ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 030497

FEDERAL AID PROJECT NO. NHPP-0046(50)

MILL & BODCAU CREEKS STRS. & APPRS. (S)

STATE HIGHWAY 82 **SECTION** 1 & 2

IN LAFAYETTE & MILLER **COUNTY**

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

GEOTECHNICAL REPORT
HIGHWAY 82 STRS. AND APPRS.(S)
BRIDGE OVER BODCAU CREEK
LAFAYETTE COUNTY, ARKANSAS

ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT NO. 030497

Prepared for:

GARVER, LLC
NORTH LITTLE ROCK

Prepared by:

GEOTECHNOLOGY, INC.
MEMPHIS, TENNESSEE

Date:

AUGUST 13, 2020

Geotechnology Project No.:

J028499.03A

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

GEOTECHNOLOGY INC
FROM THE GROUND UP

August 13, 2020

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

Re: Geotechnical Report
Highway 82 Strs. and Apprs.(S)
Bridge Over Bodcau Creek
Lafayette County, Arkansas
Geotechnology Project No. J028499.03A

Dear Mr. Ruddell:

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

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

Respectfully submitted,

GEOTECHNOLOGY, INC.

Dale M Smith

Dale M. Smith, P.E.
Geotechnical Manager

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8/13/20

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GEOTECHNICAL REPORT
HIGHWAY 82 STRS. AND APPRS.(S)
BRIDGE OVER BODCAU CREEK
LAFAYETTE COUNTY, ARKANSAS
August 13, 2020 | Geotechnology Project No. J028499.03A

CHAPTER 1. SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design and construction for the proposed improvements to Highway 82 (Hwy 82) in Lafayette County, Arkansas (Station 202+00.00 to Station 223+18.45). The referenced improvements consist of the replacement of Bridge No 02122 over Bodcau Creek. The new six-span bridge (Station 210+79.06 to Station 214+39.39) will be approximately 360-foot-long and constructed in two phases. During phase 1, a portion of the new bridge will be constructed to the south of the existing bridge. Facilitating traffic to the new bridge will be require widening of the existing approaches. In phase 2 traffic will be redirected to the partially completed bridge, and the existing bridge will be demolished and the remaining portion of the bridge completed. When complete, the new bridge will be approximately 78 feet wide. The site location 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 the borings, in-situ testing, sampling and laboratory testing are included in the report. A total of 14 borings were drilled at intervals along the proposed Highway 82 bridge over Bodcau Creek as shown in Figure 2. The boring logs, along with field and laboratory test results, are enclosed. The collected data have been analyzed and the physical properties of the in-situ soils summarized. General site conditions are discussed, along with recommendations for subgrade preparation. Important information prepared by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association for studies of this type is presented in Appendix A for your review.

CHAPTER 2. GENERAL INFORMATION

Planned Modifications

It is our understanding the existing bridge over Bodcau Creek will remain in use through the first phase of construction before being demolished and replaced in phase 2. The existing bridge approaches will be widened to facilitate traffic across the widened bridge.

The modifications to the approaches will require widening of the existing bridge approaches; beginning at Station 208+20.00, the existing road-way will be widened to the south to allow for five lanes of traffic (two in the eastbound and west bound directions and one center turn lane). Widening will end at the western bridge abutment at Station 210+79.06. The widening will require a wedge of fill to be placed on the southern shoulders of the existing road way between Station 208+20.00 and Station 210+79.06 with a maximum fill height of 8 feet at the bridge abutment. The planned side slopes of the western approach are 3 horizontal units for every 1 vertical unit (3H:1V).

The proposed six-span bridge will cross Bodcau Creek. It is our understanding that minimal grade changes will be required at the bent locations. The bridge abutments will require up to 10 feet of fill and 11 feet of cut. A 2H:1V slope is planned for the bridge abutments.

Widening of the eastern bridge approach will extend from the eastern bridge abutment at Station 214+39.39 until the end of project at Station 217+00.00. The proposed widening will require a wedge of fill to be placed in the southern shoulders of the existing road way between Stations 214+39.39 and 217+00.00, with a maximum fill height of 10 feet occurring at the eastern bridge abutment. The planned side slopes of the eastern bridge approach are 3V:1H.

Topography

The proposed Hwy 82 bridge over Bodcau Creek is located in Lafayette County, Arkansas. According to provided plans[1], the elevations at the west and east abutments are El 258.90[2] and 258.80, respectively, with a maximum of approximately 34 feet of relief across the proposed alignment.

Drainage

The drainage system in the project area consists of the Bodcau Bayou Watershed. The Bodcau Bayou Watershed, in turn, is part of the overall drainage system of the Red River Basin.

Geology

Lafayette County is located in southwestern Arkansas, in the Gulf Coastal Plain. The Gulf Coastal Plain extends across the southern United States and is bounded to the north by the Ouachita Mountains. Approximately 50 million years ago, prior to tectonic uplift, the area was covered by the Gulf of Mexico. The Coastal Plain is characterized by flat to rolling topography.

The geology in the Bodcau Creek area is characterized by an upper layer of alluvium which features predominately alluvial deposits of present streams. Below the alluvium, the geology is generally characterized by the Wilcox and Claiborne Groups which feature mainly non-marine sands, silty sands, clays and gravels. Some thick deposits of lignite are featured within both Groups.

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[1] *Arkansas Department of Transportation Construction Plans for State Highway Mill & Bodcau Creeks STRS. & Apprs. (S) Miller and Lafayette Counties Route 82 Sections 1& 2, Federal Aid Project NHPP-0046(50) Job 030497*. Provided by Garver, dated January 24, 2019.

[2] Elevations are referenced to NAVD 1988 (NAVD 88) in units of feet.

CHAPTER 3. GEOTECHNICAL EXPLORATION

A total of 14 borings were drilled at selected locations near the bridge approaches and the alignment of the proposed bridge. The borings were drilled to approximate depths ranging from 15 to 100 feet. Six cores were performed through the existing pavement. Proposed Boring B-3 was not drilled during exploration due to the presence of rip rap below the bridge and inability to access the sides of the bents.

The borings were drilled on March 14, 2019 and August 6 through 12, 2019 using a rotary drill rig (CME 55LC and CME 550X), hollow-stem augers and wet rotary methods. Sampling procedures included Standard Penetration Test (SPT) and thin-wall (Shelby) tube methods. SPT's were conducted at 2.5, 5, and 10-foot depth intervals using 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 along with an explanation of the terms and symbols used on the boring logs. Included on each boring log are elevation data estimated from the provided plans. Included in Table 1 are in situ tests and measurements made as part of the fieldwork and recorded on the boring logs.

Table 1. Field Tests and Measurements

Item	Test Method
Soil Classification	ASTM D 2488/ D 3282
Standard Penetration Test (SPT)	ASTM D 1586/ AASHTO T206
Thin-Walled (Shelby) Tube Sampling	ASTM D 1587/ AASHTO T207

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

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

CHAPTER 4. LABORATORY REVIEW AND TESTING

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

Table 2. Summary of Laboratory Tests and Methods.

Laboratory Test	ASTM	AASHTO
Moisture Content	D 2216	T 265
Atterberg Limits	D 4318	T 98
Grain Size Analysis	D 422	T 88
Percent Finer Than No. 200 Sieve	D 1140	T 11
Unconsolidated-Undrained Triaxial Compression	D 2850	T 296
Direct Shear	D 3080	T 236
One-Dimensional Consolidation	D 2435	T 216
pH of Soil	D 4972	T 289
Soil Electrical Resistivity	G 57	T 288
Moisture-Density (Standard Effort)	D 698	T 99
California Bearing Ratio (CBR)	D 1883	T 193

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

CHAPTER 5. SUBSURFACE CONDITIONS

Existing Pavement

Borings BC-12 through BC-15 were drilled in the existing pavement at the bridge approaches for the purpose of obtaining pavement thickness and subgrade information beneath the existing road-way. A summary of the pavement materials and thicknesses is provided in Table 3.

Table 3. Summary of Encountered Pavement Materials and Thicknesses.

Boring No.	Surface		Base	
	Material	Thickness (in.)	Material	Thickness (in.)
BC-12	Asphalt	3½	Sand and Gravel	8½
BC-13	Asphalt	2	Sand and Gravel	10
BC-14	Asphalt	2½	Silty Sand	9½
BC-15	Asphalt	10	Silty Sand	20

*Asphalt Core Only

Subgrade Materials

The borings were drilled in the alignment of the proposed bridge and approaches, and were drilled through either asphalt or approximately 3 inches of topsoil or gravel. Underlying the topsoil, asphalt, or gravel the soils generally consisted of interbedded fine- and coarse-grained soils underlain by predominately coarse-grained soils extending to the 100-foot maximum depth of exploration. The boring logs, with more detailed soil descriptions, are included in Appendix C. The laboratory testing was used to determine the USCS and AASHTO classifications as presented in Appendix E.

The upper, interbedded fine- and coarse-grained soils were classified as high plasticity “fat” clay (CH), AASHTO A-2-7; low plasticity “lean” clay (CL), AASHTO A-6, A-2-7; silt (ML), AASHTO A-4, with sand; clayey sand (SC), AASHTO A-4, A-6; silty sand (SM), AASHTO A-2-4; poorly graded gravel (GP), AASHTO A-1; poorly-graded sand with clay (SP-SC), AASHTO A-1-b; poorly graded sand with silt (SP-SM), AASHTO A-1-b; and poorly graded sand (SP), AASHTO A-3, A-1-b. Coarse-grained soils in the interbedded layer ranged from very loose to medium dense in consistency and fine-grained soils ranged from very soft to medium stiff.

The lower, predominately coarse-grained soils were classified as poorly graded sand (SP), AASHTO A-3, A-1-b; poorly graded sand with silt (SP-SM), AASHTO A-1-b; silty sand (SM), AASHTO A-2-4; and clayey sand (SC), AASHTO A-4, A-6. The coarse-grained soils ranged from medium dense to very dense in consistency.

Groundwater

Groundwater was encountered while drilling in the borings at the depths indicated in Table 4. The presence of groundwater in Borings B-1, B-3 through B-6, B-8, and B-10 may have been masked by the effects of wet rotary drilling which introduces water. Groundwater levels could vary significantly over time due to the effects of seasonal variation in precipitation, recharge, flood levels in Bodcau Creek or other factors not evident at the time of exploration.

Table 4. Summary of Groundwater Depths.

Boring No.	Groundwater Depth (ft.)	Groundwater Elevation (ft.)
BC-2	29	227
BC-7	7	228
BC-9	9	232
BC-11	9	235

CHAPTER 6. ENGINEERING EVALUATION, ANALYSIS, AND RECOMMENDATIONS

Site Preparation and Earthwork

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

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

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

Side Slopes. Existing slopes steeper than 4H:1V should be benched prior to placing new fill. Slope ratios of 3H:1V or flatter are recommended for all cut and fill slopes along the proposed alignment. Fill material consists of import cohesive fill as indicated by Garver.

Cut Areas. It is our understanding up to 11 feet of cut will be required to achieve design grade at the existing eastern abutment and up to 4 feet at the western abutment, as indicated on the provided plans. Based on the stratigraphy, excavations will terminate in silty sand, lean clay, fat clay, or silt. After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 95% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D 698/AASTHO T 99). Areas supporting pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

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

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

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

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

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

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

Pavement Design Information

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

Table 5. Summary of Compaction and CBR Test Results.

Boring No.	Depth (ft.)	USCS/ AAHSTO	Liquid Limit (%)	Plasticity Index	Proctor Results		CBR Results				Percent Compaction (%)
					Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)	Blows per Layer	Dry Unit Weight (pcf)	Moisture Content (%)	CBR	
BC-13	1 – 5	CL	31	17	121.0	10.0	25	113.0	13.0	3.1	93.4
		A-6(9)					56	119.4	11.0	6.6	98.7
BC-14	1 – 5	SC	27	15	132.2	6.5	--	--	--	--	--
		A-2-6(0)					--	--	--	--	--

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

Table 6. CBR Interpolation/Extrapolation.

Boring No.	Depth (ft.)	USCS/ AASHTO	CBR at 95% Compaction
BC-13	1 – 5	CL	4.2
		A-6(9)	

Based on the test results and the data presented in the previous table and to account for potential variability at the site, a CBR of 4.0 is recommended for design of pavements for this project. A CBR value of this magnitude will result in a relatively thick, expensive pavement structure. We recommend a 3-foot undercut below the base of pavements and backfilling with better (larger CBR) materials. Two materials are considered herein: A-4 (design CBR value of 8.0) and A-3 (design CBR value of 10.0).

The design CBR values mentioned in the previous paragraph were correlated to Resilient Modulus (M_R) and Resistance (R) values. The correlation was performed using a graph provided by ARDOT from AASHTO (1993) and is presented in Table 7.

Table 7. Soil Design Parameter Recommendations for Pavement Design.

	Soil Classification/Source		
	A-6 (In-Situ)	**A-4 (Import)**	**A-3 (Import)**
	CBR = 4	**CBR = 8**	**CBR = 10**
MR (psi)	2,900	4,400	5,000
Resistance (R) Value	9	20	25

Seismic Considerations

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

Earthquake Forces. It is our understanding the bridge and approaches will be designed in accordance with the AASHTO publication “LRFD Bridge Design Specifications”, eighth edition (2017), with 2017 interims.

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

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

<table>
<tr><th colspan="3">Latitude 33.36692°N/Longitude 93.522740°W</th></tr>
<tr><th>Category/ Parameter</th><th>Designation/ Value</th><th>Reference</th></tr>
<tr><td>Seismic Site Class</td><td>D</td><td>AASHTO LRFD 2017 Table 3.10.3.1-1</td></tr>
<tr><td>S_S</td><td>0.119g</td><td rowspan="11">Computed using design maps provided by the USGS (http://earthquake.usgs.gov/ws/designmaps) using the indicated latitude and longitude coordinates of the project site. The USGS tool used references AASHTO 2009.</td></tr>
<tr><td>S_1</td><td>0.047g</td></tr>
<tr><td>F_a</td><td>1.600</td></tr>
<tr><td>F_v</td><td>2.400</td></tr>
<tr><td>F_{PGA}</td><td>1.600</td></tr>
<tr><td>t_s</td><td>0.619</td></tr>
<tr><td>t_0</td><td>0.124</td></tr>
<tr><td>S_{DS}</td><td>0.190g</td></tr>
<tr><td>S_{D1}</td><td>0.117g</td></tr>
<tr><td>PGA</td><td>0.051g</td></tr>
<tr><td>As</td><td>0.081g</td></tr>
</table>

Liquefaction and Dynamic Settlement. A study was performed to evaluate the liquefaction and dynamic settlement potential at the site. Both field and laboratory data were used to perform the analysis. The field measurements included the depth of the water table and the SPT N-values. The laboratory data included USCS classification and soil unit weight. An earthquake magnitude (M_W) of 7.7 with a probability of exceedance of 7% in 75 years was considered. A site peak ground acceleration of 0.081g was utilized as obtained from the referenced Seismic Design Maps. Groundwater was assumed to be at approximately El 230.

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, the potential for liquefaction at the site is relatively low.

Due to the low potential for liquefaction at the site, downdrag on piles supporting project structures has not been considered.

Approach Embankment Settlement

Based on the cross sections provided and the proposed pile cap elevations, up to 10 feet of fill will be required at the proposed abutments to bring the site to grade. Up to 6 inches of settlement is estimated to occur under the weight of new fill placed at the bridge approaches and abutments.

We recommend a settlement monitoring program be implemented and survey data be forwarded to Geotechnology so that construction can commence as soon as settlement is essentially completed.

Settlement Monitoring Program. Settlement plates, or other appropriate methods should be utilized. Settlement plates should be installed approximately 1-foot below the existing ground surface and extend in 5-foot calibrated increments as the height of fill increases. To protect the riser pipes, fill should be hand compacted within a 4-foot radius of each plate. A typical settlement plate detail is presented in Figure 3 in Appendix B. We recommend settlement plates be placed no further than 50-feet apart, with at least one in the deepest areas of fill at both abutments. The project surveyor should be retained to monitor the settlement plate riser pipe. Settlement at the site should be measured twice weekly during fill placement and weekly after filling is completed. Further construction at the abutments should not commence until after the settlement due to the fill placement is practically complete. Provided the fill is placed in accordance with the Site Preparation and Earthwork section of this report, we anticipate fill induced settlement will be practically complete approximately four weeks after the finished grade is achieved.

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

Global Stability

Based on plans provided by Garver, the abutment slopes for the existing bridge are covered in rip rap and slope 2H:1V. 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 the following table. A pseudo-static seismic acceleration of 0.041g, corresponding to one-half the peak ground acceleration (per FHWA Publication HI-99-012) was utilized. Fill material consists of cohesive soils as provided by Garver; a water elevation of El 228, as obtained from the borings, and was utilized for the short-term and seismic condition analyses and a water elevation of 249.3, as obtained from the preliminary plans from Graver, was used for the long-term condition analyses. Section profiles with calculated critical failure arcs and utilized soil parameters are presented in Appendix F for the selected analyses. The models did not consider the effect of foundation piles driven at the abutments that would provide additional restraining force to stabilize the slopes.

Table 9. Results of Slope Stability Analyses.

Location	Description	Slope Height (ft.)	Calculated Factor of Safety		
			Short-Term Static[a,c]	Long-Term Static[a,d]	Seismic[b,c]
West Abutment	2:1 8’ Fill Slope	10	2.450	1.682	1.994
Side Slope Station 210+00	3:1 Fill Slope	10	3.676	1.991	3.073
East Abutment	2:1 Fill Slope	4	2.063	1.674	1.847
Side Slope Station 215+00	3:1 Fill Slope	12	3.398	1.963	2.881

[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.

[c] Based on a groundwater elevation of El 228 as obtained by the borings.

[d] Based on a groundwater elevation of El 249.3 as obtained by the preliminary plans provided by Garver.

Deep Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2017).

It is our understanding the proposed intermediate bents will be supported using 24- or 30-inch, closed-ended, steel pipe piles and abutments (end bents) will be supported using either HP12x53 or HP14x73 H-piles. Intermediate bents have been designated as Bent 2 through Bent 6 from west to east for the analysis. Geotechnology should be notified if a different foundation type is to be considered. Synthetic profiles have been developed for the intermediate and end bent locations based upon the soil profile encountered in the borings, approximate boring elevations, and the proposed final grade. Nominal resistance curves showing the resistance due to skin friction and the total resistance (skin friction + end bearing) for the abutments and bents are presented in Appendix H. Uplift resistance (tension) may be calculated using the resistance provided by skin friction.

<u>Resistance Factors</u>. Resistance factors should be applied to the nominal resistances provided. In general, a factor of 0.45 may be used for piles in compression and 0.35 in tension. Based on AASHTO LRFD (2017) higher resistance factor may be used in accordance with the level 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 computed software WEAP. The recommended minimum hammer energies for each pile type are provided in Table 11.

Table 11. Minimum Hammer Energies.

Pile Size	Location	Embedment Length (feet)	Required Capacity (tons/kips)	Minimum Rated Hammer Energy (kip—feet)
14x73[a]	End Bents (Bent Nos. 1 and 7)	74	205 / 410	20
30”[b]	Intermediate Bents (Bent Nos. 2 through 6)	86	425 / 850	59

[a] H-Pile.

[b] Closed-ended pile with ½-inch thick walls.

<u>Static Pile Load Testing</u>. 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.

<u>Dynamic Testing of Driven Piles</u>. 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 lengths and dimensions of the foundations and the soil characteristics. The lateral resistance of pile foundations can be computed using the computer program LPILE to model the behavior of a single pile or shaft. Soil parameters are provided in Appendix 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 pile or pile group is 1.0.

Corrosion Potential. In addition to laboratory soil classification and strength testing, pH and soil resistivity testing was also conducted. The purpose of corrosion and soil resistivity testing is to provide soil data 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 12. Results of pH and Soil Resistivity Testing.

Boring	Sample No.	Sample Depth (foot)	pH	Soil Resistivity (ohm-cm)
BC-1	SS-1 – SS-4	1	4.53	7,410
BC-1	ST-6	15	4.05	12,540
BC-1	ST-8	20	--	8,550
BC-1	SS-9 – SS-12	23.5	7.32	912
BC-1	SS-16 – SS-18	58.5	5.39	2,109
BC-2	ST-5	10	3.77	--
BC-2	SS-8 – SS-11	23.5	7.42	627
BC-5	SS-4 – SS-6	18.5	6.25	5,700
BC-5	SS-7 – SS-10	33.5	5.36	855
BC-6	SS-3 – SS-5	18.5	6.40	3,135
BC-7	SS-4 – SS-6	8.5	5.31	1,368
BC-7	SS-12 – SS-14	48.5	4.29	741
BC-8	SS-9 – SS-11	33.5	3.33	1,653
BC-9	SS-8 – SS-9	28.5	5.44	1,710
BC-10	ST-3	5	--	12,540
BC-10	SS-5 – SS-8	13.5	3.73	9,690
BC-11	SS-5 – SS-6	13.5	3.81	11,970

Based on the results of the pH and soil resistivity testing and the criteria set forth in the AASHTO LRFD Bridge Design Specifications, low pH and resistivity were measured in multiple samples indicating strong corrosion or deterioration potential in the soils at the depths represented by these samples.

CHAPTER 7. RECOMMENDED ADDITIONAL SERVICES

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

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

CHAPTER 8. LIMITATIONS

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

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

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

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

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

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

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

APPENDIX A – IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

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

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

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.***

GBA
GEOPROFESSIONAL
BUSINESS
ASSOCIATION

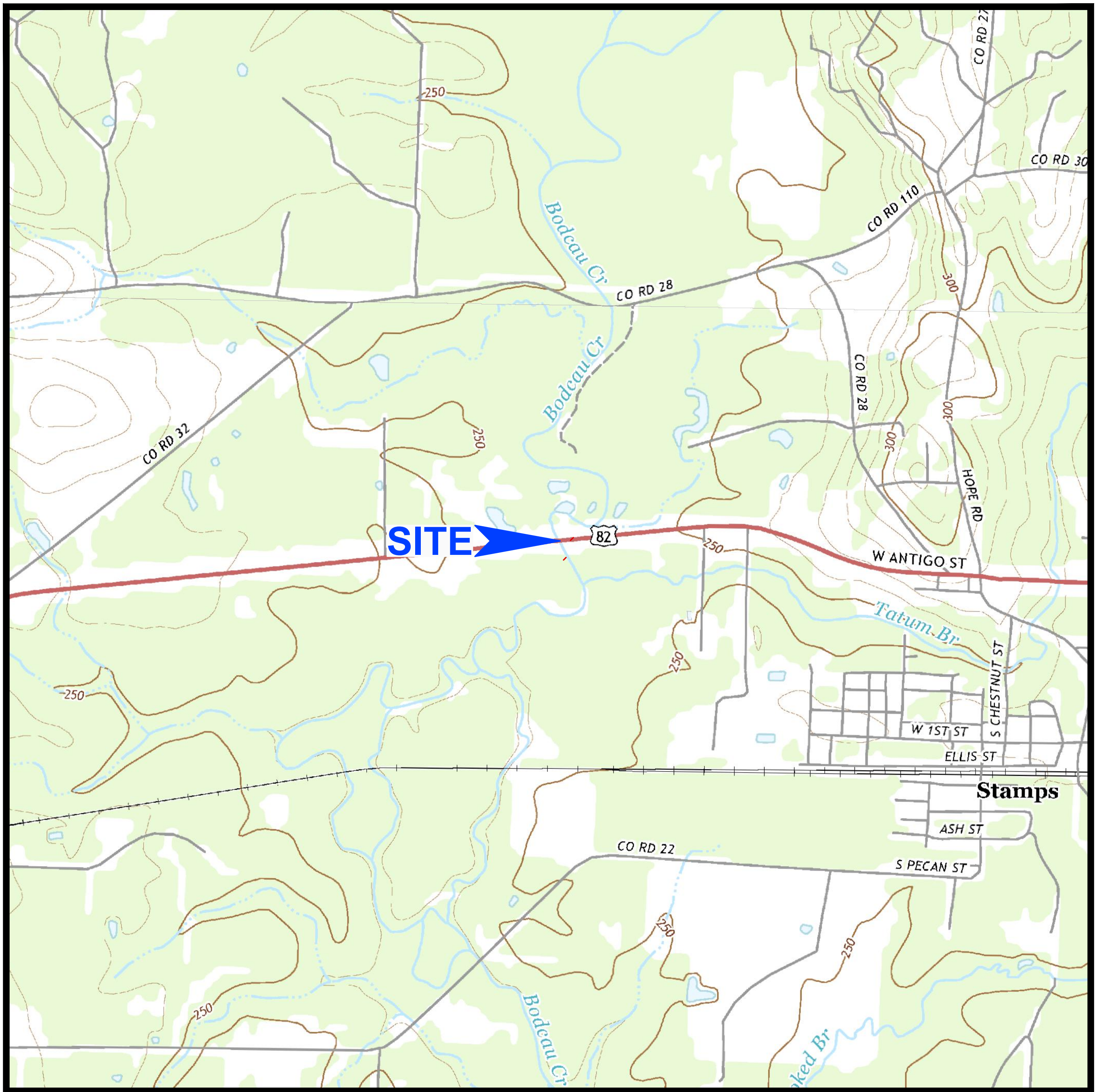
Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B – FIGURES

Figure 1 – Site Location and Topography

Figure 2 – Aerial Photograph of Site and Boring Locations

Figure 3 – Settlement Plate Detail



NOTES

1. Plan adapted from 7.5 minute U.S.G.S. maps for Lewisville and Old Town, Arkansas quadrangles, last revised in 2017.

0 2,000 4,000

SCALE IN FEET

N

Drawn By: WAH	Ck'd By: JDM	App'vd By: DM S
Date: 8-20-19	Date: 12-2-19	Date: 12-2-19

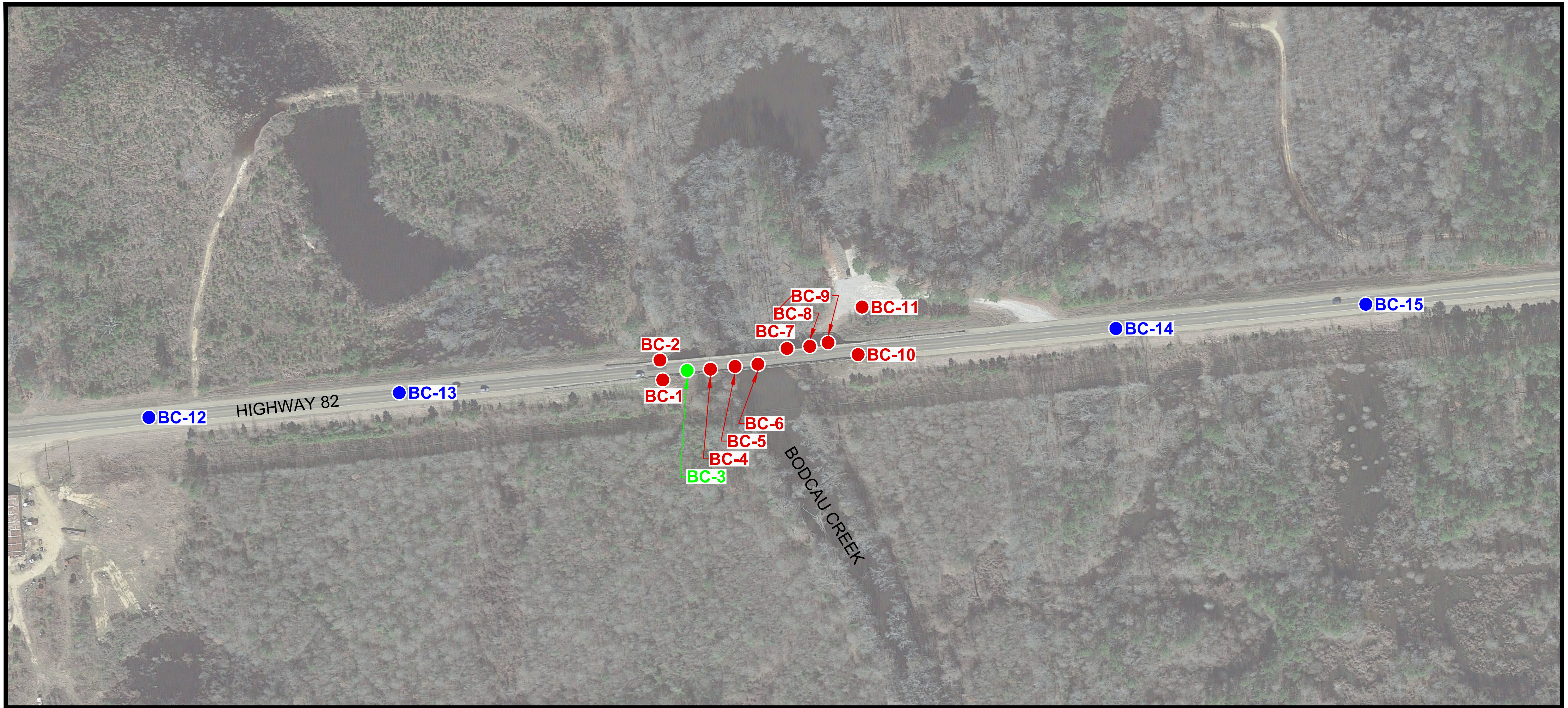
GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

SITE LOCATION AND TOPOGRAPHY

Project Number
J028499.03A

FIGURE 1



NOTES

1. Plan adapted from an January 20, 2014 aerial photograph courtesy of Google Earth.

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

3. Boring BC-3 was not drilled due to the presence of rip rap underneath the bridge and inability to access the side of the bent.

LEGEND

🔴 Boring Location (August 2019)
🔵 Boring Location (March 2019)
🟢 Boring Location not Drilled Due to Inability to Acess

0 200 400
SCALE IN FEET

N

Drawn By: WAH	Ck'd By: JDM	App'vd By: DMS
Date: 8-20-19	Date: 12-2-19	Date: 12-2-19

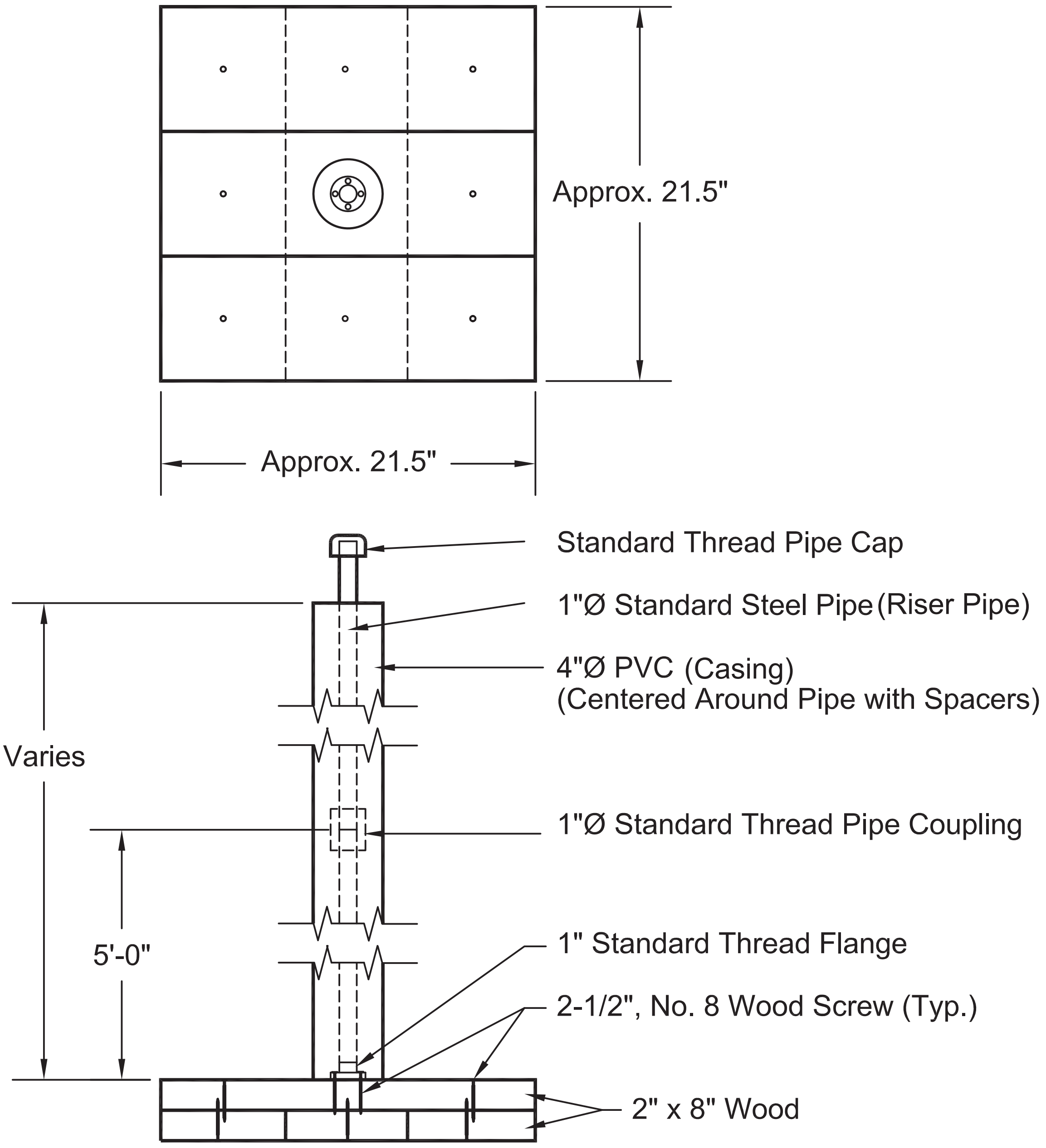
GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

AERIAL PHOTOGRAPH OF SITE AND BORING LOCATIONS

Project Number
J028499.03A

FIGURE 2



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

Drawn By: WAH	Ck'd By: FC	App'vd By: DMS
Date: 5/30/2019	Date: 5/30/2019	Date: 5/30/2019

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

SETTLEMENT PLATE DETAIL

Project Number
J028499.03

FIGURE 3

APPENDIX C – BORING INFORMATION

Boring Logs

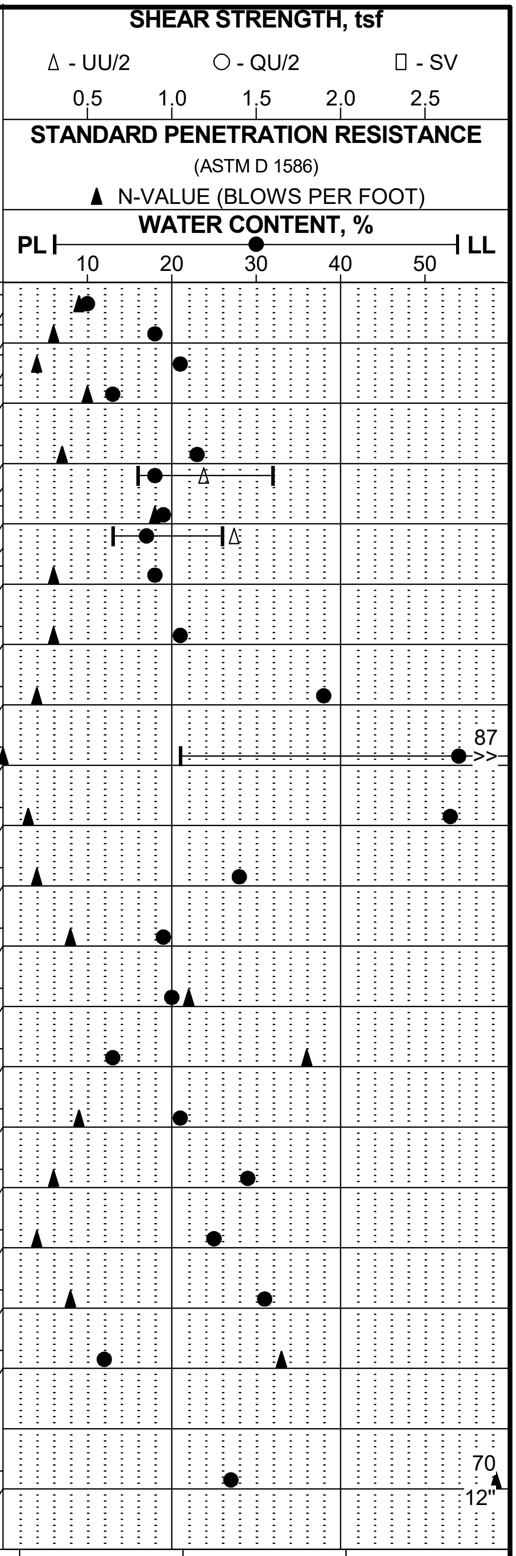
Boring Log Terms and Symbols

Surface Elevation: 256

Datum: NAVD 88

Completion Date: 8/7/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	TOPSOIL: 3 inches of grass with brown silt and trace gravel	5-5-4	SS1
5	Loose to very loose, tan and gray to gray and brown, sandy SILT - ML 59.2% passing No. 200 sieve	3-3-3	SS2
	little clay	0-1-3	SS3
10	little clay	4-5-5	SS4
15	Medium stiff to very stiff, brown and gray, sandy, LEAN CLAY - (CL)	3-3-4	SS5
	76.4% passing No. 200 sieve	113	ST6
20	56.0% passing No. 200 sieve	8-9-9	SS7
	66% passing No. 200 sieve	117	ST8
25	Medium stiff, gray to tan and gray, sandy, LEAN CLAY - CL 51.3% passing No. 200 sieve	2-3-3	SS9
30	Medium stiff to very soft, brown and red, FAT CLAY - (CH) 58.4% passing No. 200 sieve	2-3-3	SS10
35	99.5% passing No. 200 sieve	1-2-2	SS11
40		0-0-0	SS12
45	little sand	1-2-1	SS13
50	little sand	1-2-2	SS14
55	Loose to medium dense, gray, SILTY SAND - SM	7-5-3	SS15
60		7-10-12	SS16
65	Dense, gray SAND, some gravel - SP	14-20-16	SS17
70	Loose, gray, SILTY SAND - SM 28.8% passing No. 200 sieve	9-5-4	SS18
75	Loose to very loose, gray, CLAYEY SAND - SC 49.0% passing No. 200 sieve	4-3-3	SS19
80		4-2-2	SS20
85	Loose, gray, SILTY SAND - SM	2-3-5	SS21
90	Dense to very dense, tan and gray to gray SAND, some gravel - SP	14-16-17	SS22
95			
100	Boring terminated at 100 feet.	8-20 -50/6"	SS23



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
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC- 1

Project No. J028499.03

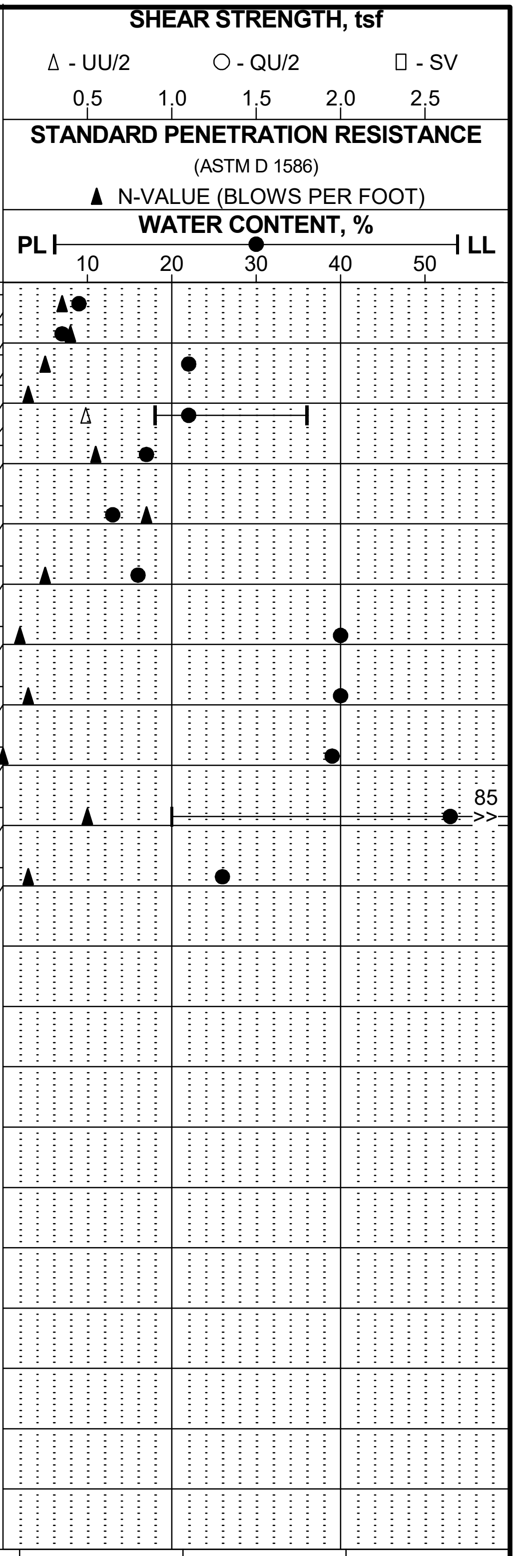
Surface Elevation: 256

Datum NAVD 88

Completion Date: 8/6/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL
	TOPSOIL: 3 inches of grass and brown silt
	Medium stiff, brown, LEAN CLAY, trace roots - CL
5	Loose, brown and gray, CLAYEY SAND, trace gravel - SC
	Medium stiff to soft, brown and gray, FAT CLAY, trace silt and sand - CH
10	Medium stiff to stiff, brown and gray, LEAN CLAY, trace sand - (CL) 84.2% passing No. 200 sieve
20	Medium dense, gray and tan, SILTY SAND - SM
25	Loose, gray, CLAYEY SAND - SC
30	Soft to very soft, gray to red, FAT CLAY - (CH) ⊽
45	Gray, CLAYEY SAND - SC
50	Soft, gray, sandy, FAT CLAY - CH
	Boring terminated at 50 feet.

DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
6-4-3	SS1
5-4-4	SS2
2-3-2	SS3
2-2-1	SS4
98	ST5
2-5-6	SS6
5-6-11	SS7
2-2-3	SS8
2-1-1	SS9
1-1-2	SS10
0-0-0	SS11
0-2-8	SS12
0-1-2	SS13



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 2002 WL J028499.03 ARDOT 030497 - BODCAU CREEK.GPJ GTINC 063001.GPJ 12/6/19

GROUNDWATER DATA

ENCOUNTERED AT 29 FEET ⊽

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM __ FEET
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC- 2

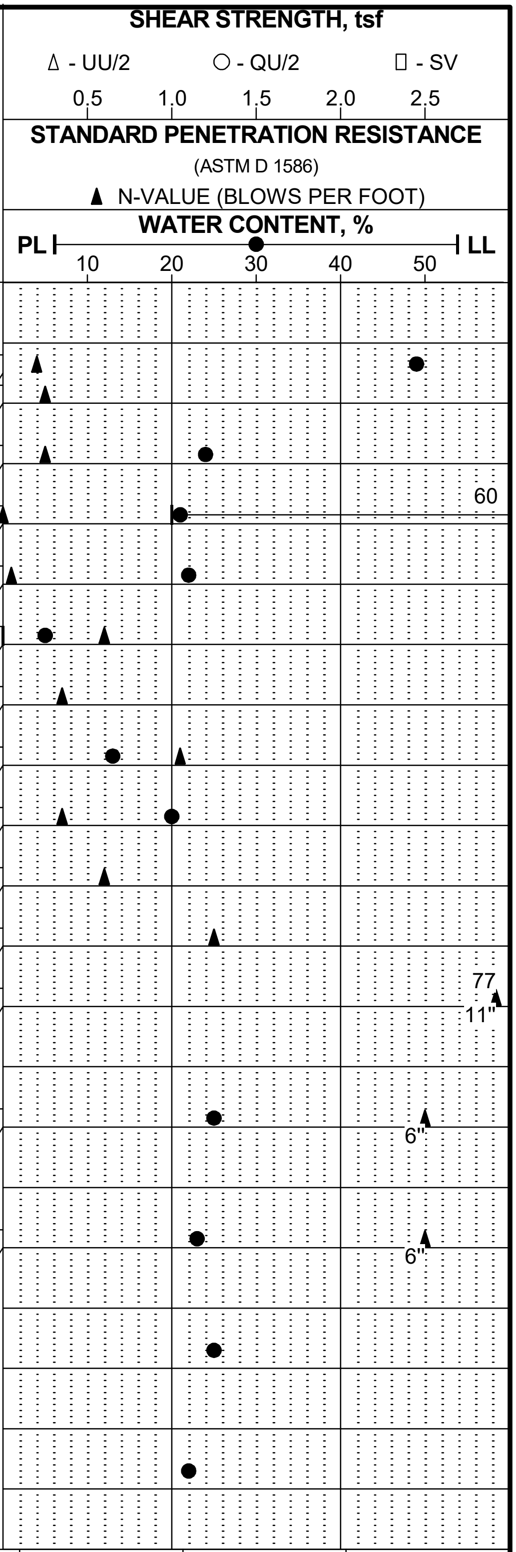
Project No. J028499.03

Surface Elevation: 237

Datum NAVD 88

Completion Date: 8/12/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Medium stiff to very soft, brown and gray to brown, FAT CLAY - (CH)			
5				
			1-1-3	SS1
10			1-2-3	SS2
15			1-2-3	SS3
20	trace silt		0-0-0	SS4
25	Very loose to medium dense, gray, SILTY SAND with clay - SM 47.1% passing No. 200 sieve		0-0-1	SS5
30	little clay		4-6-6	SS6
35	Loose, gray SAND with silt - SP-SM 11.8% passing No. 200 sieve		4-4-3	SS7
40	Medium dense, gray SAND, trace gravel		10-9-12	SS8
45	Loose, gray, SILTY SAND - SM		6-3-4	SS9
50	Medium dense, gray, CLAYEY SAND, trace silt - SC		4-4-8	SS10
55	Medium dense to very dense, gray, SILTY SAND - SM trace clay		9-9-16	SS11
60			14-27 -50/5"	SS12
65				
70			25-50/6"	SS13
75				
80	Very dense, gray SAND with silt - SP-SM Boring terminated at 80 feet.		28-50/6"	SS14
85				
90				
95				
100				



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM

WASHBORING FROM 0 FEET

BMF DRILLER JDM LOGGER

CME 550X DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 92 %

REMARKS: Boring drilled through approximately 8-inch asphalt and concrete bridge deck located approximately 20 feet above ground surface and 5 feet into creek bed.

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC- 4

Project No. J028499.03

Surface Elevation: 227

Datum NAVD 88

Completion Date: 8/11/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
0	Soft, brown to gray, FAT CLAY - CH		
7.5		0-1-1	SS1
10		0-0-2	SS2
15	little silt	0-1-2	SS3
20	Loose, gray and brown to gray, SILTY SAND - SM	3-3-5	SS4
25		5-4-5	SS5
30	Medium dense to loose, tan, gray and black GRAVEL, trace sand - GP	6-13-16	SS6
35		3-4-8	SS7
40		1-1-6	SS8
45	Medium dense to very dense, gray, SILTY SAND - SM	8-13-17	SS9
50		17-28-42	SS10
55		22-40 -50/6"	SS11
60		28-50/5"	SS12
70		27-50/6"	SS13
80	trace clay	20-50/5"	SS14
90		34-50/6"	SS15
100	Very dense, gray SAND with silt - SP-SM	50/6"	SS16
	Boring terminated at 100 feet.		

SHEAR STRENGTH, tsf

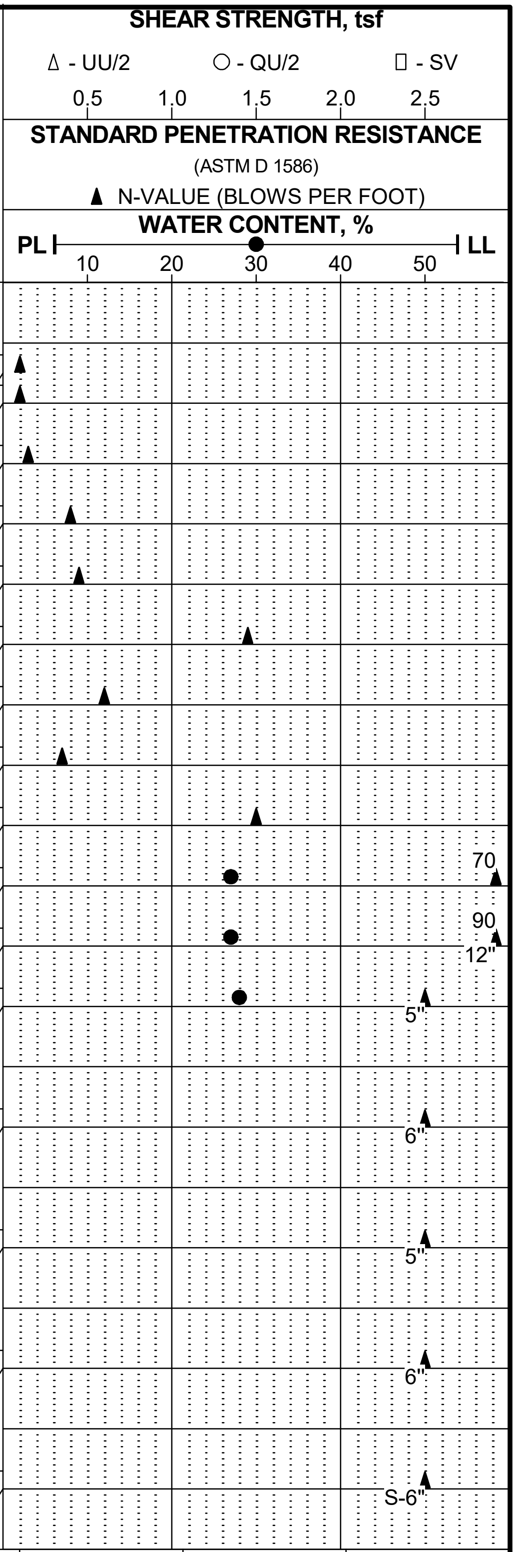
Δ - UU/2 ○ - QU/2 ▯ - SV

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

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 3/4 HOLLOW STEM

WASHBORING FROM 0 FEET

BMF DRILLER JDM LOGGER

CME 550X DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 92 %

REMARKS: Boring drilled through approximately 8-inch asphalt and concrete bridge deck located approximately 30 feet above ground surface and 5 feet into creek bed.

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC- 5

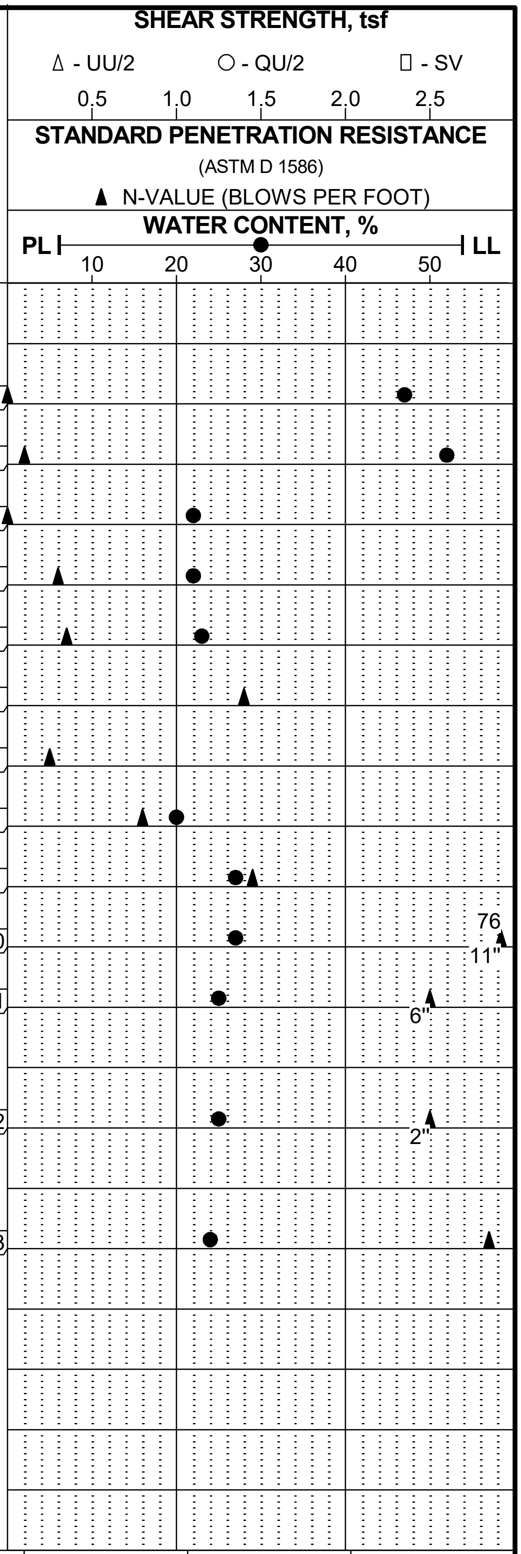
Project No. J028499.03

Surface Elevation: 226

Datum NAVD 88

Completion Date: 8/10/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Very soft to soft, gray and brown to gray, FAT CLAY - CH		
5			
10		0-0-0	SS1
15	sandy	0-0-2	SS2
20	Very soft to medium stiff, gray, sandy SILT - ML 79.1% passing No. 200 sieve	1-0-0	SS3
25		2-2-4	SS4
30		2-3-4	SS5
35	Medium dense, gray and tan SAND, little gravel - SP	10-12-16	SS6
40	Loose, gray and tan GRAVEL, trace sand - GP	2-2-3	SS7
45	Stiff to very stiff, gray, sandy, FAT CLAY - CH	6-7-9	SS8
50		8-12-17	SS9
55	Very dense, gray, SILTY SAND - SM	14-26 -50/5"	SS10
60		20-50/6"	SS11
65			
70		38-50/2"	SS12
75			
80	Very dense, gray, CLAYEY SAND, trace gravel - SC Boring terminated at 80 feet.	17-27-30	SS13
85			
90			
95			
100			



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM

WASHBORING FROM 0 FEET

BMF DRILLER JDM LOGGER

CME 550X DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 92 %

REMARKS: Boring drilled through approximately 8-inch asphalt and concrete bridge deck located approximately 35 feet above ground surface and approximately 8 feet into creek bed.

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC- 6

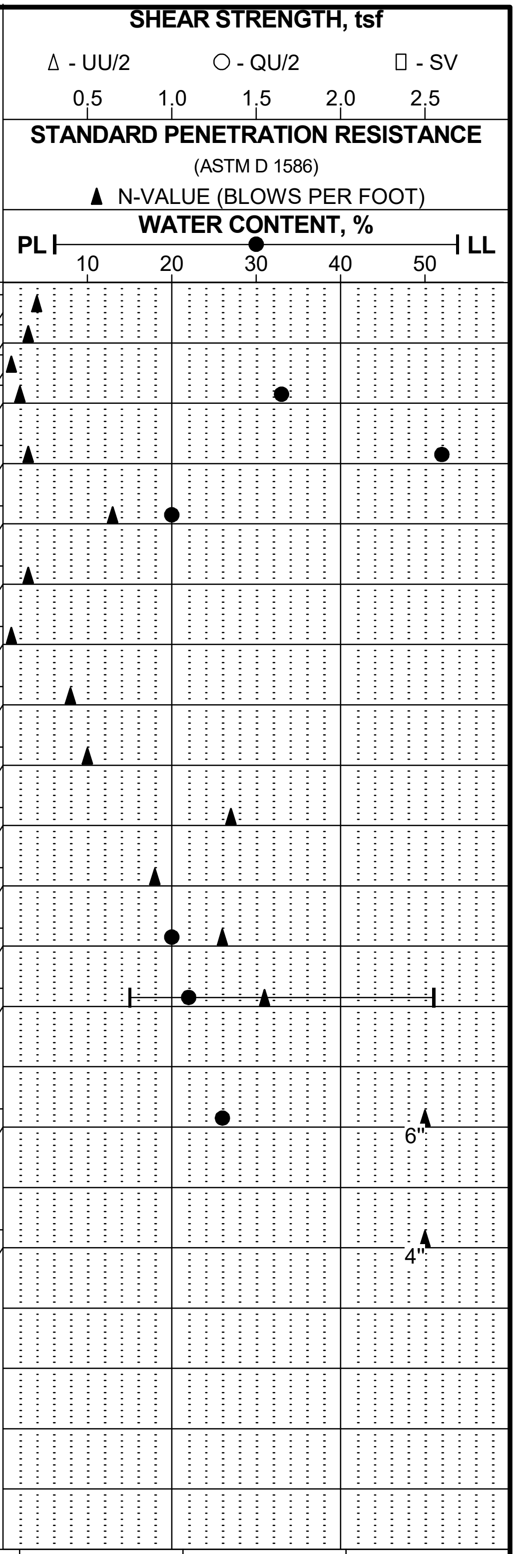
Project No. J028499.03

Surface Elevation: 235

Datum NAVD 88

Completion Date: 8/6/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Gray GRAVEL with sand - GP			
	Very loose, black SAND and gravel, trace roots - SP		1-1-3	SS1
5			2-2-1	SS2
		⊽	2-0-1	SS3
10	Very loose, gray, CLAYEY SAND - SC		0-1-1	SS4
15			0-0-3	SS5
20	Stiff to soft, gray, sandy, FAT CLAY - CH		6-6-7	SS6
25			1-1-2	SS7
30	Very loose, gray, CLAYEY SAND - SC		0-0-1	SS8
35	Loose, gray, SILTY SAND - SM		3-3-5	SS9
40			4-6-4	SS10
45	Medium dense, gray SAND, trace gravel - SP		14-13-14	SS11
50	Very stiff to hard, brown and gray to gray and brown, FAT CLAY, some sand - (CH)		9-9-9	SS12
55			8-12-14	SS13
60			10-12-19	SS14
65				
70	Very dense, gray and black, SILTY SAND - SM		36-50/6"	SS15
75				
80	Boring terminated at 80 feet.		28-50/4"	SS16
85				
90				
95				
100				



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

LOG OF BORING 2002 WL J028499.03 ARDOT 030497 - BODCAU CREEK.GPJ GTINC 0638301.GPJ 12/6/19

GROUNDWATER DATA

ENCOUNTERED AT 7 FEET ⊽

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 10 FEET
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC- 7

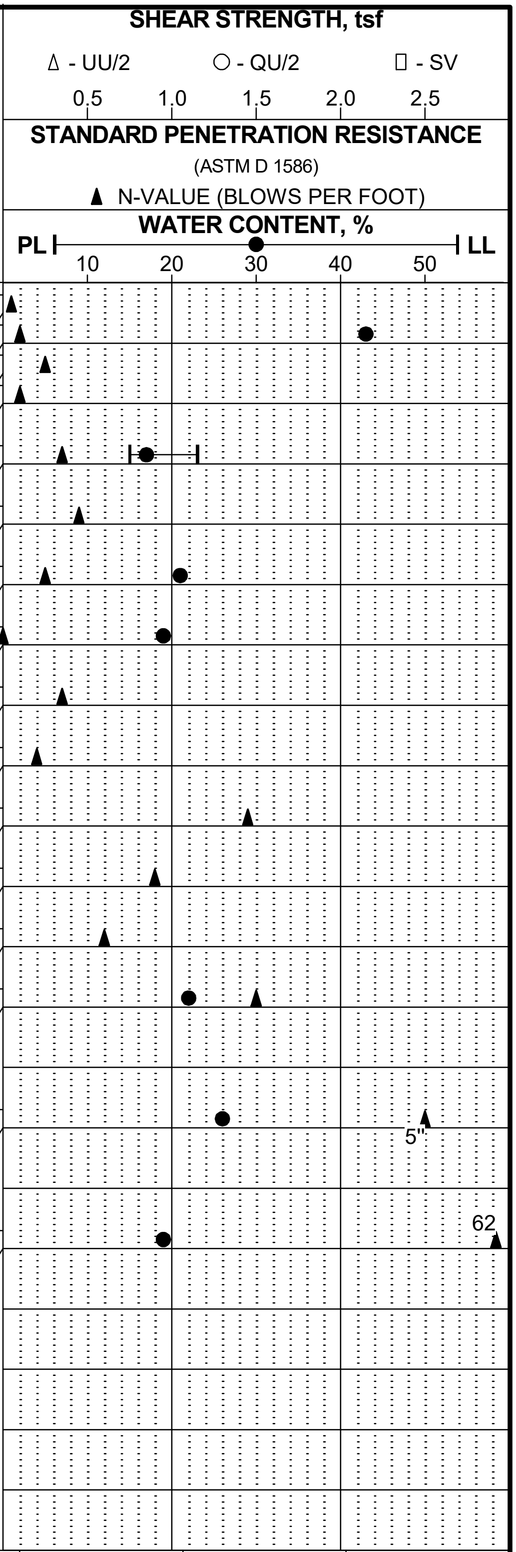
Project No. J028499.03

Surface Elevation: 237

Completion Date: 8/8/19

Datum NAVD 88

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Gray GRAVEL with sand - GP	1-1-0	SS1
5	Very loose to loose, brown and gray to brown, CLAYEY SAND - SC	0-1-1	SS2
	wood debris	1-2-3	SS3
10	wood debris	1-1-1	SS4
	trace gravel		
15	Medium stiff, gray, sandy, LEAN CLAY - (CL)	1-2-5	SS5
20	Loose, gray, SILTY SAND, some clay - SM	2-4-5	SS6
25	trace tree roots	6-4-1	SS7
30	Very soft, gray, sandy, FAT CLAY, trace silt - CH 58.4% passing No. 200 sieve	0-0-0	SS8
35	Loose to very loose, gray, SILTY SAND - SM 18.3% passing No. 200 sieve	2-3-4	SS9
40		2-2-2	SS10
45	Medium dense, gray and black to tan and gray SAND, little gravel - SP	4-8-21	SS11
50		6-8-10	SS12
55	Stiff to hard, gray, FAT CLAY - CH little silt, sand and gravel	1-6-6	SS13
60	some silt	10-12-18	SS14
65			
70	Very dense, gray, SILTY SAND - SM	16-50/5"	SS15
75			
80	Very dense, gray, CLAYEY SAND - SC Boring terminated at 80 feet.	21-26-36	SS16
85			
90			
95			
100			



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 10 FEET
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC- 8

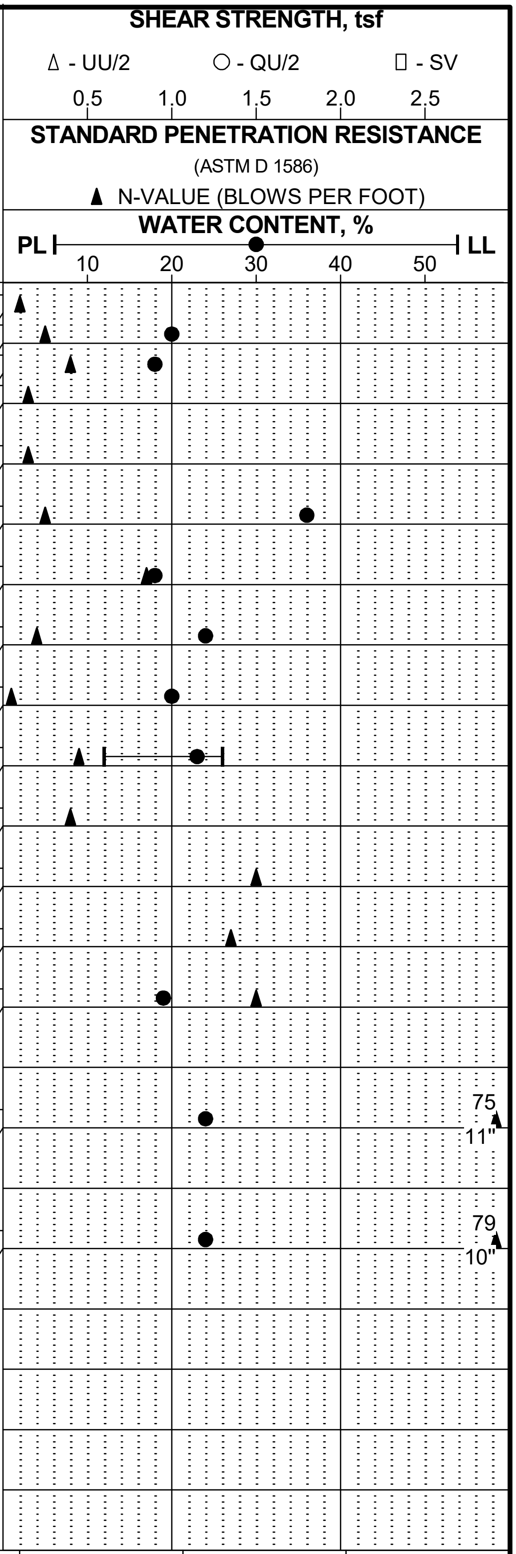
Project No. J028499.03

Surface Elevation: 241

Completion Date: 8/8/19

Datum NAVD 88

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	TOPSOIL: 3 inches of grass with brown silt			
	Loose to very loose, gray and brown to orange and gray, SILTY SAND - SM		1-1-1	SS1
5			1-2-3	SS2
			3-3-5	SS3
10	▽		1-1-2	SS4
15			1-2-1	SS5
20	Loose, gray, CLAYEY SAND - SC		1-2-3	SS6
25	Medium dense, gray, SILTY SAND, trace clay - SM		7-8-9	SS7
30	Soft to very soft, gray, sandy, FAT CLAY - CH		1-2-2	SS8
35			0-0-1	SS9
40	Stiff, gray, sandy, LEAN CLAY - (CL)		1-4-5	SS10
45	Loose, gray, SILTY SAND - SM		3-3-5	SS11
50	Medium dense, gray SAND with silt, trace gravel - SP-SM		14-16-14	SS12
55	Medium dense, tan and gray SAND, some gravel - SP		10-12-15	SS13
60	Hard, gray, silty, FAT CLAY, little sand - CH		12-13-17	SS14
65				
70	Very dense, gray, SILTY SAND - SM trace clay		16-25 -50/5"	SS15
75				
80	Boring terminated at 80 feet.		25-29 -50/4"	SS16
85				
90				
95				
100				



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

GROUNDWATER DATA

ENCOUNTERED AT 9 FEET ▽

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 10 FEET
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC- 9

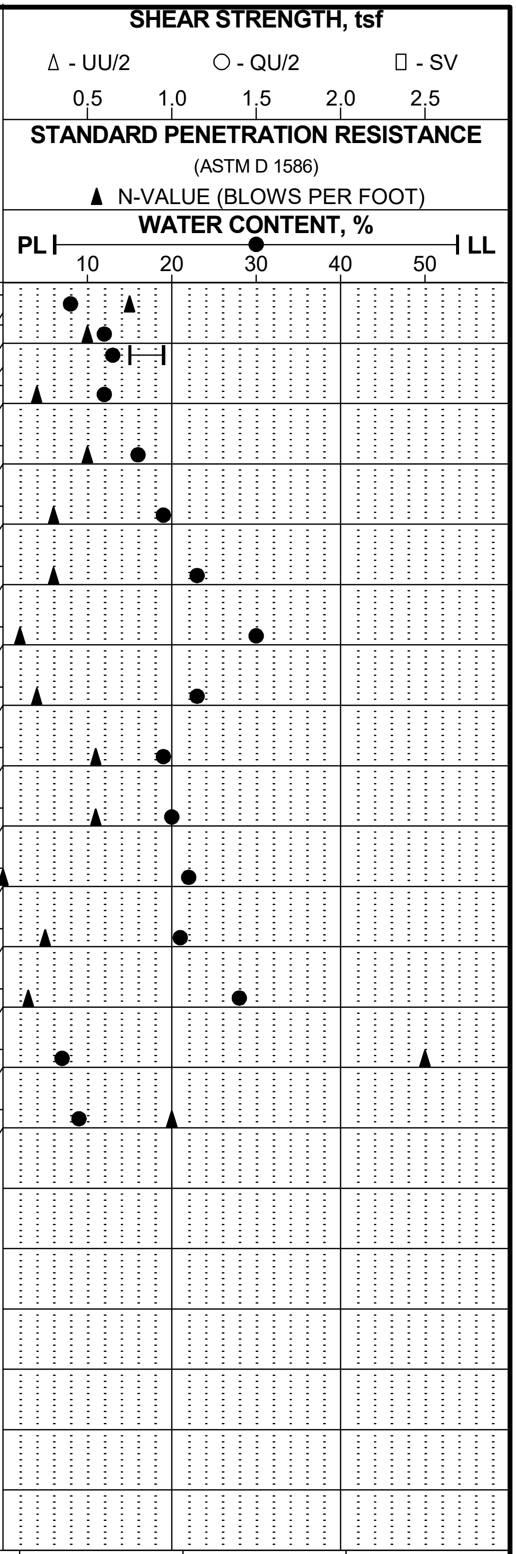
Project No. J028499.03

Surface Elevation: 256

Datum NAVD 88

Completion Date: 8/9/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	ASPHALT: 6 inches		
	Base Material: Brown and gray silt with sand and trace gravel	10-8-7	SS1
5	Medium dense, brown SAND with clay, trace gravel - SP-SC	3-4-6	SS2
			ST3
10	Loose, brown, SILTY, CLAYEY SAND, trace gravel - (SC-SM) 28.8% passing No. 200 sieve	2-2-2	SS4
15	Very loose, brown, gray and orange SAND with clay - SP-SC	3-4-6	SS5
	Loose to very loose, gray and tan to orange and gray, SILTY SAND - SM		
20		4-3-3	SS6
25		3-3-3	SS7
30	little clay	1-1-1	SS8
35	Soft to stiff, orange and gray, sandy SILT - ML 56.2% passing No. 200 sieve	1-1-3	SS9
40		2-7-4	SS10
45		4-5-6	SS11
50	Very soft, gray, silty, FAT CLAY - CH	0-0-0	SS12
55	Loose, gray, CLAYEY SAND - SC	2-2-3	SS13
60	Very loose, gray, SILTY SAND, trace clay - SM 38.5% passing No. 200 sieve	2-1-2	SS14
65	Very dense to medium dense, tan and gray SAND, little gravel - SP	24-28-22	SS15
70	Boring terminated at 70 feet.	7-10-10	SS16
75			
80			
85			
90			
95			
100			



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 10 FEET
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

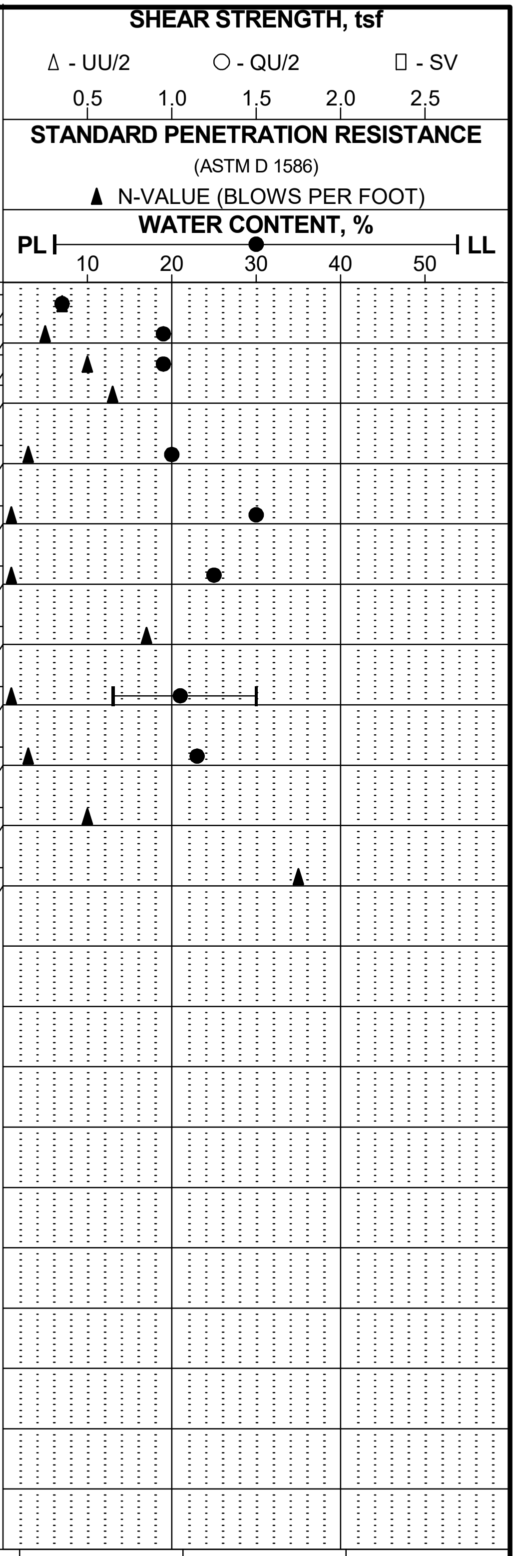
LOG OF BORING: BC-10

Project No. J028499.03

Surface Elevation: 244 Completion Date: 8/6/19

Datum NAVD 88

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Fill: 2 inches of white and gray gravel and sand			
	Loose, gray GRAVEL with sand - GP		3-4-3	SS1
5	Loose, gray, CLAYEY SAND - SC		2-2-3	SS2
	Loose to medium dense, gray and tan, SILTY SAND - SM ▽		3-5-5	SS3
10			2-5-8	SS4
15	Soft to very soft, orange and gray, sandy SILT, little clay - ML 12.4% passing No. 200 sieve		1-1-2	SS5
20			0-0-1	SS6
25	Very soft, gray, sandy, FAT CLAY - CH		0-1-0	SS7
30	Medium dense, gray SAND - SP		6-9-8	SS8
35	Very soft to soft, gray, sandy, LEAN CLAY - (CL)		1-0-1	SS9
40			0-1-2	SS10
45	Loose, gray SAND with clay - SP-SC		6-6-4	SS11
50	Dense, gray SAND - SP		15-17-18	SS12
	Boring terminated at 50 feet.			
55				
60				
65				
70				
75				
80				
85				
90				
95				
100				



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

GROUNDWATER DATA

ENCOUNTERED AT 9 FEET ▽

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 15 FEET
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC-11

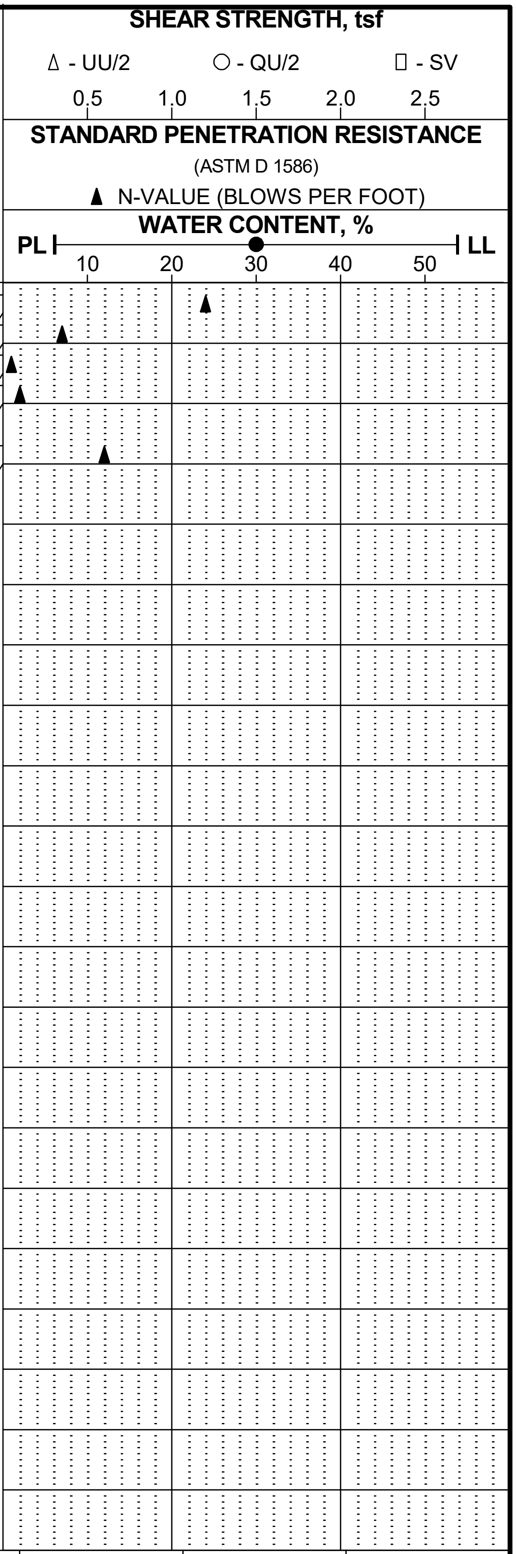
Project No. J028499.03

Surface Elevation: 256

Datum: NAVD 88

Completion Date: 3/14/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	ASPHALT: 3.5 inches			
	Base Material: 8.5 inches of black sand, trace gravel		10-6-18	SS1
5	Very stiff to medium stiff, gray SILT - ML trace gravel		8-3-4	SS2
			2-0-1	SS3
10	Very soft to soft, gray, LEAN CLAY - CL trace roots		1-1-1	SS4
15	Stiff, gray and white SILT, trace clay - ML		3-4-8	SS5
	Boring terminated at 15 feet.			



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM __ FEET
BF DRILLER JDM LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 90 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 3/18/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC-12

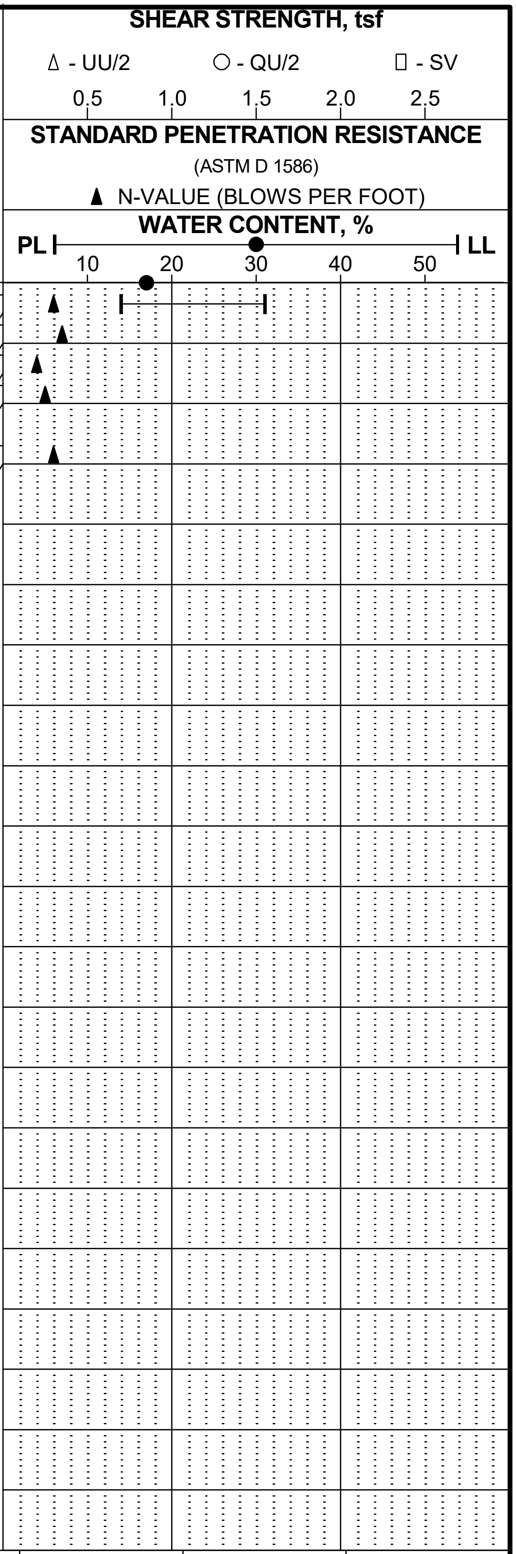
Project No. J028499.03

Surface Elevation: 256

Datum NAVD 88

Completion Date: 3/14/19

DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
ASPHALT: 2 inches			
Base Material: 10 inches of black sand, trace gravel		2-3-3	SS1
Medium stiff, brown and gray, sandy, LEAN CLAY, trace gravel - (CL) 68.7% passing No. 200 sieve		1-3-4 2-2-2	SS2 SS3
Loose to very loose, gray, CLAYEY SAND, little silt - SC		1-2-3	SS4
Medium stiff, gray to gray and tan SILT - ML trace clay		2-2-4	SS5
Boring terminated at 15 feet.			



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 2002 WL J028499.03 ARDOT 030497 - BODCAU CREEK.GPJ GTINC 063001.GDT 12/6/19

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM

WASHBORING FROM __ FEET

BF DRILLER JDM LOGGER

CME 55 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 90 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 3/18/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC-13

Project No. J028499.03

Surface Elevation: 256

Datum NAVD 88

Completion Date: 3/14/19

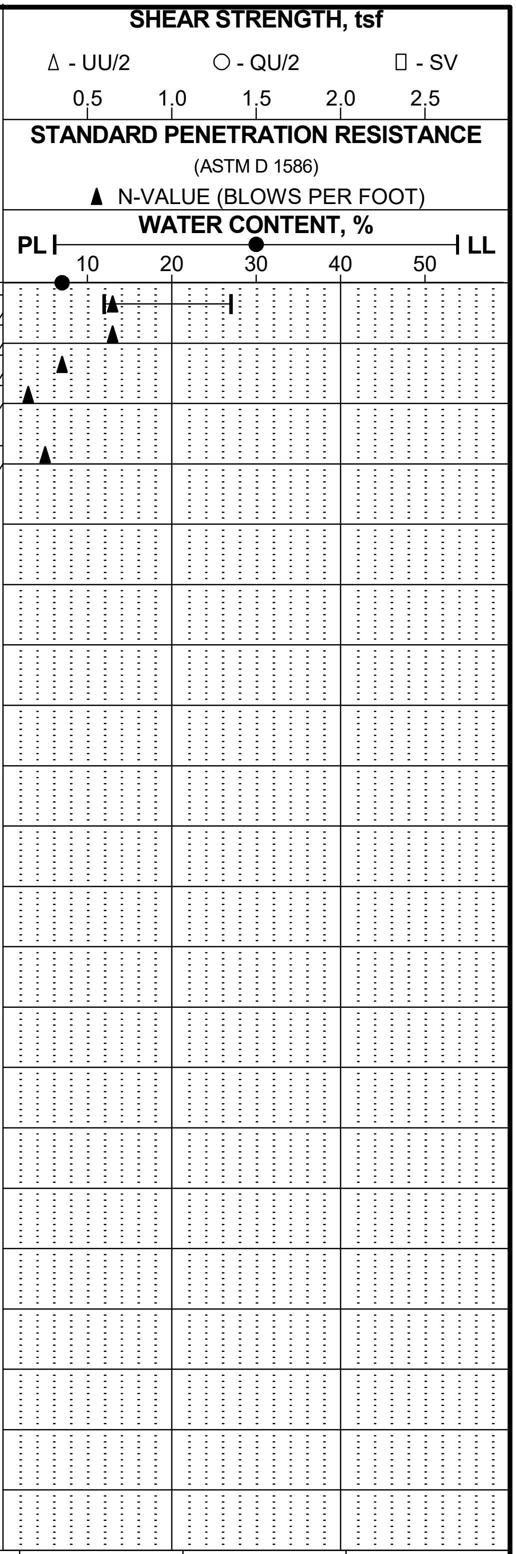
DESCRIPTION OF MATERIAL

DEPTH IN FEET | GRAPHIC LOG | DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD | SAMPLES

- ASPHALT: 2.5 inches
- Base Material: 9.5 inches of black sand with silt, trace gravel
- Medium dense, brown and gray, CLAYEY SAND, little gravel - (SC)
- 23.5% passing No. 200 sieve
- Medium dense, gray, SILTY SAND - SM
- Loose to very loose, gray and brown to gray, CLAYEY SAND - SC
- Loose, gray, SILTY SAND - SM

Boring terminated at 15 feet.

DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
7-7-6	SS1
6-6-7	SS2
2-2-5	SS3
1-1-2	SS4
2-3-2	SS5



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 2002 WL J028499.03 ARDOT 030497 - BODCAU CREEK.GPJ GTINC 0638301.GPJ 12/6/19

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM

WASHBORING FROM __ FEET

BF DRILLER JDM LOGGER

CME 55 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 90 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 3/18/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

LOG OF BORING: BC-14

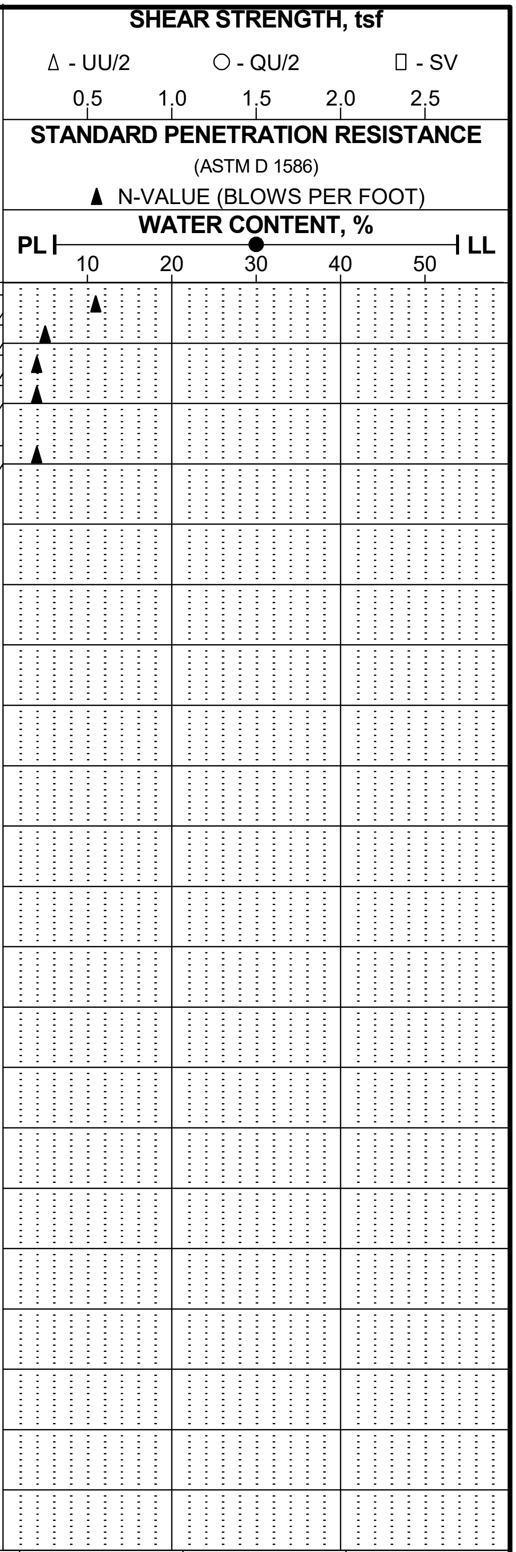
Project No. J028499.03

Surface Elevation: 256

Datum NAVD 88

Completion Date: 3/14/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	ASPHALT: 10 inches			
	Base Material: Black sand, trace gravel and silt		3-4-7	SS1
5	Loose, gray and brown, CLAYEY SAND - SC		2-2-3	SS2
	Soft, gray, sandy, LEAN CLAY - CL		2-1-3	SS3
10	Soft, gray SILT - ML		2-1-3	SS4
15	Very loose, gray, SILTY SAND, trace clay - SM		1-2-2	SS5
	Boring terminated at 15 feet.			
20				
25				
30				
35				
40				
45				
50				
55				
60				
65				
70				
75				
80				
85				
90				
95				
100				



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

LOG OF BORING 2002 WL J028499.03 ARDOT 030497 - BODCAU CREEK.GPJ GTINC 063801.GPJ 12/6/19

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM __ FEET
BF DRILLER JDM LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 90 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 3/18/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas

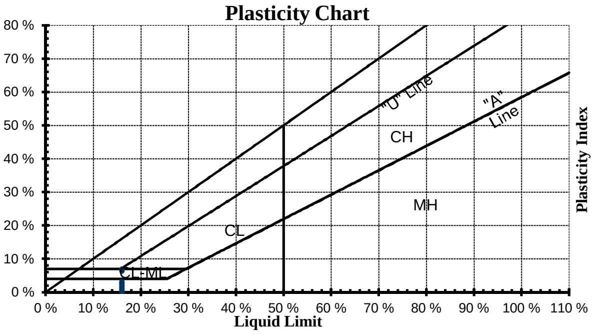
LOG OF BORING: BC-15

Project No. J028499.03

BORING LOG: TERMS AND SYMBOLS

LEGEND

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



SOIL GRAIN SIZE

US STANDARD SIEVE

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

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

300 76.2 19.1 4.76 2.00 0.42 0.074 0.005

SOIL GRAIN SIZE IN MILLIMETERS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

STRENGTH OF COHESIVE SOILS

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

DENSITY OF GRANULAR SOILS

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

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

RELATIVE COMPOSITION

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

OTHER TERMS

Layer - Inclusion greater than 3 inches thick.

Seam - Inclusion 1/8-inch to 3 inches thick

Parting - Inclusion less than 1/8-inch thick

Pocket - Inclusion of material that is smaller than sample diameter

GEOTECHNOLOGY INC
FROM THE GROUND UP

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

APPENDIX D – LABORATORY TEST DATA

Atterberg Limits

Grain Size Distributions

Unconsolidated-Undrained Triaxial Compression

One-Dimensional Consolidation

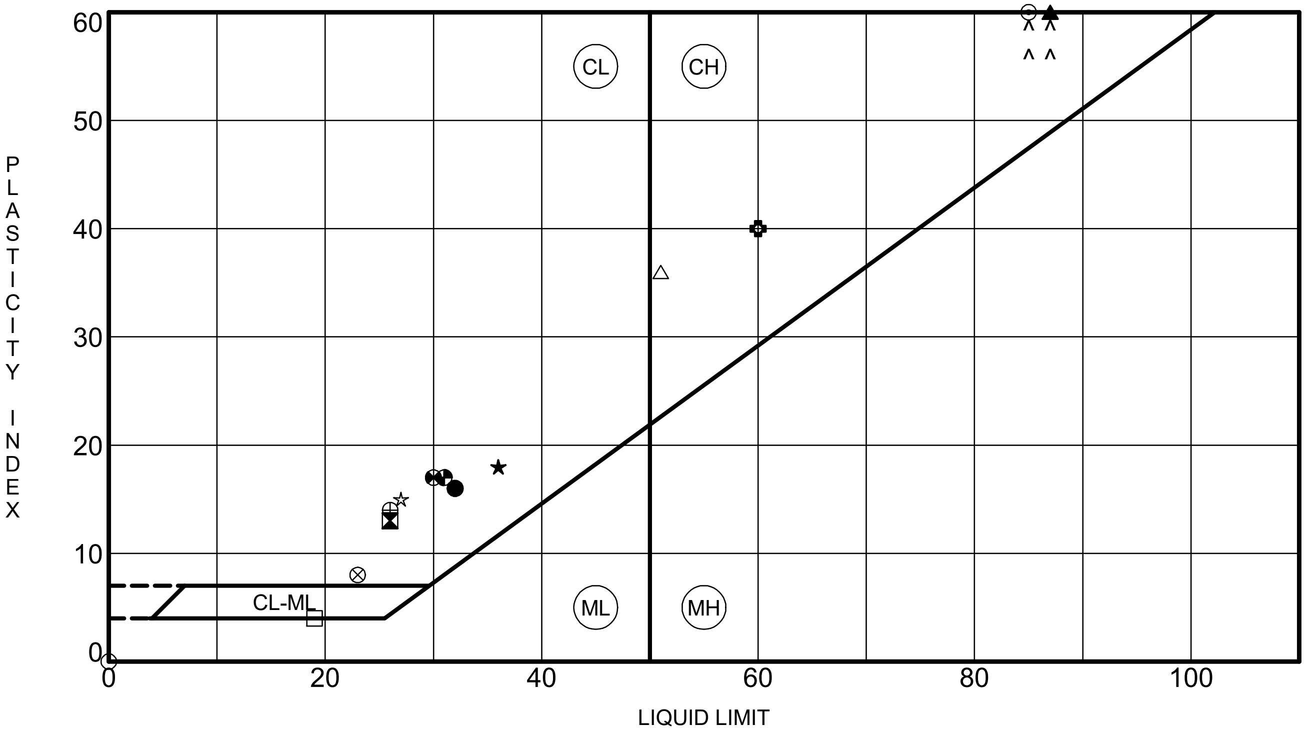
Direct Shear

Resistivity

pH

Standard Proctor Curves

CBR Results



	Specimen Identification		LL	PL	PI	Fines	Classification
●	BC- 1	15.0	32	16	16	76	LEAN CLAY(CL)
⧗	BC- 1	20.0	26	13	13	56	SANDY LEAN CLAY(CL)
▲	BC- 1	38.5	87	21	66		FAT CLAY(CH)
★	BC- 2	10.0	36	18	18	84	LEAN CLAY(CL)
⊙	BC- 2	43.5	85	20	65		FAT CLAY(CH)
✪	BC- 4	18.5	60	20	40		FAT CLAY(CH)
○	BC- 4	28.5	NP	NP	NP		SILTY SAND(SM)
△	BC- 7	58.5	51	15	36		FAT CLAY(CH)
⊗	BC- 8	13.5	23	15	8		SANDY LEAN CLAY(CL)
⊕	BC- 9	38.5	26	12	14		SANDY LEAN CLAY(CL)
□	BC-10	5.0	19	15	4	29	SILTY, CLAYEY SAND(SC-SM)
⊛	BC-11	33.5	30	13	17		SANDY LEAN CLAY(CL)
◕	BC-13	1.0	31	14	17	69	LEAN CLAY(CL), A-6(9)
☆	BC-14	1.0	27	12	15	24	CLAYEY SAND(SC),A-2-6(0)

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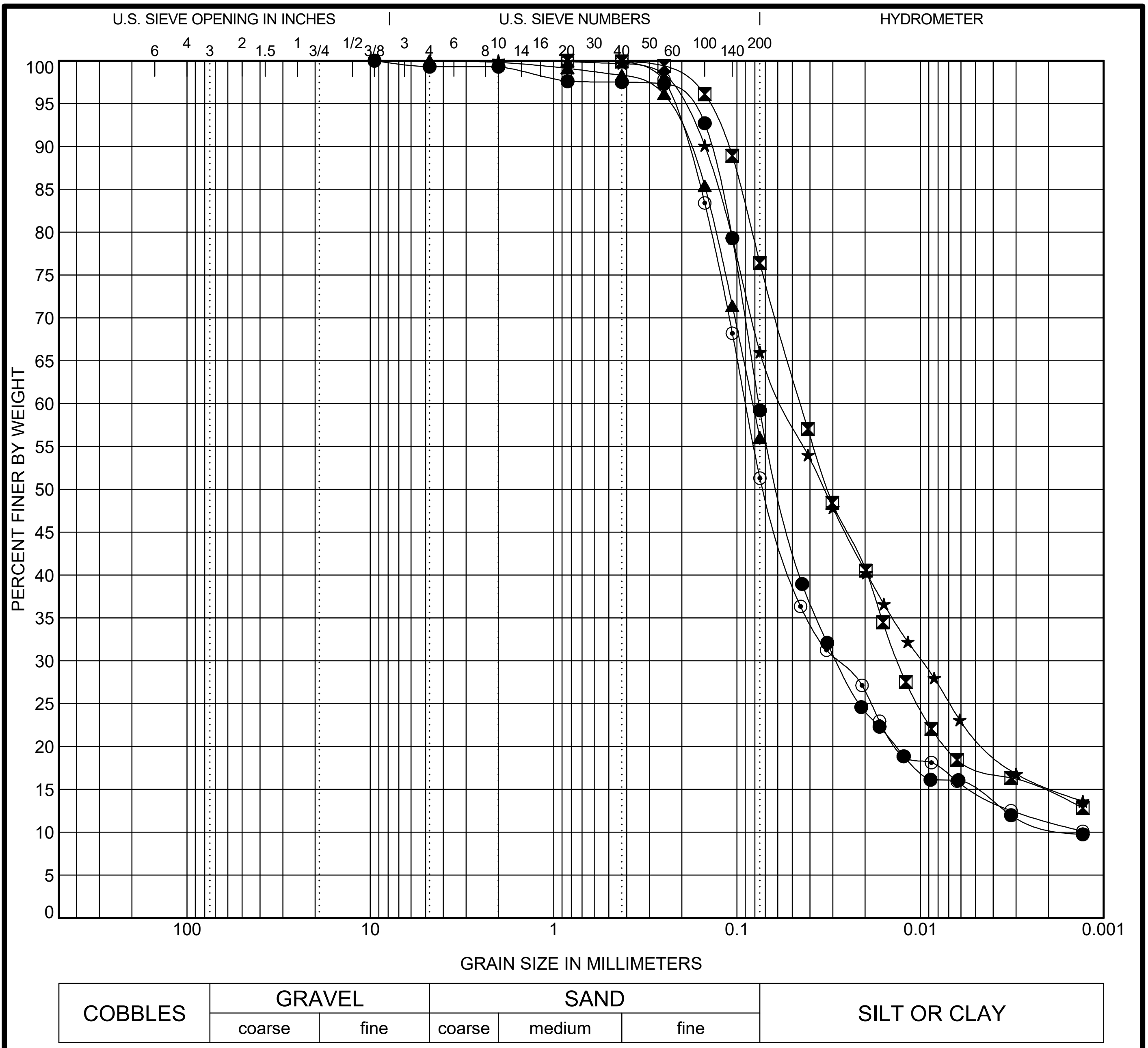
FROM THE GROUND UP

ATTERBERG LIMITS RESULTS

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas
J028499.03



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**BC- 1**	**1.0**	**SANDY SILT(ML)**				**7.47**	**52.89**
⧗	**BC- 1**	**15.0**	**LEAN CLAY(CL)**	**32**	**16**	**16**		
▲	**BC- 1**	**20.0**	**SANDY LEAN CLAY(CL)**	**26**	**13**	**13**		
★	**BC- 1**	**20.1**	**SANDY SILT(ML)**					
⊙	**BC- 1**	**23.5**	**SANDY FAT CLAY(CH)**					

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BC- 1**	**1.0**	**9.5**	**0.076**	**0.029**	**0.001**	**0.7**	**40.1**	**44.5**	**14.7**
⧗	**BC- 1**	**15.0**	**0.84**	**0.045**	**0.013**		**0.0**	**23.6**	**58.7**	**17.7**
▲	**BC- 1**	**20.0**	**4.75**	**0.082**			**0.0**	**44.0**	**56.0**	
★	**BC- 1**	**20.1**	**2**	**0.055**	**0.01**		**0.0**	**34.0**	**44.7**	**21.3**
⊙	**BC- 1**	**23.5**	**0.84**	**0.09**	**0.028**		**0.0**	**48.7**	**36.5**	**14.8**

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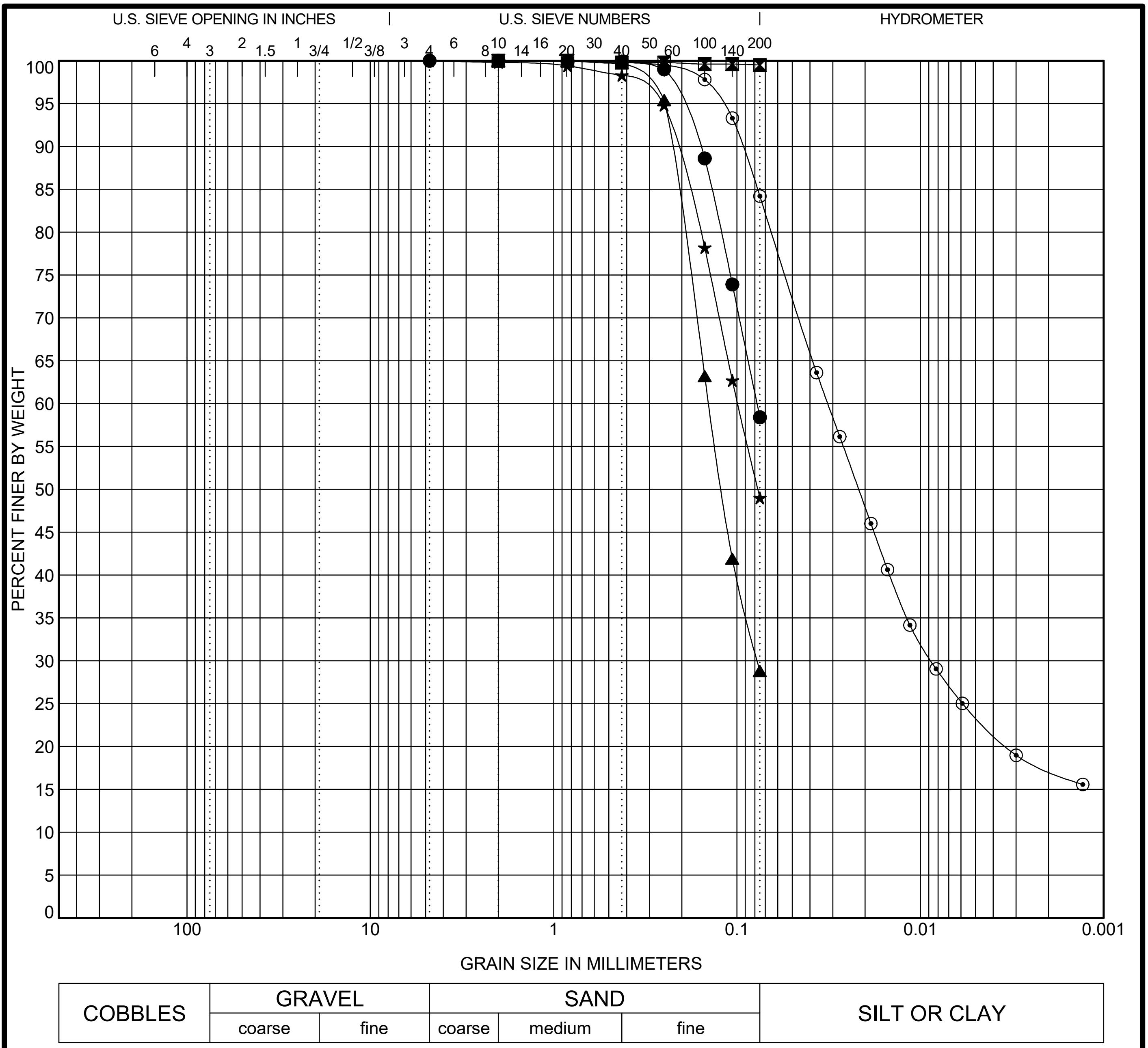
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas
J028499.03



	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**BC- 1**	**28.5**	**SANDY FAT CLAY(CH)**					
⧗	**BC- 1**	**33.5**	**FAT CLAY(CH)**					
▲	**BC- 1**	**68.5**	**SILTY SAND(SM)**					
★	**BC- 1**	**73.5**	**CLAYEY SAND(SC)**					
⊙	**BC- 2**	**10.0**	**LEAN CLAY(CL)**	**36**	**18**	**18**		

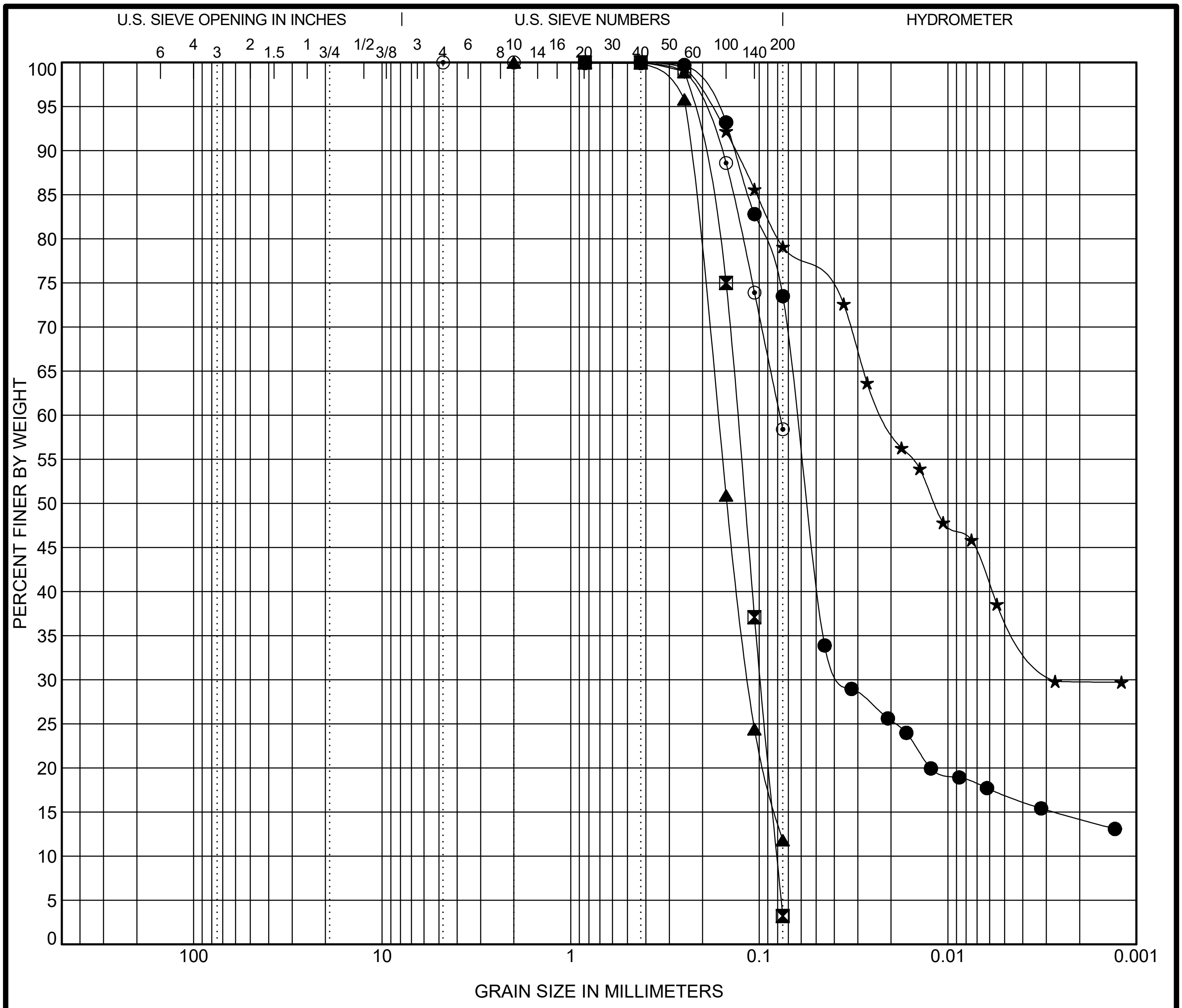
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BC- 1**	**28.5**	**4.75**	**0.078**			**0.0**	**41.6**	**58.4**	
⧗	**BC- 1**	**33.5**	**2**				**0.0**	**0.5**	**99.5**	
▲	**BC- 1**	**68.5**	**2**	**0.142**	**0.077**		**0.0**	**71.2**	**28.8**	
★	**BC- 1**	**73.5**	**4.75**	**0.099**			**0.0**	**51.0**	**49.0**	
⊙	**BC- 2**	**10.0**	**2**	**0.032**	**0.009**		**0.0**	**15.8**	**60.7**	**23.5**

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas
J028499.03

US GRAIN SIZE J028499.03 ARDOT 030497 - BODCAU CREEK.GPJ US LAB.GDT 12/3/19



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**BC- 4**	**23.5**	**SILTY SAND(SM)**					
⊠	**BC- 4**	**23.6**	**POORLY GRADED SAND(SP)**				**0.92**	**1.63**
▲	**BC- 4**	**33.5**	**POORLY GRADED SAND with SILT(SP-SM)**				**1.10**	**2.33**
★	**BC- 6**	**18.5**	**SANDY SILT(ML)**					
⊙	**BC- 8**	**28.5**	**SANDY FAT CLAY(CH)**					

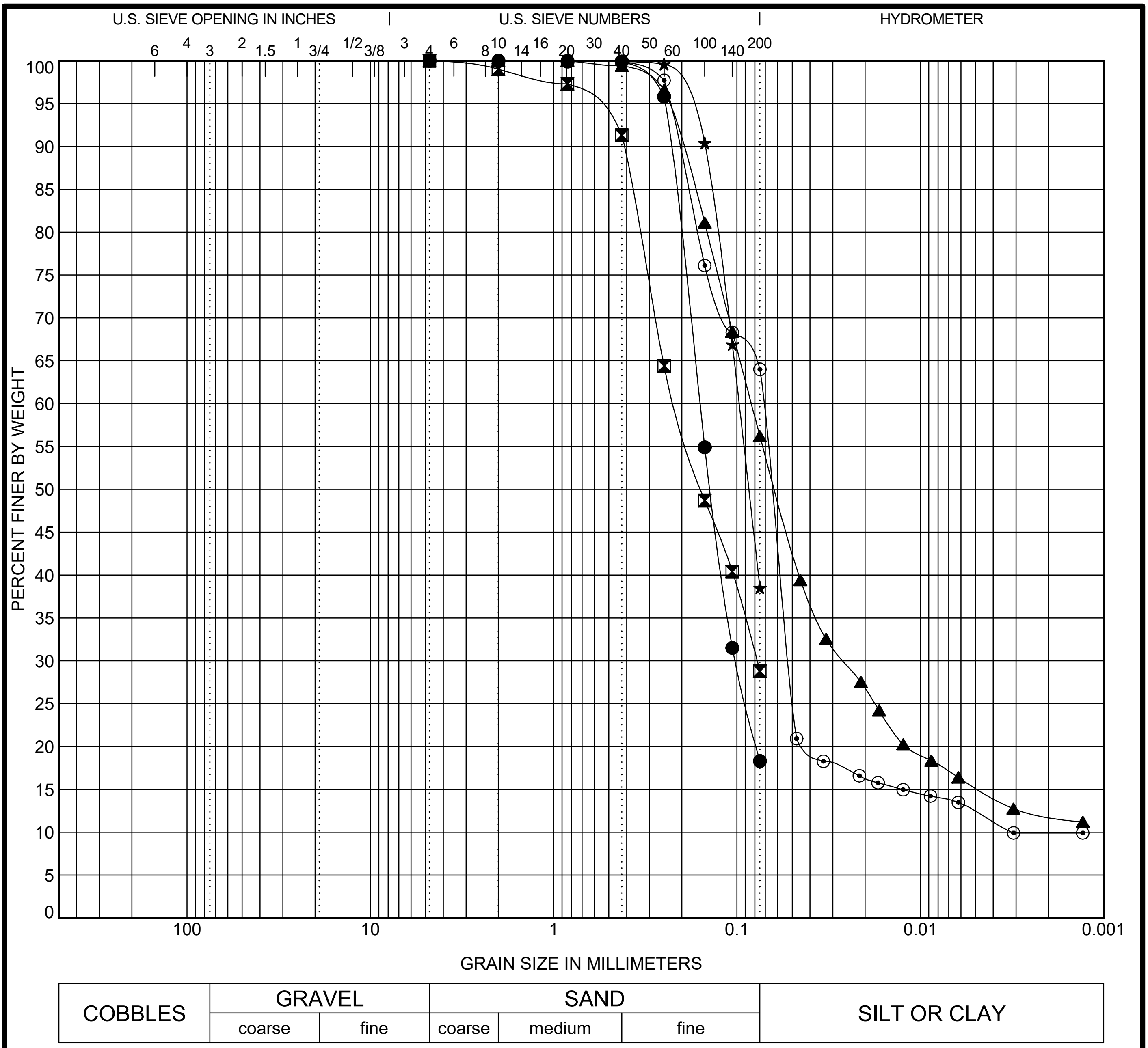
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BC- 4**	**23.5**	**0.84**	**0.063**	**0.035**		**0.0**	**26.5**	**56.5**	**17.0**
⊠	**BC- 4**	**23.6**	**0.84**	**0.131**	**0.099**	**0.08**	**0.0**	**96.8**	**3.2**	
▲	**BC- 4**	**33.5**	**2**	**0.166**	**0.114**		**0.0**	**88.2**	**11.8**	
★	**BC- 6**	**18.5**	**0.84**	**0.022**	**0.003**		**0.0**	**20.9**	**41.7**	**37.4**
⊙	**BC- 8**	**28.5**	**4.75**	**0.078**			**0.0**	**41.6**	**58.4**	

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas
J028499.03

US GRAIN SIZE J028499.03 ARDOT 030497 - BODCAU CREEK.GPJ US LAB.GDT 12/3/19



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**BC- 8**	**33.5**	**SILTY SAND(SM)**					
⧗	**BC-10**	**5.0**	**SILTY, CLAYEY SAND(SC-SM)**	**19**	**15**	**4**		
▲	**BC-10**	**33.5**	**SANDY SILT(ML)**					
★	**BC-10**	**58.5**	**SILTY SAND(SM)**					
⊙	**BC-11**	**13.5**	**SILTY SAND(ML)**				**11.99**	**22.79**

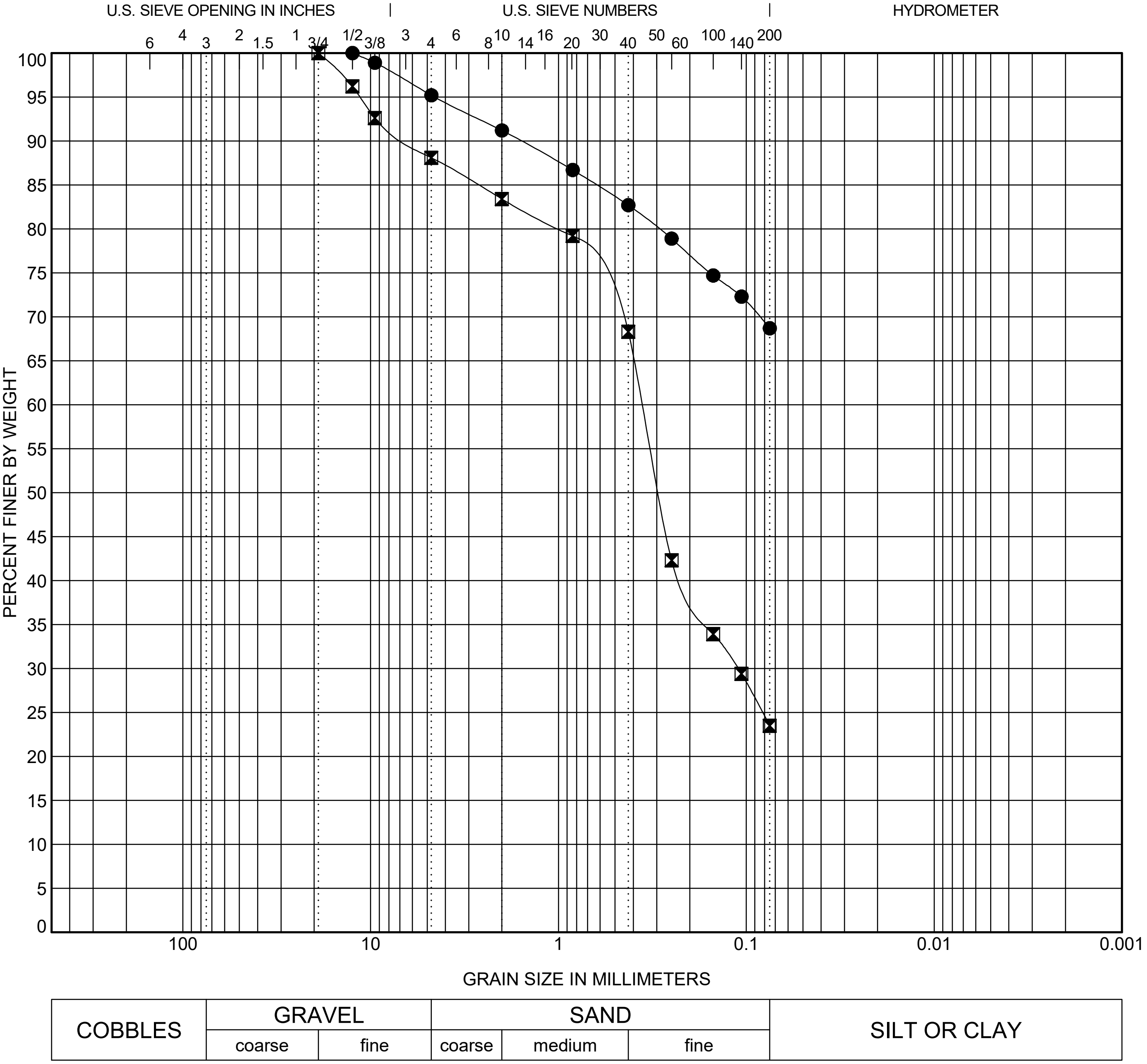
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BC- 8**	**33.5**	**4.75**	**0.16**	**0.102**		**0.0**	**81.7**	**18.3**	
⧗	**BC-10**	**5.0**	**4.75**	**0.217**	**0.078**		**0.0**	**71.2**	**28.8**	
▲	**BC-10**	**33.5**	**2**	**0.084**	**0.026**		**0.0**	**43.8**	**40.9**	**15.3**
★	**BC-10**	**58.5**	**2**	**0.097**			**0.0**	**61.5**	**38.5**	
⊙	**BC-11**	**13.5**	**2**	**0.072**	**0.052**	**0.003**	**0.0**	**36.0**	**51.6**	**12.4**

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

Bodcau Creek Bridge Replacement
Lafayette County, Arkansas
J028499.03

US GRAIN SIZE J028499.03 ARDOT 030497 - BODCAU CREEK.GPJ US LAB.GDT 12/3/19



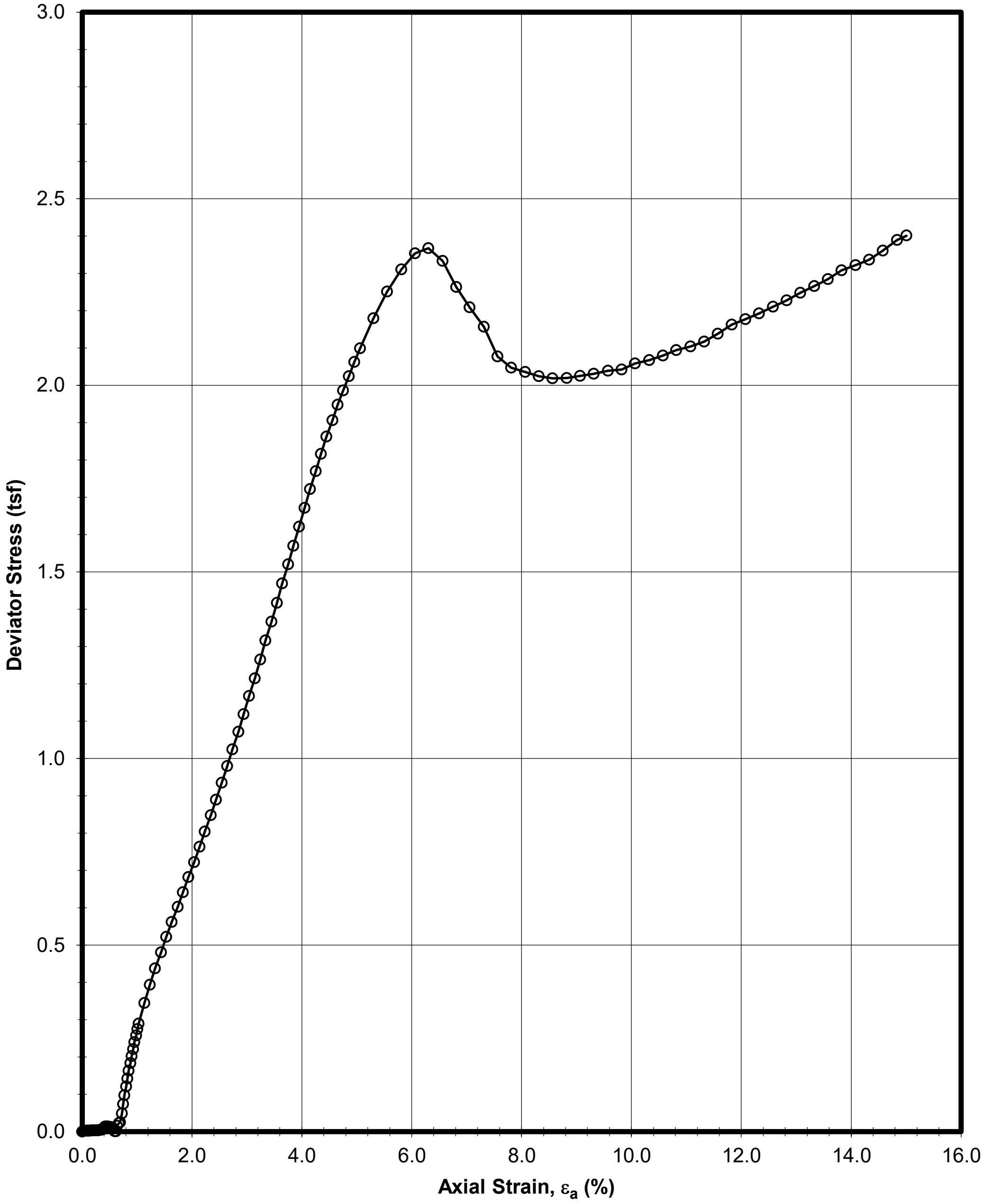
Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**BC-13**	**1.0**	**LEAN CLAY(CL), A-6(9)**	**31**	**14**	**17**		
☒	**BC-14**	**1.0**	**CLAYEY SAND(SC),A-2-6(0)**	**27**	**12**	**15**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BC-13**	**1.0**	**12.5**				**4.8**	**26.5**	**68.7**	
☒	**BC-14**	**1.0**	**19**	**0.359**	**0.111**		**11.9**	**64.6**	**23.5**	

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

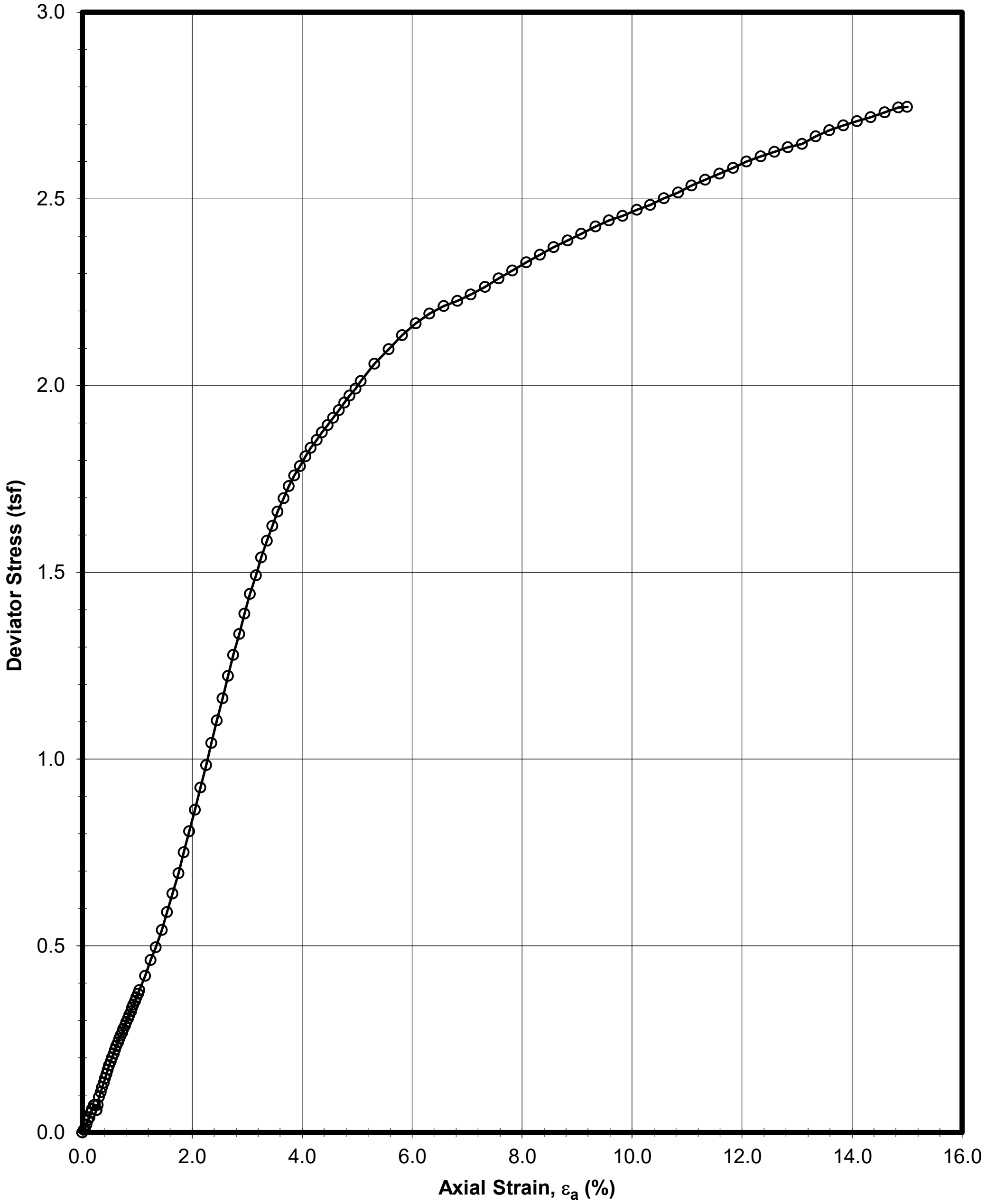
Bodcau Creek Bridge Replacement
Lafayette County, Arkansas
J028499.03



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

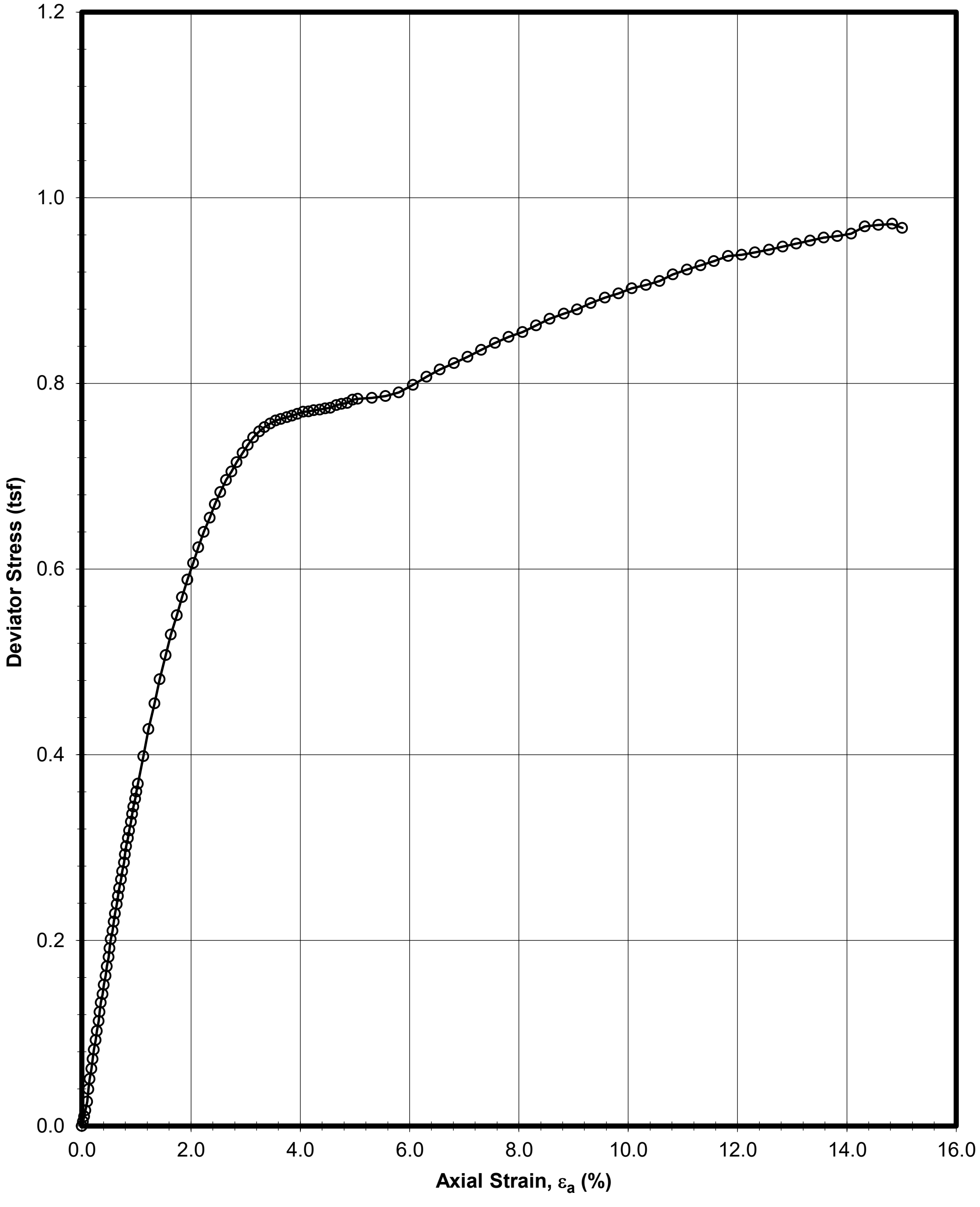
ASTM D 2850
Project No.: J028499.03
Boring: BC-1
Sample: ST-6 - Depth: 15 ft.

GEOTECHNOLOGY INC
FROM THE GROUND UP



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J028499.03
Boring: BC-1
Sample: ST-8 - Depth: 20 ft.



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J028499.03
Boring: BC-2
Sample: ST-5 - Depth: 10 ft.

GEOTECHNOLOGY INC
FROM THE GROUND UP

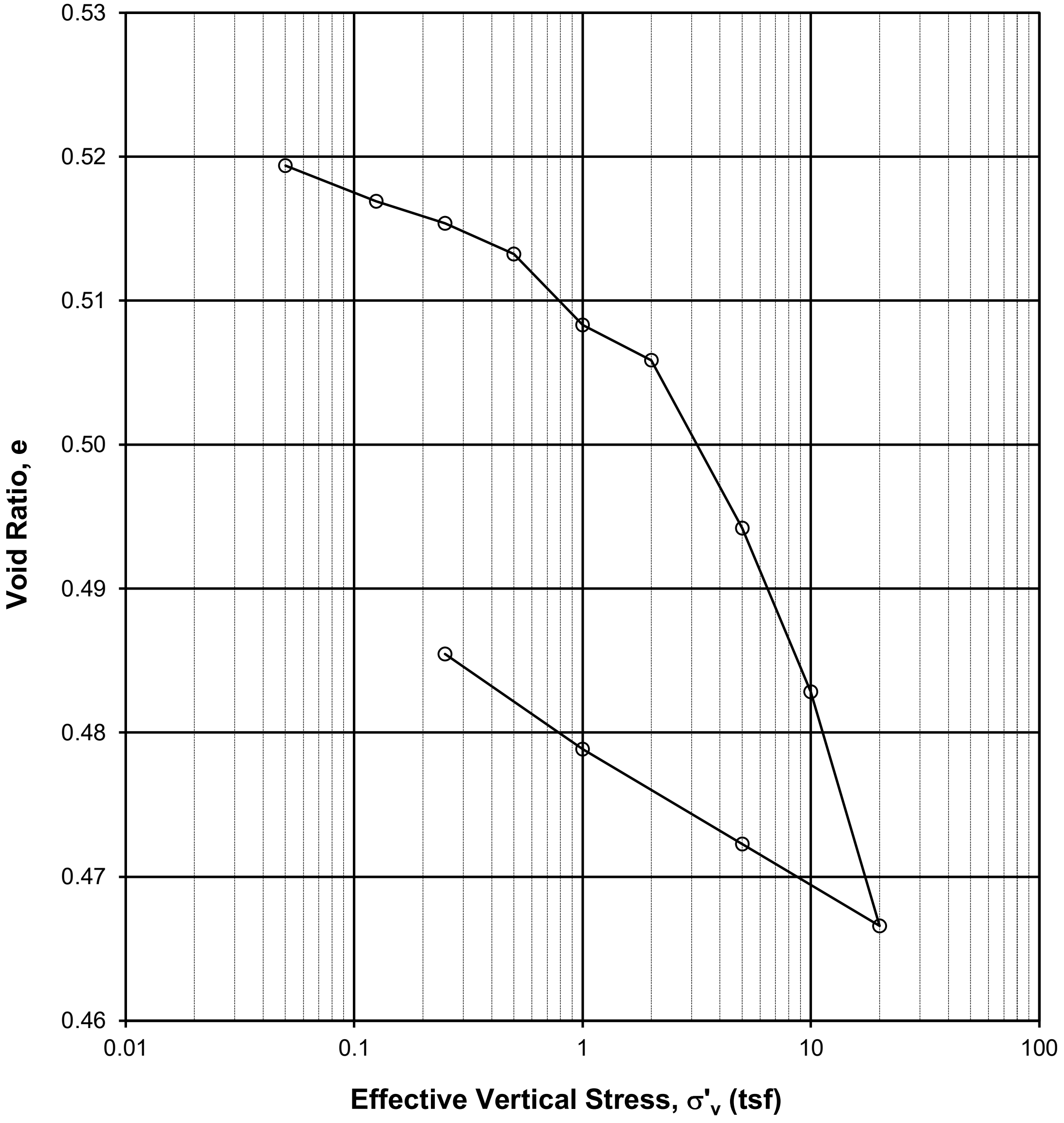
Liquid Limit= 32 Plastic Limit= 16 Plasticity Index = 16 USCS: CL

Compression Index, C_c = 0.04

Recompression Index, C_r = 0.01

Void Ratio, e_o = 0.519

Preconsolidation Pressure = 2.75 tsf

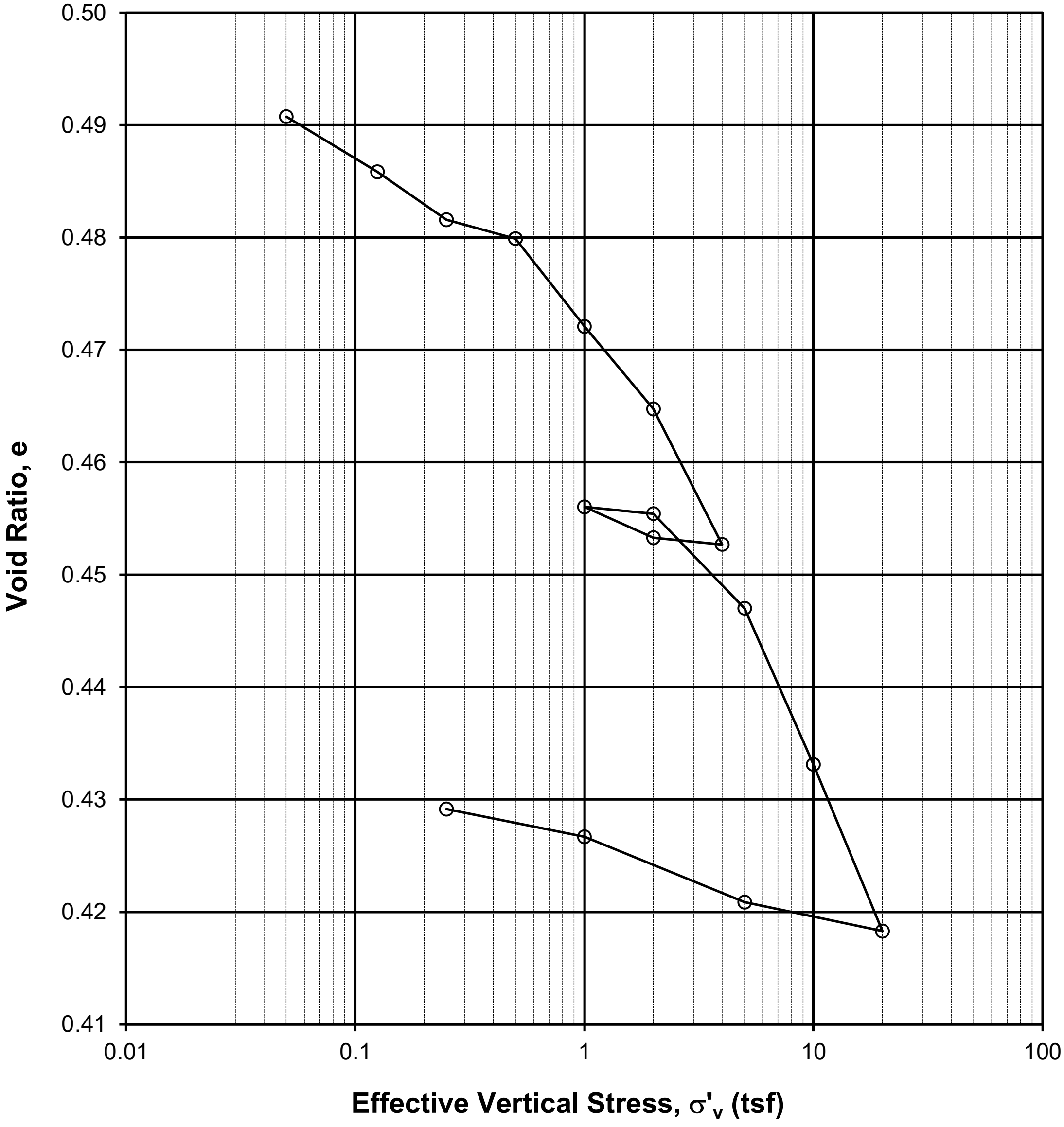


1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435
Project No.: J028499.03
Boring: BC-1
Sample: ST-6 - Depth: 15.0

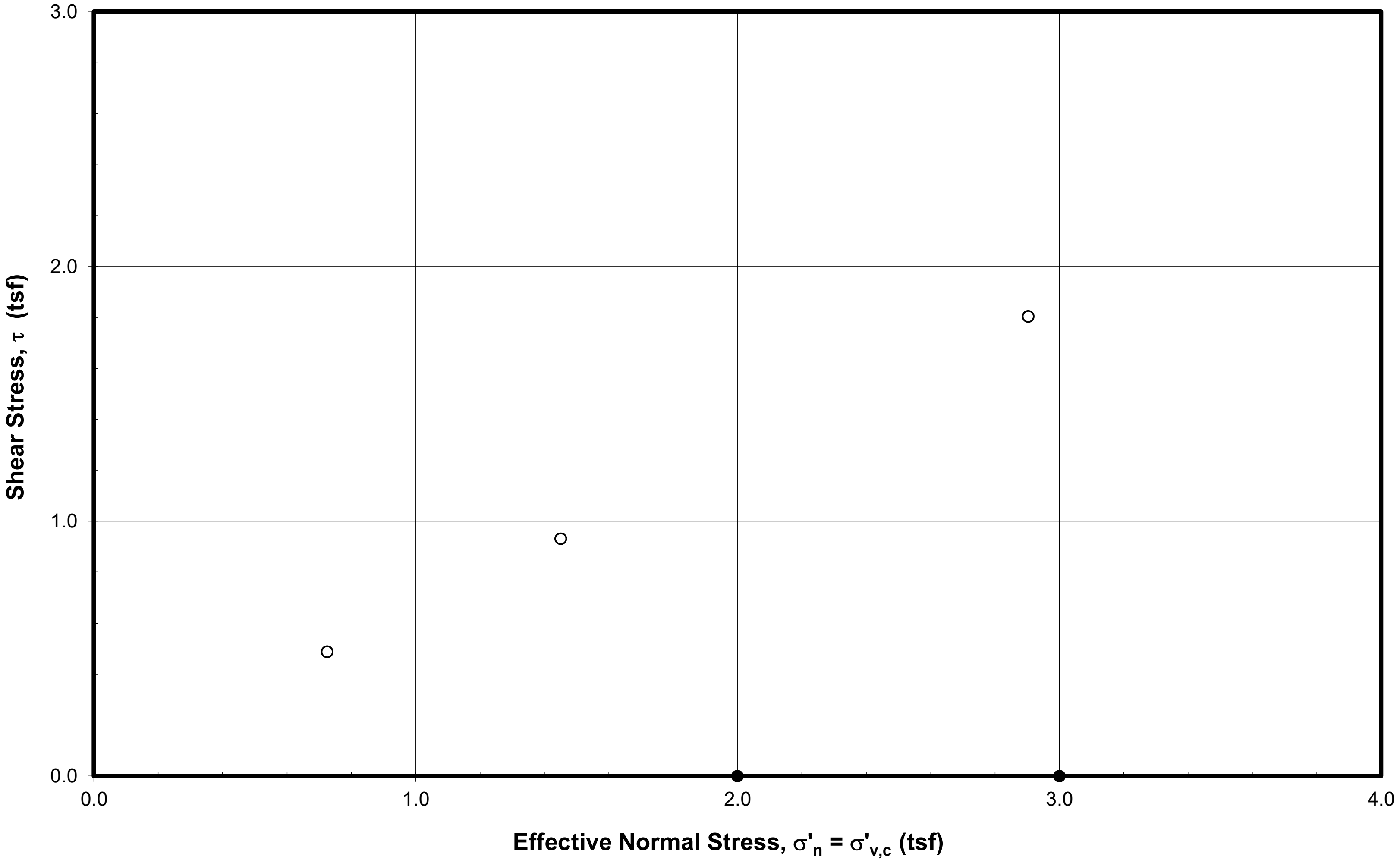
Liquid Limit= 26 Plastic Limit= 13 Plasticity Index = 13 USCS: CL

Compression Index, C_c = 0.05 Void Ratio, e_o = 0.491
Recompression Index, C_r = 0.01 Preconsolidation Pressure = 1.29 tsf



1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435
Project No.: J028499.03
Boring: BC-1
Sample: ST-8 - Depth: 20.0



DRAINED DIRECT SHEAR TEST

ASTM D 3080

Boring: BC-2 Sample: ST-5 -Depth: 10.0ft

TEST REPORT

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

Project No.:	**J028499.03**	**January 6, 2020**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-1**	
Sample ID:	**SS- 1-4**	
Depth (ft):	**1.0’**	

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	11.0
#2	13,000	0.57	7,410.00	17.7
#3	14,000	0.57	7,980.00	22.8

Minimum Soil Resistivity **7,410.00**

TEST REPORT

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

Project No.:	**J028499.03**	**October 17, 2019**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-1**	
Sample ID:	**ST-6**	
Depth (ft):	**15**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	49,000	0.57	27,930.00	10.6
#2	24,000	0.57	13,680.00	17.6
#3	22,000	0.57	12,540.00	25.3
#4	23,000	0.57	13,110.00	28.2

Minimum Soil Resistivity **12,540.00**

TEST REPORT

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

Project No.:	**J028499.03**	**October 2, 2019**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-1**	
Sample ID:	**ST-8**	
Depth (ft):	**20**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	16,000	0.57	9,120.00	10.7
#2	15,000	0.57	8,550.00	18.4
#3	16,000	0.57	9,120.00	25.2

Minimum Soil Resistivity **8,550.00**

TEST REPORT

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

Project No.:	**J028499.03**	**January 6, 2020**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-1**	
Sample ID:	**SS- 9-12**	
Depth (ft):	**23.5'**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	2,600	0.57	1,482.00	12.0
#2	1,600	0.57	912.00	19.5
#3	1,700	0.57	969.00	25.2

Minimum Soil Resistivity **912.00**

TEST REPORT

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

Project No.: J028499.03 — **January 3, 2020**
Project Name: ARDOT 030497 Bridge Replacement over Bodcau Creek — **Page 1 of 1**
Boring Number: BC-1
Sample ID: SS-16-18
Depth (ft): 58.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	7,200	0.57	4,104.00	10.2
#2	4,100	0.57	2,337.00	16.5
#3	3,700	0.57	2,109.00	23.0
#4	3,800	0.57	2,166.00	30.4
	Minimum Soil Resistivity		**2,109.00**	

TEST REPORT

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

Project No.:	**J028499.03**	**January 6, 2020**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-2**	
Sample ID:	**SS- 8-11**	
Depth (ft):	**23.5’**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	2,400	0.57	1,368.00	13.1
#2	1,200	0.57	684.00	21.8
#3	1,100	0.57	627.00	27.7
#4	1,200	0.57	684.00	35.4

Minimum Soil Resistivity **627.00**

TEST REPORT

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

Project No.:	**J028499.03**	**January 6, 2020**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-5**	
Sample ID:	**SS- 4-6**	
Depth (ft):	**18.5'**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	18,000	0.57	10,260.00	10.0
#2	11,000	0.57	6,270.00	16.7
#3	10,000	0.57	5,700.00	23.2
#4	11,000	0.57	6,270.00	31.0

Minimum Soil Resistivity **5,700.00**

TEST REPORT

Prepared For:

Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J028499.03**	**January 8, 2020**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-5**	
Sample ID:	**SS- 7-10**	
Depth (ft):	**33.5’**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	5,600	0.57	3,192.00	12.0
#2	2,300	0.57	1,311.00	18.2
#3	1,800	0.57	1,026.00	25.2
#4	1,500	0.57	855.00	33.4
#5	1,600	0.57	912.00	44.6

Minimum Soil Resistivity **855.00**

TEST REPORT

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

Project No.:	**J028499.03**	**January 8, 2020**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-6**	
Sample ID:	**SS- 3-5**	
Depth (ft):	**18.5'**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	16,000	0.57	9,120.00	10.3
#2	6,500	0.57	3,705.00	19.0
#3	5,500	0.57	3,135.00	26.1
#4	5,900	0.57	3,363.00	35.8

Minimum Soil Resistivity **3,135.00**

TEST REPORT

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

Project No.: J028499.03 — **January 9, 2020**
Project Name: ARDOT 030497 Bridge Replacement over Bodcau Creek — **Page 1 of 1**
Boring Number: BC-7
Sample ID: SS- 4-6
Depth (ft): 8.5’

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	7,700	0.57	4,389.00	10.3
#2	2,600	0.57	1,482.00	19.0
#3	2,400	0.57	1,368.00	26.1
#4	2,500	0.57	1,425.00	35.8

Minimum Soil Resistivity **1,368.00**

TEST REPORT

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

Project No.: J028499.03 — **January 10, 2020**
Project Name: ARDOT 030497 Bridge Replacement over Bodcau Creek — **Page 1 of 1**
Boring Number: BC-7
Sample ID: SS- 12-14
Depth (ft): 48.5’

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	5,300	0.57	3,021.00	13.0
#2	1,600	0.57	912.00	18.3
#3	1,300	0.57	741.00	24.6
#4	1,400	0.57	798.00	33.7

Minimum Soil Resistivity **741.00**

TEST REPORT

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

Project No.:	**J028499.03**	**January 10, 2020**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-8**	
Sample ID:	**SS- 9-11**	
Depth (ft):	**33.5’**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	8,600	0.57	4,902.00	10.1
#2	5,000	0.57	2,850.00	17.2
#3	2,900	0.57	1,653.00	25.5
#4	2,900	0.57	1,653.00	31.7
#5	3,100	0.57	1,767.00	37.1

Minimum Soil Resistivity **1,653.00**

TEST REPORT

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

Project No.:	**J028499.03**	**January 10, 2020**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-9**	
Sample ID:	**SS- 8-9**	
Depth (ft):	**28.5'**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	5,900	0.57	3,363.00	11.0
#2	3,000	0.57	1,710.00	18.1
#3	3,300	0.57	1,881.00	24.5

Minimum Soil Resistivity **1,710.00**

TEST REPORT

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

Project No.: J028499.03 — **October 2, 2019**
Project Name: ARDOT 030497 Bridge Replacement over Bodcau Creek — **Page 1 of 1**
Boring Number: BC-10
Sample ID: ST-3
Depth (ft): 5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	26,000	0.57	14,820.00	10.9
#2	22,000	0.57	12,540.00	17.9
#3	25,000	0.57	14,250.00	24.2

Minimum Soil Resistivity **12,540.00**

TEST REPORT

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

Project No.:	**J028499.03**	**January 10, 2020**
Project Name:	**ARDOT 030497 Bridge Replacement over Bodcau Creek**	**Page 1 of 1**
Boring Number:	**BC-10**	
Sample ID:	**SS- 5-8**	
Depth (ft):	**13.5’**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	40,000	0.57	22,800.00	10.4
#2	21,000	0.57	11,970.00	17.1
#3	20,000	0.57	11,400.00	24.8
#4	17,000	0.57	9,690.00	32.5
#5	19,000	0.57	10,830.00	46.0

Minimum Soil Resistivity **9,690.00**

TEST REPORT

Prepared For:

Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.: J028499.03 **January 10, 2020**
Project Name: ARDOT 030497 Bridge Replacement over Bodcau Creek **Page 1 of 1**
Boring Number: BC-11
Sample ID: SS- 5-6
Depth (ft): 13.5’

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	36,000	0.57	20,520.00	12.2
#2	21,000	0.57	11,970.00	19.6
#3	24,000	0.57	13,680.00	27.2

Minimum Soil Resistivity **11,970.00**

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

GEOTECHNOLOGY INC
FROM THE GROUND UP

DATE	PROJECT NAME	PROJECT NO.
10/16/2019	Bodcau Creek	J028499.03

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

Boring No.	Sample No.	Depth (ft)	Visual Identification (Color, Group Name & Symbol)	Soil : Water Ratio (g/g) or (g/mL)	pH of Solution (Meter/ Paper)[1]	Tare No. Air Drying	Jar Number	Remarks
BC-1	ST-6	15.00		1/1	4.05 ------------- 20.9			

BC-2	ST-5	10.00		1/1	3.77 ------------- 21.2			

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

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

Tested By: LG
Date: 10/16/19

Calculated By: AIM
Date: 10/16/19

Checked By: JDM
Date: 12/04/19

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

GEOTECHNOLOGY INC
FROM THE GROUND UP

DATE	PROJECT NAME	PROJECT NO.
1/9/2020	Bodcau Creek	J028499.03

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

Boring No.	Sample No.	Depth (ft)	Visual Identification (Color, Group Name & Symbol)	Soil : Water Ratio (g/g) or (g/mL)	pH of Solution (Meter/ Paper)[1]	Tare No. Air Drying	Jar Number	Remarks
BC-1	SS 1-4	1'-10'		1/1	4.53 ------------- 22.7			
BC-1	SS 9-12	23.5'-40'		1/2	7.32 ------------- 22.5			
BC-1	SS 16-18	58.5'-70'		1/1	5.39 ------------- 22.4			
BC-2	SS 8-11	23.5'-40'		1/2	7.42 ------------- 22.4			
BC-5	SS 4-6	18.5'-30'		1/1	6.25 ------------- 22.4			
BC-5	SS 7-10	33.5'-50'		1/1	5.36 ------------- 22.3			
BC-6	SS 3-5	18.5'-30'		1/1	6.4 ------------- 22.4			
BC-7	SS 4-6	8.5'-20'		1/1	5.31 ------------- 19.9			
BC-7	SS 12-14	48.5'-60'		1/2	4.29 ------------- 19.7			
B-8	SS 9-11	33.5'-45'		1/1	3.33 ------------- 19.8			
BC-9	SS 8-9	28.5'-35'		1/2	5.44 ------------- 20.5			
BC-10	SS 5-8	13.5'-30'		1/1	3.73 ------------- 20.5			
BC-11	SS 5-6	13.5'-20'		1/1	3.81 ------------- 20.6			
					-------------			

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

Tested By: TH
Date: 01/09/20

Calculated By: AIM
Date: 01/10/20

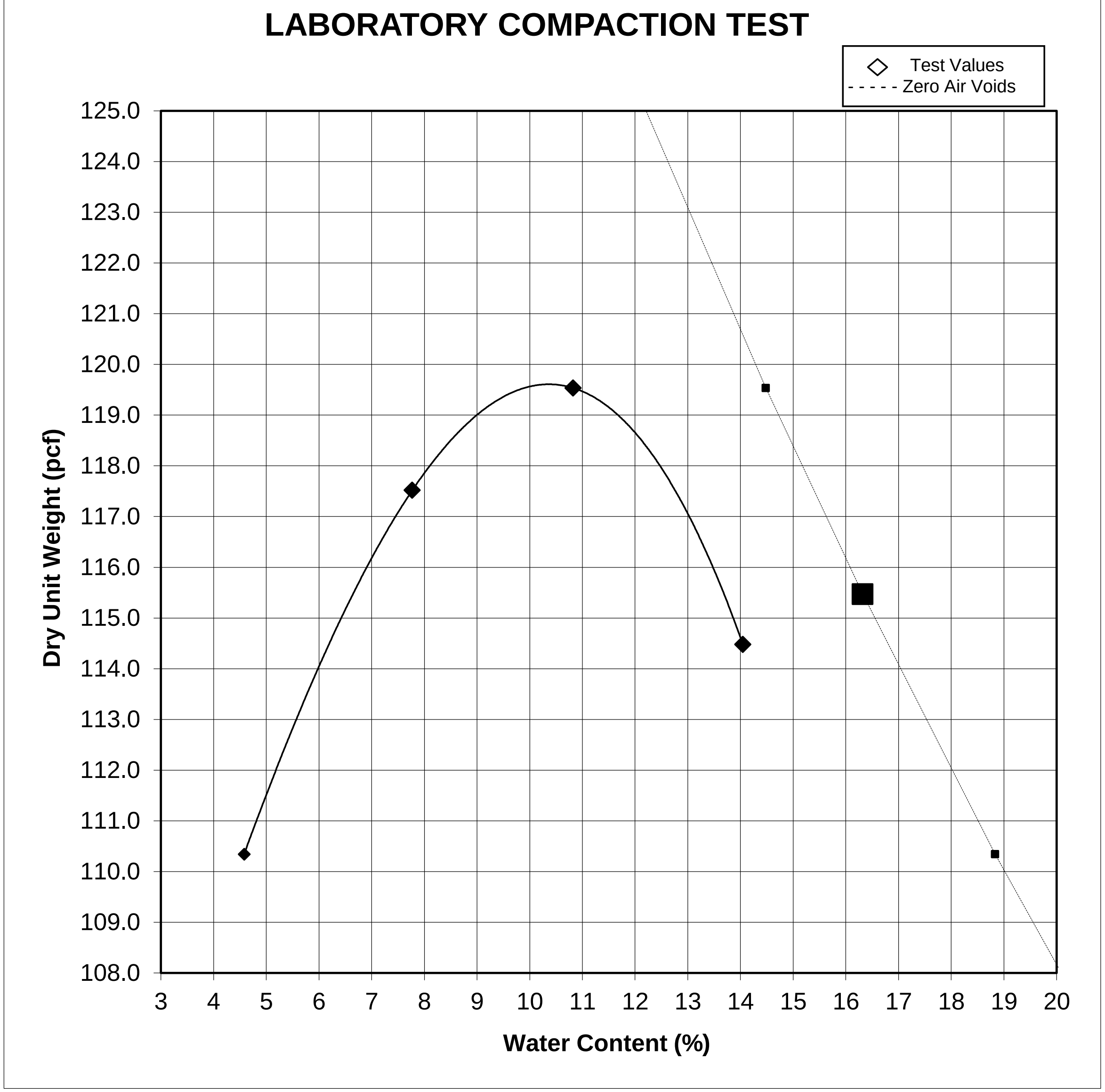
Checked By: JDM
Date: 03/09/20

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

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: ARDOT 030497 - Bodcau Creek Bridg
Client: Garver USA
Sample Source: BC-13, 1-5'
Supplier:

LABORATORY COMPACTION TEST



Test Information

Project No.: J028499.03
Test Date: 03/26/19
Proctor No.: BC-13

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

Sample Description

Brown Sandy Lean Clay

Sample Properties

Moisture Content NA
Liquid Limit 31
Plastic Limit 14
Plasticity Index 17
Specific Gravity: 2.650 Estimated
Classification CL

Test Results:

Maximum Dry Unit Weight (pcf): 119.6
Optimum Water Content (%): 10.4

Oversize Correction Values:

Maximum Dry Unit Weight (pcf): 121.0
Optimum Water Content (%): 10.0

Tested By: TA
Date: 03/26/19

Input By: ALY
Date: 03/28/19

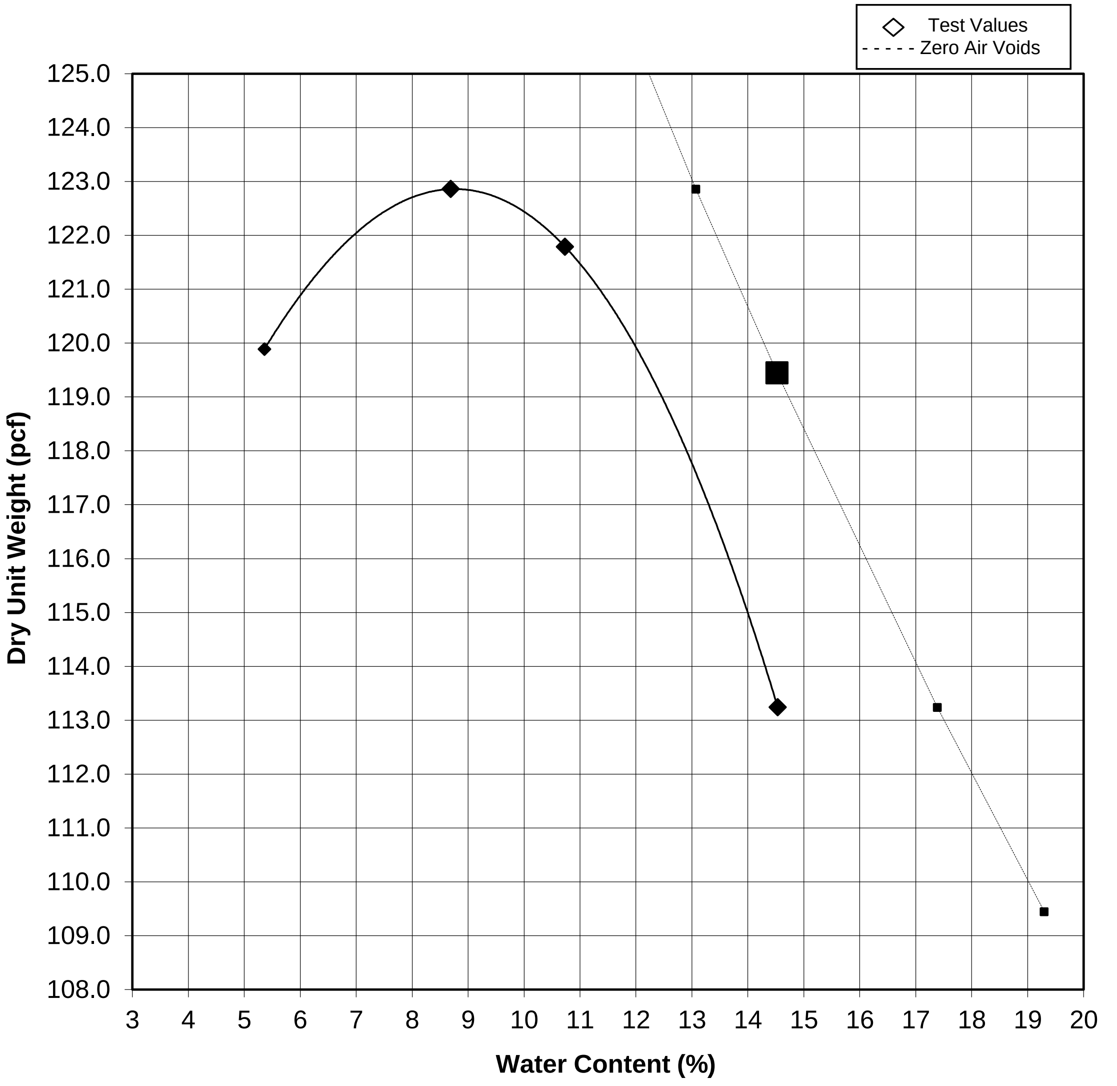
Checked By: HP
Date: 03/28/19

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

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: ARDOT 030497 - Bodcau Creek Bridg
Client: Garver USA
Sample Source: BC-14, 1-5'
Supplier:

LABORATORY COMPACTION TEST



Test Information

Project No.: J028499.03
Test Date: 03/27/19
Proctor No.: BC-14

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

Sample Description

Brown Clayey Sand with Gravel and Asphalt

Sample Properties

Moisture Content NA
Liquid Limit 27
Plastic Limit 12
Plasticity Index 15
Specific Gravity: 2.650 Estimated
Classification SC

Test Results:

Maximum Dry Unit Weight (pcf): 122.9
Optimum Water Content (%): 8.8

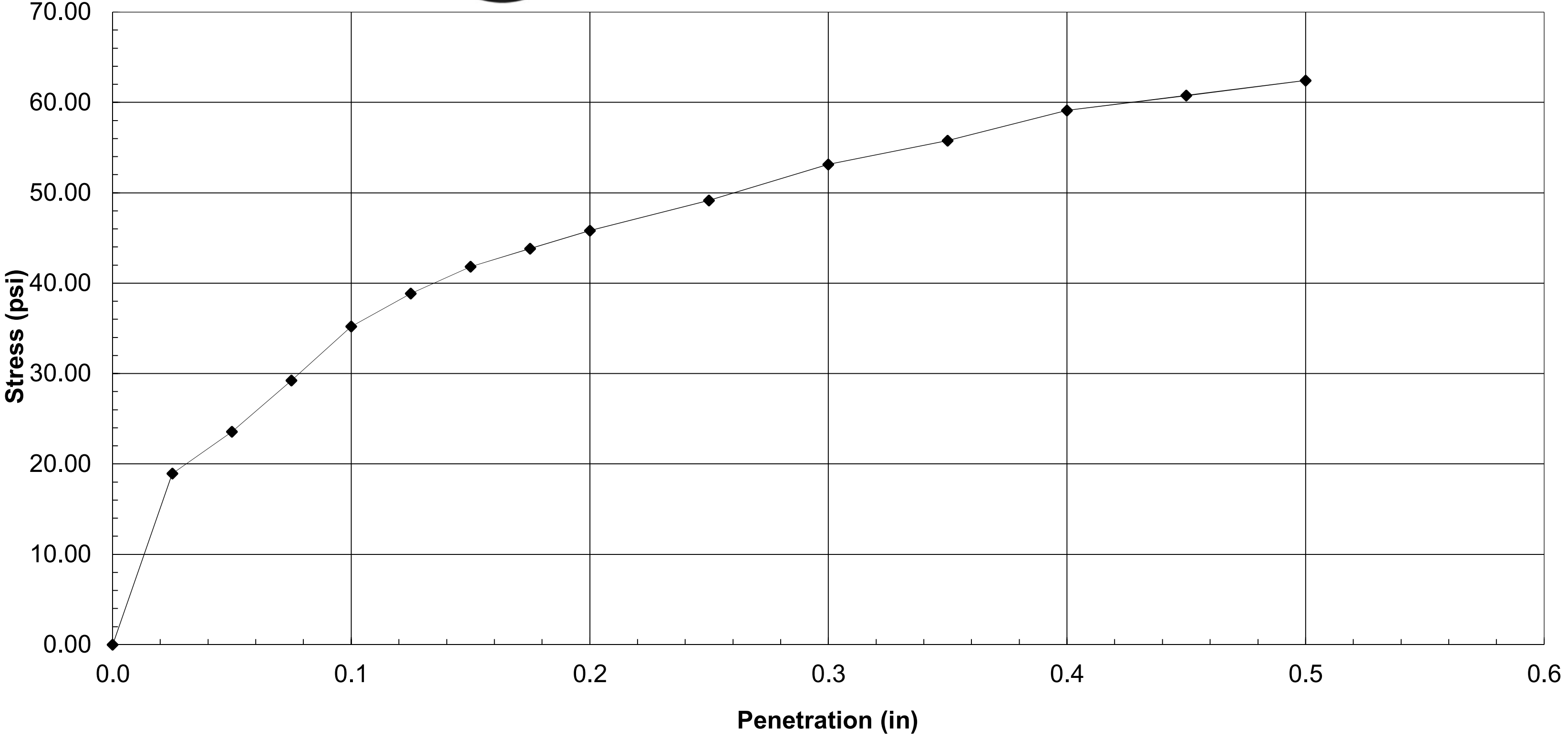
Oversize Correction Values:

Maximum Dry Unit Weight (pcf): 132.2
Optimum Water Content (%): 6.5

Tested By: MP
Date: 03/27/19

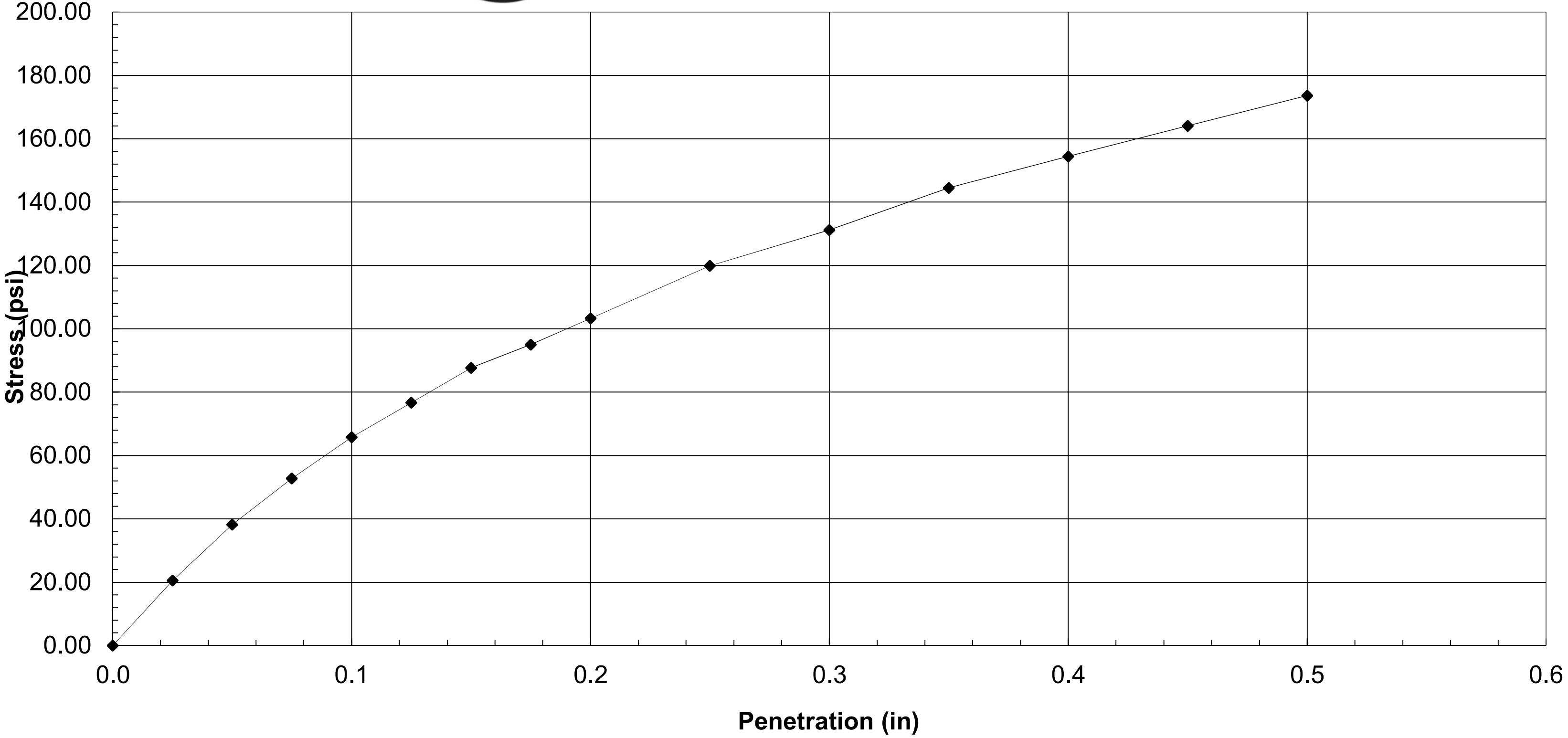
Input By: ALY
Date: 03/28/19

Checked By: HP
Date: 03/28/19



CALIFORNIA BEARING RATIO (CBR) TEST

ASTM D 1883
Project No.: J028499.02
Boring: BC-13
Sample: 25 Blows - Depth: 0 ft.



CALIFORNIA BEARING RATIO (CBR) TEST

ASTM D 1883
Project No.: J028499.02
Boring: BC-13
Sample: 56 Blows - Depth: 0 ft.

APPENDIX E – AASHTO AND USCS CLASSIFICATIONS

SUMMARY OF CLASSIFICATION RESULTS
Highway 82 Strs. & Apprs. (S): Bodcau Creek Bridge
Lafayette County: Arkansas
ARDOT 030497

Boring	Depth (feet)	Liquid Limit (LL) (%)	Plastic Limit (PL) (%)	Plasticity Index (PI) (%)	Sieve Analysis Percent Passing								GI	AASHTO CLASS.	USCS CLASS.
					2 in.	1 in.	3/4 in.	3/8 in.	#4	#10	#40	#200			
BC-1	1	--	--	--	100.0	100.0	100.0	100.0	99.3	99.3	97.5	59.2	0	A-4	ML
BC-1	15	32	16	16	100.0	100.0	100.0	100.0	100.0	100.0	99.9	76.4	10	A-6	CL
BC-1	20	26	13	13	100.0	100.0	100.0	100.0	100.0	99.8	98.3	56.0	4	A-6	CL
BC-1	23.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.9	51.3	0	A-4	ML
BC-1	28.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.9	58.4	0	A-4	ML
BC-1	33.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.8	99.5	0	A-2-7	CH
BC-1	38.5	87	21	66	--	--	--	--	--	--	--	--	0	A-2-7	CH
BC-1	68.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.7	28.8	0	A-2-4	SM
BC-1	73.5	--	--	--	100.0	100.0	100.0	100.0	100.0	99.8	98.3	49.0	0	A-4	SM
BC-2	10	36	18	18	100.0	100.0	100.0	100.0	100.0	100.0	99.8	84.2	14	A-6	CL
BC-2	43.5	85	20	65	--	--	--	--	--	--	--	--	0	A-2-7	CH
BC-4	18.5	60	20	40	--	--	--	--	--	--	--	--	0	A-2-7	CH
BC-4	23.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	100.0	73.5	0	A-4	ML
BC-4	28.5	0	0	0	--	--	--	--	--	--	--	--	0	A-1-a	ML
BC-4	33.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.9	11.8	0	A-2-4	SP-SM
BC-6	18.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	100.0	79.1	0	A-4	ML
BC-7	58.5	51	15	36	--	--	--	--	--	--	--	--	0	A-2-7	CH
BC-8	13.5	23	15	8	--	--	--	--	--	--	--	--	0	A-2-4	CL
BC-8	28.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.9	58.4	0	A-4	ML
BC-8	33.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.9	18.3	0	A-2-4	SM
BC-9	38.5	26	12	14	--	--	--	--	--	--	--	--	0	A-2-6	CL
BC-10	5	19	15	4	100.0	100.0	100.0	100.0	100.0	99.7	93.2	20.4	0	A-2-4	SC-SM
BC-10	33.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.4	56.2	0	A-4	ML
BC-10	58.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.9	38.5	0	A-4	SM
BC-11	13.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.9	64.0	0	A-4	ML
BC-11	33.5	30	13	17	--	--	--	--	--	--	--	--	0	A-2-6	CL
BC-13	1	31	14	17	100.0	100.0	100.0	98.9	95.2	91.2	82.7	68.7	9	A-6	CL
BC-14	1	27	12	15	100.0	100.0	100.0	92.6	88.1	83.4	68.3	23.5	0	A-2-6	SC

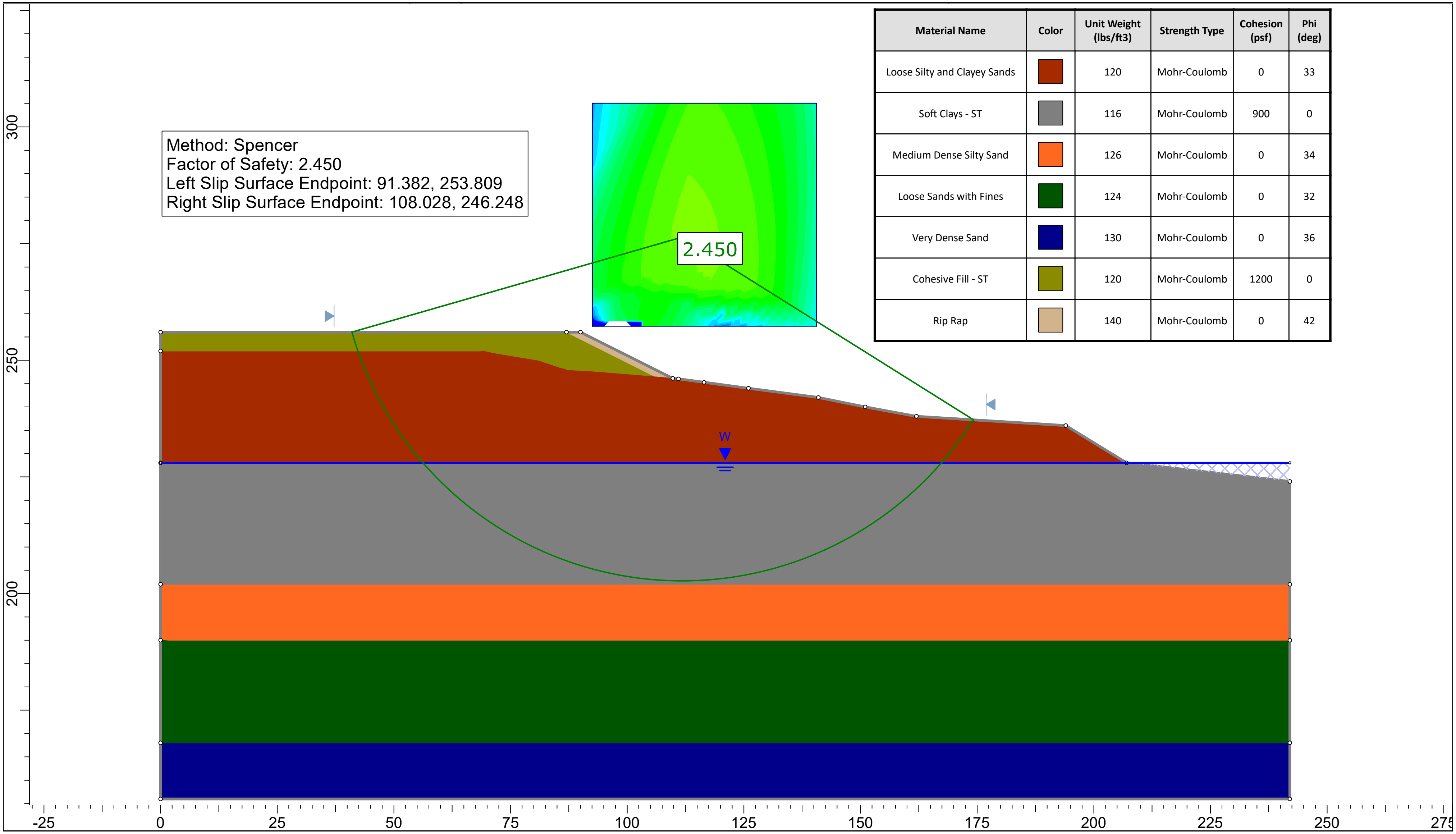
APPENDIX F – GLOBAL STABILITY ANALYSES

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: West Abutment.slmd
Name: Group 1
Description: Short Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - West Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 2.450
Left Slip Surface Endpoint: 91.382, 253.809
Right Slip Surface Endpoint: 108.028, 246.248

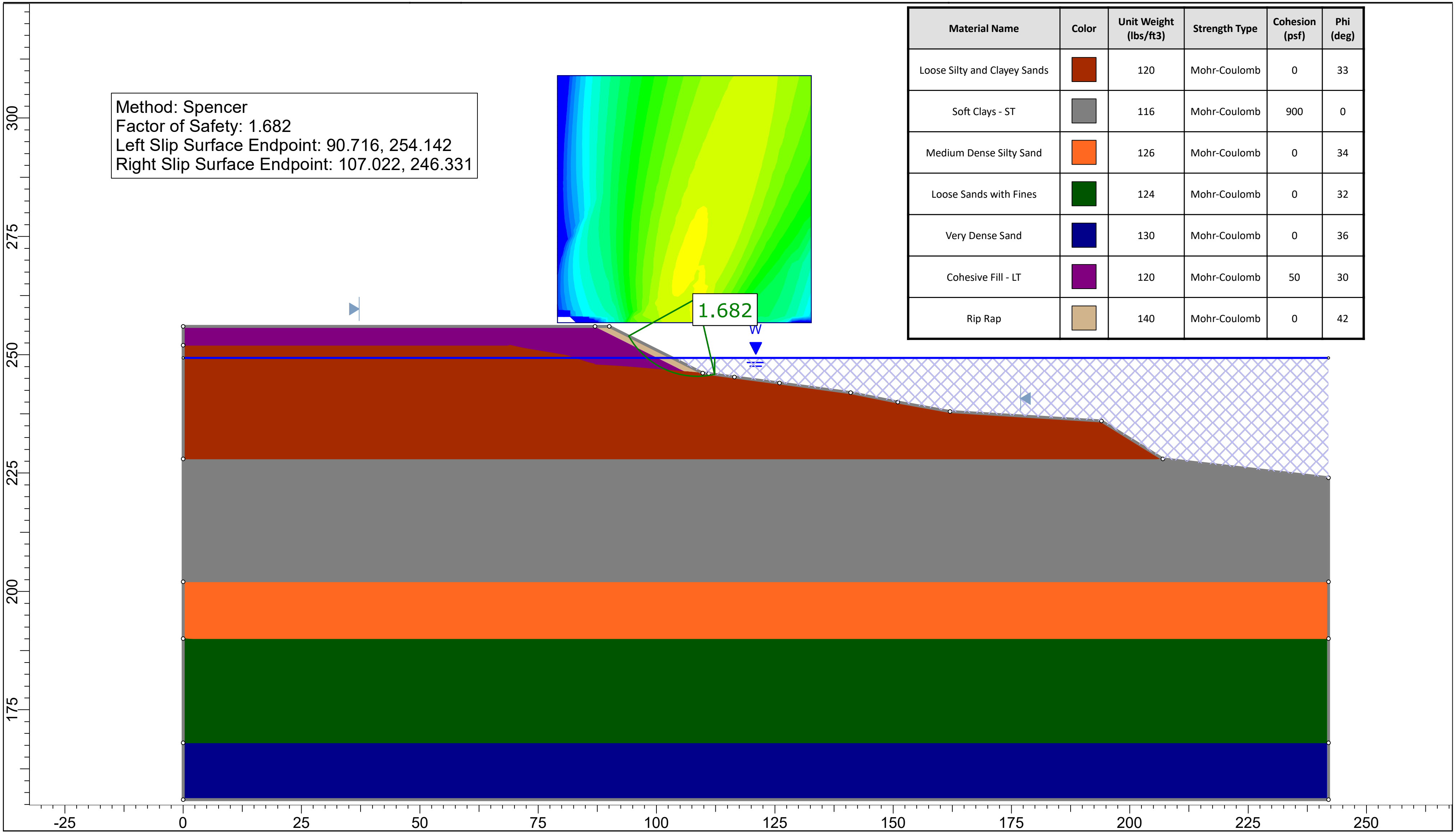
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose Silty and Clayey Sands	🟥	120	Mohr-Coulomb	0	33
Soft Clays - ST	⬛	116	Mohr-Coulomb	900	0
Medium Dense Silty Sand	🟧	126	Mohr-Coulomb	0	34
Loose Sands with Fines	🟩	124	Mohr-Coulomb	0	32
Very Dense Sand	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - ST	🟨	120	Mohr-Coulomb	1200	0
Rip Rap	🟫	140	Mohr-Coulomb	0	42

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: West Abutment.slmd
Name: Group 1
Description: Long Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - West Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 1.682
Left Slip Surface Endpoint: 90.716, 254.142
Right Slip Surface Endpoint: 107.022, 246.331

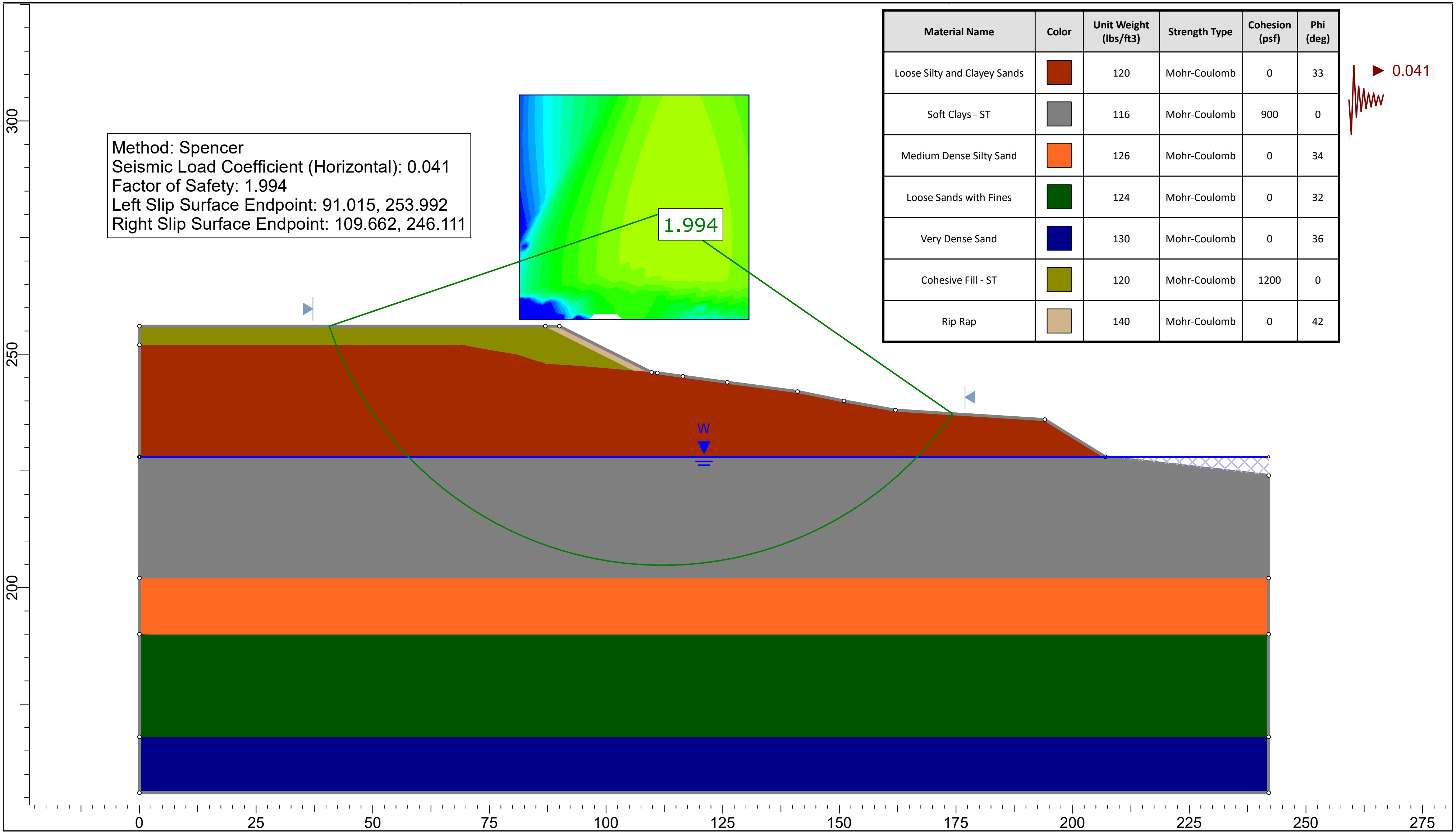
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose Silty and Clayey Sands	🟥	120	Mohr-Coulomb	0	33
Soft Clays - ST	◼	116	Mohr-Coulomb	900	0
Medium Dense Silty Sand	🟧	126	Mohr-Coulomb	0	34
Loose Sands with Fines	🟩	124	Mohr-Coulomb	0	32
Very Dense Sand	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - LT	🟪	120	Mohr-Coulomb	50	30
Rip Rap	🟫	140	Mohr-Coulomb	0	42

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: West Abutment.slmd
Name: Group 1
Description: Seismic Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - West Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Seismic Load Coefficient (Horizontal): 0.041
Factor of Safety: 1.994
Left Slip Surface Endpoint: 91.015, 253.992
Right Slip Surface Endpoint: 109.662, 246.111

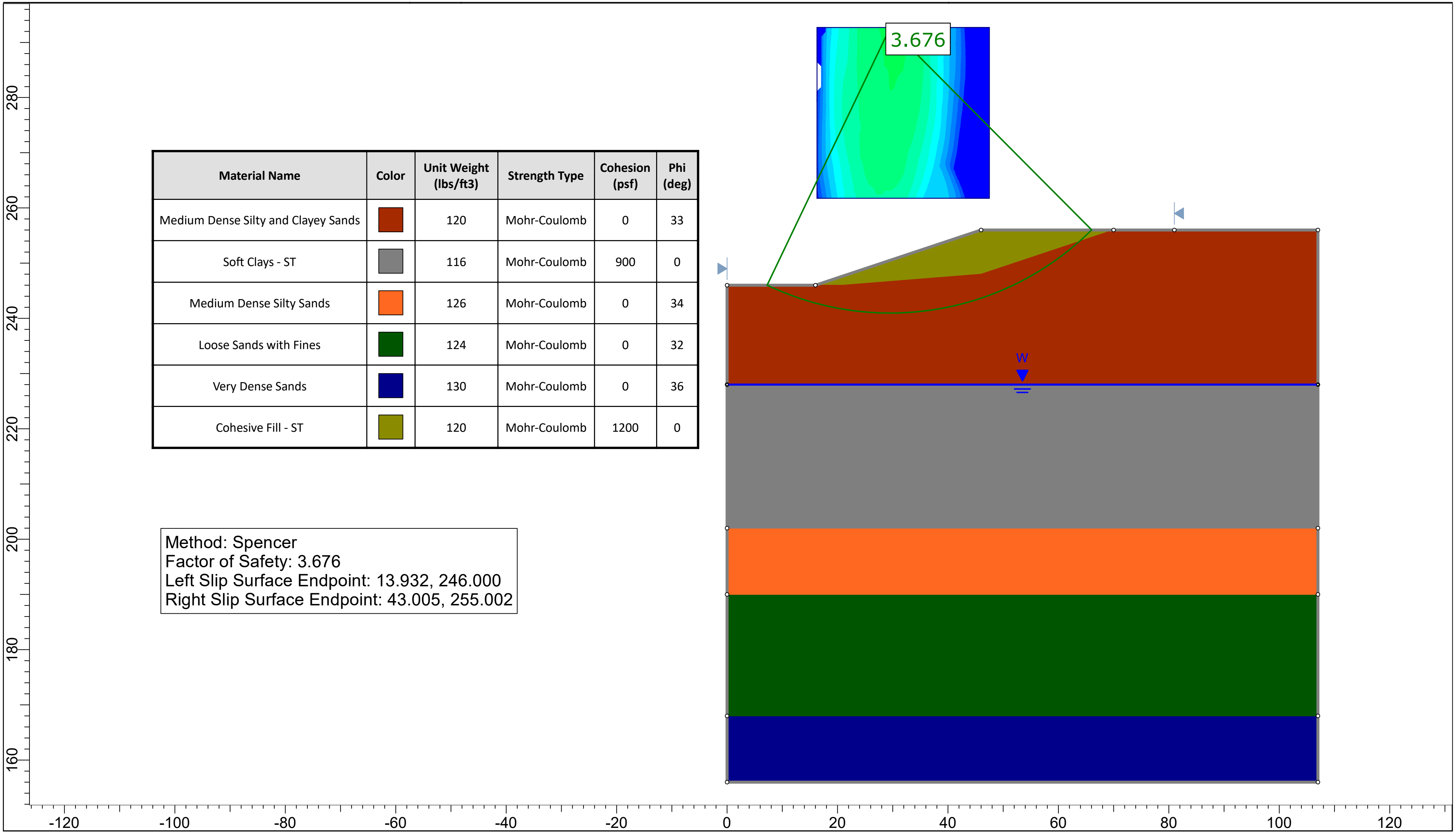
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose Silty and Clayey Sands	🟥	120	Mohr-Coulomb	0	33
Soft Clays - ST	⬛	116	Mohr-Coulomb	900	0
Medium Dense Silty Sand	🟧	126	Mohr-Coulomb	0	34
Loose Sands with Fines	🟩	124	Mohr-Coulomb	0	32
Very Dense Sand	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - ST	🟨	120	Mohr-Coulomb	1200	0
Rip Rap	🟫	140	Mohr-Coulomb	0	42

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Slope.slmd
Name: Group 1
Description: Short Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - STA 210+00
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Medium Dense Silty and Clayey Sands	🟥	120	Mohr-Coulomb	0	33
Soft Clays - ST	⬛	116	Mohr-Coulomb	900	0
Medium Dense Silty Sands	🟧	126	Mohr-Coulomb	0	34
Loose Sands with Fines	🟩	124	Mohr-Coulomb	0	32
Very Dense Sands	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - ST	🟨	120	Mohr-Coulomb	1200	0

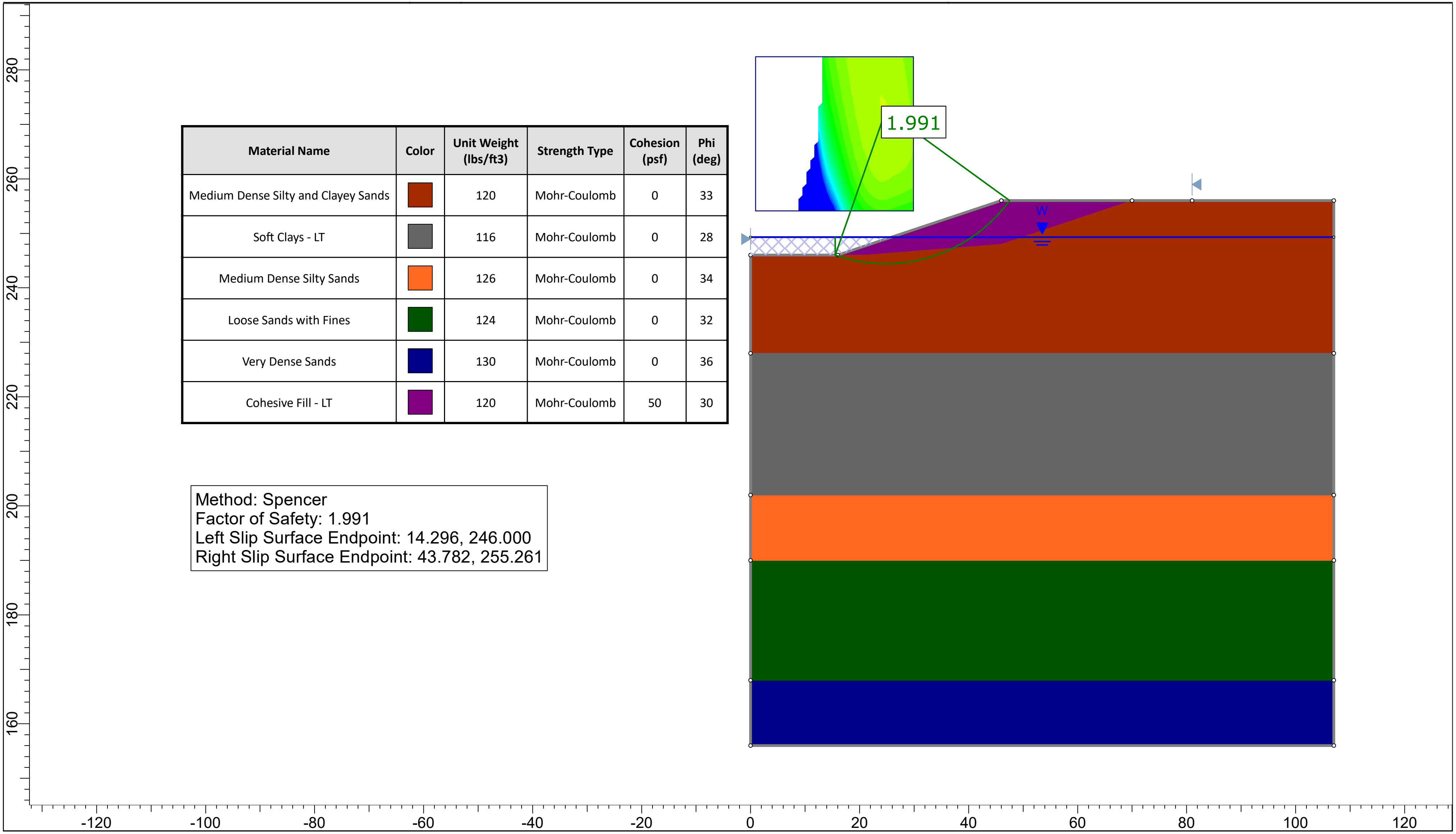
Method: Spencer
Factor of Safety: 3.676
Left Slip Surface Endpoint: 13.932, 246.000
Right Slip Surface Endpoint: 43.005, 255.002

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Slope.slmd
Name: Group 1
Description: Long Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - STA 210+00
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Medium Dense Silty and Clayey Sands	🟫	120	Mohr-Coulomb	0	33
Soft Clays - LT	⬛	116	Mohr-Coulomb	0	28
Medium Dense Silty Sands	🟧	126	Mohr-Coulomb	0	34
Loose Sands with Fines	🟩	124	Mohr-Coulomb	0	32
Very Dense Sands	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - LT	🟪	120	Mohr-Coulomb	50	30

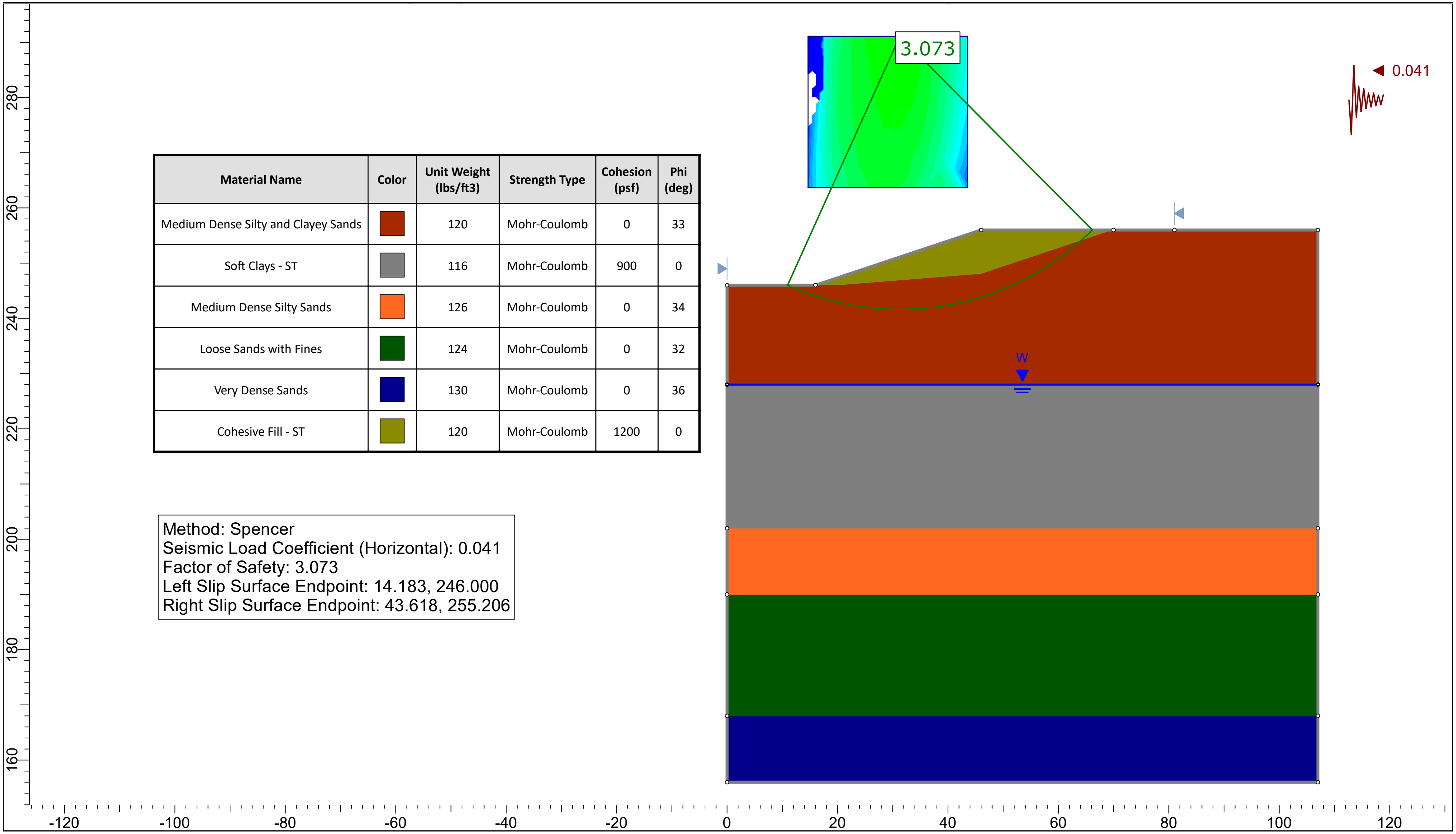
Method: Spencer
Factor of Safety: 1.991
Left Slip Surface Endpoint: 14.296, 246.000
Right Slip Surface Endpoint: 43.782, 255.261

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Slope.slmd
Name: Group 1
Description: Seismic Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - STA 210+00
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Medium Dense Silty and Clayey Sands	🟥	120	Mohr-Coulomb	0	33
Soft Clays - ST	⬛	116	Mohr-Coulomb	900	0
Medium Dense Silty Sands	🟧	126	Mohr-Coulomb	0	34
Loose Sands with Fines	🟩	124	Mohr-Coulomb	0	32
Very Dense Sands	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - ST	🟨	120	Mohr-Coulomb	1200	0

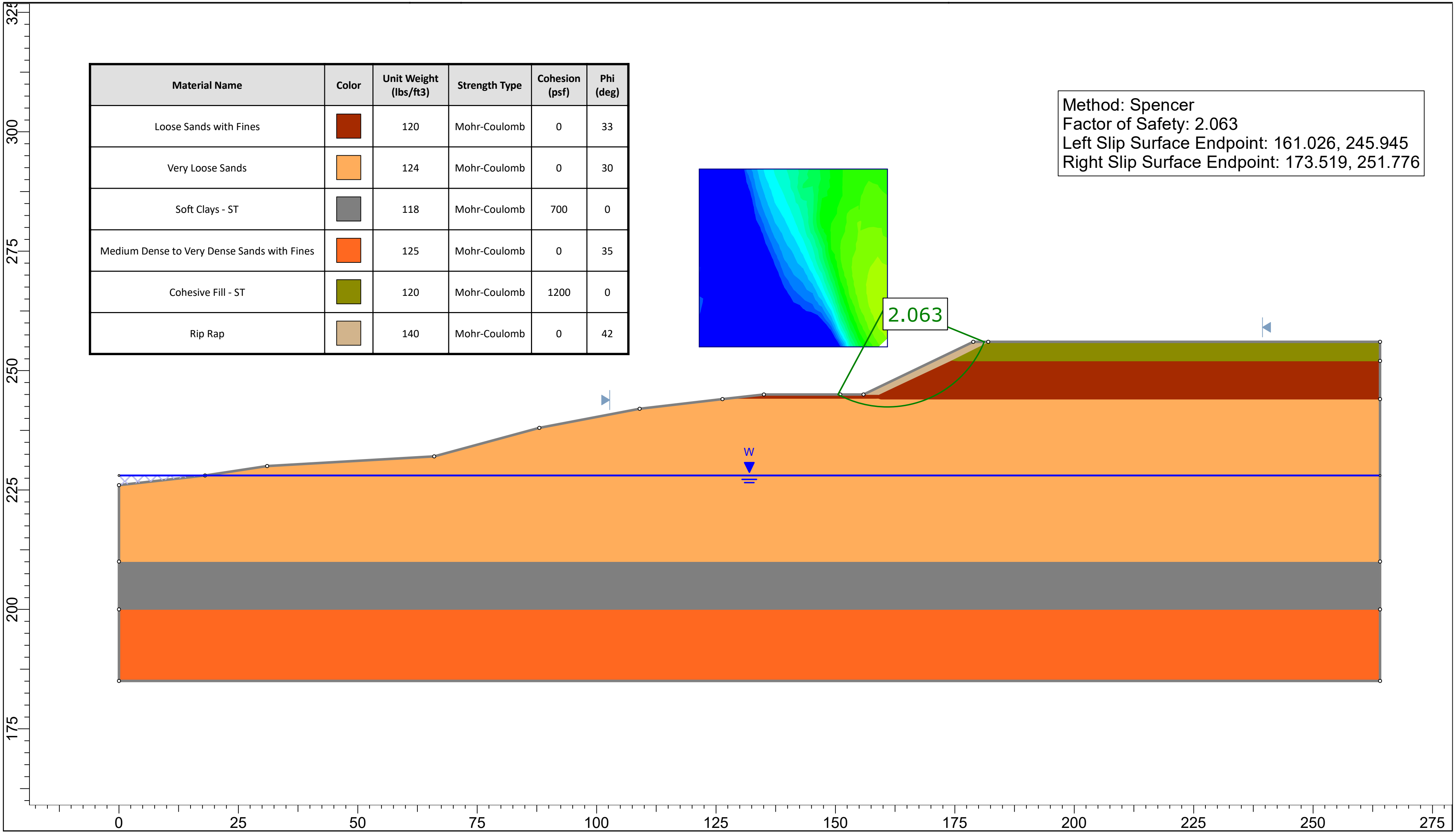
Method: Spencer
Seismic Load Coefficient (Horizontal): 0.041
Factor of Safety: 3.073
Left Slip Surface Endpoint: 14.183, 246.000
Right Slip Surface Endpoint: 43.618, 255.206

File Name: East Abutment.slmd
Name: Group 1
Description: Short Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - East Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	33
Very Loose Sands	🟧	124	Mohr-Coulomb	0	30
Soft Clays - ST	⬛	118	Mohr-Coulomb	700	0
Medium Dense to Very Dense Sands with Fines	🟧	125	Mohr-Coulomb	0	35
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0
Rip Rap	🟫	140	Mohr-Coulomb	0	42



Method: Spencer
Factor of Safety: 2.063
Left Slip Surface Endpoint: 161.026, 245.945
Right Slip Surface Endpoint: 173.519, 251.776

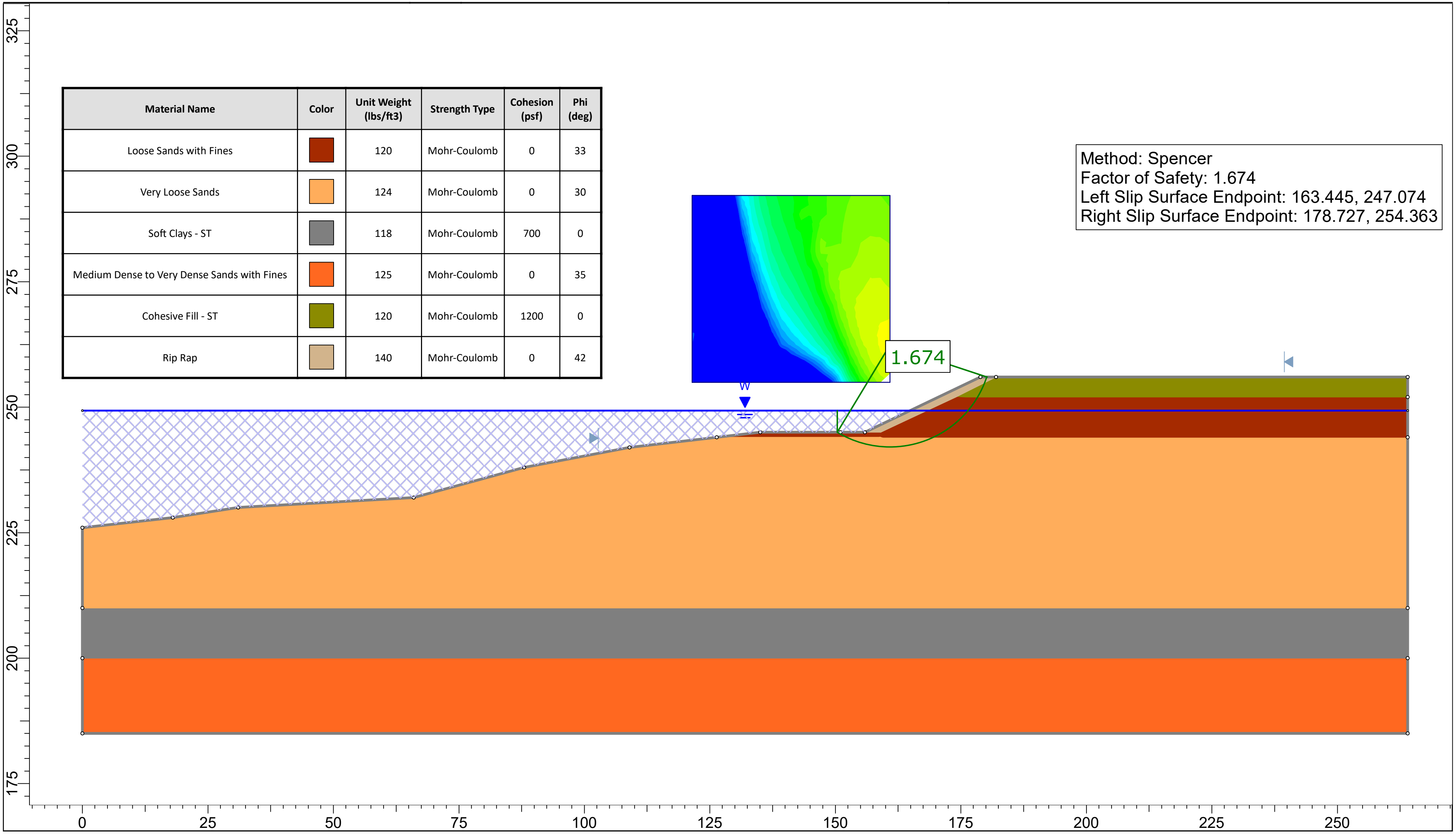
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: East Abutment.slmd
Name: Group 1
Description: Long Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - East Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	33
Very Loose Sands	🟧	124	Mohr-Coulomb	0	30
Soft Clays - ST	⬛	118	Mohr-Coulomb	700	0
Medium Dense to Very Dense Sands with Fines	🟧	125	Mohr-Coulomb	0	35
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0
Rip Rap	🟫	140	Mohr-Coulomb	0	42



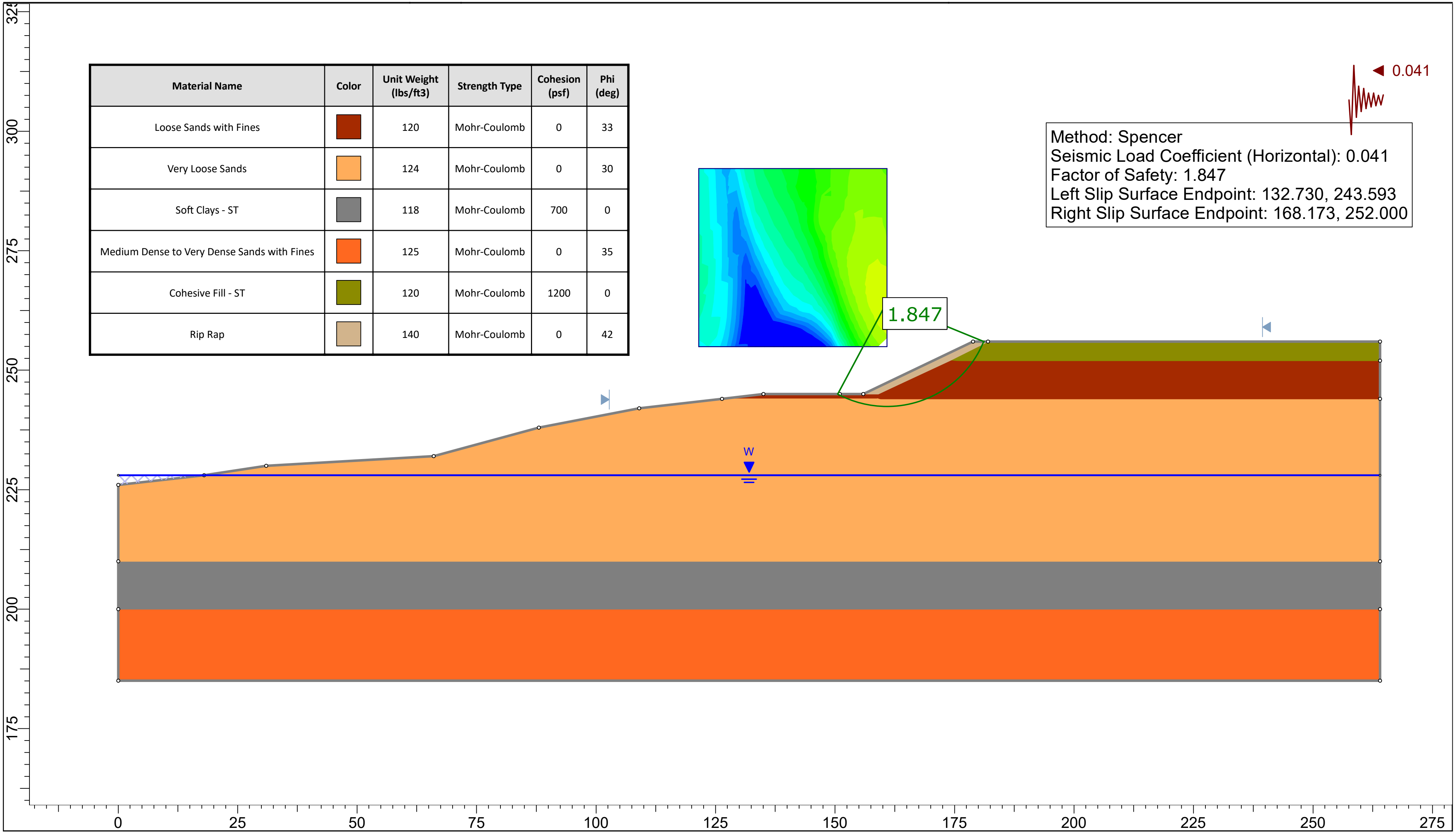
Method: Spencer
Factor of Safety: 1.674
Left Slip Surface Endpoint: 163.445, 247.074
Right Slip Surface Endpoint: 178.727, 254.363

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: East Abutment.slmd
Name: Group 1
Description: Seismic Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - East Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	33
Very Loose Sands	🟧	124	Mohr-Coulomb	0	30
Soft Clays - ST	⬛	118	Mohr-Coulomb	700	0
Medium Dense to Very Dense Sands with Fines	🟧	125	Mohr-Coulomb	0	35
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0
Rip Rap	🟫	140	Mohr-Coulomb	0	42

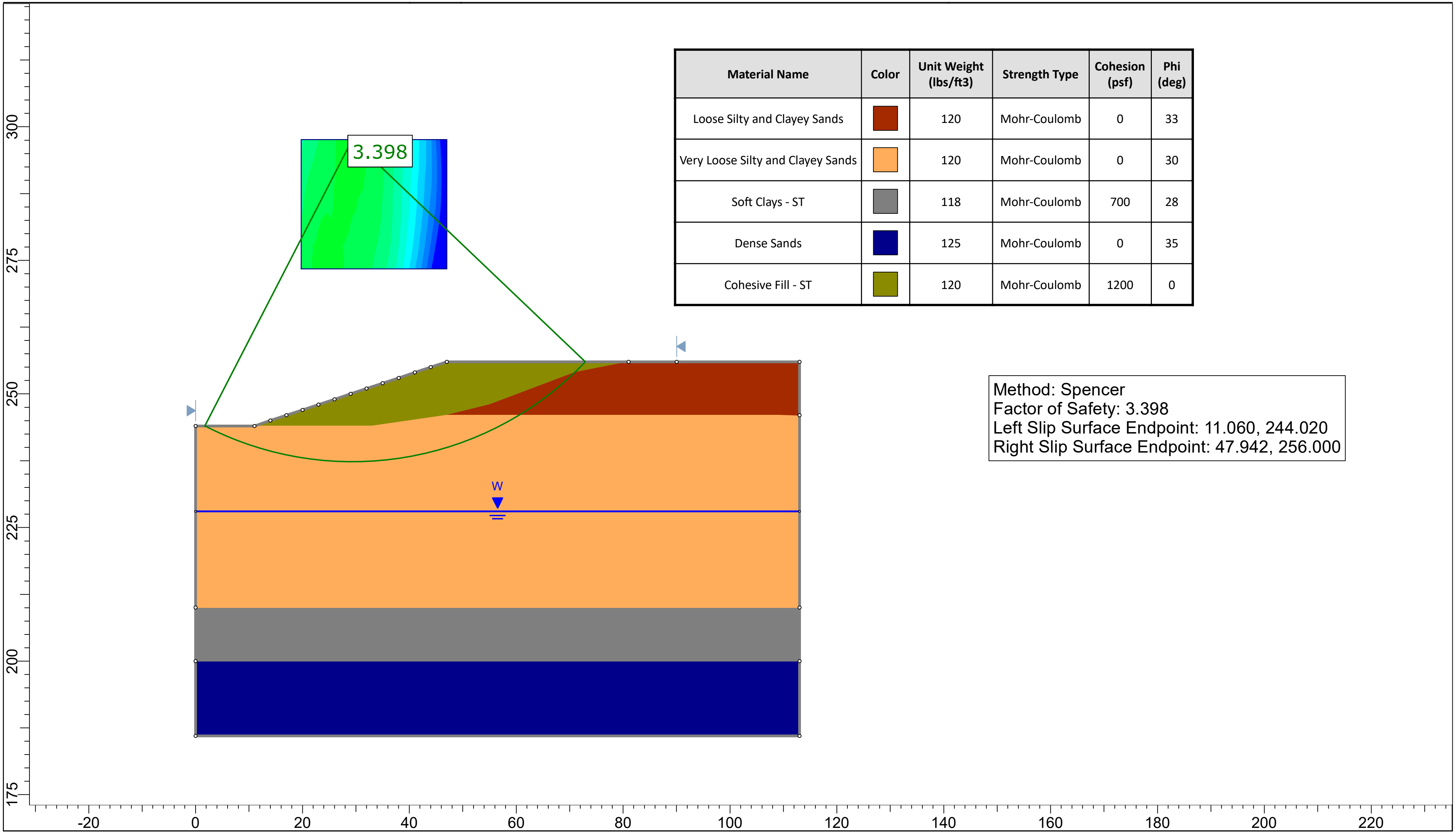
Method: Spencer
Seismic Load Coefficient (Horizontal): 0.041
Factor of Safety: 1.847
Left Slip Surface Endpoint: 132.730, 243.593
Right Slip Surface Endpoint: 168.173, 252.000

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Slope.slmd
Name: Group 1
Description: Short Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - STA 215+00
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose Silty and Clayey Sands	🟥	120	Mohr-Coulomb	0	33
Very Loose Silty and Clayey Sands	🟧	120	Mohr-Coulomb	0	30
Soft Clays - ST	⬛	118	Mohr-Coulomb	700	28
Dense Sands	🟦	125	Mohr-Coulomb	0	35
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0

Method: Spencer
Factor of Safety: 3.398
Left Slip Surface Endpoint: 11.060, 244.020
Right Slip Surface Endpoint: 47.942, 256.000

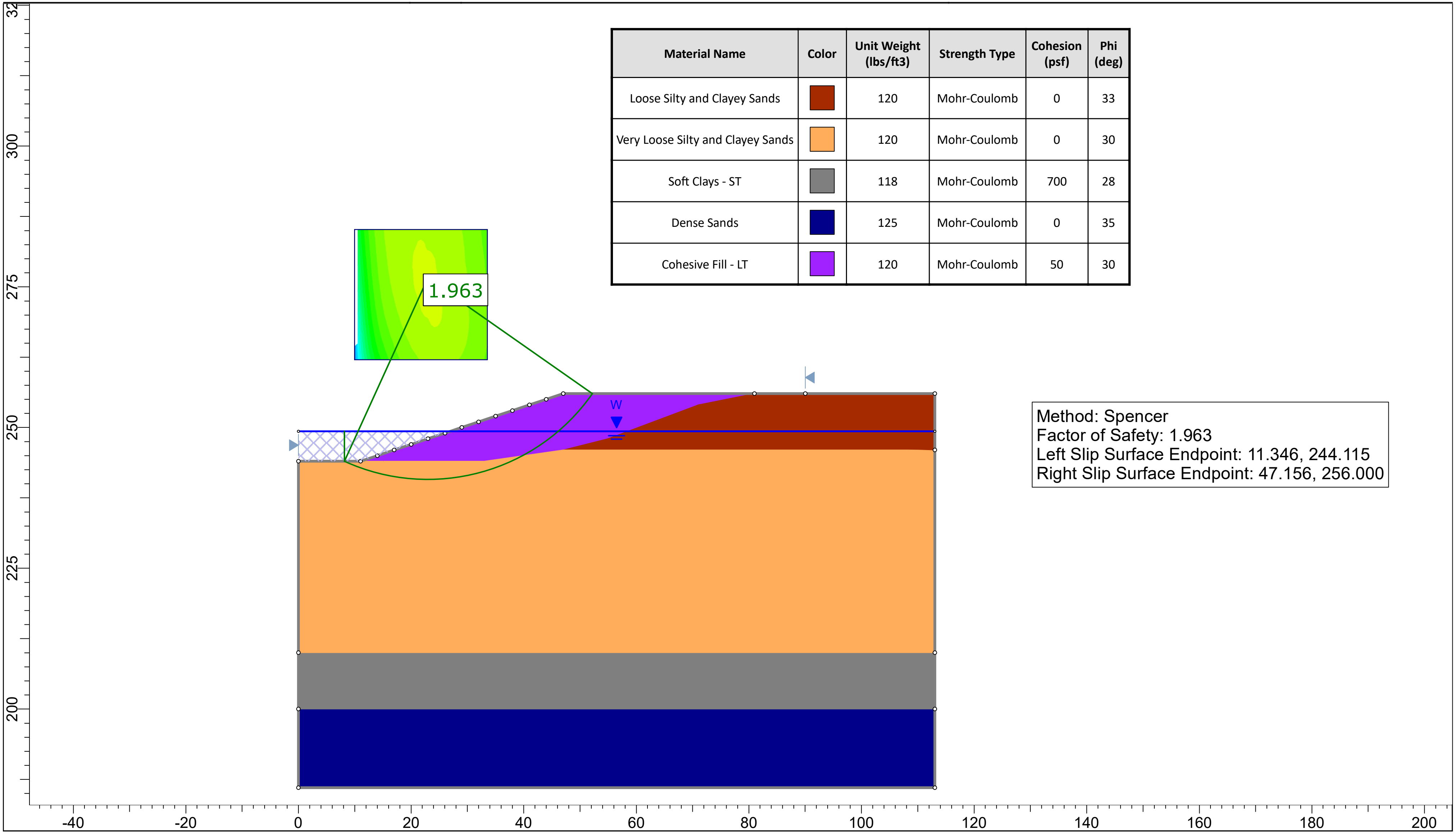
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Slope.slmd
Name: Group 1
Description: Long Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - STA 215+00
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose Silty and Clayey Sands	🟥	120	Mohr-Coulomb	0	33
Very Loose Silty and Clayey Sands	🟧	120	Mohr-Coulomb	0	30
Soft Clays - ST	◼	118	Mohr-Coulomb	700	28
Dense Sands	🟦	125	Mohr-Coulomb	0	35
Cohesive Fill - LT	🟪	120	Mohr-Coulomb	50	30



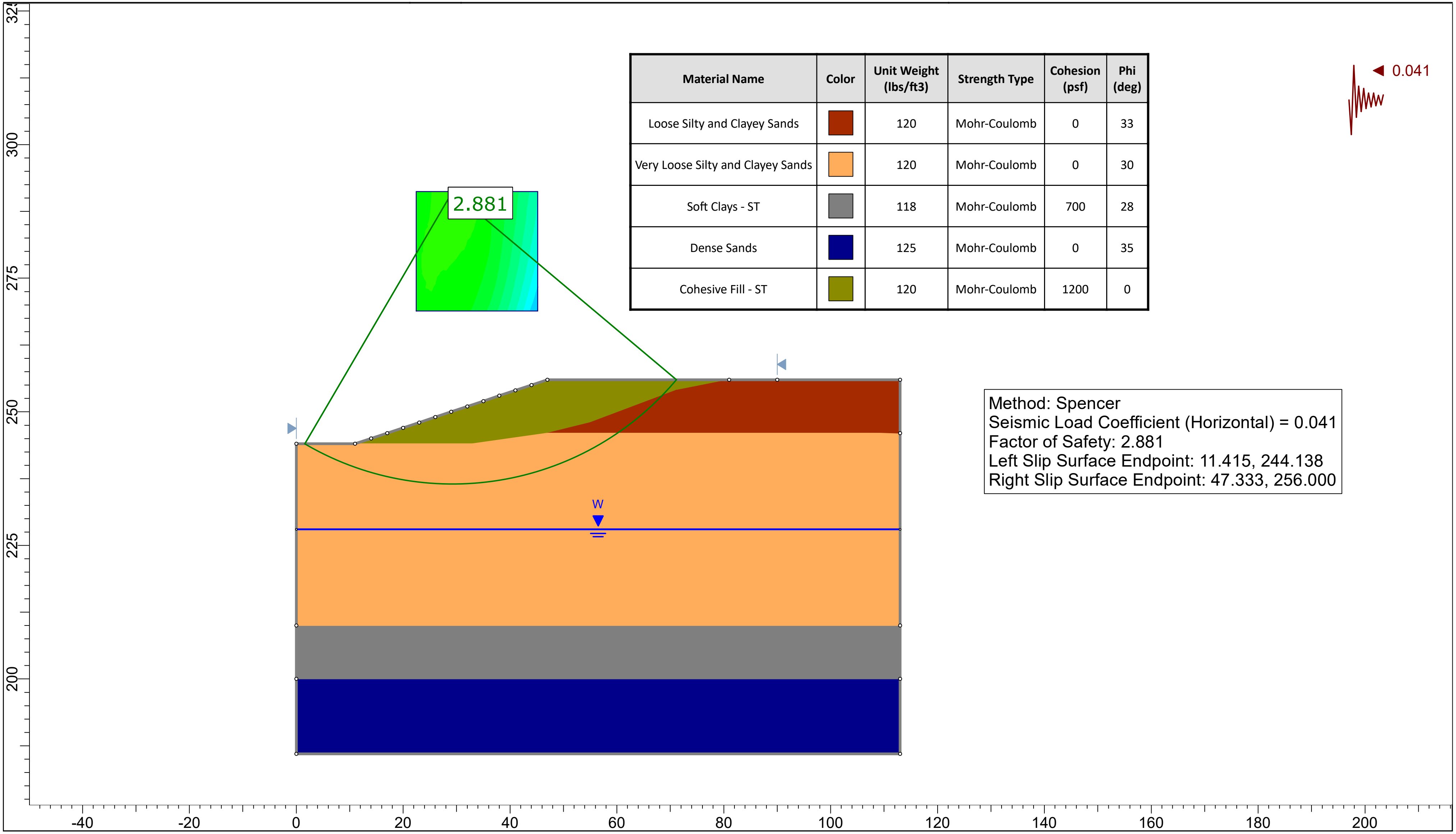
Method: Spencer
Factor of Safety: 1.963
Left Slip Surface Endpoint: 11.346, 244.115
Right Slip Surface Endpoint: 47.156, 256.000

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Slope.slmd
Name: Group 1
Description: Seismic Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Bodcau Creek - STA 215+00
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose Silty and Clayey Sands	🟫	120	Mohr-Coulomb	0	33
Very Loose Silty and Clayey Sands	🟧	120	Mohr-Coulomb	0	30
Soft Clays - ST	◼	118	Mohr-Coulomb	700	28
Dense Sands	🟦	125	Mohr-Coulomb	0	35
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0

Method: Spencer
Seismic Load Coefficient (Horizontal) = 0.041
Factor of Safety: 2.881
Left Slip Surface Endpoint: 11.415, 244.138
Right Slip Surface Endpoint: 47.333, 256.000

APPENDIX G – SOIL PARAMETERS FOR SYNTHETIC PROFILES

BODCAU CREEK BRIDGE INTERNAL BENTS 2, 3, & 4 – BORINGS BC- 4 THROUGH BC-6

ASSUMED PILE CUTOFF ELEVATION: EL 230

ZONE	SOIL TYPES / LPILE SOIL[b]	DEPTH (ELEVATION)		WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD[b] PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[a]
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)		
1	Soft Clays / Soft Clay (Matlock)	230	210	117	500	--	--	28	0.01	100
2	Loose/Medium Dense Sands with Silt / Sand (Reese)	210	183	124	--	32	--	32	--	20
3	Very Stiff/Hard Clays / Stiff Clay w/ Free Water (Reese)	183	173	119	4,000	--	--	30	0.005	1,500
4	Very Dense Sands with Silt / Sand (Reese)	173	130	130	--	36	--	36	--	125

[a] Pounds per cubic inch.
[b] For lateral load analysis only.

Assumed groundwater at El 228.

BODCAU CREEK BRIDGE INTERNAL BENTS 5 & 6 – BORINGS BC- 7 THROUGH BC-9

ASSUMED PILE CUTOFF ELEVATION: EL 230

ZONE	SOIL TYPES / LPILE SOIL[b]	DEPTH (ELEVATION)		WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD[b] PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[a]
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ' (DEGREE)		
1	Very Loose/Loose Sands / Sand (Reese)	230	218	120	--	28	--	28	--	10
2	Soft Clays / Stiff Clay w/ Free Water (Reese)	218	203	117	800	--	--	28	0.01	30
3	Loose/Medium Dense Sands / Sand (Reese)	203	187	124	--	34	--	34	--	60
4	Very Stiff/Hard Clays / Stiff Clay w/ Free Water (Reese)	187	167	120	4,000	--	--	30	0.004	1,500
5	Very Dense Sands with Silt	167	140	130	--	36	--	36	--	125

[a] Pounds per cubic inch.

[b] For lateral load analysis only.

Assumed groundwater at El 228.

BODCAU CREEK BRIDGE WEST ABUTMENT – BORINGS BC-1 & BC-2

ASSUMED PILE CUTOFF ELEVATION: EL 250

ZONE	SOIL TYPES / LPILE SOIL[b]	DEPTH (ELEVATION)		WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD[b] PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[a]
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)		
1	Loose Sands with Fines / Sand (Reese)	250	228	120	--	33	--	33	--	20
2	Soft/Very Soft Clays / Stiff Clay w/ Free Water (Reese)	228	202	116	900	--	--	28	0.02	30
3	Medium Dense Sands with Silt / Sand (Reese)	202	190	126	--	34	--	34	--	60
4	Loose Sands with Fines / Sand (Reese)	190	168	124	--	32	--	32	--	15
5	Very Dense Sands / Sand (Reese)	168	156	130	--	36	--	36	--	125

[a] Pounds per cubic inch.
[b] For lateral load analysis only.

Assumed groundwater at El 228.

BODCAU CREEK EAST ABUTMENT – BORINGS BC-9 THROUGH BC-11

ASSUMED PILE CUTOFF ELEVATION: EL 250

ZONE	SOIL TYPES / LPILE SOIL[b]	DEPTH (ELEVATION)		WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD[b] PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[a]
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)		
1	Loose/Very Loose Sands with Fines / Sand (Reese)	250	210	120	--	33	--	33	--	20
2	Soft Clays / Stiff Clay w/ Free Water (Reese)	210	200	118	500	--	--	28	0.01	100
3	Medium Dense Sand / Sand (Reese)	200	183	124	--	34	--	34	--	60
4	Hard Clays / Stiff Clay w/ Free Water (Reese)	183	173	120	2,400	--	--	30	0.005	750
5	Very Dense Sands / Sand (Reese)	173	160	130	--	36	--	36	--	125

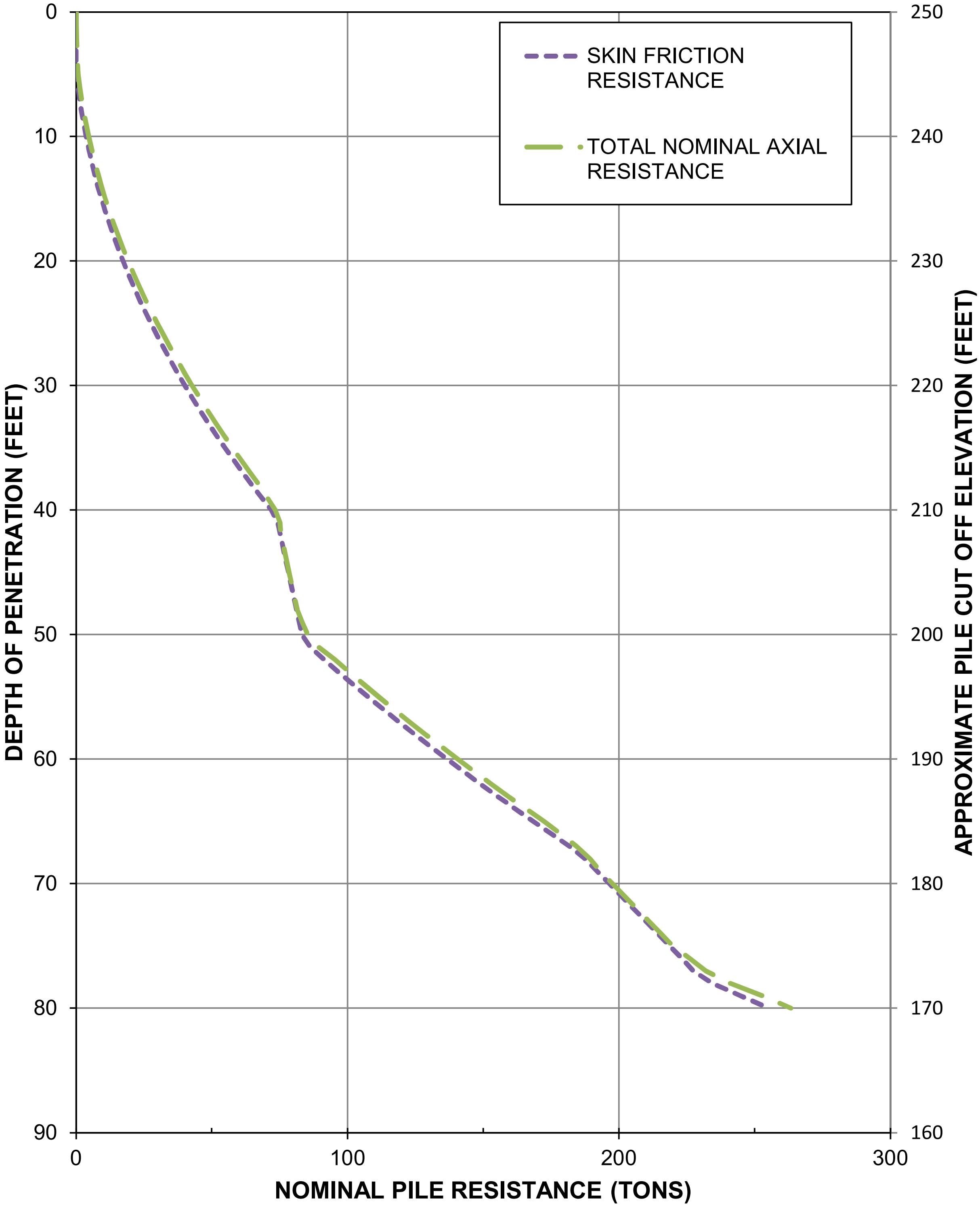
[a] Pounds per cubic inch.
[b] For lateral load analysis.

Assumed groundwater at El 228.

APPENDIX H – NOMINAL RESISTANCE CURVES

BODCAU CREEK BRIDGE EAST END BENT
ARDOT 030497 HWY 82

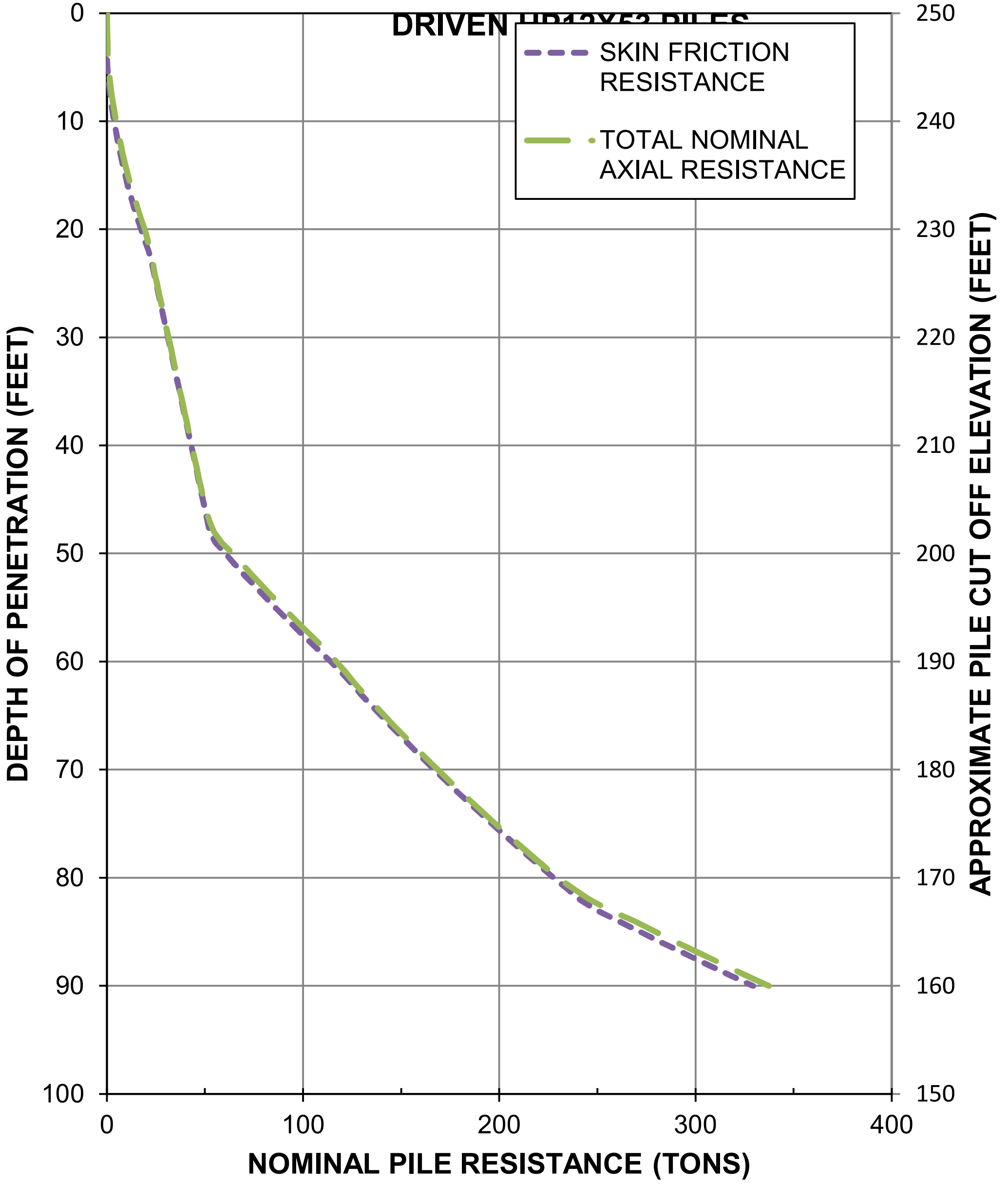
NOMINAL RESISTANCE CURVES
DRIVEN HP12X53 PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

BODCAU CREEK BRIDGE WEST END BENT
ARDOT 030497 HWY 82

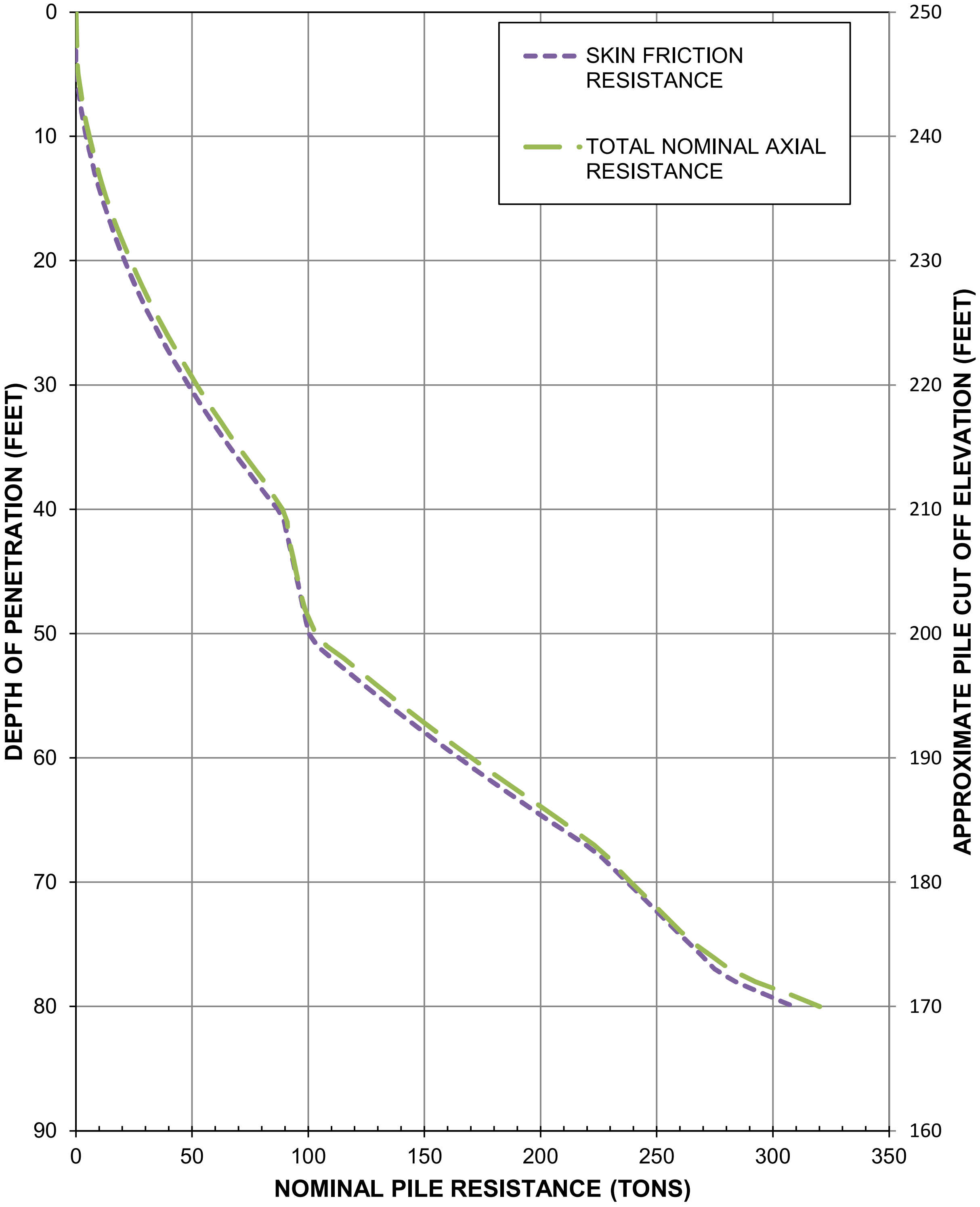
NOMINAL RESISTANCE CURVES
DRIVEN HP12X53 PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

BODCAU CREEK BRIDGE EAST END BENT
ARDOT 030497 HWY 82

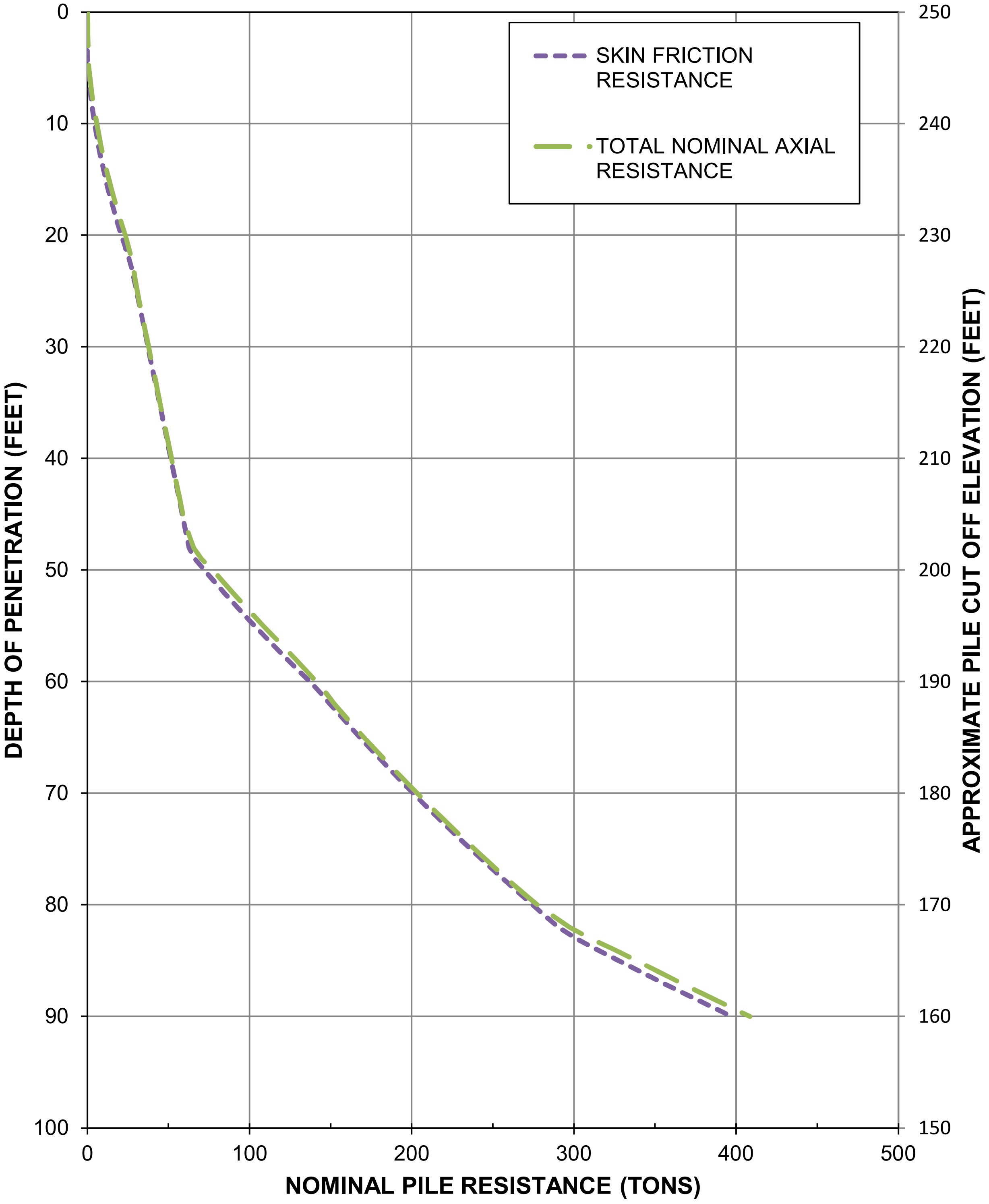
NOMINAL RESISTANCE CURVES
DRIVEN HP14X73 PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

BODCAU CREEK BRIDGE WEST END BENT
ARDOT 030497 HWY 82

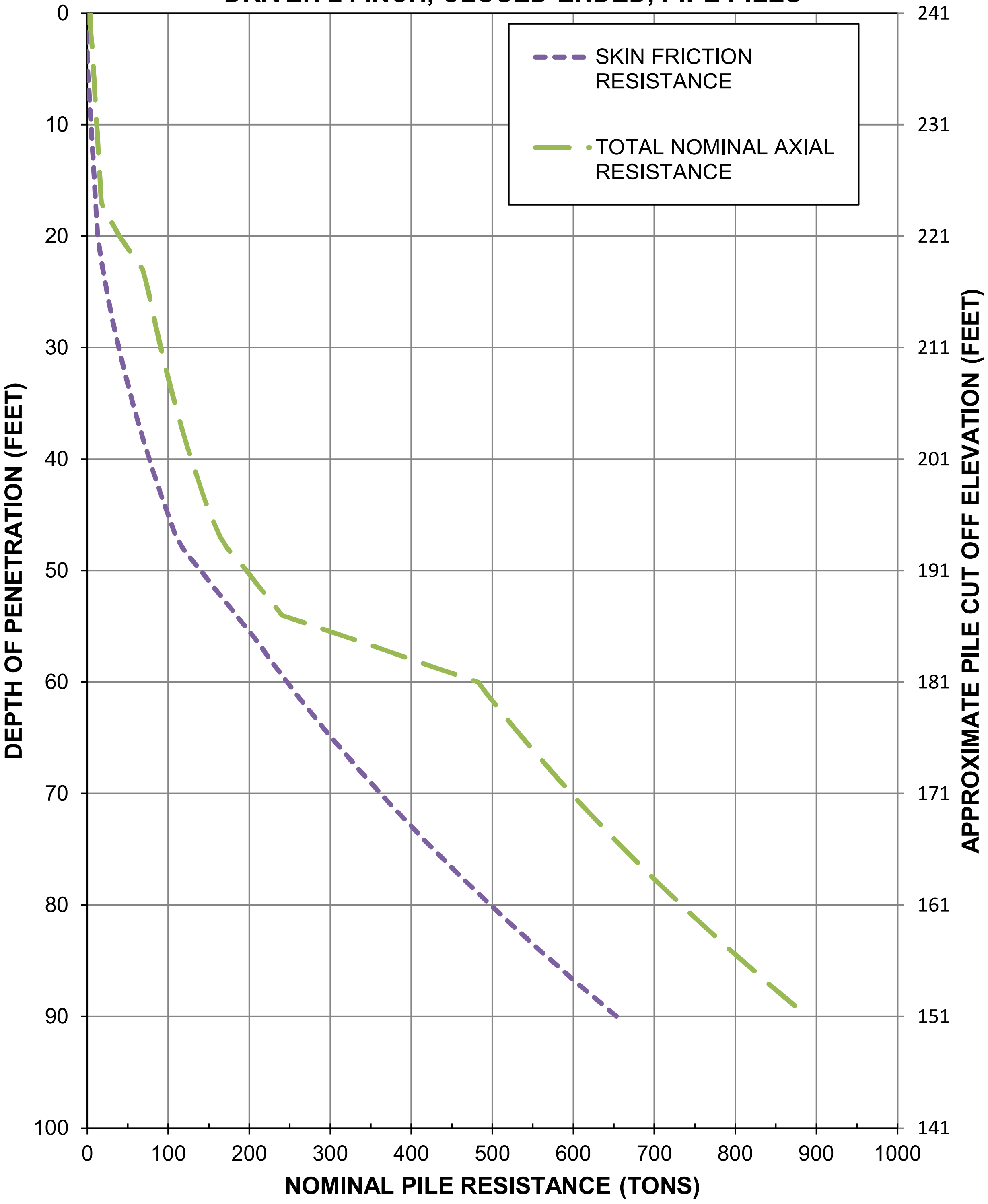
NOMINAL RESISTANCE CURVES
DRIVEN HP14X73 PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

BODCAU CREEK BRIDGE INTERMEDIATE BENTS 2, 3, 4
ARDOT 030497 HWY 82

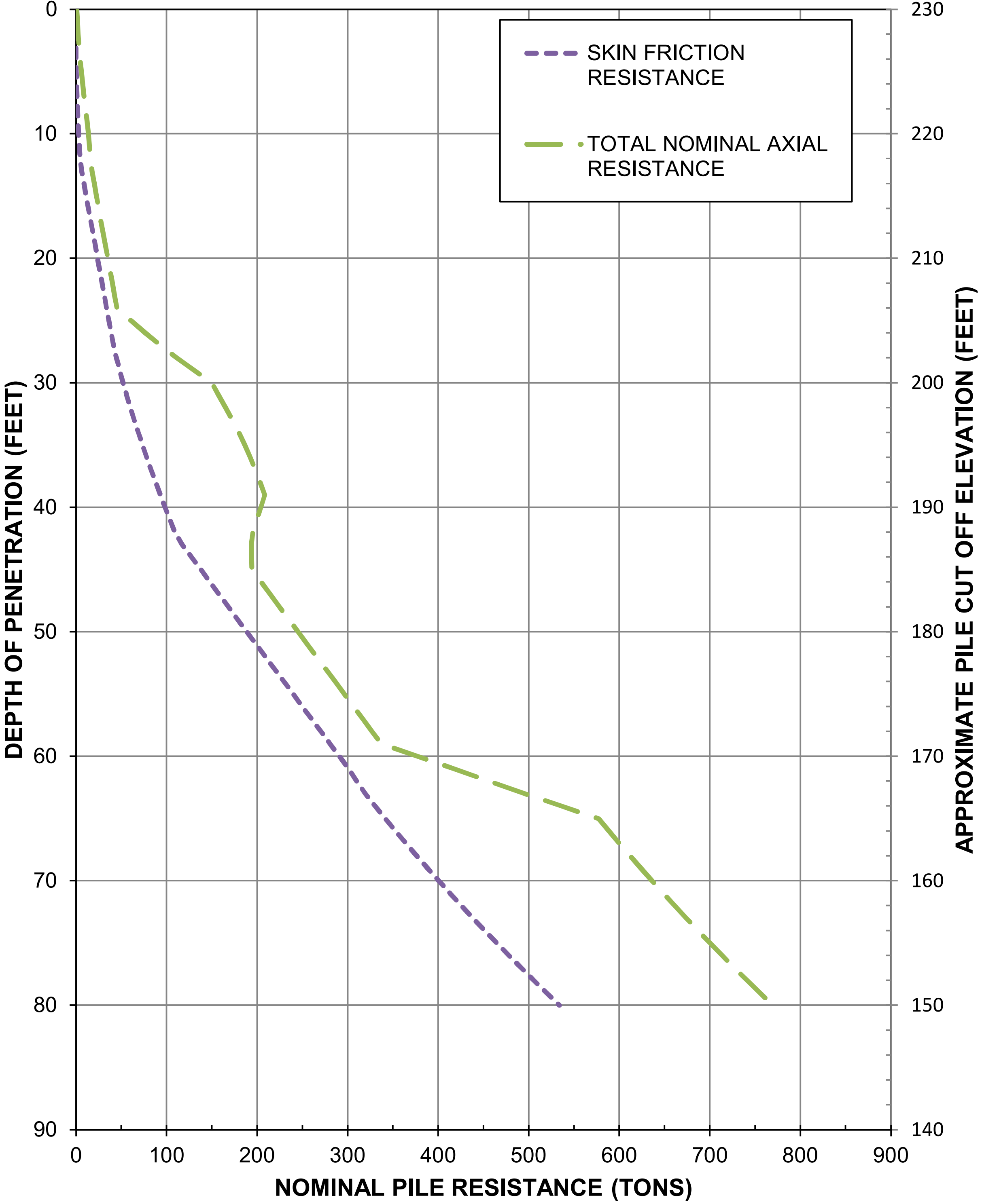
NOMINAL RESISTANCE CURVES
DRIVEN 24 INCH, CLOSED-ENDED, PIPE PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

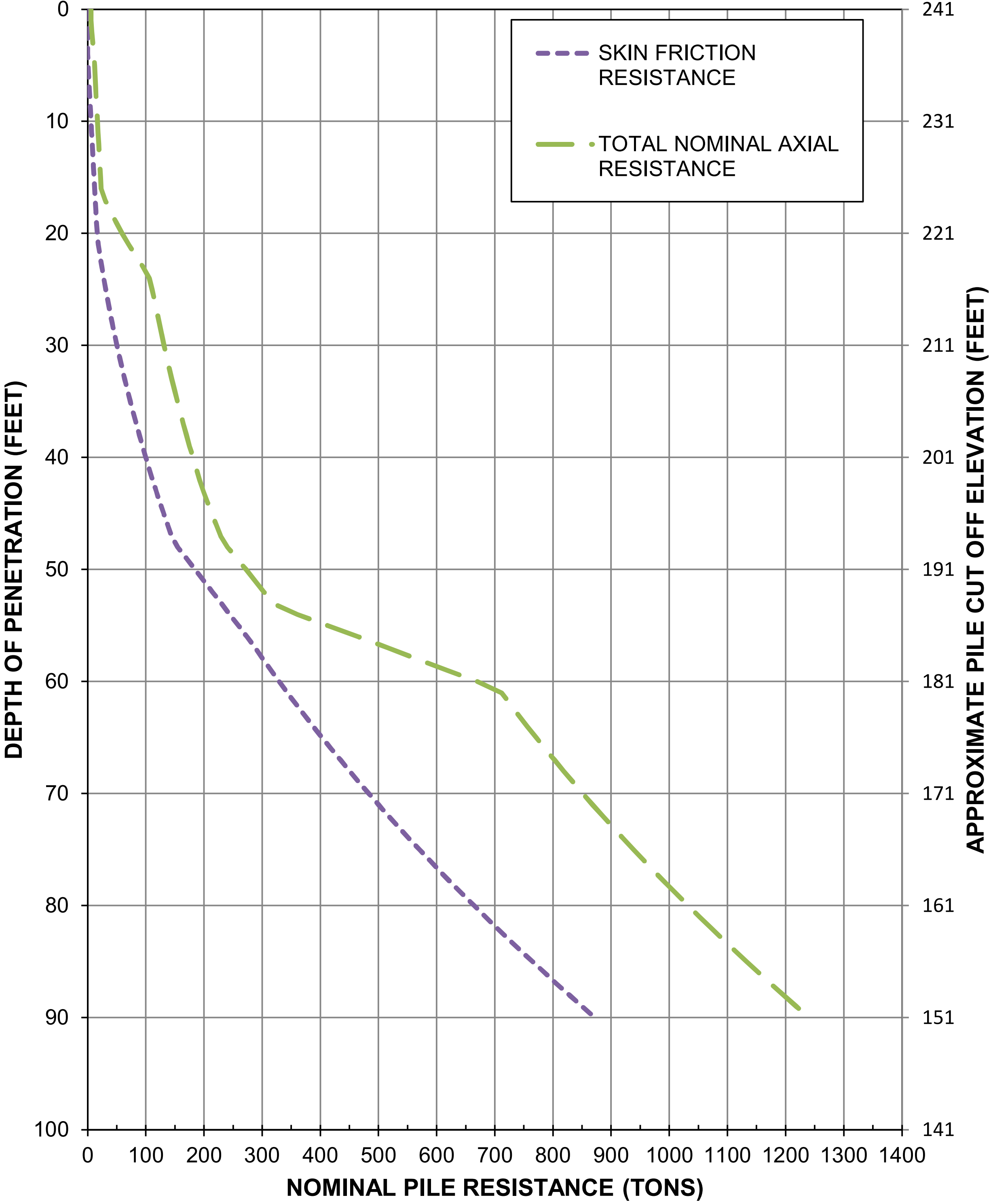
BODCAU CREEK BRIDGE INTERMEDIATE BENTS 5 & 6
ARDOT 030497 HWY 82

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



BODCAU CREEK BRIDGE INTERMEDIATE BENTS 3 & 4
ARDOT 030497 HWY 82

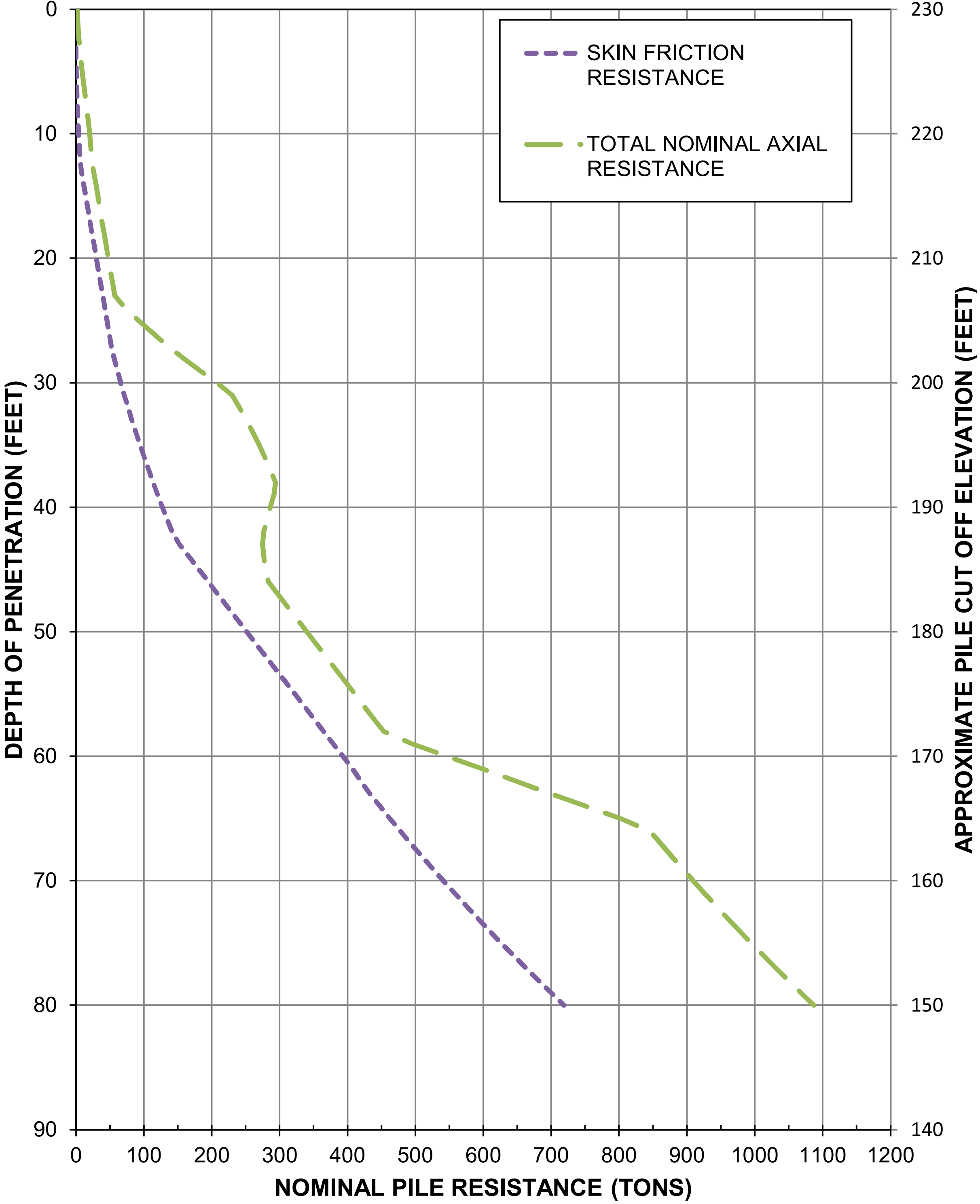
NOMINAL RESISTANCE CURVES
DRIVEN 30 INCH, CLOSED-ENDED, PIPE PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

BODCAU CREEK BRIDGE INTERMEDIATE BENT 5
ARDOT 030497 HWY 82

NOMINAL RESISTANCE CURVES
DRIVEN 30 INCH, CLOSED-ENDED, PIPE PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

GEOTECHNICAL EXPLORATION
HIGHWAY 82 STRS. & APPRS. (S)
BRIDGE OVER MILL CREEK
MILLER COUNTY, ARKANSAS

ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT NO. 030497

Prepared for:

GARVER, LLC
NORTH LITTLE ROCK

Prepared by:

GEOTECHNOLOGY, INC.
MEMPHIS, TENNESSEE

Date:

AUGUST 13, 2020

Geotechnology Project No.:

J028499.03B

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

GEOTECHNOLOGY INC
FROM THE GROUND UP

August 13, 2020

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

Re: Geotechnical Exploration
Highway 82 Strs. & Apprs. (S)
Bridge Over Mill Creek
Miller County, Arkansas
Geotechnology Project No. J028499.03B

Dear Mr. Ruddell:

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

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

Respectfully submitted,
GEOTECHNOLOGY, INC.

Dale M Smith

Dale M. Smith, P.E.
Geotechnical Manager

ALY/JDM/DBA/DMS/ASE:jdm

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GEOTECHNICAL EXPLORATION
HIGHWAY 82 STRS. & APPRS. (S)
BRIDGE OVER MILL CREEK
MILLER COUNTY, ARKANSAS

August 13, 2020 | Geotechnology Project No. J028499.03B

CHAPTER 1. SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design and construction, and other related features for the proposed improvements to Highway 82 (Hwy 82) in Miller County, Arkansas (Station 102+00.00 to Station 120+93.46). The referenced improvements consist of the replacement of Bridge No 02549 over Mill Creek. The new three-span bridge (Station 110+71.66 to Station 112+14.33) will be approximately 143-foot-long and constructed in two phases. During phase 1, a portion of the new bridge will be constructed to the south of the existing bridge. Facilitating traffic to the new bridge will require widening of the existing approaches. In phase 2 the existing bridge will be demolished; traffic will be redirected to the partially completed bridge, and the existing bridge will be demolished and the remaining portion of the bridge completed. When complete, the new bridge will be approximately 75 feet wide. The site location 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 10 borings were drilled at intervals along the proposed Highway 82 bridge over Mill Creek as shown in Figure 2. The boring logs, along with field and laboratory test results, are enclosed. The collected data have been analyzed and the physical properties of the in-situ soils summarized. General site conditions are discussed, along with recommendations for subgrade preparation. Important information prepared by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association for studies of this type is presented in Appendix A for your review.

CHAPTER 2. GENERAL INFORMATION

Planned Modifications

It is our understanding the existing bridge over Mill Creek will remain in use through the first phase of construction before being demolished and replaced in phase 2. The existing bridge approaches will be widened to facilitate traffic across the widened bridge.

The modifications to the approaches will require widening of the existing bridge approaches; beginning at Station 108+00.00, the existing road-way will be widened to the south to allow for five lanes of traffic (two in the eastbound and west-bound directions and one center turn lane).

Widening will end at the western bridge abutment at Station 110+71.66. The widening will require a wedge of fill to be placed on the southern shoulders of the existing road way between Station 108+00.00 and Station 110+71.66 with a maximum fill height of 10 feet at the bridge abutment. The planned side slopes of the western approach are 3 horizontal units for every 1 vertical unit (3H:1V) to the north and 2H:1V to the south.

The proposed bridge over Mill Creek will consist of an approximately 143-foot long, three-span bridge from Station 110+71.66 to 112+14.33. It is our understanding that minimal grade changes will be required at the bent locations. The bridge abutments will require up to 10 feet of fill and no cut. A 2H:1V slope is planned at the bridge abutment locations.

Widening of the eastern bridge approach will extend from the eastern bridge abutment at Station 112+14.33 to Station 114+92. The proposed widening will require a wedge of fill to be placed in the southern shoulder of the existing road way between Stations 112+14.33 and 114+92, with a maximum fill height of 10 feet occurring at the eastern bridge abutment. The planned side slopes of the eastern bridge approach are 3V:1H.

Topography

The proposed Hwy 82 bridge over Mill Creek is located in Miller County, Arkansas. According to the provided plans[1], the elevations at the west and east abutments are El 262.94[2] and 262.97, respectively with a maximum of approximately 19 feet of relief across the proposed alignment.

Drainage

The drainage system in the project area consists of the McKinney-Posten Bayous Watershed. The McKinney-Posten Bayous Watershed, in turn, is part of the overall drainage system of the Red River Basin.

Geology

Miller County is located in southwestern Arkansas, in the Gulf Coastal Plain. The Gulf Coastal Plain extends across the southern United States and is bounded to the north by the Ouachita Mountains. Approximately 50 million years ago, prior to tectonic uplift, the area was covered by the Gulf of Mexico. The Coastal Plain is characterized by flat to rolling topography.

The geology in the Mill Creek area is characterized by an upper layer of alluvium which features predominately alluvial deposits of present streams. Below the alluvium, the geology is generally

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[1] *Arkansas Department of Transportation Construction Plans for State Highway Mill & Bodcau Creeks STRS. & Apprs. (S) Miller and Lafayette Counties Route 82 Sections 1 & 2, Federal Aid Proj. NHPP-0046(50) Job 030497.* Provided by Garver on March 7, 2020.

[2] Elevations are referenced to NAVD 1988 (NAVD 88) in units of feet.

characterized by the Wilcox and Claiborne Groups which feature mainly non-marine sands, silty sands, clays and gravels. Some thick deposits of lignite are featured within both Groups.

CHAPTER 3. GEOTECHNICAL EXPLORATION

A total of ten borings were drilled at selected locations near the bridge approaches and the alignment of the proposed bridge. An additional boring, MC-11b, was drilled approximately 40 feet east from MC-11 due to split-spoon and auger refusal at 5 feet. The borings were drilled to approximate depths of 5 to 100 feet. Seven cores were performed through the existing pavement. Proposed Borings MC-4 and MC-5 were not drilled during exploration due to the presence of rip rap below the bridge and inability to access the sides of the bents.

The borings were drilled March 14, August 13, and August 20 through 22, 2019 using a rotary drill rig (CME 55 and CME 550X), hollow-stem augers and wet-rotary methods. Sampling procedures included Standard Penetration Test (SPT) and thin-wall (Shelby) tube methods. SPT’s were conducted at 2.5-, 5-, and 10-foot depth intervals using 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 along with an explanation of the terms and symbols used on the boring logs. Included on each boring log is the elevation estimated from the provided plans. Included in Table 1 are in situ tests and measurements made as part of the fieldwork and recorded on the boring logs.

Table 1. Field Tests and Measurements

Item	Test Method
Soil Classification	ASTM D 2488/ D 3282
Standard Penetration Test (SPT)	ASTM D 1586/ AASHTO T206
Thin-Walled (Shelby) Tube Sampling	ASTM D 1587/ AASHTO T207

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

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

CHAPTER 4. LABORATORY REVIEW AND TESTING

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

Table 2. Summary of Laboratory Tests and Methods.

Laboratory Test	ASTM	AASHTO
Moisture Content	D 2216	T 265
Atterberg Limits	D 4318	T 98
Grain Size Analysis	D 422	T 88
Unconsolidated-Undrained Triaxial Compression	D 2850	T 296
Direct Shear	D 3080	T 236
pH of Soil	D 4972	T 289
Soil Electrical Resistivity	G 57	T 288
Moisture-Density (Standard Effort)	D 698	T 99
California Bearing Ratio (CBR)	D 1883	T 193

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

CHAPTER 5. SUBSURFACE CONDITIONS

Existing Pavement

Borings MC-1, and MC-6 through MC-11 were drilled in the existing bridge approaches for the purpose of obtaining pavement thickness and subgrade information beneath the existing road way. A summary of the pavement materials and thicknesses is provided in Table 3.

Table 3. Summary of Encountered Pavement Materials and Thicknesses.

Boring No.	Surface		Base	
	Material	Thickness (in.)	Material	Thickness (in.)
MC-1	Asphalt	6	Sand and Gravel	6
MC-6	Asphalt	9	Silty Sand	3
MC-7	Asphalt	9	Sandy Silt	3
MC-8	Asphalt	6	Sand and Gravel	6
MC-9	Asphalt	8½	Sand and Gravel	3½
MC-10	Asphalt	4	Sand and Gravel	8
MC-11	Asphalt	8	Sand and Gravel	4

Subgrade Materials

The borings were drilled in the alignment of the proposed bridge and approaches, and were drilled through asphalt and approximately 3 to 6 inches of topsoil. Underlying the topsoil or pavement, the soils generally consisted of coarse-grained, predominately sandy soil underlain by layers of fine-grained soil which in turn was underlain by coarse-grained soils extending to the 100-foot maximum depth of exploration. The borings logs, with more detailed soil descriptions are included in Appendix C. The laboratory testing used to determine AASHTO and USCS classifications is presented in Appendix E.

The upper, interbedded coarse- and fine-grained soils were classified as poorly graded sand (SP), AASHTO A-3, A-1-b, high plasticity “fat” clay (CH), AASHTO A-2-7, and silt (ML) AASHTO A-4, with sand, clayey sand (SC) AASHTO A-4, A-6, and silty sand (SM) AASHTO A-2-4. Coarse-grained soils in the upper soils ranged from very loose to loose in consistency and fine-grained soils ranged from very soft to medium dense.

The upper sandy soils were underlain by fine-grained, predominately clay soils classified as low plasticity, “lean”, clay (CL), AASHTO A-6, A-2-7, silt (ML), AASHTO A-4, and high plasticity “fat” clay (CH), AASHTO A-2-7. Apparent very hard lignite was encountered at a depth of approximately 48.5 feet within the fine-grained soils in Boring MC-6. The fine-grained soils ranged in consistency from very soft to hard.

The fine-grained soils were underlain by coarse-grained soil classified as poorly-graded sand with silt (SP-SM), AASHTO A-1-b, A-3, A-2-4, and silty sand (SM), AASHTO A-2-4. Based on field test results, the coarse-grained soils were very dense in consistency.

Groundwater

Groundwater was encountered in Boring MC-3 at a depth of approximately 9 feet (El 241) while drilling and was not encountered in the other borings. Groundwater may have been masked by mud-rotary drilling operations. Groundwater levels could vary significantly over time due to the effects of seasonal variation in precipitation, recharge, flood levels in Mill Creek or other factors not evident at the time of exploration.

CHAPTER 6. ENGINEERING EVALUATION, ANALYSIS, AND RECOMMENDATIONS

Site Preparation and Earthwork

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

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

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

Side Slopes. Existing slopes steeper than 1V:4H should 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. Fill material consists of cohesive soil as indicated by Garver.

Cut Areas. It is our understanding up to 8 feet of cut will be required to achieve design grade at the existing eastern abutment. Based on the stratigraphy, excavations will terminate in silty sand, lean clay, fat clay, or silt. After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 95% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D 698/AASHTO T 99). Areas supporting pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

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

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

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

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

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

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

Pavement Design Information

Composite bulk samples of the auger cuttings were collected from selected borings. Atterberg limits and standard Proctor compaction tests (ASTM D 698/AASHTO T99) were performed on each composite sample. California Bearing Ratio (CBR) tests (ASTM D 1883/ AASHTO T193) were conducted on soaked samples remolded in standard CBR molds using compaction efforts of 25 and 56 blows per layer. The test results are summarized in Table 4.

Table 4. Summary of Compaction and CBR Test Results.

Boring No.	Depth (ft.)	USCS/ AAHSTO	Liquid Limit (%)	Plastic Limit (%)	Proctor Results		CBR Results				Percent Compaction (%)
					Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)	Blows per Layer	Dry Unit Weight (pcf)	Moisture Content (%)	CBR	
MC-9	1-5	CL A-6(6)	34	20	117.3	12.4	25	109.3	15.2	3.1	93.2
							56	116.2	12.8	8.5	99.1
MC-10	1-5	CL A-6(12)	36	21	127.1	10.4	25	113.9	13.3	6.5	89.6
							56	120.1	11.2	17.4	94.5

The results in the previous table were interpolated/extrapolated to estimate the CBR values at 95 percent compaction, which is typically considered a minimum compaction value to be achieved in the field. The mean and standard deviation of the interpretation were also calculated. The results are presented in Table 5.

Table 5. CBR Interpolation/Extrapolation.

Boring No.	Depth (ft.)	USCS/ AASHTO	CBR at 95% Compaction
MC-9	1 – 5	CL A-6(6)	4.9
MC-10	1 – 5	CL A-6(12)	18.5

Based on the test results and the data presented in the previous table, a CBR of 4.0 is recommended for design of pavements for this project. A CBR value of this magnitude will result in a relatively thick, expensive pavement structure. We recommend a 3-foot undercut below the base of pavements and backfilling with better (larger CBR) materials. Two materials are considered herein: A-4 (design CBR value of 8.0) and A-3 (design CBR value of 10.0).

The design CBR values mentioned in the previous paragraph were correlated to Resilient Modulus (M_R) and Resistance (R) values. The correlation was performed using a graph provided by ARDOT from AASHTO (1993) and is presented in Table 6.

Table 6. Soil Design Parameter Recommendations for Pavement Design.

	Soil Classification/Source		
	A-7-6 (In-Situ)	**A-4 (Import)**	**A-3 (Import)**
	CBR = 4	**CBR = 8**	**CBR = 10**
MR (psi)	2,900	4,400	5,00
Resistance (R) Value	9	20	25

Seismic Considerations

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

Earthquake Forces. It is our understanding the bridge and approaches will be designed in accordance with the AASHTO publication “LRFD Bridge Design Specifications”, eighth edition (2017), with 2017 interims.

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

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

Latitude 33.42894°N/Longitude 93.900380°W		
Category/ Parameter	Designation/ Value	Reference
Seismic Site Class	D	AASHTO LRFD 2017 Table 3.10.3.1-1
S_S	0.110g	Computed using design maps provided by the USGS (http://earthquake.usgs.gov/ws/designmaps) using the indicated latitude and longitude coordinates of the project site. The USGS tool used references AASHTO 2009.
S_1	0.047g	
F_a	1.600	
F_v	2.400	
F_{PGA}	1.600	
t_s	0.635	
t_0	0.127	
S_{DS}	0.177g	
S_{D1}	0.112g	
PGA	0.047g	
As	0.076g	

Liquefaction and Dynamic Settlement. A study was performed to evaluate the liquefaction and dynamic settlement potential at the site. Both field and laboratory data were used to perform the analysis. The field measurements included the depth of the water table and the SPT N-values. The laboratory data included USCS classification and soil unit weight. An earthquake magnitude (Mw) of 7.7 with a probability of exceedance of 7% in 75 years was considered. A site-adjusted peak ground acceleration, A_S, of 0.076g was utilized as obtained from the referenced Seismic Design Maps. Groundwater was assumed to be at approximately El 241.

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, the potential for liquefaction at the site is relatively low.

Due to the low potential for liquefaction at the site, downdrag on piles supporting project structures has not been considered.

Approach Embankment Settlement

Based on the cross sections provided and the proposed pile cap elevations, up to 10 feet of fill will be required at the proposed abutments to bring the site to grade. Up to 6 inches of settlement is estimated to occur under the weight of new fill placed at the bridge approaches and abutments.

We recommend a settlement monitoring program be implemented and survey data be forwarded to Geotechnology so that construction can commence as soon as settlement is essentially completed.

Settlement Monitoring Program. Settlement plates, or other appropriate methods should be utilized. Settlement plates should be installed approximately 1-foot below the existing ground surface and extended in 5-foot calibrated increments as the height of fill increases. To protect the riser pipes, fill should be hand compacted within a 4-foot radius of each plate. A typical settlement plate detail is presented on Figure 3 in Appendix B. We recommend settlement plates be placed no further than 50-feet apart, with at least one in the deepest areas of fill at both abutments. The project surveyor should be retained to monitor the settlement plate riser pipe. Settlement at the site should be measured twice weekly during fill placement and weekly after filling is completed. Further construction at the abutments should not commence until after the settlement due to the fill placement is practically complete. Provided the fill is placed in accordance with the Site Preparation and Earthwork section of this report, we anticipate fill induced settlement will be practically complete approximately four weeks after the finished grade is achieved.

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

Global Stability

Based on plans provided by Garver, the abutment slopes for the existing bridge are covered in rip rap and slope 1V:2H. 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 the following table. A pseudo-static seismic acceleration of 0.038g, corresponding to one-half the peak ground acceleration (per FHWA Publication HI-99-012) was utilized. Fill material consists of cohesive soils as provided by Garver; a water elevation of El 241, as obtained from the borings, was utilized for the short-term and seismic condition analyses and a water elevation of El 255.3, as obtained from the preliminary plans from Garver, was used for the long-term condition analyses. Section profiles with calculated critical failure arcs and utilized soil parameters are presented in Appendix F for the selected analyses. The models did not consider the effect of foundation piles driven at the abutments that would provide additional restraining force to stabilize the slopes.

Table 8. Results of Slope Stability Analyses.

Location	Description	Slope Height (ft.)	Calculated Factor of Safety		
			Short-Term Static[a,c]	Long-Term Static[a,d]	Seismic[b,c]
West Abutment	1V:2H Fill Slope	10	3.307	1.748	2.988
South Side Slope Station 110+00	1V:3H Fill Slope	10	3.872	1.703	3.271
East Abutment	1V:2H Fill Slope	10	3.129	2.732	2.839
Side Slope Station 112+50	1V:3H Fill Slope	8	3.734	1.625	3.371

[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.

[c] Based on a groundwater elevation of El 241 as obtained in the borings.

[d] Based on a groundwater elevation of El 255.3 as obtained by the preliminary plans provided by Garver.

Deep Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2017).

It is our understanding the proposed intermediate bents will be supported using 20-inch, closed-end, steel pipe piles and abutments (end bents) will be supported using either HP12x53 or HP14x73 H-piles. Intermediate bents have been designated as Bent 2 for the western bent and Bent 3 for the eastern bent for the analysis. Geotechnology should be notified if a different foundation type is to be considered. Synthetic profiles have been developed for the intermediate and end bents locations based upon the soil profile encountered in the borings, approximate boring elevations, and the proposed final grade. Nominal resistance curves showing the resistance due to skin friction and the total resistance (skin friction + end bearing) for the end and intermediate bents are presented in Appendix H. Uplift resistance (tension) may be calculated using the resistance provided by skin friction.

Resistance Factors. Resistance factors should be applied to the nominal resistances provided. In general, a factor of 0.45 may be used for piles in compression and 0.35 in tension. Based on AASHTO LRFD (2017) higher resistance factor may be used in accordance with the level 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 energies for each pile type are provided in Table 10.

Table 10. Minimum Hammer Energies.

Pile Size	Location	Embedment Length (feet)	Required Capacity (tons/kips)	Minimum Rated Hammer Energy (kip-feet)
12x53[a]	End Bents (Bents 1 and 4)	72	185 / 370	13
20”[b]	Intermediate Bents (Bents 2 and 3)	62	325 / 650	28

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

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 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 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.

Corrosion Potential. In addition to laboratory soil classification and strength testing, pH and soil resistivity testing was also conducted. The purpose of corrosion and soil resistivity testing is to provide soil data 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 11. Results of pH and Soil Resistivity Testing.

Boring	Sample No.	Sample Depth (foot)	pH	Soil Resistivity (ohm-cm)
MC-1	ST-3	5	4.91	17,100
MC-1	SS-5 – SS-8	13.5	--	7,980
MC-1	SS-14 – SS-17	58.5	--	1,824
MC-2	ST-4	8	4.53	10,545
MC-3	SS-3 – SS-6	6	--	3,420
MC-3	SS-8 – SS-10	28.5	--	1,311
MC-6	SS-4 – SS-8	8.5	--	9,690
MC-6	ST-5	10	3.90	11,400
MC-6	SS-14 – SS-16	53.5	--	1,197
MC-7	SS-4 – SS-7	8.5	--	9,120

Based on the results of the pH and soil resistivity testing and the criteria set forth in the AASHTO LRFD Bridge Design Specifications, low pH and resistivity were measured in multiple samples indicating strong corrosion or deterioration potential in the soils at the depths represented by these samples.

CHAPTER 7. RECOMMENDED ADDITIONAL SERVICES

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

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

CHAPTER 8. LIMITATIONS

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

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

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

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

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

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

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

APPENDIX A – IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

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

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

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

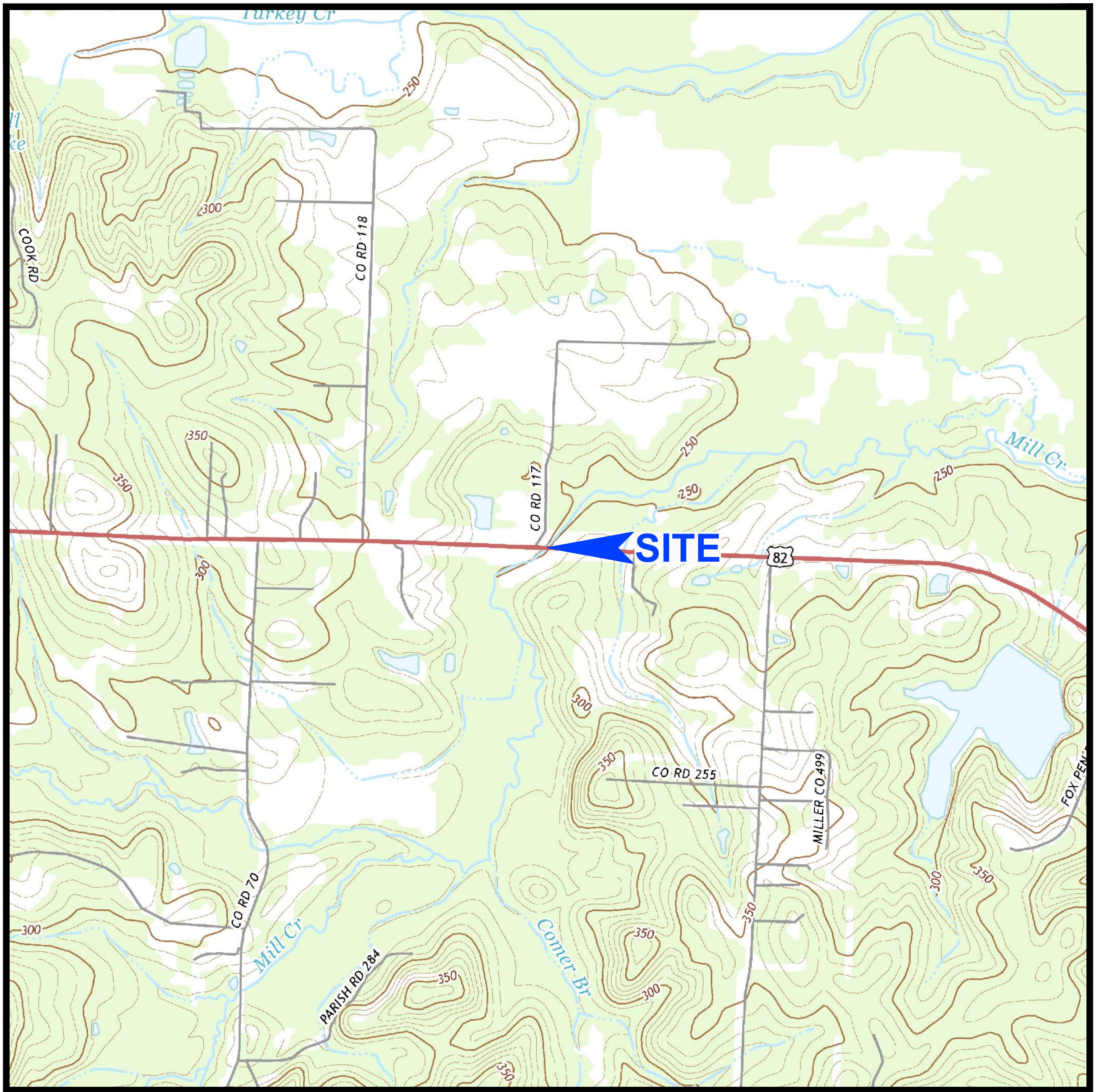
Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B – FIGURES

Figure 1 – Site Location and Topography

Figure 2 – Aerial Photograph of Site and Boring Locations

Figure 3 – Settlement Plate Detail



NOTES

1. Plan adapted from 7.5 minute U.S.G.S. maps for Lewisville and Old Town, Arkansas quadrangles, last revised in 2017.

0 2,000 4,000

SCALE IN FEET

N

Drawn By: WAH	Ck'd By: JDM	App'vd By: DMS
Date: 8-20-19	Date: 12-2-19	Date: 12-2-19

GEOTECHNOLOGY INC

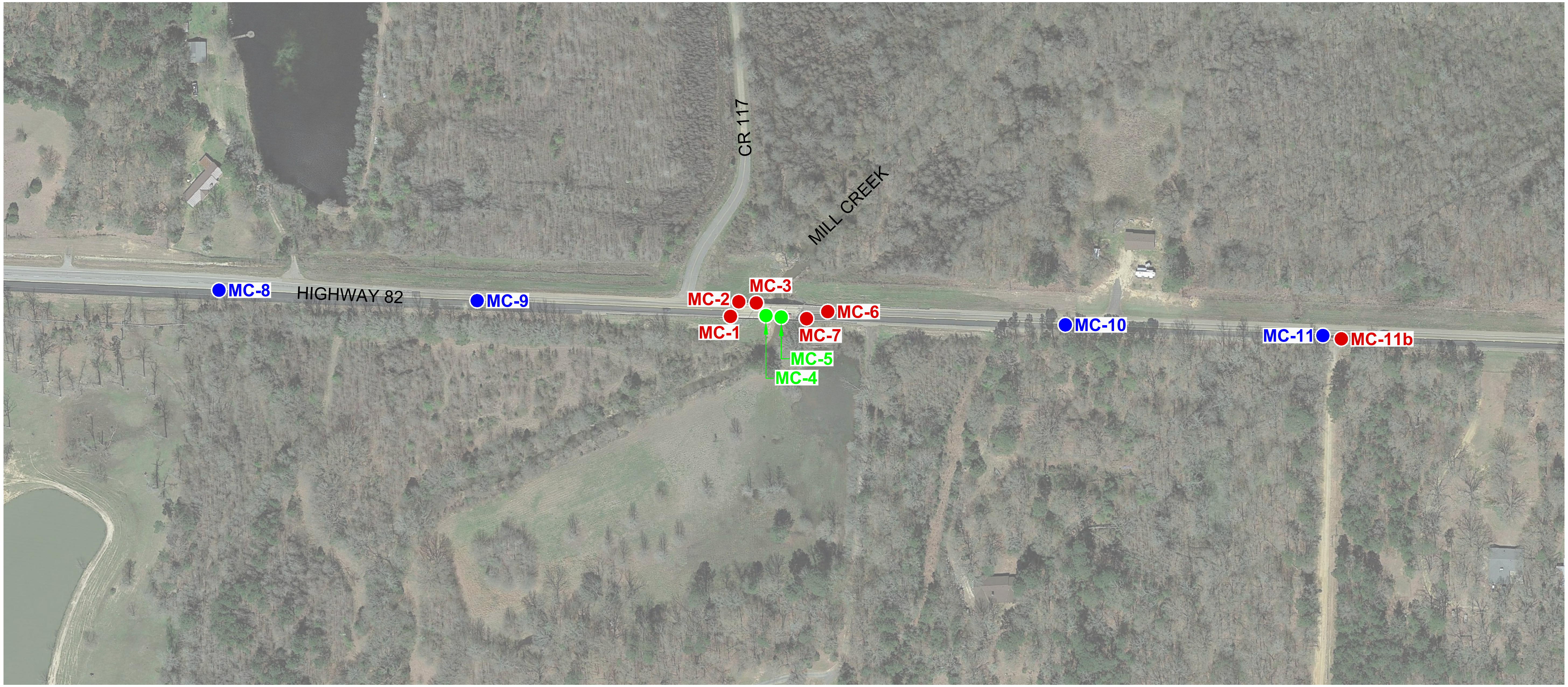
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County, Arkansas

SITE LOCATION AND TOPOGRAPHY

Project Number
J028499.03B

FIGURE 1



NOTES

1. Plan adapted from an January 20, 2014 aerial photograph courtesy of Google Earth.
2. Borings were located in the field with reference to site features and are shown approximate only.
3. Borings MC-4 and MC-5 were not drilled due to the presence of rip rap below the bridge and inability to access sides of bents.

LEGEND

🔴	Boring Location (August 2019)
🔵	Boring Location (March 2019)
🟢	Boring Locations Not Drilled Due to Inability to Access.

0 200 400

SCALE IN FEET

N

Drawn By: WAH	Ck'd By: JDM	App'vd By: DMS
Date: 8-20-19	Date: 12-2-19	Date: 12-2-19

GEOTECHNOLOGY INC

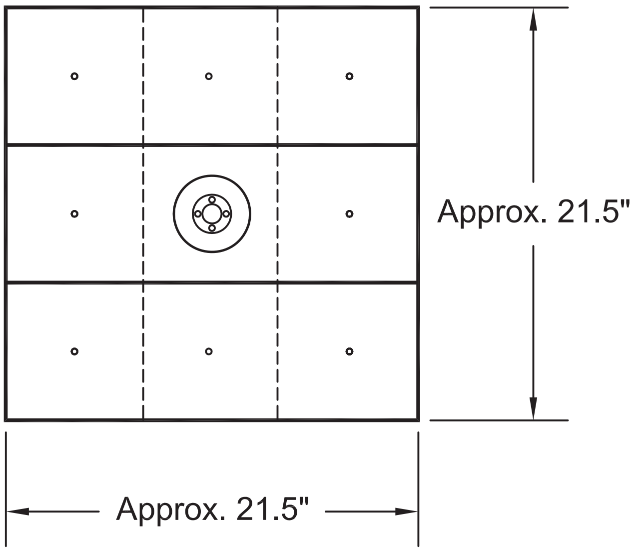
FROM THE GROUND UP

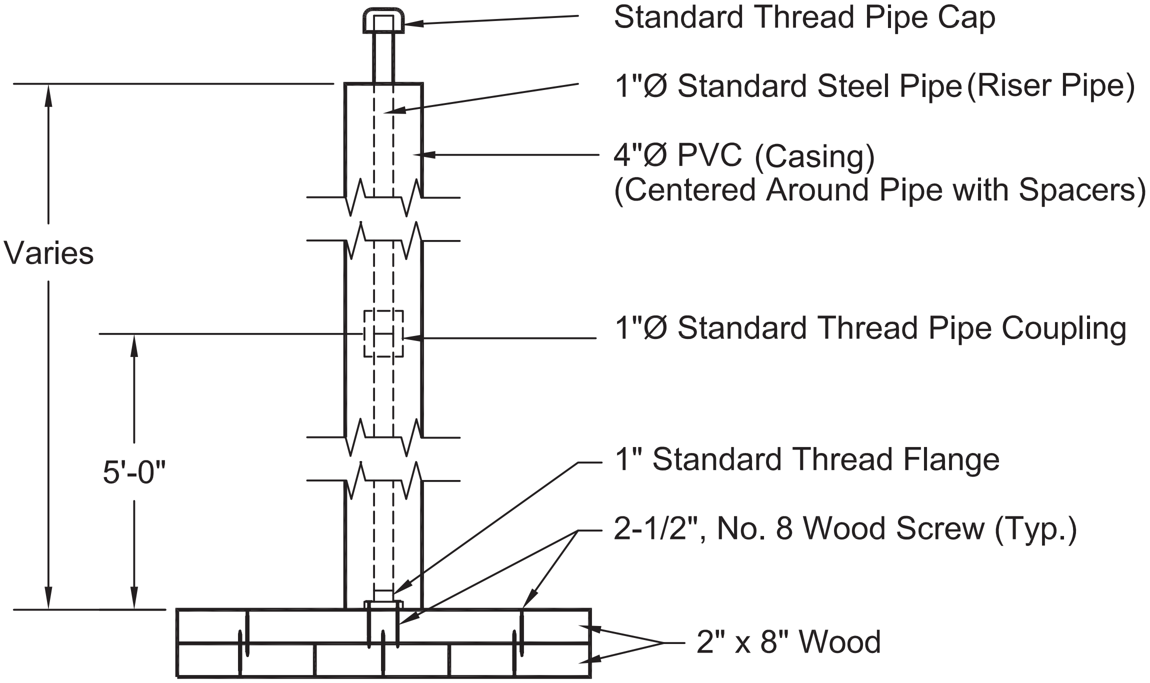
Mill Creek Bridge Replacement
Miller County, Arkansas

AERIAL PHOTOGRAPH OF SITE AND BORING LOCATIONS

Project Number
J028499.03B

FIGURE 2





NOTES

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

Drawn By: WAH	Ck'd By: FC	App'vd By: DMS
Date: 5/30/2019	Date: 5/30/2019	Date: 5/30/2019

GEOTECHNOLOGY INC
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County, Arkansas

SETTLEMENT PLATE DETAIL

Project Number J028499.03	FIGURE 3

APPENDIX C – BORING INFORMATION

Boring Logs

Boring Log Terms and Symbols

Surface Elevation: 260

Datum NAVD 88

Completion Date: 8/21/19

SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 ☐ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

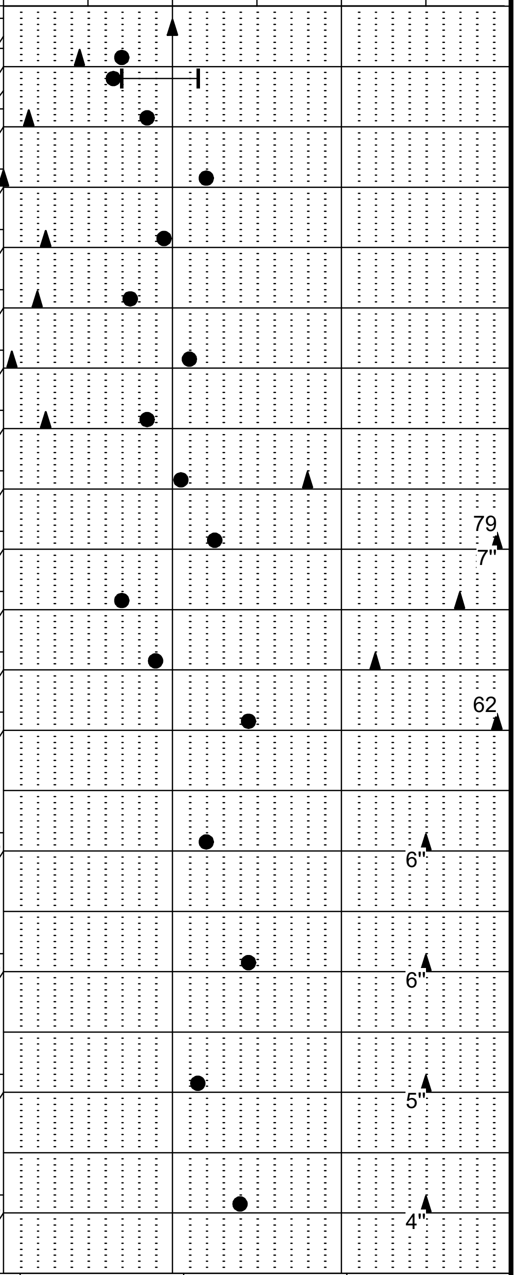
WATER CONTENT, %

PL ●— LL

10 20 30 40 50

DESCRIPTION OF MATERIAL

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	ASPHALT: 6 inches		
	Base Material: Black and brown sand, little silt and gravel	1-8-12	SS1
5	Medium dense, brown and black SAND, trace silt and gravel - SP	3-3-6	SS2
		116	ST3
10	Stiff, gray and brown, sandy, FAT CLAY - CH	1-1-2	SS4
	Gray and brown, CLAYEY SAND - (SC) 43.8% passing No. 200 sieve		
15	Very loose, brown, SILTY SAND - SM	0-0-0	SS5
	Very loose, gray, CLAYEY SAND - SC		
20	Medium stiff to very soft, brown and gray to gray, sandy SILT - ML 61.6% passing No. 200 sieve	2-1-4	SS6
25		2-1-3	SS7
30		0-0-1	SS8
35		1-2-3	SS9
40	Hard, gray to brown, FAT CLAY - CH sandy	8-16-20	SS10
45	silty trace black decayed organic material	12-29 -50/1"	SS11
50	Very dense, gray, SILTY SAND - SM	11-22-32	SS12
55	Dense, gray SAND with silt - SP-SM	12-19-25	SS13
60	Hard, gray, sandy, FAT CLAY - CH	25-33-29	SS14
65			
70	Very dense, gray, SILTY SAND - SM 21.3% passing No. 200 sieve	31-50/6"	SS15
75			
80		22-50/6"	SS16
85			
90		23-50/5"	SS17
95			
100	Boring terminated at 100 feet.	16-50/4"	SS18



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 10 FEET
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/23/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County

LOG OF BORING: MC- 1

Project No. J028499.03

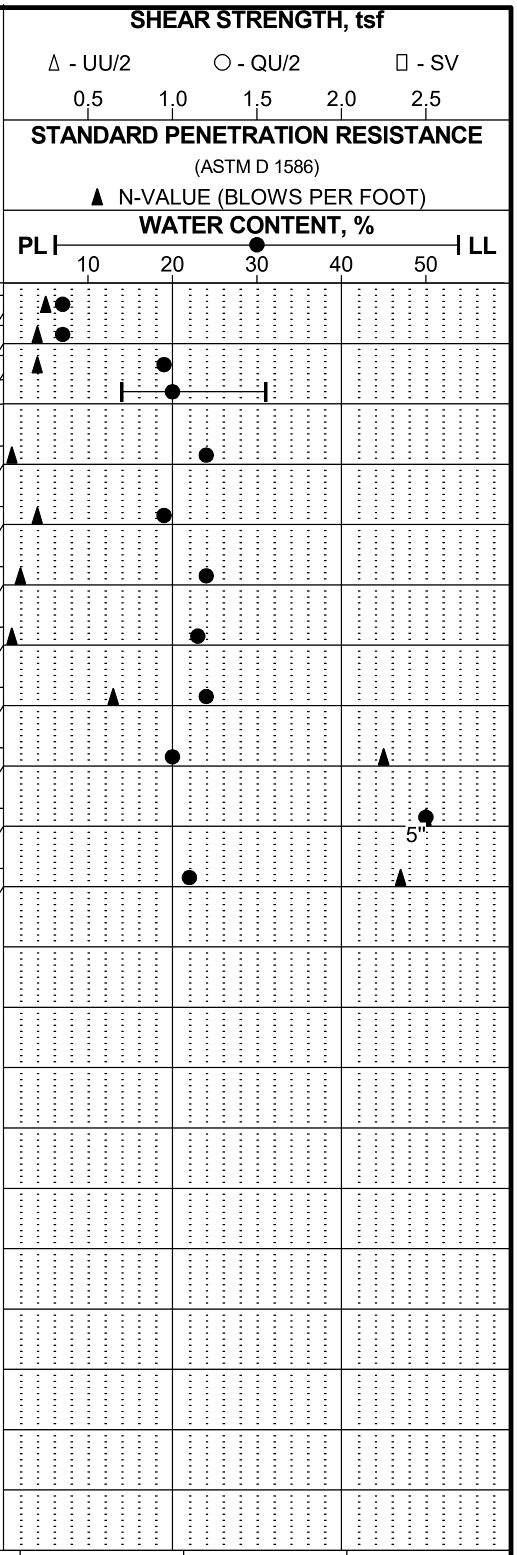
LOG OF BORING 2002 WL J028499.03 ARDOT 030497 - MILL CREEK.GPJ GTINC 0638301.GDT 12/6/19

Surface Elevation: 260

Datum NAVD 88

Completion Date: 8/13/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	TOPSOIL: 3 inches of grass with brown silt		
	Medium dense, brown, sandy, LEAN CLAY, trace gravel - CL	3-2-3	SS1
5	Soft, brown, sandy SILT - ML	2-1-3	SS2
	Soft to very soft, brown to gray, LEAN CLAY with sand - (CL)	2-2-2	SS3
10	71.3 % passing No. 200 sieve	128	ST4
15	70.7% passing No. 200 sieve	0-0-1	SS5
20	Very loose, brown, SILTY SAND - SM	3-2-2	SS6
25	Very loose, gray, CLAYEY SAND - SC	0-0-2	SS7
30	Very loose to very dense, gray to gray and black, SILTY SAND - (SM)	1-1-0	SS8
35	30% passing No. 200 sieve	2-4-9	SS9
40		12-19-26	SS10
45	trace organics	12-50/5"	SS11
50		11-20-27	SS12
	Boring terminated at 50 feet.		
55			
60			
65			
70			
75			
80			
85			
90			
95			
100			



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

LOG OF BORING 2002 WL J028499.03 ARDOT 030497 - MILL CREEK.GPJ GTINC 0638301.GPJ 12/6/19

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM __ FEET
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/14/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County

LOG OF BORING: MC- 2

Project No. J028499.03

Surface Elevation: 250

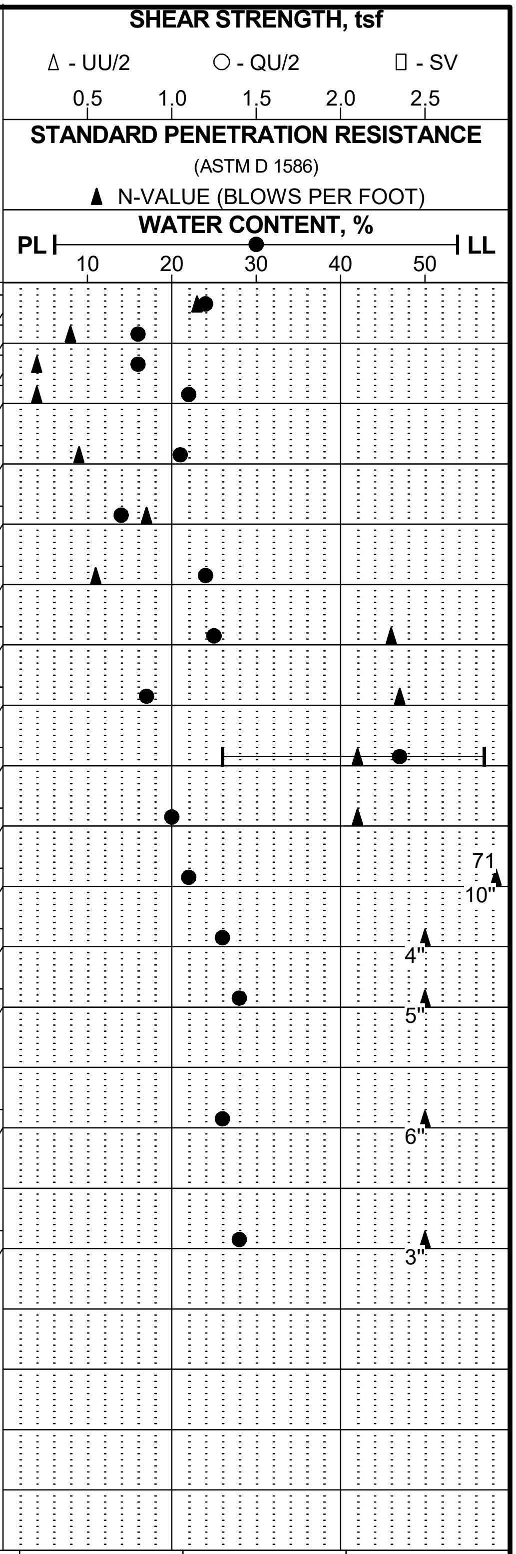
Completion Date: 8/20/19

Datum NAVD 88

DESCRIPTION OF MATERIAL

DEPTH IN FEET	DESCRIPTION OF MATERIAL
	TOPSOIL: 3 inches of grass with brown silt
	Very stiff, brown, sandy, FAT CLAY, trace roots and silt - CH
5	Medium stiff, brown and gray SILT, trace organics, clay and sand - ML
10	Soft to very stiff, brown and gray to tan and gray, sandy SILT - ML 60.7% passing No. 200 sieve trace clay
25	Medium dense to dense, gray, SILTY SAND, trace black decayed organic material - SM 49.9% passing No. 200 sieve
40	Hard, gray, sandy, FAT CLAY, trace black devayed organic material - (CH)
45	Dense to very dense, gray, SILTY SAND - SM
50	trace clay
55	Very dense, gray SAND with silt - SP-SM
70	Very dense, gray SAND - SP
80	Very dense, gray SAND with silt - SP-SM Boring terminated at 80 feet.

DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
14-13-10	SS1
5-4-4	SS2
1-2-2	SS3
1-2-2	SS4
4-5-4	SS5
5-11-6	SS6
1-2-9	SS7
9-18-28	SS8
15-23-24	SS9
9-15-27	SS10
14-20-22	SS11
15-21 -50/4"	SS12
18-50/4"	SS13
21-50/5"	SS14
18-50/6"	SS15
30-50/3"	SS16



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 9 FEET ∇

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 10 FEET
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/23/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County

LOG OF BORING: MC- 3

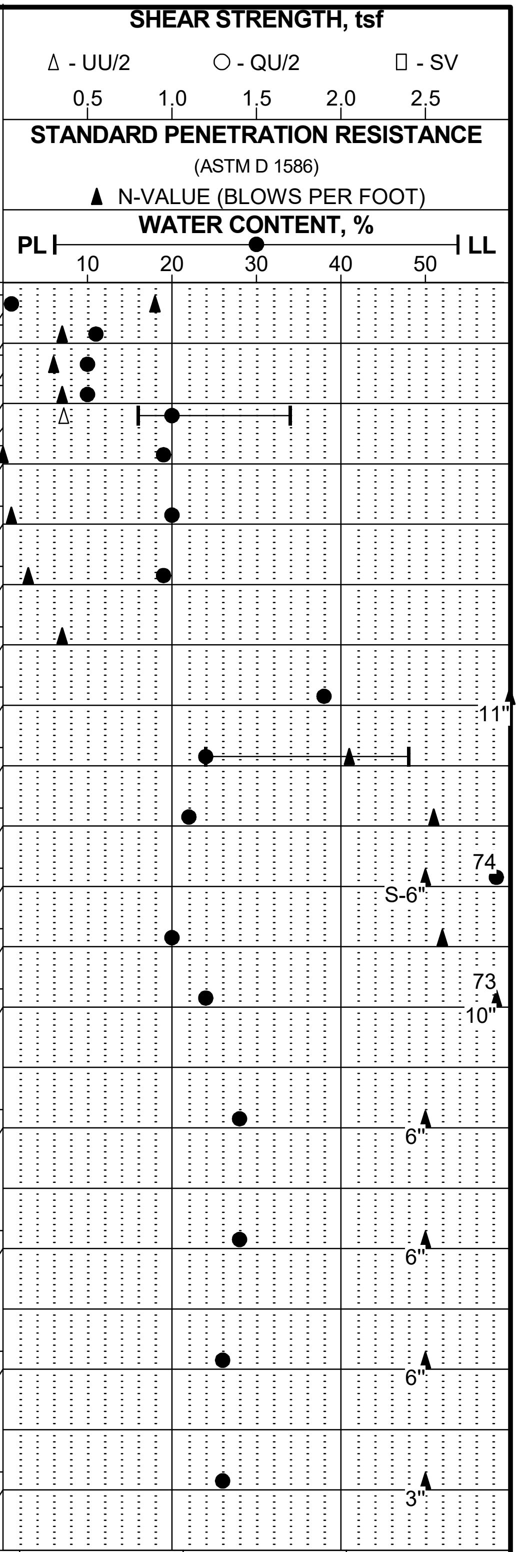
Project No. J028499.03

Surface Elevation: 260

Datum NAVD 88

Completion Date: 8/22/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	ASPHALT: 9 inches		
	Base Material: Brown and gray silt, little sand and gravel	9-12-6	SS1
5	Medium dense, gray and black SAND, little silt and gravel - SP	2-4-3	SS2
		3-3-3	SS3
10	Loose, orange and tan to red and orange, SILTY SAND - SM trace clay	3-3-4	SS4
		107	ST5
15	Loose, red, CLAYEY SAND, trace silt - (SC) 38.4% passing No. 200 sieve	0-0-0	SS6
20	Very soft, gray, sandy SILT - ML 71.1% passing No. 200 sieve trace clay	0-0-1	SS7
25		1-1-2	SS8
30	Loose to very dense, tan and gray SAND - SP	3-3-4	SS9
35	trace gravel and black decayed organic material	9-10 -50/5"	SS10
40	Hard, gray, LEAN CLAY, trace sand - (CL)	10-17-24	SS11
45		13-20-31	SS12
50	Very dense, black decayed LIGNITE	50/6"	SS13
55	Hard, gray, silty, sandy, FAT CLAY - CH	13-21-31	SS14
60	Very dense, gray, SILTY SAND - SM 43.0% passing No. 200 sieve	13-23 -50/4"	SS15
65			
70		19-50/6"	SS16
75			
80	Very dense, gray SAND with silt - SP-SM	24-50/6"	SS17
85			
90		24-50/6"	SS18
95			
100		26-50/3"	SS19
	Boring terminated at 100 feet.		



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM

WASHBORING FROM 10 FEET

BMF DRILLER JDM LOGGER

CME 550X DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/23/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County

LOG OF BORING: MC- 6

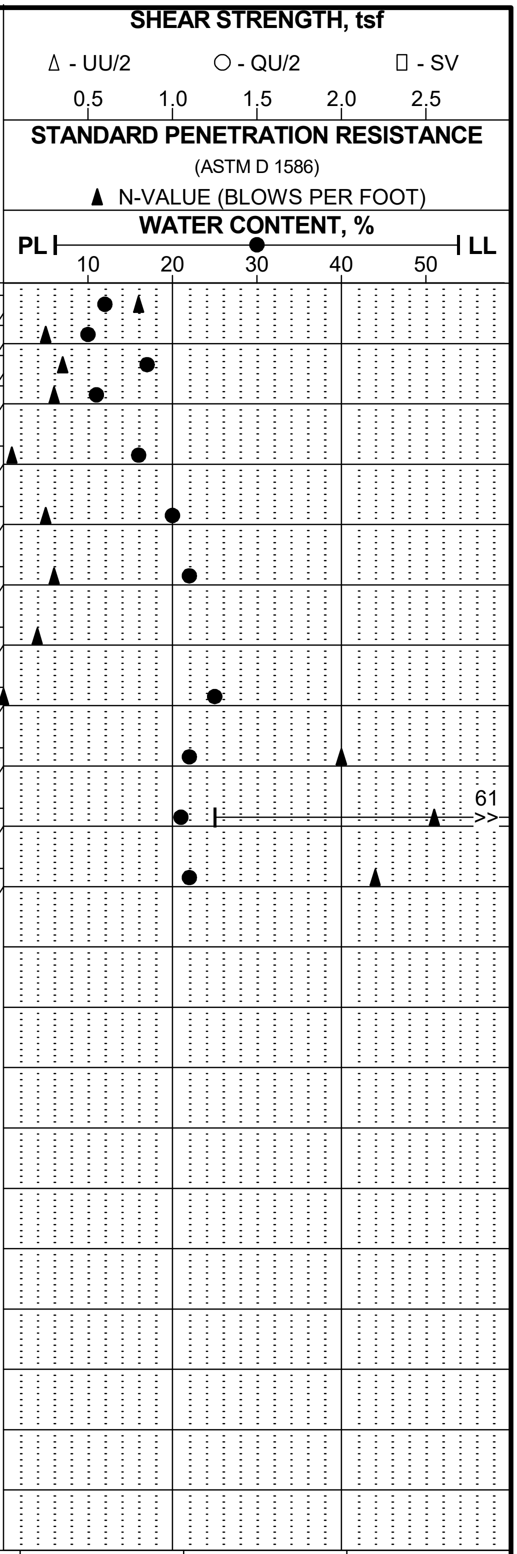
Project No. J028499.03

Surface Elevation: 260

Datum NAVD 88

Completion Date: 8/22/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	ASPHALT: 9 inches			
	Base Material: Black and brown sand with silt and gravel		5-7-9	SS1
5	Medium dense to very loose, tan and orange to gray and white, SILTY SAND, trace clay - SM		3-2-3	SS2
			3-3-4	SS3
10			3-3-3	SS4
15			1-0-1	SS5
20	Medium stiff, gray and white, sandy SILT - ML 50.3% passing No. 200 sieve		0-2-3	SS6
25			3-3-3	SS7
30	Very loose, brown and gray SAND - SP		2-2-2	SS8
35	Very loose to dense, gray, SILTY SAND, little clay - SM 42.3% passing No. 200 sieve		0-0-0	SS9
40			9-20-20	SS10
45	Hard, gray, sandy, silty, FAT CLAY - (CH)		14-20-31	SS11
50	little black decayed organic material		35-24-20	SS12
	Boring terminated at 50 feet.			
55				
60				
65				
70				
75				
80				
85				
90				
95				
100				



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

LOG OF BORING 2002 WL J028499.03 ARDOT 030497 - MILL CREEK.GPJ GTINC 0638301.GPJ 12/5/19

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM 10 FEET
BMF DRILLER JDM LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 8/23/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County

LOG OF BORING: MC- 7

Project No. J028499.03

Surface Elevation: 260

Datum NAVD 88

Completion Date: 3/14/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	ASPHALT: 6 inches		
	Base Material: Black sand, trace gravel and silt	16-8-7	SS1
5	Stiff, brown, sandy, LEAN CLAY - CL	2-1-1	SS2
	Very loose, brown, silty SAND, trace clay - SM	0-5-10	SS3
10	Medium dense, tan SAND - SP	1-1-3	SS4
	Very loose to loose, tan to tan and gray, silty SAND - SM trace clay		
15		4-4-7	SS5
	Boring terminated at 15 feet.		

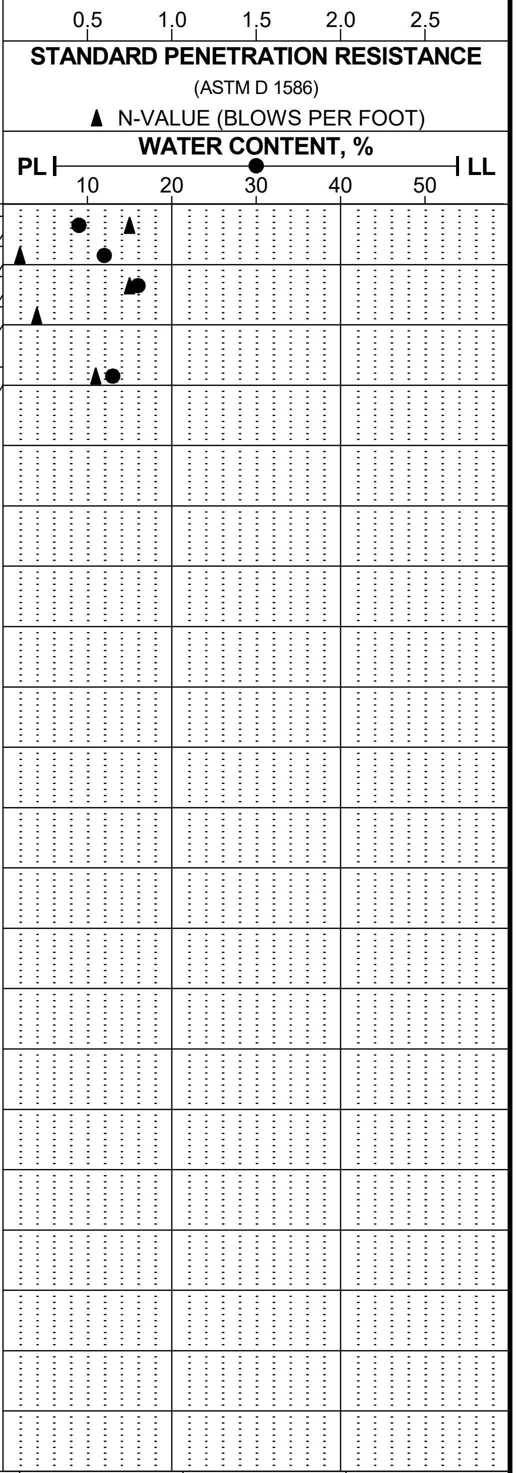
SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 ▯ - SV

STANDARD PENETRATION RESISTANCE (ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

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 3/4 HOLLOW STEM

WASHBORING FROM __ FEET

BMF DRILLER JDM LOGGER

CME 55 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 90 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 3/18/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County

LOG OF BORING: MC- 8

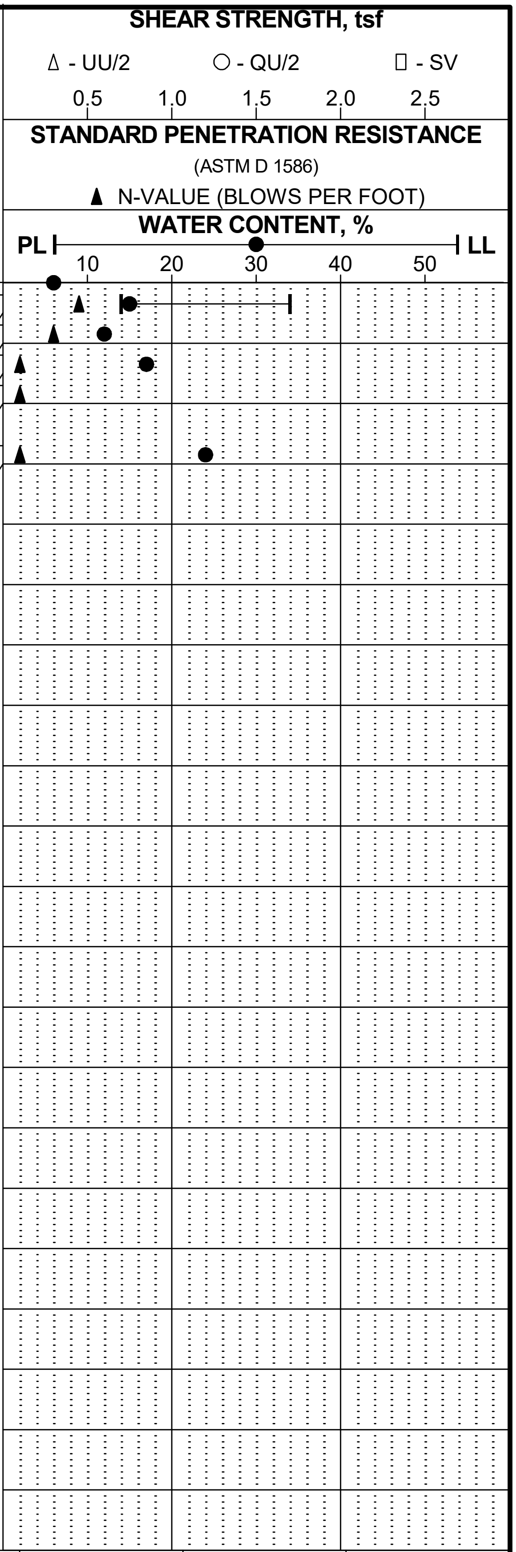
Project No. J028499.03

Surface Elevation: 260

Datum NAVD 88

Completion Date: 3/14/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	ASPHALT: 8.5 inches			
	Base Material: Black sand, trace gravel and silt		3-4-5	SS1
5	Stiff, brown, sandy, LEAN CLAY - CL 50.6% passing No. 200 sieve		0-3-3	SS2
			2-1-1	SS3
10	Tan and gray, clayey SAND, little silt - SC		1-1-1	SS4
	Medium stiff to soft, brown to tan and gray, sandy, FAT CLAY - CH			
15	Soft, gray and tan, LEAN CLAY, little sand - CL		1-1-1	SS5
	Boring terminated at 15 feet.			
20				
25				
30				
35				
40				
45				
50				
55				
60				
65				
70				
75				
80				
85				
90				
95				
100				



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

LOG OF BORING 2002 WL J028499.03 ARDOT 030497 - MILL CREEK.GPJ GTINC 0638301.GPJ 12/6/19

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM __ FEET
BMF DRILLER JDM LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 90 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 3/18/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County

LOG OF BORING: MC- 9

Project No. J028499.03

Surface Elevation: 260

Datum NAVD 88

Completion Date: 3/14/19

DESCRIPTION OF MATERIAL

DEPTH IN FEET

ASPHALT: 4 inches

Base Material: Black sand, trace gravel and silt

Stiff, tan and gray, LEAN CLAY with sand, trace gravel - (CL)
4.7% Gravel
74.6% passing No. 200 sieve

Medium dense, tan and gray SAND with clay, trace gravel - SP-SC

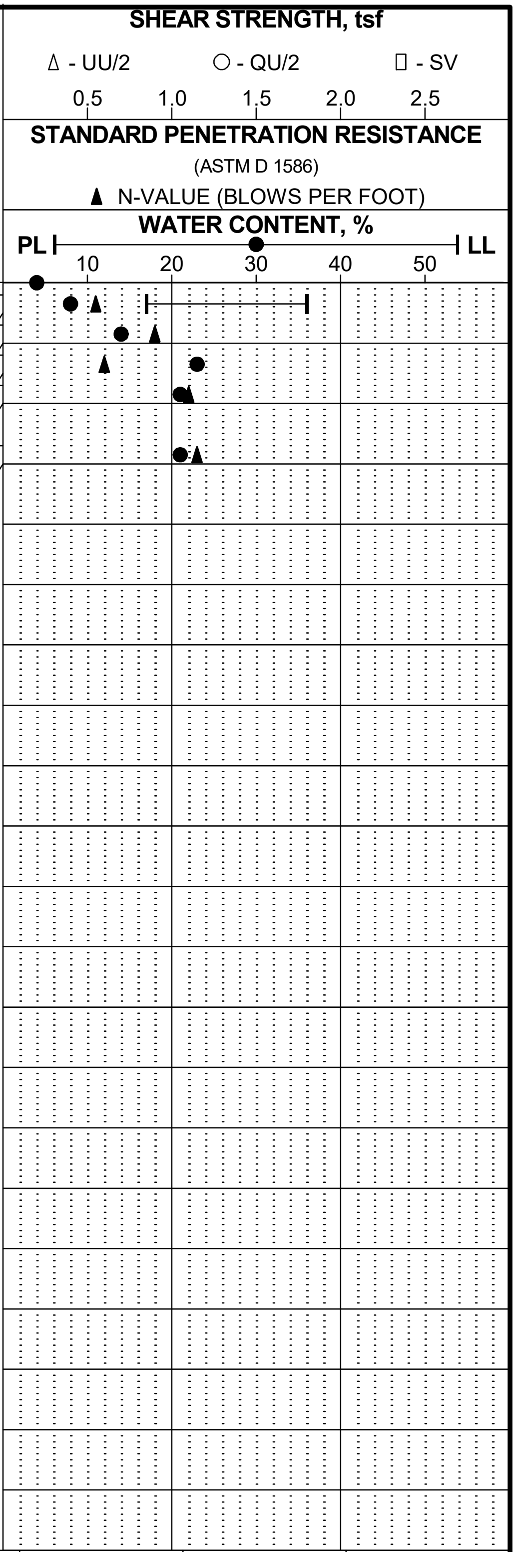
Medium dense, gray, silty SAND - SM

Medium dense, gray and tan SAND, trace clay - SP

Very stiff, gray, FAT CLAY - CH

Boring terminated at 15 feet.

DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
5-6-5	SS1
3-9-9	SS2
2-6-6	SS3
3-10-12	SS4
8-11-12	SS5



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 2002 WL J028499.03 ARDOT 030497 - MILL CREEK.GPJ GTINC 0638301.GPJ 12/6/19

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM __ FEET
BMF DRILLER JDM LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 90 %

REMARKS:

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 3/18/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County

LOG OF BORING: MC-10

Project No. J028499.03

Surface Elevation: 260

Completion Date: 3/14/19

Datum NAVD 88

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	ASPHALT: 8 inches		
	Base Material: Black sand, trace gravel and silt	3-6-6	SS1
5	Stiff, gray, sandy, LEAN CLAY - CL	3-7-50/1"	SS2
	Medium dense, gray, silty SAND, trace clay - SM		
10	Split-spoon refusal at approximately 4.75 feet.		

SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 ▯ - SV

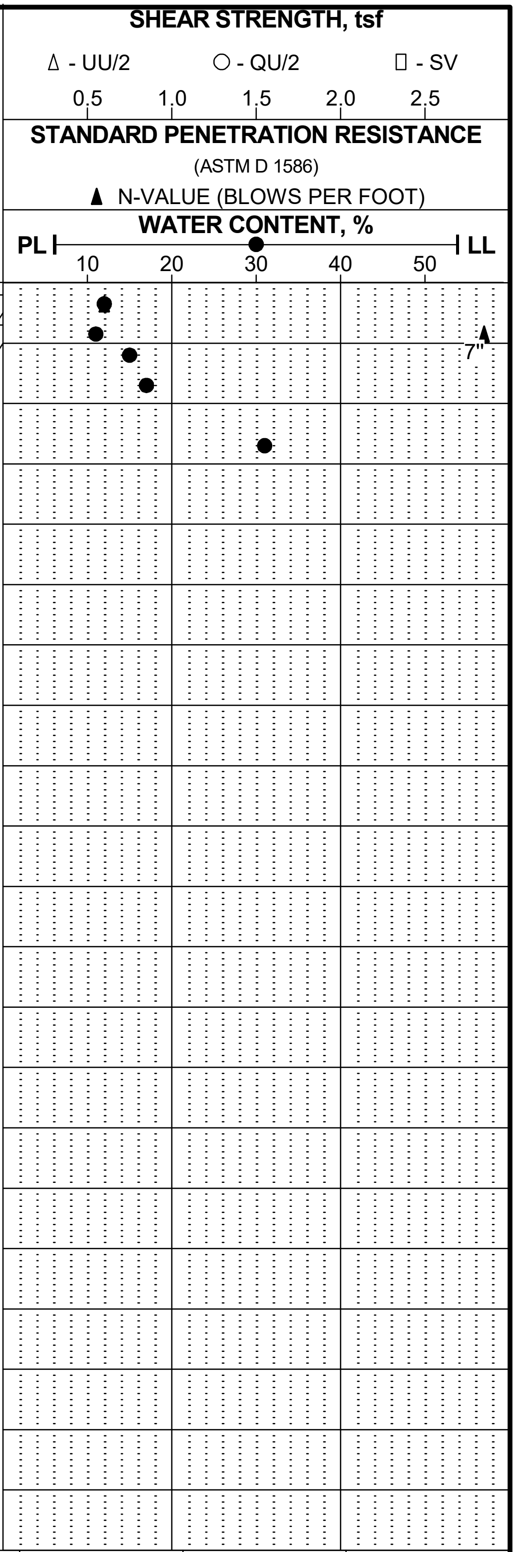
STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ├──●──┤ LL



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM

WASHBORING FROM __ FEET

BMF DRILLER JDM LOGGER

CME 55 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 90 %

REMARKS: Auger refusal encountered at approximately 4.75 feet.

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 3/18/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC

FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County

LOG OF BORING: MC-11

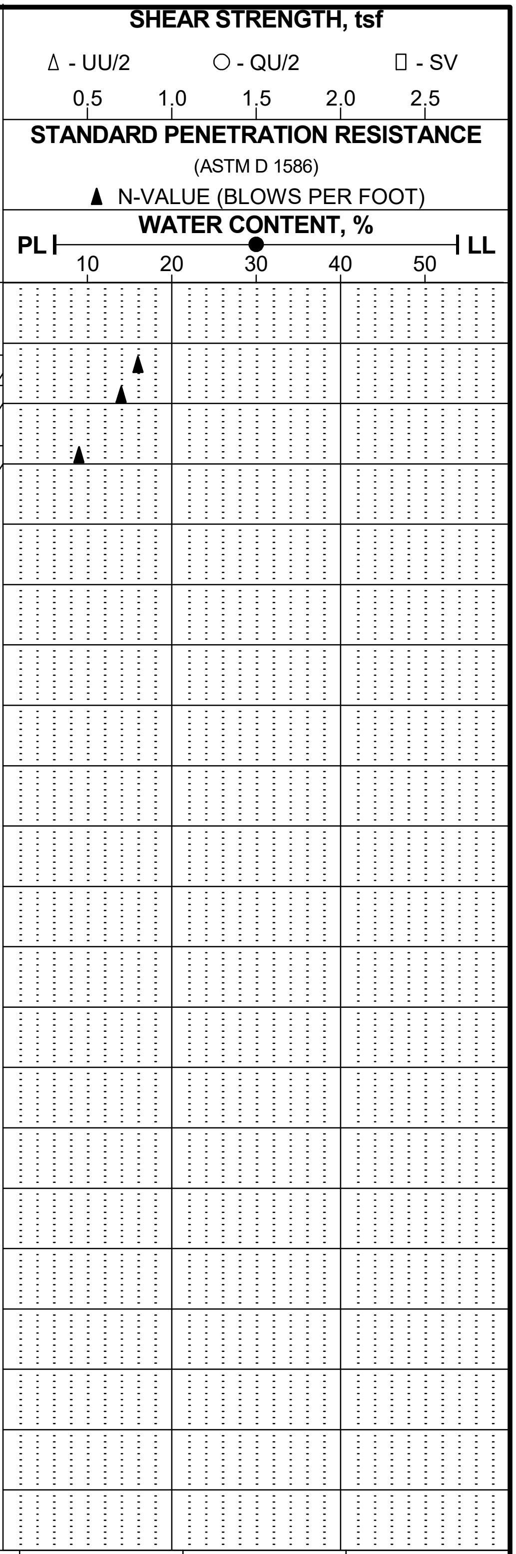
Project No. J028499.03

Surface Elevation: 260

Datum NAVD 88

Completion Date: 3/14/19

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
0–5	Refer to Boring MC-11.			
5–13	Medium dense, brown and gray to gray and orange, silty SAND, trace gravel - SM		6-8-8	SS3
			3-6-8	SS4
13	∇			
14–15	Stiff, gray and tan, FAT CLAY - CH		5-3-6	SS5
15	Boring terminated at 15 feet.			



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

GROUNDWATER DATA

ENCOUNTERED AT 13 FEET ∇

DRILLING DATA

__ AUGER 3 3/4 HOLLOW STEM
WASHBORING FROM __ FEET
BMF DRILLER JDM LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 90 %

REMARKS: Boring offset approximately 40 feet east of MC-11 and continued to full depth of exploration.

Drawn by: JDM	Checked by: ASM	App'vd. by: DMS
Date: 3/18/19	Date: 11/4/19	Date: 11/4/19

GEOTECHNOLOGY INC
FROM THE GROUND UP

Mill Creek Bridge Replacement
Miller County

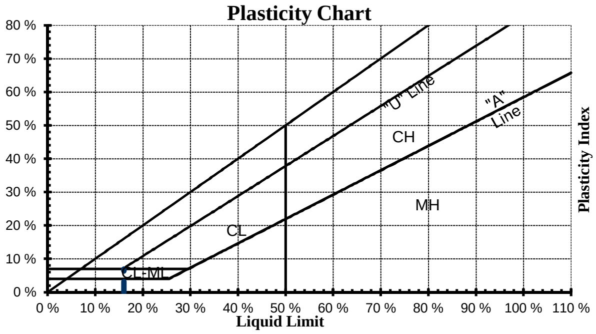
LOG OF BORING: MC-11b

Project No. J028499.03

BORING LOG: TERMS AND SYMBOLS

LEGEND

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



SOIL GRAIN SIZE

US STANDARD SIEVE

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

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

300 76.2 19.1 4.76 2.00 0.42 0.074 0.005

SOIL GRAIN SIZE IN MILLIMETERS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

STRENGTH OF COHESIVE SOILS

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

DENSITY OF GRANULAR SOILS

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

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

RELATIVE COMPOSITION

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

OTHER TERMS

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

GEOTECHNOLOGY INC
FROM THE GROUND UP

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

APPENDIX D – LABORATORY TEST DATA

Atterberg Limits

Grain Size Distributions

Unconsolidated-Undrained Triaxial Compression

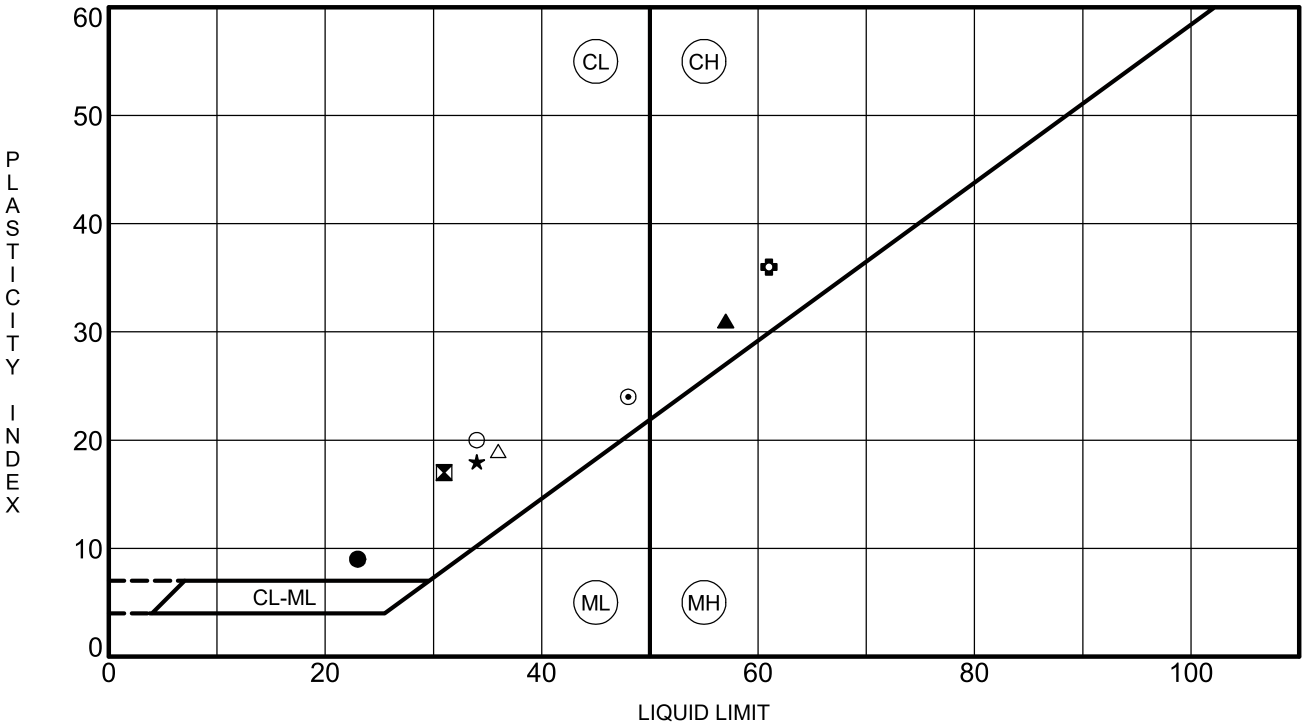
Direct Shear

Resistivity

pH

Standard Proctor Curves

CBR Results

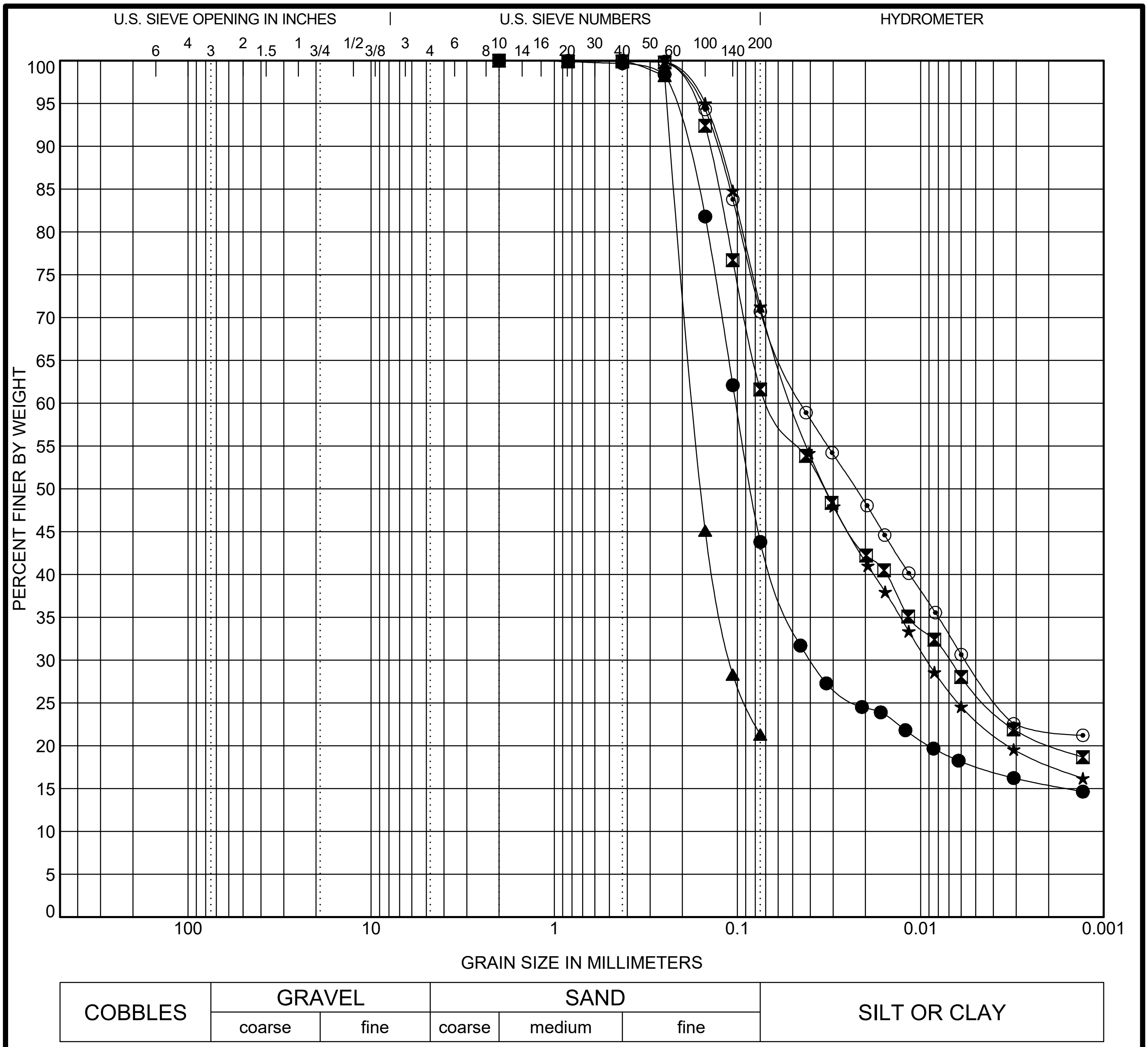


	Specimen Identification		LL	PL	PI	Fines	Classification
●	**MC- 1**	**5.0**	**23**	**14**	**9**	**44**	**CLAYEY SAND(SC)**
⧗	**MC- 2**	**8.0**	**31**	**14**	**17**	**71**	**LEAN CLAY with SAND(CL)**
▲	**MC- 3**	**38.5**	**57**	**26**	**31**		**FAT CLAY(CH)**
★	**MC- 6**	**10.0**	**34**	**16**	**18**	**38**	**CLAYEY SAND(SC)**
⊙	**MC- 6**	**38.5**	**48**	**24**	**24**		**LEAN CLAY(CL)**
✿	**MC- 7**	**43.5**	**61**	**25**	**36**		**FAT CLAY(CH)**
○	**MC- 9**	**1.0**	**34**	**14**	**20**	**51**	**SANDY LEAN CLAY(CL)**
△	**MC-10**	**1.0**	**36**	**17**	**19**	**75**	**LEAN CLAY with SAND(CL)**

GEOTECHNOLOGY INC
FROM THE GROUND UP

ATTERBERG LIMITS RESULTS

Mill Creek Bridge Replacement
Miller County
J028499.03



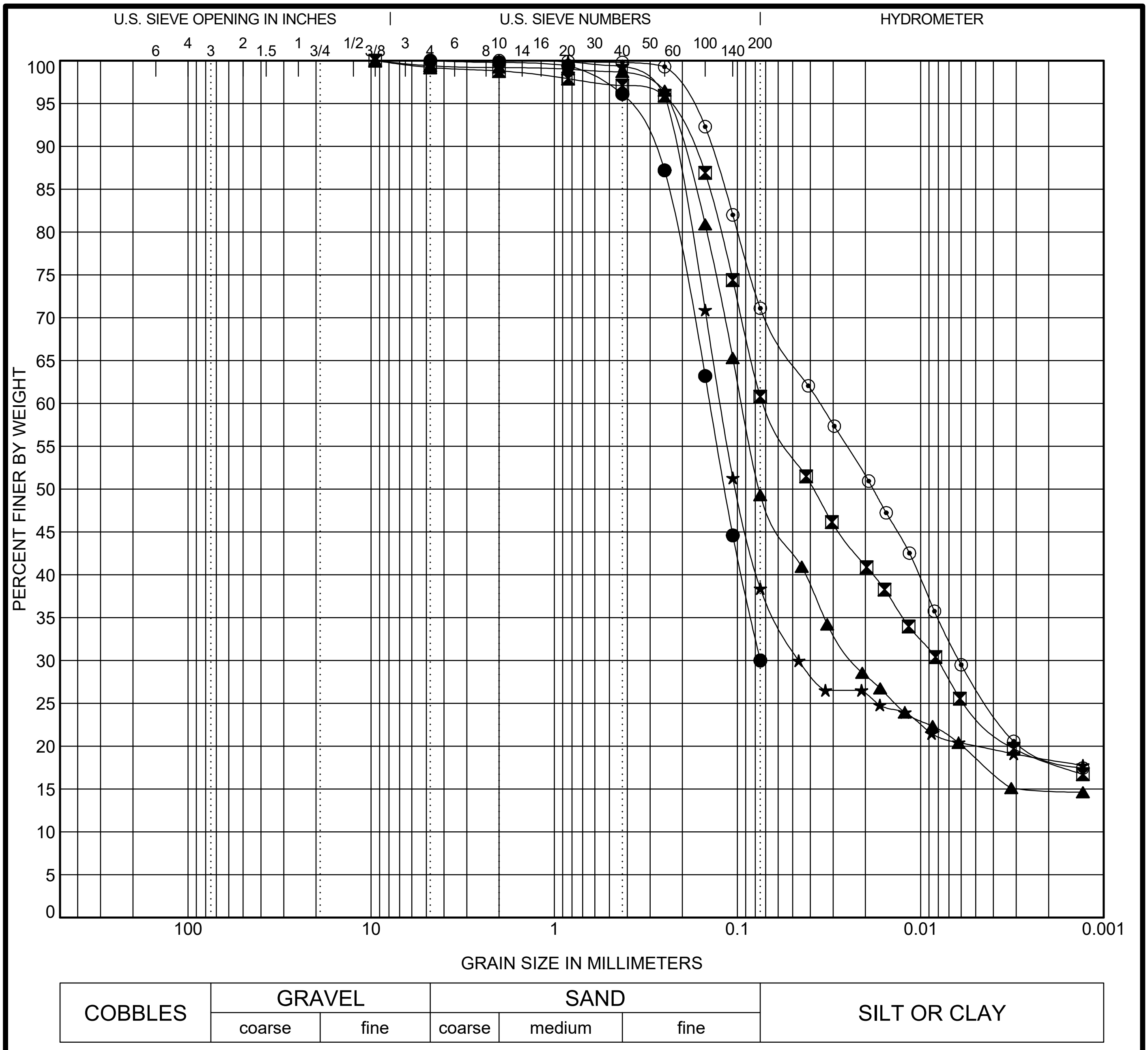
Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**MC- 1**	**5.0**	**CLAYEY SAND(SC)**	**23**	**14**	**9**		
☒	**MC- 1**	**18.5**	**SANDY SILT(ML)**					
▲	**MC- 1**	**68.5**	**SILTY SAND(SM)**					
★	**MC- 2**	**8.0**	**LEAN CLAY with SAND(CL)**	**31**	**14**	**17**		
⊙	**MC- 2**	**13.5**	**LEAN CLAY with SAND(CL)**					

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**MC- 1**	**5.0**	**2**	**0.102**	**0.04**		**0.0**	**56.2**	**26.2**	**17.6**
☒	**MC- 1**	**18.5**	**2**	**0.067**	**0.007**		**0.0**	**38.4**	**35.3**	**26.3**
▲	**MC- 1**	**68.5**	**0.84**	**0.173**	**0.11**		**0.0**	**78.7**	**21.3**	
★	**MC- 2**	**8.0**	**0.425**	**0.05**	**0.009**		**0.0**	**28.7**	**48.1**	**23.2**
⊙	**MC- 2**	**13.5**	**2**	**0.045**	**0.006**		**0.0**	**29.3**	**42.3**	**28.4**

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

Mill Creek Bridge Replacement
Miller County
J028499.03



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**MC- 2**	**28.5**	**SILTY SAND(SM)**					
☒	**MC- 3**	**6.0**	**SANDY SILT(ML)**					
▲	**MC- 3**	**23.5**	**SILTY SAND(SM)**					
★	**MC- 6**	**10.0**	**CLAYEY SAND(SC)**	**34**	**16**	**18**		
⊙	**MC- 6**	**13.5**	**SANDY SILT(ML)**					

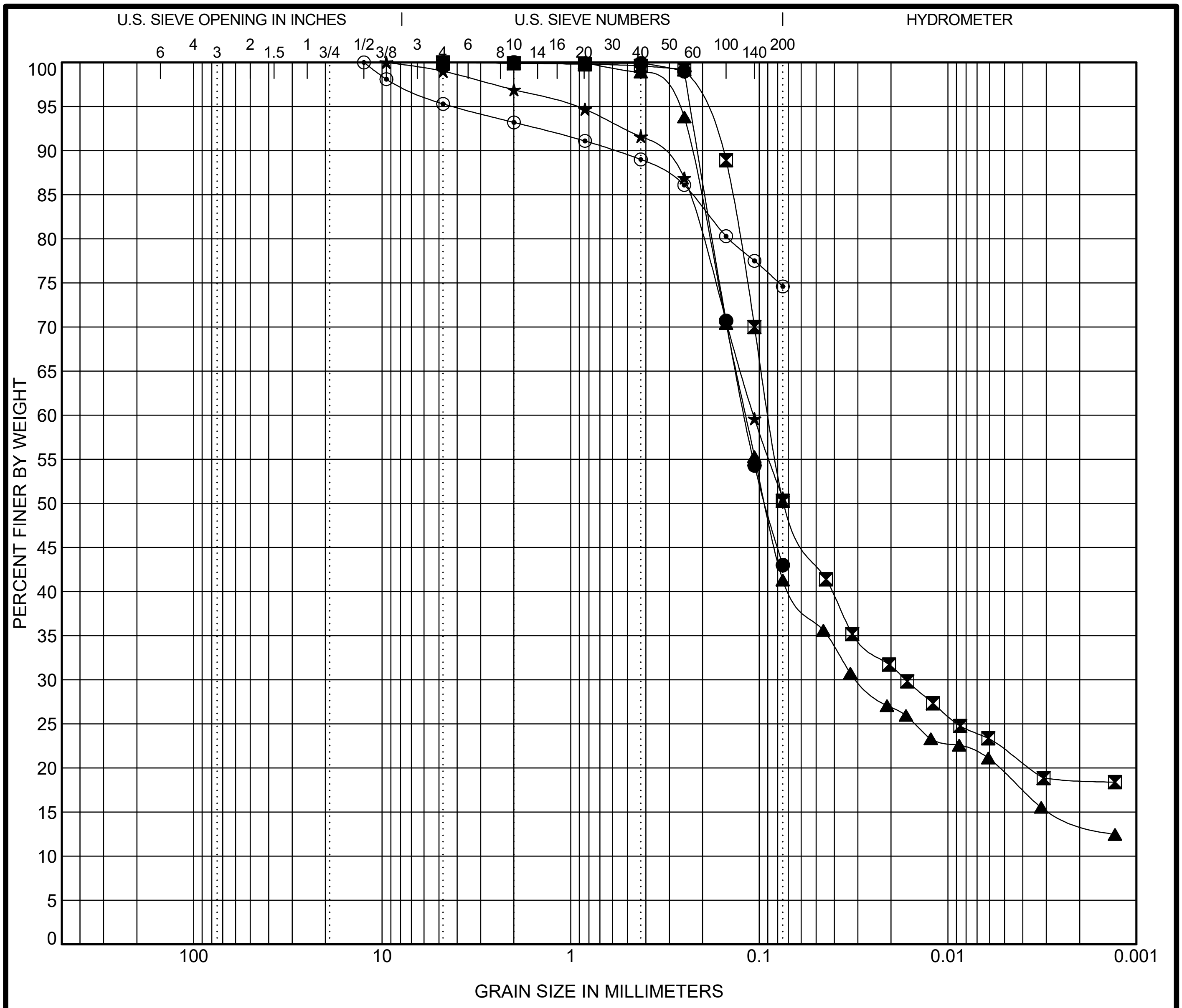
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**MC- 2**	**28.5**	**4.75**	**0.141**	**0.075**		**0.0**	**70.0**	**30.0**	
☒	**MC- 3**	**6.0**	**9.5**	**0.071**	**0.008**		**0.8**	**38.4**	**37.0**	**23.8**
▲	**MC- 3**	**23.5**	**9.5**	**0.095**	**0.023**		**0.6**	**50.1**	**30.6**	**18.7**
★	**MC- 6**	**10.0**	**2**	**0.124**	**0.046**		**0.0**	**61.6**	**18.4**	**20.0**
⊙	**MC- 6**	**13.5**	**4.75**	**0.036**	**0.006**		**0.0**	**28.9**	**44.1**	**27.0**

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

Mill Creek Bridge Replacement
Miller County
J028499.03

US GRAIN SIZE J028499.03 ARDOT 030497 - MILL CREEK.GPJ US LAB.GDT 12/3/19



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

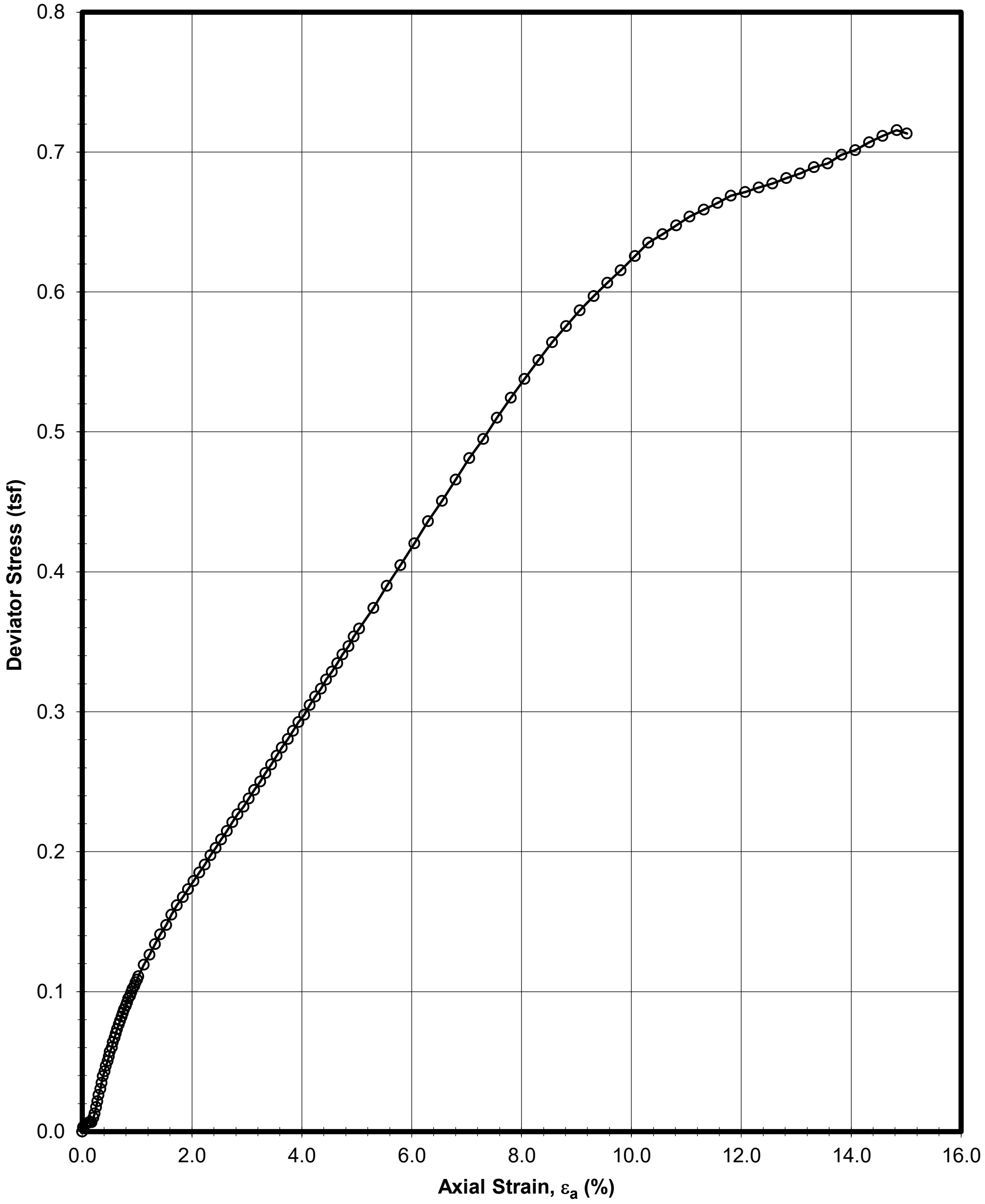
Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**MC- 6**	**58.5**	**SILTY SAND(SM)**					
⧗	**MC- 7**	**18.5**	**SANDY SILT(ML)**					
▲	**MC- 7**	**33.5**	**SILTY SAND(SM)**					
★	**MC- 9**	**1.0**	**SANDY LEAN CLAY(CL)**	**34**	**14**	**20**		
⊙	**MC-10**	**1.0**	**LEAN CLAY with SAND(CL)**	**36**	**17**	**19**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**MC- 6**	**58.5**	**4.75**	**0.12**			**0.0**	**57.0**	**43.0**	
⧗	**MC- 7**	**18.5**	**4.75**	**0.089**	**0.017**		**0.0**	**49.7**	**28.3**	**22.0**
▲	**MC- 7**	**33.5**	**4.75**	**0.118**	**0.03**		**0.0**	**58.7**	**21.9**	**19.4**
★	**MC- 9**	**1.0**	**9.5**	**0.107**			**1.0**	**48.4**	**50.6**	
⊙	**MC-10**	**1.0**	**12.5**				**4.7**	**20.7**	**74.6**	

GEOTECHNOLOGY INC
FROM THE GROUND UP

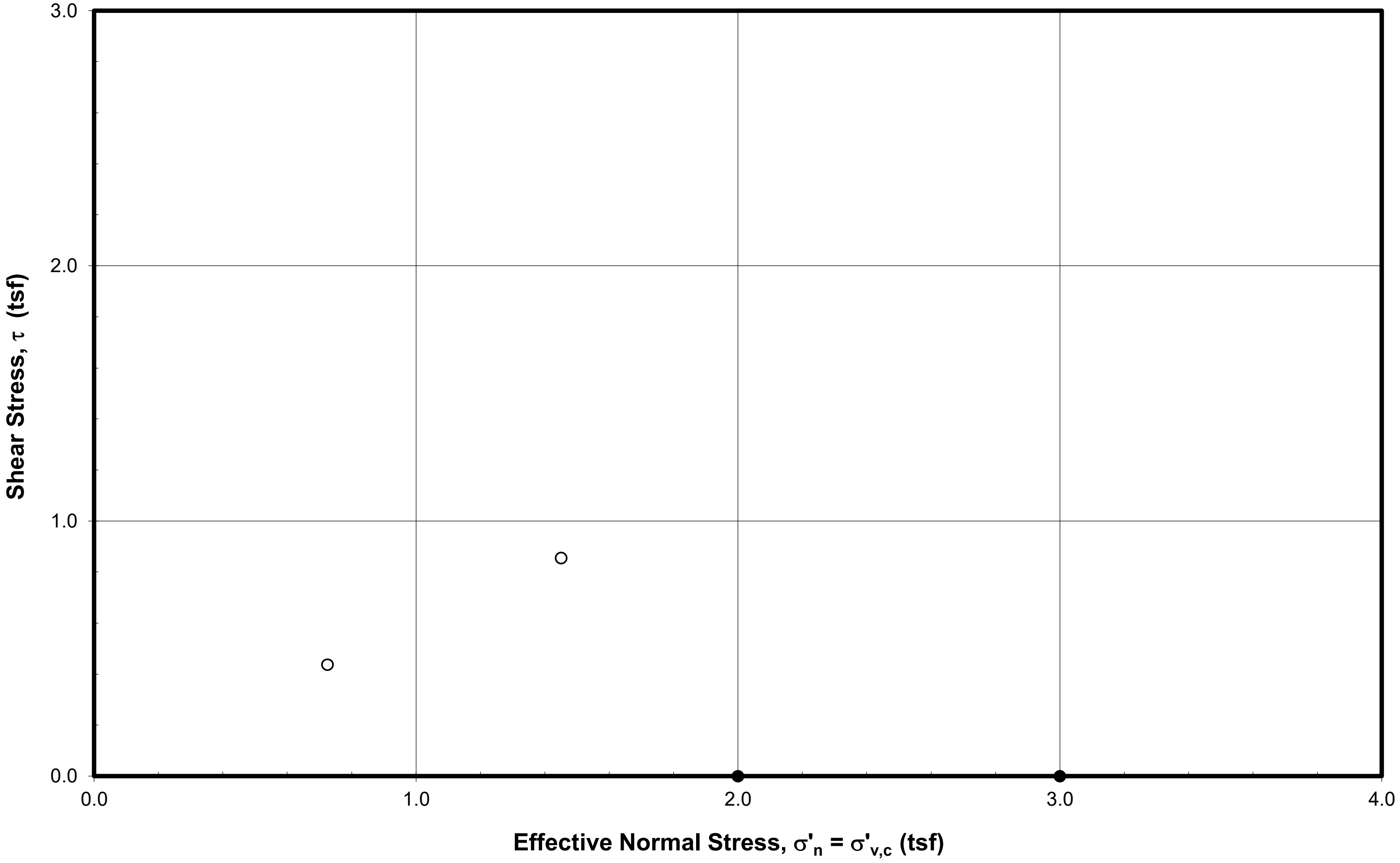
GRAIN SIZE DISTRIBUTION

Mill Creek Bridge Replacement
Miller County
J028499.03



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850
Project No.: J028499.01
Boring: MC-6
Sample: ST-5 - Depth: 10 ft.



DRAINED DIRECT SHEAR TEST

ASTM D 3080

Boring: MC-6 Sample: ST-5 -Depth: 10ft

TEST REPORT

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

Project No.:	**J028499.01**	**November 20, 2019**
Project Name:	**ARDOT 030497 Bridge Replacement over Mill Creek**	**Page 1 of 1**
Boring Number:	**MC-1**	
Sample ID:	**ST-3**	
Depth (ft):	**5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	52,000	0.57	29,640.00	10.3
#2	30,000	0.57	17,100.00	17.2
#3	32,000	0.57	18,240.00	25.5

Minimum Soil Resistivity **17,100.00**

TEST REPORT

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

Project No.: J028499.01 — **December 17, 2019**
Project Name: ARDOT 030497 Bridge Replacement over Mill Creek — **Page 1 of 1**
Boring Number: MC-1
Sample ID: SS5-8
Depth (ft): 13.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	18,000	0.57	10,260.00	11.9
#2	14,000	0.57	7,980.00	19.9
#3	15,000	0.57	8,550.00	25.6

Minimum Soil Resistivity **7,980.00**

TEST REPORT

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

Project No.:	**J028499.01**	**December 17, 2019**
Project Name:	**ARDOT 030497 Bridge Replacement over Mill Creek**	**Page 1 of 1**
Boring Number:	**MC-1**	
Sample ID:	**SS14-17**	
Depth (ft):	**58.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	5,900	0.57	3,363.00	13.4
#2	3,500	0.57	1,995.00	20.1
#3	3,200	0.57	1,824.00	27.1
#4	3,300	0.57	1,881.00	34.0

Minimum Soil Resistivity **1,824.00**

TEST REPORT

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

Project No.:	**J028499.03**	**November 14, 2019**
Project Name:	**ARDOT 030497 Bridge Replacement over Mill Creek**	**Page 1 of 1**
Boring Number:	**MC-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	43,000	0.57	24,510.00	9.9
#2	19,000	0.57	10,830.00	16.2
#3	18,500	0.57	10,545.00	23.0
#4	19,000	0.57	10,830.00	28.0
	Minimum Soil Resistivity		**10,545.00**	

TEST REPORT

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

Project No.: J028499.01 — **December 17, 2019**
Project Name: ARDOT 030497 Bridge Replacement over Mill Creek — **Page 1 of 1**
Boring Number: MC-3
Sample ID: SS3-6
Depth (ft): 6

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	23,000	0.57	13,110.00	11.5
#2	6,000	0.57	3,420.00	17.8
#3	15,000	0.57	8,550.00	22.7
#4	16,000	0.57	9,120.00	31.0

Minimum Soil Resistivity **3,420.00**

TEST REPORT

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

Project No.:	**J028499.01**	**December 18, 2019**
Project Name:	**ARDOT 030497 Bridge Replacement over Mill Creek**	**Page 1 of 1**
Boring Number:	**MC-3**	
Sample ID:	**SS8-10**	
Depth (ft):	**28.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	7,000	0.57	3,990.00	10.4
#2	2,700	0.57	1,539.00	17.8
#3	2,300	0.57	1,311.00	24.6
#4	2,400	0.57	1,368.00	29.5

Minimum Soil Resistivity **1,311.00**

TEST REPORT

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

Project No.:	**J028499.01**	**December 18, 2019**
Project Name:	**ARDOT 030497 Bridge Replacement over Mill Creek**	**Page 1 of 1**
Boring Number:	**MC-6**	
Sample ID:	**SS4-8**	
Depth (ft):	**8.5**	

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	10.5
#2	18,000	0.57	10,260.00	17.3
#3	17,000	0.57	9,690.00	24.0
#4	19,000	0.57	10,830.00	29.7

Minimum Soil Resistivity **9,690.00**

TEST REPORT

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

Project No.:	**J028499.01**	**November 20, 2019**
Project Name:	**ARDOT 030497 Bridge Replacements over Mill Creek**	**Page 1 of 1**
Boring Number:	**MC-6**	
Sample ID:	**ST-5**	
Depth (ft):	**10**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	36,000	0.57	20,520.00	25.6
#2	20,000	0.57	11,400.00	22.8
#3	23,000	0.57	13,110.00	33.7

Minimum Soil Resistivity **11,400.00**

TEST REPORT

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

Project No.:	**J028499.01**	**December 18, 2019**
Project Name:	**ARDOT 030497 Bridge Replacement over Mill Creek**	**Page 1 of 1**
Boring Number:	**MC-6**	
Sample ID:	**SS14-16**	
Depth (ft):	**53.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	4,800	0.57	2,736.00	10.8
#2	2,300	0.57	1,311.00	17.9
#3	2,100	0.57	1,197.00	25.7
#4	2,200	0.57	1,254.00	32.8

Minimum Soil Resistivity **1,197.00**

TEST REPORT

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

Project No.:	**J028499.01**	**December 18, 2019**
Project Name:	**ARDOT 030497 Bridge Replacement over Mill Creek**	**Page 1 of 1**
Boring Number:	**MC-7**	
Sample ID:	**SS4-7**	
Depth (ft):	**8.5**	

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	10.1
#2	16,000	0.57	9,120.00	17.3
#3	17,000	0.57	9,690.00	24.1

Minimum Soil Resistivity **9,120.00**

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

GEOTECHNOLOGY INC
FROM THE GROUND UP

DATE	PROJECT NAME Mill Creek	PROJECT NO. J028499.03

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

Boring No.	Sample No.	Depth (ft)	Visual Identification (Color, Group Name & Symbol)	Soil : Water Ratio (g/g) or (g/mL)	pH of Solution (Meter/ Paper)[1]	Tare No. Air Drying	Jar Number	Remarks
MC-1	ST-3	5.00		1/1	4.91 ------------- 21.5			

MC-2	ST-4	8.00		1/1	4.53 ------------- 21.4			

MC-6	ST-5	10.00		1/1	3.9 ------------- 21.5			

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

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

Tested By: EM
Date: 12/05/19

Calculated By: HP
Date: 12/05/19

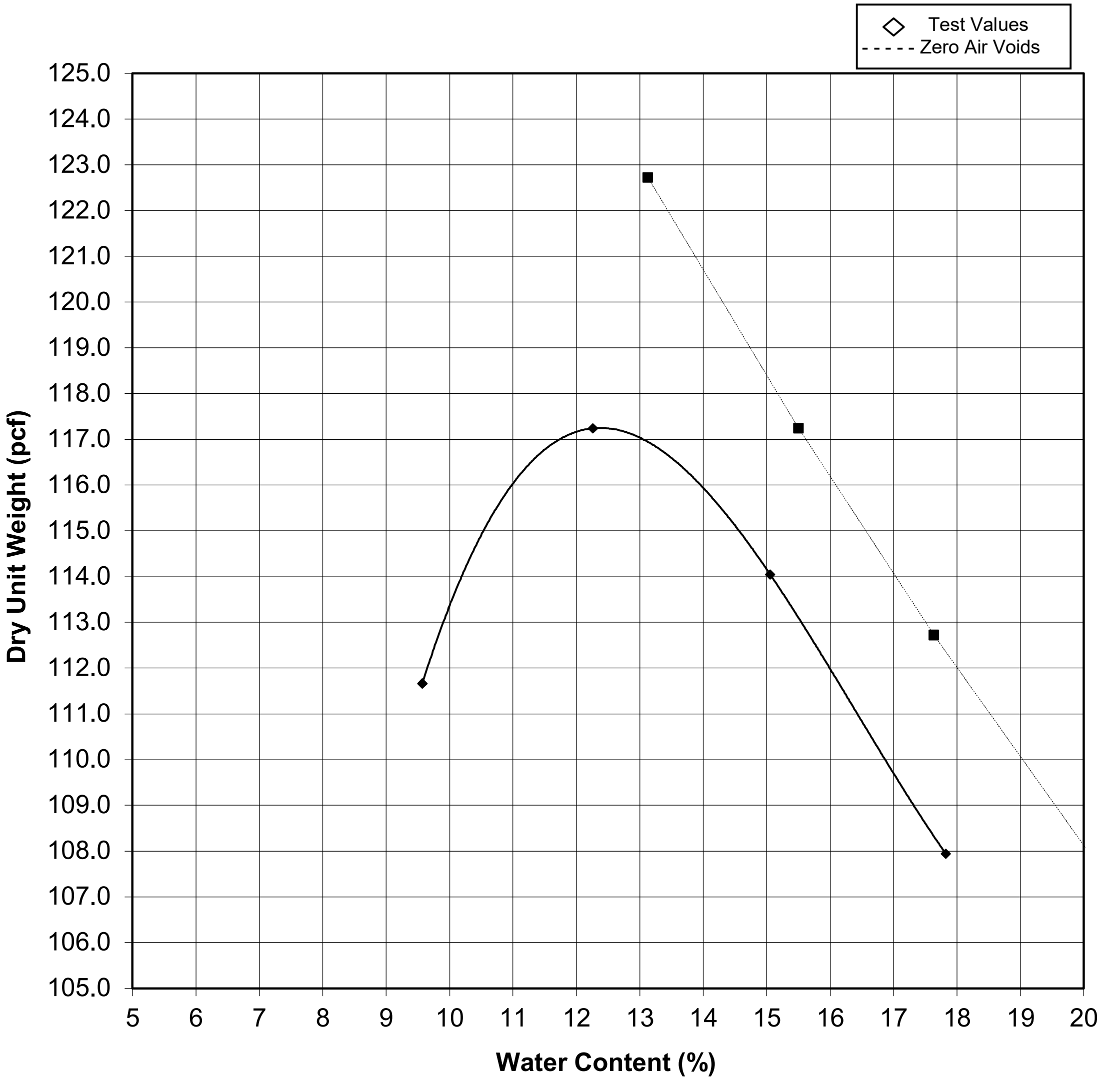
Checked By: JDM
Date: 12/05/19

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

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: ARDOT - Miller and Bodcau
Client: Garver USA
Sample Source: MC9, 1.0'-5.0'
Supplier:

LABORATORY COMPACTION TEST



Test Information

Project No.: J028499.03
Test Date: 03/21/19
Proctor No.: MC-9

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

Sample Description

Red/Brown Sandy Lean Clay (CL)/ A-6(6)

Sample Properties

Moisture Content NA
Liquid Limit 34
Plastic Limit 14
Plasticity Index 20
Specific Gravity: 2.650 Estimated
Classification CL

Test Results:

Maximum Dry Unit Weight (pcf): 117.3
Optimum Water Content (%): 12.4

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

Tested By: TA
Date: 03/21/19

Input By: HP
Date: 03/22/19

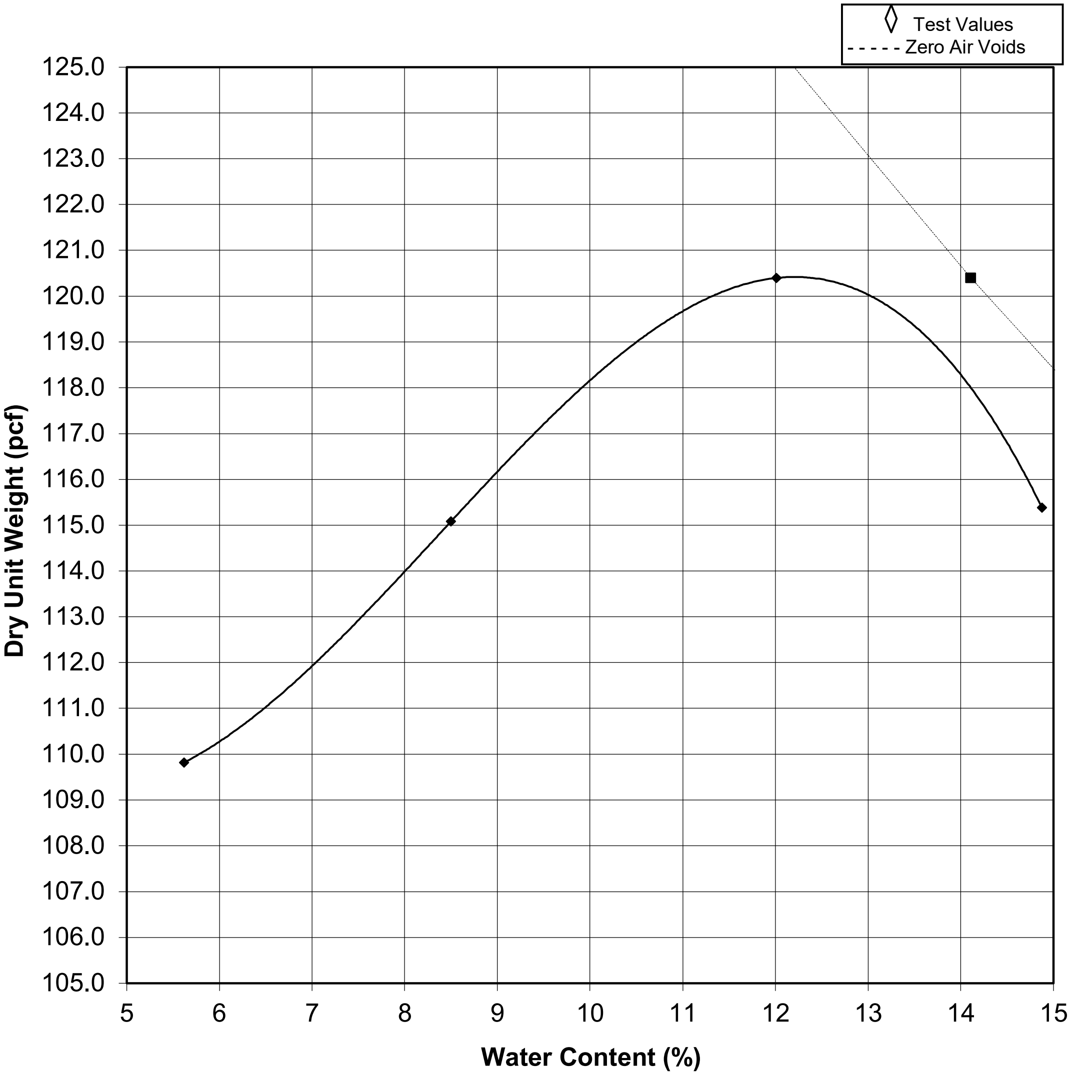
Checked By: HP
Date: 03/22/19

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

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: ARDOT - Miller and Bodcau
Client: Garver USA
Sample Source: MC10, 1.0'-5.0'
Supplier:

LABORATORY COMPACTION TEST



Test Information	
Project No.:	J028499.03
Test Date:	03/21/19
Proctor No.:	MC10
Test Method:	ASTM D 698
Rammer Type:	Mechanical
Prep. Method:	Dry

Sample Description

Red/Brown Lean Clay with Sand (CL)/ A-6(12)

Sample Properties		
Moisture Content	NA	
Liquid Limit	36	
Plastic Limit	17	
Plasticity Index	19	
Specific Gravity:	2.650	Estimated
Classification	CL	

Test Results:

Maximum Dry Unit Weight (pcf): 120.4
Optimum Water Content (%): 12.3

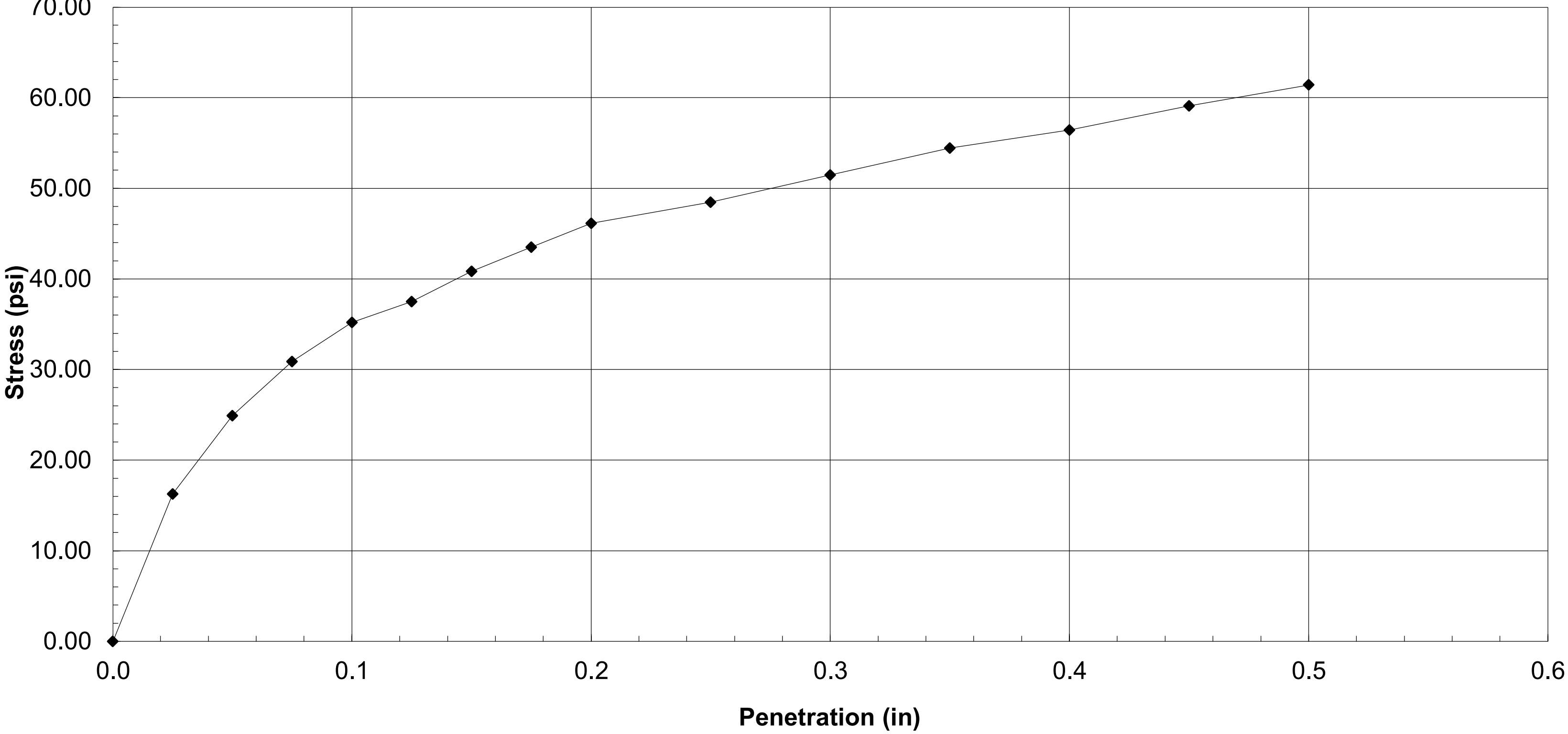
Oversize Correction Values:

Maximum Dry Unit Weight (pcf): 127.1
Optimum Water Content (%): 10.4

Tested By: TA
Date: 03/21/19

Input By: HP
Date: 03/22/19

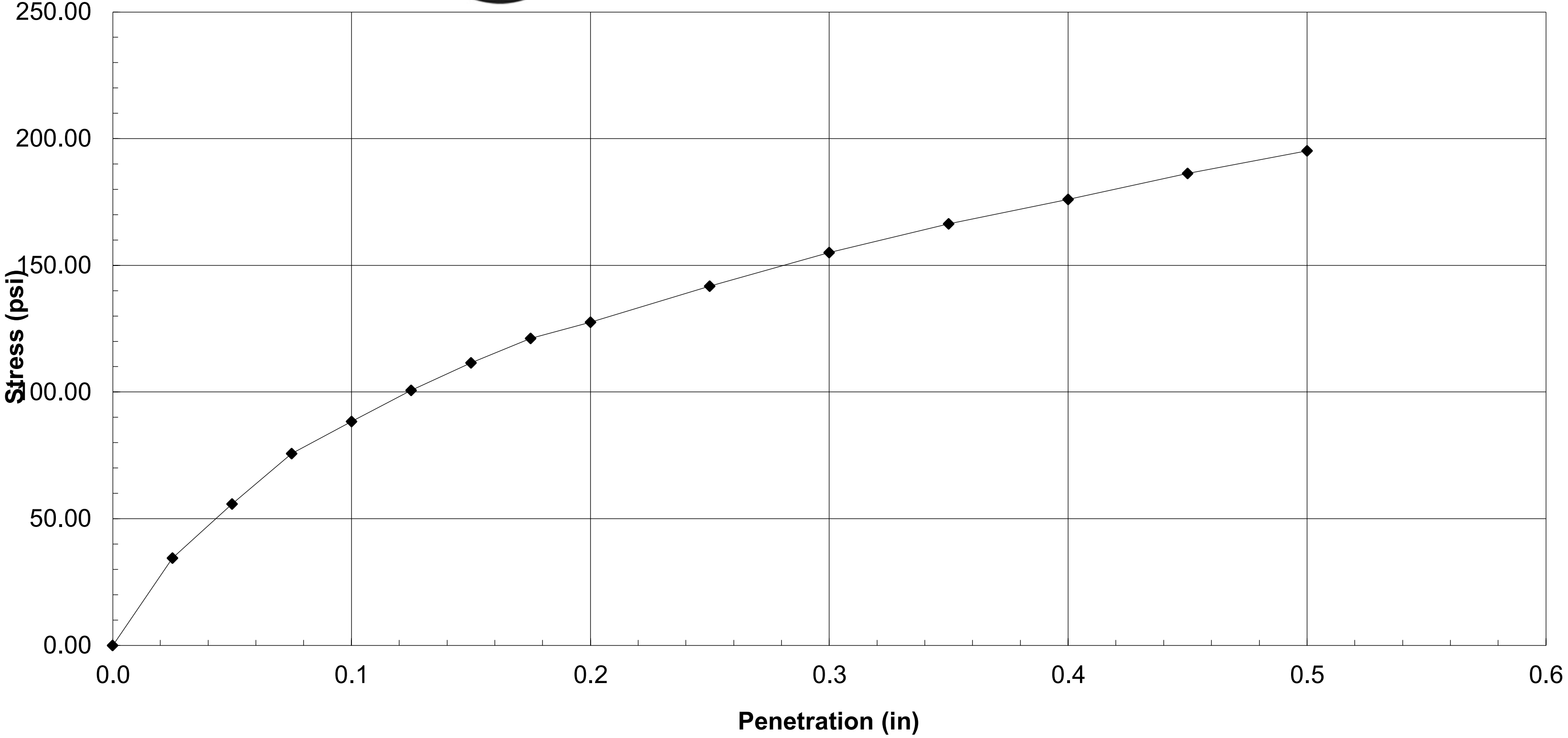
Checked By: HP
Date: 03/22/19



CALIFORNIA BEARING RATIO (CBR) TEST

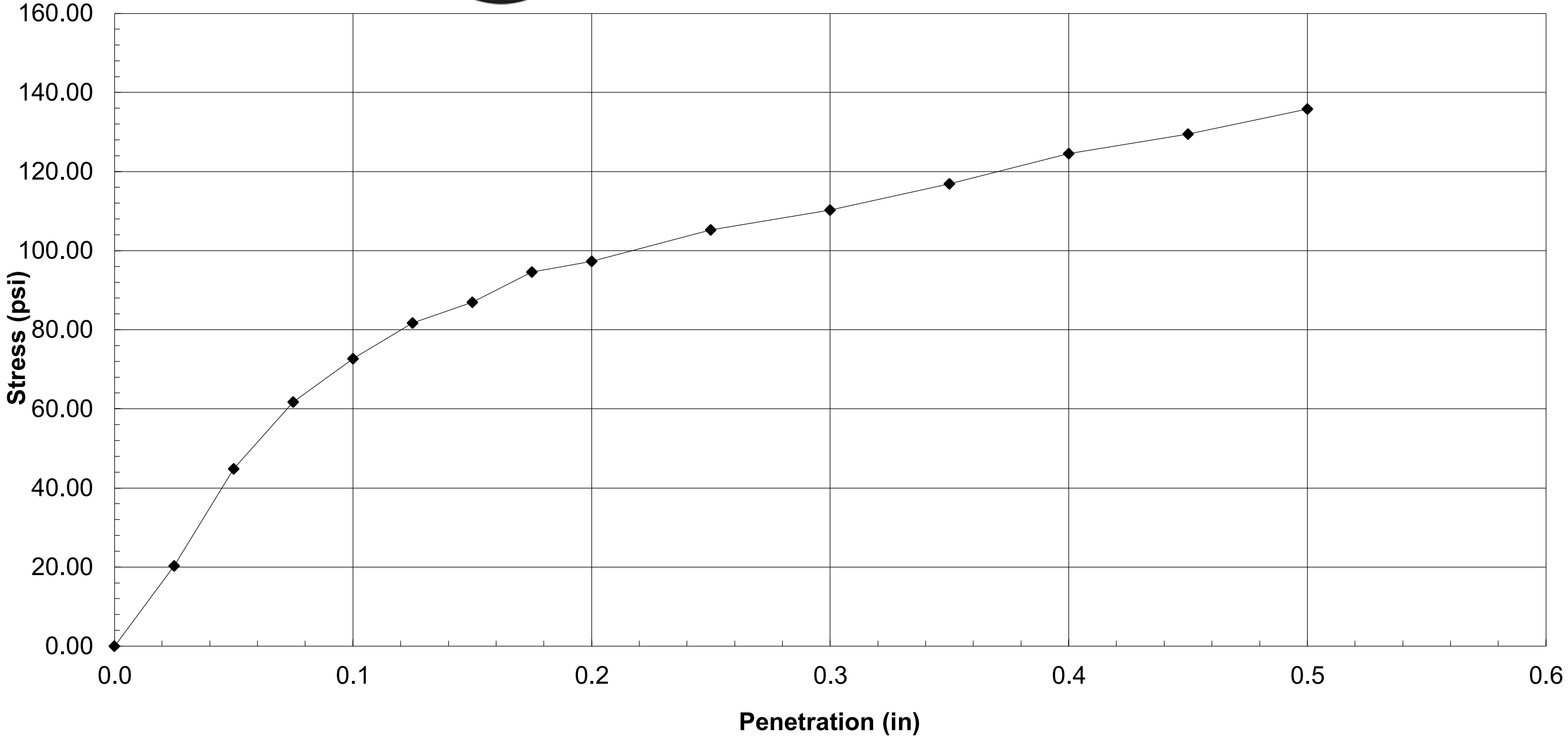
ASTM D 1883
Project No.: J028499.02
Boring: MC-9
Sample: 25 Blows - Depth: 0 ft.

GEOTECHNOLOGY INC
FROM THE GROUND UP



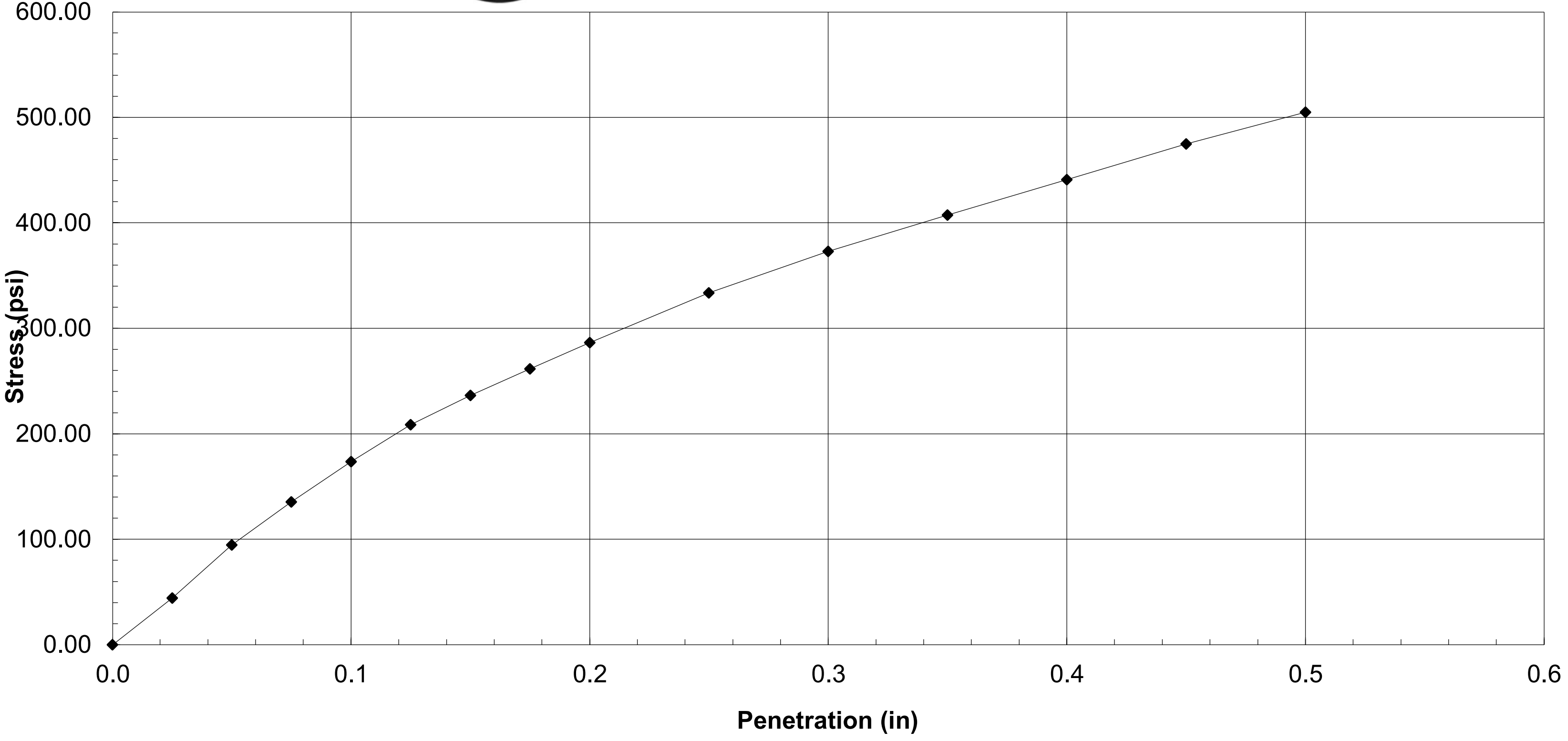
CALIFORNIA BEARING RATIO (CBR) TEST

ASTM D 1883
Project No.: J028499.03
Boring: MC-9
Sample: 56 Blows - Depth: 0 ft.



CALIFORNIA BEARING RATIO (CBR) TEST

ASTM D 1883
Project No.: J028499.03
Boring: MC-10
Sample: 25 Blows - Depth: 0 ft.



CALIFORNIA BEARING RATIO (CBR) TEST

ASTM D 1883
Project No.: J028499.03
Boring: MC-10
Sample: 56 Blows - Depth: 0 ft.

APPENDIX E – AASHTO AND USCS CLASSIFICATIONS

SUMMARY OF CLASSIFICATION RESULTS
Highway 82 Strs. & Apprs. (S): Mill Creek Bridge
Miller County: Arkansas
ARDOT 030497

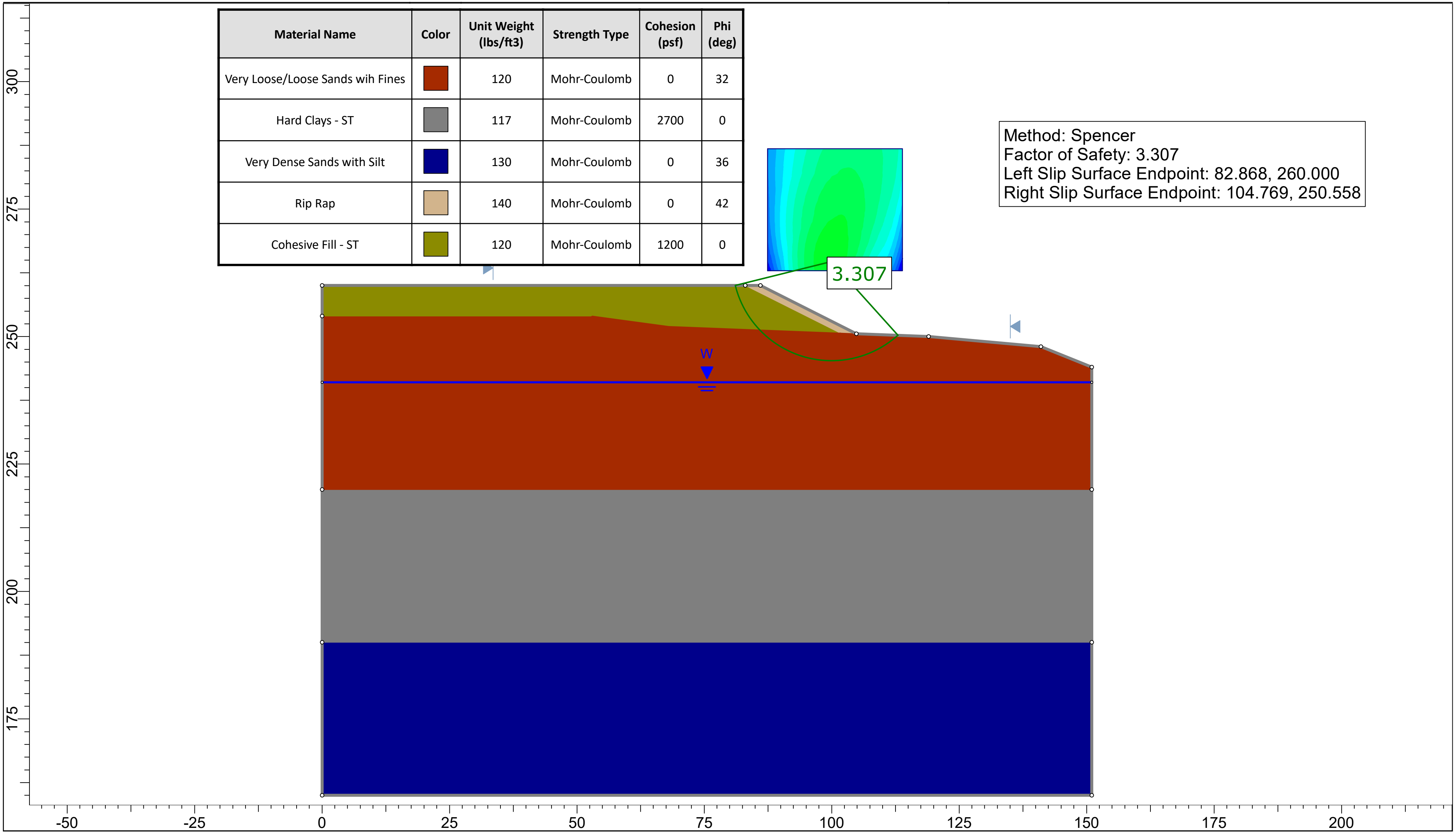
Boring	Depth (feet)	Liquid Limit (LL) (%)	Plastic Limit (PL) (%)	Plasticity Index (PI) (%)	Sieve Analysis Percent Passing								GI	AASHTO CLASS.	USCS CLASS.
					2 in.	1 in.	3/4 in.	3/8 in.	#4	#10	#40	#200			
MC-1	5	23	14	9	100.0	100.0	100.0	100.0	100.0	100.0	99.7	43.8	1	A-4	SC
MC-1	18.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.9	61.6	0	A-4	ML
MC-1	68.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	100.0	21.3	0	A-2-4	SM
MC-2	8	31	14	17	100.0	100.0	100.0	100.0	100.0	100.0	100.0	71.3	10	A-6	CL
MC-2	13.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	100.0	70.7	0	A-6	CL
MC-2	28.5	--	--	--	100.0	100.0	100.0	100.0	100.0	99.8	96.1	30.0	0	A-2-4	SM
MC-3	6	--	--	--	100.0	100.0	100.0	100.0	99.2	98.8	97.1	60.8	0	A-4	ML
MC-3	23.5	--	--	--	100.0	100.0	100.0	100.0	99.4	99.2	98.7	49.3	0	A-4	SM
MC-6	10	34	16	18	100.0	100.0	100.0	100.0	100.0	100.0	99.4	38.4	2	A-6	SC
MC-6	13.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.8	71.1	0	A-4	ML
MC-6	38.5	48	24	24	--	--	--	--	--	--	--	--	0	A-2-7	CL
MC-6	58.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	99.9	43.0	0	A-4	SM
MC-7	18.5	--	--	--	100.0	100.0	100.0	100.0	100.0	99.9	99.6	50.3	0	A-4	ML
MC-7	33.5	--	--	--	100.0	100.0	100.0	100.0	100.0	100.0	98.9	41.3	0	A-4	SM
MC-7	43.5	61	42	36	--	--	--	--	--	--	--	--	0	A-2-7	CH
MC-9	1	34	14	20	100.0	100.0	100.0	100.0	99.0	96.9	91.6	50.6	6	A-6	CL
MC-10	1	36	17	19	100.0	100.0	100.0	98.1	95.3	93.2	91.1	74.6	12	A-6	CL

APPENDIX F – GLOBAL STABILITY ANALYSES

File Name: West Abutment.slmd
Name: Group 1
Description: Short Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - West Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Loose/Loose Sands wih Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays - ST	⬛	117	Mohr-Coulomb	2700	0
Very Dense Sands with Silt	🟦	130	Mohr-Coulomb	0	36
Rip Rap	🟫	140	Mohr-Coulomb	0	42
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0

Method: Spencer
Factor of Safety: 3.307
Left Slip Surface Endpoint: 82.868, 260.000
Right Slip Surface Endpoint: 104.769, 250.558

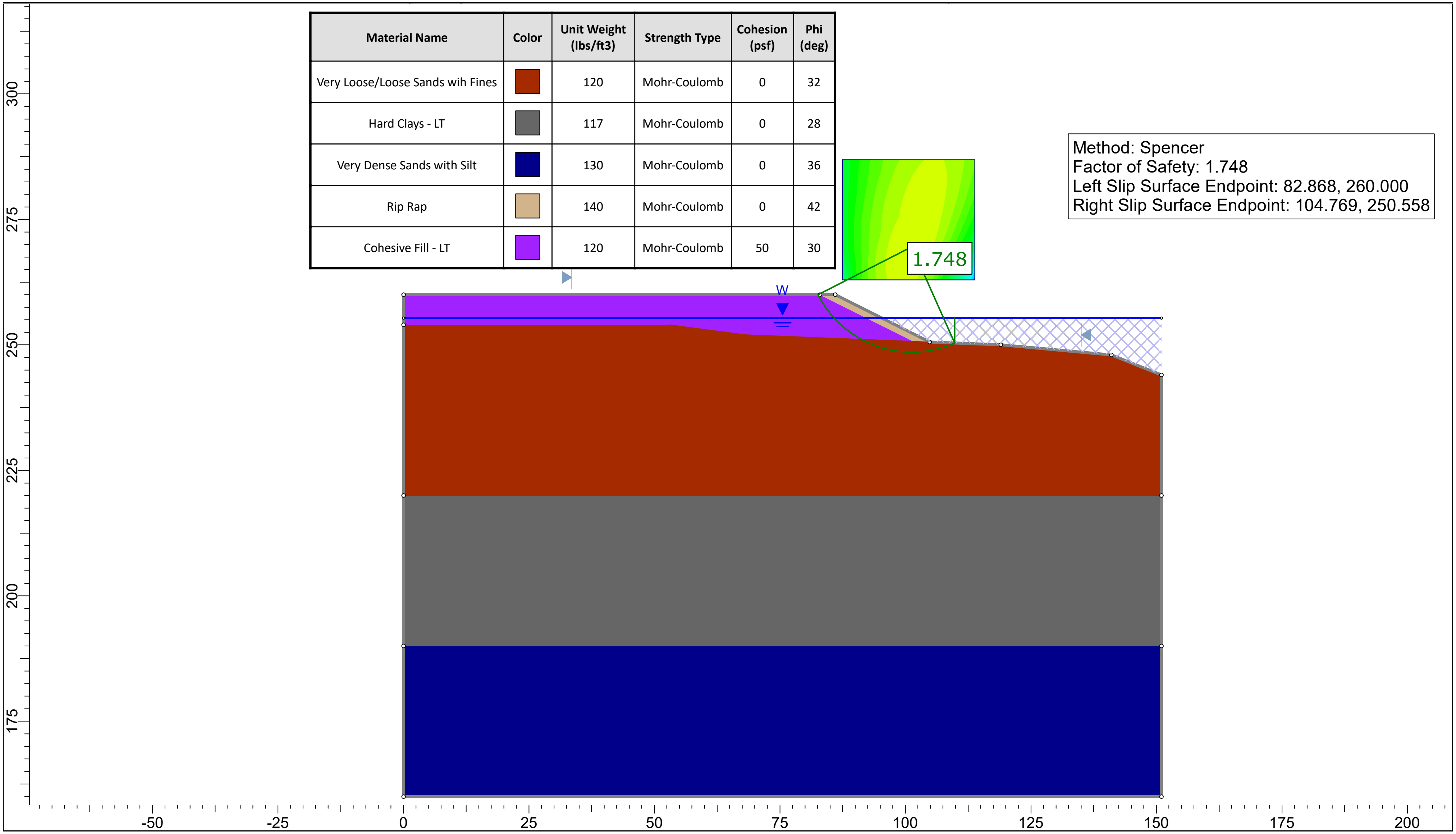
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: West Abutment.slmd
Name: Group 1
Description: Long Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - West Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Loose/Loose Sands wih Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays - LT	⬛	117	Mohr-Coulomb	0	28
Very Dense Sands with Silt	🟦	130	Mohr-Coulomb	0	36
Rip Rap	🟫	140	Mohr-Coulomb	0	42
Cohesive Fill - LT	🟪	120	Mohr-Coulomb	50	30



Method: Spencer
Factor of Safety: 1.748
Left Slip Surface Endpoint: 82.868, 260.000
Right Slip Surface Endpoint: 104.769, 250.558

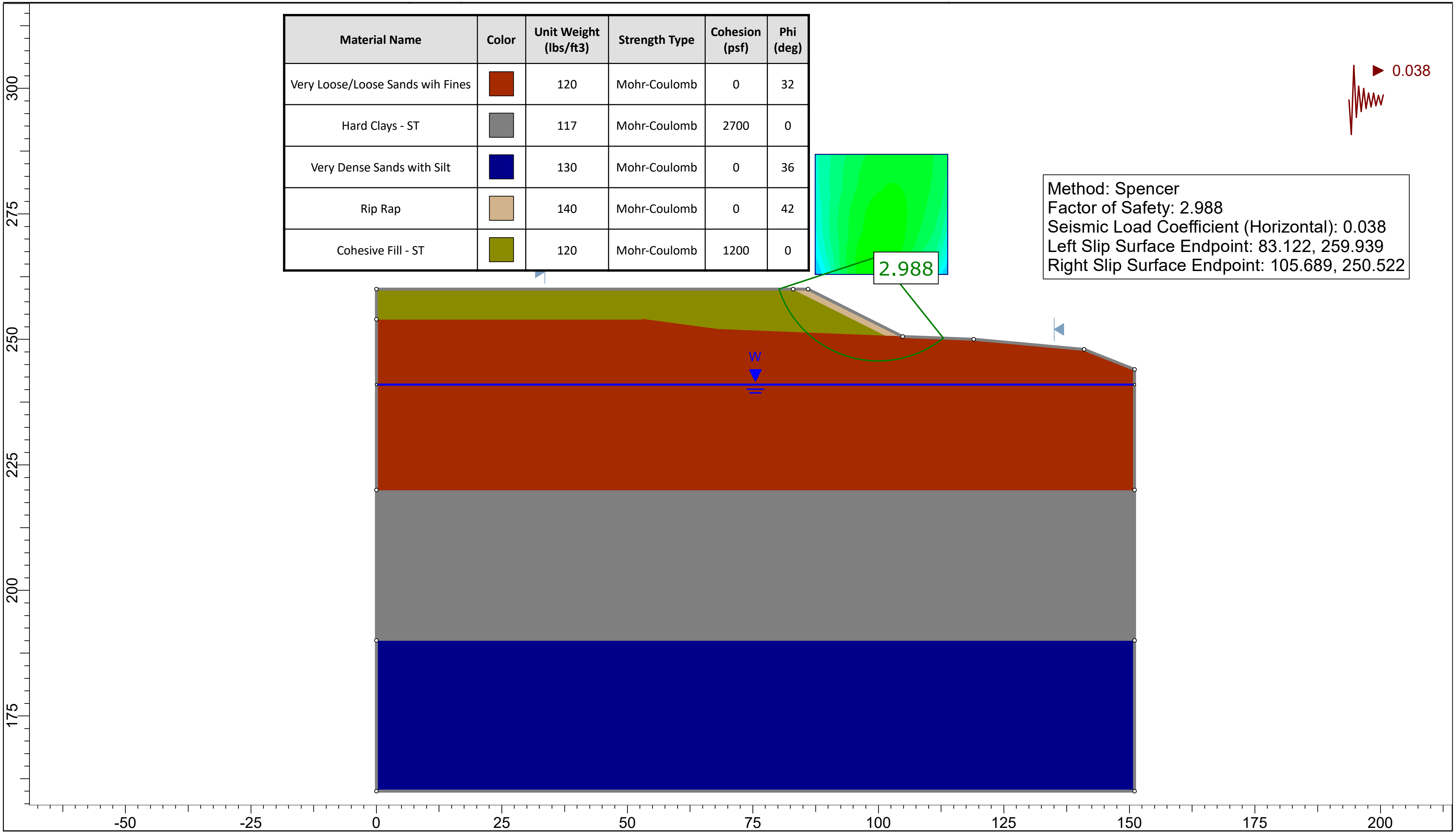
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: West Abutment.slmd
Name: Group 1
Description: Seismic Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - West Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Loose/Loose Sands wih Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays - ST	⬛	117	Mohr-Coulomb	2700	0
Very Dense Sands with Silt	🟦	130	Mohr-Coulomb	0	36
Rip Rap	🟫	140	Mohr-Coulomb	0	42
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0



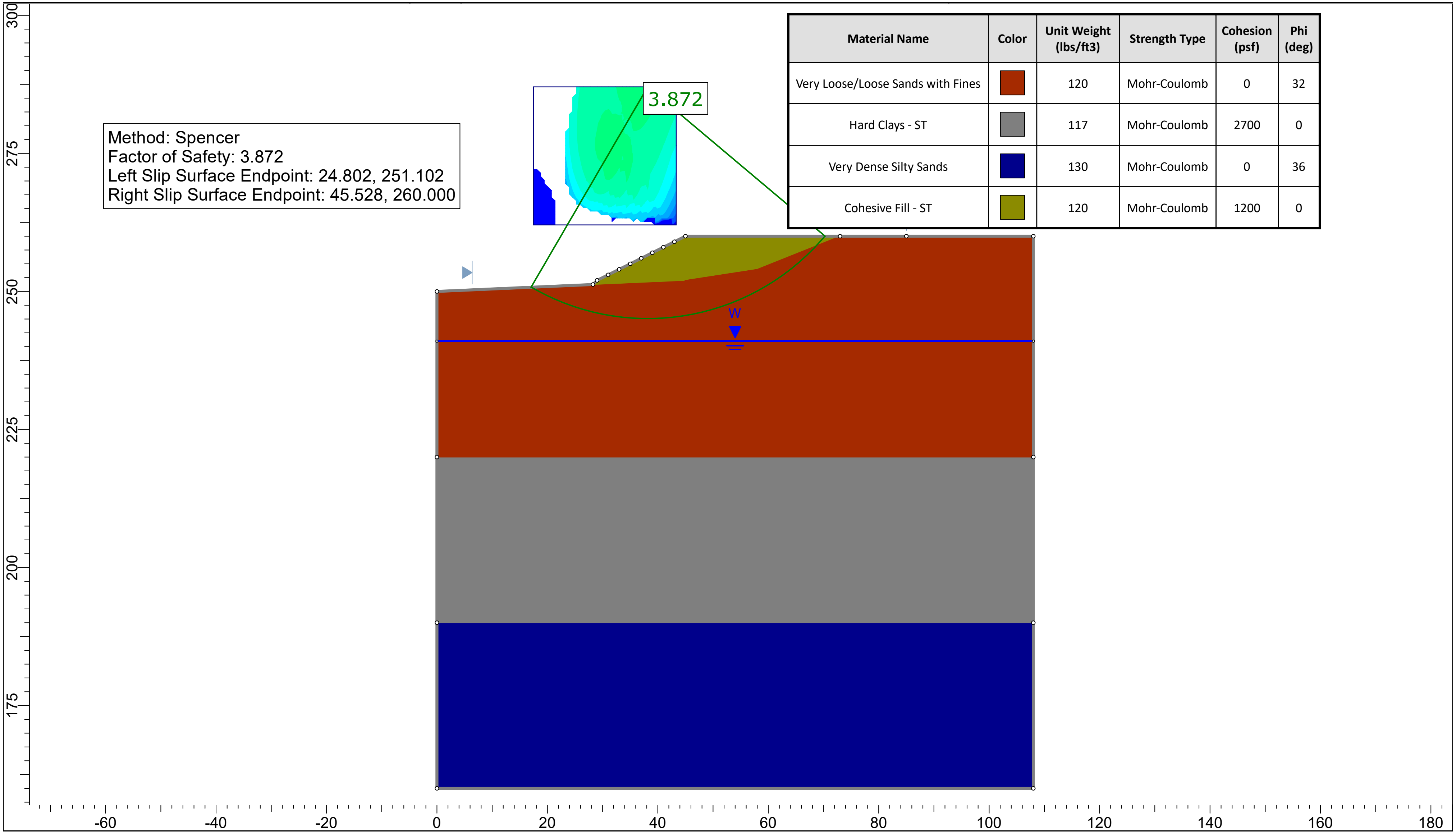
Method: Spencer
Factor of Safety: 2.988
Seismic Load Coefficient (Horizontal): 0.038
Left Slip Surface Endpoint: 83.122, 259.939
Right Slip Surface Endpoint: 105.689, 250.522

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: STA 110+00 Southern Side Slope.slmd
Name: Group 1
Description: Short Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - STA 110+00
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 3.872
Left Slip Surface Endpoint: 24.802, 251.102
Right Slip Surface Endpoint: 45.528, 260.000

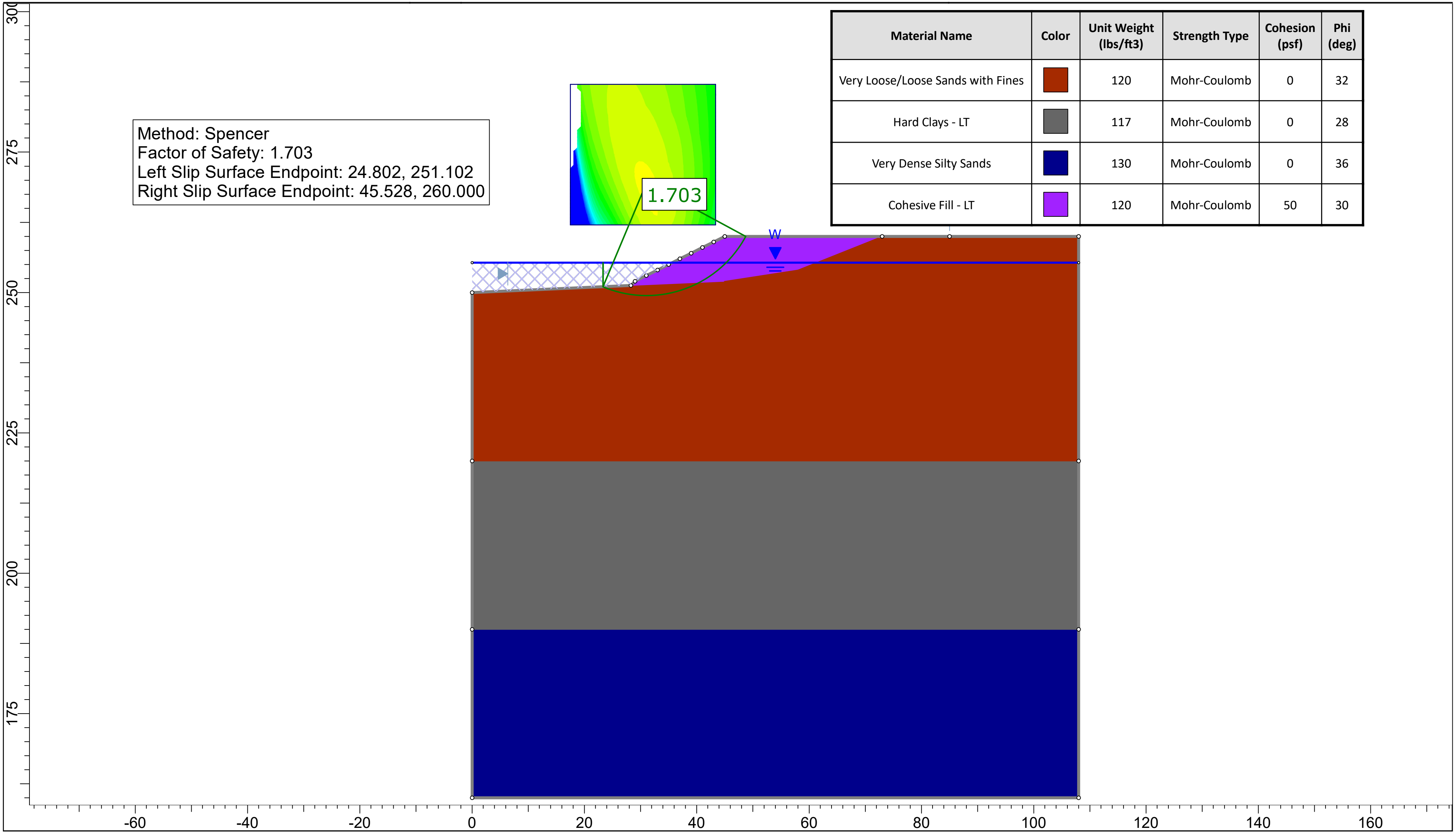
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Loose/Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays - ST	⬛	117	Mohr-Coulomb	2700	0
Very Dense Silty Sands	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: STA 110+00 Southern Side Slope.slmd
Name: Group 1
Description: Long Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - STA 110+00
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 1.703
Left Slip Surface Endpoint: 24.802, 251.102
Right Slip Surface Endpoint: 45.528, 260.000

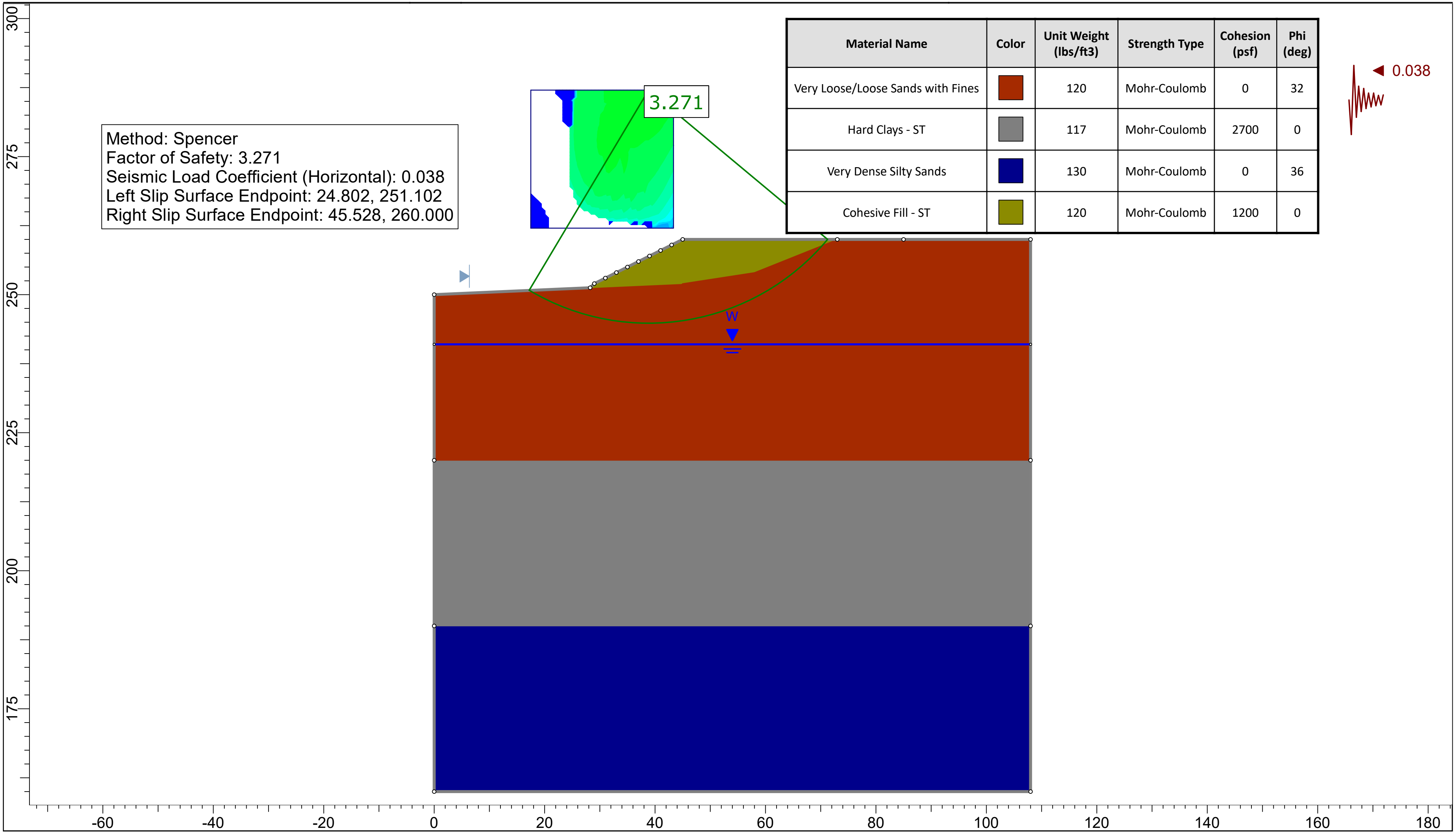
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Loose/Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays - LT	⬛	117	Mohr-Coulomb	0	28
Very Dense Silty Sands	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - LT	🟪	120	Mohr-Coulomb	50	30

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: STA 110+00 Southern Side Slope.slmd
Name: Group 1
Description: Seismic Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - STA 110+00
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 3.271
Seismic Load Coefficient (Horizontal): 0.038
Left Slip Surface Endpoint: 24.802, 251.102
Right Slip Surface Endpoint: 45.528, 260.000

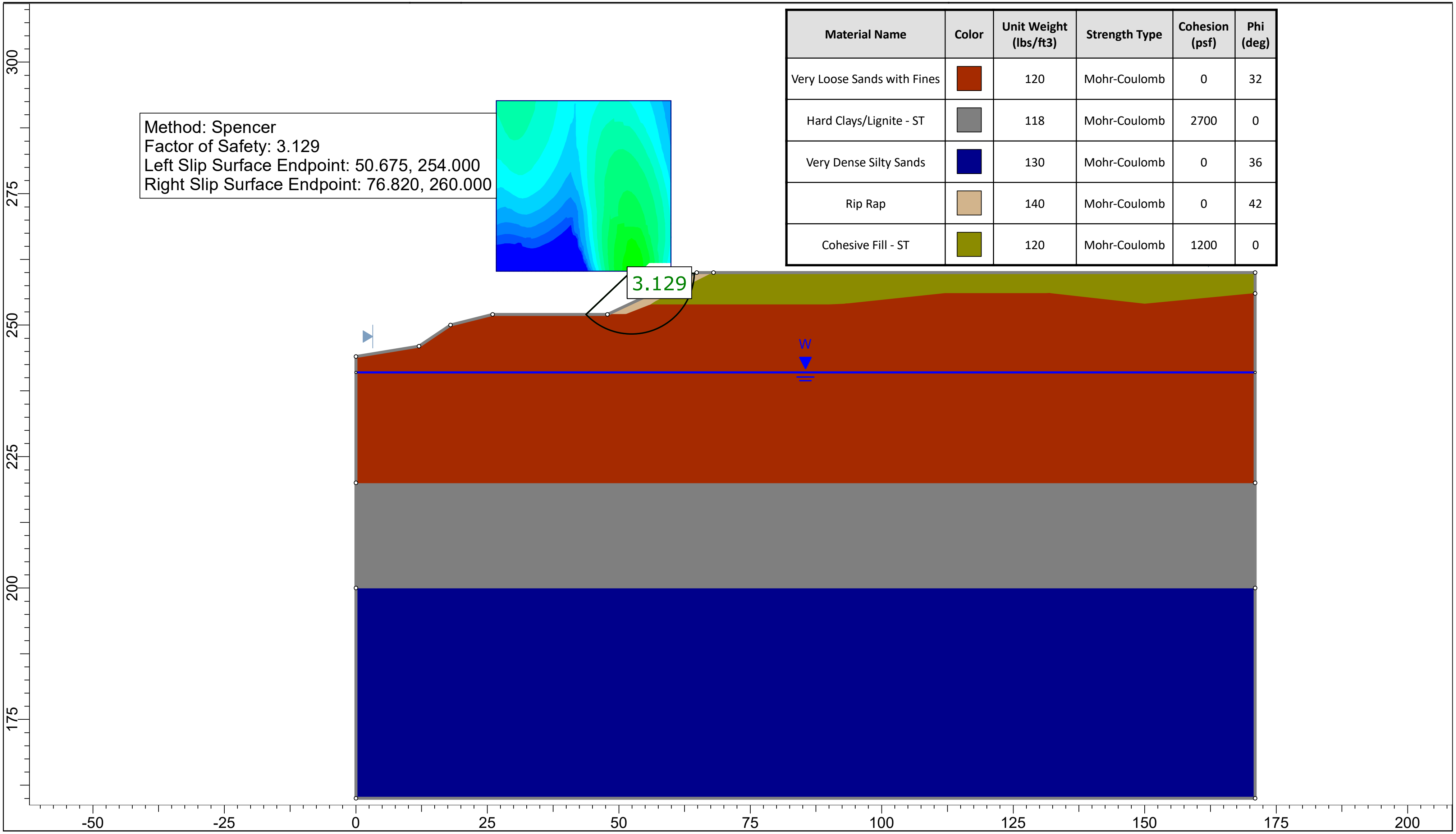
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Loose/Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays - ST	▨	117	Mohr-Coulomb	2700	0
Very Dense Silty Sands	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - ST	🟨	120	Mohr-Coulomb	1200	0

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: East Abutment.slmd
Name: Group 1
Description: Short Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - East Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 3.129
Left Slip Surface Endpoint: 50.675, 254.000
Right Slip Surface Endpoint: 76.820, 260.000

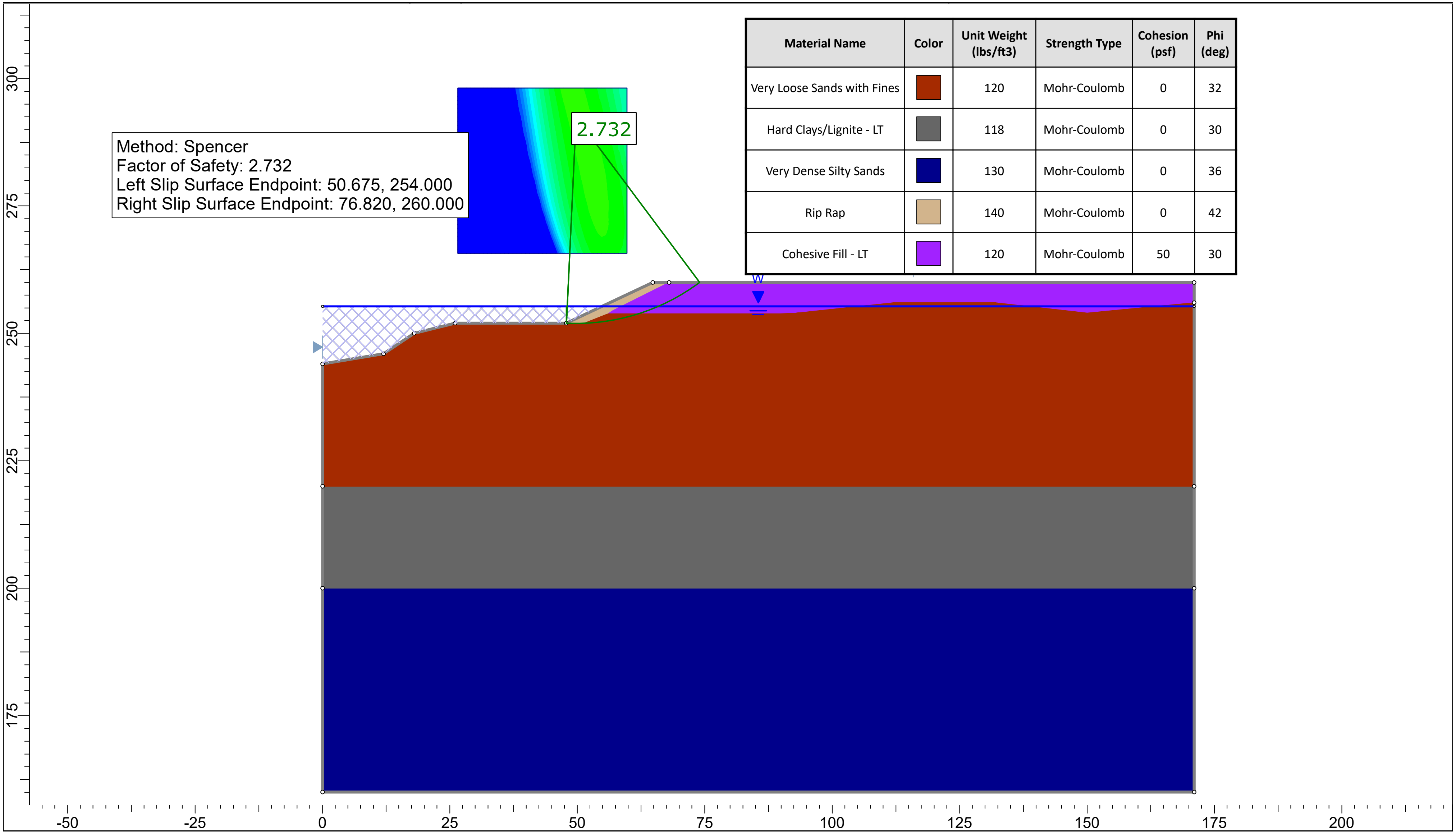
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays/Lignite - ST	⬛	118	Mohr-Coulomb	2700	0
Very Dense Silty Sands	🟦	130	Mohr-Coulomb	0	36
Rip Rap	🟫	140	Mohr-Coulomb	0	42
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: East Abutment.slmd
Name: Group 1
Description: Long Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - East Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 2.732
Left Slip Surface Endpoint: 50.675, 254.000
Right Slip Surface Endpoint: 76.820, 260.000

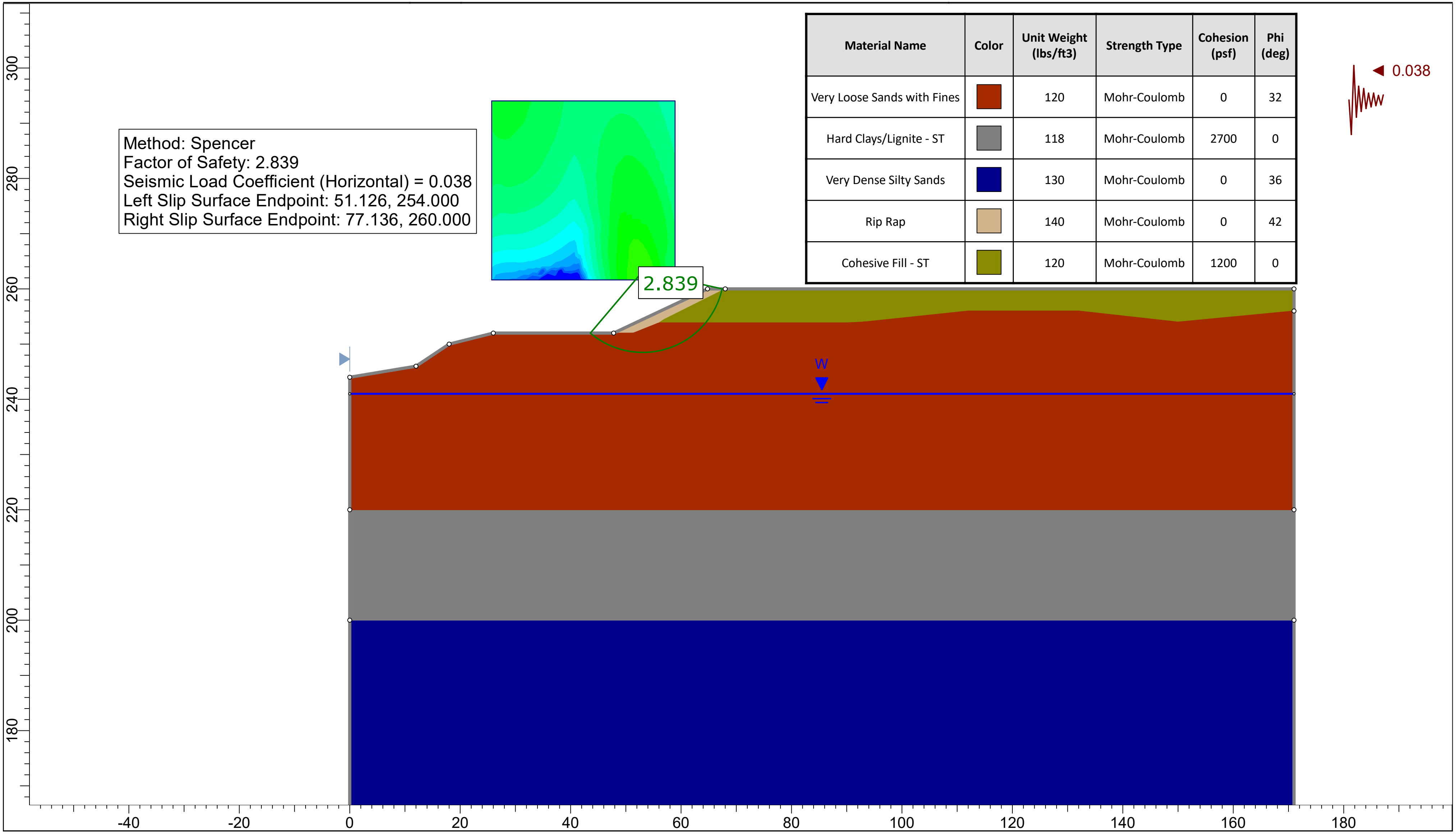
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays/Lignite - LT	⬛	118	Mohr-Coulomb	0	30
Very Dense Silty Sands	🟦	130	Mohr-Coulomb	0	36
Rip Rap	🟫	140	Mohr-Coulomb	0	42
Cohesive Fill - LT	🟪	120	Mohr-Coulomb	50	30

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: East Abutment.slmd
Name: Group 1
Description: Seismic Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - East Abutment
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 2.839
Seismic Load Coefficient (Horizontal) = 0.038
Left Slip Surface Endpoint: 51.126, 254.000
Right Slip Surface Endpoint: 77.136, 260.000

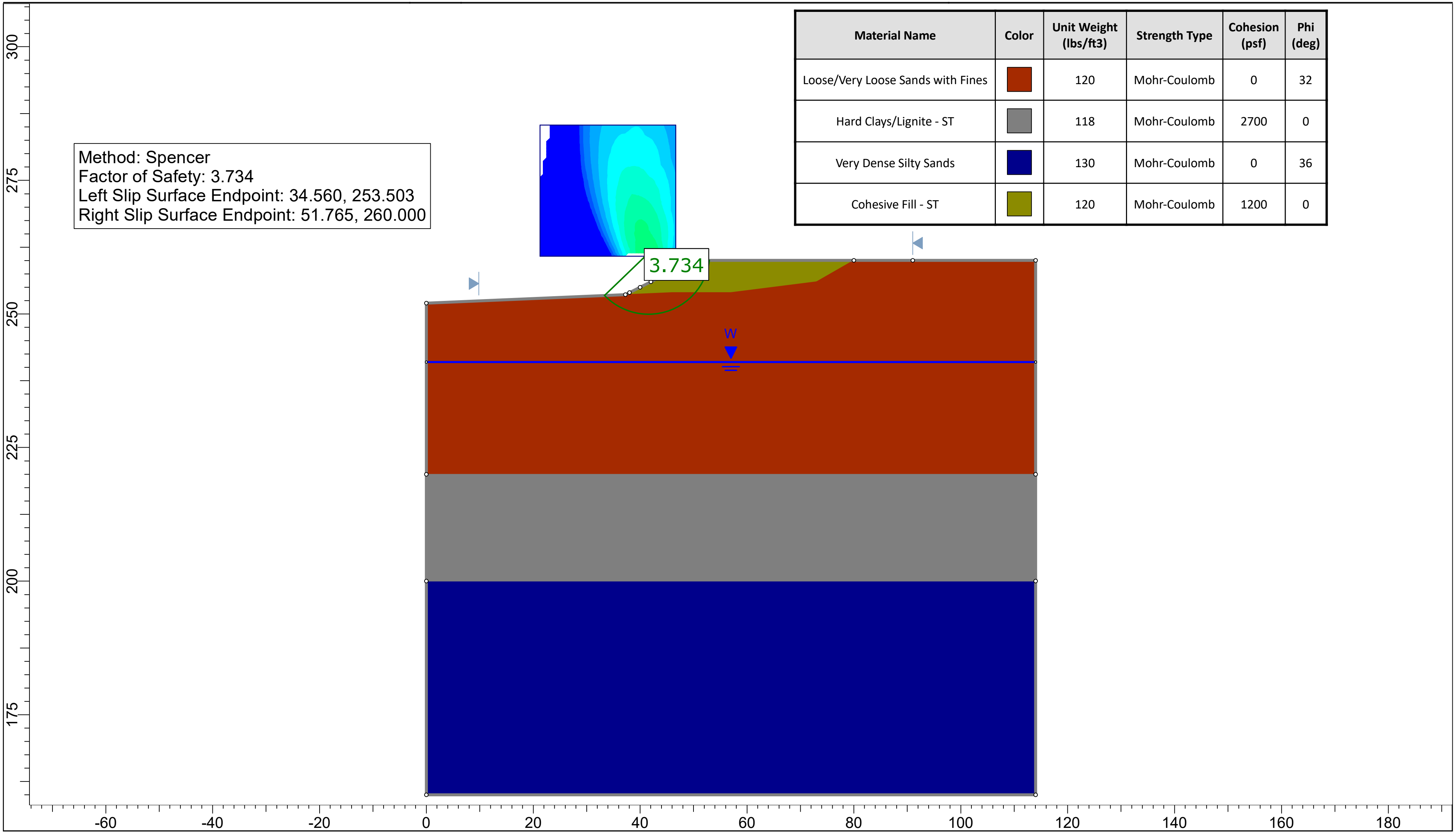
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays/Lignite - ST	⬛	118	Mohr-Coulomb	2700	0
Very Dense Silty Sands	🟦	130	Mohr-Coulomb	0	36
Rip Rap	🟨	140	Mohr-Coulomb	0	42
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: MC STA112+50 Southern Slope.slmd
Name: Group 1
Description: Short Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - STA 112+50
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 3.734
Left Slip Surface Endpoint: 34.560, 253.503
Right Slip Surface Endpoint: 51.765, 260.000

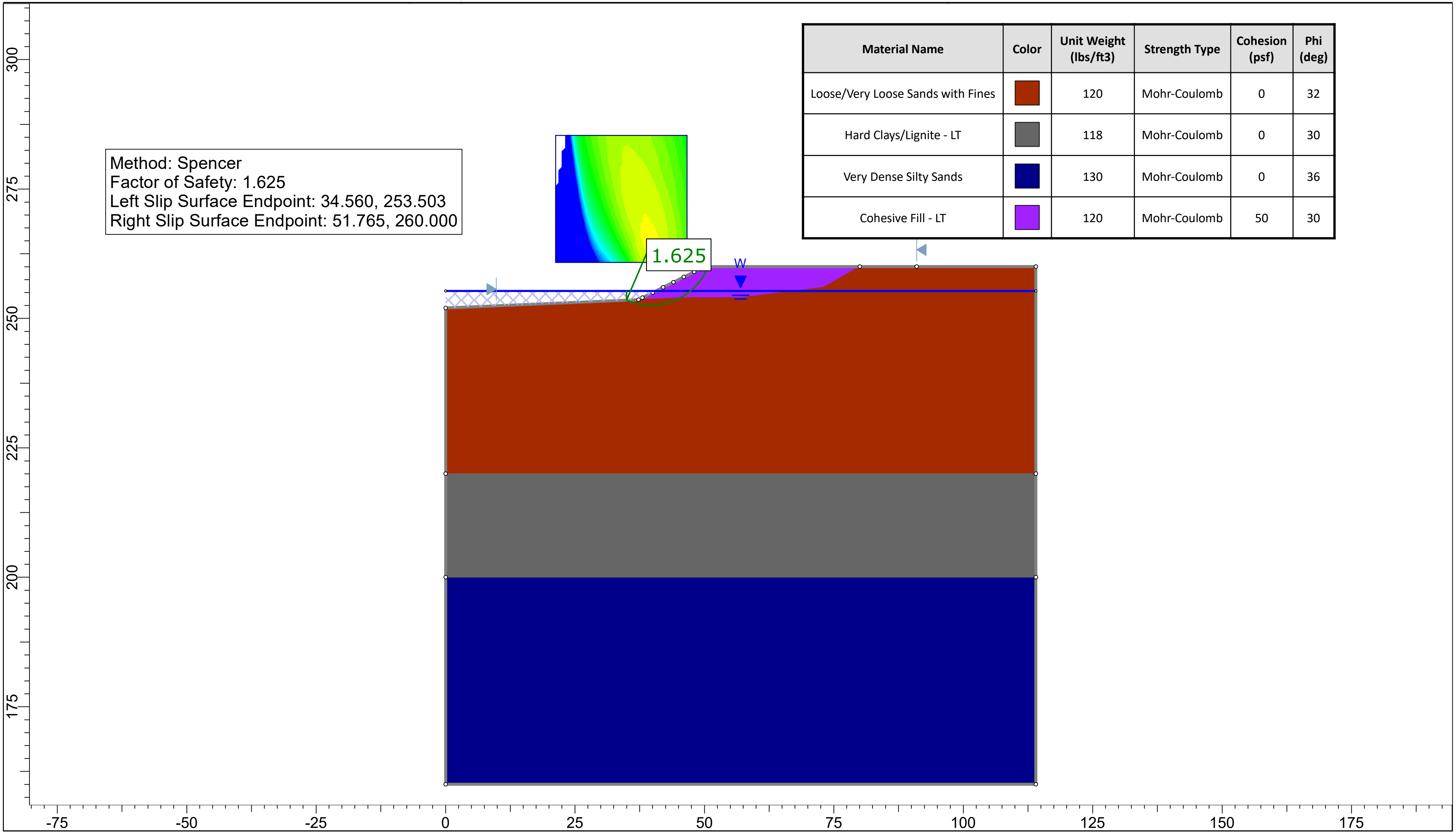
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose/Very Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays/Lignite - ST	⬛	118	Mohr-Coulomb	2700	0
Very Dense Silty Sands	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - ST	🟩	120	Mohr-Coulomb	1200	0

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: MC STA112+50 Southern Slope.slmd
Name: Group 1
Description: Long Term Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - STA 112+50
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 1.625
Left Slip Surface Endpoint: 34.560, 253.503
Right Slip Surface Endpoint: 51.765, 260.000

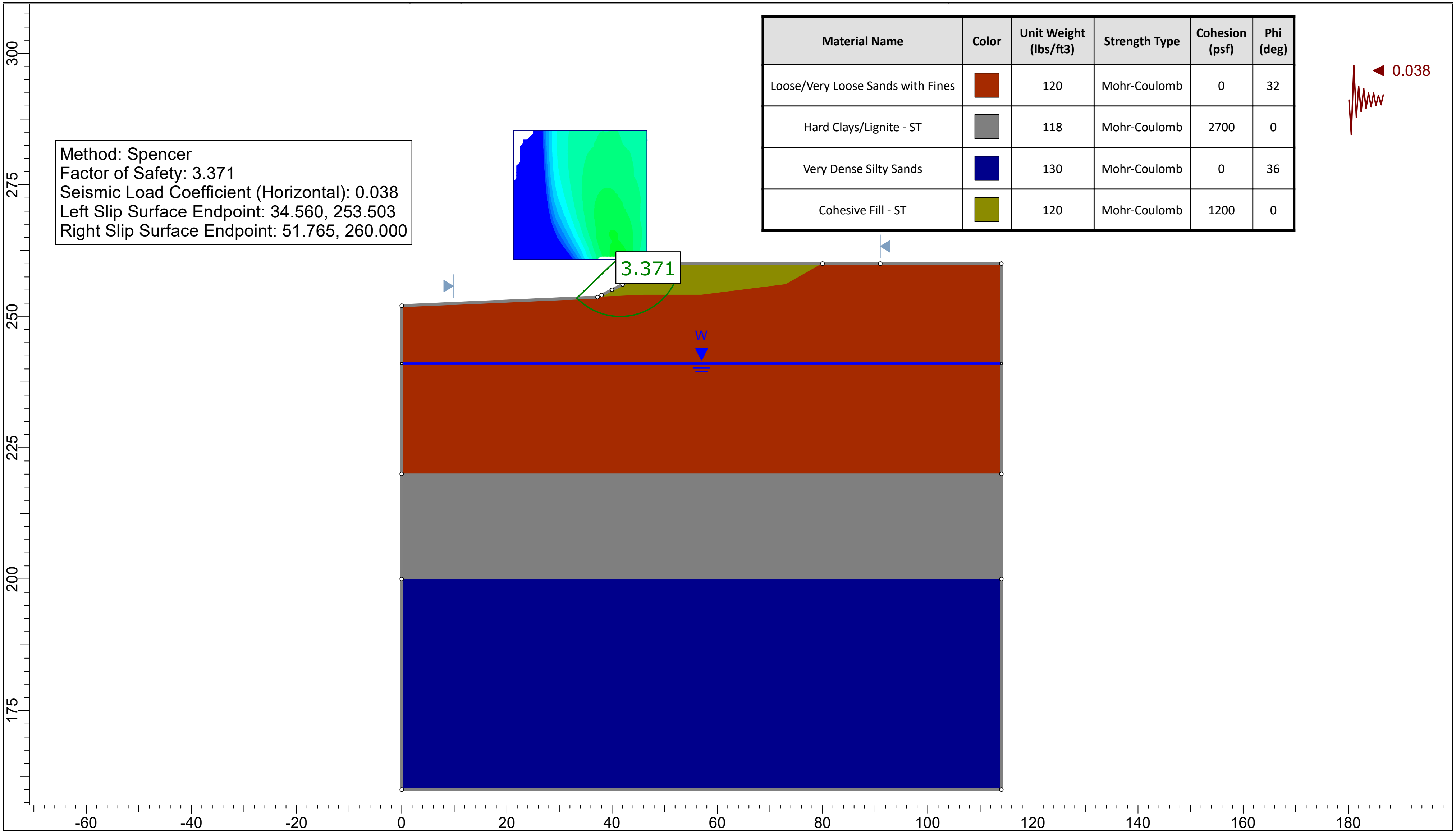
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose/Very Loose Sands with Fines	🟫	120	Mohr-Coulomb	0	32
Hard Clays/Lignite - LT	⬛	118	Mohr-Coulomb	0	30
Very Dense Silty Sands	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - LT	🟪	120	Mohr-Coulomb	50	30

GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: MC STA112+50 Southern Slope.slmd
Name: Group 1
Description: Seismic Conditions
Method: Spencer

Project Number: J028499.03
Client: Garver
Project: ArDOT 030497 - Hwy 82
Mill Creek - STA 112+50
Southern Side Slope
Date: 3/9/2020

SLIDEINTERPRET 8.031



Method: Spencer
Factor of Safety: 3.371
Seismic Load Coefficient (Horizontal): 0.038
Left Slip Surface Endpoint: 34.560, 253.503
Right Slip Surface Endpoint: 51.765, 260.000

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose/Very Loose Sands with Fines	🟥	120	Mohr-Coulomb	0	32
Hard Clays/Lignite - ST	▨	118	Mohr-Coulomb	2700	0
Very Dense Silty Sands	🟦	130	Mohr-Coulomb	0	36
Cohesive Fill - ST	🟨	120	Mohr-Coulomb	1200	0

APPENDIX G – SOIL PARAMETERS FOR SYNTHETIC PROFILES

MILL CREEK BRIDGE INTERNAL BENTS 2 & 3 – BORING MC-3

ASSUMED PILE CUTOFF ELEVATION: EL 250

ZONE	SOIL TYPES / LPILE SOIL[b]	DEPTH (ELEVATION)		WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD[b] PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[a]
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)		
1	Loose Sands with Silt / Sand (Reese)	250	222	120	--	33	--	33	--	20
2	Dense Silty Sands / Sand (Reese)	222	212	125	--	35	--	35	--	100
3	Hard Clay / Stiff Clay w/ Free Water (Reese)	212	207	117	2,700	--	--	28	0.004	800
4	Very Dense Sands with Silt / Sand (Reese)	207	170	130	--	36	--	36	--	125

[a] Pounds per cubic inch.

[b] For lateral load analysis only.

Groundwater assumed at El 241.

MILL CREEK BRIDGE WEST ABUTMENT – BORINGS MC-1 & MC-2

ASSUMED PILE CUTOFF ELEVATION: EL 256

ZONE	SOIL TYPES / LPILE SOIL[b]	DEPTH (ELEVATION)		WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD[b] PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)[a]
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)		
1	Very Loose/Loose Sands with Fines / Sand (Reese)	256	220	120	--	32	--	32	--	20
2	Hard Clays / Stiff Clay w/ Free Water (Reese)	220	190	117	2,700	--	--	28	0.004	700
3	Very Dense Sands with Silt / Sand (Reese)	190	160	130	--	36	--	36	--	125

[a] Pounds per cubic inch.
[b] For lateral load analysis only.

Groundwater assumed at El 241.

MILL CREEK EAST ABUTMENT – BORINGS MC-6 & MC-7

ASSUMED PILE CUTOFF ELEVATION: EL 256

ZONE	SOIL TYPES / LPILE SOIL	DEPTH (ELEVATION)		WET UNIT WEIGHT (PCF)	SHEAR STRENGTH PARAMETERS				LATERAL LOAD** PARAMETERS	
					UNDRAINED (SHORT TERM)		DRAINED (LONG TERM)		SOIL STRAIN, E_{50}	STATIC SOIL MODULUS (PCI)*
		FROM	TO		COHESION (PSF)	Φ (DEGREE)	EFFECTIVE COHESION (PSF)	Φ’ (DEGREE)		
1	Loose/Very Loose Sands with Fines / Sand (Reese)	256	220	120	--	32	--	32	--	20
2	Hard Clay/Lignite / Stiff Clay w/ Free Water (Reese)	220	200	118	2,700	--	--	30	0.004	700
3	Very Dense Sands with Silt / Sand (Reese)	200	160	130	--	36	--	36	--	125

[a] Pounds per cubic inch.

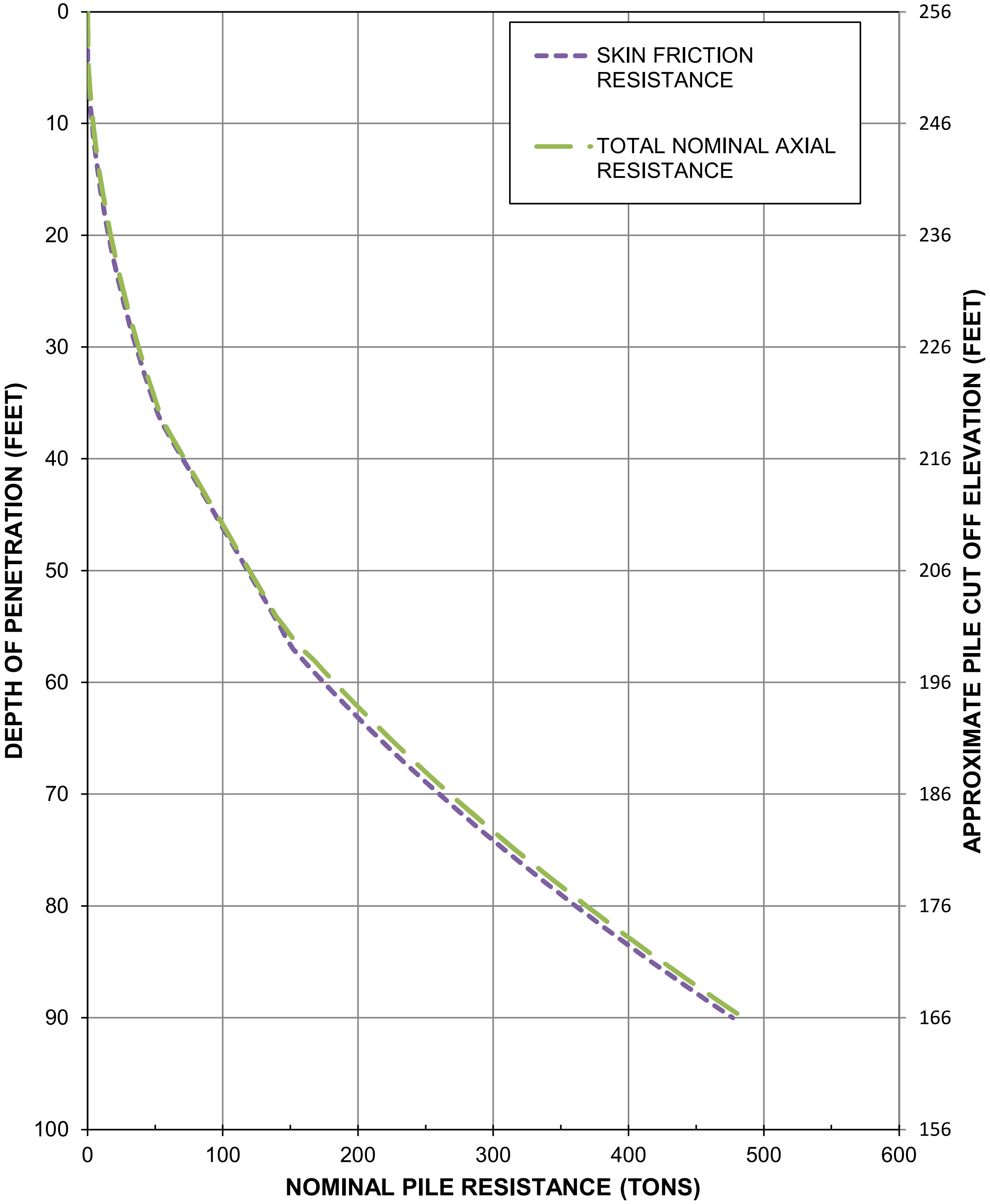
[b] For lateral load analysis only.

Groundwater assumed at El 241.

APPENDIX H – NOMINAL RESISTANCE CURVES

MILL CREEK BRIDGE EAST END BENT
ARDOT 030497 HWY 82

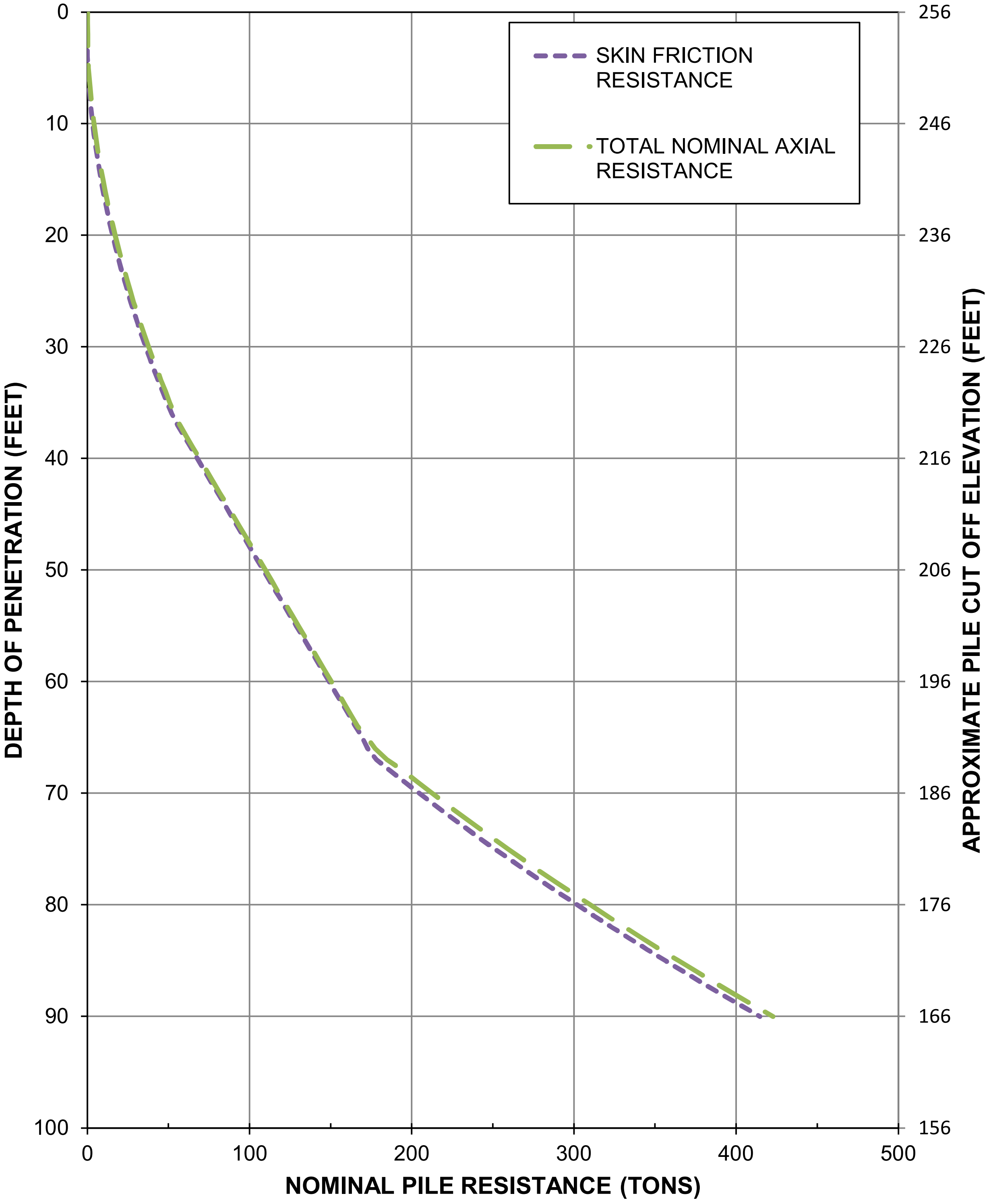
NOMINAL RESISTANCE CURVES
DRIVEN HP12X53 PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

MILL CREEK BRIDGE WEST END BENT
ARDOT 030497 HWY 82

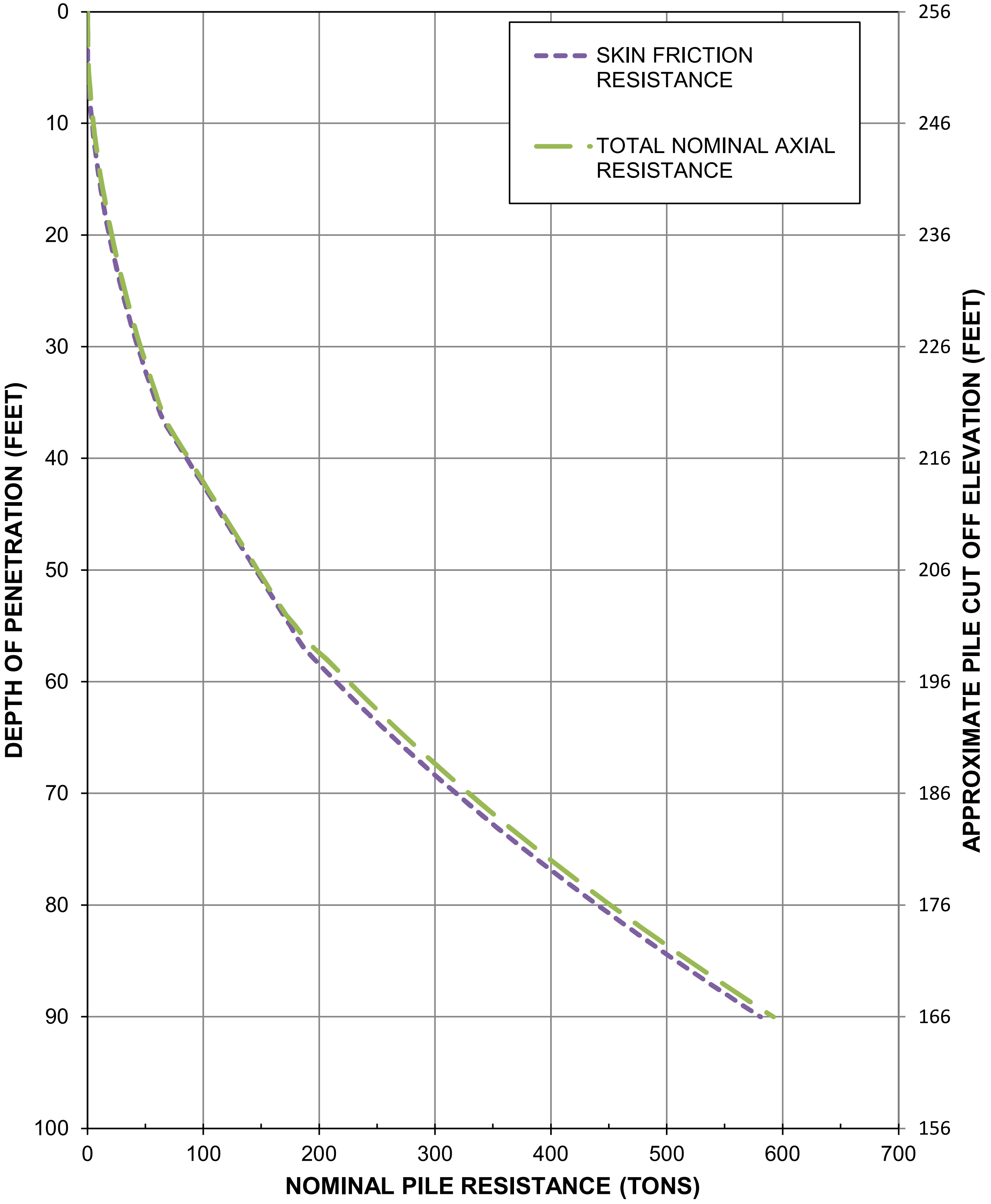
NOMINAL RESISTANCE CURVES
DRIVEN HP12X53 PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

MILL CREEK BRIDGE EAST END BENT
ARDOT 030497 HWY 82

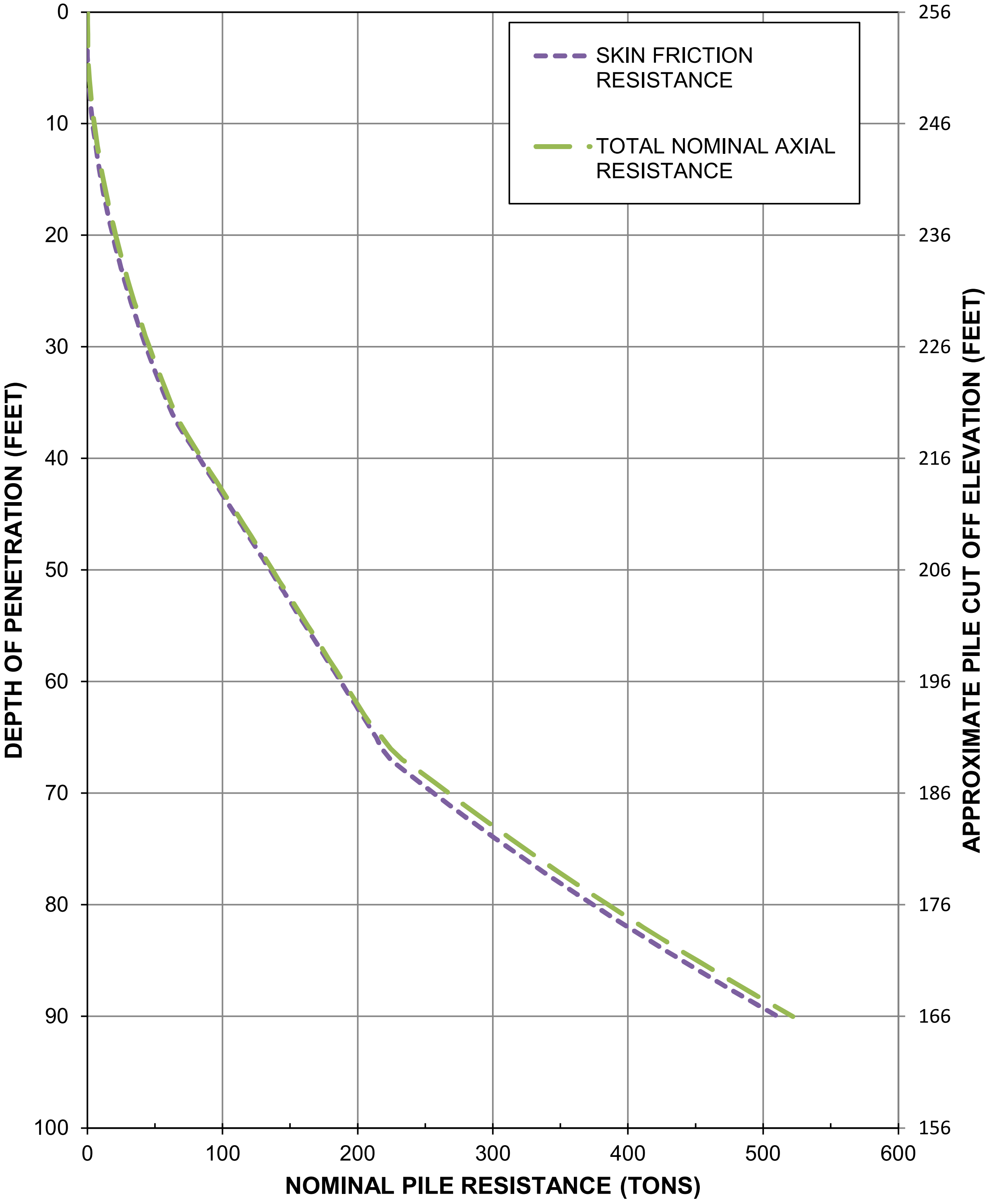
NOMINAL RESISTANCE CURVES
DRIVEN HP14X73 PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

MILL CREEK BRIDGE WEST END BENT
ARDOT 030497 HWY 82

NOMINAL RESISTANCE CURVES
DRIVEN HP14X73 PILES



GEOTECHNOLOGY PROJECT NUMBER J028499.03

MILL CREEK BRIDGE INTERMEDIATE BENTS
ARDOT 030497 HWY 82

NOMINAL RESISTANCE CURVES
DRIVEN 20 INCH, CLOSED-ENDED, PIPE PILES