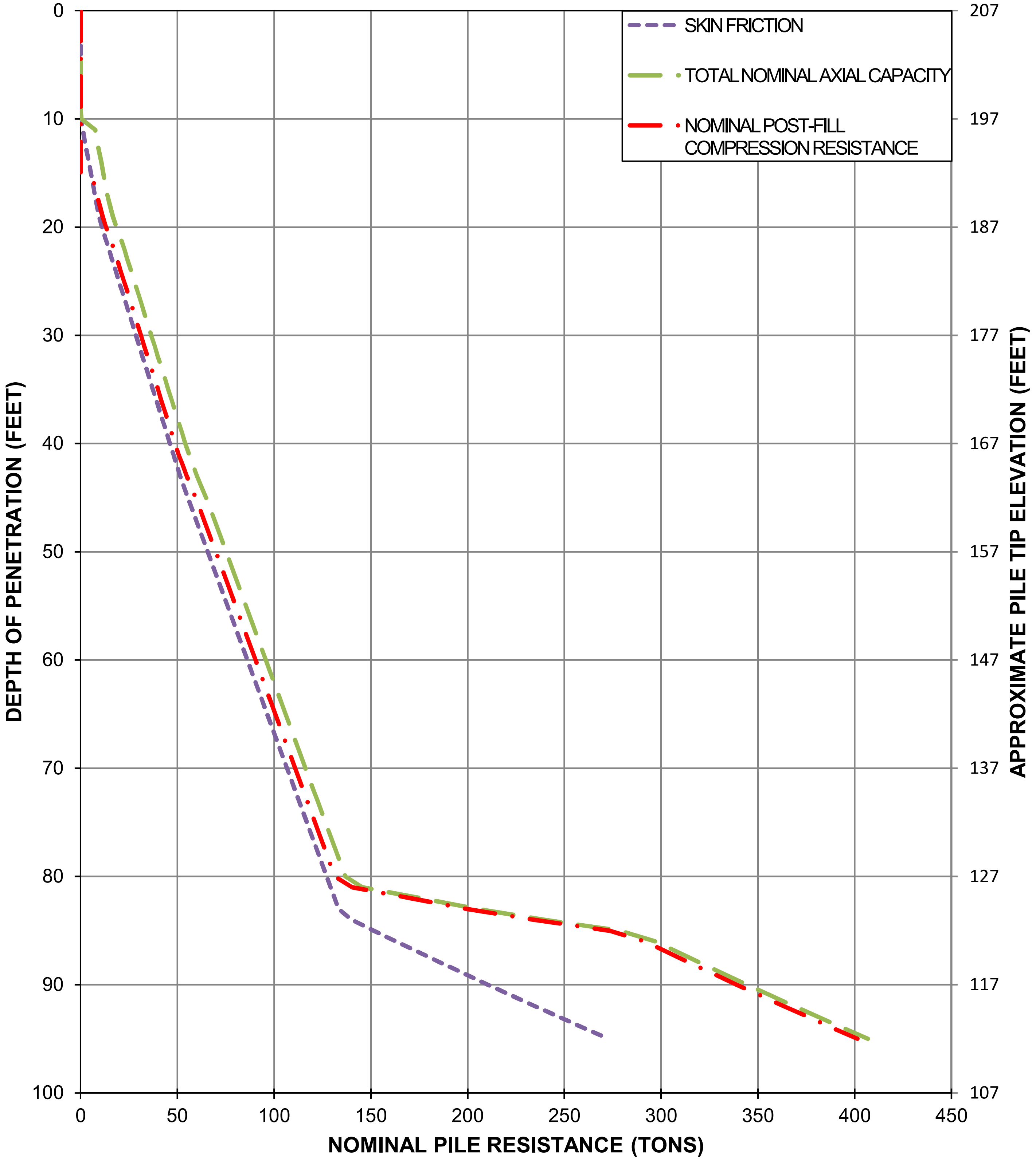
SOUTH ABUTMENT (BENT NO. 1)
HWY 63 BRIDGE NO. 01859 OVER LA GRUE BAYOU

NOMINAL RESISTANCE CURVES
DRIVEN 16-INCH, CLOSED-ENDED, PIPE PILES



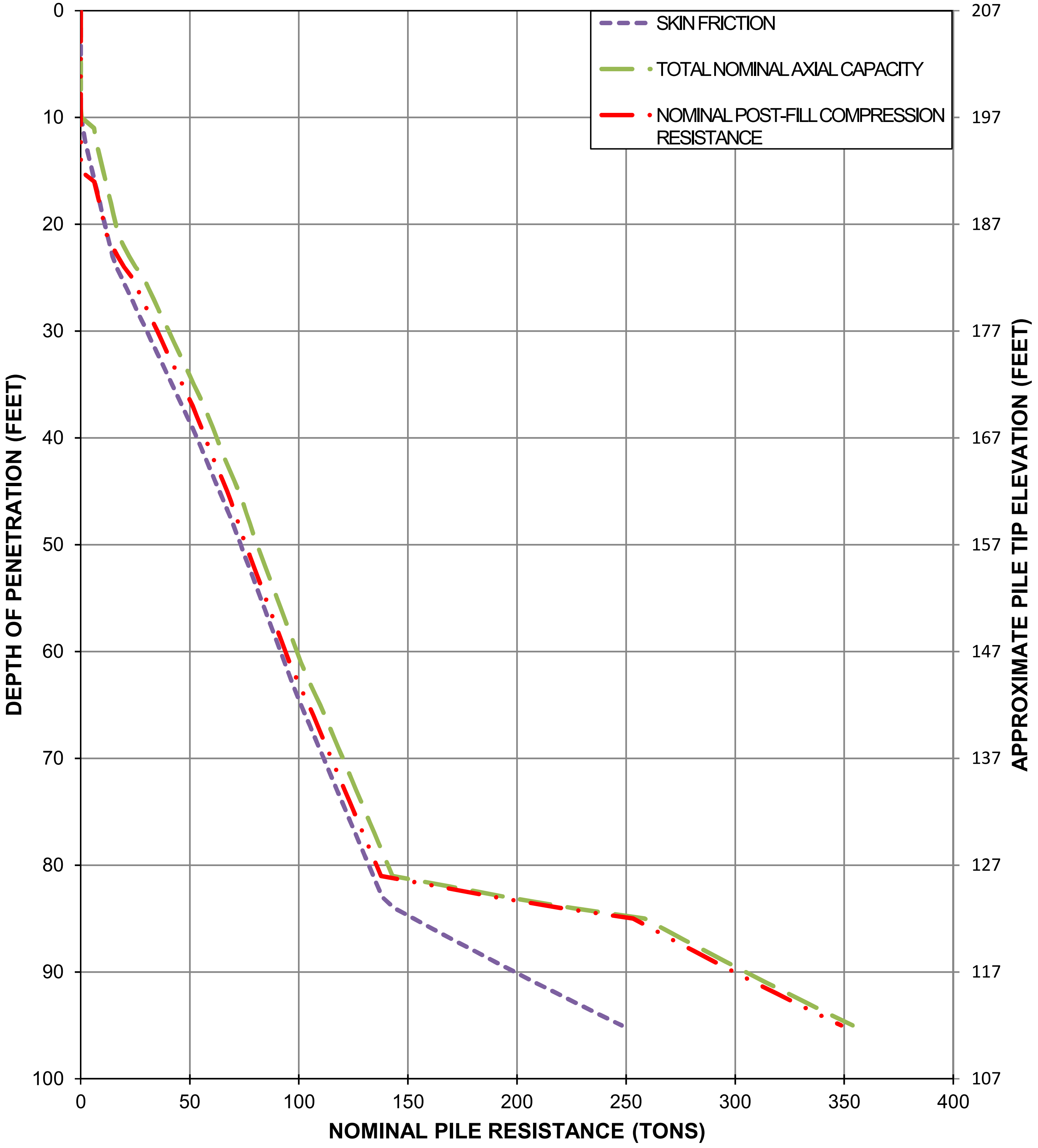
SOUTH ABUTMENT (BENT NO. 1)
HWY 63 BRIDGE NO. 01859 OVER LA GRUE BAYOU

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



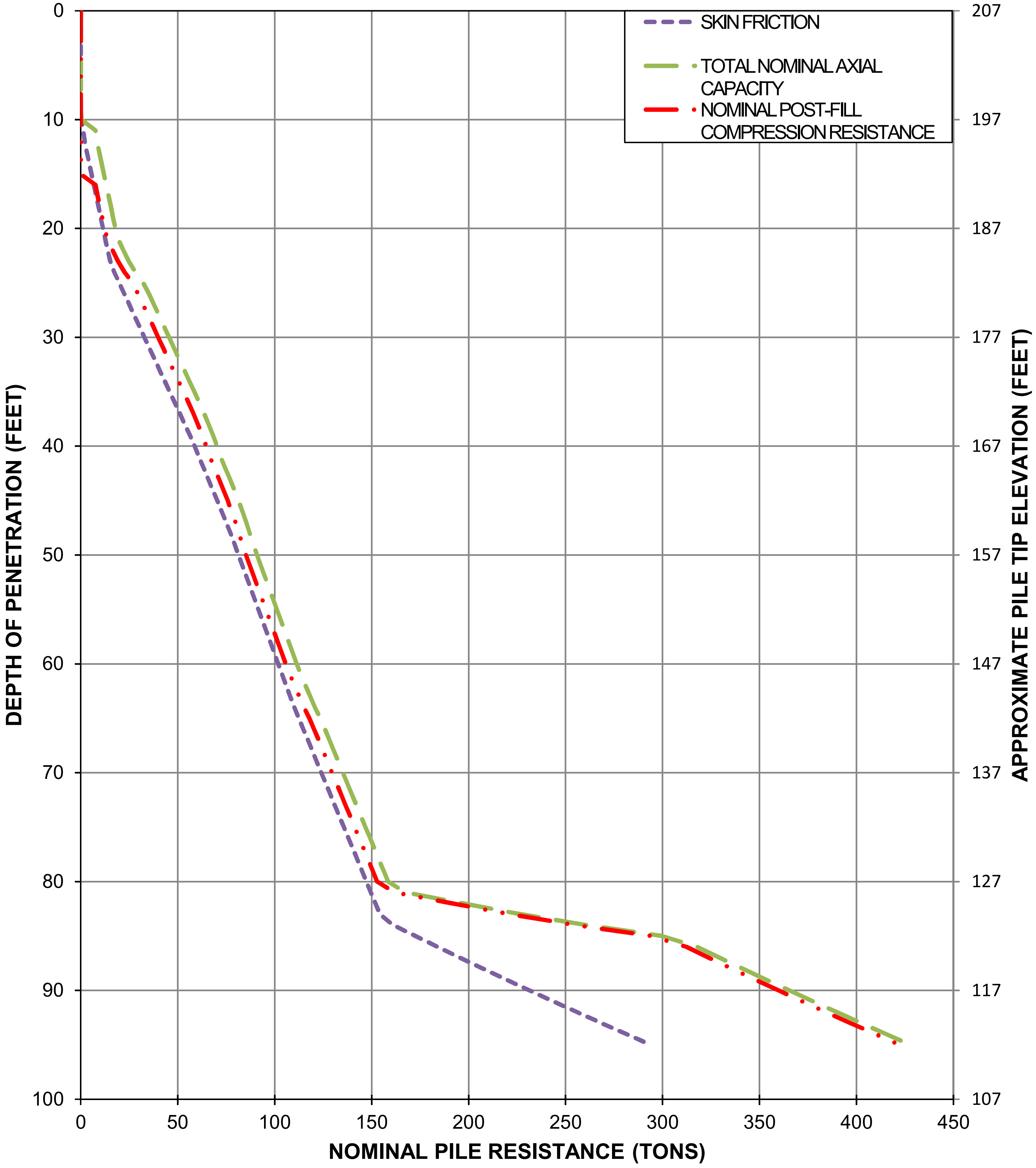
NORTH ABUTMENT (BENT NO. 2)
HWY 63 BRIDGE NO. 01859 OVER LA GRUE BAYOU

NOMINAL RESISTANCE CURVES
DRIVEN 16-INCH, CLOSED-ENDED, PIPE PILES



NORTH ABUTMENT (BENT NO. 2)
HWY 63 BRIDGE NO. 01859 OVER LA GRUE BAYOU

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



Appendix G
SOIL PARAMETERS FOR SYNTHETIC PROFILES

SOUTH ABUTMENT (BENT NO. 1) - BORING L-1

ZONE	SOIL TYPES	ELEVATION[a]		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)			
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)	SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]
1	Engineered Fill (Cohesive)	217[b]	207	120	1,500	--	50	28	0.007	500
2	Lean Clay	207	188	116	800	--	25	30	0.01	100
3	Lean Clay	188	174	120	1,000	--	--	28	0.007	500
4	Fat Clay	174	164	118	1,000	--	--	20	0.007	500
5	Fat Clay	164	134	122	1,200	--	--	20	0.007	500
6	Fat Clay	134	124	122	1,200	--	--	28	0.007	500
7	Silty Sand	124	1113	128	--	36	--	36	--	125

Note: Groundwater was not encountered in Boring L-1, but was encountered in Boring L-2 at approximate El 194. The effective unit weight should be used below the ground water level. Subtract 62.4 from the total unit weight to calculate the effective unit weight.

[a] Elevations are approximated from the provided drawing

[b] Approximate final grade at south abutment

[c] Pounds per cubic inch

[d] For lateral load analysis only

NORTH ABUTMENT (BENT NO. 2) - BORING L-2

ZONE	SOIL TYPES	ELEVATION[a]		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[D]	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)			
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)	SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[C]
1	Engineered Fill (Cohesive)	217[b]	207	120	1,500	--	50	28	0.007	500
2	Lean Clay	207	184	118	800	--	25	28	0.01	100
3	Lean Clay	184	177	121	1,600	--	--	26	0.007	500
4	Fat Clay	177	168	120	1,600	--	--	26	0.007	500
6	Fat Clay	168	158	120	1,400	--	--	20	0.007	500
7	Fat Clay	158	144	122	1,200	--	--	20	0.007	500
8	Lean Clay	144	124	124	1,400	--	--	20	0.007	500
9	Silty Sand	124	113	128	--	36	--	36	--	125

Note: Groundwater was encountered in Boring L-2 at approximate El 194. The effective unit weight should be used below the ground water level. Subtract 62.4 from the total unit weight to calculate the effective unit weight.

[a] Elevations are approximated from the provided drawing

[b] Approximate final grade at north abutment

[c] Pounds per cubic inch

[d] For lateral load analysis only

GEOTECHNICAL EXPLORATION

ROUTE 63 SECTION 11 STRUCTURES AND APPROACHES (S) – BRIDGE NO. 01860

PRAIRIE COUNTY, ARKANSAS

ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT NO. 061615
FEDERAL AID PROJECT NO. 9990

Prepared for:

GARVER USA
NORTH LITTLE ROCK, ARKANSAS

Prepared by:

GEOTECHNOLOGY, LLC
MEMPHIS, TENNESSEE

Date:

MARCH 23, 2023

Geotechnology Project No.:

J034561.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

St. Louis, MO | Erlanger, KY | Memphis, TN | Overland Park, KS | Cincinnati, OH | Fairview Heights, IL
Lexington, KY | Dayton, OH | Oxford, MS | Jonesboro, AR

GEOTECHNOLOGY
A Universal Engineering Sciences Company

March 23, 2023

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

Re: Geotechnical Exploration
ARDOT 061615
Route 63 Section 11 Structures and Approaches (S) – Bridge No. 01860
Prairie County, Arkansas
Geotechnology Project No. J034561.01

Dear Mr. Ruddell:

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

Amber Meadows
Amber Meadows
Project Engineer

Dale M Smith
Dale M. Smith, P.E.
Senior Project Manager

ABM/DMS/ASE/DBA:abm/dms

Copies submitted: Client (email)

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

Dale M Smith

3/23/23

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ROUTE 63 SECTION 11 STRUCTURES AND APPROACHES (S) – BRIDGE NO. 01860
Prairie County, Arkansas
March 23, 2023 | Geotechnology Project No. J034561.01

1.0 SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design, construction, and other related features for the proposed approach improvements and bridge replacement over La Grue Bayou along Highway 63 in Prairie County, Arkansas. The referenced features include demolition of the existing bridge and construction of a new bridge (Structure No. 01860). It is our understanding the anticipated foundation type for support of the new bridges is driven, closed-ended, pipe piles. The existing bridge approaches will be modified to facilitate traffic flow over the new bridge. A general overview of the project extents is shown on Figure 1 included in Appendix B.

The recommendations presented in this report are based on the geology, topography, and the results of the geotechnical exploration. Results of borings, in-situ testing, sampling, and laboratory testing are included in the report. A total of 2 borings were drilled in the vicinity of the site as shown on Figure 2 included in Appendix B. The boring logs, along with field and laboratory test results, are enclosed. The collected data have been analyzed and the physical properties of the in-situ soils summarized. General site conditions are discussed, along with recommendations for subgrade preparation. Important information prepared by the Geotechnical Business Association (GBA) for studies of this type is presented in Appendix A for your review.

2.0 GENERAL INFORMATION

Planned Modifications

The existing 2-lane, 101.5-foot long, 27.4-foot wide, 4-span bridge supported on precast concrete piles over La Grue Bayou (north) will be replaced with a 4-lane, 110-foot long, 77.5-foot wide, single-span bridge. The new bridge will be constructed east of the existing bridge. Riprap is planned along the abutment slopes based on the provided plans[1]; abutment slopes are anticipated to be two horizontal units for every vertical unit (2H:1V) and side slopes at the approaches are anticipated to be 3H:1V. Intersections of access drives will be modified to accommodate the new alignment. Up to approximately 5 and 12 feet of cut and fill, respectively, will be required to achieve design grades.

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[1] *Arkansas Department of Transportation Construction Plans for State Highway, La Grue Bayou, Wolf Island Slash & Honey Creek STRS. & APPRS. (S), Prairie County, Route 63 Section 11, Route 33 Section 5, Job 061615*, provided by ARDOT on September 28, 2020.

Drainage

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

Physiographic Setting & Geology

Prairie County is located in east-central Arkansas, in the Mississippi Embayment. The Mississippi Embayment is a trough-like depression containing thousands of feet of sediment and plunging southward along an axis approximating the present course of the Mississippi River. The deposits in the area consist of Holocene epoch alluvial gravel and sand. These materials are typically white to brown or gray, poorly to well sorted, fine- to coarse-quartz sand and gravel with minor silts and clays. These deposits form a broad terrace among the west side of the Mississippi River flood plan, and include both glacial outwash and non-glacial alluvium. Thickness can vary from 3 to 40 meters and may include loessal colluvium from nearby Crowley's Ridge.

3.0 GEOTECHNICAL EXPLORATION

Two borings were drilled between August 12 and 13, 2020 with a rotary drill rig (CME 55) using hollow-stem auger and wash rotary drilling methods. The borings were drilled to an approximate depth of 100 feet. 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 automatic hammers. Thin-walled Shelby tube samples were collected in cohesive soils at selected depths. Groundwater observations were made during drilling operations.

The collected samples were visually examined by field staff and transported to our laboratory for further evaluation and testing. The samples were examined in the laboratory by a geotechnical professional who prepared descriptive logs of the materials encountered. The boring logs are presented in Appendix C. An explanation of the terms and symbols used on the boring logs is also provided in Appendix C. Included on each boring log are ground surface elevation, station and offset provided by representatives of ARDOT. 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

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, pH, resistivity, and unconsolidated-undrained triaxial compression (UU) test results are also provided in Appendix D. The laboratory tests and corresponding test method standards are presented in Table 2.

Table 2. Summary of Laboratory Tests and Methods.

Laboratory Test	ASTM	AASHTO
Moisture Content	D 2216	T 265
Atterberg Limits	D 4318	T 98
Grain Size Analysis by Sieving	D 6913	T 88
Unconsolidated-Undrained Triaxial Compression	D 2850	T 296
Soil Electrical Resistivity	G 57	T 288
Soil pH	D 4972	T 289

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

5.0 SUBSURFACE CONDITIONS

General Subsurface Conditions

The borings at this site include Borings L-3 and -4 and were drilled in the northbound shoulder of the existing, southern and northern approaches, respectively. Beneath the approximately 7- and 4-inch thick layer of asphalt and 11- and 8-inch thick cement-treated base material in Borings L-3 and -4, respectively, the soils generally consisted of fine-grained soil underlain by coarse-grained soil layers to the 100-foot maximum depth of exploration. More detailed descriptions of the stratigraphy encountered at each bridge are included below and on the boring logs in Appendix C.

Fine-Grained Soil. Underlying the pavement materials, the stratigraphy generally consisted of predominantly fine-grained soils underlain by predominantly coarse-grained soil at depths of approximately 78 and 63 feet in Borings L-3 and -4, respectively. The predominantly coarse-grained soil layer extended to the 100-foot maximum depth of exploration in both borings. The fine-grained soils were classified as lean clay (CL), fat clay (CH), elastic silt (MH), and silt (ML) by the Unified Soil Classification System (USCS) and A-6, A-7-6, A-7-5, or A-4 by the AASHTO classification method. The fine-grained soils were very soft to stiff based on SPT N-values, and medium stiff to very stiff based on UU tests. The laboratory testing used to determine USCS and AASHTO classifications are presented in Appendix D.

Coarse-Grained Soil. The underlying, coarse-grained soils were classified as intermixed, poorly-graded sand (SP and SP-SM by USCS; A-1-b, A-3, or A-2-4 by AASHTO), silty sand (SM by USCS, A-1-a, A-1-b, A-2-4, A-2-6, or A-4 by AASHTO), and clayey sand (SC by USCS, A-4, A-6, and A-7-6 by AASHTO). Based on field test results, the coarse-grained soils were loose to very dense. Very dense sand (N > 50 bpf) was encountered in Boring L-3 from approximate depths of 98 to 100 feet (approximately El[2] 115 to El 113) and in Boring L-4 from approximate depths of 83 to 100 feet (approximately El 126 to El 113).

Soil pH and Resistivity Test Results

In addition to laboratory soil classification and strength testing, soil corrosion 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 to 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 below and are included in Appendix D.

Table 3. Results of Soil Corrosion Testing.

Boring	Sample No.	Sample Depth (foot)	Soil Resistivity (ohm-cm)	Soil pH
L-3	ST-6	18	1,425	2.99
	SS9	33	741	8.25
	ST-15	58	1,254	8.12
	SS17	68	1,197	7.75
L-4	SS5	13	1,881	3.85
	ST-8	28	855	8.01
	ST-11	43	741	8.04
	SS14	58	599	8.21

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.
- pH less than 5.5.
- pH between 5.5 and 8.5 in soils with high organic content.

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

- Resistivity less than 3,000 ohm-cm.

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[2] Elevations are referenced in feet; datum was not provided.

- pH less than 5.5

Results of the corrosion and deterioration testing indicate the site has potential for pile or steel reinforcement deterioration. Interpretation of the data and corrosion protection of the bridge structural components should be performed by the design team.

Groundwater

Groundwater was encountered at an approximate depth of 19½ feet during drilling operations in Boring L-4. The presence of higher groundwater levels in Boring L-3 could have been obscured by the use of mud rotary drilling methods, which introduces fluid to the borehole. Groundwater levels could vary significantly over time due to water levels in La Grue Bayou and seasonal variation in precipitation, recharge, 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 1V:4H must be benched prior to placing new fill. Slope ratios of 1V:3H or flatter are recommended for all cut and fill slopes along the proposed alignment, based on the results of global stability analyses (discussed in a subsequent section).

Cut Areas. After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 95% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D 698/AASHTO T 99). Areas supporting pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

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

Fill and Backfill Placement. Fill and backfill should be placed in level lifts up to 8 inches in loose thickness. For fill and backfill exhibiting a well-defined moisture-density relationship, each lift should be moisture-conditioned to within ±2% of the optimum moisture content and compacted with a sheepsfoot roller or self-propelled compactor to a minimum of 98% of the maximum dry unit weight as determined by the standard Proctor test. Moisture-conditioning can include: aeration and drying of wetter soils, wetting drier soils, and/or mixing wetter and drier soils into a uniform blend. The upper three feet of fill and backfill 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 at least 70% of the relatively density as evaluated from the maximum and minimum index densities measured by ASTM D4253 and D4254, respectively. The upper 3 feet of fill and backfill beneath the base of pavement should be compacted to at least 75% of the relatively density.

Moisture Considerations. The soils encountered in the borings are relatively wet and will most likely require drying. The time for drying will depend on the weather conditions during grading activities. We recommend construction take place during dry weather conditions. Wet weather conditions can cause rutting of the surficial soils which will require drying and recompacting.

Maintaining the moisture content of bearing and subgrade soils within the acceptable range is important during and after construction for the proposed structure. 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.

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

Seismic Considerations

Earthquake Risk. The project area is located within the New Madrid Seismic Zone (NMSZ). The NMSZ is located in the northern part of the Mississippi Embayment and trends in a northeast to southwest direction from southern Illinois to northeast Arkansas. In December 1811, a series of large magnitude earthquakes occurred, which were centered near New Madrid, Missouri. Three strong earthquakes occurred over a 3-month period 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", eighth edition (2017). A peak ground acceleration of 0.255g was obtained from published values.

Seismic Design Parameters. Presented in Table 4 are seismic design parameters based on a seismic hazard with 7% probability of exceedance in 75 years and the field and laboratory testing results.

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

Latitude 34.708138°N/Longitude 91.555041°W		
Category/ Parameter	Designation/ Value	Reference
Seismic Zone	2	AASHTO LRFD 2017 Table 3.10.6-1
Seismic Site Class	D	AASHTO LRFD 2017 Table 3.10.3.1-1
S_S	0.383g	Ground motion parameters obtained from the computer program supplied with the AASHTO Guidelines for the Seismic Design of Highway Bridges (2009) using the indicated latitude and longitude coordinates of the project site and the seismic site class based on boring data.
S_1	0.112g	
F_a	1.494	
F_v	2.351	
F_{PGA}	1.447	
t_s	0.462	
t_0	0.092	
S_{DS}	0.572g	
S_{D1}	0.264g	
PGA	0.176g	
As	0.255g	

Liquefaction and Dynamic Settlement. A study was performed to evaluate the liquefaction and dynamic settlement potential at the site. Field and laboratory data were used to perform the analyses. The field measurements included the assumed depth of the water table and the SPT N-values. The laboratory data included USCS/AASHTO classification and soil unit weight. An earthquake magnitude (Mw) of 7.5 was considered. A peak ground acceleration of 0.255g was utilized as obtained from the referenced Seismic Design Maps. Groundwater was set at a depth of approximately 19 feet measured from the approximate ground surface at the location of Boring L-4. Based on the analyses, potential for liquefaction is low at the site.

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. Geotechnology evaluated this condition, and more information is provided in the Global Stability section of this report.

Approach Embankment Settlement

Settlement analyses were performed to assess fill-induced settlement for the approaches. Based on the plans provided, approximately 12 feet of fill will be required at the proposed abutments to bring the sites to grade. For settlement analyses, we assumed cohesive, engineered fill will be used for the fill material. The results of the settlement analyses are shown in Table 5. If grade changes will require the placement of additional fill, Geotechnology should be contacted to perform additional settlement analyses for fill-induced settlement at the approaches.

Table 5. Summary of Estimated Settlement.

South Abutment			North Abutment		
Max Fill (feet)	Estimated Settlement (inches)		Max Fill (feet)	Estimated Settlement (inches)	
	Immediate	Consolidation		Immediate	Consolidation
12	1½	½	12	1½	½

Discussion of Fill-Induced Settlement. The results of the settlement analyses indicate immediate and long-term consolidation settlement at the approaches. We anticipate the immediate settlement to occur within a week of fill placement. Based on the one-dimensional consolidation test performed on a sample collected from Boring L-1 at the adjacent bridge (No. 01859), practical completion of consolidation induced settlement will occur within 2 to 3 weeks following fill placement.

It should be noted the one-dimensional consolidation test confines the drainage pathway to one dimension while in the field, drainage takes place in three dimensions; therefore, it is our professional opinion the estimated settlement will occur in a shorter time period. If the anticipated waiting period adversely impacts project schedule, a settlement monitoring program may be implemented and survey data be forwarded to Geotechnology so that construction can commence as soon as the remaining settlement is tolerable.

Global Stability

Based on the provided plans, abutment fill will be placed at a 2H:1V slope and side slopes will be constructed at 3H:1V slopes on top of the varying existing grades. Geotechnology performed stability analyses for deep-seated, global failure of bridge abutment slopes using the computer program Slide. 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 Table 6. A pseudo-static seismic acceleration of 0.128g, corresponding to one-half the peak ground acceleration (per FHWA Publication NHI-11-032) was utilized for the seismic condition. Section profiles with calculated critical failure surfaces and utilized soil parameters are presented in Appendix E for selected analyses.

Table 6. Slope Stability Analyses Results.

Location	Slope Ratio	Approximate Berm | Embankment Height (ft.)	Calculated Factor of Safety		
			Short-Term Static[a]	Long-Term Static[a]	Seismic[b]
South Abutment Side Slope	3:1	-- | 12	2.68	2.09	1.76
South Abutment Spill Slope	2:1	13 | 18	2.20	1.50	1.73
North Abutment Side Slope	3:1	-- | 12	2.98	2.04	2.15
North Abutment Spill Slope	2:1	13 | 18	2.89	1.56	2.22

Note: Berm height is defined as height of the 2H:1V spill slope.

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

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

The models used in this computation did not consider the relative stabilizing effect of foundation piles driven to support the abutments or armoring of abutments with rip rap or concrete. In general, foundation piles may provide additional stabilizing force to the abutment slopes, resulting in a factor of safety higher than those presented in Table 6. Existing slopes should be benched prior to placing new fill to reduce the potential for development of slip planes between new and existing fill.

Driven Pile Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2017). It is our understanding 16- and 18-inch diameter, closed-end, steel, pipe piles are being considered for support of the proposed bridge. Geotechnology should be notified if a different foundation type is being considered.

Synthetic profiles have been compiled for each abutment based upon the soil profile encountered in the borings, approximate boring elevations, and the proposed final grade. Based on the provided information, the pile cap elevation at both abutments is approximately El 207 and the upper 10 feet beneath the pile cap at each pile location will be pre-bored. Soil parameters, including LPILE parameters, for each abutment are included in Appendix G.

Nominal resistance curves showing axial resistance from skin friction and total axial capacity (skin friction + end bearing) for the abutments are presented in Appendix F. Nominal resistances at each abutment are presented in Table 7. Uplift (tension) capacities may be calculated using the resistance provided by skin friction.

Table 7. Nominal Static Axial Resistance of Driven Piles.

Location	Pile Diameter (inches)	Embedment Length (feet)	Nominal Static Resistance (tons)			Nominal Post-Fill Resistance (tons)	
			Side Resistance	End Bearing	Compression Total	Compression Total[a,b]	Drag Load
South Abutment	16	60	97	23	120	102	18
		70	142	23	165	147	18
		80	215	105	320	302	18
North Abutment		60	100	20	120	106	14
		70	160	20	180	166	14
		80	235	105	340	326	14
South Abutment	18	60	110	30	140	120	20
		70	165	35	200	180	20
		80	260	130	390	370	20
North Abutment		60	110	35	145	130	15
		70	185	35	220	205	15
		80	287	133	420	405	15

[a] Nominal post-fill resistance has not been reduced by the drag load.

[b] Based on estimated settlement due to placement of fill.

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

Table 8. Resistance Factors Based on Static Analysis Methods.

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

Based on AASHTO LRFD (2017) Table 10.5.5.2.3-1, a higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 9.

Table 9. Resistance Factors for Driven Piles.

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

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

Pile Group Considerations. The settlement of pile groups should be evaluated as per AASHTO LRFD (2017) 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 (2017) 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 (2017) sections 10.7.3.9 and 10.7.3.11.

Driven Pile Construction Considerations. Minimum hammer energies required to drive the piles were evaluated using the computer software WEAP. The recommended minimum hammer energy is provided in Table 10.

Table 10. Minimum Hammer Energies.

Pile Diameter [a] (inches)	Location	Embedment Length (feet)	Required Capacity (tons)	Minimum Rated Hammer Energy (kip-feet)
16	North and South Abutment	75	283	64

[a] Closed-ended pipe piles with ½-inch thick walls.

Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. Alternatively, potential driving criteria can be developed using wave equation analyses after the pile hammer is selected.

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 (2017) section 10.7.3.8.2. 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 (2017) 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 (2017) Table 10.5.5.2.3-1 or 2 percent of the production piles, whichever results in a larger number of tests. We recommend that the identified piles be tested at the end of initial drive (EOID) and a restrike performed at a minimum seven days after EOID.

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

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

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

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

Lateral Resistance. The lateral resistance of pile foundations depends on the length and dimensions of the foundation and the soil characteristics. The lateral resistance of pile foundations can be computed using the computer program LPILE to model the behavior of a single pile or shaft. Soil parameters are provided in Appendix G 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 (2017) to determine lateral resistance of a pile group. Alternatively, the GROUP software can be used to evaluate the lateral resistance of the pile/shaft groups. The resistance factor for lateral resistance of single piles or pile groups is 1.0.

Downdrag

The AASHTO LRFD (2017) 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 the foundations due to settlement of the soil layers. The relative movement of the soil layers versus the shaft depends on the final foundation configuration.

Downdrag Due to Fill-Induced Settlement. Based on settlement analysis performed for the 12-foot maximum fill placement at the abutments, up to 2 inches of settlement is predicted. Approximately 1½ inches of settlement is estimated to occur immediately after fill placement, and ½-inch of consolidation settlement is estimated to occur within approximately 2 to 3 weeks following fill placement. As discussed previously, we understand the contractor will pre-bore 10 feet prior to driving piles. Pile driving should not begin for at least two weeks following completion of fill placement to allow time for settlement to be essentially complete. Piles driven after fill placement prior to completion of settlement will be subject to drag loads as the soil below the fill consolidates due to the weight of the fill.

Downdrag Due to Dynamic Settlement. Based on the low liquefaction potential at this site, liquefaction-induced drag loads were not considered.

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 <u>not</u> 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 <u>not</u> be adequate to develop geotechnical design recommendations for the project.

Do <u>not</u> 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 <u>not</u> rely on an executive summary. Do <u>not</u> 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.

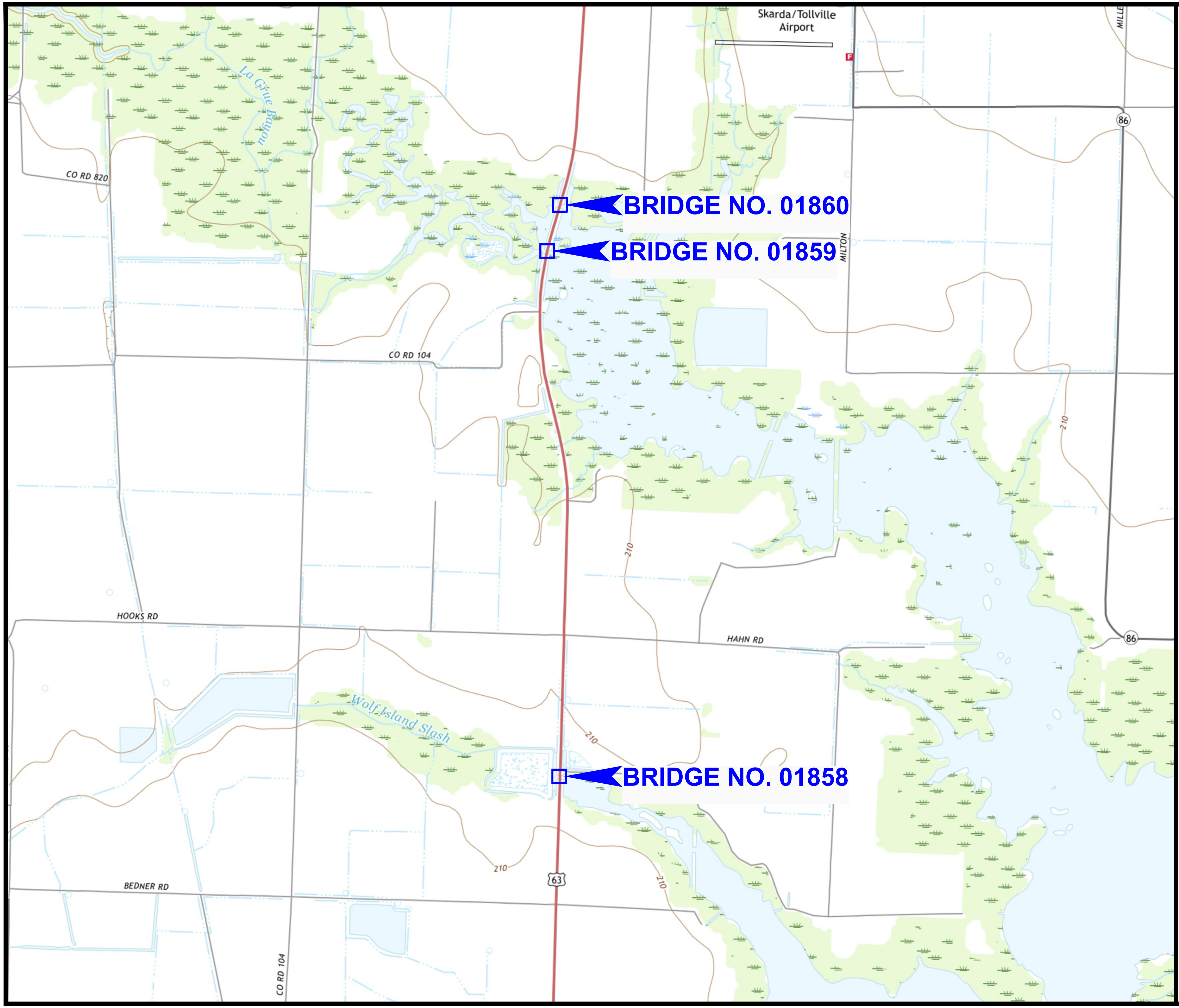
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

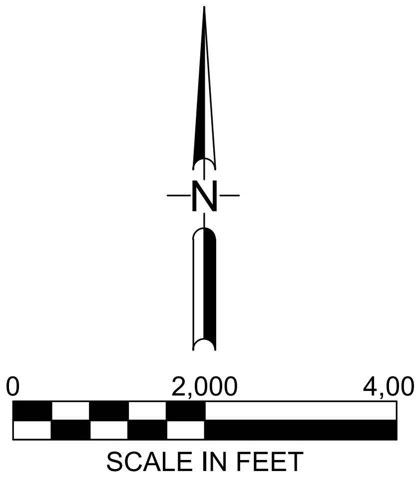
Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

Appendix B
FIGURES



NOTES

1. Plan adapted from a 7.5 minute U.S.G.S. map for Slovak, Arkansas quadrangle, last revised in 2017.



Drawn By: SDF	Ck'd By: ASM	App'vd By: DMS
Date: 12-8-20	Date: 12-17-20	Date: 12-17-20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Route 63 Section 11
Structures and Approaches
Bridge Nos. 01858, 01859 and 01860
Prairie County, Arkansas

BRIDGE LOCATIONS
AND TOPOGRAPHY

Project Number
J034561.01

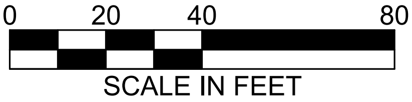
FIGURE 1

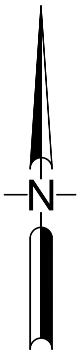
NOTES

1. Plan adapted from an October 14, 2015 aerial photograph courtesy of Google Earth.
2. Borings were located in the field with reference to site features and are shown approximate only.

LEGEND

● Boring Location





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Date: 12-8-20	Date: 12-17-20	Date: 12-17-20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Route 63 Section 11
Structures and Approaches
Bridge No. 01860
Prairie County, Arkansas

AERIAL PHOTOGRAPH OF BRIDGE 01860 AND BORING LOCATIONS

Project Number
J034561.01

FIGURE 2

Appendix C

BORING INFORMATION

Surface Elevation: 213

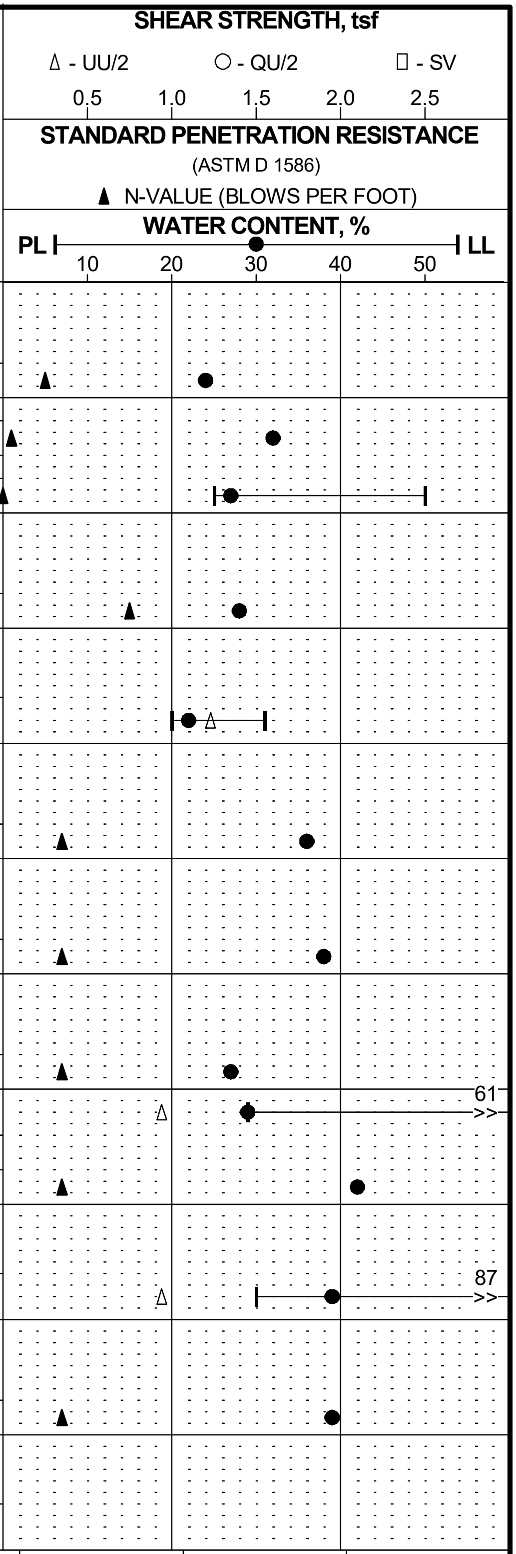
Datum msl

Completion Date: 8/13/20

Station: 225+6

Offset: -12

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Asphalt: 7 inches		
		Base Materials: 11 inches of cemet-treated soil		
		Medium stiff to very soft, gray to tan and red, FAT CLAY - (CH)	1-2-3	SS2
5	208			
		trace organics	0-0-1	SS3
		sandy	0-0-0	SS4
10	203	51% passing No. 200 sieve		
		Stiff to very stiff, tan and red, LEAN CLAY - (CL)	3-5-10	SS5
15	198			
		95% passing No. 200 sieve pH = 2.99	109	ST6
20	193	resistivity = 1,425 ohms-cm		
		trace organics		
		Medium stiff to stiff, tan and red, FAT CLAY - (CH)	2-3-4	SS7
25	188			
			3-4-3	SS8
30	183			
		pH = 8.25	3-3-4	SS9
35	178	resistivity = 741 ohms-cm		
		96% passing No. 200 sieve	94	ST10
			3-3-4	SS11
40	173			
		96% passing No. 200 sieve	84	ST12
45	168			
			2-3-4	SS13
50	163			
				*ST 14



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 10 FEET
JCG DRILLER SWF LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS: *No recovery

Drawn by: SWF	Checked by: DMS	App'vd. by: ASE
Date: 8/14/20	Date: 12/17/20	Date: 12/17/20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ARDOT 061615
Highway 63 Bridge No. 01860
Over La Grue Bayou
Prairie County, Arkansas

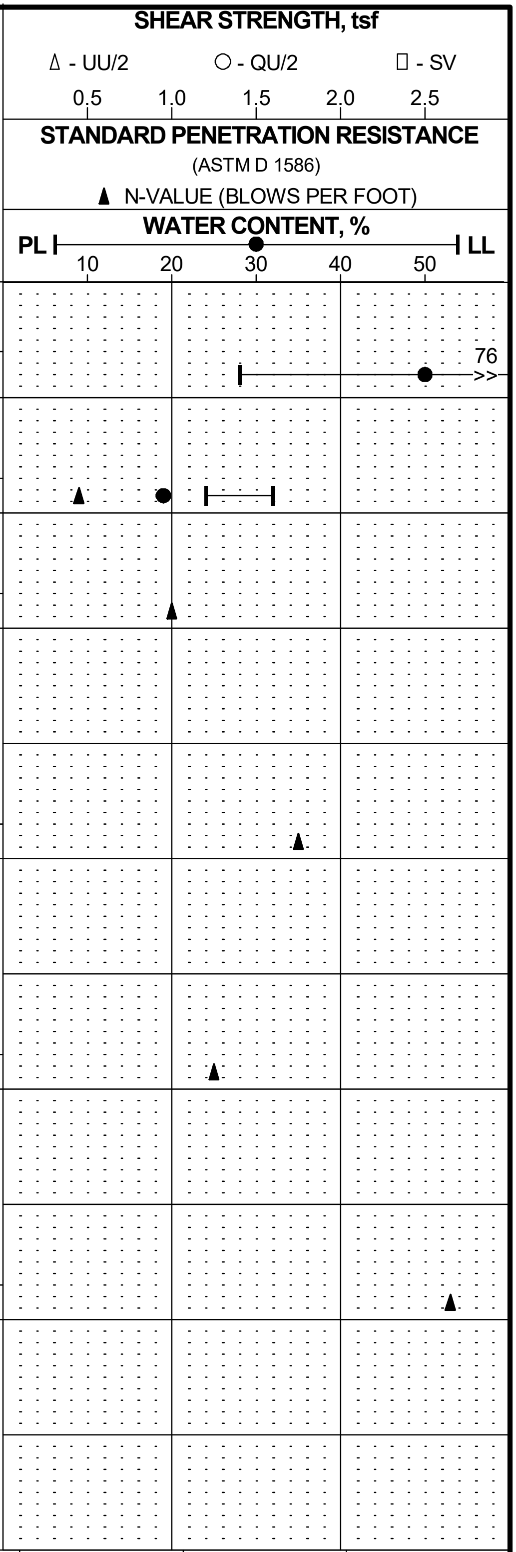
LOG OF BORING: L-3

Project No. J034561.01

Surface Elevation: 213
Datum msl

Completion Date: 8/13/20
Station: 225+6
Offset: -12

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Medium stiff to stiff, tan and red, FAT CLAY - (CH) *(continued)*	▨		
		98% passing No. 200 sieve pH = 8.12 resistivity = 1,254 ohms-cm		72	ST15
60	153				
		Stiff to very stiff, gray, sandy SILT - (ML) 62% passing No. 200 sieve		6-5-4	SS16
65	148				
		55% passing No. 200 sieve pH = 7.75 resistivity = 1,197 ohms-cm		7-9-11	SS17
70	143				
75	138				
		Dense, gray, SILTY SAND - SM		16-16-19	SS18
80	133				
85	128				
		Medium dense to very dense, gray SAND, trace silt and gravel - (SP-SM) trace organics 9% passing No. 200 sieve		11-9-16	SS19
90	123				
95	118				
				22-25-28	SS20
100	113	Boring terminated at 100 feet.			
105	108				



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 10 FEET
JCG DRILLER SWF LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS: *No recovery

Drawn by: SWF	Checked by: DMS	App'vd. by: ASE
Date: 8/14/20	Date: 12/17/20	Date: 12/17/20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ARDOT 061615
Highway 63 Bridge No. 01860
Over La Grue Bayou
Prairie County, Arkansas

CONTINUATION OF
LOG OF BORING: L-3

Project No. J034561.01

Surface Elevation: 213

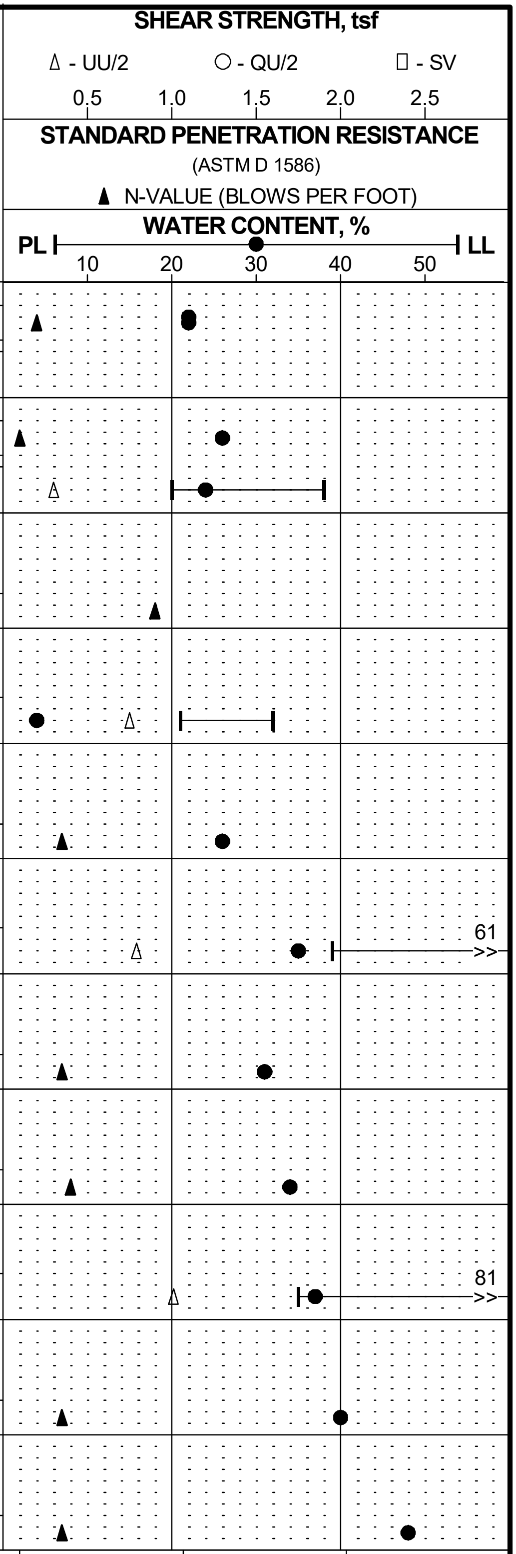
Datum msl

Completion Date: 8/12/20

Station: 226+83

Offset: -8

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Asphalt: 4 inches	■		
		Base Materials: 8 inches of cement-treated soil	▨	1-2-2	SS1
		Soft to very stiff, gray to tan and red, LEAN CLAY - (CL)	🟩		ST2
5	208				
		little organics		0-1-1	SS3
		97% passing No. 200 sieve		100	ST4
10	203				
		pH = 3.85 resistivity = 1,881 ohms-cm		3-7-11	SS5
15	198				
		94% passing No. 200 sieve	⊽	117	ST6
20	193				
		Medium stiff, tan and red, FAT CLAY - CH	🟩	1-3-4	SS7
25	188				
		Stiff, tan and red, sandy, ELASTIC SILT - (MH) 56% passing No. 200 sieve pH = 8.01 resistivity = 855 ohms-cm	🟩	87	ST8
30	183				
		Very stiff to soft, tan and red, FAT CLAY - (CH)	🟩	2-3-4	SS9
35	178				
				2-4-4	SS10
40	173				
		with sand 72% passing No. 200 sieve pH = 8.04 resistivity = 43 ohms-cm		85	ST11
45	168				
				2-3-4	SS12
50	163				
				1-3-4	SS13



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 19.5 FEET ⊽

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
JCG DRILLER SWF LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: SWF	Checked by: DMS	App'vd. by: ASE
Date: 8/14/20	Date: 12/17/20	Date: 12/17/20

GEOTECHNOLOGY
A Universal Engineering Sciences Company

ARDOT 061615
Highway 63 Bridge No. 01860
Over La Grue Bayou
Prairie County, Arkansas

LOG OF BORING: L-4

Project No. J034561.01

LOG OF BORING 2020 JDM - ELEVATIONS J034561.01.GPJ GTINC 0638301.GPJ 3/23/23

Surface Elevation: 213

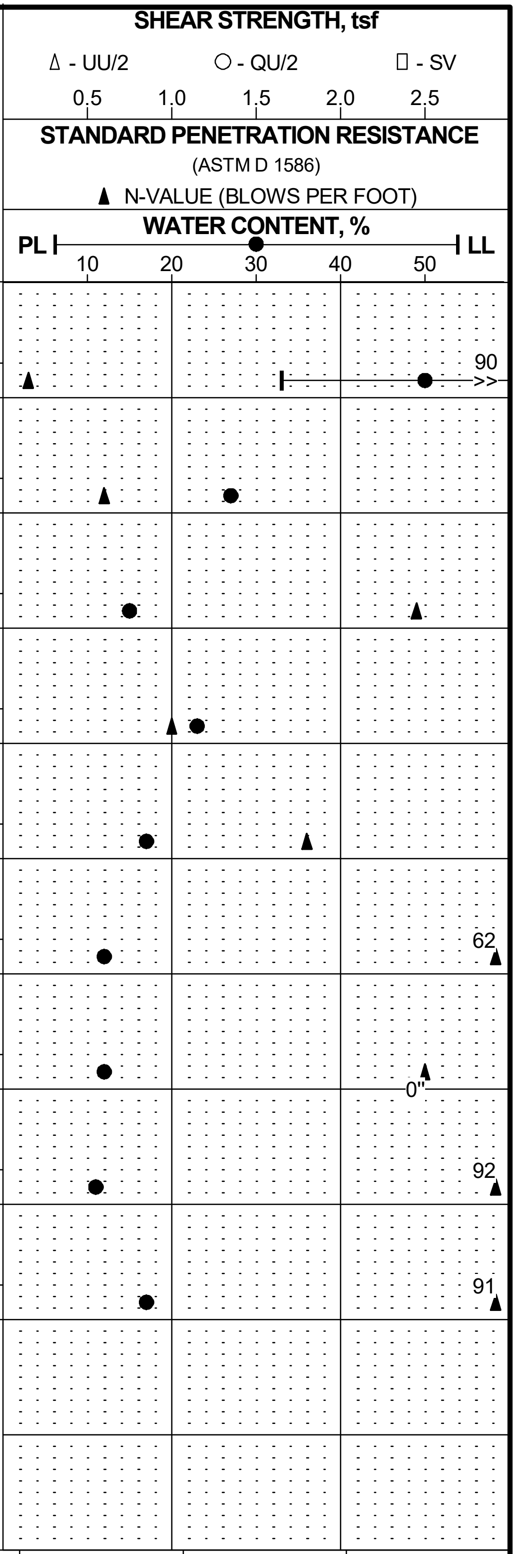
Datum msl

Completion Date: 8/12/20

Station: 226+83

Offset: -8

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Very stiff to soft, tan and red, FAT CLAY - (CH) *(continued)*			
		trace organics 97% passing No. 200 sieve pH = 8.21 resistivity = 598.5 ohms-cm		0-1-2	SS14
60	153				
		Medium dense to dense, gray, SILTY SAND - (SM) 39% passing No. 200 sieve		5-5-7	SS15
65	148				
				15-21-28	SS16
70	143				
		Medium dense, gray, CLAYEY SAND - (SC) 21% passing No. 200 sieve		14-12-8	SS17
75	138				
		Dense to very dense, gray SAND - SP		13-18-18	SS18
80	133				
		trace gravel		23-27-35	SS19
85	128				
		little gravel		23-50/0"	SS20
90	123				
		little gravel		29-42-50	SS21
95	118				
		little gravel		29-41-50	SS22
100	113	Boring terminated at 100 feet.			
105	108				



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 19.5 FEET ∇

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
JCG DRILLER SWF LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: SWF	Checked by: DMS	App'vd. by: ASE
Date: 8/14/20	Date: 12/17/20	Date: 12/17/20

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ARDOT 061615
Highway 63 Bridge No. 01860
Over La Grue Bayou
Prairie County, Arkansas

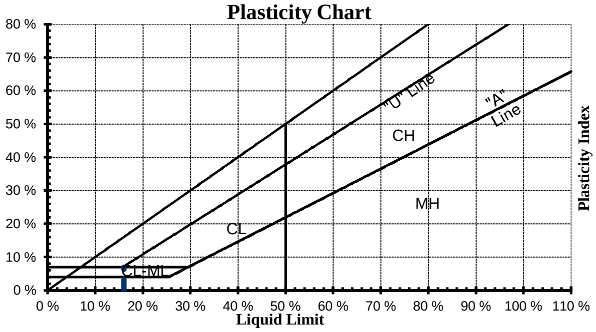
CONTINUATION OF
LOG OF BORING: L-4

Project No. J034561.01

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)


Plasticity Chart

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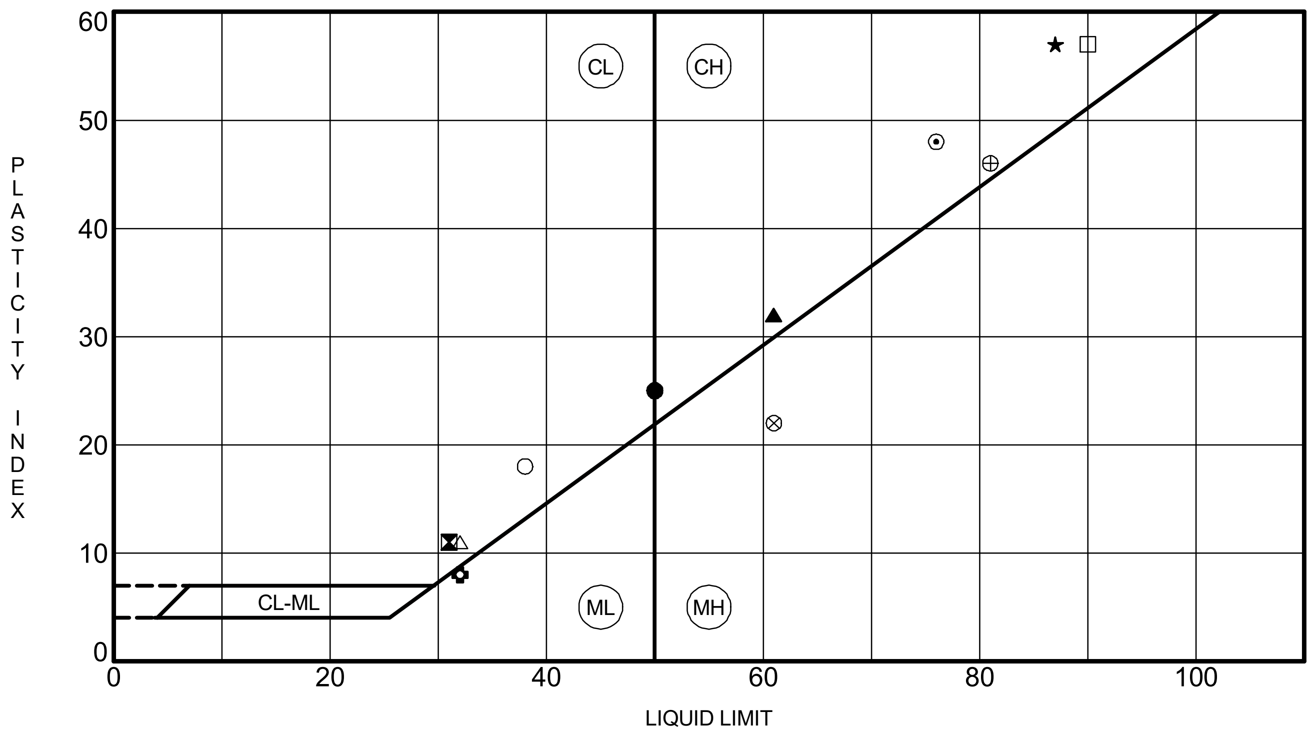
Relative composition and Unified Soil Classification System (USCS) designations are based on visual descriptions and are approximate only. If laboratory tests were performed to classify the soil, the USCS designation is shown in parenthesis.

Appendix D
LABORATORY TEST DATA



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**L-3**	**8.5**	**50**	**25**	**25**	**51**	**SANDY FAT CLAY(CH), A-7-6 (9)**
⧗	**L-3**	**18.0**	**31**	**20**	**11**	**95**	**LEAN CLAY(CL), A-6 (18)**
▲	**L-3**	**35.0**	**61**	**29**	**32**	**96**	**FAT CLAY(CH), A-7-6 (36)**
★	**L-3**	**43.0**	**87**	**30**	**57**	**99**	**FAT CLAY(CH), A-7-5 (65)**
⊙	**L-3**	**58.0**	**76**	**28**	**48**	**98**	**FAT CLAY(CH), A-7-6 (55)**
✿	**L-3**	**63.5**	**32**	**24**	**8**	**62**	**SANDY SILT(ML), A-4 (3)**
○	**L-4**	**8.0**	**38**	**20**	**18**	**97**	**LEAN CLAY(CL), A-6 (18)**
△	**L-4**	**18.0**	**32**	**21**	**11**	**94**	**LEAN CLAY(CL), A-6 (10)**
⊗	**L-4**	**28.0**	**61**	**39**	**22**	**56**	**SANDY ELASTIC SILT(MH), A-7-5 (11)**
⊕	**L-4**	**43.0**	**81**	**35**	**46**	**72**	**FAT CLAY with SAND(CH), A-7-5 (17)**
□	**L-4**	**58.5**	**90**	**33**	**57**	**97**	**FAT CLAY(CH), A-7-5 (66)**

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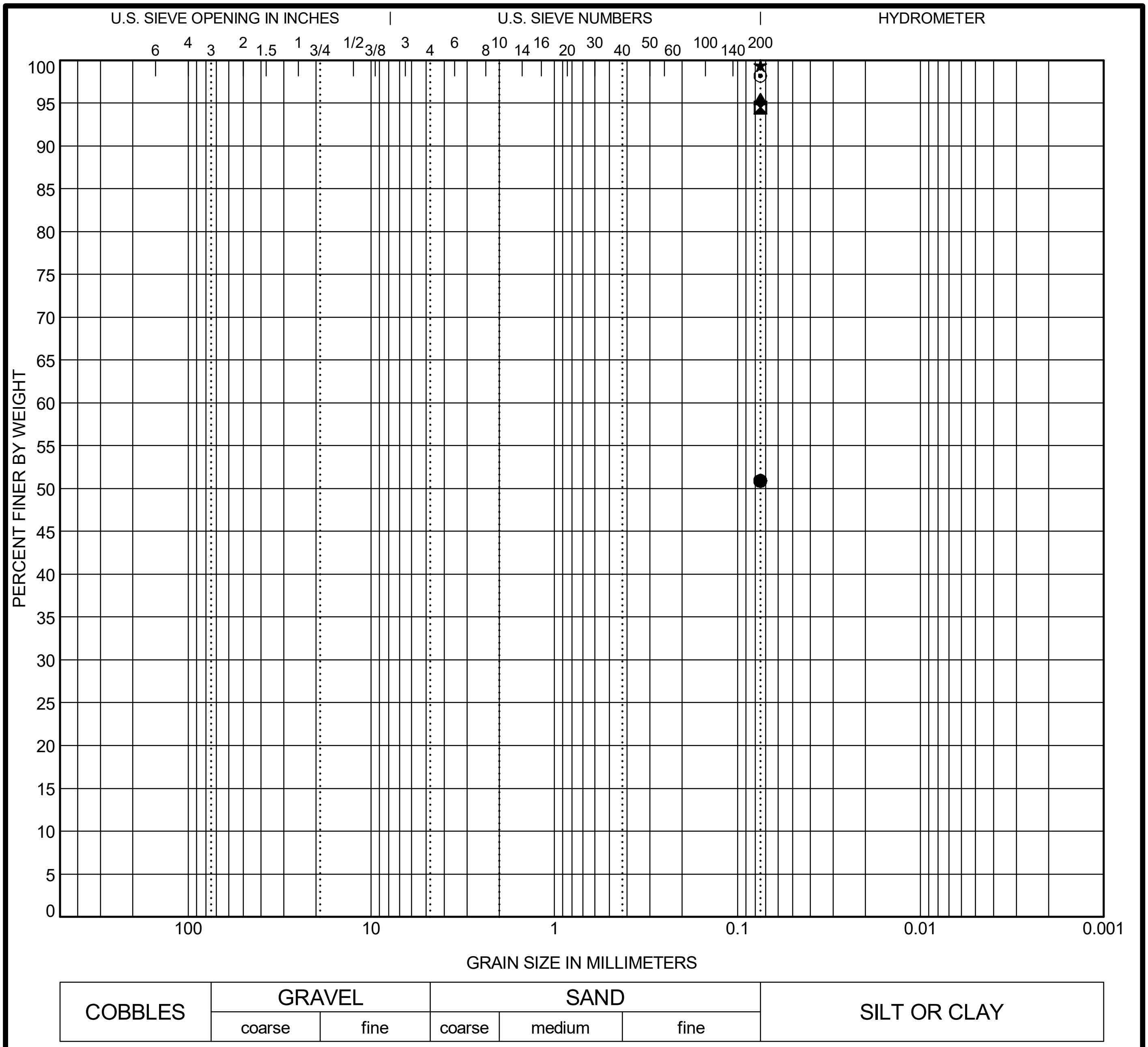
ATTERBERG LIMITS RESULTS

ARDOT 061615 Highway 63
Bridge No. 01860 Over La Grue Bayou
Prairie County, Arkansas
J034561.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**L-3**	**8.5**	**SANDY FAT CLAY(CH), A-7-6 (9)**	**50**	**25**	**25**		
⊠	**L-3**	**18.0**	**LEAN CLAY(CL), A-6 (18)**	**31**	**20**	**11**		
▲	**L-3**	**35.0**	**FAT CLAY(CH), A-7-6 (36)**	**61**	**29**	**32**		
★	**L-3**	**43.0**	**FAT CLAY(CH), A-7-5 (65)**	**87**	**30**	**57**		
⊙	**L-3**	**58.0**	**FAT CLAY(CH), A-7-6 (55)**	**76**	**28**	**48**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**L-3**	**8.5**	**0.075**				**0.0**	**0.0**	**50.9**	
⊠	**L-3**	**18.0**	**0.075**				**0.0**	**0.0**	**94.5**	
▲	**L-3**	**35.0**	**0.075**				**0.0**	**0.0**	**95.5**	
★	**L-3**	**43.0**	**0.075**				**0.0**	**0.0**	**99.4**	
⊙	**L-3**	**58.0**	**0.075**				**0.0**	**0.0**	**98.2**	

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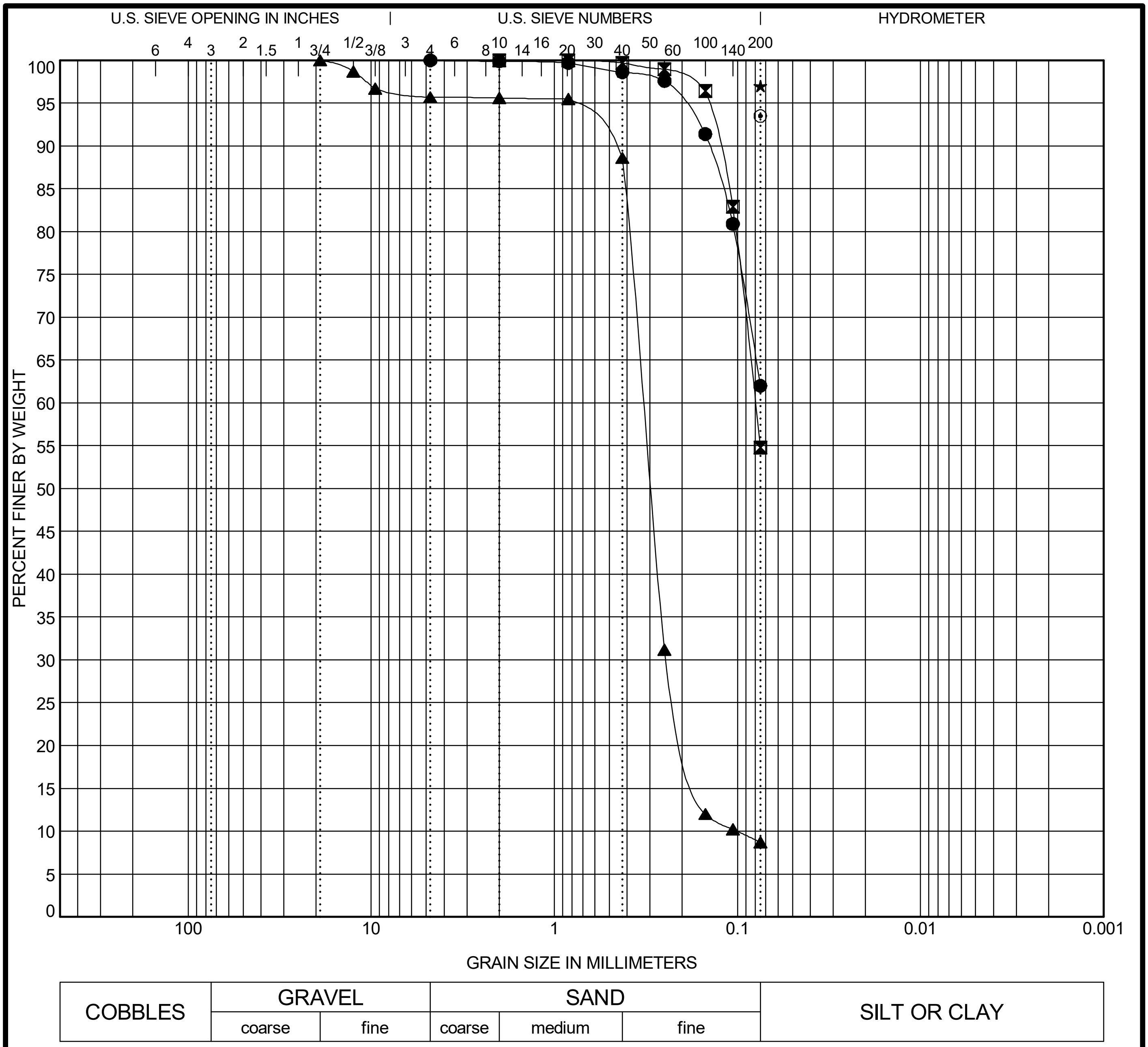
GRAIN SIZE DISTRIBUTION

ARDOT 061615 Highway 63
Bridge No. 01860 Over La Grue Bayou
Prairie County, Arkansas
J034561.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	L-3	63.5	**SANDY SILT(ML), A-4 (3)**	32	24	8		
⊠	L-3	68.5	**SANDY SILT(ML), A-4**					
▲	L-3	88.5	**POORLY GRADED SAND with SILT(SP-SM), A-3**				1.78	3.22
★	L-4	8.0	**LEAN CLAY(CL), A-6 (18)**	38	20	18		
⊙	L-4	18.0	**LEAN CLAY(CL), A-6 (10)**	32	21	11		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	L-3	63.5	4.75				0.0	38.0	62.0	
⊠	L-3	68.5	2	0.08			0.0	45.2	54.8	
▲	L-3	88.5	19	0.326	0.242	0.101	4.3	87.0	8.7	
★	L-4	8.0	0.075				0.0	0.0	97.0	
⊙	L-4	18.0	0.075				0.0	0.0	93.5	

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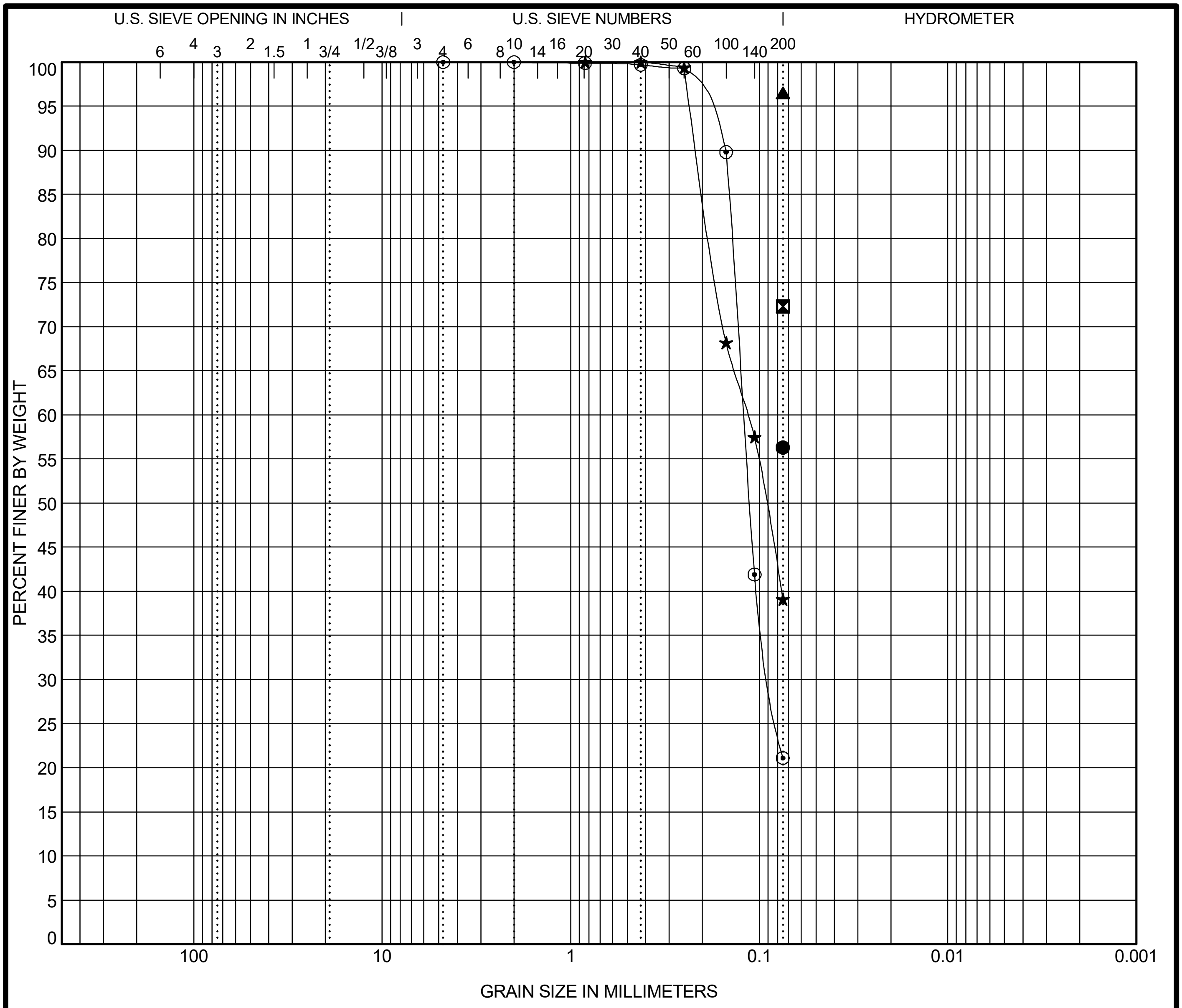
GRAIN SIZE DISTRIBUTION

ARDOT 061615 Highway 63
Bridge No. 01860 Over La Grue Bayou
Prairie County, Arkansas
J034561.01



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	L-4	28.0	**SANDY ELASTIC SILT(MH), A-7-5 (11)**	61	39	22		
⊠	L-4	43.0	**FAT CLAY with SAND(CH), A-7-5 (17)**	81	35	46		
▲	L-4	58.5	**FAT CLAY(CH), A-7-5 (66)**	90	33	57		
★	L-4	63.5	**SILTY SAND(SM), A-4**					
⊙	L-4	73.5	**CLAYEY SAND(SC), A-2-7**					

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	L-4	28.0	0.075				0.0	0.0	56.3	
⊠	L-4	43.0	0.075				0.0	0.0	72.3	
▲	L-4	58.5	0.075				0.0	0.0	96.5	
★	L-4	63.5	0.84	0.115			0.0	60.9	39.1	
⊙	L-4	73.5	4.75	0.121	0.087		0.0	78.9	21.1	

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GRAIN SIZE DISTRIBUTION

ARDOT 061615 Highway 63
Bridge No. 01860 Over La Grue Bayou
Prairie County, Arkansas
J034561.01

GEOTECHNOLOGY
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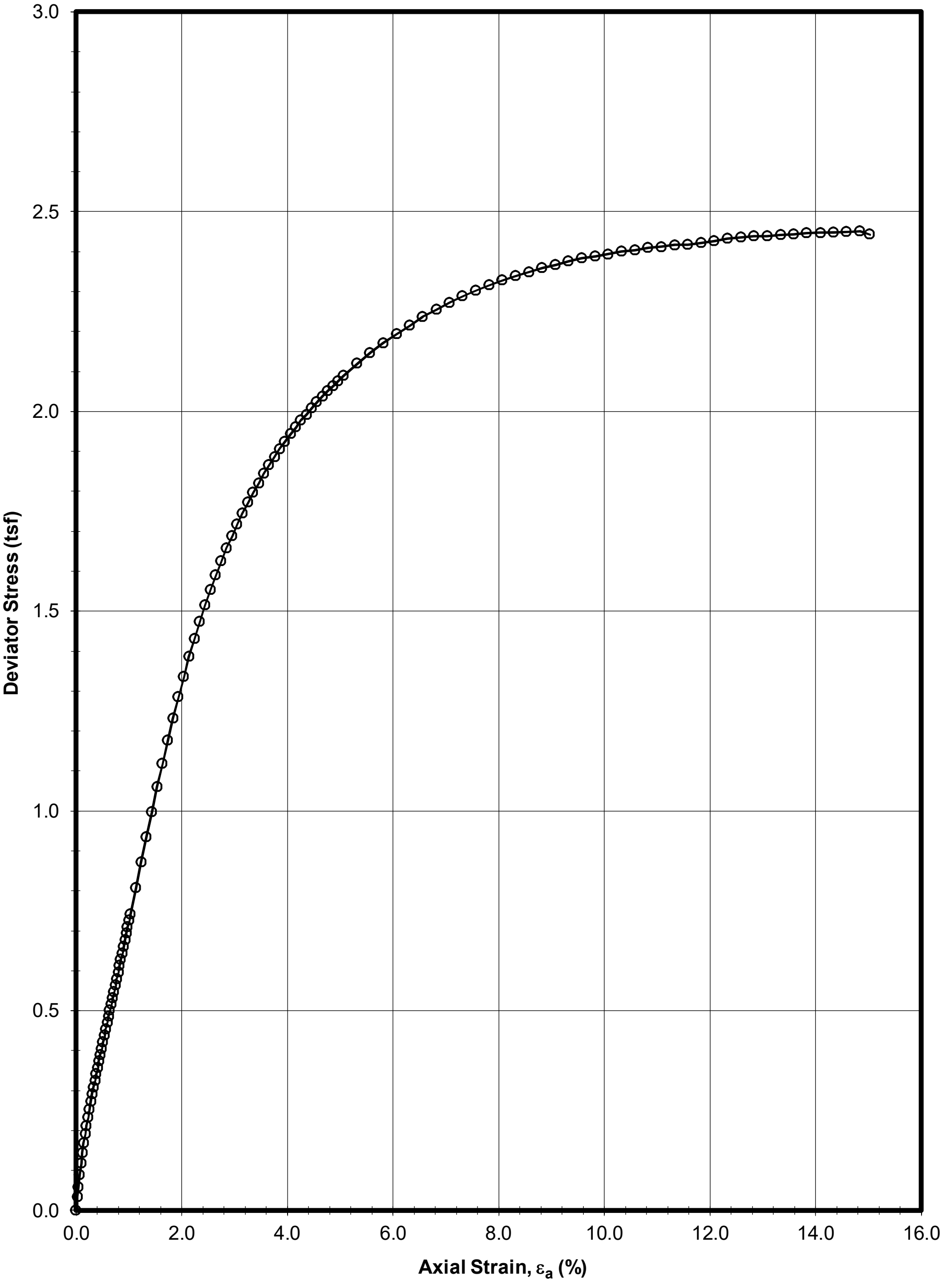
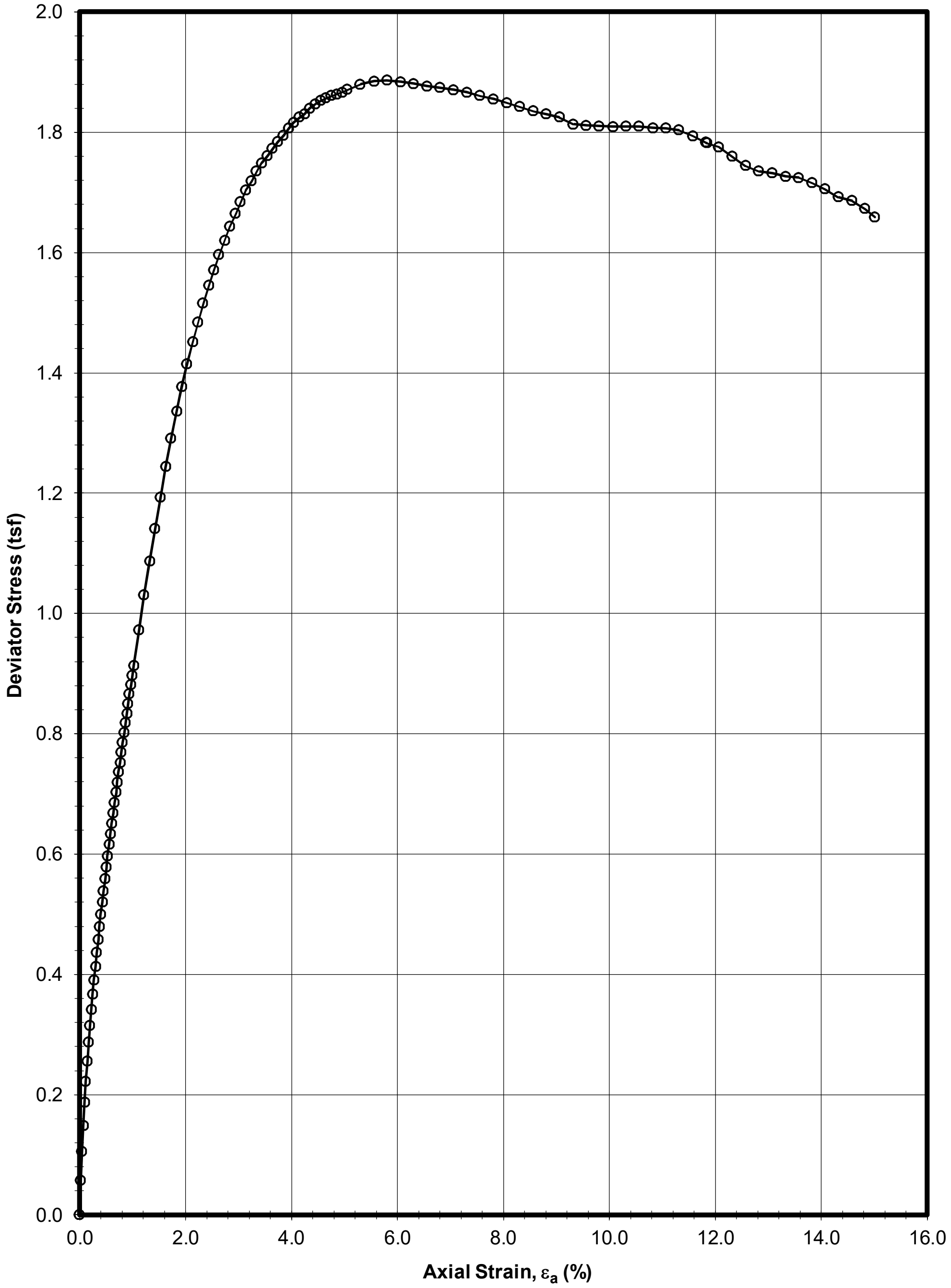


UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST
ASTM D 2850
Project No.: J034561.01
Boring: L-3
Sample: ST6 - Depth: 18 ft.



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

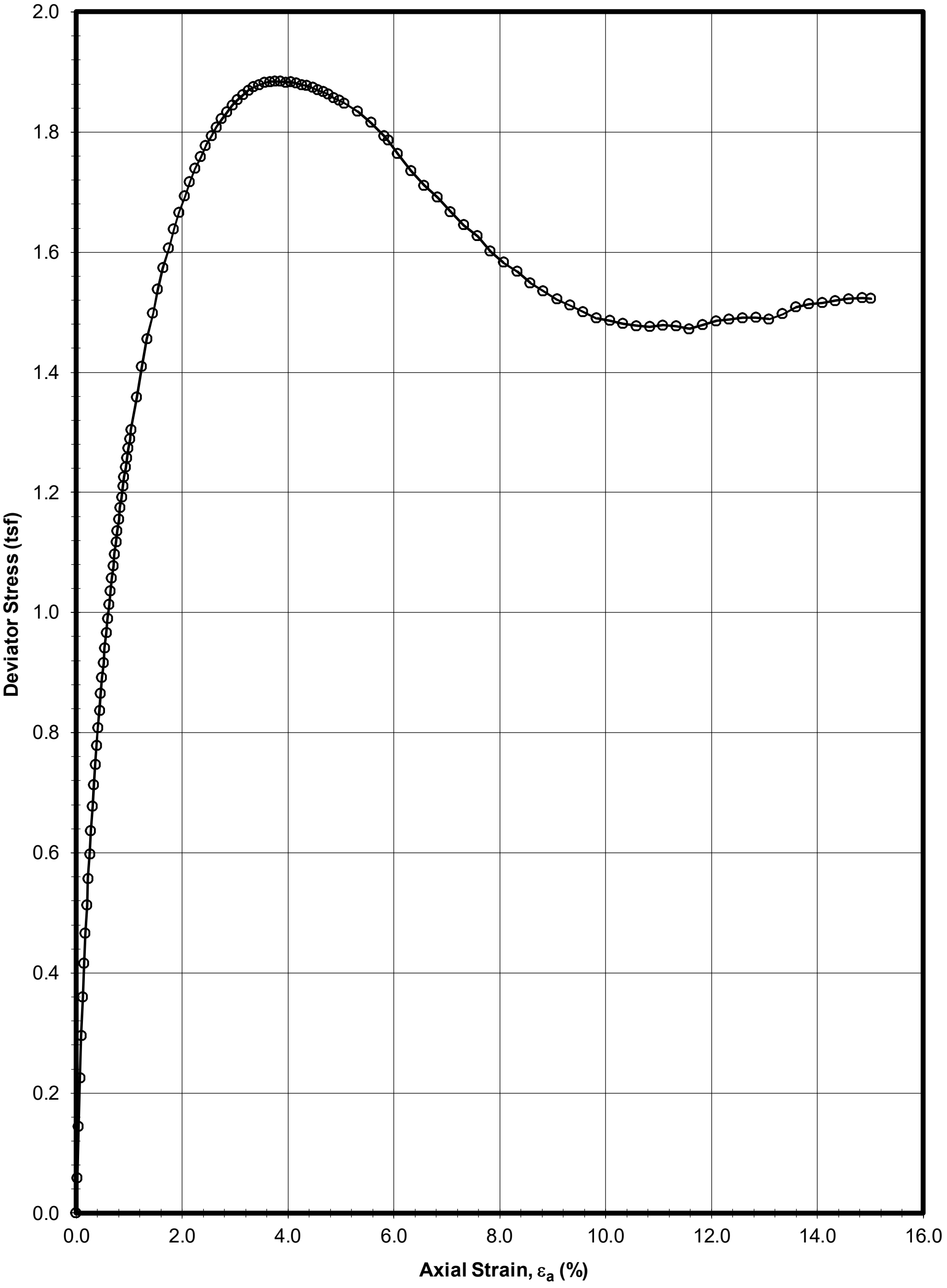
ASTM D 2850

Project No.: J034561.01

Boring: L-3

Sample: ST10 - Depth: 35 ft.

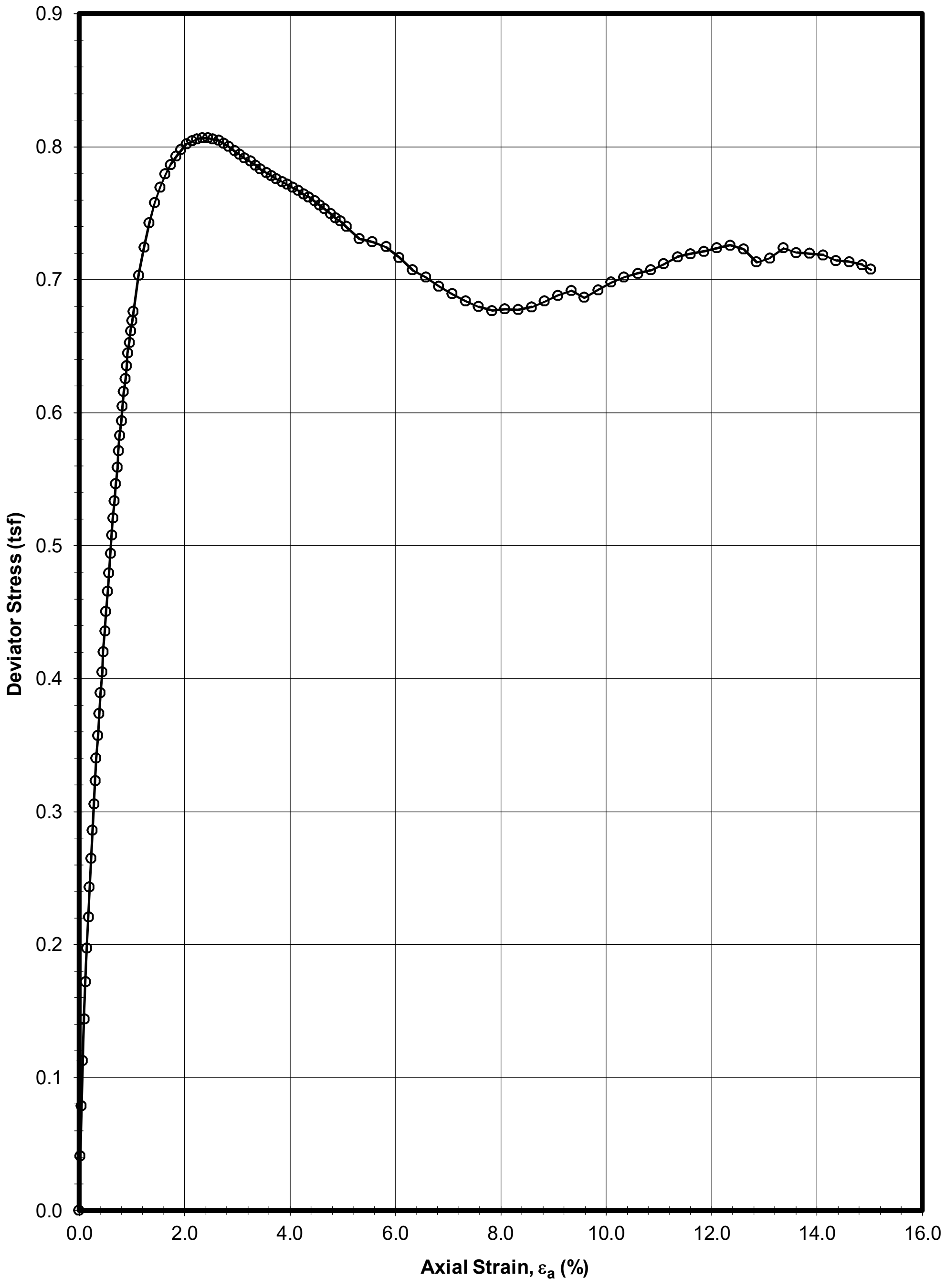
GEOTECHNOLOGY
A Universal Engineering Sciences Company



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J034561.01
Boring: L-3
Sample: ST12 - Depth: 43 ft.

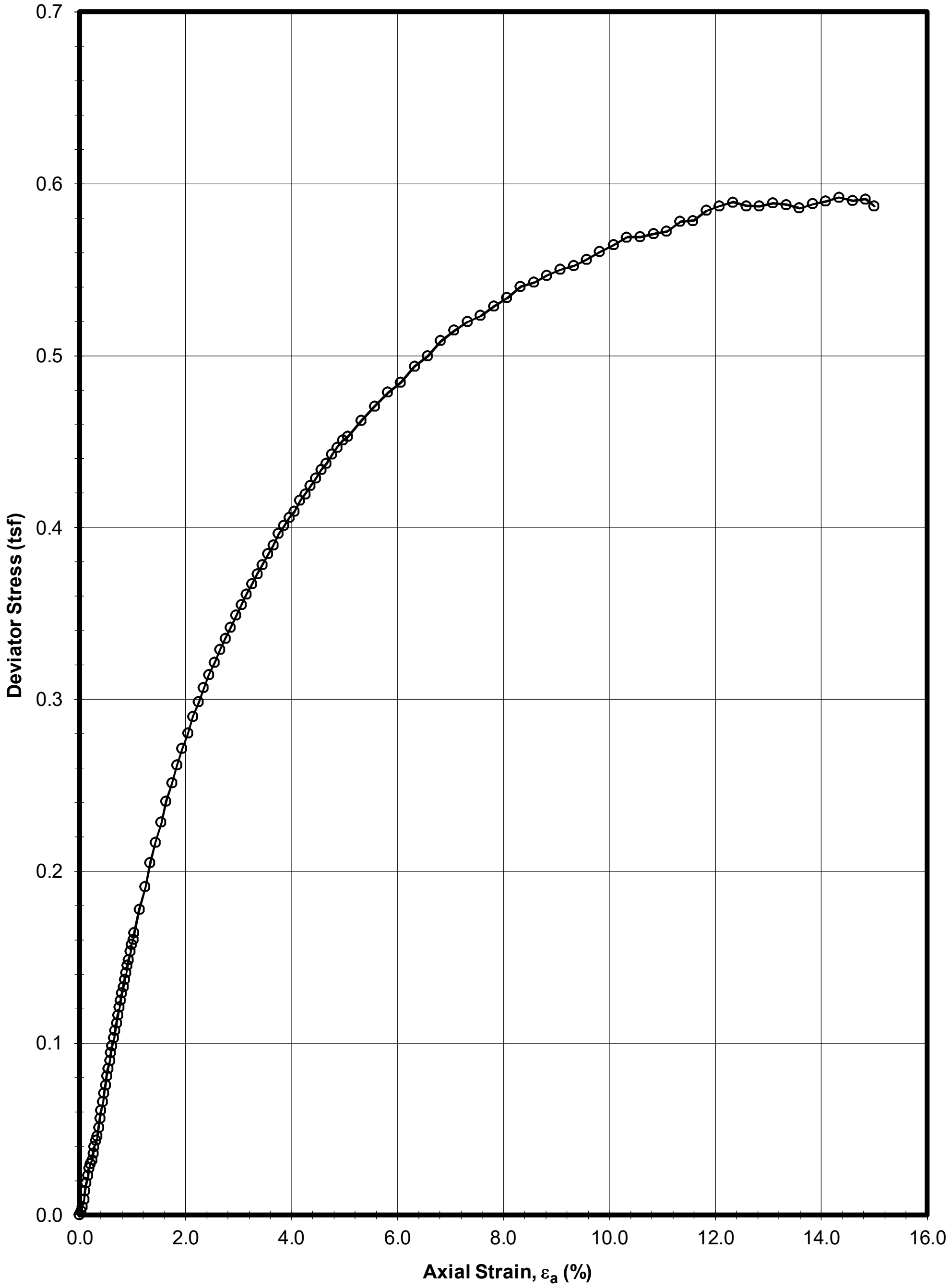
GEOTECHNOLOGY
A Universal Engineering Sciences Company



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J034561.01
Boring: L-3
Sample: ST14 - Depth: 58 ft.

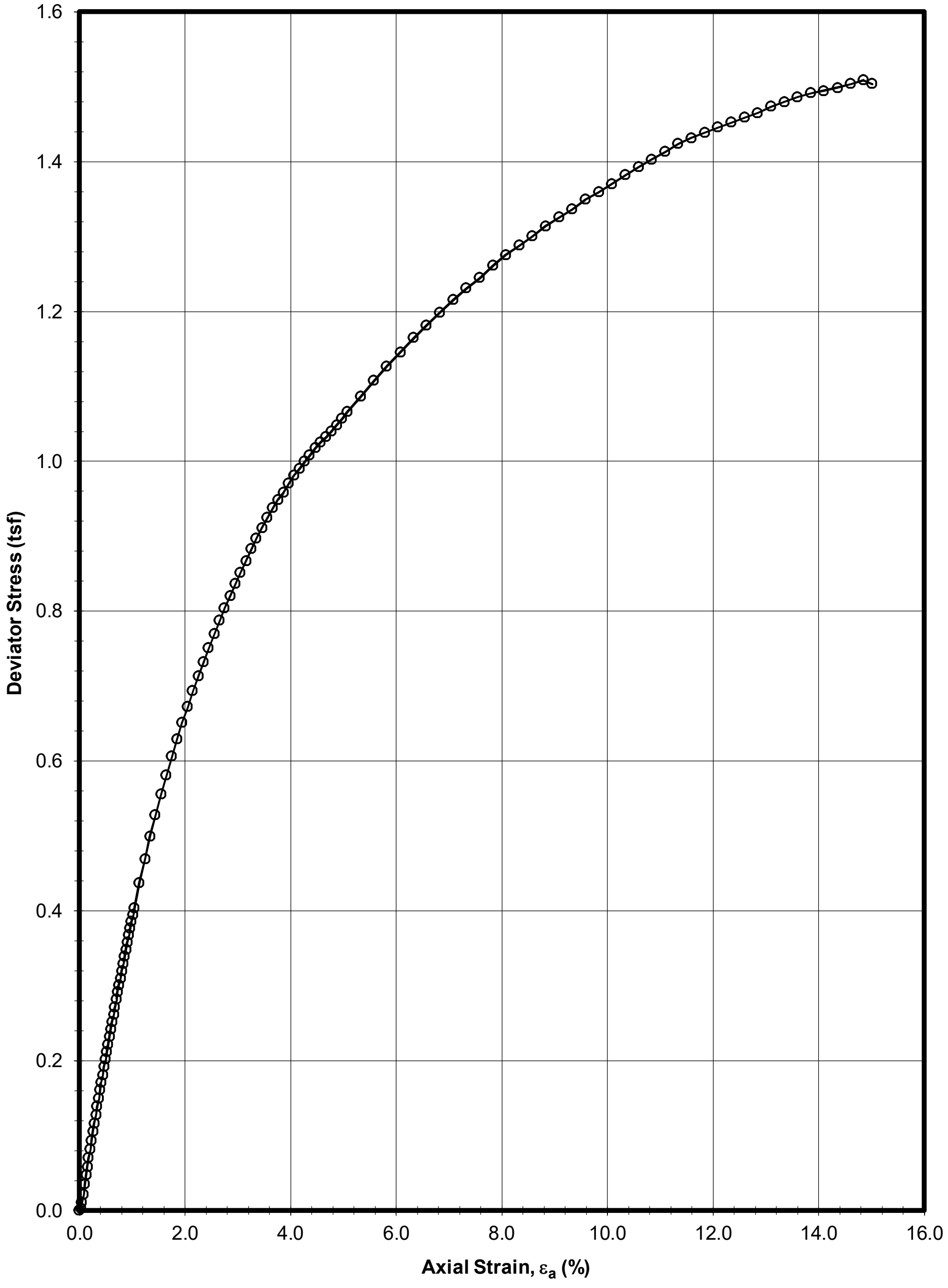
GEOTECHNOLOGY
A Universal Engineering Sciences Company



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

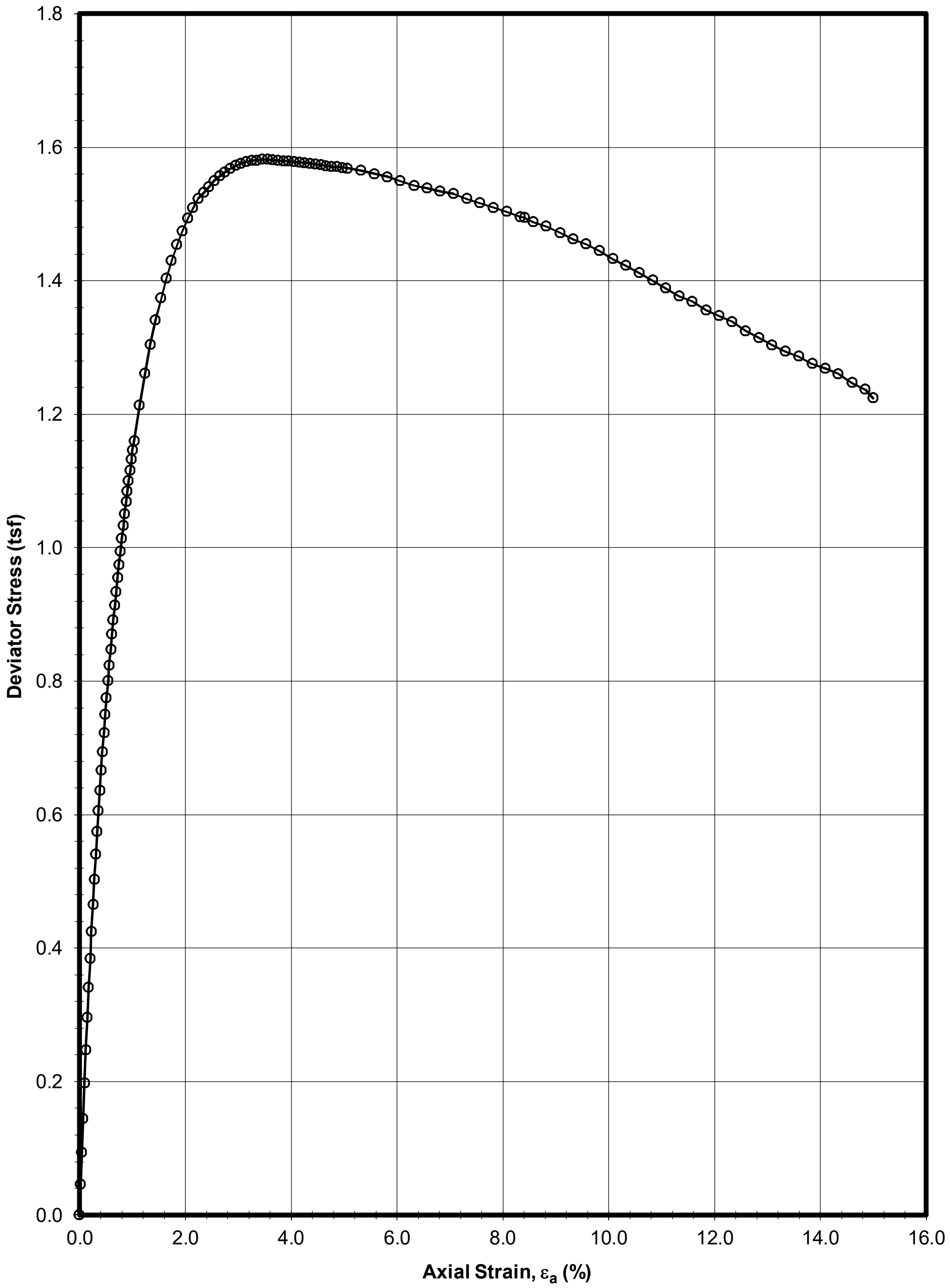
ASTM D 2850
Project No.: J034561.01
Boring: L-4
Sample: ST4 - Depth: 8 ft.

GEOTECHNOLOGY
A Universal Engineering Sciences Company



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST
ASTM D 2850
Project No.: J034561.01
Boring: L-4
Sample: ST6 - Depth: 18 ft.

GEOTECHNOLOGY
A Universal Engineering Sciences Company

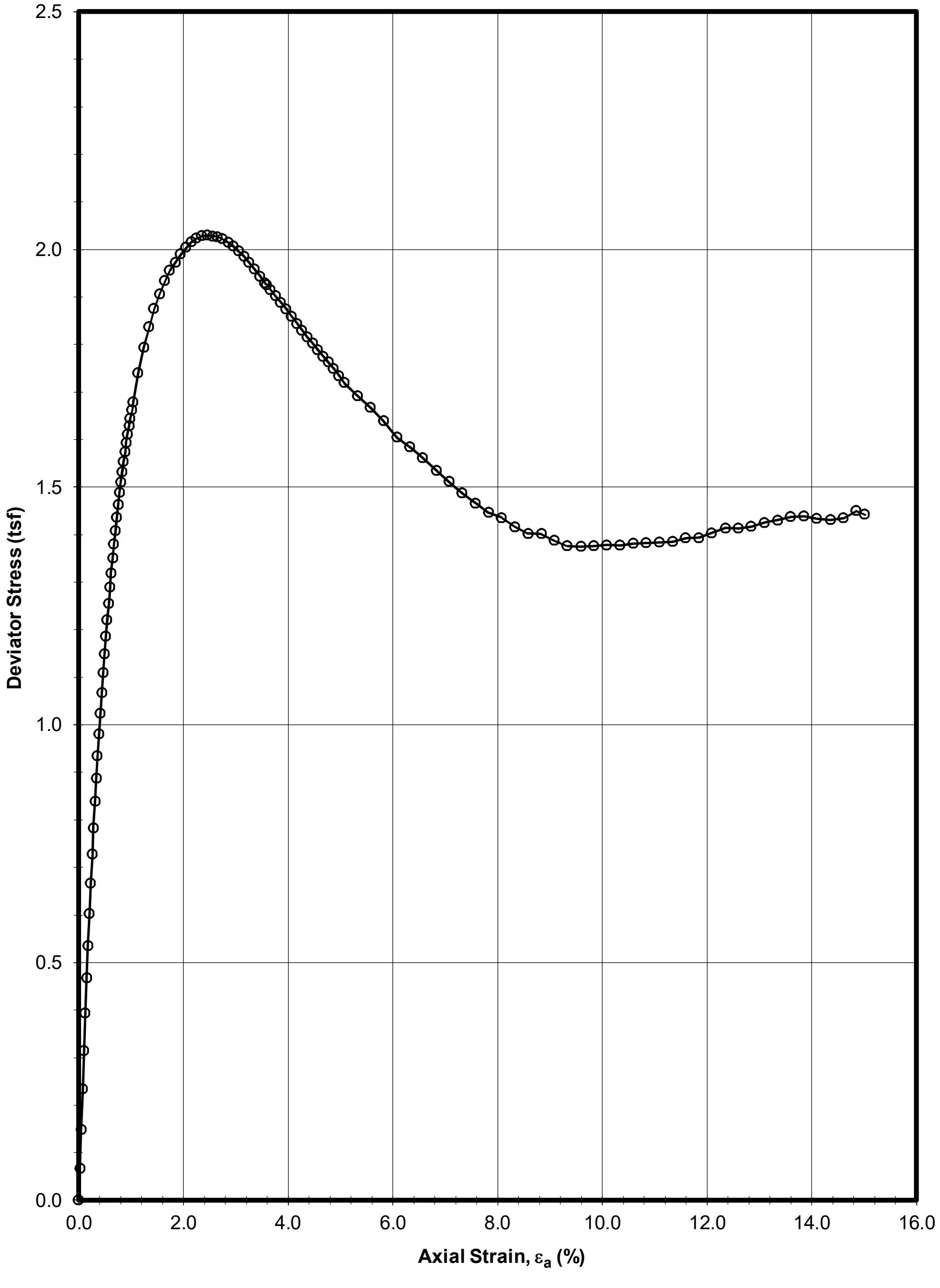


UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J034561.01
Boring: L-4
Sample: ST-8 - Depth: 28 ft.

GEOTECHNOLOGY

A Universal Engineering Sciences Company



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034561.01

Boring: L-4

Sample: ST-11 - Depth: 43 ft.

TEST REPORT

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

Project No.:	**J034561.01**	**December 16, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01860**	
Boring Number:	**L-3**	
Sample ID:	**SS-9**	
Depth (ft):	**33.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	17,500	0.57	9,975.00	13.8
#2	3,000	0.57	1,710.00	20.8
#3	1,600	0.57	912.00	27.0
#4	1,300	0.57	741.00	32.3
#5	1,400	0.57	798.00	38.9

Minimum Soil Resistivity **741.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 14, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01860**	
Boring Number:	**L-3**	
Sample ID:	**SS-16**	
Depth (ft):	**68.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	5,700	0.57	3,249.00	11.0
#2	3,100	0.57	1,767.00	18.3
#3	2,100	0.57	1,197.00	25.5
#4	2,200	0.57	1,254.00	34.3

Minimum Soil Resistivity **1,197.00**

TEST REPORT

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

Project No.: J034561.01 **December 16, 2020**
Project Name: ARDOT 061615 **Page 1 of 1**
Bridge No.: 01860
Boring Number: L-3
Sample ID: ST-6
Depth (ft): 18.0

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	3,100	0.57	1,767.00	13.4
#2	2,500	0.57	1,425.00	20.7
#3	2,700	0.57	1,539.00	25.1

Minimum Soil Resistivity **1,425.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 14, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01860**	
Boring Number:	**L-3**	
Sample ID:	**ST-15**	
Depth (ft):	**58.0**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	2,200	0.57	1,254.00	20.0
#2	2,850	0.57	1,624.50	24.4

Minimum Soil Resistivity **1,254.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 16, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01860**	
Boring Number:	**L-4**	
Sample ID:	**SS-5**	
Depth (ft):	**13.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	30,000	0.57	17,100.00	14.2
#2	10,000	0.57	5,700.00	21.7
#3	3,400	0.57	1,938.00	27.4
#4	3,300	0.57	1,881.00	34.3
#5	3,700	0.57	2,109.00	40.8

Minimum Soil Resistivity **1,881.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 16, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01860**	
Boring Number:	**L-4**	
Sample ID:	**SS-14**	
Depth (ft):	**58.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	37,000	0.57	21,090.00	15.7
#2	3,550	0.57	2,023.50	23.1
#3	1,600	0.57	912.00	30.4
#4	1,050	0.57	598.50	35.8
#5	1,100	0.57	627.00	43.7

Minimum Soil Resistivity **598.50**

TEST REPORT

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

Project No.:	**J034561.01**	**December 14, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01860**	
Boring Number:	**L-4**	
Sample ID:	**ST-8**	
Depth (ft):	**28.0**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	14,000	0.57	7,980.00	18.3
#2	1,500	0.57	855.00	27.1
#3	1,700	0.57	969.00	32.0

Minimum Soil Resistivity **855.00**

TEST REPORT

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

Project No.:	**J034561.01**	**December 14, 2020**
Project Name:	**ARDOT 061615**	**Page 1 of 1**
Bridge No.:	**01860**	
Boring Number:	**L-4**	
Sample ID:	**ST-11**	
Depth (ft):	**43.0**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	6,600	0.57	3,762.00	19.6
#2	3,000	0.57	1,710.00	26.9
#3	1,650	0.57	940.50	33.5
#4	1,300	0.57	741.00	39.0
#5	1,350	0.57	769.50	46.0

Minimum Soil Resistivity **741.00**

Appendix E
SELECTED GLOBAL STABILITY ANALYSES

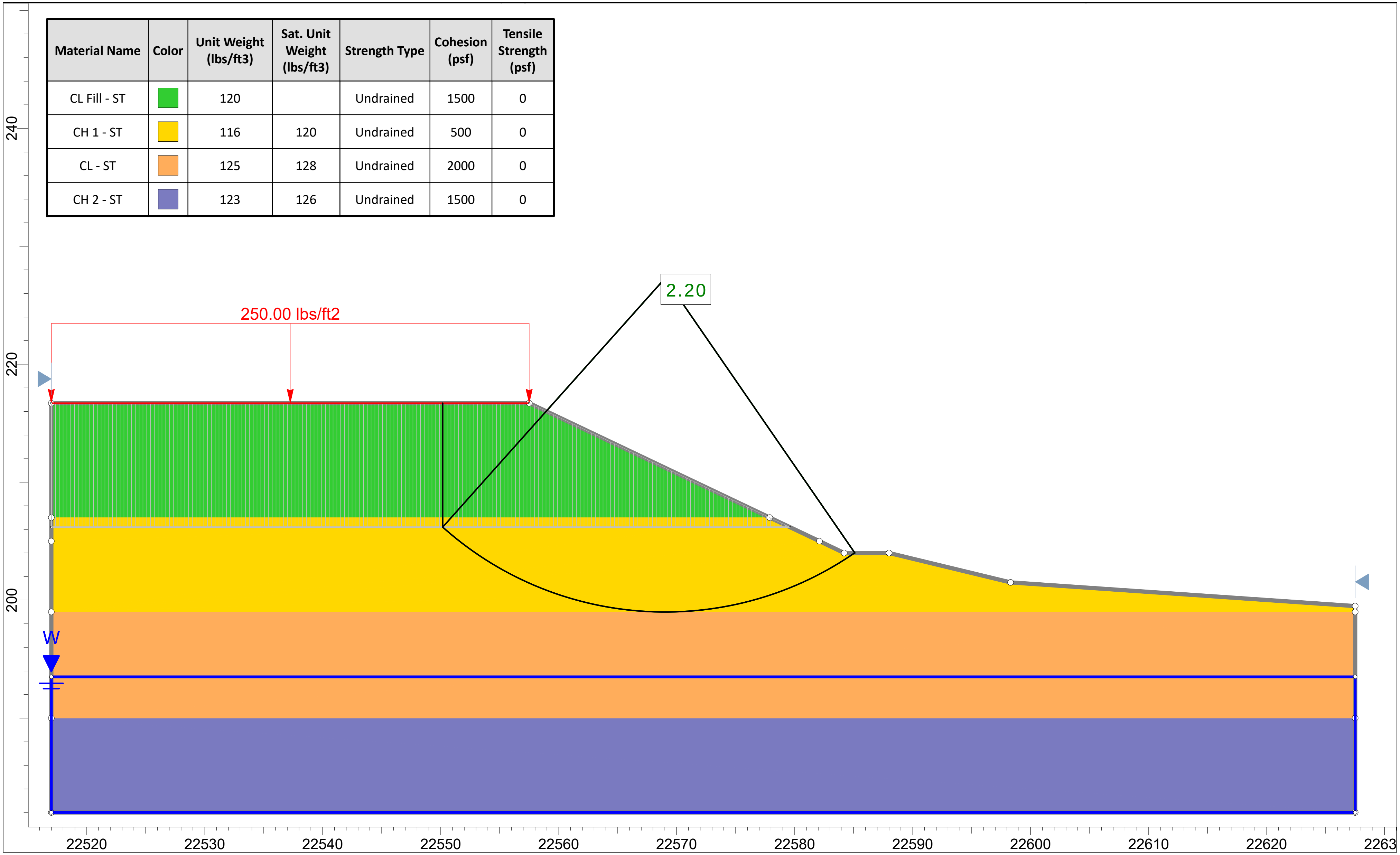
GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: South Abutment Spill Slope
Analysis Description: Short Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/17/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CH 1 - ST	🟨	116	120	Undrained	500	0
CL - ST	🟧	125	128	Undrained	2000	0
CH 2 - ST	🟪	123	126	Undrained	1500	0



GEOTECHNOLOGY

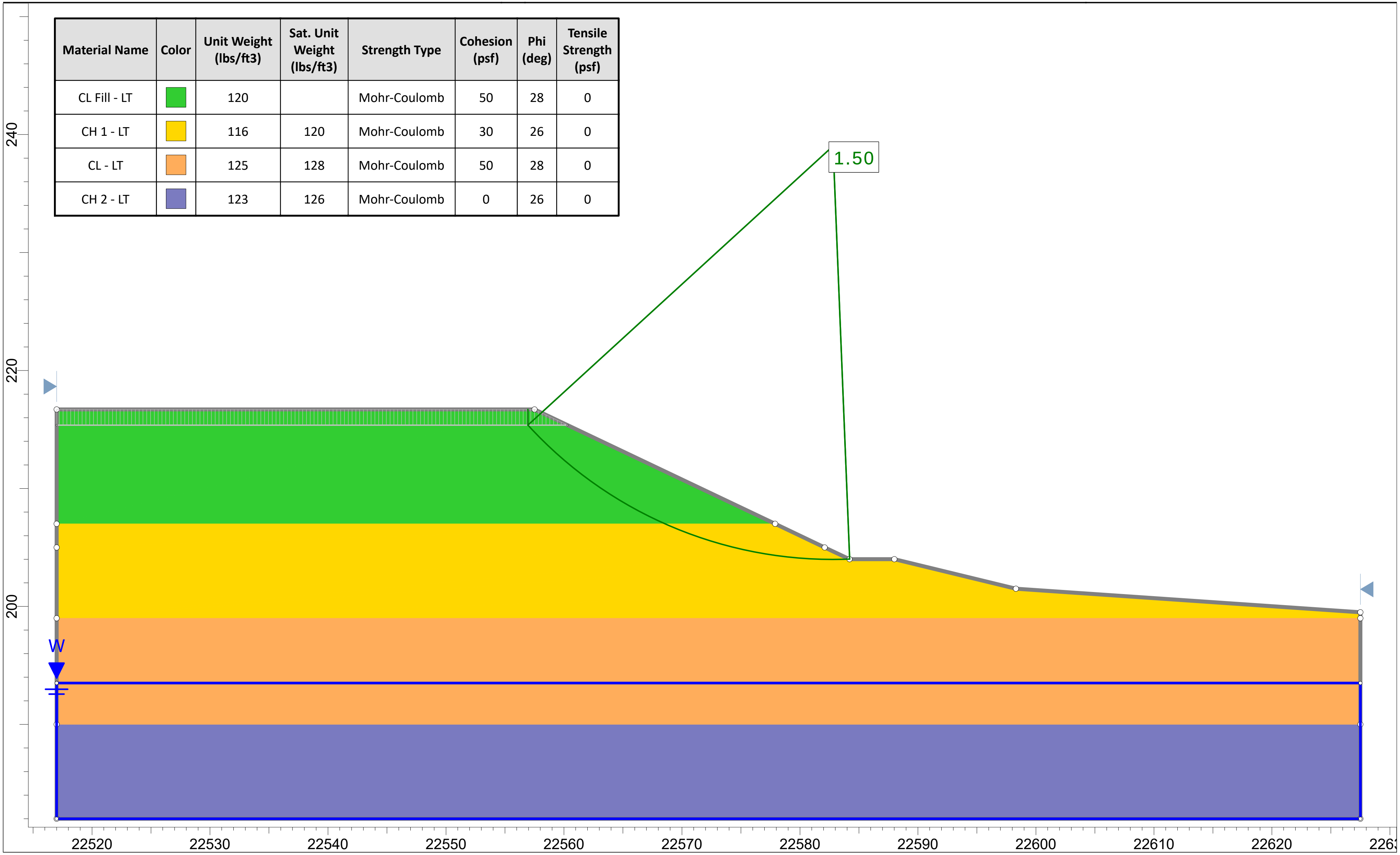
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: South Abutment Spill Slope
Analysis Description: Long Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 3/1/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - LT	🟩	120		Mohr-Coulomb	50	28	0
CH 1 - LT	🟨	116	120	Mohr-Coulomb	30	26	0
CL - LT	🟧	125	128	Mohr-Coulomb	50	28	0
CH 2 - LT	🟪	123	126	Mohr-Coulomb	0	26	0

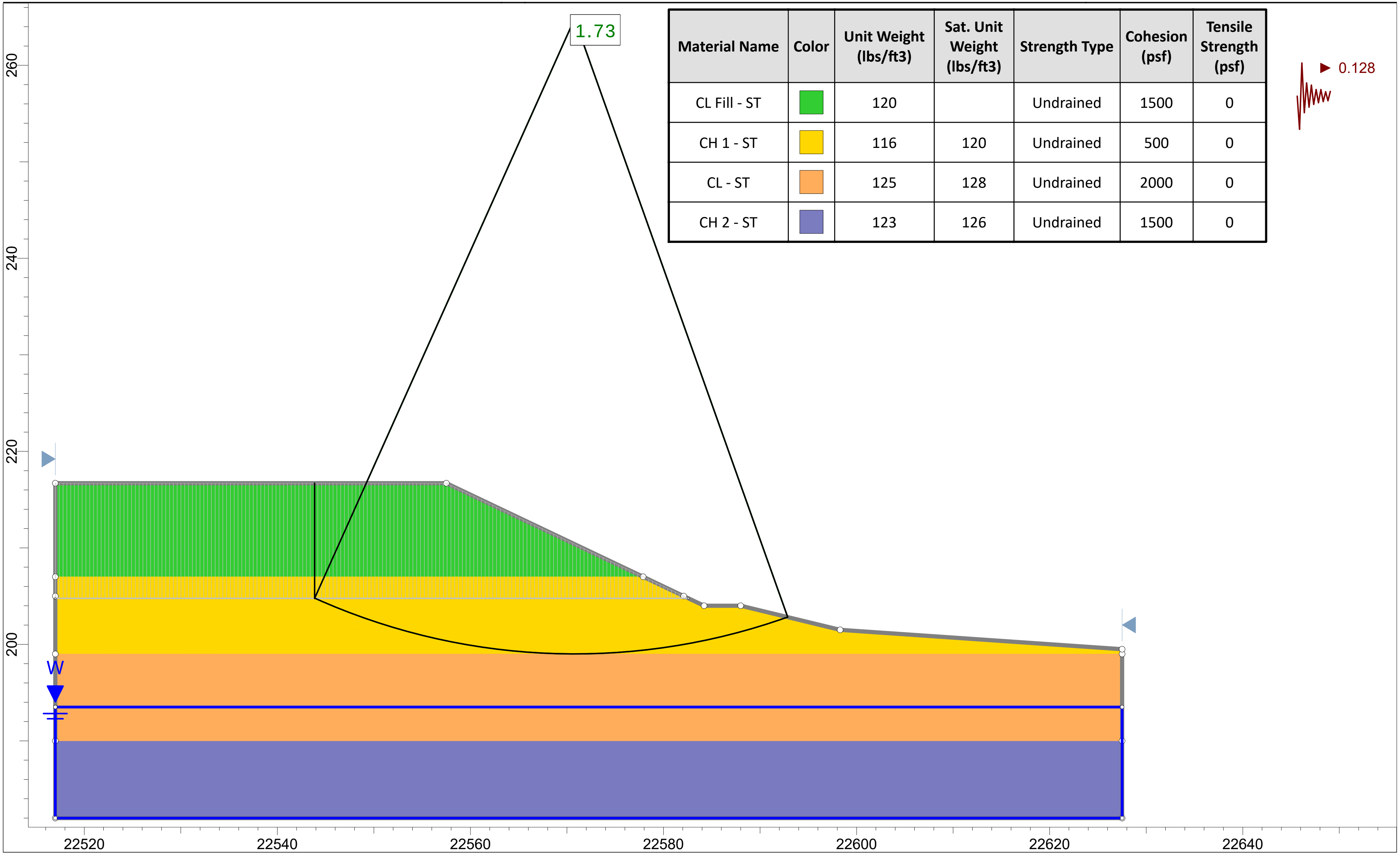


GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: South Abutment Spill Slope
Analysis Description: Seismic
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/17/2021

SLIDEINTERPRET 8.032



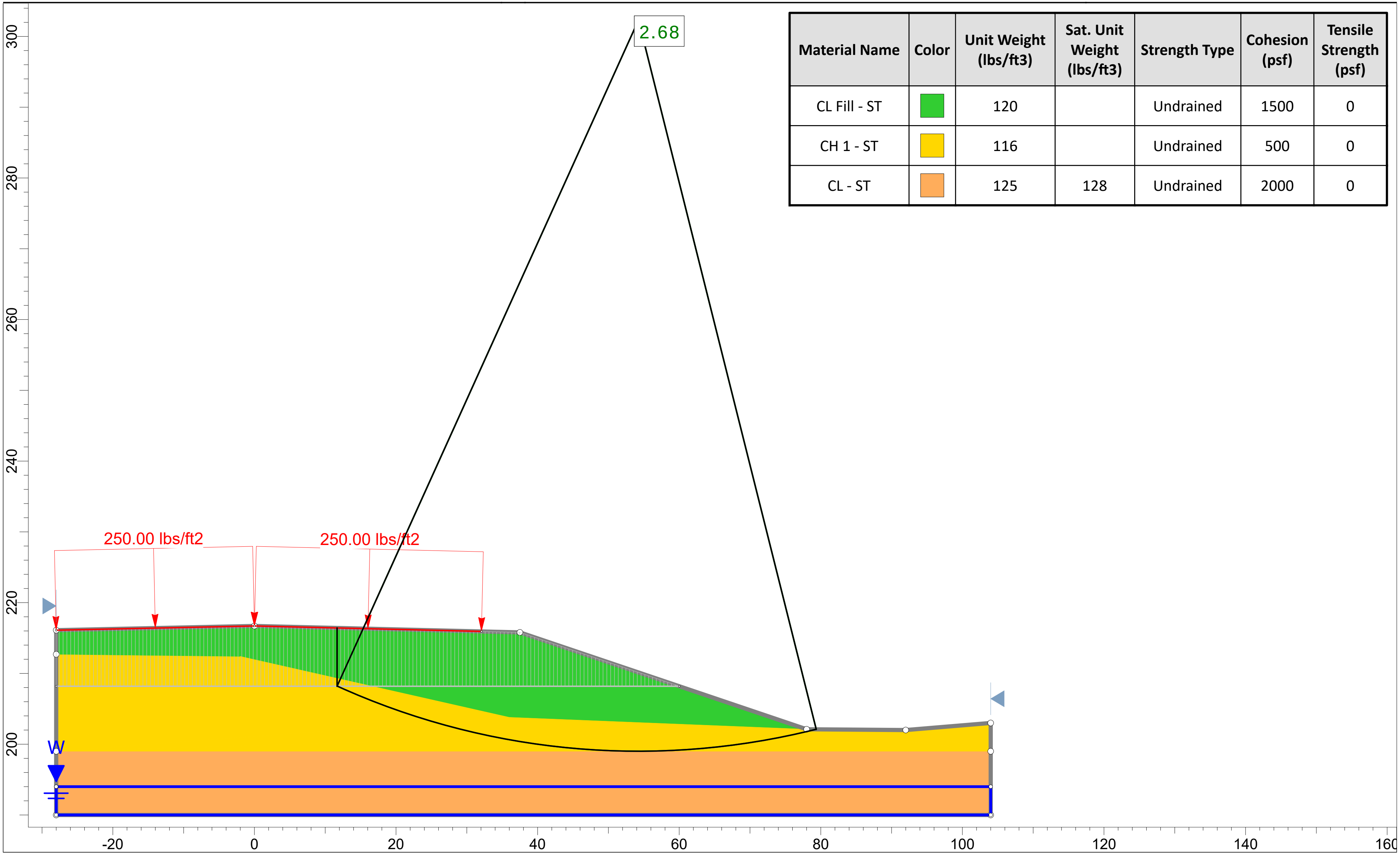
Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CH 1 - ST	🟨	116	120	Undrained	500	0
CL - ST	🟧	125	128	Undrained	2000	0
CH 2 - ST	🟪	123	126	Undrained	1500	0

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: South Abutment Side Slope Station 225+60
Analysis Description: Short Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/18/2021

SLIDEINTERPRET 8.032



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CH 1 - ST	🟨	116		Undrained	500	0
CL - ST	🟧	125	128	Undrained	2000	0

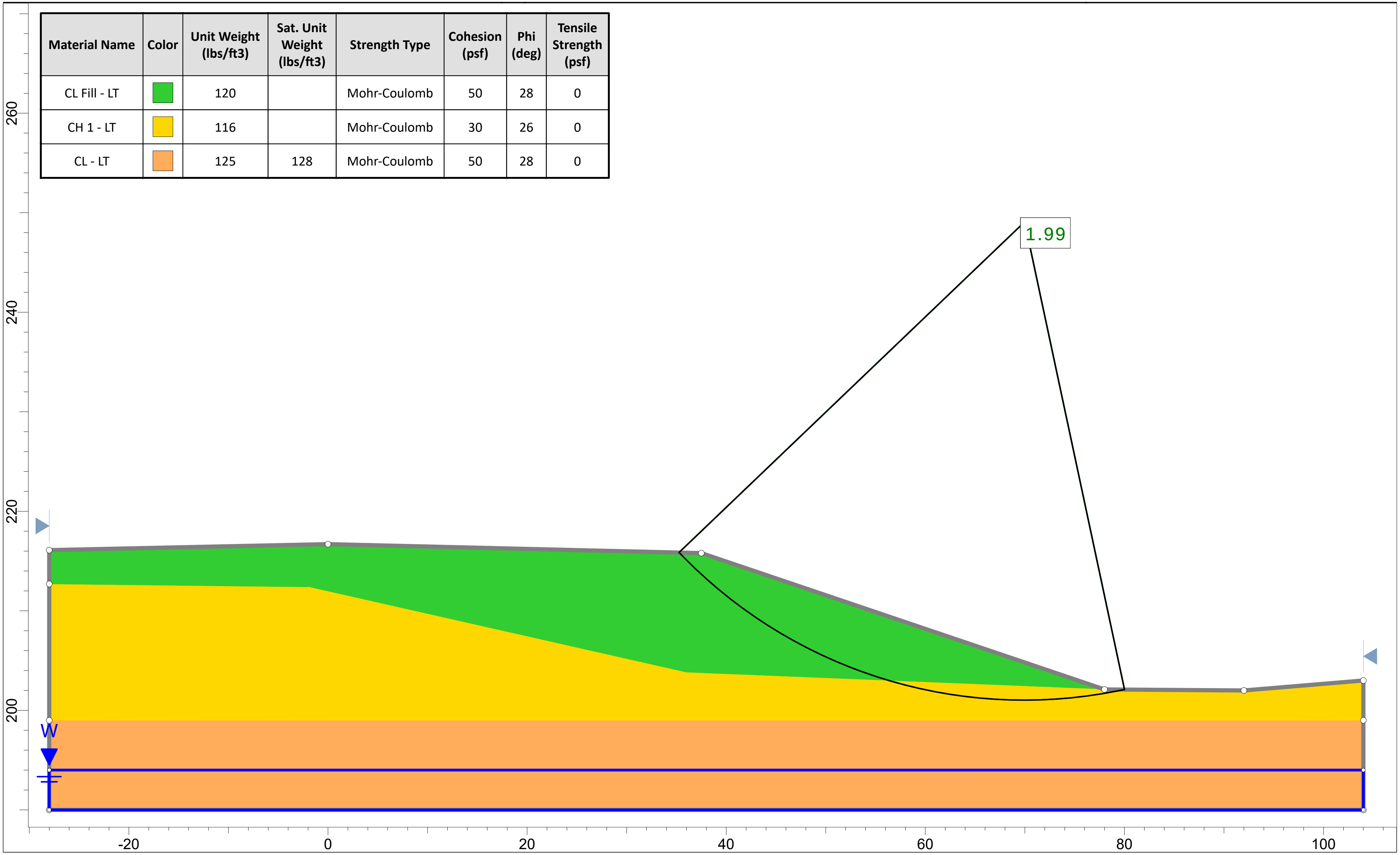
GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: South Abutment Side Slope Station 225+60
Analysis Description: Long Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 3/1/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill - LT	🟩	120		Mohr-Coulomb	50	28	0
CH 1 - LT	🟨	116		Mohr-Coulomb	30	26	0
CL - LT	🟧	125	128	Mohr-Coulomb	50	28	0

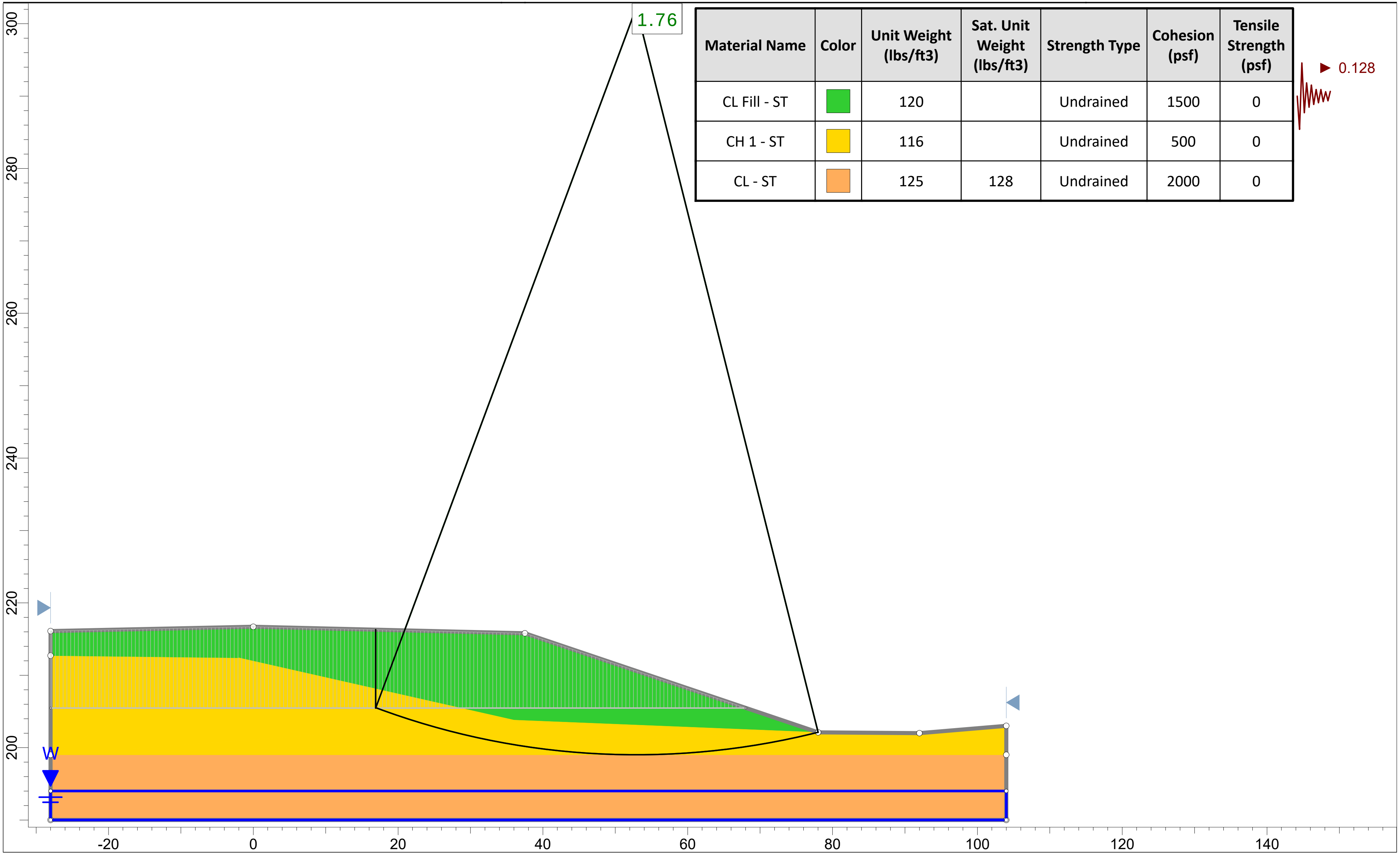


Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: South Abutment Side Slope Station 225+60
Analysis Description: Seismic
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/18/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill - ST	🟩	120		Undrained	1500	0
CH 1 - ST	🟨	116		Undrained	500	0
CL - ST	🟧	125	128	Undrained	2000	0



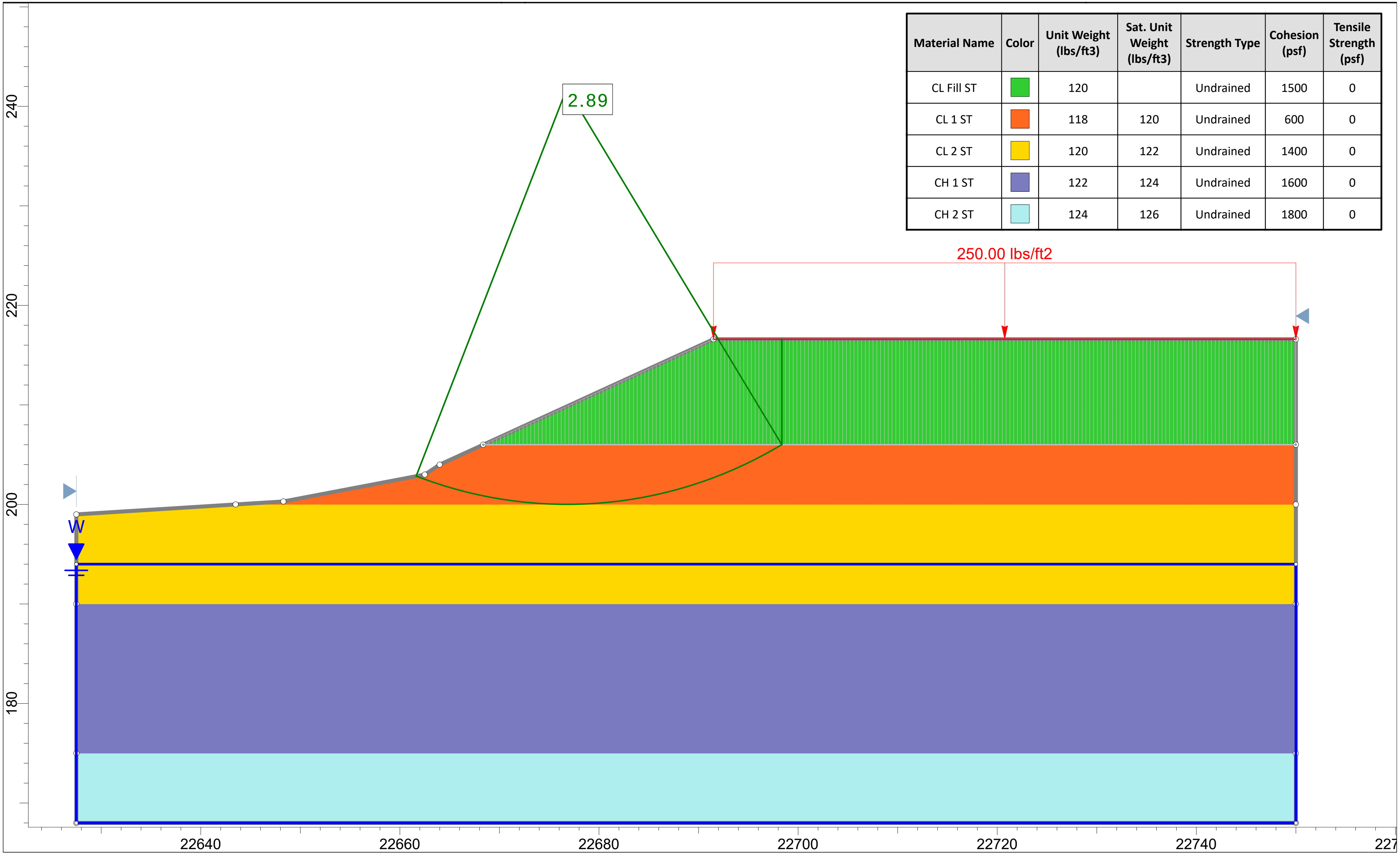
GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: North Abutment Spill Slope
Analysis Description: Short Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/18/2021

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST	🟩	120		Undrained	1500	0
CL 1 ST	🟧	118	120	Undrained	600	0
CL 2 ST	🟨	120	122	Undrained	1400	0
CH 1 ST	🟪	122	124	Undrained	1600	0
CH 2 ST	🟦	124	126	Undrained	1800	0

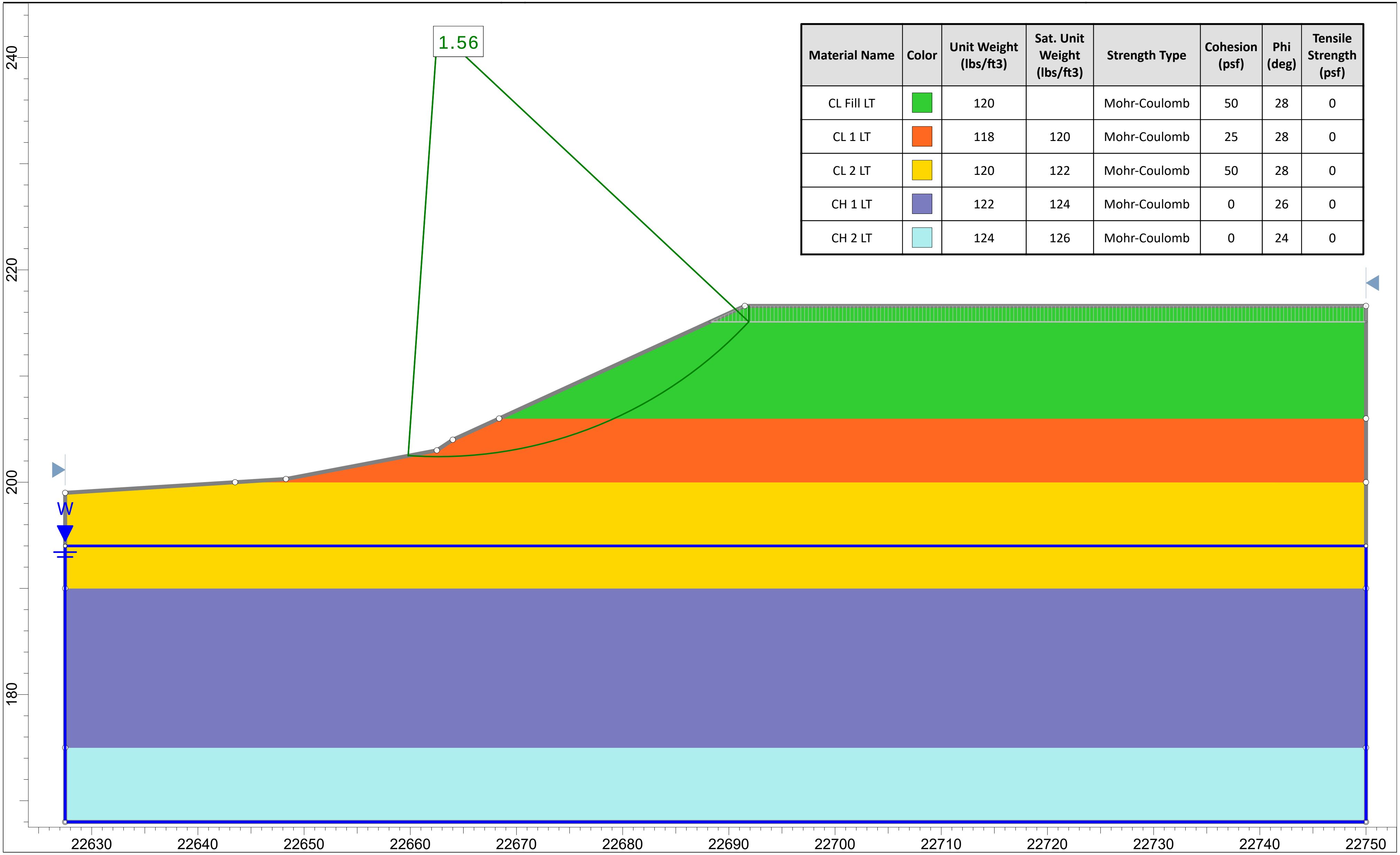


GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: North Abutment Spill Slope
Analysis Description: Long Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 3/1/2021

SLIDEINTERPRET 8.032



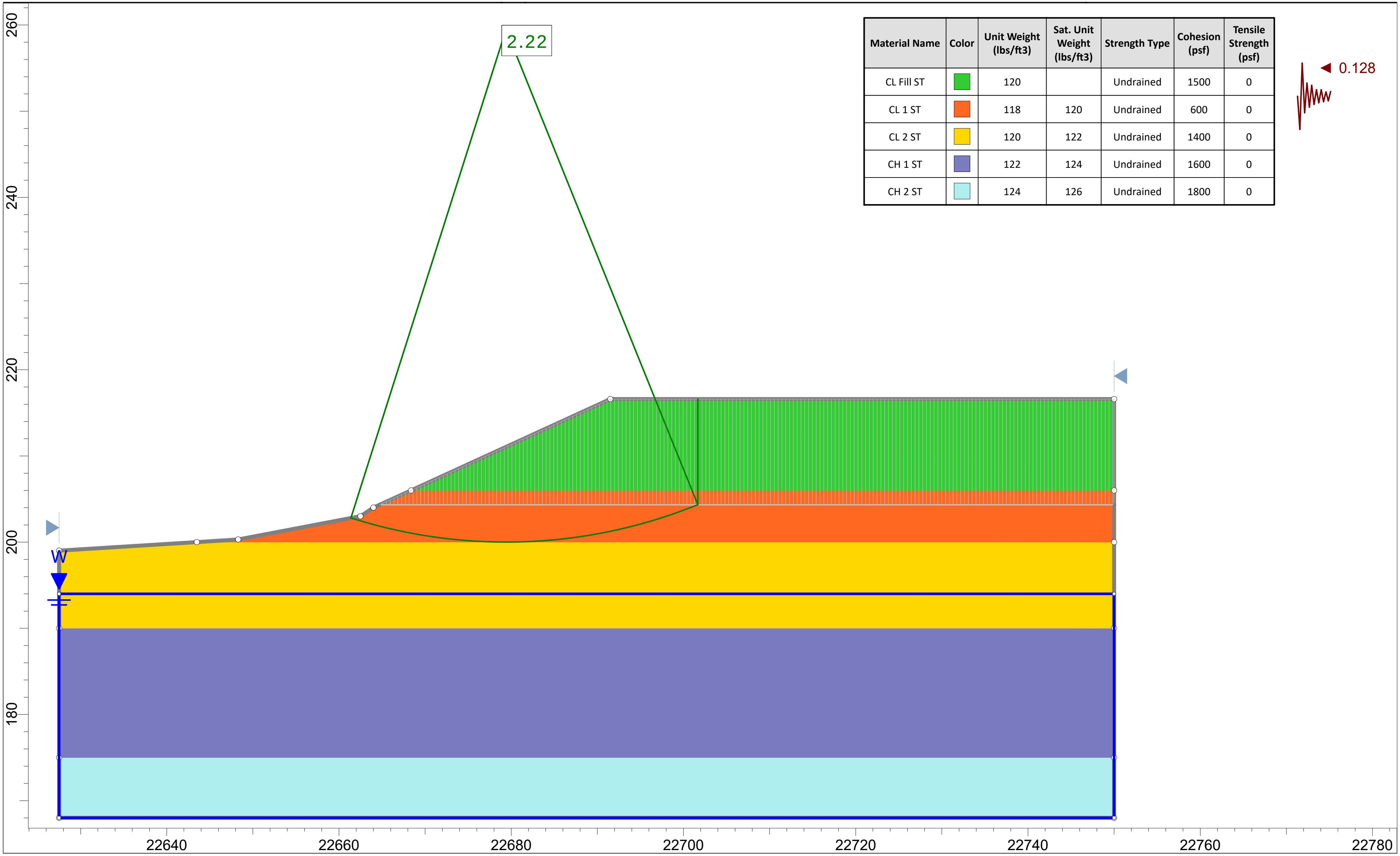
Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill LT	🟩	120		Mohr-Coulomb	50	28	0
CL 1 LT	🟧	118	120	Mohr-Coulomb	25	28	0
CL 2 LT	🟨	120	122	Mohr-Coulomb	50	28	0
CH 1 LT	🟪	122	124	Mohr-Coulomb	0	26	0
CH 2 LT	🟦	124	126	Mohr-Coulomb	0	24	0

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: North Abutment Spill Slope
Analysis Description: Seismic
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/18/2021

SLIDEINTERPRET 8.032



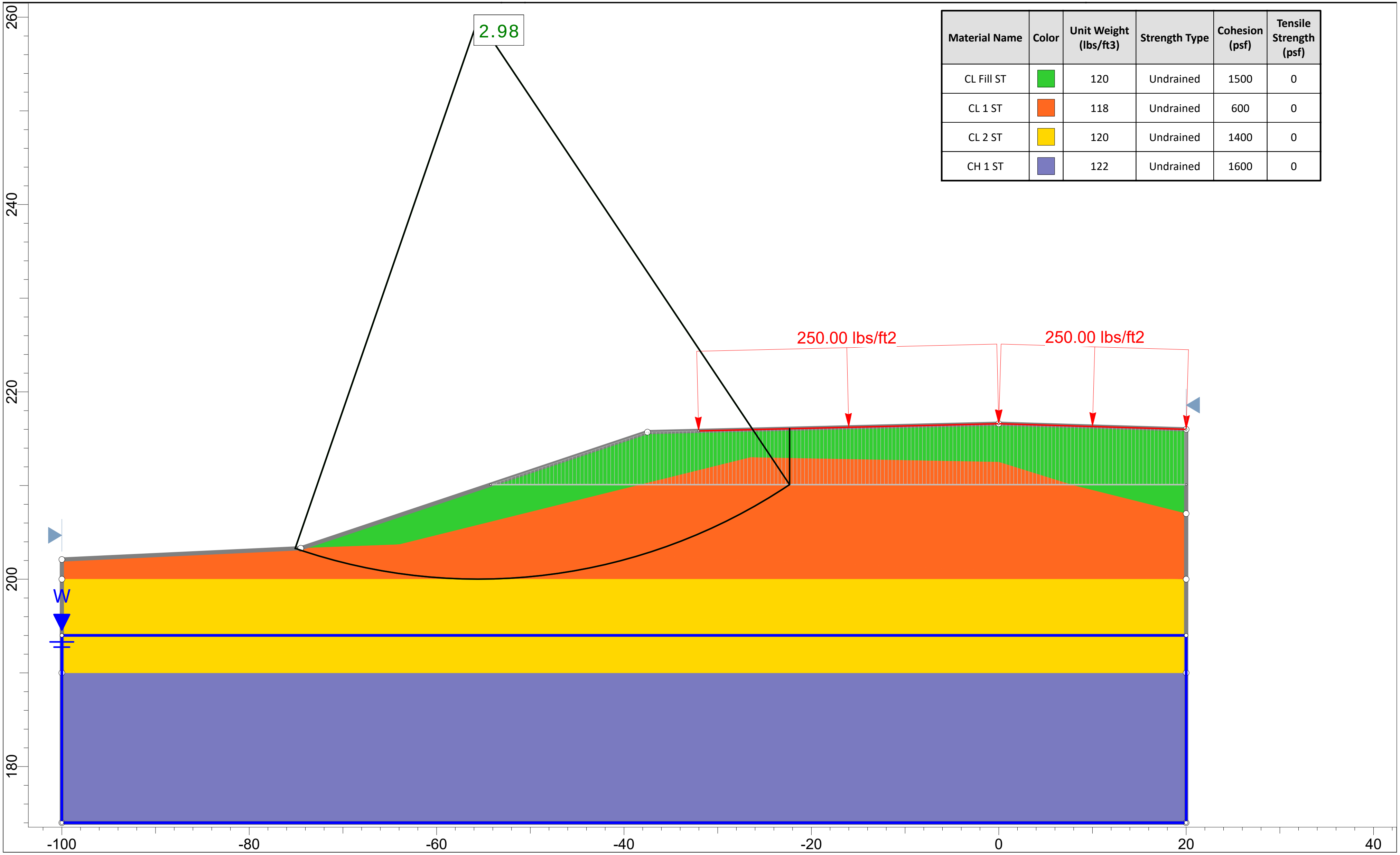
Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST	🟩	120		Undrained	1500	0
CL 1 ST	🟧	118	120	Undrained	600	0
CL 2 ST	🟨	120	122	Undrained	1400	0
CH 1 ST	🟪	122	124	Undrained	1600	0
CH 2 ST	🟦	124	126	Undrained	1800	0

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: North Abutment Side Slope Station 226+80
Analysis Description: Short Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/18/2021

SLIDEINTERPRET 8.032



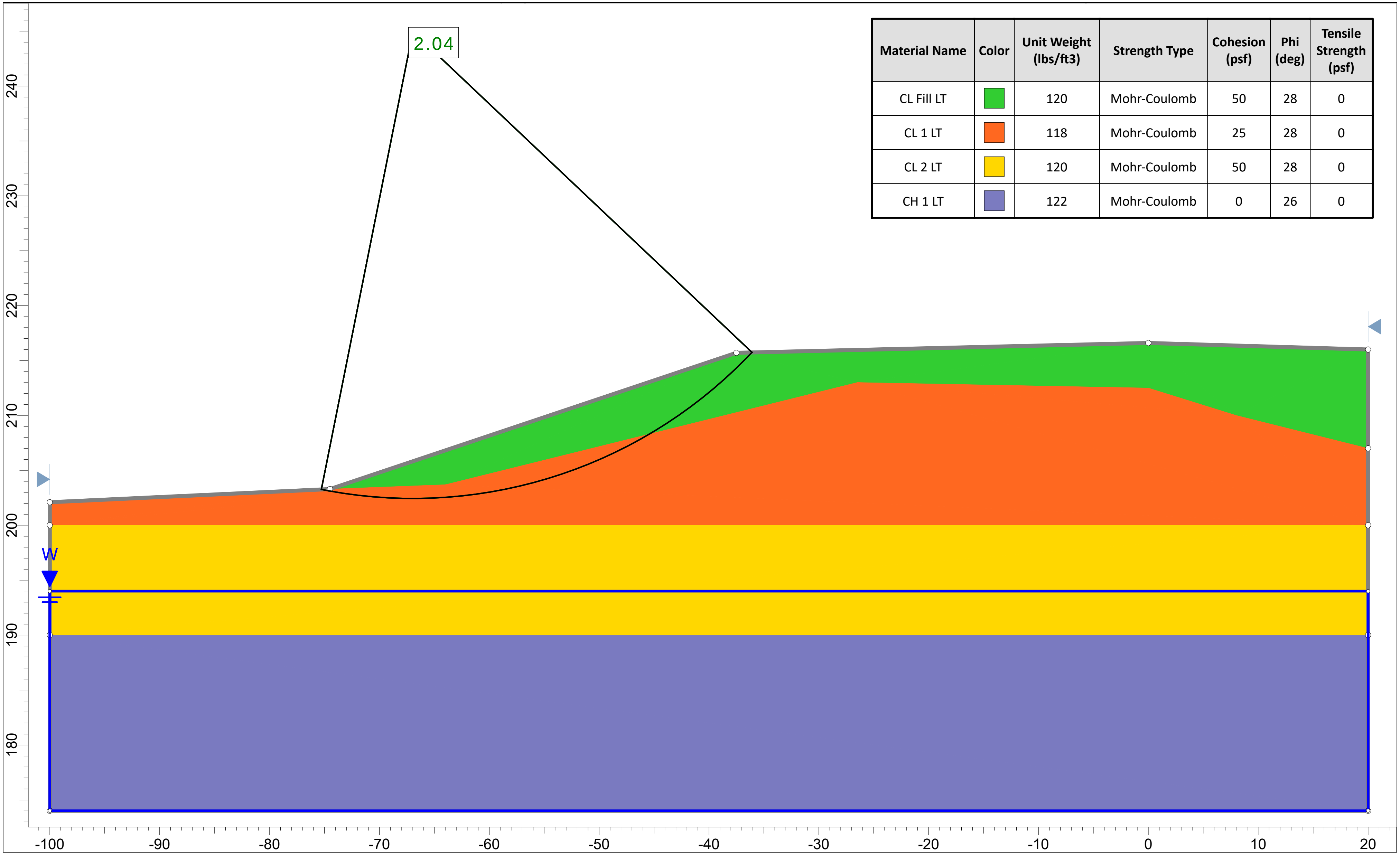
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST	🟩	120	Undrained	1500	0
CL 1 ST	🟧	118	Undrained	600	0
CL 2 ST	🟨	120	Undrained	1400	0
CH 1 ST	🟪	122	Undrained	1600	0

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: North Abutment Side Slope Station 226+80
Analysis Description: Long Term
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 3/1/2021

SLIDEINTERPRET 8.032



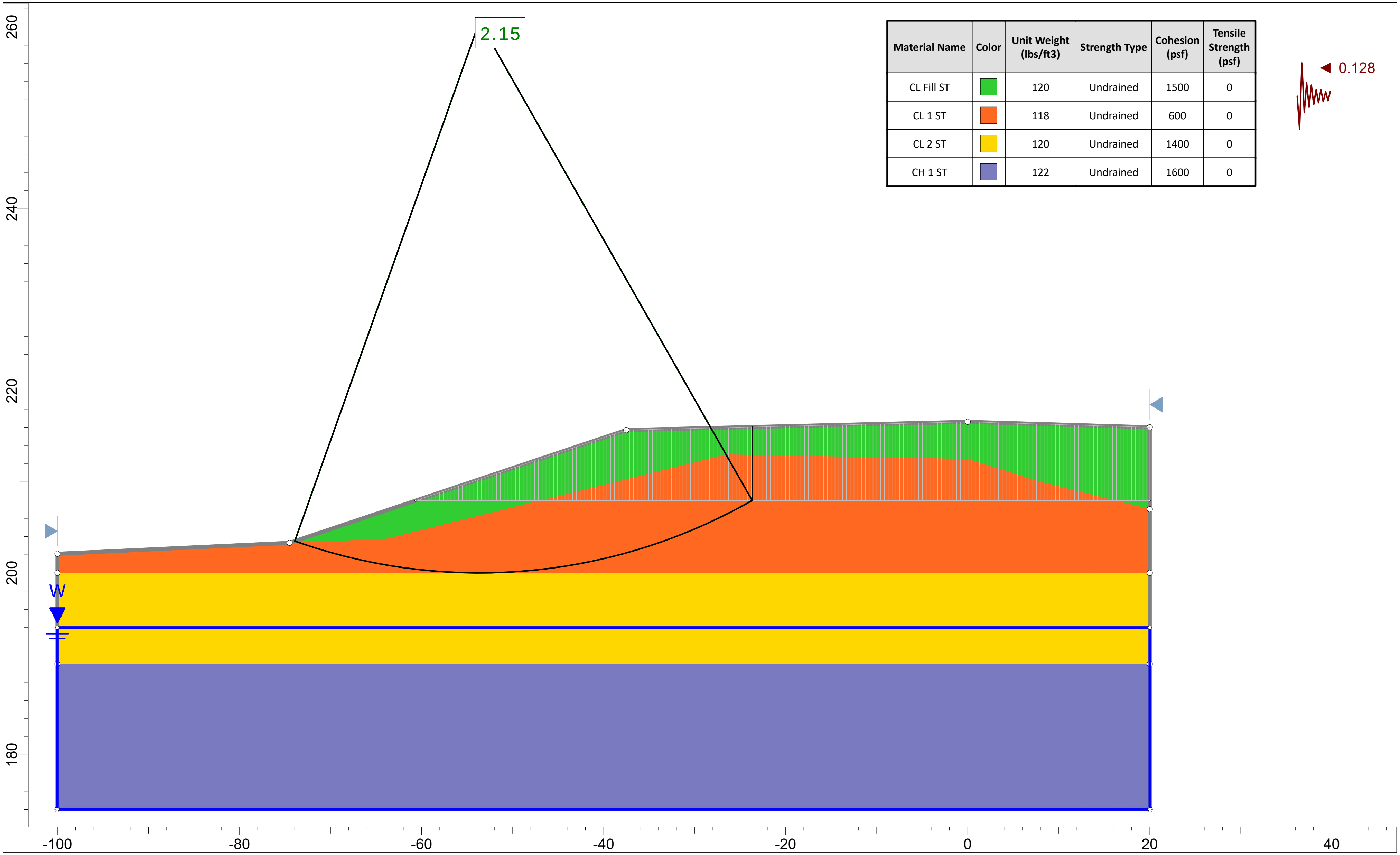
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Tensile Strength (psf)
CL Fill LT	🟩	120	Mohr-Coulomb	50	28	0
CL 1 LT	🟧	118	Mohr-Coulomb	25	28	0
CL 2 LT	🟨	120	Mohr-Coulomb	50	28	0
CH 1 LT	🟪	122	Mohr-Coulomb	0	26	0

GEOTECHNOLOGY
A Universal Engineering Sciences Company

Project Name: Highway 63 Bridge No. 01860 Over La Grue Bayou North
Analysis Name: North Abutment Side Slope Station 226+80
Analysis Description: Seismic
Method: Spencer

ARDOT Project No.: 061615
Geotechnology Project No.: J034561.01
Location: Prairie County, Arkansas
Date: 2/18/2021

SLIDEINTERPRET 8.032



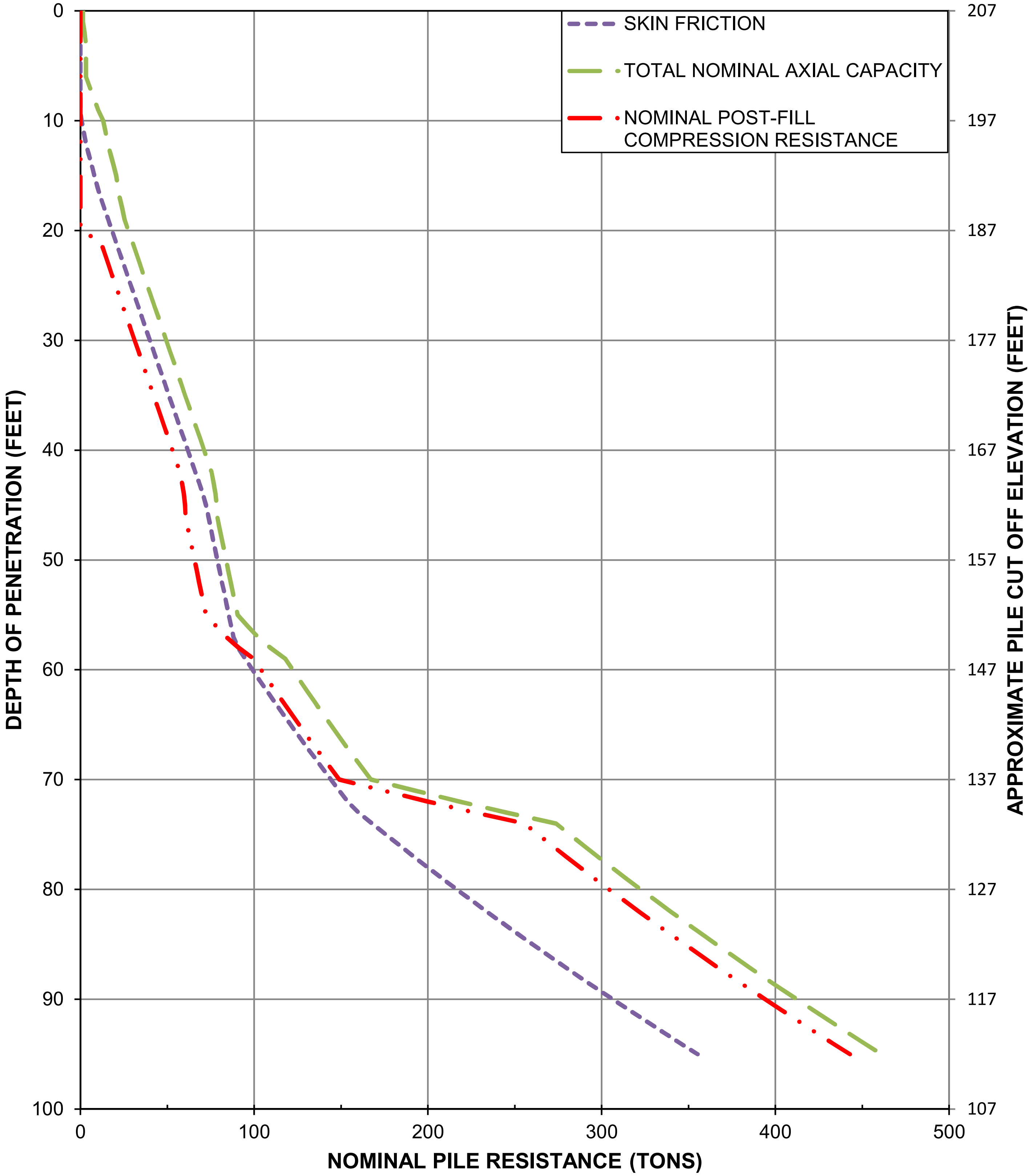
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST	🟩	120	Undrained	1500	0
CL 1 ST	🟧	118	Undrained	600	0
CL 2 ST	🟨	120	Undrained	1400	0
CH 1 ST	🟪	122	Undrained	1600	0

Appendix F

NOMINAL RESISTANCE CURVES FOR DRIVEN PILES

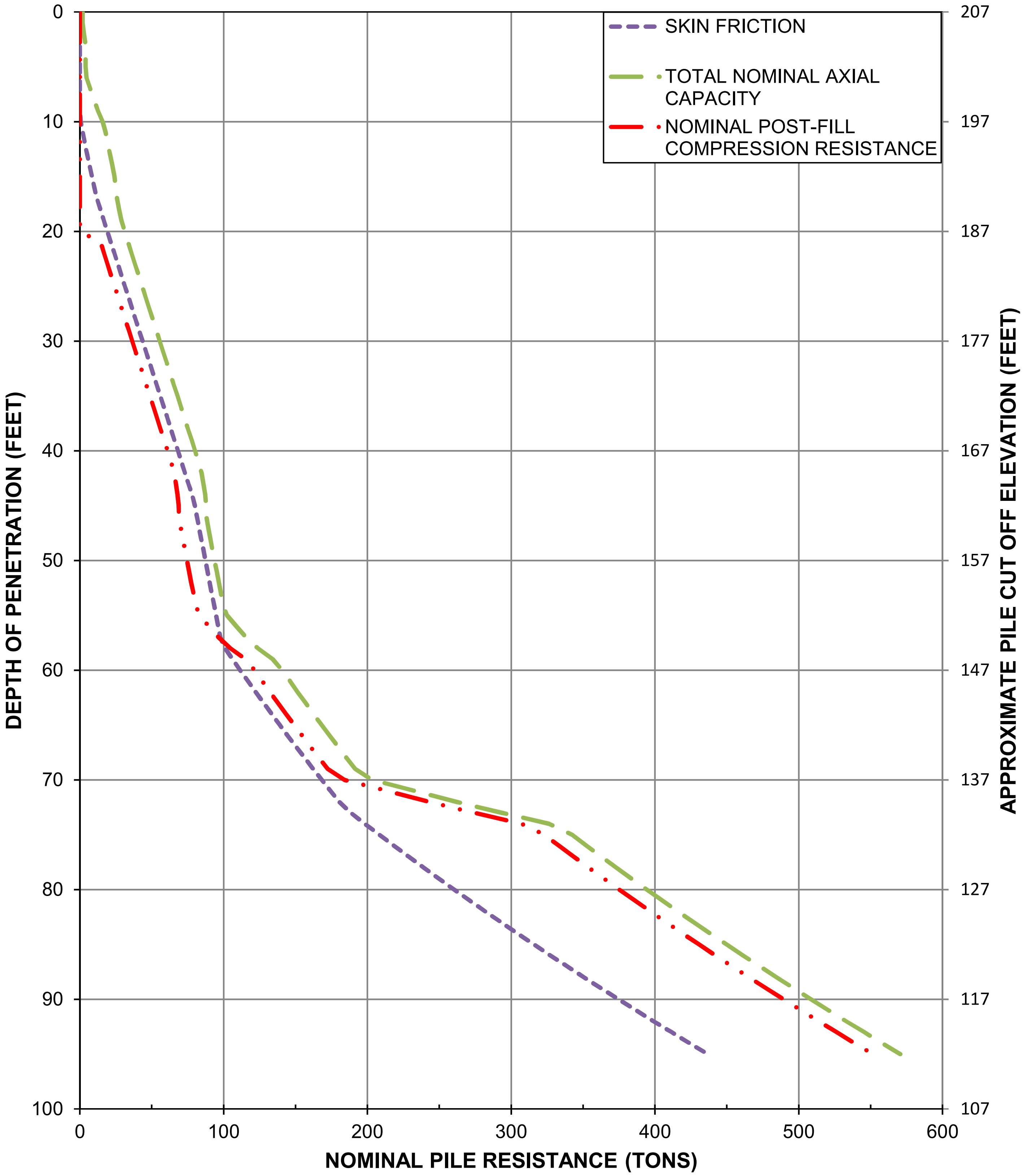
SOUTH ABUTMENT (BENT NO. 1)
HWY 63 BRIDGE NO. 01860 OVER LA GRUE BAYOU

NOMINAL RESISTANCE CURVES
DRIVEN 16 INCH, CLOSED-ENDED, PIPE PILES



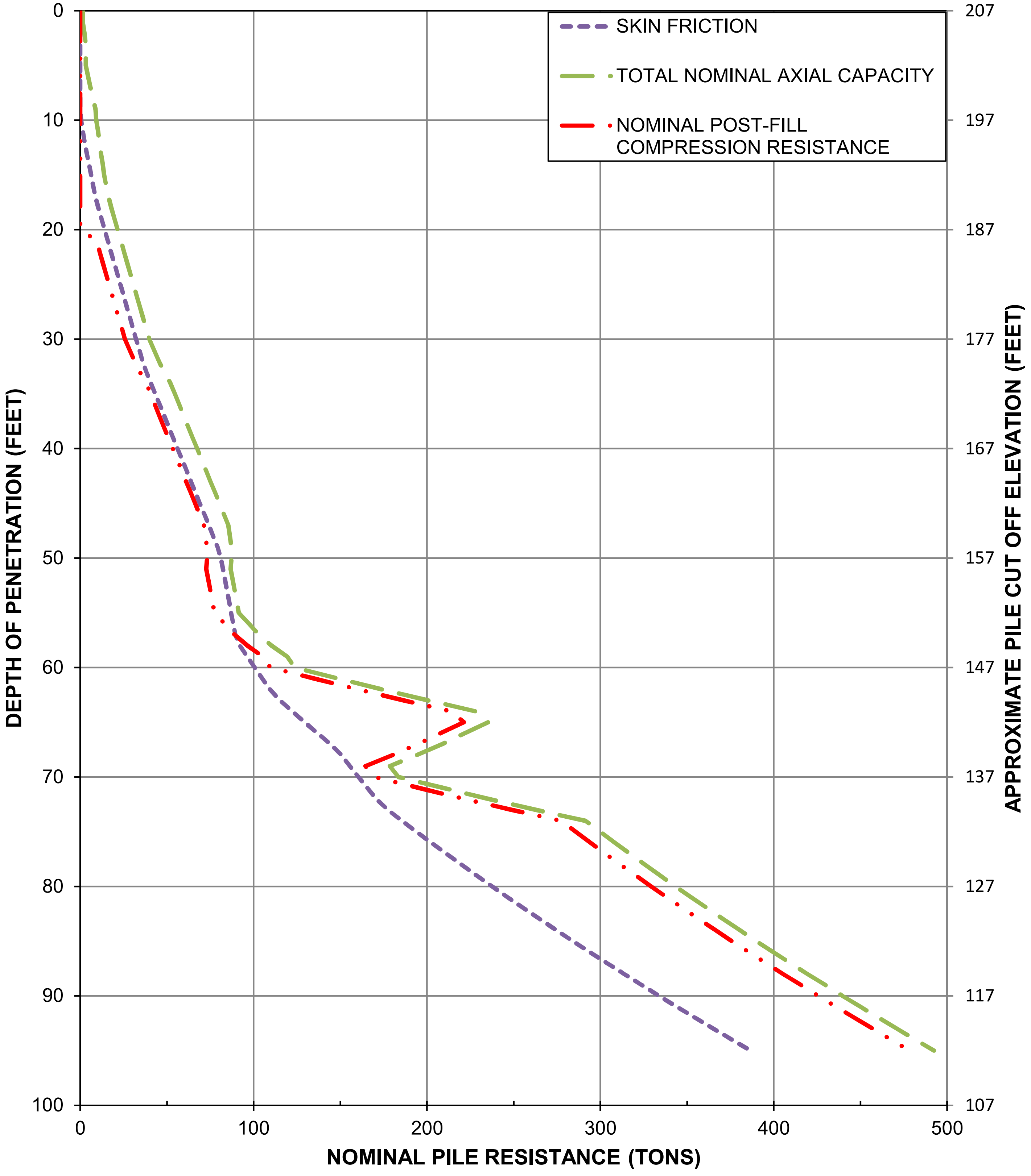
SOUTH ABUTMENT (BENT NO. 1)
HWY 63 BRIDGE NO. 01860 OVER LA GUE BAYOU

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



NORTH ABUTMENT (BENT NO. 2)
HWY 63 BRIDGE NO. 01860 OVER LA GRUE BAYOU

NOMINAL RESISTANCE CURVES
DRIVEN 16 INCH, CLOSED-ENDED, PIPE PILES

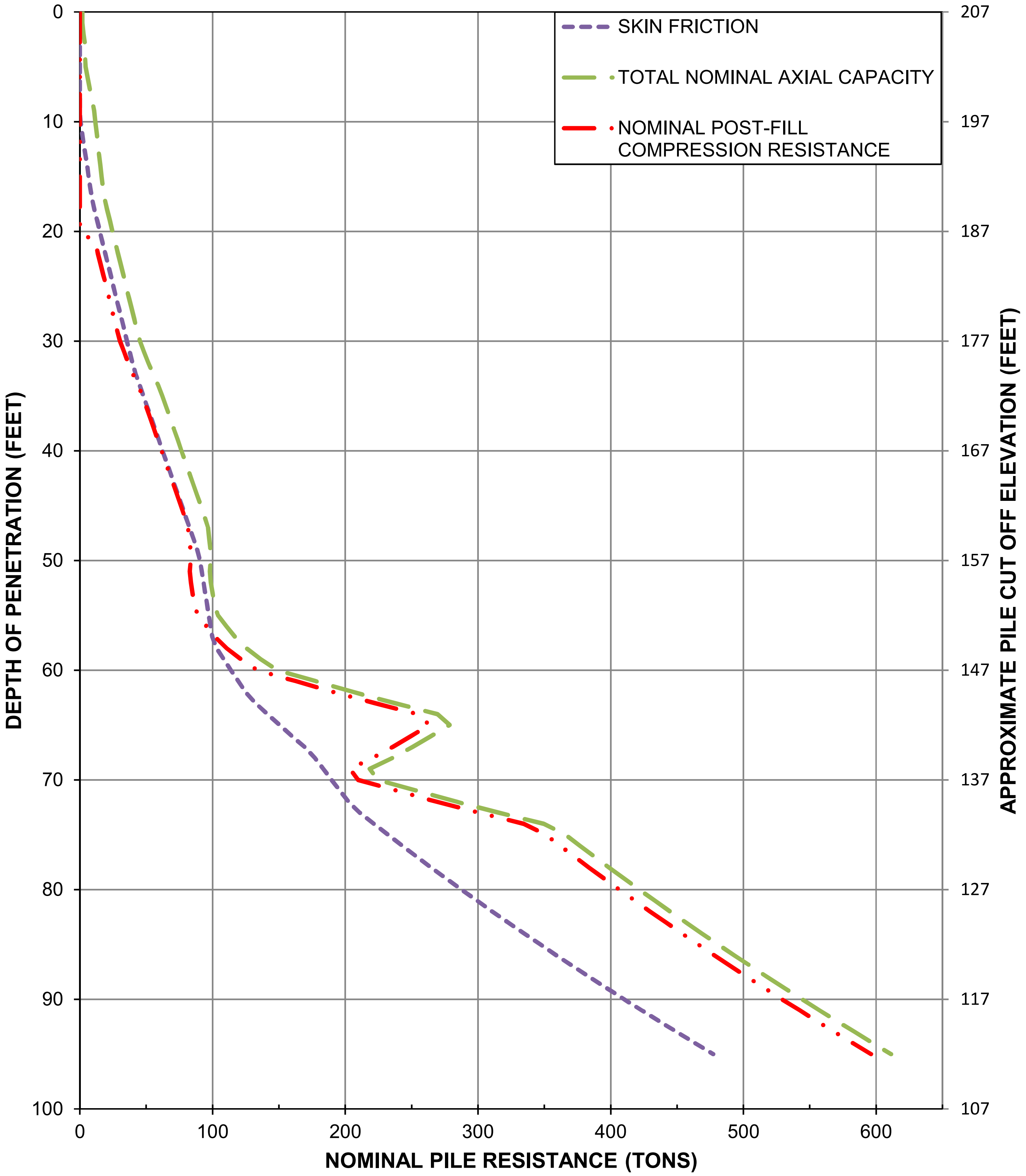


GEOTECHNOLOGY PROJECT NUMBER J034561.01

ARDOT 061615

NORTH ABUTMENT (BENT NO. 2)
HWY 63 BRIDGE NO. 01860 OVER LA GRUE BAYOU

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



GEOTECHNOLOGY PROJECT NUMBER J034561.01

ARDOT 061615

Appendix G
SOIL PARAMETERS FOR SYNTHETIC PROFILES

SOUTH ABUTMENT (BENT NO. 1) - BORING L-3											
ZONE	SOIL TYPES	ELEVATION[a]		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)			
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)	SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]
1	Engineered Fill (Cohesive)	217[b]	207	120	1,500	--	50	28	0.007	500
2	Fat Clay	207	199	116	500	--	30	26	0.01	100
3	Lean Clay	199	190	125	2,000	--	--	28	0.007	500
4	Fat Clay	190	163	123	1,500	--	--	26	0.007	500
5	Fat Clay	163	150	123	800	--	--	26	0.01	100
6	Sandy Silt	150	135	114	--	32	--	32	--	20
7	Silty Sand	135	113	128	--	36	--	36	--	125

Note: Groundwater was not encountered in Boring L-3, but was encountered in Boring L-4 at approximate El 194. The effective unit weight should be used below the ground water level. Subtract 62.4 from the total unit weight to calculate the effective unit weight.

[a] Elevations are approximated from the provided drawing

[b] Approximate final grade at south abutment

[c] Pounds per cubic inch

[d] For lateral load analysis only

NORTH ABUTMENT (BENT NO. 2) - BORING L-4

ZONE	SOIL TYPES	ELEVATION[a]		TOTAL UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD PARAMETERS[d]	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)			
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)	SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[c]
1	Engineered Fill (Cohesive)	217[b]	207	120	1,500	--	50	28	0.007	500
2	Lean Clay	207	200	118	600	--	--	28	0.01	100
3	Lean Clay	200	190	120	1,400	--	--	28	0.007	500
4	Fat Clay	190	175	122	1,600	--	--	26	0.007	500
6	Fat Clay	175	150	120	1,800	--	--	24	0.007	500
7	Silty Sand	150	145	125	--	32	--	32	--	60
8	Silty Sand	145	140	128	--	36	--	36	--	125
9	Clayey Sand	140	135	125	--	32	--	32	--	60
10	Sand	135	113	130	--	36	--	36	--	125

Note: Groundwater was encountered in Boring L-4 at approximate El 194. The effective unit weight should be used below the ground water level. Subtract 62.4 from the total unit weight to calculate the effective unit weight.

[a] Elevations are approximated from the provided drawing

[b] Approximate final grade at north abutment

[c] Pounds per cubic inch

[d] For lateral load analysis only