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February 24, 2021

Cindy Rich, P.E.
Neel-Schaffer, Inc.
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Jackson, Mississippi 39201

Report No. 200518 – Site 1

Geotechnical Exploration
Site 1
ARDOT SR230 Bridge Replacements
Craighead and Lawrence Counties, Arkansas

Dear Ms. Rich:

Submitted here is the report of our geotechnical exploration for the above-captioned project. This exploration was authorized by Task Order 108 to the Subconsultant Agreement between Neel-Schaffer, Inc. and Burns Cooley Dennis, Inc. dated September 17, 2020.

We appreciate the opportunity to be of service. If you should have any questions concerning this report, please do not hesitate to call us.

Very truly yours,

BURNS COOLEY DENNIS, INC.

Alexander B. Reeb, Ph.D., P.E.

A. E. (Eddie) Templeton, P.E.

ABR/AET/khb
Copy Submitted: (via e-mail)

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1.0 INTRODUCTION

1.1 Project Description

Plans are being made for the construction of replacement bridges and box culverts at ten sites along Highway 230 between Alicia and Bono in Craighead and Lawrence Counties, Arkansas. Site 1 is located in Lawrence County where Highway 230 crosses Village Creek. At this site, a new bridge will be constructed on a new alignment just north of the existing bridge.

The new bridge will be 120 ft long and consist of three spans of approximately equal spacing. It is our understanding that new fill will be placed to raise the grade at the new abutments above the grade of the existing bridge. The abutment spill-through slopes will be constructed as 2H:1V slopes, and the abutment side slopes will be constructed as 3H:1V slopes. The abutment bents are to be supported by 18-in. diameter, closed-ended steel pipe piles, and the interior bents are to be supported by 24-in. diameter, closed-ended steel pipe piles. A preliminary layout showing the proposed construction is presented on Figure 1 of this report.

1.2 Purposes

The specific purposes of this exploration were:

1) to review the exploratory soil borings made within the area planned for construction of the new bridge;

2) to verify field classifications and to evaluate pertinent physical properties of the soils encountered in the borings by means of visual examination of the soil samples in the laboratory and routine tests performed on the samples;

3) to perform analyses to investigate liquefaction, slope stability, settlement, pile capacity, and downdrag; and

4) to provide geotechnical recommendations for design and construction of the bridge.

Our scope of work for the bridge does not include providing recommendations for roadway subgrades and pavements. Discussion and recommendations pertaining to roadway subgrades and pavements are provided under separate cover.

2.0 FIELD EXPLORATION

2.1 General

Subsurface soil conditions within the area planned for construction of the bridge were explored by means of four deep borings. Borings S1-1, S1-2, S1-3, and S1-4 were performed by McCray Drilling under contract to SoilTech Consultants, Inc. The approximate locations of the borings are shown on Figure 1.

All soils were classified in general accordance with the Unified Soil Classification System. A synopsis of the Unified Soil Classification System (USCS) is presented on Figure 2 along with symbols and terminology typically utilized on graphical soil boring logs. Graphical logs of the borings are presented on Figures 3 through 6. The graphical logs illustrate the types of soil and stratification encountered with depth below the existing ground surface at the individual boring locations. Approximate GPS coordinates for the boring locations are shown at the bottom of the graphical boring logs within the “Comments” section.

2.2 Drilling Methods and Groundwater Observations

The four borings were made to an exploration depth of 100 ft using a CME-750X buggy-mounted drill rig. Borings S1-1, S1-2, S1-3, and S1-4 were initially advanced to a depth of 55 ft, 45 ft, 30 ft and 50 ft, respectively, by dry augering and then were extended to completion using rotary wash drilling procedures. Groundwater was encountered at a depth of 44.5 ft, 44 ft, 27 ft, and 48 ft in Borings S1-1, S1-2, S1-3, and S1-4, respectively.

2.3 Sampling Methods

Disturbed samples of soils were obtained by driving a standard 2-in. OD split-spoon sampler 18 in. into the soil with a 140-lb hammer falling freely a distance of 30 in. The depths at which the split-spoon samples were taken are illustrated as crossed rectangular symbols under the "Samples" column of the graphic logs. Standard penetration test (SPT) blow counts resulting from split-spoon sampling are recorded under the "Blows Per Ft" column of the graphic logs. The SPT blow counts are the “raw” field values. The recommended hammer energy correction factor is indicated in the “Comments” section of the logs. Relatively undisturbed samples of the soils encountered in the borings were obtained by pushing a 3-in. OD Shelby tube sampler approximately 2 ft into the soil. The Shelby tube samples were obtained within the depth intervals illustrated as shaded portions of the "Samples" column of the graphic logs. The Shelby tube and/or split-spoon samples were generally obtained at approximate 3-ft to 5-ft intervals of depth. Disturbed auger cutting samples were taken near the ground surface in the borings. The depths at which the auger cutting samples were taken are illustrated as small I-shaped symbols under the "Samples" column of the graphic boring logs.

2.4 Field Classification, Sample Preservation and Borehole Abandonment

All soils encountered during drilling were examined and classified in the field by a geotechnical engineering technician. Representative portions of the split-spoon samples and the auger cutting samples were sealed in jars to provide material for visual examination and testing in the laboratory. The Shelby tubes were capped and the ends sealed with wax in the field to prevent moisture loss and structural disturbance while they were transported to the testing laboratory. At the testing laboratory, the Shelby tube samples were extruded, and an approximate 6-in. long portion of each sample was temporarily sealed in plastic wrap to prevent moisture loss during the period between sample extrusion and testing. Additional portions of each Shelby tube sample were sealed in jars to provide additional material for visual examination and testing. The boreholes were grouted after completion of drilling and sampling.

3.0 LABORATORY TESTING

3.1 General

All of the soil samples were examined in the laboratory and tests were performed on selected samples to verify field classifications and to assist in evaluating the strength and volume change properties of the soils encountered. The types of laboratory tests performed are described in the following paragraphs.

3.2 Strength Properties

The undrained shear strength characteristics of the fine-grained soils encountered in the borings were investigated by means of visual estimates of consistency and from the results of unconfined compression tests and unconsolidated undrained (UU) triaxial compression tests performed on selected undisturbed Shelby tube samples. The results of the unconfined compression tests in terms of cohesion are plotted as small open circles in the data sections of the graphic logs. The cohesions resulting from the UU triaxial compressions test are plotted as small open triangles in the data section of the graphic boring logs. The water content and dry density were also determined for each unconfined and UU triaxial compression test specimen. The water contents are plotted as small shaded circles in the data section of the graphic logs. The dry densities are tabulated to the nearest lb per cu ft under the “Dry Density” column of the graphic boring logs.

3.3 Consolidation Tests

The compressibility characteristics of the fine-grained soils encountered in the borings were investigated by means of a one-dimensional consolidation test performed on a representative undisturbed Shelby tube sample. The results of the consolidation test, including a plot of void ratio versus effective vertical stress, are presented in Appendix A.

3.4 Classification Tests

The classifications and volume change properties of the fine-grained soils encountered in the borings were investigated by means of Atterberg liquid and plastic limit tests performed on selected representative samples. The results of the liquid and plastic limit tests are plotted as small crosses interconnected by dashed lines in the data section of the graphic boring logs. In accordance with the Unified Soil Classification System, fine-grained soils are classified as either clays or silts of low or high plasticity based on the results of Atterberg limit tests. The numerical difference between the liquid limit and plastic limit is defined as the plasticity index (PI). The magnitudes of the liquid limit and plasticity index and the proximity of the natural water content to the plastic limit are indicators of the potential for a fine-grained soil to shrink or swell upon changes in moisture content or to consolidate under loading. The proximity of the natural water content to the plastic limit is also an indicator of soil strength.

The classifications of some samples were investigated by means of minus No. 200 sieve tests. The percentages of fines resulting from the minus No. 200 sieve tests are tabulated at the appropriate depths under the “% Passing No. 200 Sieve” column of the graphic boring logs.

The classifications of some samples were investigated by means of sieve and hydrometer analyses. Particle size distribution curves from these tests are presented in Appendix A. The percentages of fines resulting from the sieve tests are also tabulated at the appropriate depths under the “% Passing No. 200 Sieve” column of the graphic boring logs

3.5 Water Content Tests

Water content tests were performed on samples to corroborate field classifications and to extend the usefulness of the strength, plasticity, and field SPT blow count data. The results of the water content tests are plotted as small shaded circles in the data section of the graphic boring logs. The water content data have been interconnected on the logs to illustrate a continuous profile with depth.

3.6 Soluble Sulfates, pH, and Resistivity Tests

Laboratory testing was performed on selected samples from the borings to determine the percent of soluble sulfate by mass, soil pH, and soil resistivity. Sulfate testing was performed on all five samples, and soil pH and resistivity testing was performed on three of the five samples. Results of the tests are presented in Table 1.

Table 1 - Soluble Sulfates, pH, and Resistivity Test Results

Boring	Sample Depth (ft)	USCS	Sulfate (SO_4), % by mass	Average pH	Resistance (ohm-cm)
S1-2	5	CL	0.012	7.37	1400
S1-2	53.5	SP	0.024	-	-
S1-3	4	CL	0.033	6.84	1100
S1-4	4	CL	0.034	6.71	1600
S1-4	78.5	SP	0.019	-	-

4.0 GENERAL SUBSURFACE CONDITIONS

4.1 General

A general description of subsurface soil and groundwater conditions revealed by the borings made for this exploration is provided in the following paragraphs. The graphical logs shown on Figures 3 through 6 should be referred to for specific soil and groundwater conditions encountered at each boring location. Stick logs of the borings are shown in profile with the proposed bridge section on Figure 7 to aid in visualizing subsurface soil conditions. Tabulated adjacent to the stick logs are Atterberg liquid and plastic limits, water contents, dry densities, cohesions, percentages of fines passing the No. 200 sieve and field SPT blow counts.

4.2 Geology

The project site is located within the physiographic province known as the Mississippi River Alluvial Plain. Geological maps indicate Quaternary age deposits are continuous throughout the project area. The Quaternary deposits at the site include alluvial sediments from both the Holocene and Pleistocene series. Sediments typically include a substratum zone of sands and gravels overlain by a top stratum of clays and silts.

Tertiary deposits are present below the Quaternary deposits. Tertiary deposits within the project vicinity are expected to consist of hard clays, sandy clays and silty clays containing organics and lignite interbedded with very dense sand strata. Geological maps suggest that the elevation of top of the Tertiary deposits may be at about El 100 to 125 ft MSL.

4.3 Soil Stratification

As shown on the Figure 7 profile, the soils encountered at the site were grouped into the zones outlined below. The zones were generally based on the soil classifications and interpreted strengths used in design. The borings generally indicate fill materials and fine-grained top stratum soils overlying alluvial sands.

- Zone 1 – Soft to stiff silty clay (CL) and medium stiff to very stiff clay (CH)
- Zone 2 – Loose to dense sand (SP) with trace of gravel and slightly silty sand (SP-SM), and very loose to loose silty sand (SM)
- Zone 3 – Medium dense to dense sand (SP) with trace of gravel

Zone 1 soils were generally encountered from the ground surface down to depths ranging from about 15 to 20 ft. Zone 2 soils were encountered beneath the Zone 1 soils down to depths ranging from about 87 to 95 ft. Zone 3 soils were encountered beneath the Zone 2 soils and extend to the boring termination depths.

Zone 2 was further divided into Zones 2A, 2B, and 2C based on the estimated likelihood of liquefaction and potential for strength loss due to an earthquake. The soils encountered in Zones 2A and 2C were generally identified as having a high likelihood of liquefaction and significant strength loss. The soils encountered in Zone 2B were generally identified as having a moderate likelihood of liquefaction but no significant strength loss.

We understand that new fill materials will be placed along the new alignment to create the approach embankments. The thickness of the proposed new fill at abutments along the bridge centerline is illustrated on the profile.

4.4 Groundwater

Groundwater was encountered during auger drilling at a depth of 44.5 ft, 44 ft, 27 ft, and 48 ft in Borings S1-1, S1-2, S1-3, and S1-4, respectively. Groundwater cannot be observed during rotary wash drilling. In our opinion, groundwater conditions at the site will be influenced by rainfall, surface drainage, and by the rise and fall of water levels in the nearby ditches, creeks, ponds or other bodies of water. The regional groundwater is primarily influenced by the Mississippi River. Groundwater conditions at the site can also be influenced by man-made changes. Surficial soils can become saturated and weak to relatively shallow depths during periods of prolonged and heavy rainfall.

5.0 ENGINEERING ANALYSES AND DISCUSSION

5.1 General

The purposes of this study were to perform analyses and develop geotechnical recommendations for: 1) seismic design including site classification, liquefaction, and seismic compression; 2) slope stability including proposed slope grading and configuration to provide acceptable factors of safety; and 3) deep foundation design including axial capacity curves, downdrag, lateral analysis parameters, and drivability analysis. A discussion of our analyses is provided in the following subsections.

5.2 Seismic

Seismic evaluations and analyses were generally performed based on the guidance provided by ARDOT and the recommendations discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual and in Idriss and Boulanger (2008).

5.2.1 Site Classification. Soil shear wave velocity data are not available for the bridge site. The site class was determined from SPT blow counts and undrained shear strength data in accordance with definitions provided in Table 3.10.3.1-1 of the AASHTO LRFD 2017 Bridge Design Specifications. We recommend that a site class E be utilized to determine the site coefficient and spectral response acceleration for this bridge site. The site is classified as within Seismic Zone 4 per Table 3.10.6 1.

The acceleration design response spectrum was developed using the computer program "AASHTO Seismic Design Parameters" version 2.10 developed by the U.S. Geological Survey.

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The recommended design values are presented subsequently in tabular format. Plots of the design spectrum are included as Figures 8 and 9.

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 35.894250
Longitude = -91.072980

Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.348	PGA - Site Class B
0.2	0.667	Ss - Site Class B
1.0	0.171	S1 - Site Class B

Spectral Response Accelerations SDs and SD1
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1

Site Class E - Fpga = 1.06, Fa = 1.37, Fv = 3.29
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.369	As - Site Class E
0.2	0.914	SDs - Site Class E
1.0	**0.563**	**SD1 - Site Class E: Seismic Zone 4**

Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	Sd in.	
0.000	0.369	0.000	T = 0.0, Sa = As
0.123	0.914	0.136	
0.200	0.914	0.357	T = 0.2, Sa = SDs
0.616	0.914	3.390	T = Ts, Sa = SDs
0.700	0.805	3.852	
0.800	0.704	4.402	
1.000	0.563	5.503	T = 1.0, Sa = SD1
1.200	0.469	6.603	
1.400	0.402	7.704	
1.600	0.352	8.804	
1.800	0.313	9.905	
2.000	0.282	11.005	
2.200	0.256	12.106	
2.400	0.235	13.207	
2.600	0.217	14.307	
2.800	0.201	15.408	

3.000	0.188	16.508
3.200	0.176	17.609
3.400	0.166	18.709
3.600	0.156	19.810
3.800	0.148	20.910
4.000	0.141	22.011

5.2.1 Liquefaction Triggering. Liquefaction triggering evaluations were performed using the Microsoft Excel workbook developed by Cox and Griffiths (2011)[1] and provided by ARDOT. The liquefaction evaluations were performed using all three procedures available in the workbook: Youd et al. (2001)[2], Cetin et al. (2004)[3], Idriss and Boulanger (2008)[4].

The design earthquake magnitude (M_w) was estimated using the Unified Hazard Tool on the U.S. Geological Survey (USGS) website. Deaggregations were computed using the 2008 (v3.3.3) edition of the National Seismic Hazard Mapping Project (NSHMP). A return period of 5% in 50 years (i.e., 975 years) was used in the deaggregation. The resulting modal earthquake magnitude of 7.7 was input in the liquefaction triggering workbook.

The liquefaction triggering evaluation was performed for each of the borings. The liquefaction triggering workbook input is provided for each boring in Appendix B. As recommended by Cox and Griffiths (2011), a blow count N-value of 1 was used input in the workbook at sample depths where SPT blow counts were not measured. For these cases, the Factor of Safety (FS) against liquefaction was not calculated. Comparison plots that show the resulting liquefaction FS values vs. elevation for each of the three evaluation procedures are provided as Figures 10, 11, 12, and 13 for Borings S1-1, S1-2, S1-3, and S1-4, respectively.

5.2.2 Seismic Compression. Potential seismic compression was calculated for all soil layers that were identified as likely to liquefy (i.e., FS<=1.0) based on the Idriss and Boulanger

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[1] Cox, B. R., and Griffiths, S. C. (2011). *Practical Recommendations for Evaluation and Mitigation of Soil Liquefaction in Arkansas*, MBTC 3017, Mack-Blackwell Rural Trans. Center at the U. of Arkansas.

[2] Youd, T. L., Idriss, I.M., et al. (2001). "Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops of evaluation of liquefaction resistance of soils." *J. of Geotech. and Geoevir. Engrg*., Vol. 127(4): 297-313.

[3] Cetin, K.O., Seed, R.B., Kiureghain, A.D., Tokimatsu, K., Harder, L.F., Kayen, R.E., Moss, R.E.S. (2004). “Standard Penetration Test-Based Probabilistic and Deterministic Assesment of Seismic Soil Liquefaction Potential.” *J.of Geotech. and Geoevir. Engrg*., Vol. 130(12): 1314-1340.

[4] Idriss, I. M., and Boulanger, R. W. (2008). "Soil Liquefaction during Earthquakes." *MNO-12*, Earthquake Engineering Research Institute.

(2008) liquefaction triggering criteria. The seismic compression calculations were performed following two different procedures: Tomkimatsu & Seed (1987)[5] and Idriss and Boulanger (2008). The Tomkimatsu & Seed (1987) procedure for calculating seismic compression is discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

Plots that show the distribution of estimated seismic compression vs. elevation for the two procedures are provided as Figures 14, 15, 16, and 17 for Borings S1-1, S1-2, S1-3, and S1-4, respectively. For reference, the top and bottom elevation of the boring is indicated by a horizontal dashed line on each plot. As shown in these figures, the total estimated settlements at the boring locations due to seismic compression range from about 11 to 20 inches depending on the analysis method.

5.2.3 Residual Strengths of Liquefied Soils. Residual strengths for post-earthquake stability analyses were estimated for soils that were identified as likely to liquefy (i.e., FS<=1.0) based on the Idriss and Boulanger (2008) liquefaction triggering criteria. The residual strengths were estimated using the procedures outlined in Idriss and Boulanger (2008) and based on the correlation proposed by Olson and Johnson (2008)[6]. The correlations proposed by Olson and Johnson (2008) are included in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

5.3 Slope Stability

Slope stability analyses were performed for the proposed conditions using the SLOPE/W computer program and the Spencer Method. The stability analyses were performed for end of construction, long term, pseudo-static, and post-seismic conditions. We understand that the target factors of safety are 1.5 for end of construction and long-term conditions, and 1.1 for pseudo-static and post-earthquake conditions. Analyses were performed for the spill-though slopes and for the embankment side slopes. A traffic surcharge load of 250 psf was applied in pavement areas in the analyses.

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[5] Tokimatsu, K. and Seed, H.B. (1987). “Evaluation of settlements in sand due to earthquake shaking.” *J. of Geotech. Engrg.*, Vol. 113(8): 861-878.

[6] Olson, S. M. and Johnson, C. I. (2008). “Analyzing Liquefaction-induced Lateral Spreads Using Strength Ratios.” *J. of Geotech. and Geoenviron. Engrg.*, 134(8): 1035–1049.

The end of construction analyses use undrained strengths for cohesive soils and drained strengths for cohesionless soils. The long-term analyses use drained strengths for all soils. The pseudo-static analyses use undrained strengths for cohesive soils, drained strengths for cohesionless soils, and include a seismic coefficient equal to 0.5 times the site class specific PGA (i.e., 0.5*F_{PGA}*PGA) as suggested in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual. The post-earthquake analyses use undrained strengths for cohesive soils, residual strengths for cohesionless soils that were identified as likely to liquefy, and drained strengths for cohesionless soils that were not identified as likely to liquefy. For cohesive soils that were estimated to have peak undrained strengths of approximately 1,500 psf or less, undrained strengths equal to 0.8 times the peak undrained strengths were used in the post-earthquake analyses to account for possible cyclic softening.

The stability analyses indicate that slope stabilization measures are required to achieve acceptable factors of safety for pseudostatic and post-seismic conditions, and the slope stabilization could be accomplished with multiple layers of geosynthetic reinforcement. In our analyses, we assumed that the geosynthetic reinforcement would have an allowable tensile strength of 5,200 lbs/ft. Each geosynthetic layer shall be continuous along its length, and it shall be placed to lay flat, pulled tight and pinned or weighted down to its position until the subsequent soil layer can be placed.

At the west approach embankment, 3 layers of geosynthetic reinforcement that are oriented parallel to the roadway alignment are required to stabilize the west abutment spill-through slope. The geosynthetic should extend from the mid-point of the spill-through slope back about 75 ft to at least Sta. 109+80. The bottom layer of geosynthetic should extend back an additional 16 ft to at least Sta. 109+64. The geosynthetic should be placed such that the full width of the embankment is covered between the top edges of the side slopes, the distance between which measures about 40 ft. The layers of geosynthetic should be placed at 2-ft vertical spacing, and the bottom layer should be placed at the bottom of the embankment.

At the east approach embankment, 6 layers of geosynthetic reinforcement that are oriented parallel to the roadway alignment are required to stabilize the west abutment spill-through slope. The geosynthetic should extend from the mid-point of the spill-through slope back about 100 ft to at least Sta. 112+72. The bottom layer of geosynthetic should extend back an additional 16 ft to at least Sta. 112+88. The geosynthetic should be placed such that the full width of the embankment is covered between the top edges of the side slopes, the distance between which measures about 40 ft. The layers of geosynthetic should be placed at 2-ft vertical spacing, and the bottom layer should be placed at the bottom of the embankment.

At the east approach embankment, an additional layer of geosynthetic reinforcement that is oriented perpendicular to the roadway alignment is required to stabilize the north side slope. The geosynthetic should extend from the mid-point of the north side slope of the proposed approach embankment back to and against the north side slope of the existing approach embankment. This additional layer of geosynthetic is required between Sta 111+72 and 112+88. This additional layer of geosynthetic should be placed at 1-ft vertical spacing above the bottom layer of geosynthetic that is to be placed parallel to the roadway alignment.

Additional layers of geosynthetic reinforcement that are oriented perpendicular to the roadway alignment are not required at the west approach embankment.

A summary of the slope stability Factor of Safety (FS) values is provided in Table 2. The analyzed geometries, soil properties, and critical failure surfaces are shown in Figures 18 to 33.

Table 2 - Slope Stability FS Results Summary

Conditions	Req’d	West Abutment Spill-Through	East Abutment Spill-Through	West Abutment North Side Slope (110+43)	East Abutment North Side Slope (111+72)
End of Construction	1.5	3.12	2.91	3.16	3.29
Long Term	1.5	2.38	2.82	1.73	2.88
Pseudostatic	1.1	1.71	1.11	1.57	1.20
Post-Earthquake	1.1	1.10	1.12	1.36	1.15

5.4 Embankment Consolidation Settlement

Settlement analyses were performed using the computer program Settle3D by Rocscience to estimate compression of the natural soils. Soil stratification and parameters used in the analyses were based on the conditions encountered at the boring locations and the results of routine laboratory tests performed on samples from the borings. The analyses considered a 12-ft tall embankment. The embankment fill was assumed to be silty or sandy clay (CL) with a unit weight of 120 lbs per cu ft. Compressible clay (CL, CH) foundation soils extended from the ground surface to a depth of about 12 feet below the proposed embankment fill. For the compressible foundation soils, we considered an initial void ratio, e_0, of 0.66; a compression index, C_c, of 0.28; a recompression index, C_r, of 0.03; and pre-consolidation pressure, P_c, of 7,600 psf based on the consolidation test results. The ground water table was assumed to be 1 foot below the existing ground surface. The calculated settlement resulting from consolidation of the relatively weak soils is about 2 in.

Based on typical values of compression of fill presented in NAVFAC DM 7.2, we assumed that 1 percent strain would occur in the compacted embankment fill; therefore, the calculated settlement of the embankment fill itself is about 1.5 in.

Based on these analyses, the total settlement for the approach embankments is expected to be on the order of 3 to 4 in. Approximately 50 percent of the settlement is expected to occur during bridge construction. No settlement problems due to consolidation settlement are anticipated at this site, and no special mitigation will be required.

5.5 Deep Foundations

We understand that driven 18-in. and 24-in. diameter, closed-ended steel pipe piles are proposed for the abutment bents and interior bents, respectively. Analyses were performed to evaluate the abutment bents and interior bents pile capacities based on the guidance provided by ARDOT and the recommendations discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

5.5.1 Axial Pile Capacity. Axial pile capacity curves were computed based on the pile type shown on the provided plans and the subsurface soil conditions encountered in the borings. Scour was not considered in our analyses. If significant scour is anticipated, we should be contacted to provide revised capacity curves.

The pile capacities were estimated based on the FHWA design procedure using the ENSOFT computer program APile v2015. The compression capacity of an individual pile consists of a combination of skin friction around the perimeter of the pile shaft and end bearing at the tip. The skin friction in the upper 5 ft of soil was neglected. Separate calculations were performed to determine pile capacities with and without consideration of seismic effects. For the calculations that consider seismic effects, the pile skin friction was reduced by 90% for liquefiable soil layers between the ground surface and a depth of 50 ft and the pile skin friction was reduced by 50% for liquefiable soil layers below a depth of 50 ft.

The pile capacity curves are presented in Figures 34, 35, and 36, for the west abutment, east abutment, and interior bents, respectively. The pile capacity curves are presented as nominal (ultimate) values that do not include a resistance factor. An appropriate resistance factor should be applied to the nominal values presented on the pile capacity curves. Guidance on resistance factors is provided in Section 6.2. We recommend that the piles extend at least 10 feet into Zone 3 (see Figure 7 profile) to ensure that the piles are tipped below the deepest soil layer with a high likelihood of liquefaction (i.e., Zone 2C).

5.5.2 Downdrag. The seismic compression of the liquefiable soil layers can result in drag loads and increased pile settlement. Pile drag loads occur when the soils surrounding a pile settle more than the pile and apply negative skin friction to the pile. These drag loads increase the compressive loads in the pile that should be considered as part of the pile structural design. Structural capacity determination of the piles is not in our scope for this investigation.

The depth at which the pile and the soils settle the same amount is referred to as the neutral plane. Below the neutral plane, the pile settles more than the surrounding soils. The depth of the neutral plane depends on the soil settlement profile, the pile length, the distribution of pile skin friction and end bearing, and the load applied to the top of the pile. The soil settlement profiles were based on the distributions of seismic compression. The distributions of pile skin friction and end bearing were based on the axial pile capacity curves that consider reduced skin friction in the liquefiable soil layers. We used unfactored dead loads provided by Neel Schaffer, Inc. as the loads applied to the tops of the piles. For the interior bent piles, we added the self-weight of the pile stick-up (between the ground surface and the bottom of the pile cap) to the unfactored deadloads.

The downdrag analysis results are summarized in the following tables. Table 3 and Table 4 present the results for the west abutment bent for loads of 65 kips and 80 kips, respectively. Table 5 and Table 6 present the results for the east abutment bent for loads of 65 kips and 80 kips, respectively. Table 7 presents the results for the interior bents for a load of 87 kips. For each case, results are provided for a range of possible pile lengths.

Table 3 - Downdrag Analysis Results for West Abutment with Load of 65 kips

	Pile Length (ft) below El 247 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	264	306	344	344	344
Top of Pile Settlement (in.)	2.5	0.9	0.1	0.1	0.1
Neutral Plane Depth (ft)	82.0	85.9	88.0	88.0	88.0

Table 4 - Downdrag Analysis Results for West Abutment with Load of 80 kips

	Pile Length (ft) below El 247 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	256	294	344	344	344
Top of Pile Settlement (in.)	2.9	1.1	0.1	0.1	0.1
Neutral Plane Depth (ft)	81.0	85.4	88.0	88.0	88.0

Table 5 - Downdrag Analysis Results for East Abutment with Load of 65 kips

	Pile Length (ft) below El 248 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	227	273	367	377	377
Top of Pile Settlement (in.)	4.9	3.1	0.2	0.1	0.1
Neutral Plane Depth (ft)	77.8	83.8	93.8	94.0	94.0

Table 6 - Downdrag Analysis Results for East Abutment with Load of 80 kips

	Pile Length (ft) below El 248 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	219	265	357	377	377
Top of Pile Settlement (in.)	5.0	3.5	0.5	0.1	0.1
Neutral Plane Depth (ft)	76.7	82.8	93.3	94.0	94.0

Table 7 - Downdrag Analysis Results for Interior Bents with Load of 87 kips

	Pile Length (ft) below El 235 ft				
	85	90	100	110	120
Maximum Drag Load (kips)	295	347	362	362	362
Top of Pile Settlement (in.)	1.7	0.2	0.1	0.1	0.1
Neutral Plane Depth (ft)	77.9	80.7	81.0	81.0	81.0

5.5.3 Lateral Analysis Parameters. If lateral loads applied to the piles are substantial, a lateral load analysis should be performed. The piles should be designed so that angular rotation and deflection at the tops of the piles are maintained within structurally tolerable limits. We recommend that the response of the piles to applied moment and lateral loading be analyzed utilizing the method developed by Dr. Lymon C. Reese of the University of Texas or a similar analysis procedure. Computer programs (e.g., LPILE) are available for this method of analysis. The analysis method utilizes finite difference approximations to solve for deflection, moment, soil modulus and soil reaction for a single pile. Soil response to the laterally loaded pile is represented in the analysis by a set of nonlinear “p-y" curves that are developed for various depths along the pile and for the different soil types. The "p-y" curves essentially indicate the soil reaction in force per unit length of pile versus deflection for a given pile diameter. A tabulation of recommended soil parameters that can be used in the lateral pile analysis are presented in Table 8. The LPILE default values of E_{50} and k, which are correlated based on the cohesion and friction angle, can be used in the lateral pile analysis.

Table 8 - Recommended Soil Parameters for Lateral Pile Analysis

Soil Zone	p-y Curve Type	Effective Unit Weight (pcf)	Cohesion (psf)	Internal Friction Angle (degrees)
New Fill	Stiff Clay w/o Free Water (Reese)	57.6	1500	-
1	Soft Clay (Matlock)	59.6	750	-
2A, 2B, 2C	Sand (Reese)	57.6	-	34
3	Sand (Reese)	57.6	-	35

Liquefaction of sands and cyclic softening of clay soils can result in significant short-term strength losses that can reduce lateral pile capacity. Accordingly, Table 9 provides a separate set of soil parameters that should be used instead of the values in Table 8 in the lateral pile analysis for seismic conditions.

Table 9 - Recommended Post-Earthquake Soil Parameters for Lateral Pile Analysis

Soil Zone	p-y Curve Type	Effective Unit Weight (pcf)	Cohesion (psf)	Internal Friction Angle (degrees)
New Fill	Stiff Clay w/o Free Water (Reese)	57.6	1200	-
1	Soft Clay (Matlock)	59.6	600	-
2A	Soft Clay (Matlock)	57.6	180	-
2B	Sand (Reese)	57.6	-	34
2C	Soft Clay (Matlock)	57.6	400	-
3	Sand (Reese)	57.6	-	35

5.5.4 Drivability Analysis. A "drivability" type wave equation analysis relating blow counts to pile penetration, ultimate static pile capacities, dynamic pile driving stresses, minimum recommended hammer energy and hammer strokes was performed using the program GRLWEAP v.2010. The unit skin friction and end-bearing values in each soil layer were developed based on the results of unconsolidated undrained (UU) triaxial compression tests, supplemented by the results of the field standard penetration tests and visual estimates of consistency and the static analysis program in GRLWEAP. A 72% pile hammer efficiency and a shaft gain/loss factor of 0.833 and a toe gain/loss factor of 1.0 were used in the analysis. A maximum driving stress of 90% of the steel yield strength was considered for these analyses.

Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. An open-ended D36 diesel hammer was utilized for the drivability analyses of both pile sizes. Hammer and pile cushion information was based on manufacturer-recommended values. Both the 18-in. and 24-in. diameter steel pipe piles were assumed to be installed close-ended. In the analyses, the piles at the abutments and interior bents are assumed to be driven from the plan pile cap bottom elevations to the recommended tip elevations. Graphical and tabulated results of the drivability analyses are provided in Appendix C. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed prior to hammer acceptance and beginning of driving. The resulting minimum hammer energy to drive the piles at the abutment and interior bents is provided in Table 10.

Table 10 - Results of Drivability Analyses

Location	Hammer Type	Minimum Hammer Energy (kip-ft.)
Abutment Bents	D36	80
Interior Bents	D36	80

The parameters used in the wave equation analysis were based on general information available at the time of the analysis; however, actual field conditions may be different. We recommend prudent use of the wave equation analysis results. Soil response, hammer performance, and pile stresses and drivability should be verified by dynamic measurements using the Pile Driving Analyzer (PDA) on site and subsequent data analysis with the CAPWAP program. The actual suitability and final acceptance of a hammer system for a given project can only be determined after demonstration of satisfactory field performance, which is typically evaluated during the Test Pile Driving Program with PDA dynamic pile measurements and related data analyses.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Pile Design and Installation

Driving refusal for the steel pipe piles may occur in the dense sands encountered in Zone 3 (see Figure 7 profile). If refusal occurs at depths shallower than the required minimum depth, then jetting will be required to achieve additional penetration. However, the final 5 ft of pile penetration must be achieved by driving. Driven piles should be installed in accordance with AHTD Standard Specification Section 805 PILING.

The pile capacity curves presented in this report do not reflect the effects of jetting. As described in FHWA-NHI-16-009, Design and Construction of Driven Pile Foundations, the use of jetting will result in greater soil disturbance than considered in standard static pile capacity calculations. Some field studies have reported that the pile side resistance may be reduced by about 50 percent over the jetted depth. If jetting is necessary, we should be contracted to provide revised

axial capacities. Dynamic load testing should be performed during construction to more accurately determine the ultimate capacity of the piles after jetting.

6.2 Test Piles, Dynamic Load Testing, and Resistance Factors

Based on Table 10.5.5.2.3-1 of the AASHTO LRFD 2017 Bridge Design Specifications and considering that the soil profiles consist predominantly of sand, a resistance factor of 0.45 should generally be applied for axial compression and a resistance factor of 0.35 should generally be applied for tension. A higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 11.

Table 11 - Pile Resistance Factors based on Condition/Resistance Determination Method

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 condition, but no less than 2% of the 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 the production piles.	0.65
	Wave equation analysis, without pile dynamic measurements or load test by with field confirmation of hammer performance.	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only).	0.40

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

As discussed in Section 10.5.5.3.3 of the Bridge Design Specifications, a resistance factor of 1.0 should be applied for axial compression and a resistance factor of 0.80 should be applied for tension when designing the foundations to resist earthquake loading.

We recommend a minimum of two test piles (one at an abutment bent and one at an interior bent) be driven to evaluate pile capacities and drivability, prior to ordering the production piles. The test pile lengths should be selected considering the estimated pile capacities, minimum penetration requirements, and the anticipated driving resistance. The test piles can be driven at permanent pile locations.

We recommend that dynamic pile load testing be performed on the test piles in accordance with ASTM D 4945. The results of the dynamic pile load test should be used to establish driving criteria for the production piles. The embedment length of the piles may be increased based on the PDA evaluation. All testing should be performed prior to ordering production piles in case the design lengths change due to the testing.

The dynamic pile load testing data collection should be performed by an engineer with a minimum of one year of dynamic pile testing field 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/or Foundation QA. Pile driving modeling and analysis of PDA data should be performed by an engineer with a minimum of five years of 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/or Foundation QA.

6.3 Embankment Construction

Embankment construction shall conform with Section 210 and all other applicable requirements of the latest AHTD Standard Specification for Highway Construction. The fill material for embankment construction should classify as AASHTO A-6, A-5, or A-4 with a liquid limit less than 45 and a plasticity index less than or equal to 25. The fill materials should be compacted to not less than 95 percent of standard Proctor maximum dry density (AASHTO T99) at moisture contents within 3 percentage points of the optimum moisture content. Fill material with a plasticity index less than 10 or that is susceptible to erosion shall have a minimum 18-inch clay plating (measured perpendicular to the finished slopes). Clay plating shall consist of material having a plasticity index in the range of 10 to 25 that supports vegetation and that is not highly susceptible to erosion.

As an initial site preparation step, existing utilities or pipes and any other subsurface obstructions that might interfere with earthwork, bridge, and/or drainage ditch construction should

be removed and/or relocated. Stripping should then be performed within the construction areas to remove organic-laden surficial soils, vegetation, debris, brush or roots. Temporary excavation slopes should not be steeper than 1H:1V. We recommend that excavations be left open for the shortest possible duration to minimize exposure of the bearing soils to rainfall. Drainage should be maintained away from the excavations during construction.

Prior to placement of any fill materials, the soils exposed after excavation should be inspected. Any obviously weak soils should be excavated and replaced with properly compacted backfill. The effort required to mitigate any unstable soils will be influenced by the season of the year when earthwork is performed. The soils may be drier during the hot late summer and could weaken during heavy rain events. We recommend that earthwork be performed during a dry summer or fall season, if the schedule permits. The vertical and lateral extent of excavation required to remove any weak soils must be determined in the field during earthwork construction. In order to minimize the amount of excavation, we recommend that a representative of Burns Cooley Dennis, Inc. be present to observe excavation operations and assist in evaluating the depth and lateral extent of any excavation required.

In areas where embankments are to be constructed over existing ditches, we understand that the work will conform with the requirements presented in the AHTD Special Provision for Embankment Construction, which is provided in Appendix D. This special provision requires that the ditches shall be undercut 2 feet to remove all highly organic, wet material and backfilled with Stone Backfill prior to embankment construction. The remaining embankment shall be constructed of Select Material (Class SM-2). Synthetic Filter Blanket and Dumped Riprap shall be placed on the slopes of embankments constructed of SM-2 from the top of the Stone Backfill to at least 2 feet above the high-water elevation. The remainder of embankments construction of SM-2 or other material that is susceptible to erosion shall have a minimum 18-inch clay plating (measured perpendicular to the finished slopes). Clay plating shall consist of material having a plasticity index in the range of 10 to 25 that supports vegetation and that is not highly susceptible to erosion.

As discussed in Section 210.09 of the AHTD Standard Specification, where fill materials are to be placed and compacted against a slope, the slope shall be continuously benched as the fill lifts are placed and compacted.

Laboratory classification tests, including grain size analyses and Atterberg limit determinations, should be performed on the backfill soils initially and routinely during earthwork

operations to check for compliance with the recommendations provided herein. Field moisture and density tests should be performed at frequencies that satisfy the requirements specified in Section 210.02 of the AHTD Standard Specification.

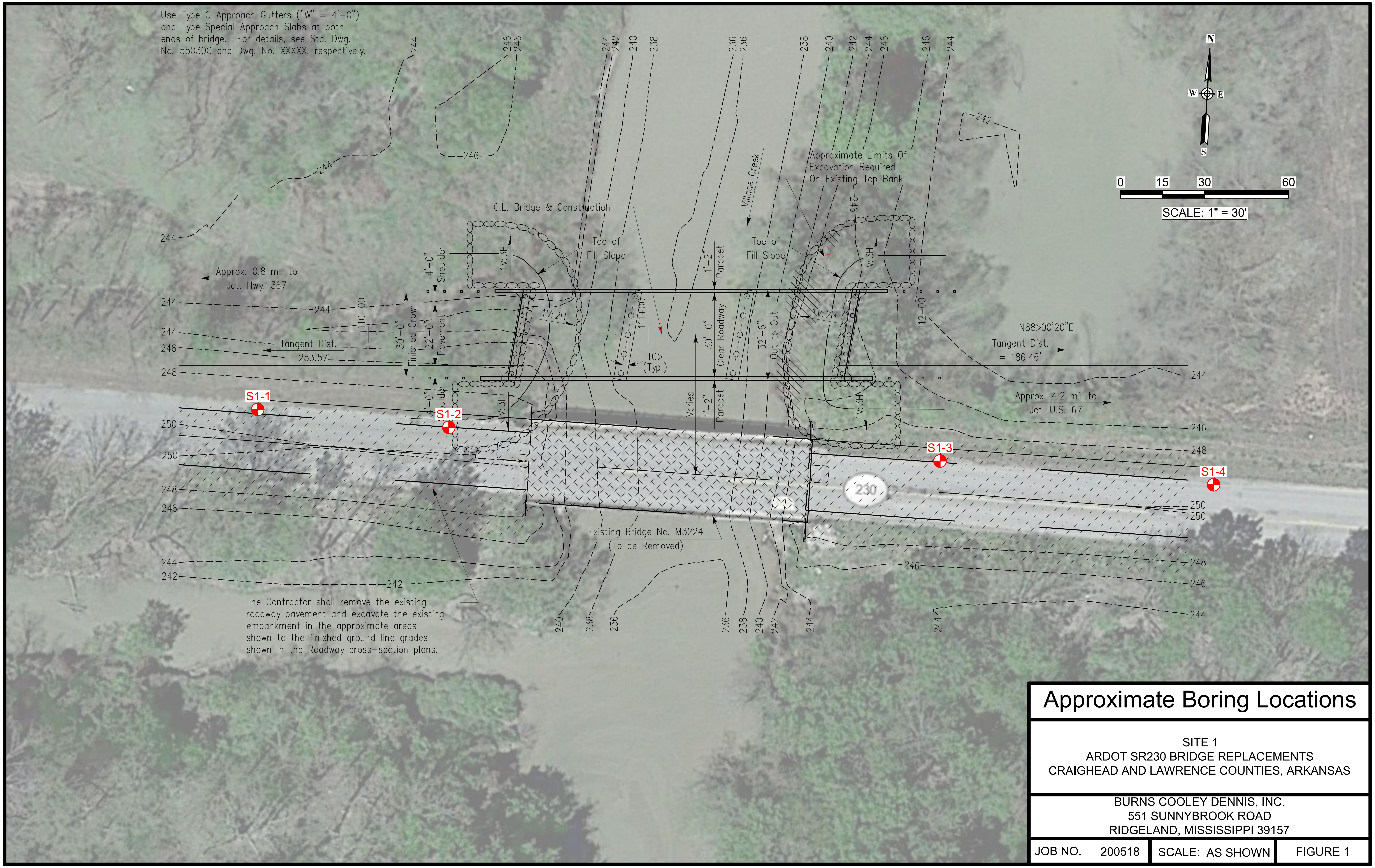
7.0 REPORT LIMITATIONS

The analyses, conclusions, and recommendations discussed in this report are based on conditions as they existed at the time of the exploration and further on the assumption that the exploratory borings are representative of subsurface conditions throughout the areas investigated. It should be noted that actual subsurface conditions between and beyond the borings might differ from those encountered at the boring locations. If subsurface conditions are encountered during construction that vary from those discussed in this report, Burns Cooley Dennis, Inc. should be notified immediately in order that we may evaluate the effects, if any, on earthwork and foundation design and construction.

Burns Cooley Dennis, Inc. should be retained for a general review of final design drawings and specifications. It is advised that we also be retained to observe earthwork for the project, to perform and observe the pile testing, and to develop the pile driving criteria. Our involvement during construction would give opportunity for us to help confirm that our recommendations are valid or to modify them accordingly. Burns Cooley Dennis, Inc. cannot assume responsibility or liability for the adequacy of recommendations if we do not observe construction.

This report has been prepared for the exclusive use of Neel-Schaffer, Inc. for specific application to the geotechnical-related aspects of design and construction of the ARDOT SR230 Bridge Replacements in Craighead and Lawrence Counties, Arkansas. The only warranty made by us in connection with the services provided is we have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing in the same or similar locality. No other warranty, express or implied, is made or intended.

FIGURES



Approximate Boring Locations

SITE 1
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO.	200518	SCALE: AS SHOWN	FIGURE 1

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL & LETTER		DESCRIPTION
COARSE-GRAINED SOILS More than half of material larger than No. 200 sieve size	GRAVELS More than half of coarse fraction larger than No.4 sieve size	Clean Gravels (Little or no fines)	⁘	GW	WELL GRADED GRAVEL, GRAVEL-SAND MIXTURE
			⁘	GP	POORLY GRADED GRAVEL, GRAVEL-SAND MIXTURE
		Gravels with fines (Appreciable amount of fines)	▥	GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURE
			▨	GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY MIXTURE
	SANDS More than half of coarse fraction smaller than No.4 sieve size	Clean Sands (Little or no fines)	⁙	SW	WELL GRADED SAND, GRAVELLY SAND
			⁙	SP	POORLY GRADED SAND, GRAVELLY SAND
		Sands with fines (Appreciable amount of fines)	▥	SM	SILTY SAND, SAND-SILT MIXTURE
			▨	SC	CLAYEY SAND, SAND-CLAY MIXTURE
FINE-GRAINED SOILS More than half of material smaller than No. 200 sieve size	SILTS AND CLAYS	Liquid limit less than 50	▥	ML	SILT WITH LITTLE OR NO PLASTICITY
			▨	ML	CLAYEY SILT, SILT WITH SLIGHT TO MEDIUM PLASTICITY
			▥	ML	SANDY SILT
			▨	CL	SILTY CLAY, LOW TO MEDIUM PLASTICITY
			▨	CL	SANDY CLAY, LOW TO MEDIUM PLASTICITY (30% TO 50% SAND)
	SILTS AND CLAYS	Liquid limit greater than 50	▥	MH	SILT, HIGH PLASTICITY
			▨	CH	CLAY, HIGH PLASTICITY
			▨	OH	ORGANIC CLAY OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			▦	PT	PEAT, HUMUS, SWAMP SOIL

TERMS CHARACTERIZING SOIL STRUCTURE

Slickensided - Clays with polished and striated planes created as a result of volume changes related to shrinking, swelling and/orchanges in overburden pressure.

Fissured - Clays with a blocky or jointed structure generally created by seasonal shrinking and swelling.

Laminated - Composed of thin alternating layers of varying color and texture.

Calcareous - Containing appreciable quantities of calcium carbonate.

Parting - Paper thin (less than 1/8 inch).

Seam - 1/8 inch to 3 inch thickness.

Layer - Greater than 3 inches in thickness.

TERMS CHARACTERIZING SOIL STRUCTURE

COARSE-GRAINED SOILS		FINE-GRAINED SOILS		
	PENETRATION RESISTANCE, N		COHESION	PENETRATION RESISTANCE, N
DENSITY	Blows per Foot	Consistancy	Kips/Sq.Ft	Blows per Foot
Very loose	0 - 4	Very Soft	<0.25	0 - 1
Loose	5 - 10	Soft	0.25 - 0.50	2 - 4
Medium Dense	11 - 30	Medium Stiff	0.50 - 1.00	5 - 8
Dense	31 - 50	Stiff	1.00 - 2.00	9 - 15
Very Dense	>4.00	Very Stiff	2.00 - 4.00	16 - 30
		Hard	>4.00	>30

PARTICLE SIZE IDENTIFICATION

Cobbles - Greater than 3 inches

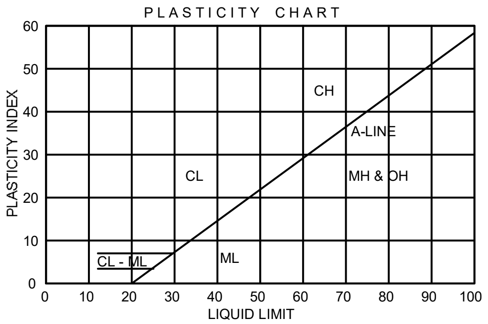
Gravel - Coarse-3/4 inch to 3 inches
Fine-4.76 mm to 3/4 inch

Sand - Coarse-2 mm to 4.76 mm
Medium-0.42 mm to 2 mm
Fine-0.074 mm to 0.42 mm

Silt & Clay - Less than 0.074 mm

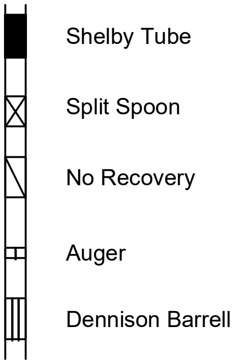
RELATIVE COMPOSITION

Slightly	5 - 15%
With	16 - 29%
Sandy (or gravelly)	30 - 50%



FOR CLASSIFICATION OF FINE GRAINED SOILS

SAMPLE TYPES
(Shown in Sample Column)



CLASSIFICATION, SYMBOLS AND TERMS USED ON GRAPHICAL BORING LOGS

LOG OF BORING NO. S1-1

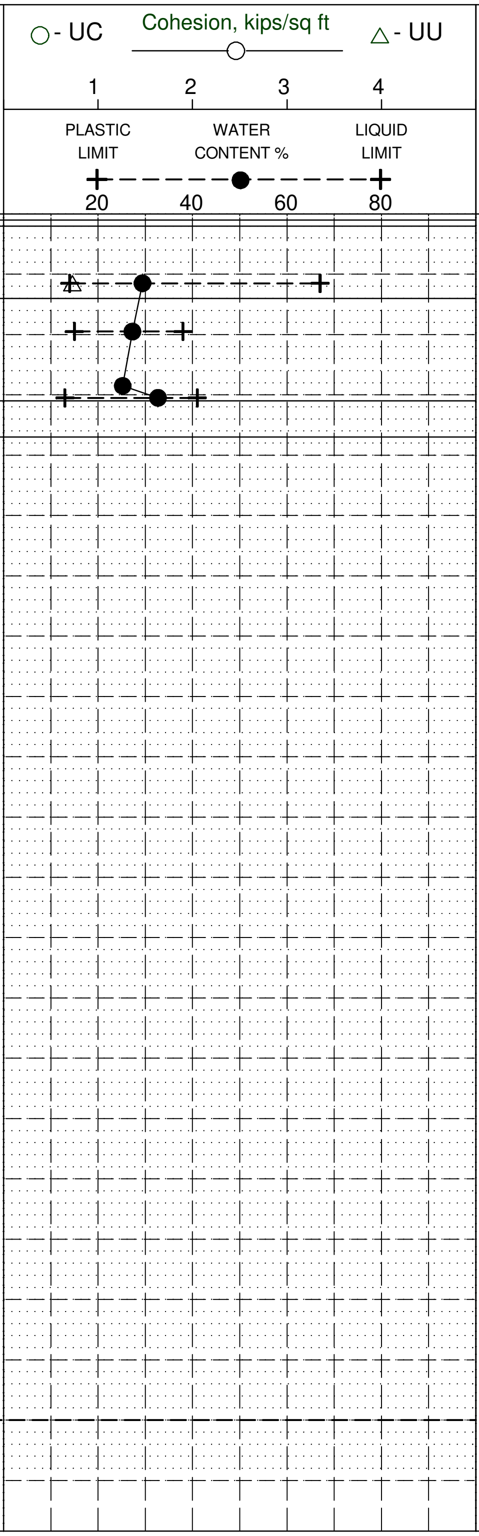
ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 55', then rotary wash to completion.

LOCATION: Sta. 109+50 (Approximate) +/- 27' Right of Construction C/L

SURFACE EL: 249 ±ft

DEPTH, ft	DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
	Asphalt Pavement (6")			
	Medium dense tan silty fine sand (SM) with gravel	6	95	
	Medium stiff tan clay (CH)			
10	Stiff gray silty clay (CL), slightly sandy			90.5
	- soft, gray and tan below 13'			
	Very stiff light gray clay (CH)			
20	Medium dense light gray fine sand (SP)			1.5
25		13		4.9
30	- loose 28' - 33'	9		
35		20		2.2
40		13		1.1
45		15		2.3
50		30		4.7
55		29		3.4
60		15		1.9
65	- with trace of gravel 63' - 93'	18		2.4
70		29		4.1
75	- loose 73' - 78'	10		2.5
80		14		2.6
85		12		4.8
90	- dense 88' - 93'	45		4.5
95		22		2.4
100	- with trace of gravel below 98'	19		2.7



BORING DEPTH: 100 ft

DATE: 08/20/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 53' 39.29" - W 91° 4' 22.72"

GROUNDWATER DATA: Free water encountered at an approximate depth of 44.5' during auger drilling. Water level remained at an approximate depth of 44.5' after about 15 minutes.

FIGURE 3

LOG OF BORING NO. S1-2

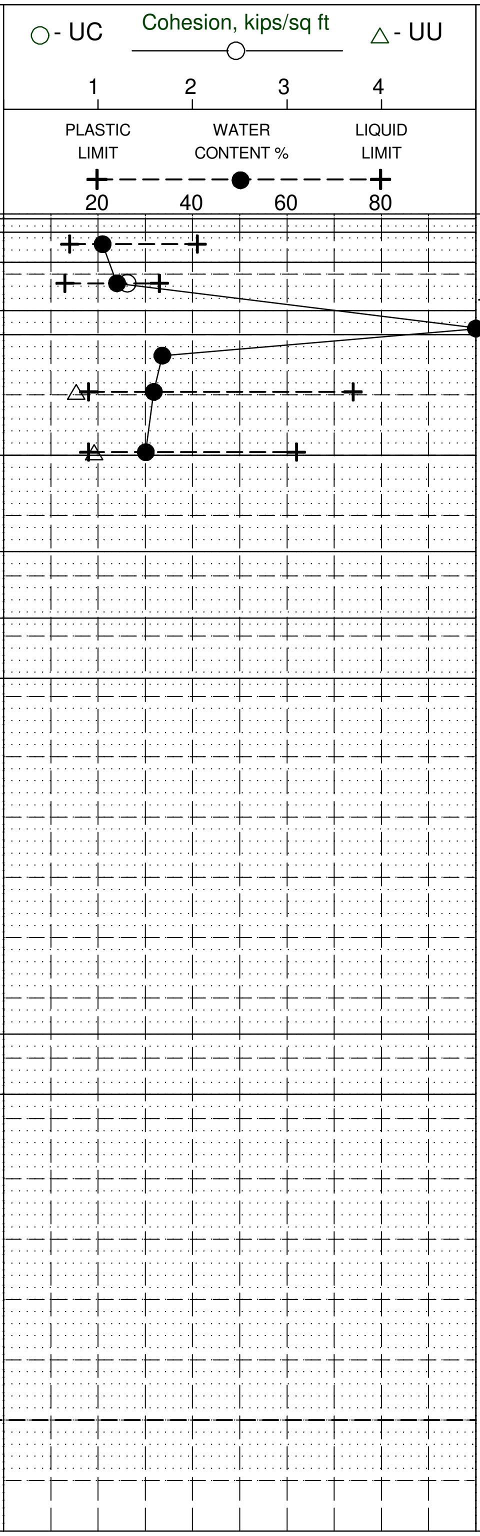
ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 45', then rotary wash to completion.

LOCATION: Sta. 110+29 (Approximate) +/- 33' Right of Construction C/L

SURFACE EL: 249 ±ft

DEPTH, ft	DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
	Asphalt Pavement (5")			
	Medium dense tan and light gray silty fine sand (SM) with gravel	5		68.4
5	Medium stiff tan and light gray sandy clay (CL) with trace of gravel		103	
	Stiff gray silty clay (CL) with gravel to 5'			100.87
10	Wood	11		
	Medium stiff light gray clay (CH), slightly sandy			86.8
15			93	
20			92	
	Loose light gray fine sand (SP)			
25		8		3.8
30	Very loose light gray silty fine sand (SM), slightly clayey with trace of organics	4		24.8
35	Medium dense light gray fine sand (SP-SM), slightly silty	26		5.8
40	Dense light gray and tan fine sand (SP)	32		2.3
45	- medium dense, with trace of organics 43' - 48'	18		2.2
50		37		3.9
55	- medium dense below 53' - with trace of organics 53' - 58' - fine to medium below 58'	16		2.3
60		27		3.2
65		19		3.7
70	Dense light gray and tan fine sand (SP-SM), slightly silty	41		5.3
75	Loose light gray and tan fine to coarse sand (SP) - with gravel to 83' - medium dense 78' - 88' - fine below 78'	9		2.5
80		13		2.5
85		14		3.3
90	- dense 88' - 93'	49		3.4
95	- medium dense 93' - 98'	11		2.5
100	- dense, with trace of gravel below 98'	35		3.5
105				



BORING DEPTH: 100 ft

DATE: 08/24/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 53' 39.23" - W 91° 4' 22.01"

GROUNDWATER DATA: Free water encountered at an approximate depth of 44' during auger drilling. Water level at an approximate depth of 43.8' after about 1 hour.

LOG OF BORING NO. S1-3

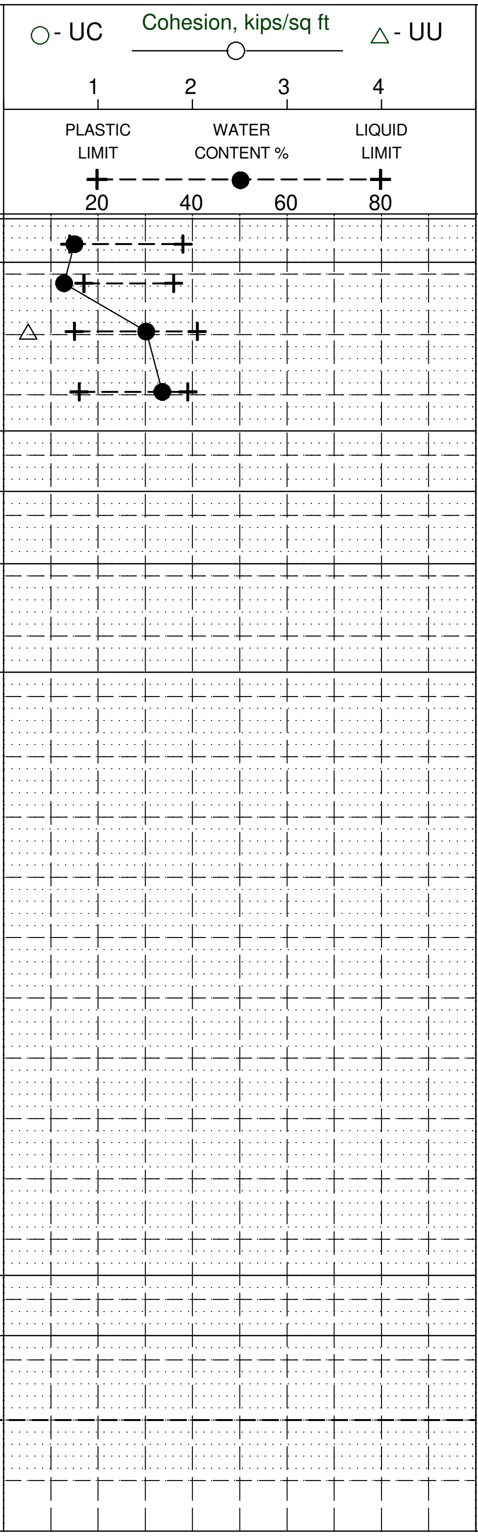
ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 30', then rotary wash to completion.

LOCATION: Sta. 112+04 (Approximate)
+/- 45' Right of Construction C/L

SURFACE EL: 249 ±ft

DEPTH, ft	DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
0	Asphalt Pavement (5")			
	Loose tan clayey fine to coarse sand (SC) with gravel	7		42.6
5	Stiff tan and light gray silty clay (CL) - with trace of organics to 8' - soft, with sand 8' - 13' - medium stiff below 13'			
10			96	81.4
15				
20	Loose light gray fine sand (SP-SM), slightly silty			5.9
25	Loose light gray silty fine sand (SM)	5		16.3
30	Medium dense light gray fine sand (SP-SM), slightly silty	19		6.4
35		25		7.4
40	Medium dense light gray fine to coarse sand (SP) - fine 43' - 63' - with trace of organics 48' - 53' - loose, fine to coarse 63' - 68' - dense, fine to medium 68' - 73' - fine 73' - 78' - fine to medium below 78'	25		2.6
45		13		3.5
50		16		4.6
55		19		3.1
60		15		3.1
65		8		1.8
70		48		4.0
75		21		4.2
80		15		4.5
85		23		4.4
90	Loose light gray fine to coarse sand (SP-SM), slightly silty, with trace of gravel	10		5.7
95	Medium dense light gray fine to coarse sand (SP) with trace of gravel	23		4.5
100		27		2.8
105				



BORING DEPTH: 100 ft

DATE: 08/26/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 53' 39.14" - W 91° 4' 20.01"

GROUNDWATER DATA: Free water encountered at an approximate depth of 27' during auger drilling. Water level remained at an approximate depth of 27' after about 15 minutes.

FIGURE 5

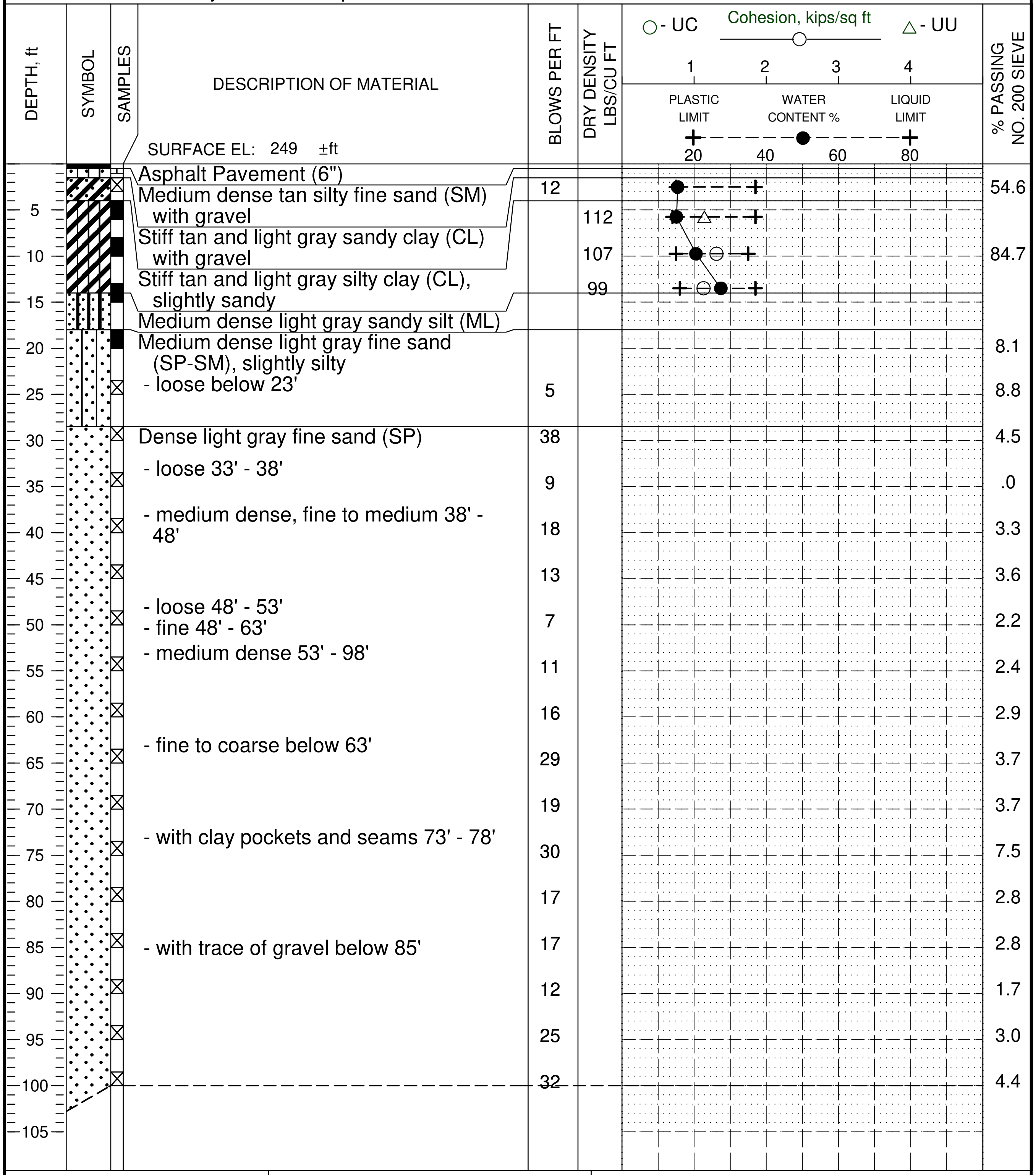
LOG OF BORING NO. S1-4

ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 50', then rotary wash to completion.

LOCATION: Sta. 113+01 (Approximate) +/- 54' Right of Construction C/L

SURFACE EL: 249 ±ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
Asphalt Pavement (6")			
Medium dense tan silty fine sand (SM) with gravel	12		54.6
Stiff tan and light gray sandy clay (CL) with gravel		112	
		107	84.7
Stiff tan and light gray silty clay (CL), slightly sandy		99	
Medium dense light gray sandy silt (ML)			
Medium dense light gray fine sand (SP-SM), slightly silty			8.1
- loose below 23'	5		8.8
Dense light gray fine sand (SP)	38		4.5
- loose 33' - 38'	9		.0
- medium dense, fine to medium 38' - 48'	18		3.3
	13		3.6
- loose 48' - 53' - fine 48' - 63'	7		2.2
- medium dense 53' - 98'	11		2.4
	16		2.9
- fine to coarse below 63'	29		3.7
	19		3.7
- with clay pockets and seams 73' - 78'	30		7.5
	17		2.8
- with trace of gravel below 85'	17		2.8
	12		1.7
	25		3.0
	32		4.4

BORING DEPTH: 100 ft

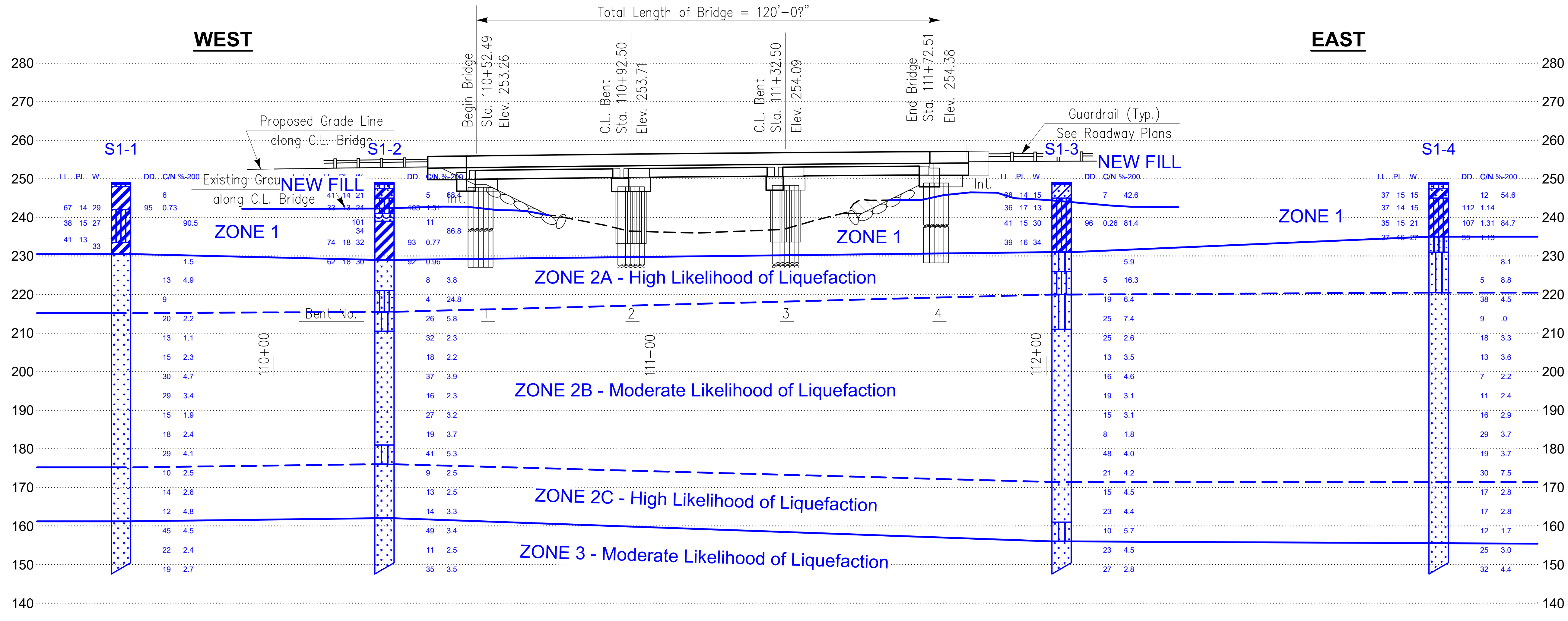
DATE: 08/27/20

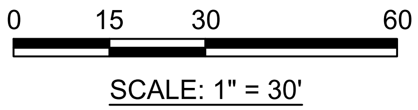
COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 53' 39.14" - W 91° 4' 18.79"

GROUNDWATER DATA: Free water encountered at an approximate depth of 48' during auger drilling. Water level remained at an approximate depth of 48' after about 30 minutes.

200518 1/27/2021 10:28:12 AM

FIGURE 6





ZONE 1

Soft to stiff silty clay (CL) and medium stiff to very stiff clay (CH)

ZONE 2

Loose to dense sand (SP) with trace of gravel and sand (SP-SM), slightly silty, and very loose to loose silty sand (SM)

ZONE 3

Medium dense to dense sand (SP) with trace of gravel

Note: The SPT blow count "N" values are raw values. They have not been corrected for hammer energy. A hammer energy correction factor of 1.36 applies to borings S1-1, S1-2, S1-3 & S1-4.

Soil Profile

SITE 1
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO.	200518	SCALE: AS SHOWN	FIGURE 7

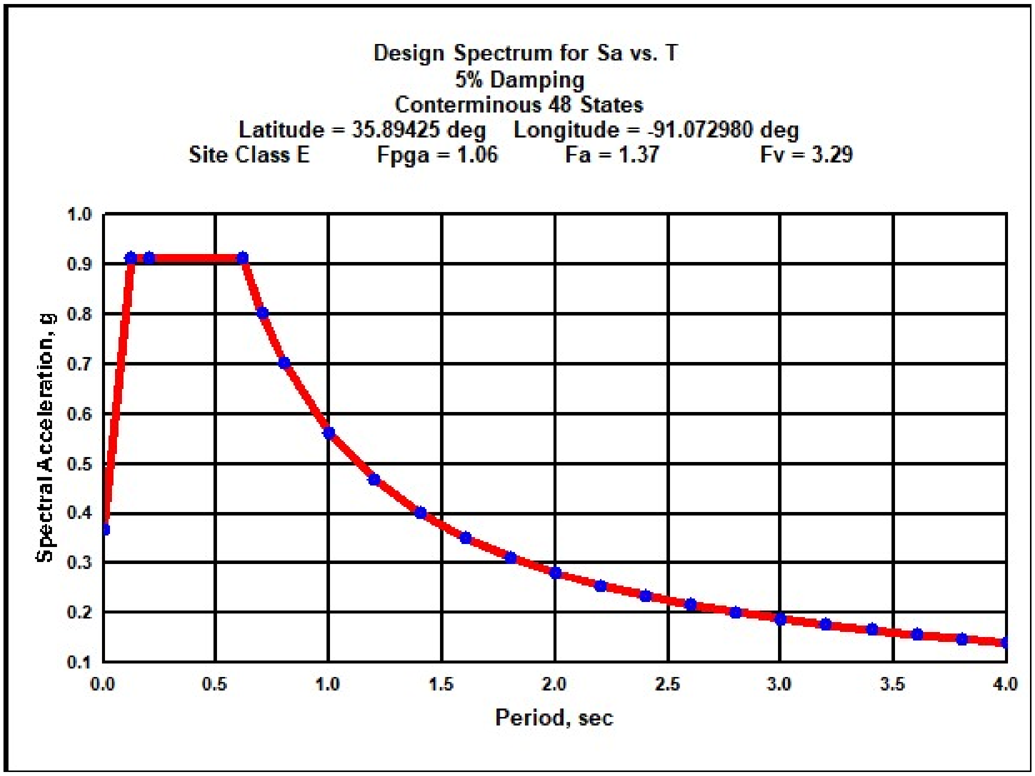


Figure 8 - Seismic Design Spectrum for Sa vs. T

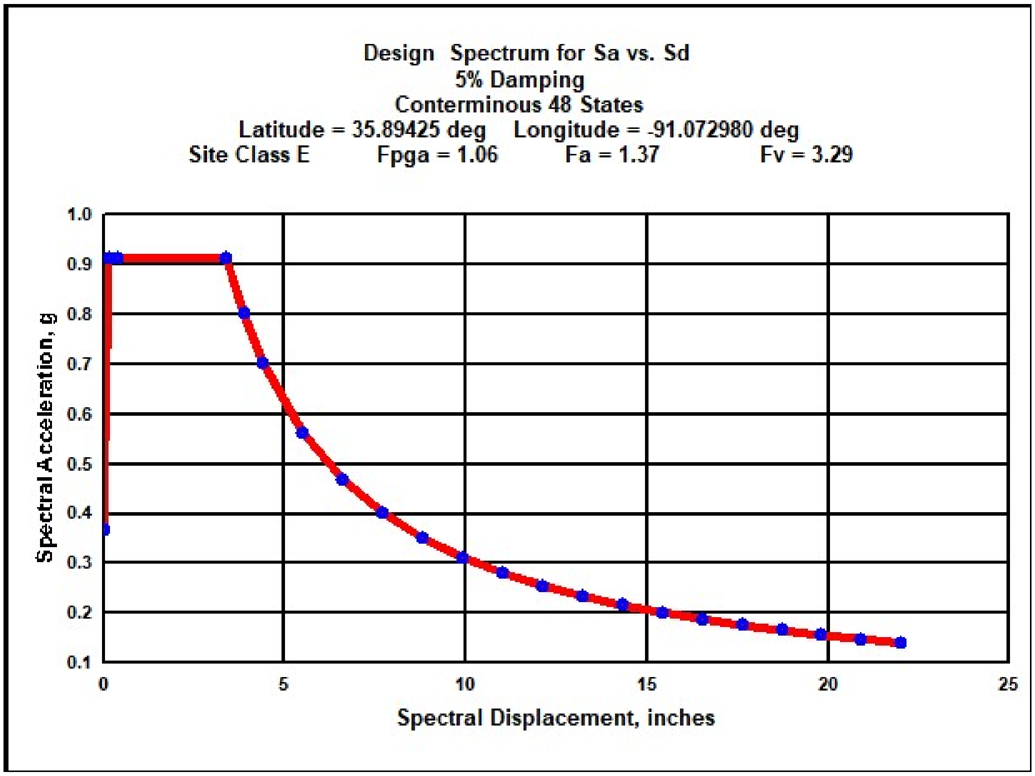


Figure 9 - Seismic Design Spectrum for Sa vs. Sd

Figure 10 - Liquefaction Triggering FS Values for S1-1 (Top of Boring at EL 249 ft)

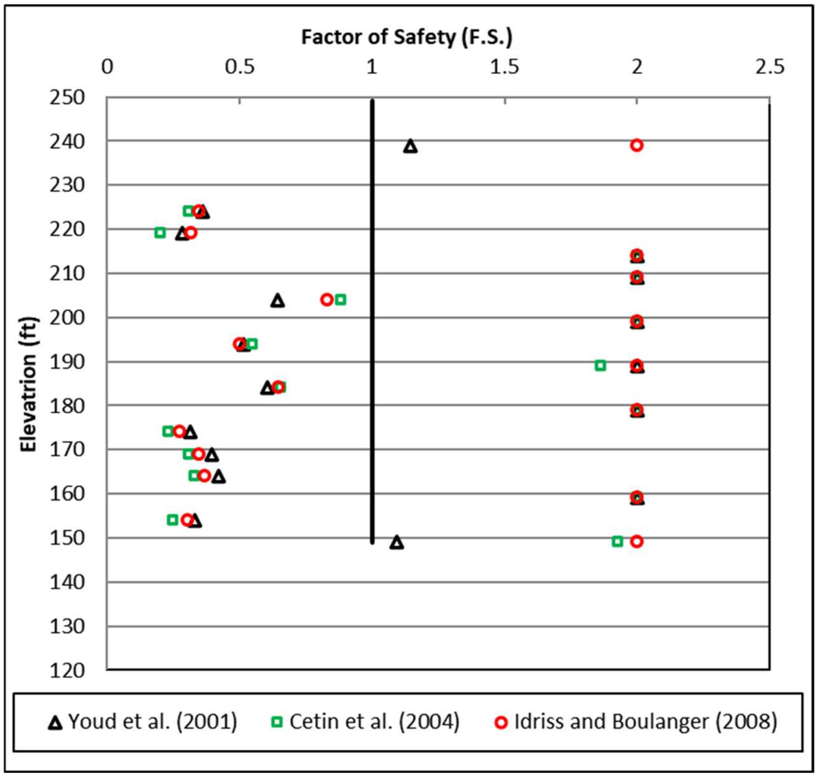


Figure 11 - Liquefaction Triggering FS Values for S1-2 (Top of Boring at EL 249 ft)

Figure 12 - Liquefaction Triggering FS Values for S1-3 (Top of Boring at EL 249 ft)

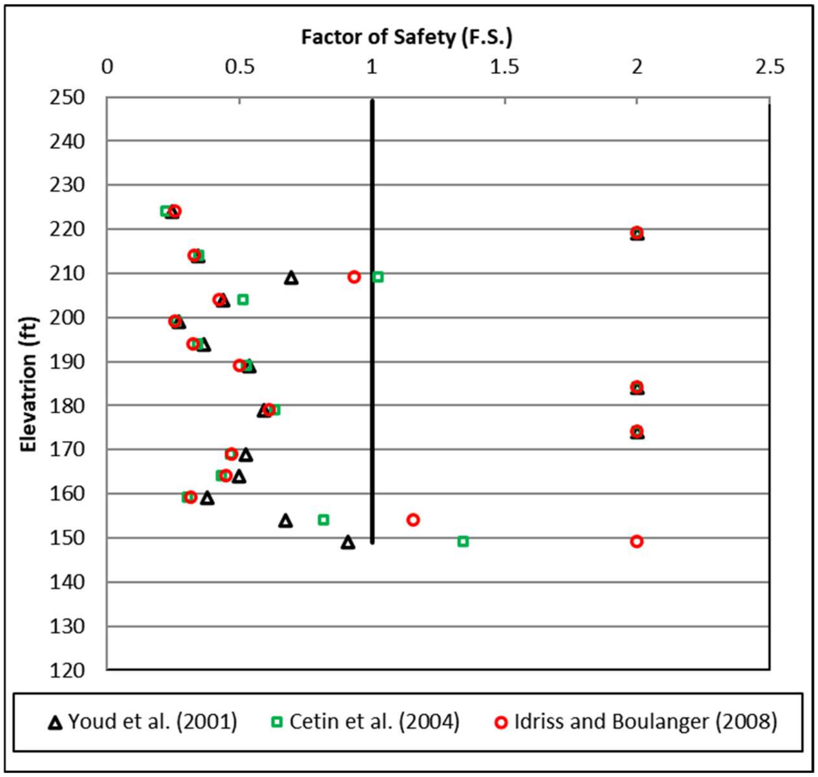


Figure 13 - Liquefaction Triggering FS Values for S1-4 (Top of Boring at EL 249 ft)

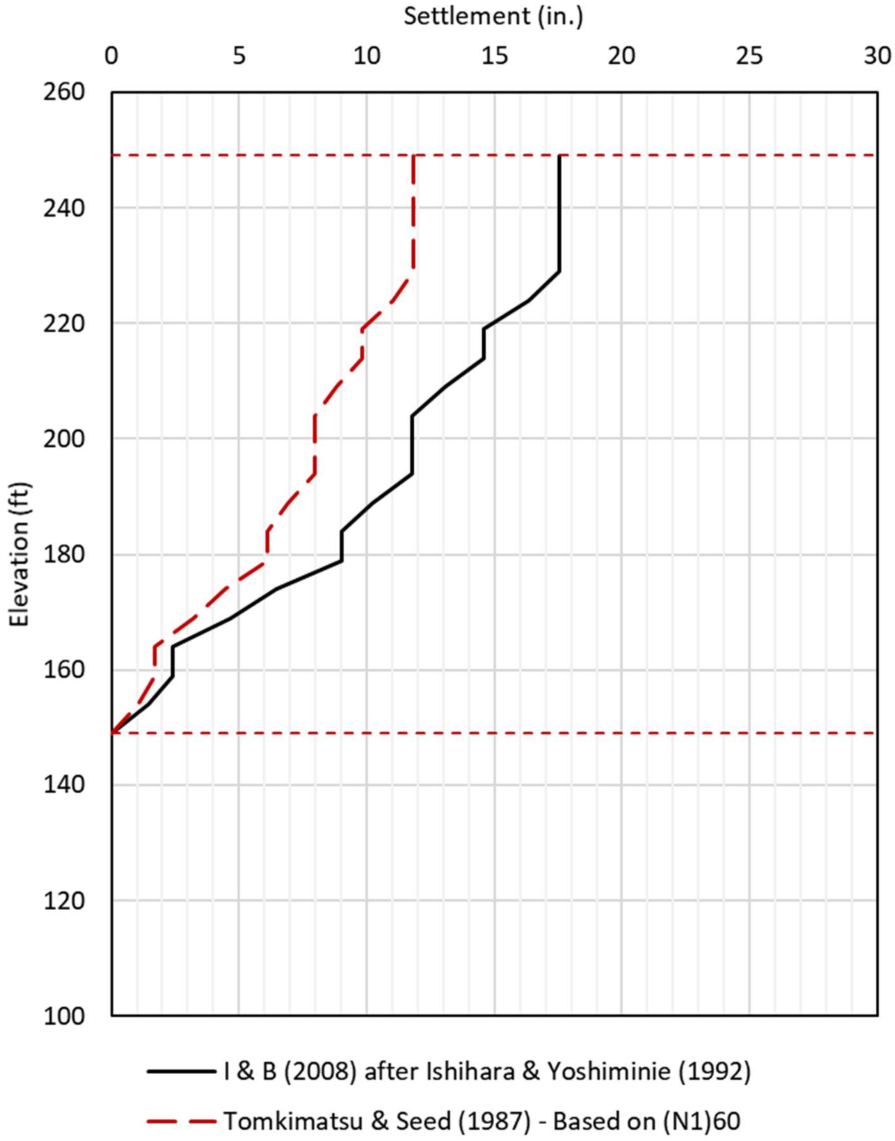


Figure 14 – Seismic Compression for S1-1 (Top of Boring at EL 249 ft)

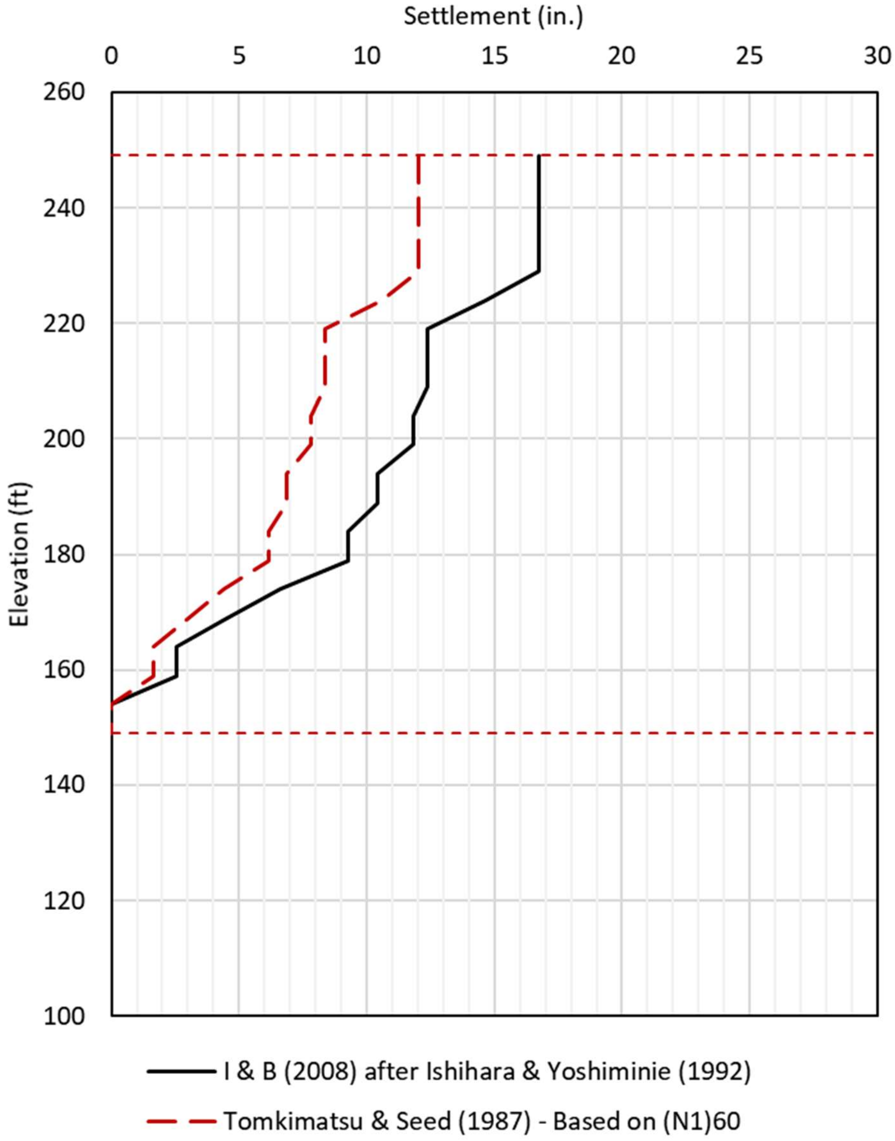


Figure 15 - Seismic Compression for S1-2 (Top of Boring at EL 249 ft)

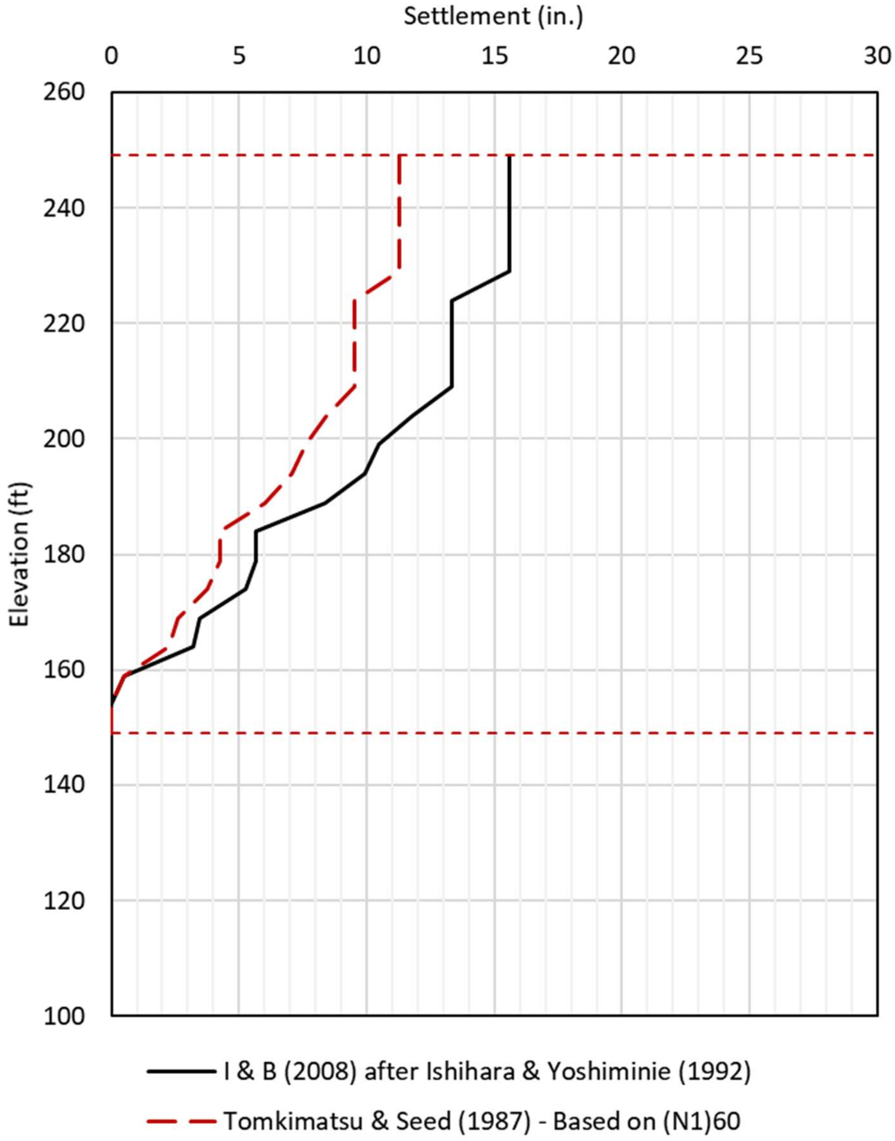


Figure 16 - Seismic Compression for S1-3 (Top of Boring at EL 249 ft)

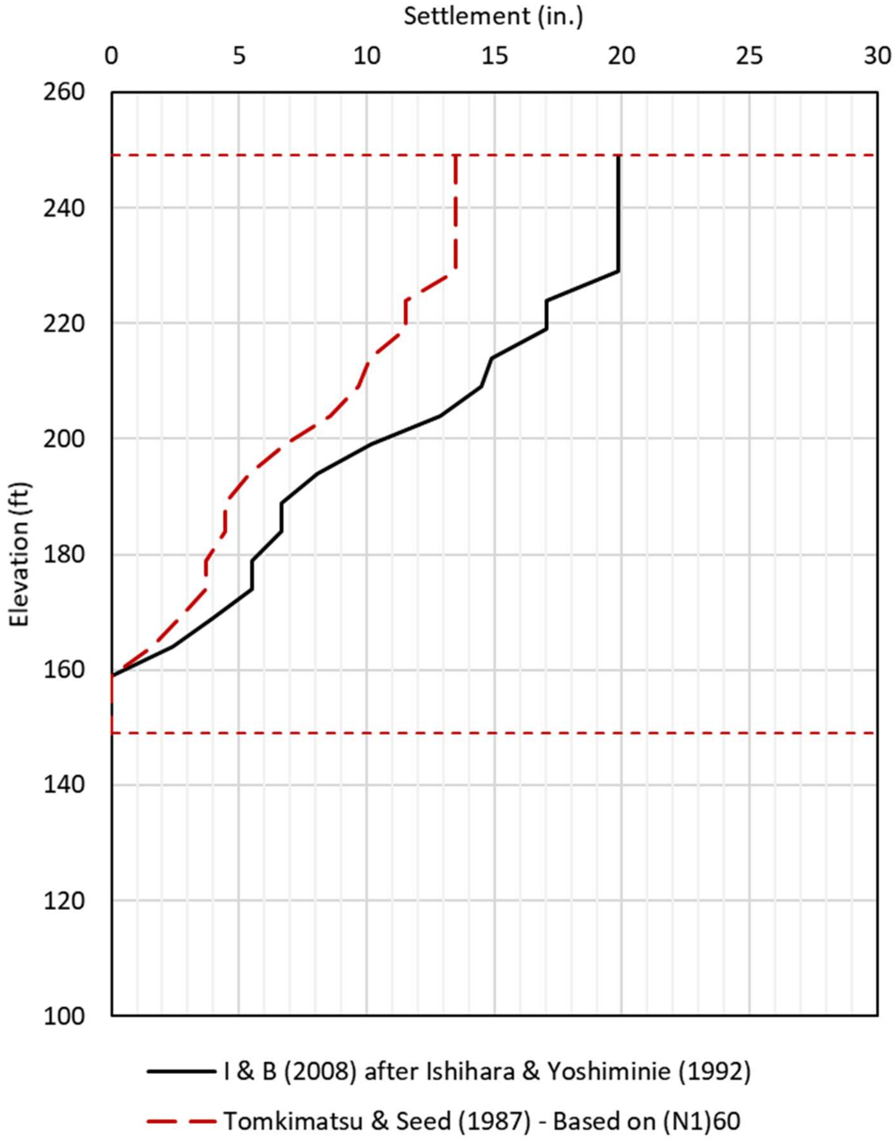
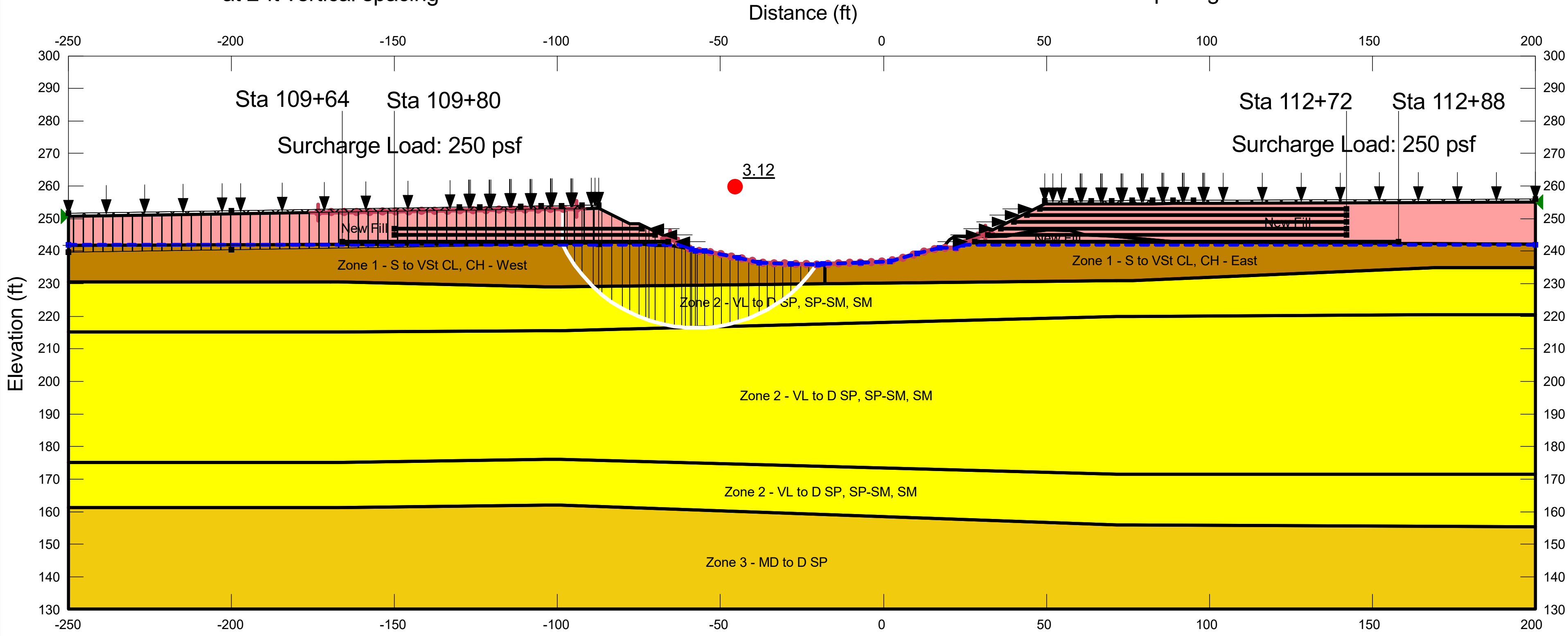


Figure 17 - Seismic Compression for S1-4 (Top of Boring at EL 249 ft)

West Abutment
3 layers of geosynthetic reinforcement
with tensile strength = 5,200 lbf
at 2-ft vertical spacing

East Abutment
6 layers of geosynthetic reinforcement
with tensile strength = 5,200 lbf
at 2-ft vertical spacing



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - S to VSt CL, CH - East	Mohr-Coulomb	122	500	0
🟫	Zone 1 - S to VSt CL, CH - West	Mohr-Coulomb	122	750	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟨	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

West Abutment Spill-Through
End of Construction

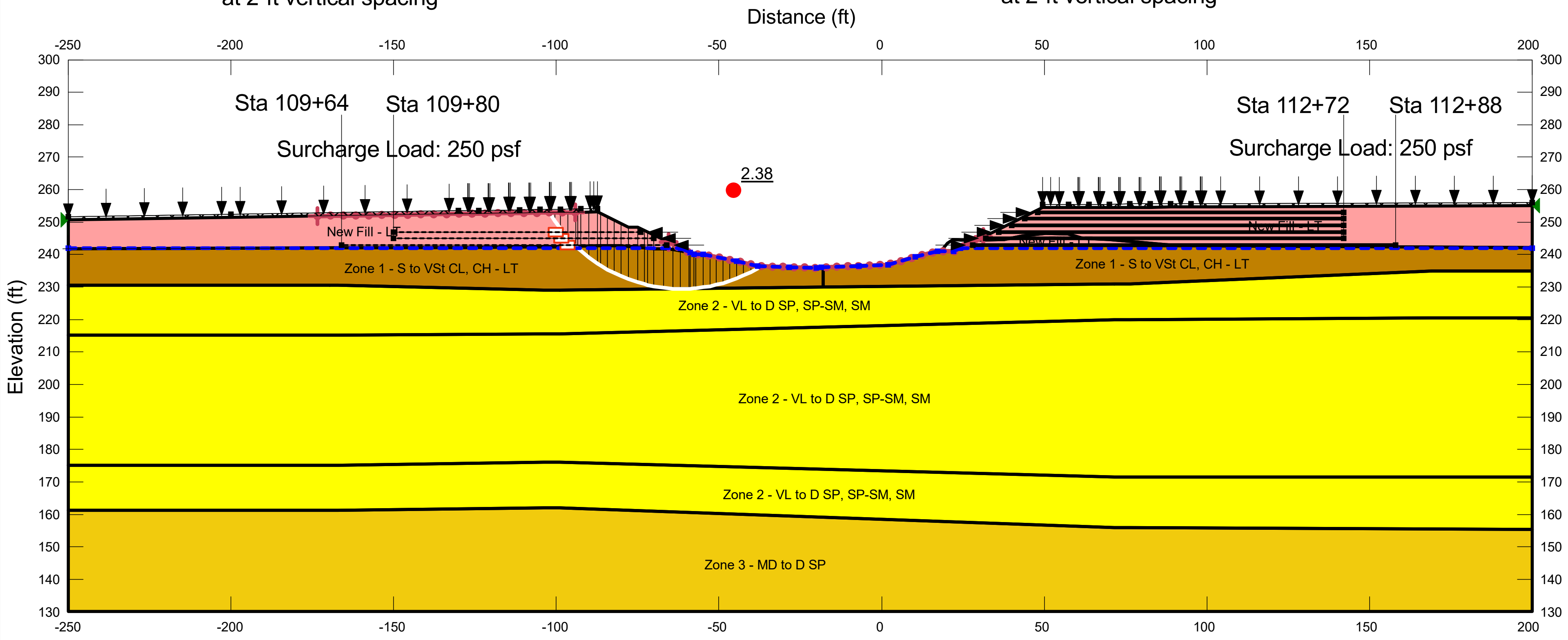
SR 230 Site 1
Craighead County, AR

Figure 18

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

West Abutment
3 layers of geosynthetic reinforcement
with tensile strength = 5,200 lbf
at 2-ft vertical spacing

East Abutment
6 layers of geosynthetic reinforcement
with tensile strength = 5,200 lbf
at 2-ft vertical spacing



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
🟫	Zone 1 - S to VSt CL, CH - LT	Mohr-Coulomb	122	50	21
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

West Abutment Spill-Through
Long Term

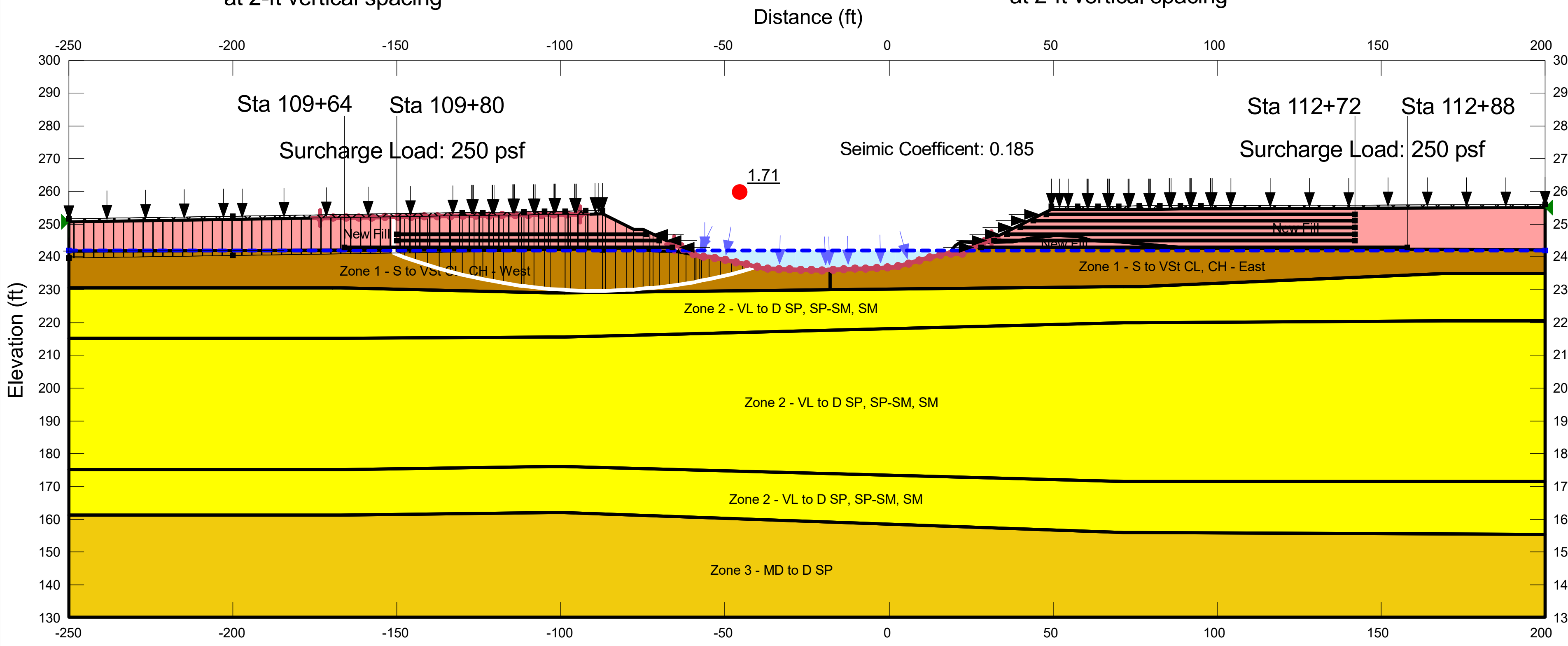
SR 230 Site 1
Craighead County, AR

Figure 19

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

West Abutment
3 layers of geosynthetic reinforcement
with tensile strength = 5,200 lbf
at 2-ft vertical spacing

East Abutment
6 layers of geosynthetic reinforcement
with tensile strength = 5,200 lbf
at 2-ft vertical spacing



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - S to VSt CL, CH - East	Mohr-Coulomb	122	500	0
🟫	Zone 1 - S to VSt CL, CH - West	Mohr-Coulomb	122	750	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟨	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

West Abutment Spill-Through
Pseudostatic

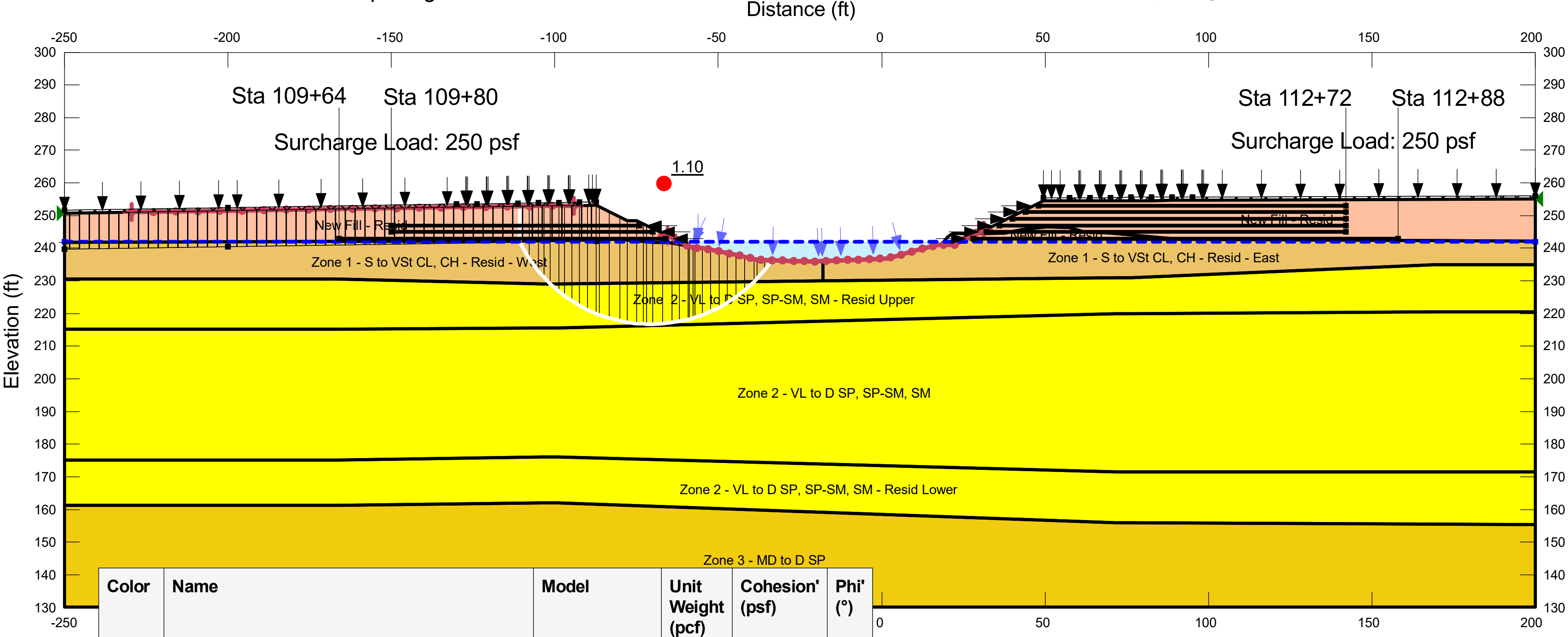
SR 230 Site 1
Craighead County, AR

Figure 20

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

West Abutment
3 layers of geosynthetic reinforcement
with tensile strength = 5,200 lbf
at 2-ft vertical spacing

East Abutment
6 layers of geosynthetic reinforcement
with tensile strength = 5,200 lbf
at 2-ft vertical spacing



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
🟨	Zone 2 - VL to D SP, SP-SM, SM - Resid Upper	Mohr-Coulomb	120	180	0
🟫	Zone 1 - S to VSt CL, CH - Resid - East	Mohr-Coulomb	122	400	0
🟫	Zone 1 - S to VSt CL, CH - Resid - West	Mohr-Coulomb	122	600	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟨	Zone 2 - VL to D SP, SP-SM, SM - Resid Lower	Mohr-Coulomb	120	400	0
🟨	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

West Abutment Spill-Through
Post-Seismic

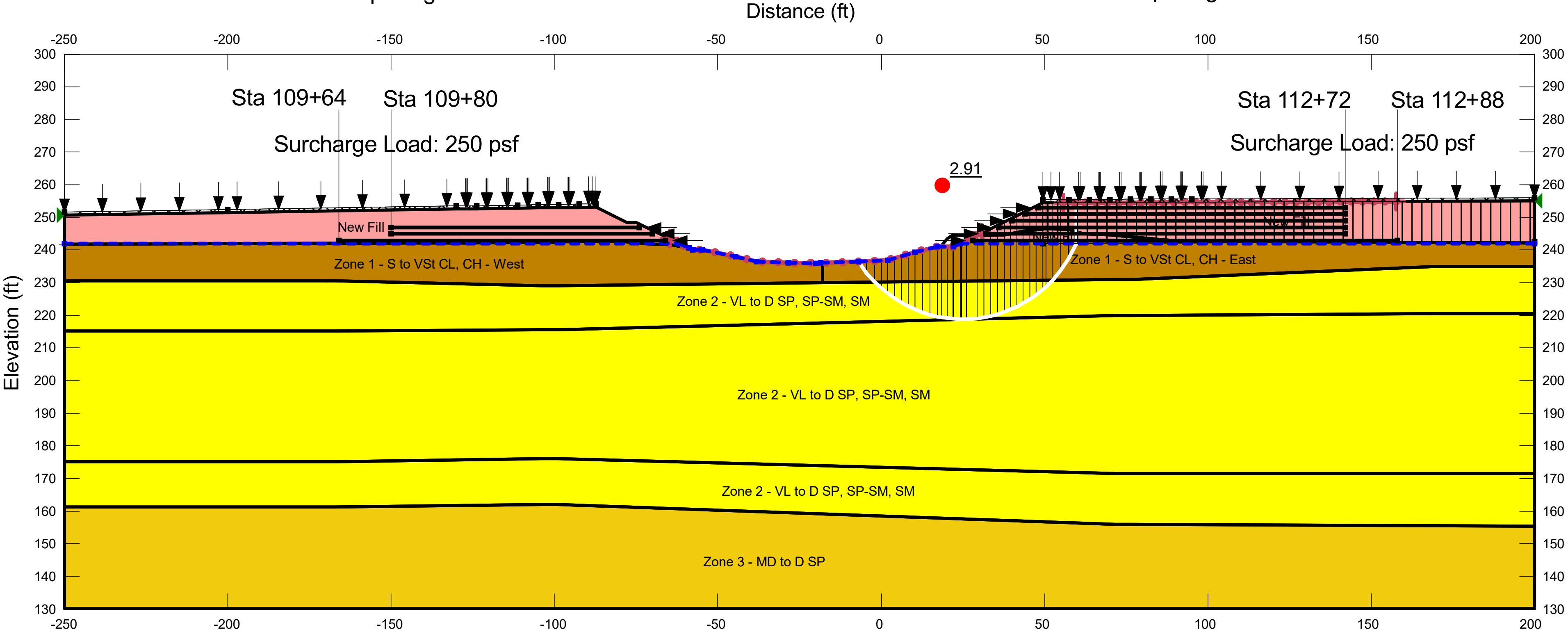
SR 230 Site 1
Craighead County, AR

Figure 21

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

West Abutment
3 layers of geosynthetic reinforcement
with tensile strength = 5,200 lbf
at 2-ft vertical spacing

East Abutment
6 layers of geosynthetic reinforcement
with tensile strength = 5,200 lbf
at 2-ft vertical spacing

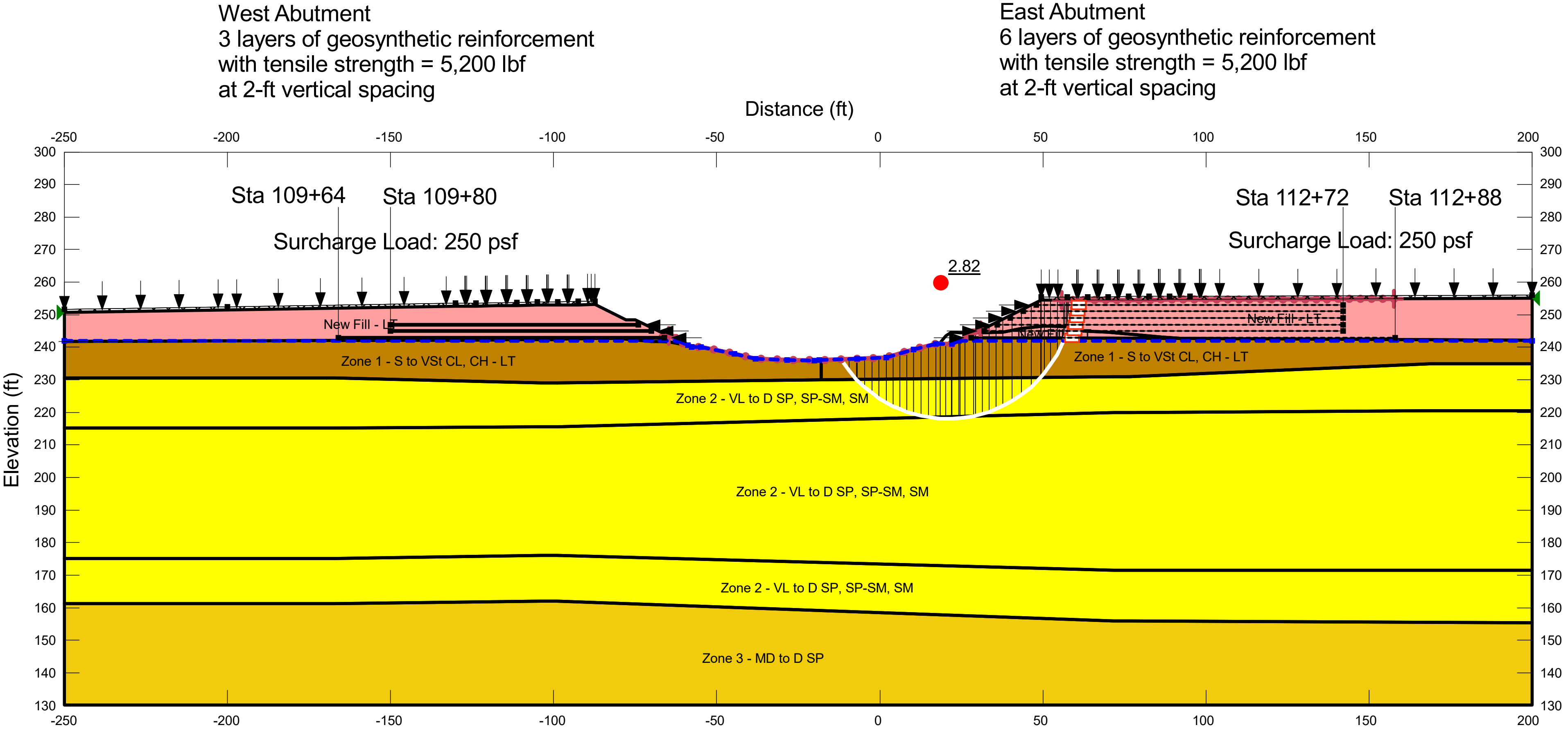


Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - S to VSt CL, CH - East	Mohr-Coulomb	122	500	0
🟫	Zone 1 - S to VSt CL, CH - West	Mohr-Coulomb	122	750	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟨	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

East Abutment Spill-Through
End of Construction

SR 230 Site 1
Craighead County, AR

Figure 22



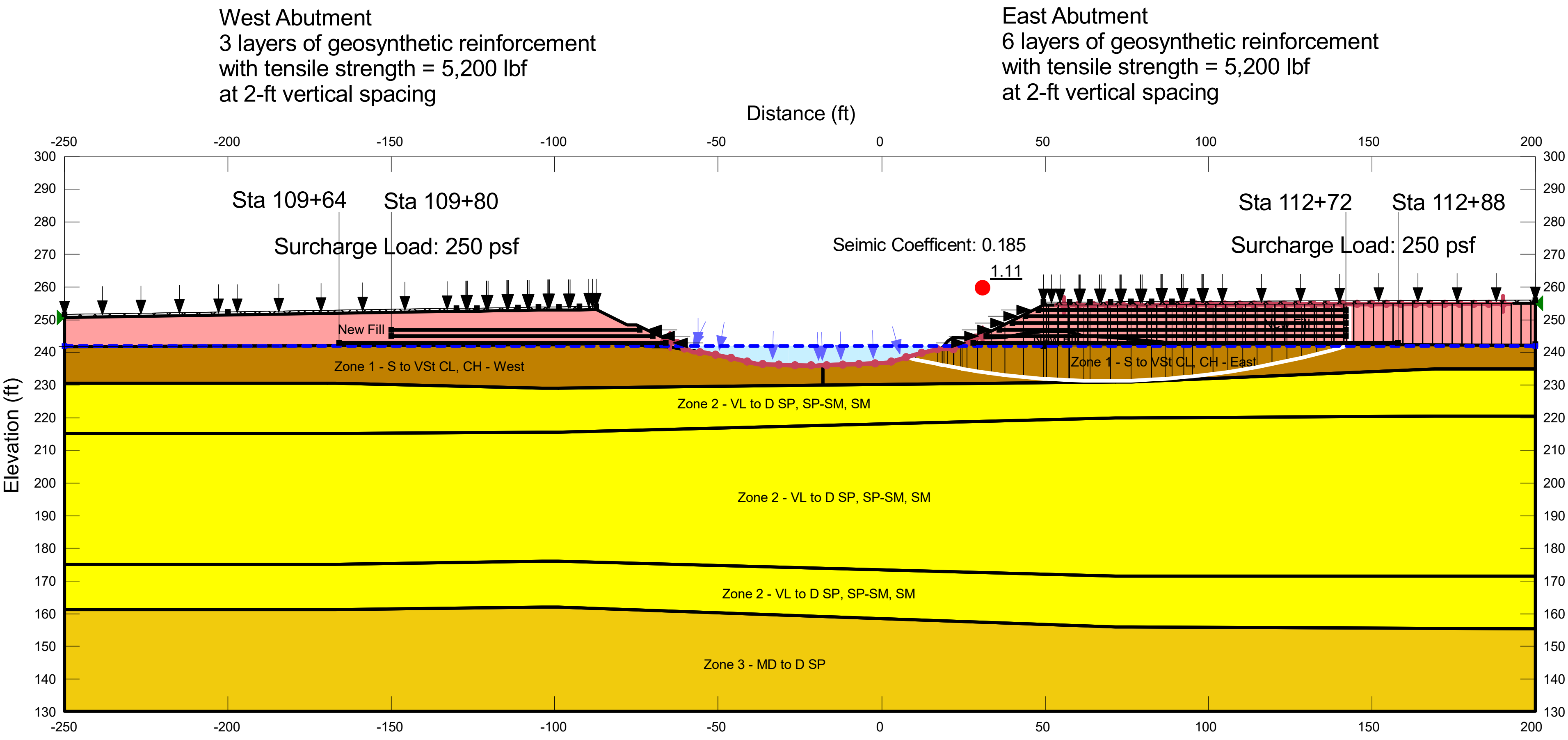
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
🟫	Zone 1 - S to VSt CL, CH - LT	Mohr-Coulomb	122	50	21
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

East Abutment Spill-Through
Long Term

SR 230 Site 1
Craighead County, AR

Figure 23

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



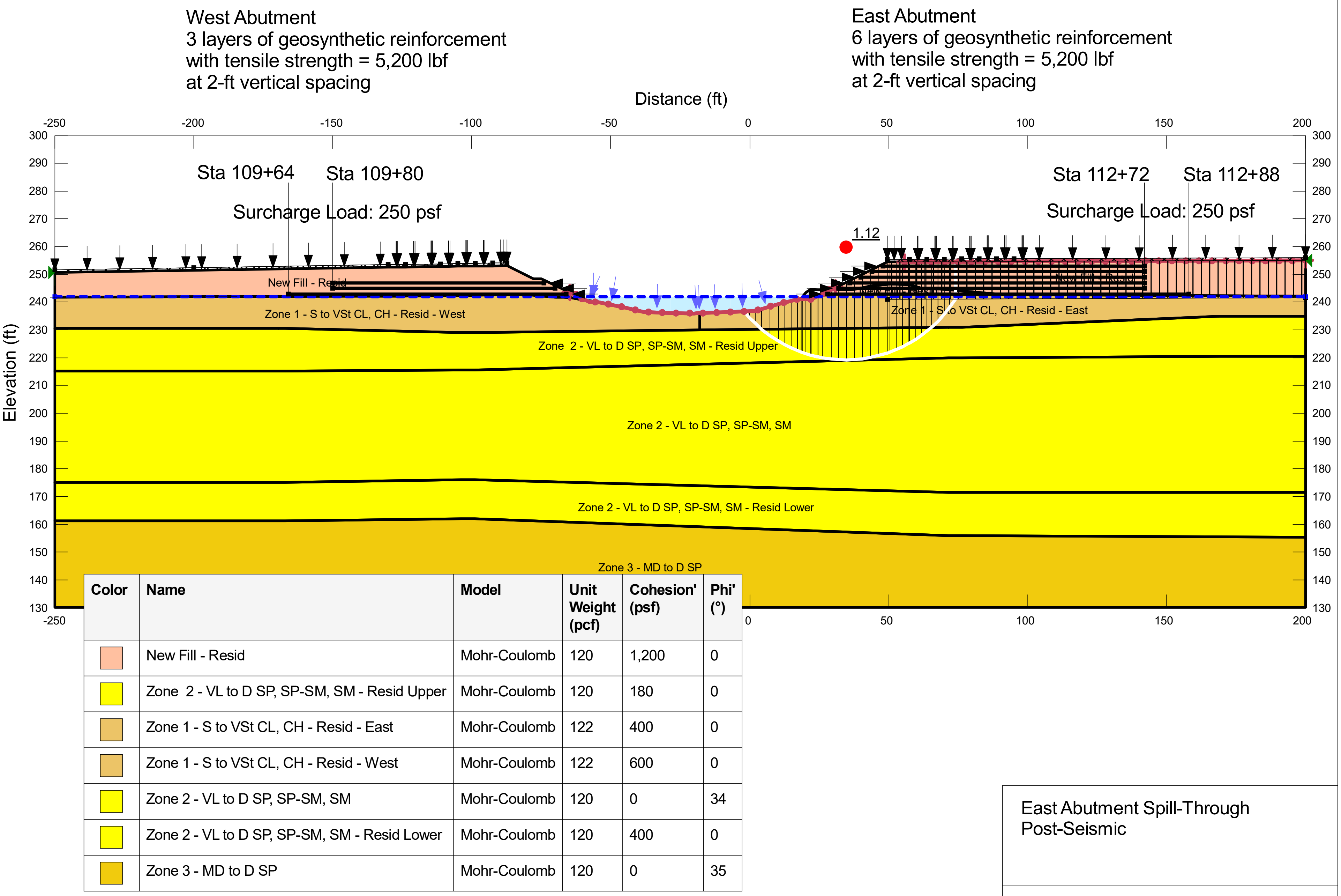
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - S to VSt CL, CH - East	Mohr-Coulomb	122	500	0
🟫	Zone 1 - S to VSt CL, CH - West	Mohr-Coulomb	122	750	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟨	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

East Abutment Spill-Through
Pseudostatic

SR 230 Site 1
Craighead County, AR

Figure 24

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



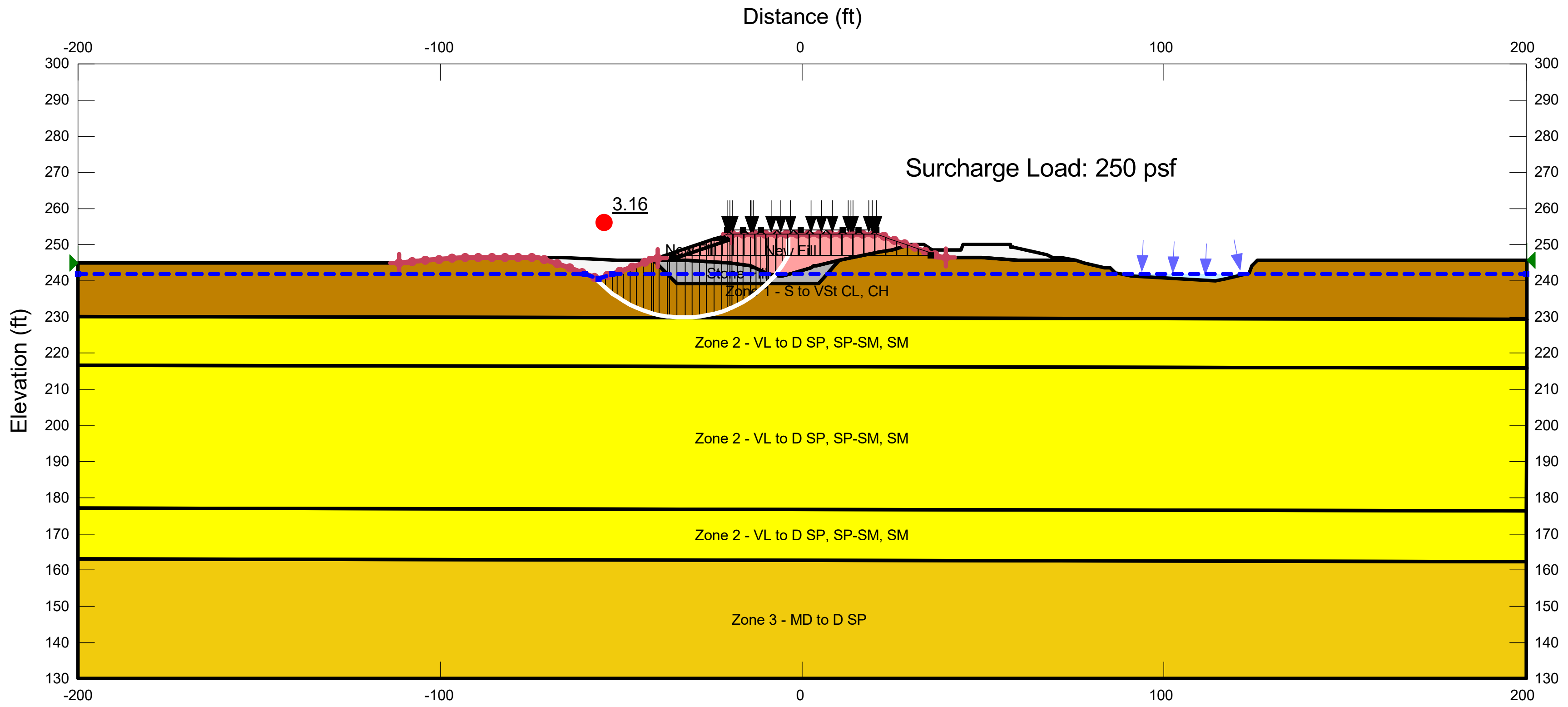
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
🟨	Zone 2 - VL to D SP, SP-SM, SM - Resid Upper	Mohr-Coulomb	120	180	0
🟫	Zone 1 - S to VSt CL, CH - Resid - East	Mohr-Coulomb	122	400	0
🟫	Zone 1 - S to VSt CL, CH - Resid - West	Mohr-Coulomb	122	600	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟨	Zone 2 - VL to D SP, SP-SM, SM - Resid Lower	Mohr-Coulomb	120	400	0
🟨	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

East Abutment Spill-Through
Post-Seismic

SR 230 Site 1
Craighead County, AR

Figure 25

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



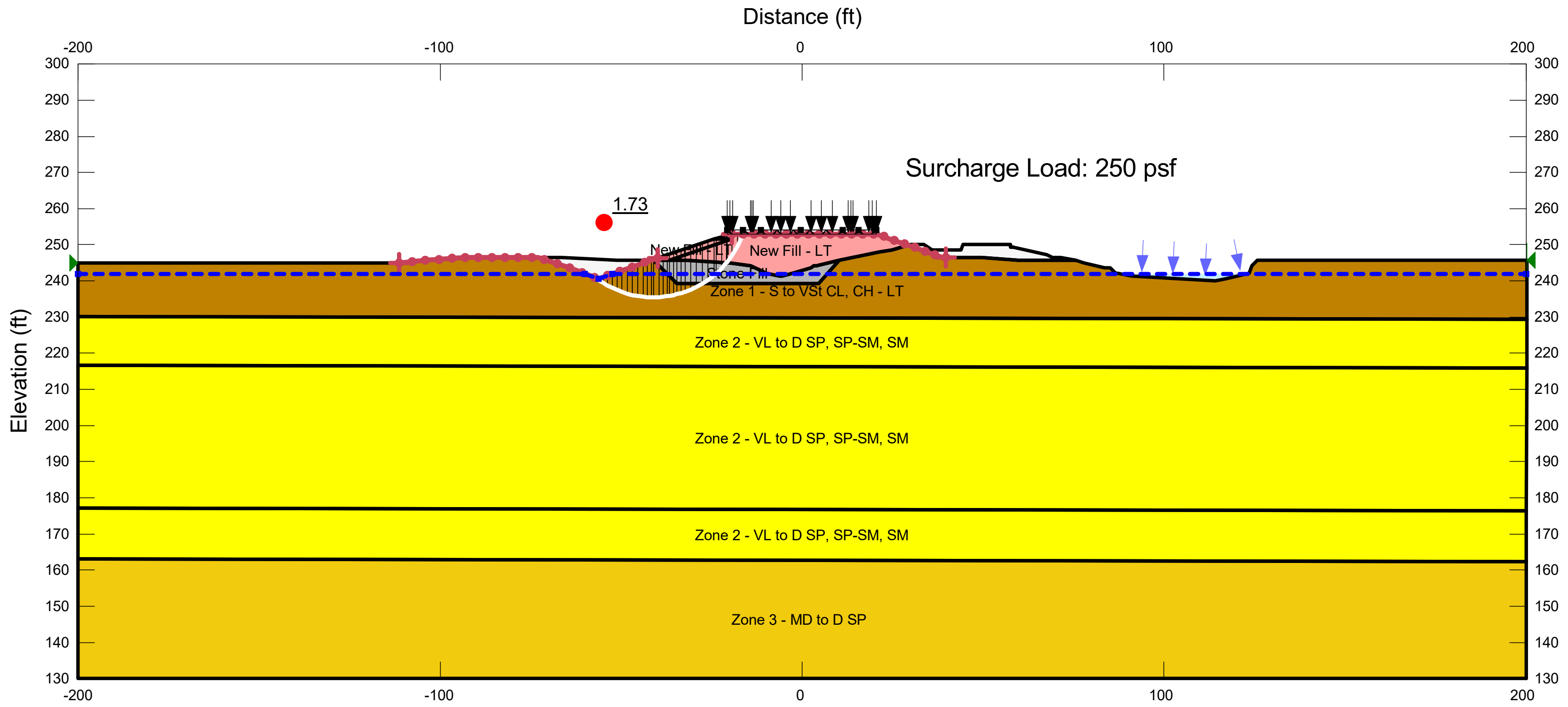
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
⬜	Stone Fill	Mohr-Coulomb	120	0	38
🟫	Zone 1 - S to VSt CL, CH	Mohr-Coulomb	122	750	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

West Abutment Side Slope
End of Construction

SR 230 Site 1
Craighead County, AR

Figure 26

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



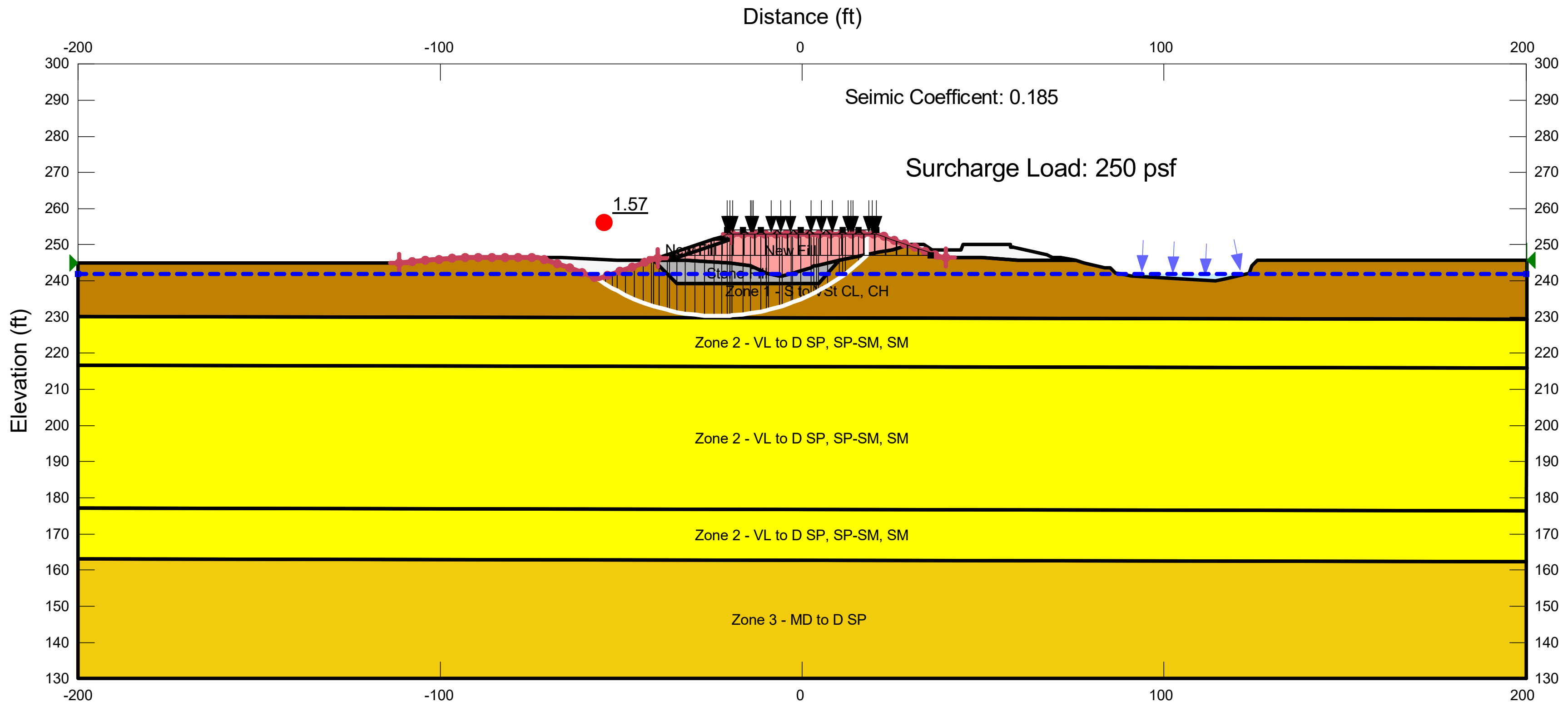
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
⬜	Stone Fill	Mohr-Coulomb	120	0	38
🟫	Zone 1 - S to VSt CL, CH - LT	Mohr-Coulomb	122	50	21
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

West Abutment Side Slope
Long Term

SR 230 Site 1
Craighead County, AR

Figure 27

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



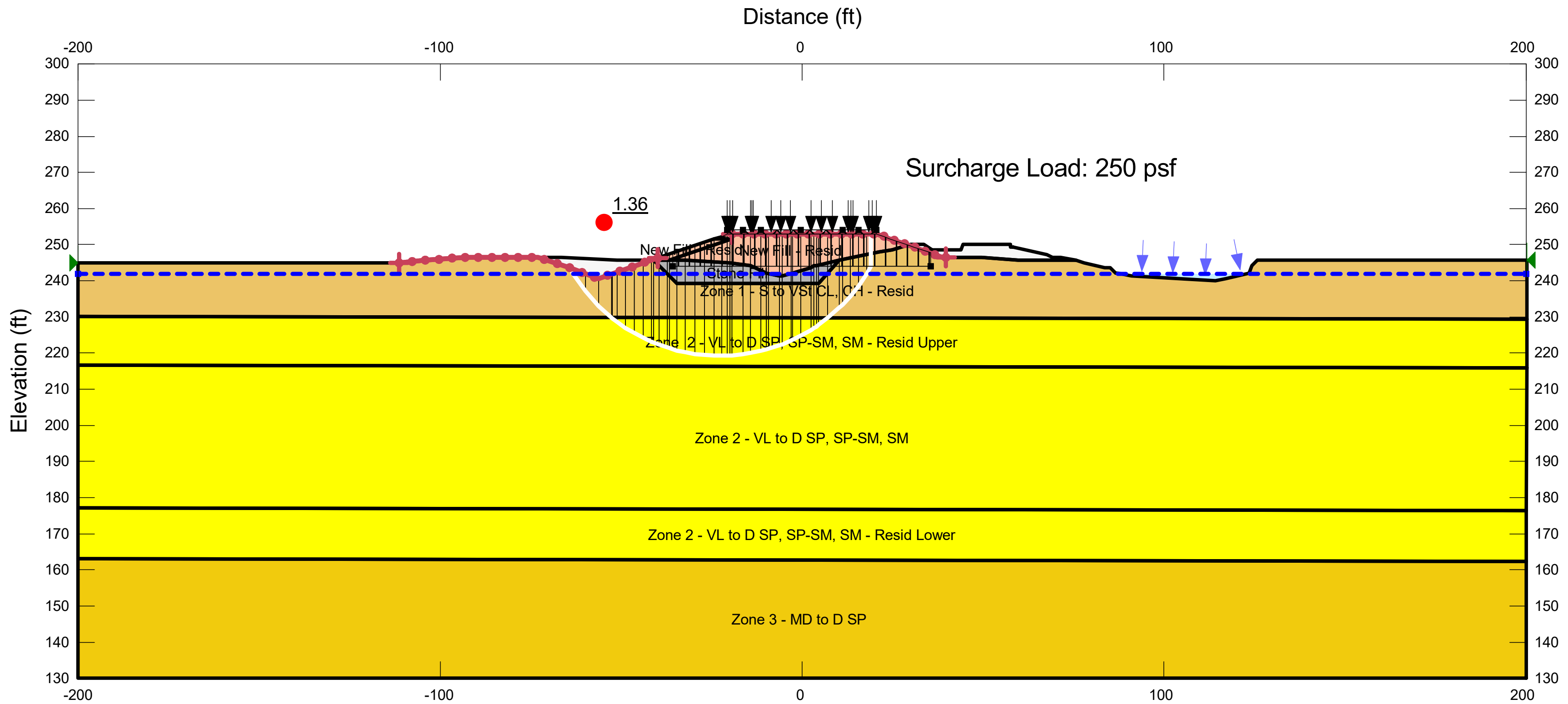
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
⬜	Stone Fill	Mohr-Coulomb	120	0	38
🟫	Zone 1 - S to VSt CL, CH	Mohr-Coulomb	122	750	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

West Abutment Side Slope
Pseudostatic

SR 230 Site 1
Craighead County, AR

Figure 28

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



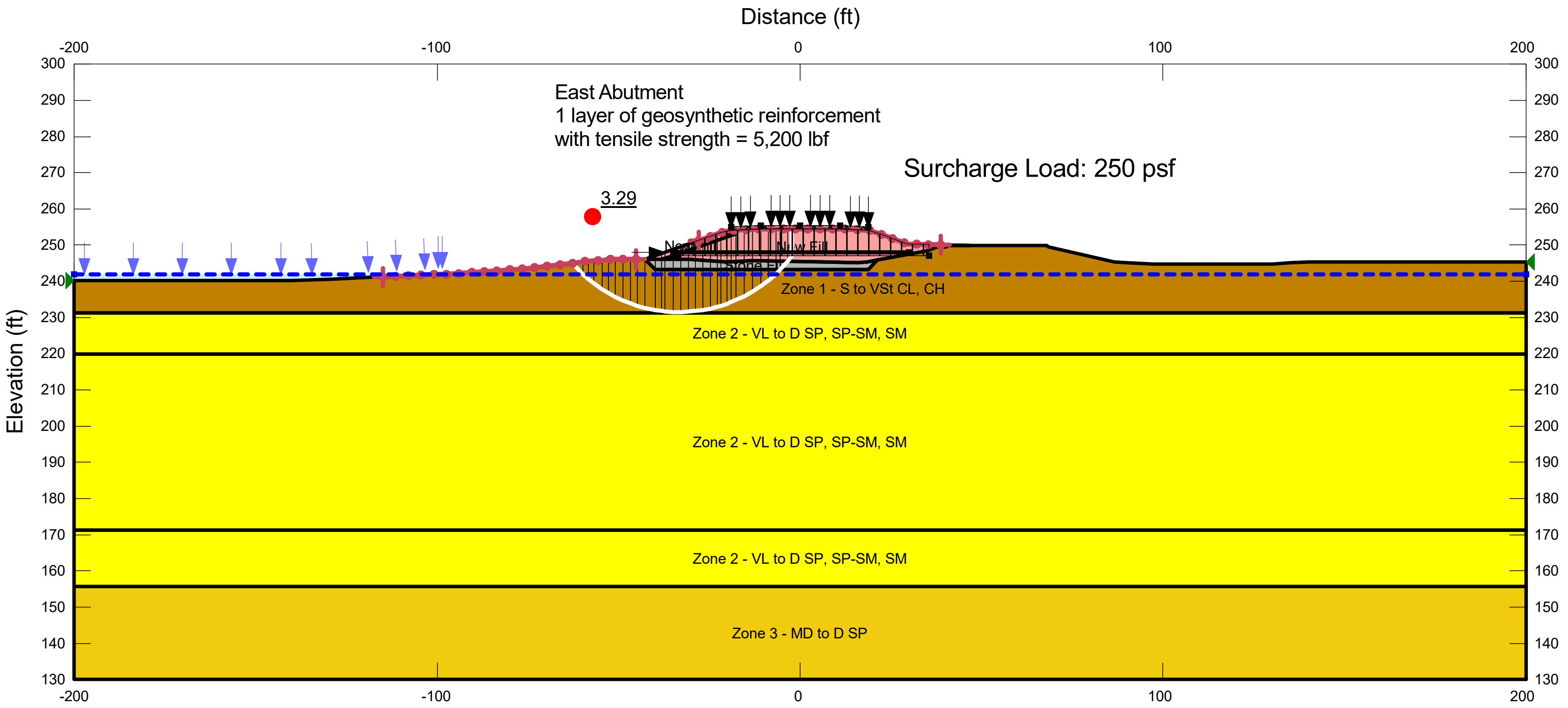
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
⬜	Stone Fill	Mohr-Coulomb	120	0	38
🟨	Zone 2 - VL to D SP, SP-SM, SM - Resid Upper	Mohr-Coulomb	120	180	0
🟫	Zone 1 - S to VSt CL, CH - Resid	Mohr-Coulomb	122	600	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟨	Zone 2 - VL to D SP, SP-SM, SM - Resid Lower	Mohr-Coulomb	120	400	0
🟨	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

West Abutment Side Slope
Post-Seismic

SR 230 Site 1
Craighead County, AR

Figure 29

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



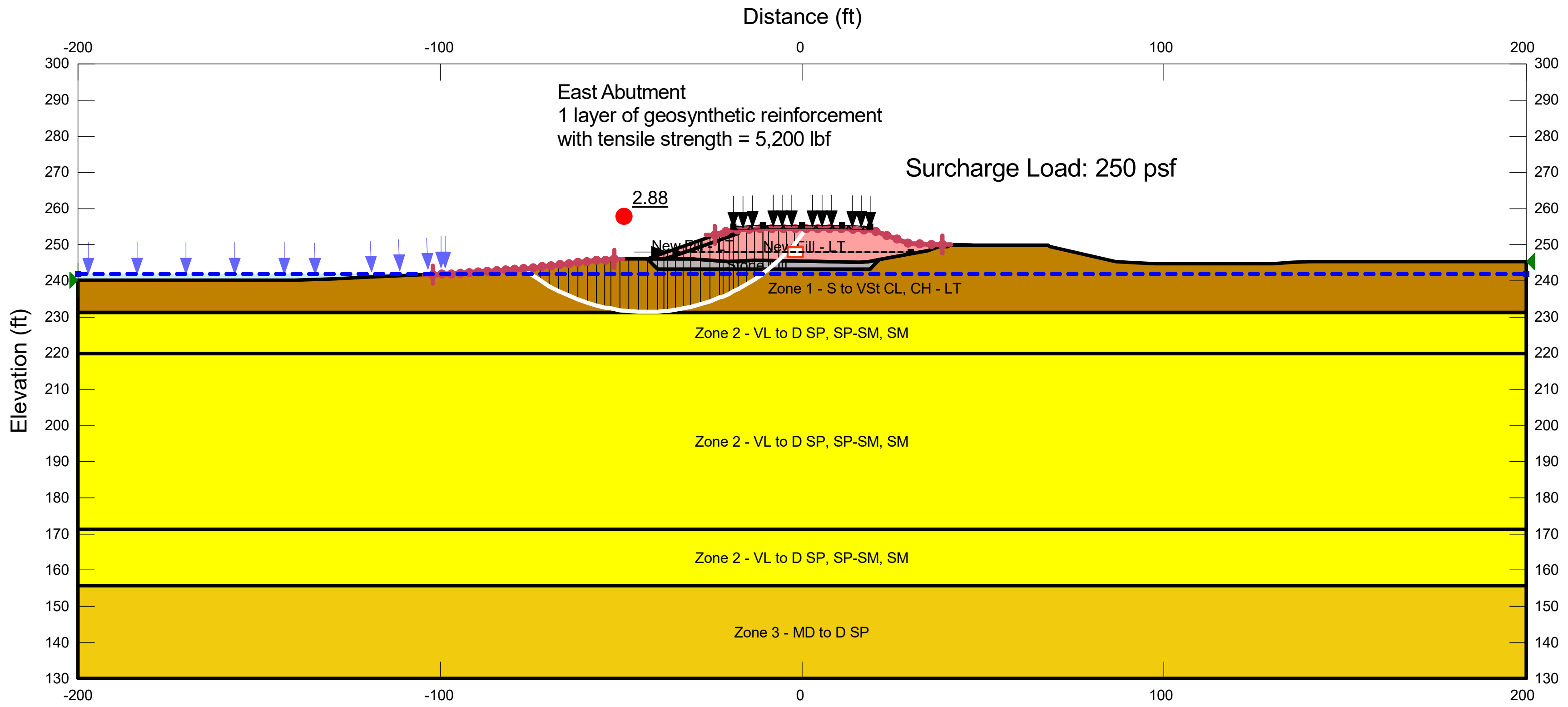
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
⬜	Stone Fill	Mohr-Coulomb	120	0	38
🟫	Zone 1 - S to VSt CL, CH	Mohr-Coulomb	122	500	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

East Abutment Side Slope
End of Construction

SR 230 Site 1
Craighead County, AR

Figure 30

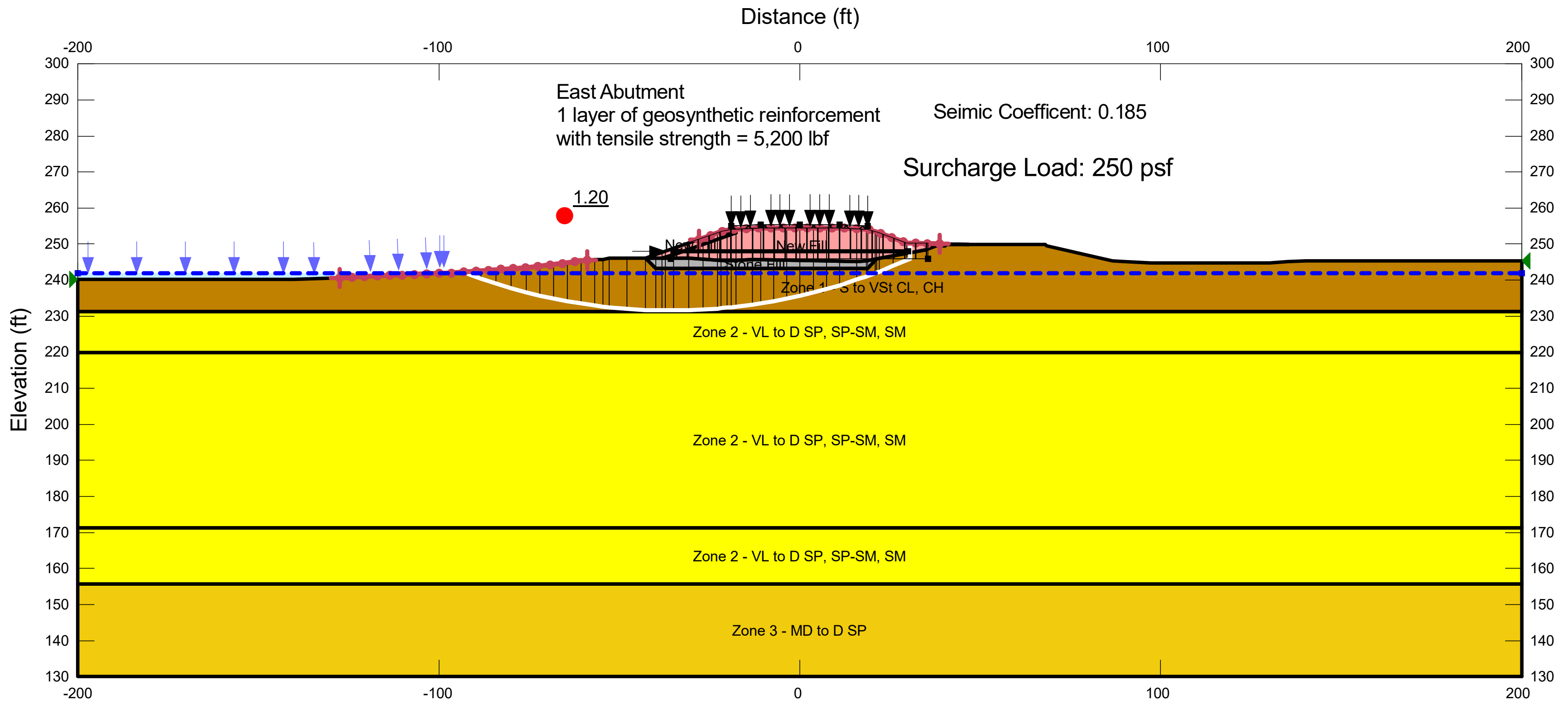


Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
⬜	Stone Fill	Mohr-Coulomb	120	0	38
🟫	Zone 1 - S to VSt CL, CH - LT	Mohr-Coulomb	122	50	21
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

East Abutment Side Slope
Long Term

SR 230 Site 1
Craighead County, AR

Figure 31



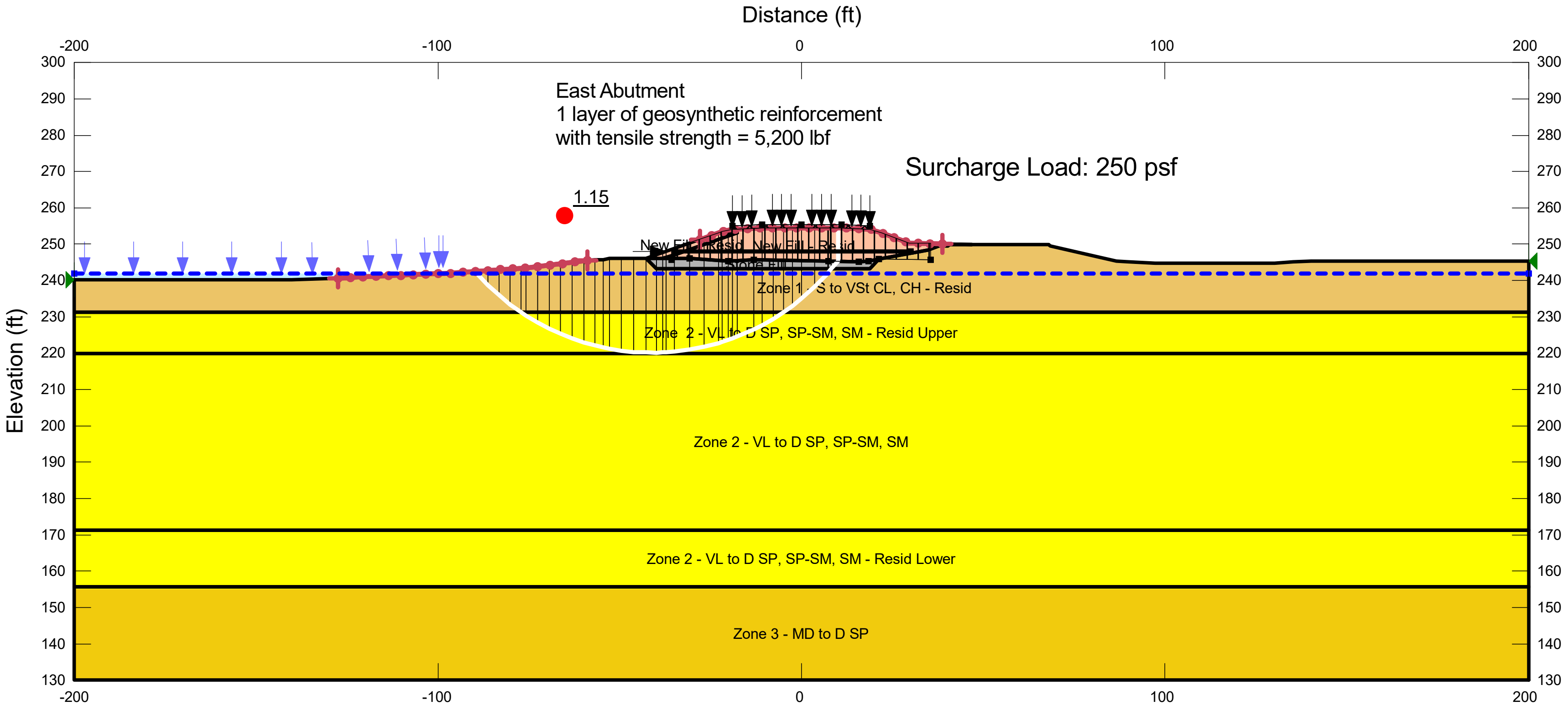
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
⬜	Stone Fill	Mohr-Coulomb	120	0	38
🟫	Zone 1 - S to VSt CL, CH	Mohr-Coulomb	122	500	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

East Abutment Side Slope
Pseudostatic

SR 230 Site 1
Craighead County, AR

Figure 32

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



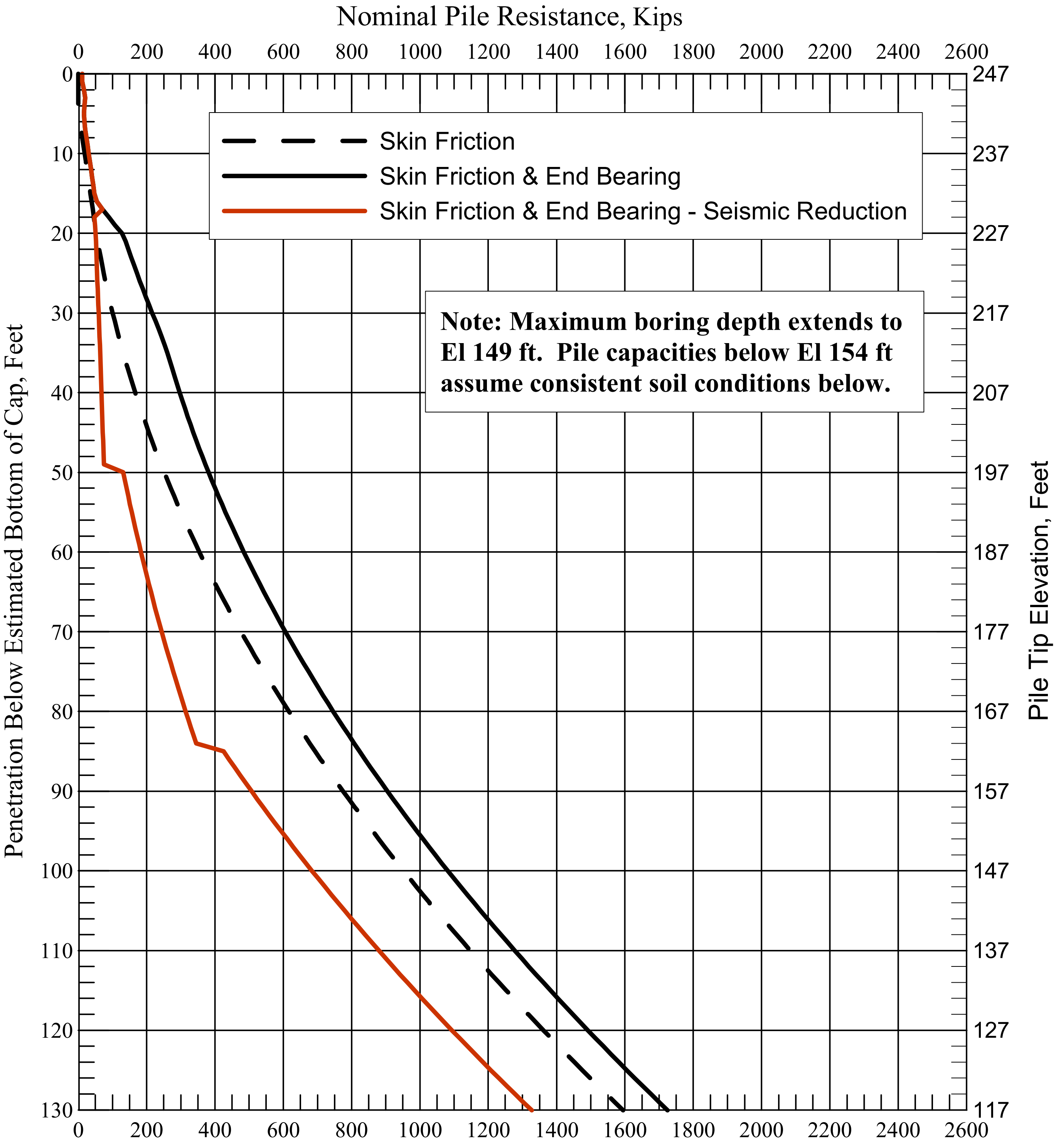
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
⬜	Stone Fill	Mohr-Coulomb	120	0	38
🟨	Zone 2 - VL to D SP, SP-SM, SM - Resid Upper	Mohr-Coulomb	120	180	0
🟫	Zone 1 - S to VSt CL, CH - Resid	Mohr-Coulomb	122	400	0
🟨	Zone 2 - VL to D SP, SP-SM, SM	Mohr-Coulomb	120	0	34
🟨	Zone 2 - VL to D SP, SP-SM, SM - Resid Lower	Mohr-Coulomb	120	400	0
🟨	Zone 3 - MD to D SP	Mohr-Coulomb	120	0	35

East Abutment Side Slope
Post-Seismic

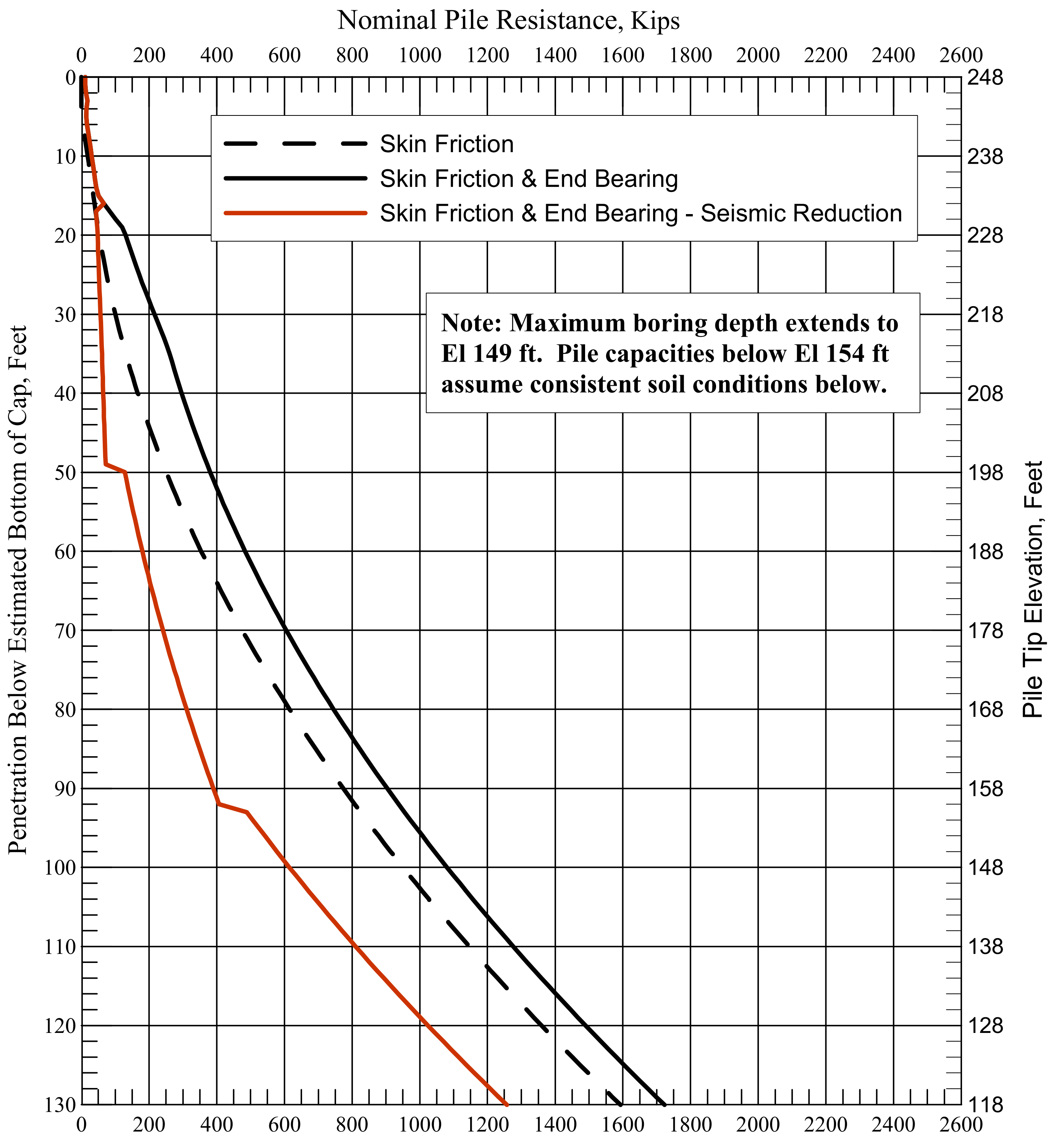
SR 230 Site 1
Craighead County, AR

Figure 33

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



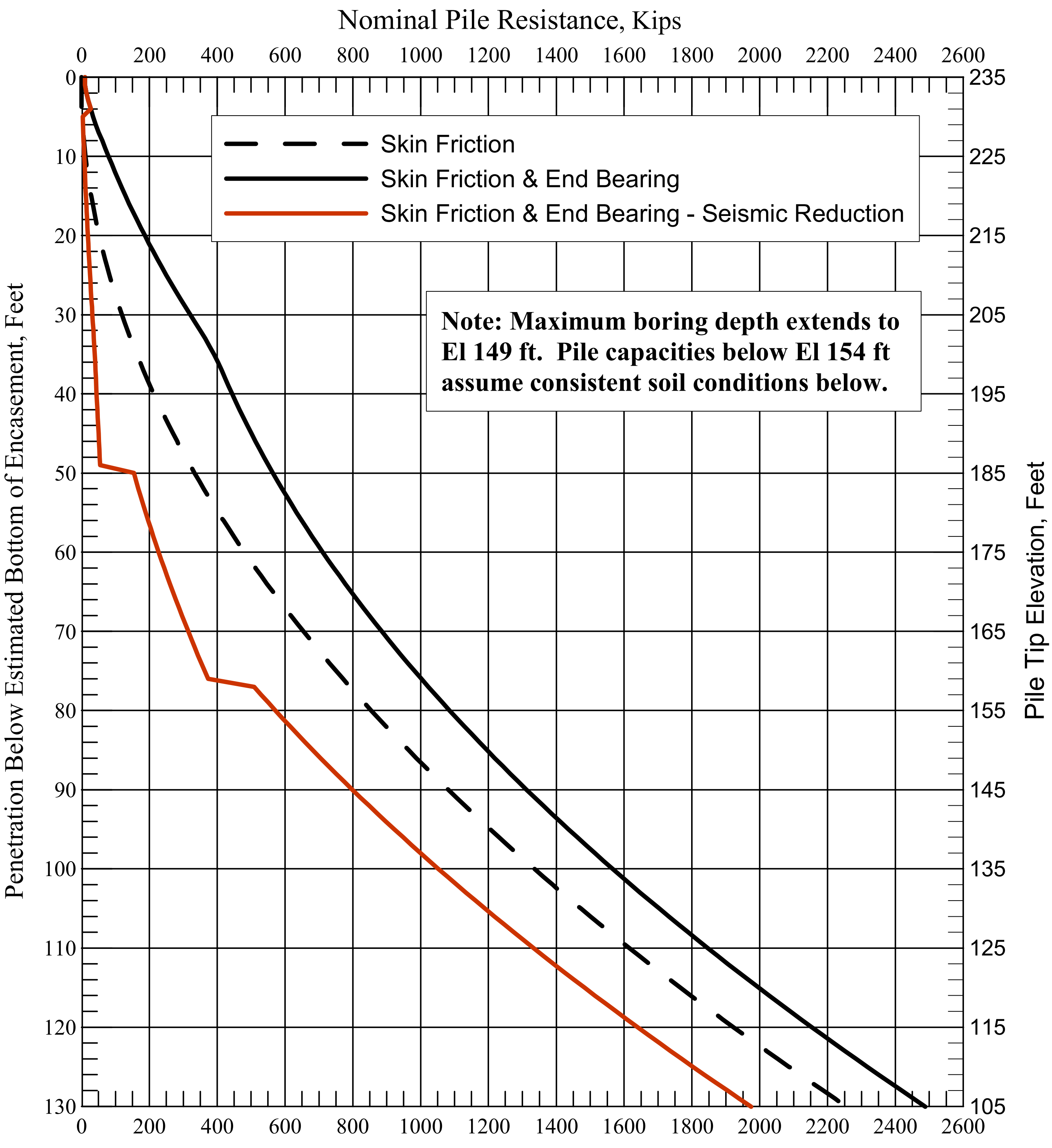
NOMINAL PILE RESISTANCE CURVES
SITE 1
WEST ABUTMENT
18" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS



NOMINAL PILE RESISTANCE CURVES
SITE 1
EAST ABUTMENT
18" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.

FIGURE 35

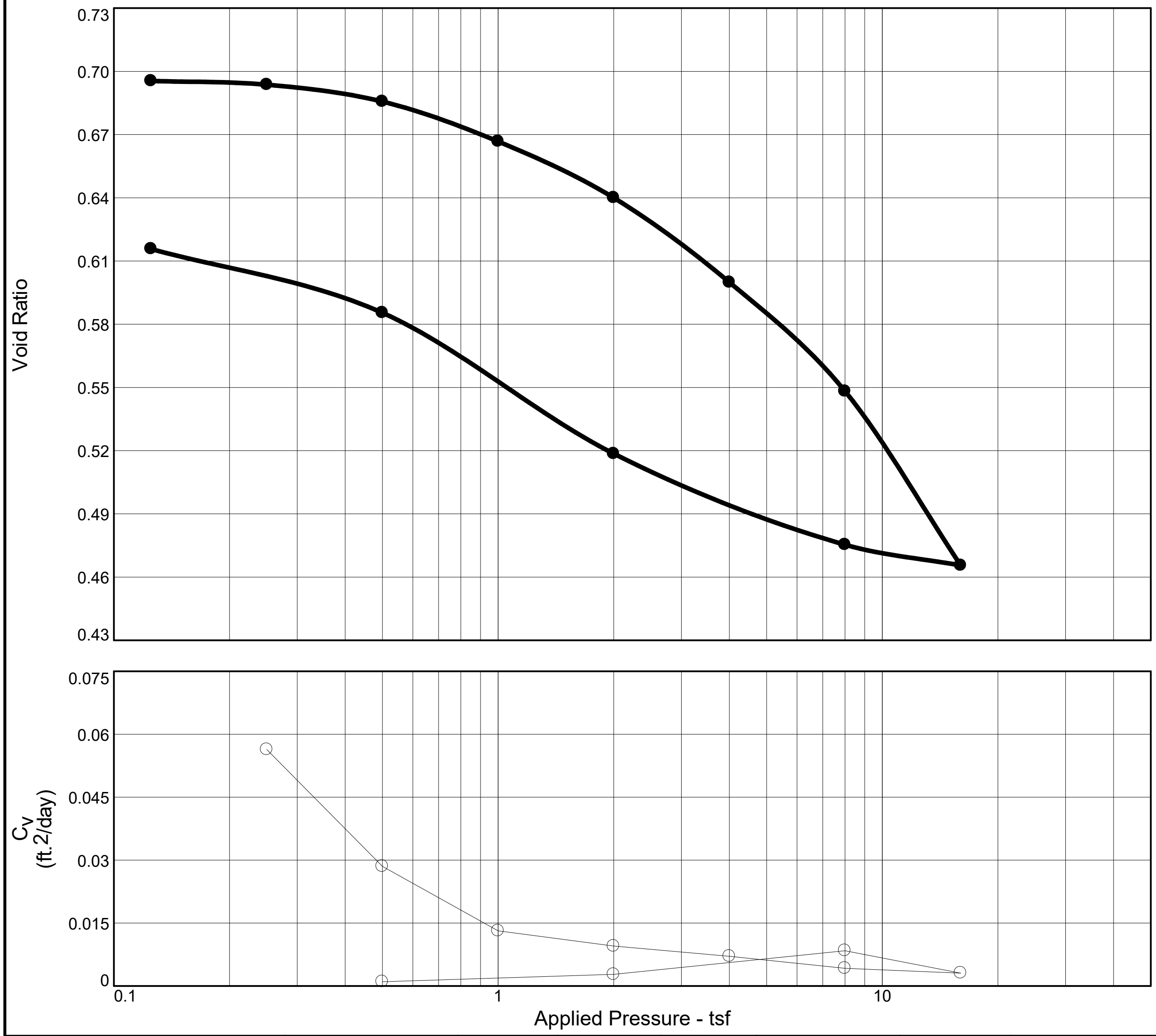


NOMINAL PILE RESISTANCE CURVES
SITE 1
INTERIOR BENTS
24" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

APPENDIX A

Consolidation Test Results and Particle Size Distribution Curves

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (tsf)	C_c	Initial Void Ratio
Saturation	Moisture							
98.8 %	24.4 %	100.4	39	23	2.67	3.8	0.28	0.660

MATERIAL DESCRIPTION	USCS	AASHTO
Medium stiff tan and light gray silty clay (CL)	CL	

Project No. 200518 **Client:** ARDOT

Project: SR-230 Alicia to Bono, AR

Source of Sample: S1-3 **Depth:** 14.5' **Sample Number:** 5C

BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Remarks:

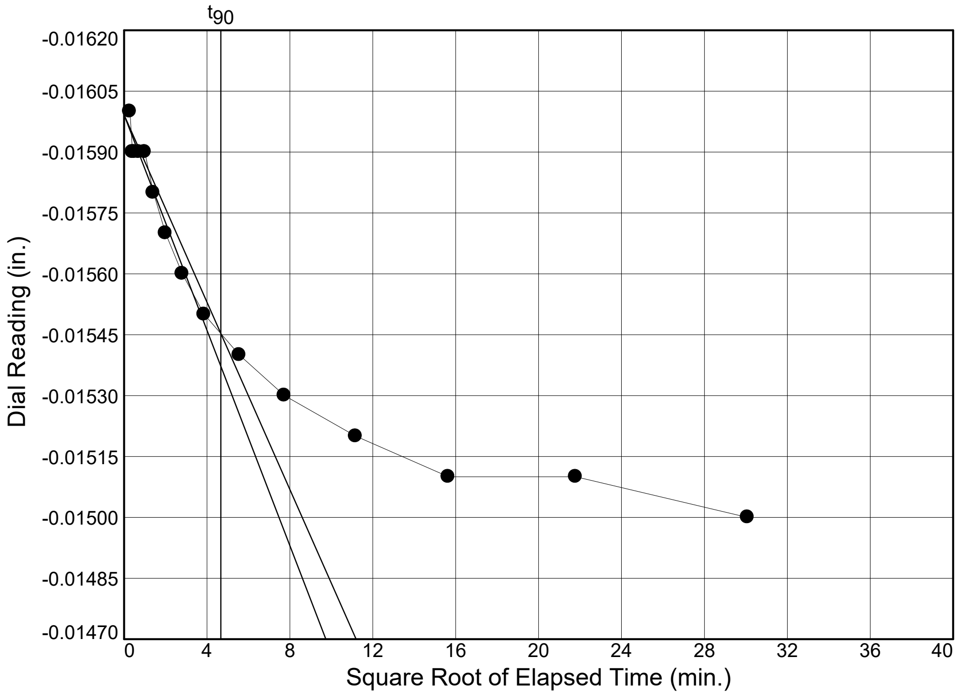
Figure

Checked By: ____________________

Dial Reading vs. Time

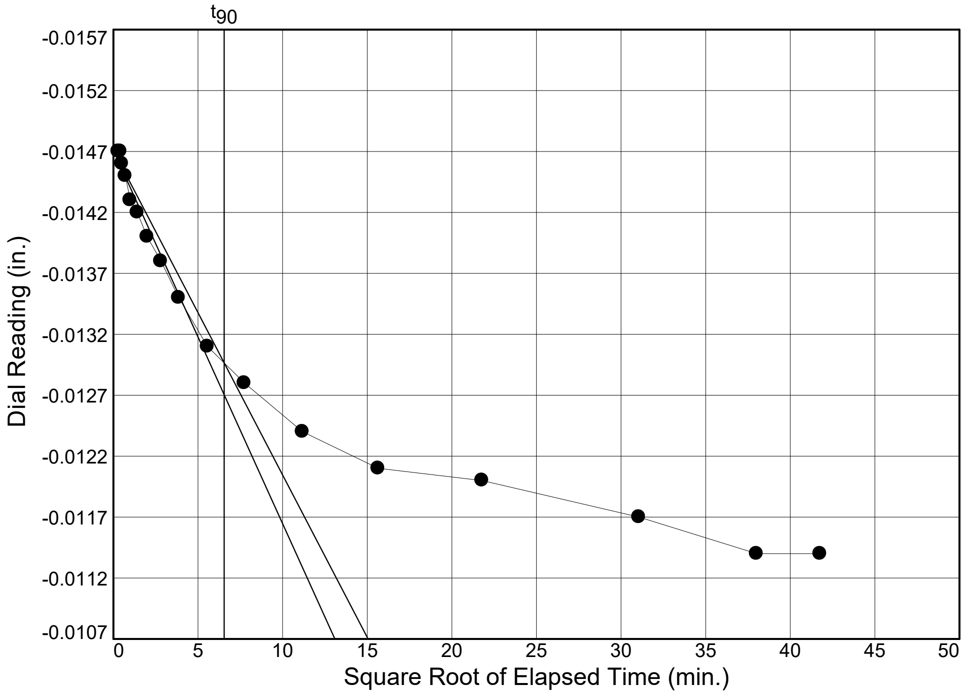
Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S1-3 Depth: 14.5' Sample Number: 5C



Load No.= 2
Load= 0.25 tsf
D_0 = -0.0160
D_{90} = -0.0155
D_{100} = -0.0154
T_{90} = 21.79 min.

C_v @ T_{90}
0.056 ft.^{2}/day



Load No.= 3
Load= 0.50 tsf
D_0 = -0.0147
D_{90} = -0.0130
D_{100} = -0.0128
T_{90} = 42.80 min.

C_v @ T_{90}
0.029 ft.^{2}/day

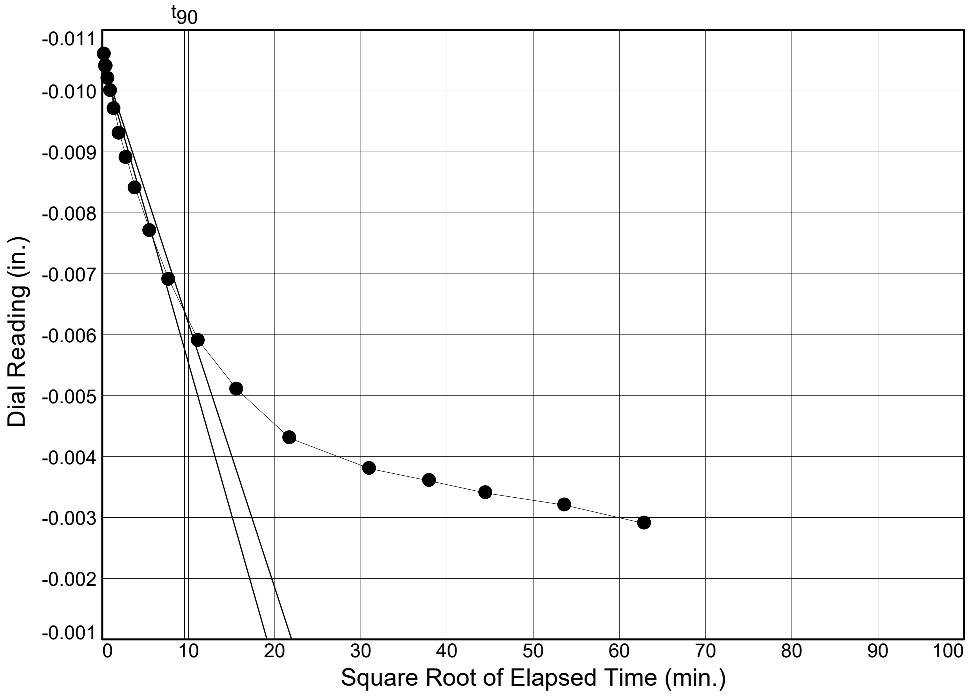
Figure

BURNS COOLEY DENNIS, INC.

Dial Reading vs. Time

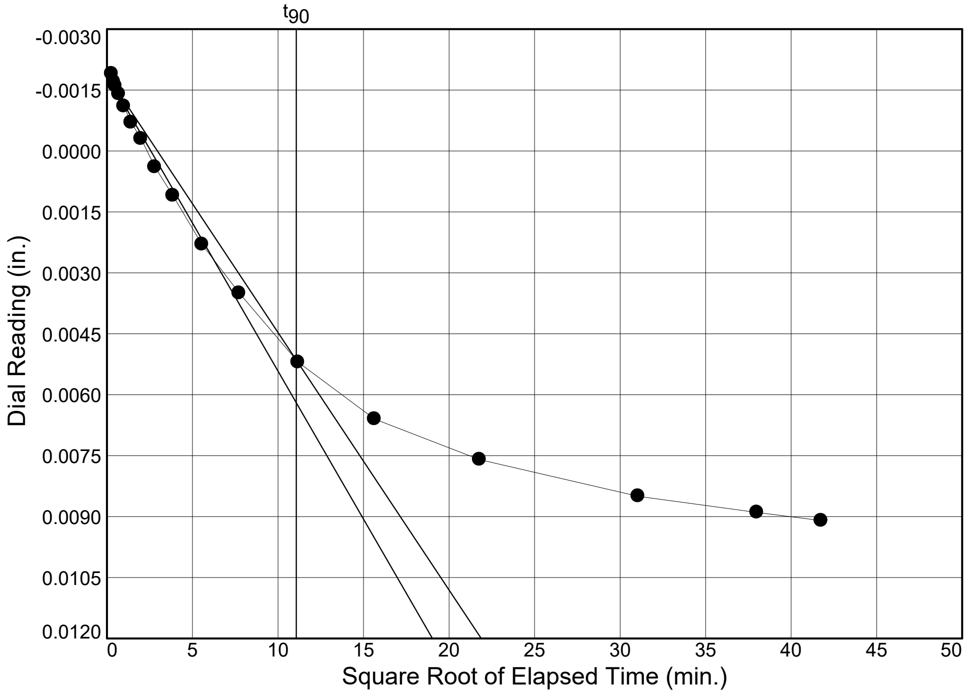
Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S1-3 **Depth:** 14.5' **Sample Number:** 5C



Load No.= 4
Load= 1.00 tsf
D_0 = -0.0105
D_{90} = -0.0064
D_{100} = -0.0059
T_{90} = 91.27 min.

C_v @ T_{90}
0.013 ft.^{2}/day



Load No.= 5
Load= 2.00 tsf
D_0 = -0.0019
D_{90} = 0.0051
D_{100} = 0.0059
T_{90} = 122.63 min.

C_v @ T_{90}
0.010 ft.^{2}/day

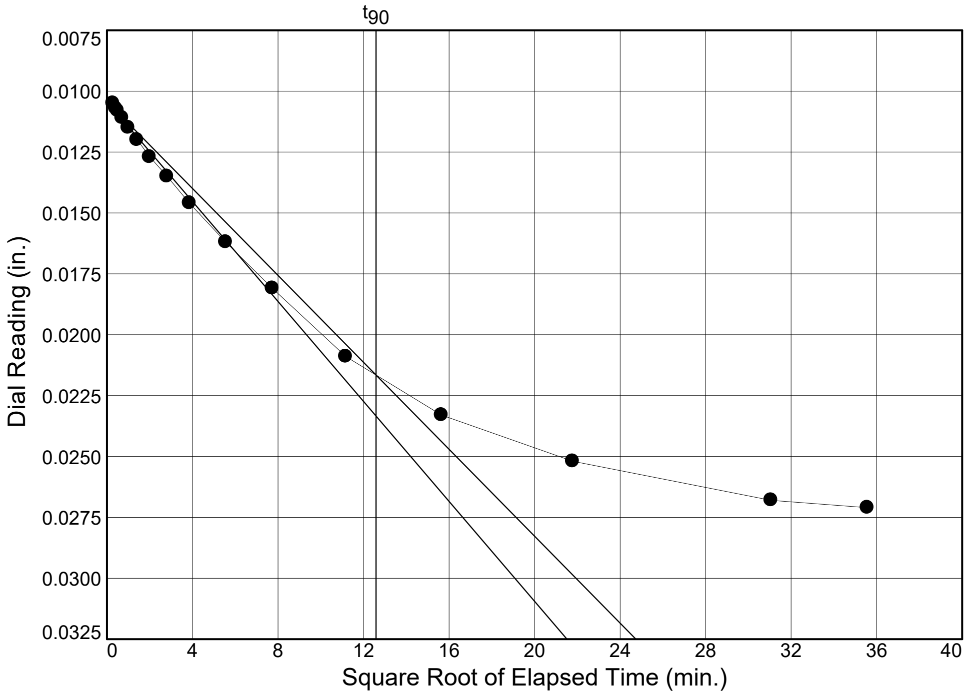
Figure

BURNS COOLEY DENNIS, INC.

Dial Reading vs. Time

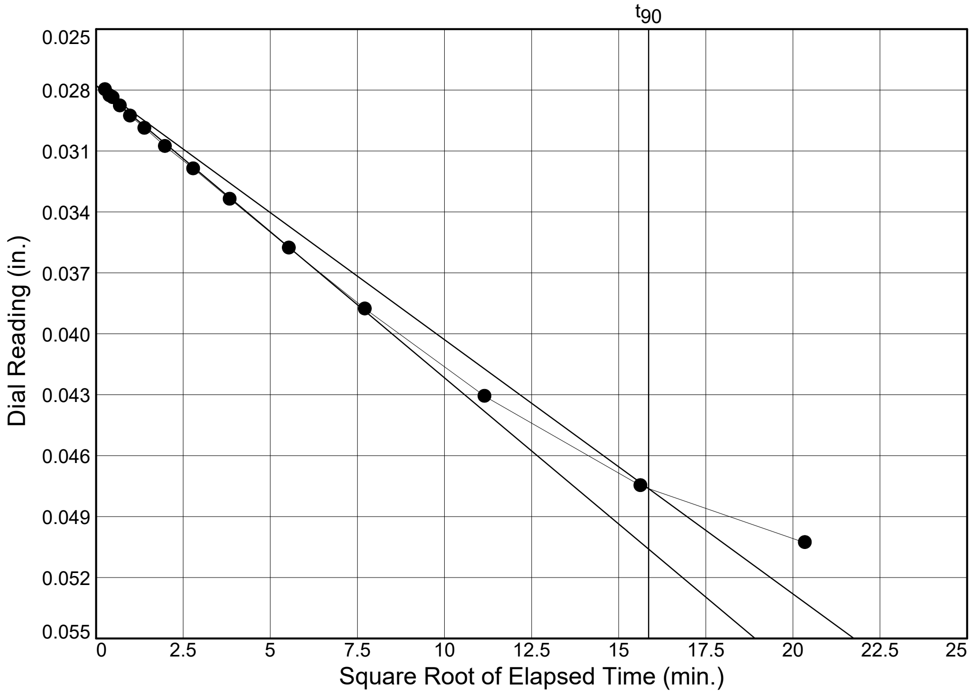
Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S1-3 Depth: 14.5' Sample Number: 5C



Load No.= 6
Load= 4.00 tsf
D_0 = 0.0104
D_{90} = 0.0217
D_{100} = 0.0229
T_{90} = 158.34 min.

C_v @ T_{90}
0.007 ft.^{2}/day



Load No.= 7
Load= 8.00 tsf
D_0 = 0.0278
D_{90} = 0.0476
D_{100} = 0.0498
T_{90} = 251.52 min.

C_v @ T_{90}
0.004 ft.^{2}/day

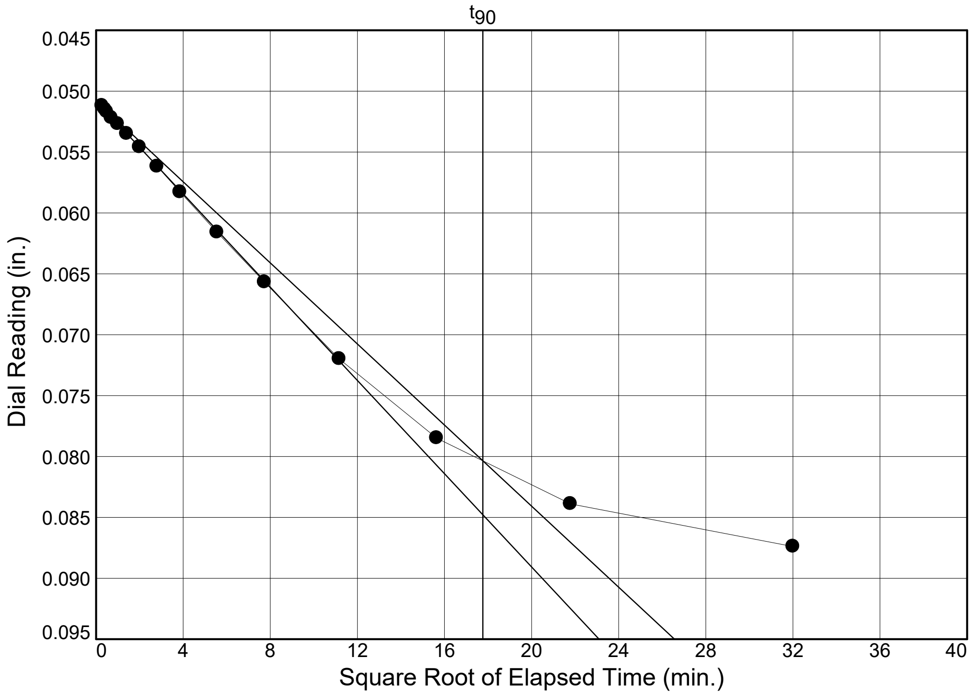
Figure

BURNS COOLEY DENNIS, INC.

Dial Reading vs. Time

Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S1-3 Depth: 14.5' Sample Number: 5C



Load No.= 8

Load= 16.00 tsf

D_0 = 0.0508

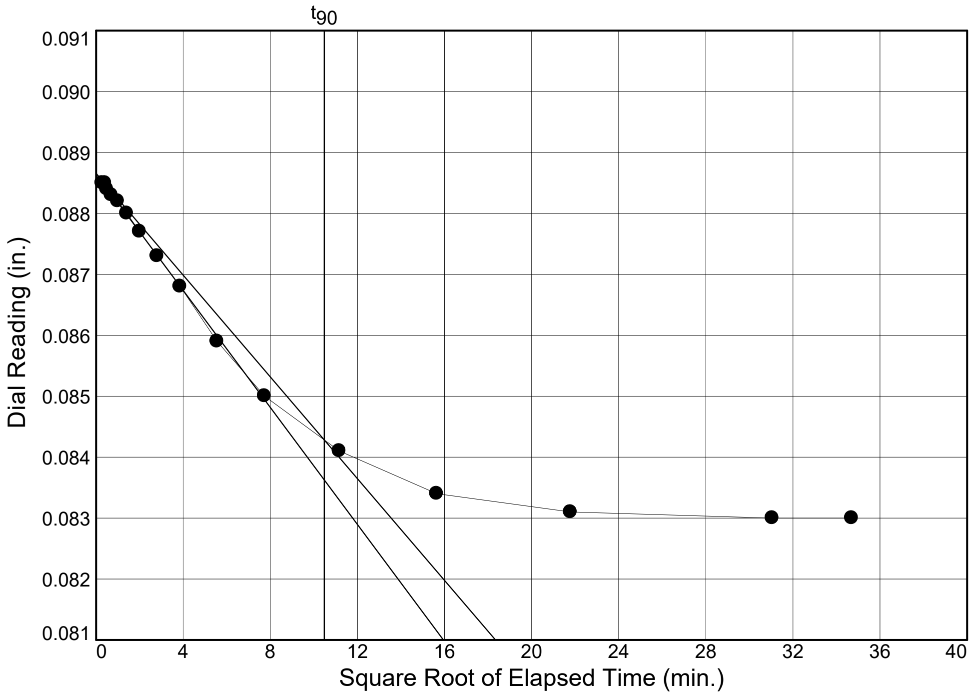
D_{90} = 0.0804

D_{100} = 0.0836

T_{90} = 315.51 min.

C_v @ T_{90}

0.003 ft.^{2}/day



Load No.= 9

Load= 8.00 tsf

D_0 = 0.0887

D_{90} = 0.0843

D_{100} = 0.0838

T_{90} = 109.80 min.

C_v @ T_{90}

0.008 ft.^{2}/day

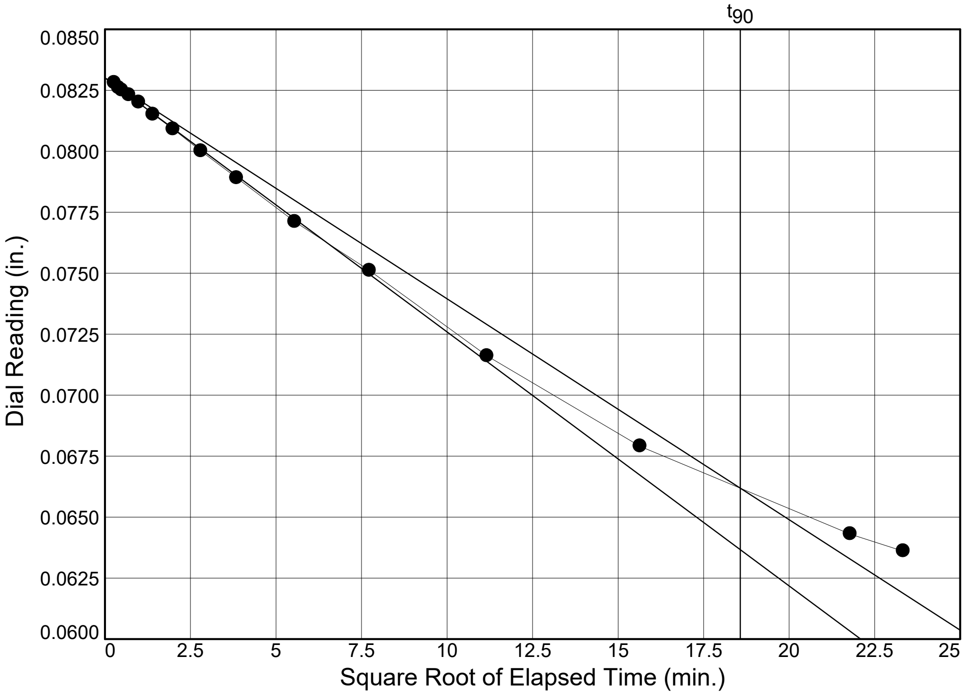
Figure

BURNS COOLEY DENNIS, INC.

Dial Reading vs. Time

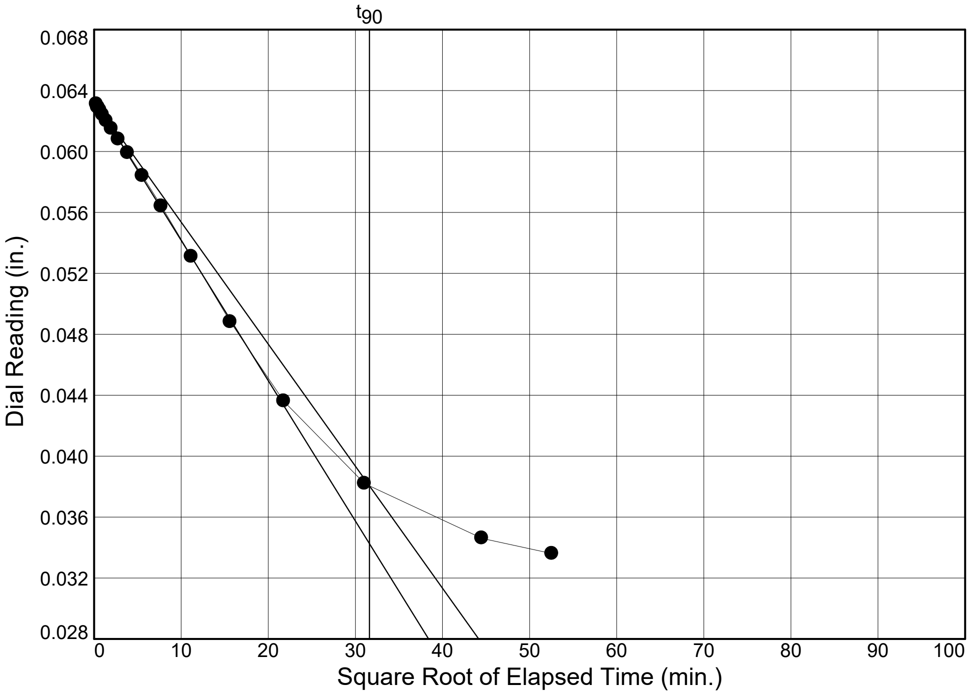
Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S1-3 Depth: 14.5' Sample Number: 5C



Load No.= 10
Load= 2.00 tsf
D_0 = 0.0830
D_{90} = 0.0662
D_{100} = 0.0643
T_{90} = 344.95 min.

C_v @ T_{90}
0.003 ft.^{2}/day

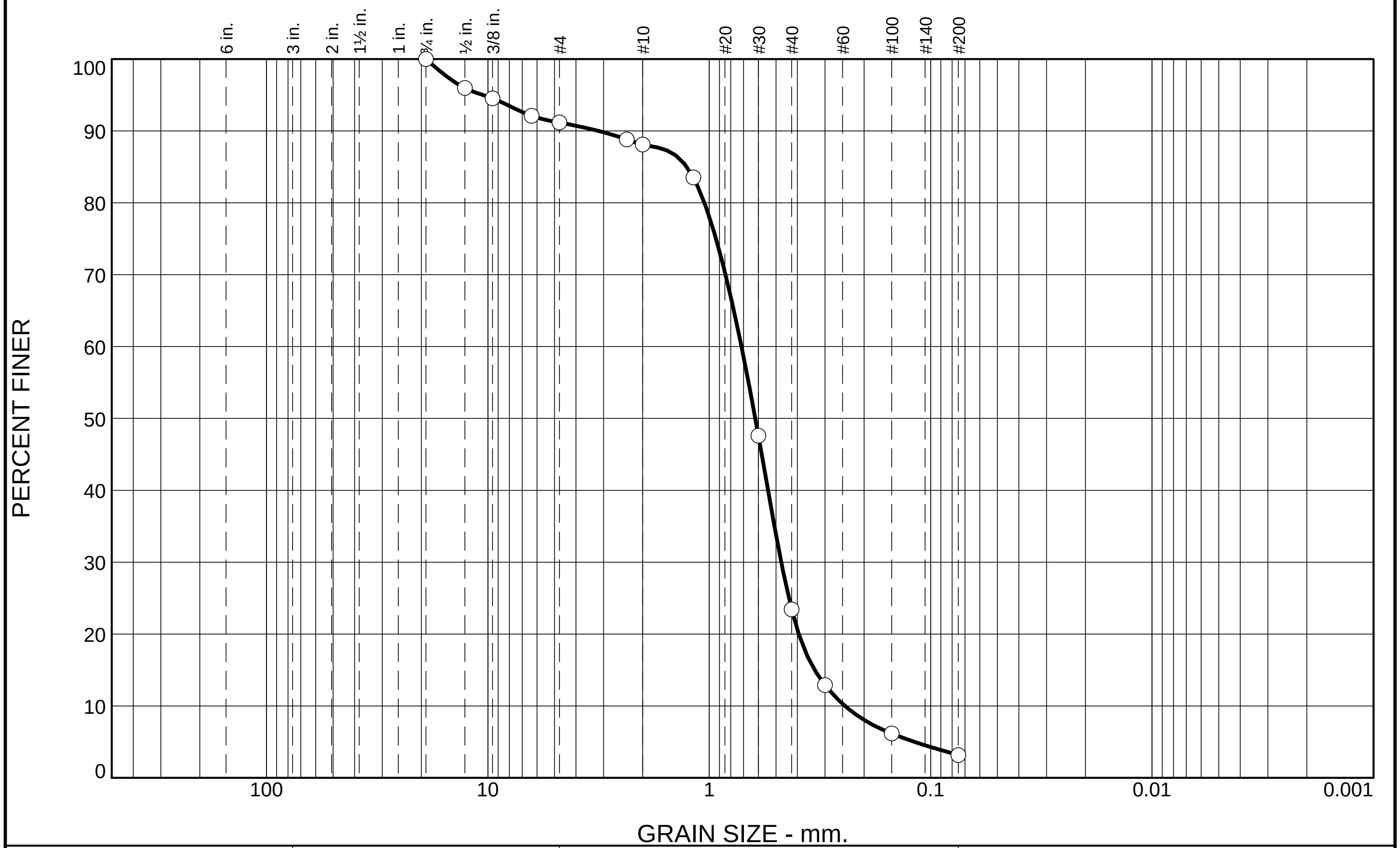


Load No.= 11
Load= 0.50 tsf
D_0 = 0.0634
D_{90} = 0.0381
D_{100} = 0.0352
T_{90} = 999.21 min.

C_v @ T_{90}
0.001 ft.^{2}/day

Figure

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	8.8	3.1	64.7	20.2	3.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	96.0		
3/8	94.5		
1/4	92.1		
#4	91.2		
#8	88.8		
#10	88.1		
#16	83.5		
#30	47.6		
#40	23.4		
#50	12.9		
#100	6.2		
#200	3.2		

* (no specification provided)

Soil Description

Gray fine to coarse sand (SP) with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.1593 D_{85}= 1.2627 D_{60}= 0.7152
D_{50}= 0.6197 D_{30}= 0.4736 D_{15}= 0.3348
D_{10}= 0.2441 C_u= 2.93 C_c= 1.28

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S1-2 **Depth:** 58.5'
Sample Number: 15 **Date:** 09-14-20

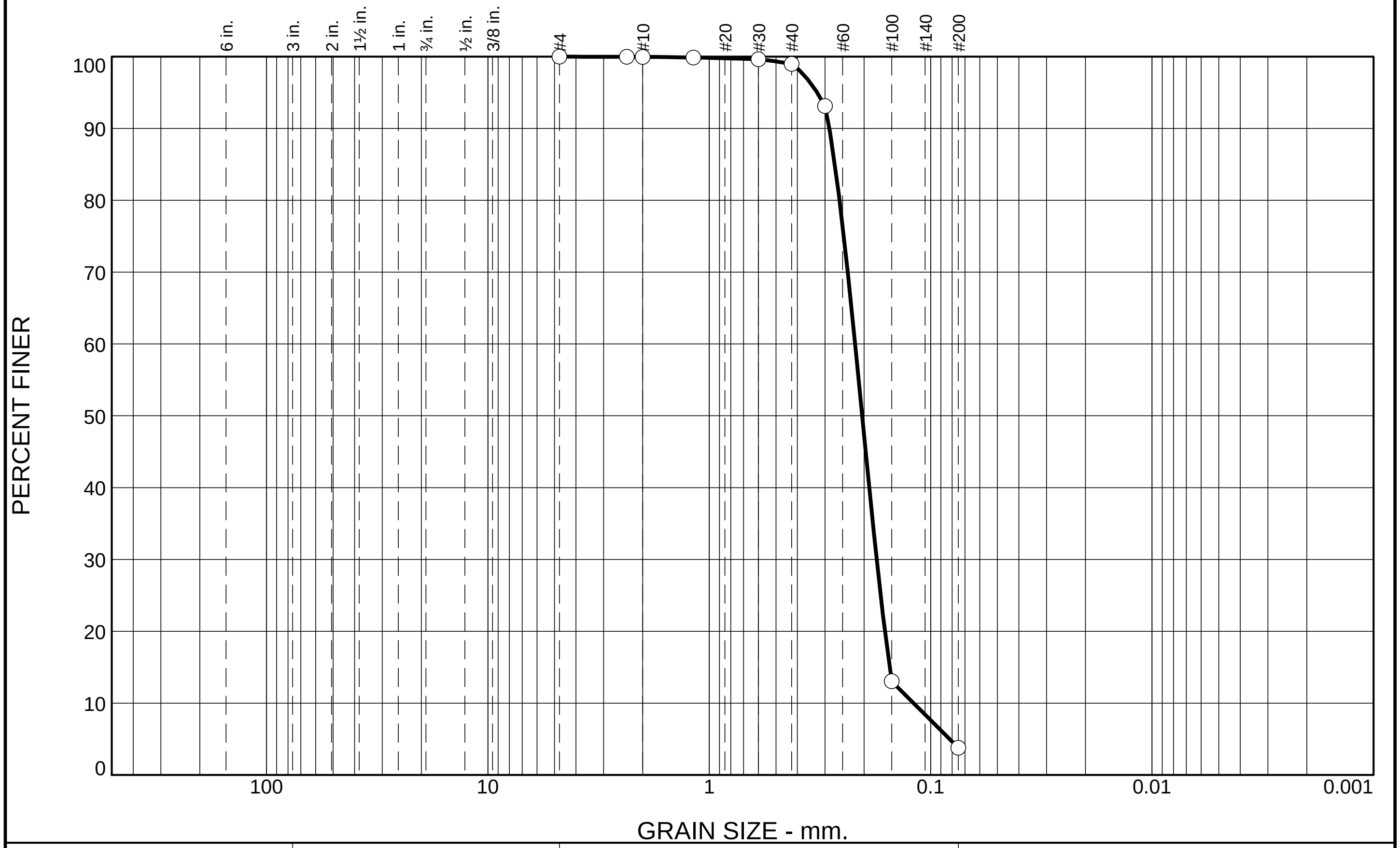
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: tw **Checked By:**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.0	95.2	3.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	100.0		
#10	100.0		
#16	99.9		
#30	99.6		
#40	99.0		
#50	93.1		
#100	13.0		
#200	3.8		

* (no specification provided)

Soil Description

Gray fine sand (SP)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2868 D_{85}= 0.2708 D_{60}= 0.2195
D_{50}= 0.2039 D_{30}= 0.1754 D_{15}= 0.1533
D_{10}= 0.1195 C_u= 1.84 C_c= 1.17

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S1-2 **Depth:** 23.5'
Sample Number: 8
Date: 09-14-20

BURNS COOLEY DENNIS, INC.

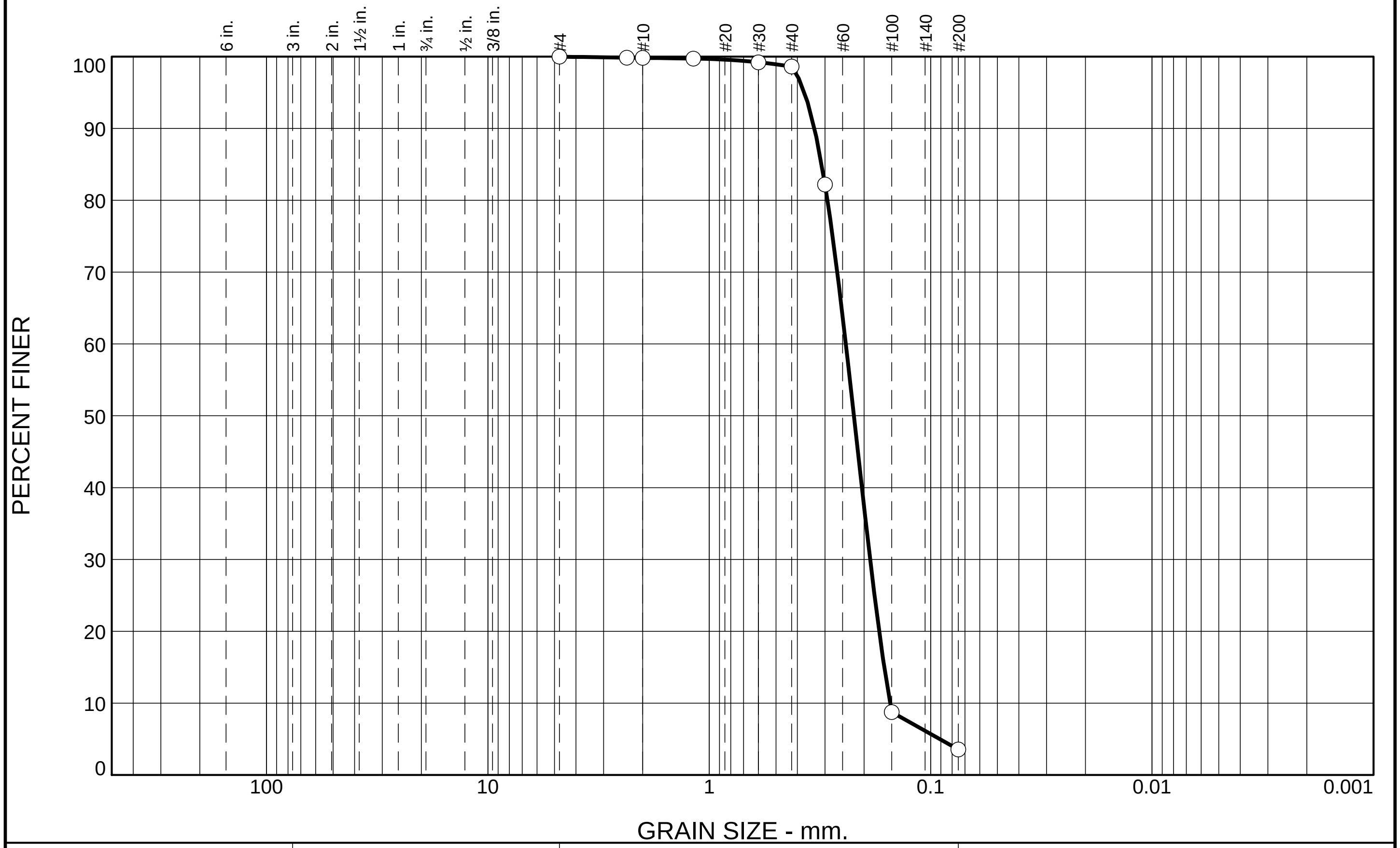
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: tw **Checked By:**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.2	1.2	95.1	3.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#10	99.8		
#16	99.7		
#30	99.2		
#40	98.6		
#50	82.2		
#100	8.7		
#200	3.5		

* (no specification provided)

Soil Description

Gray fine sand (SP)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3352 D_{85}= 0.3109 D_{60}= 0.2417
D_{50}= 0.2222 D_{30}= 0.1877 D_{15}= 0.1622
D_{10}= 0.1526 C_u= 1.58 C_c= 0.96

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S1-3 **Depth:** 43.5'
Sample Number: 11

Date: 09-14-20

BURNS COOLEY DENNIS, INC.

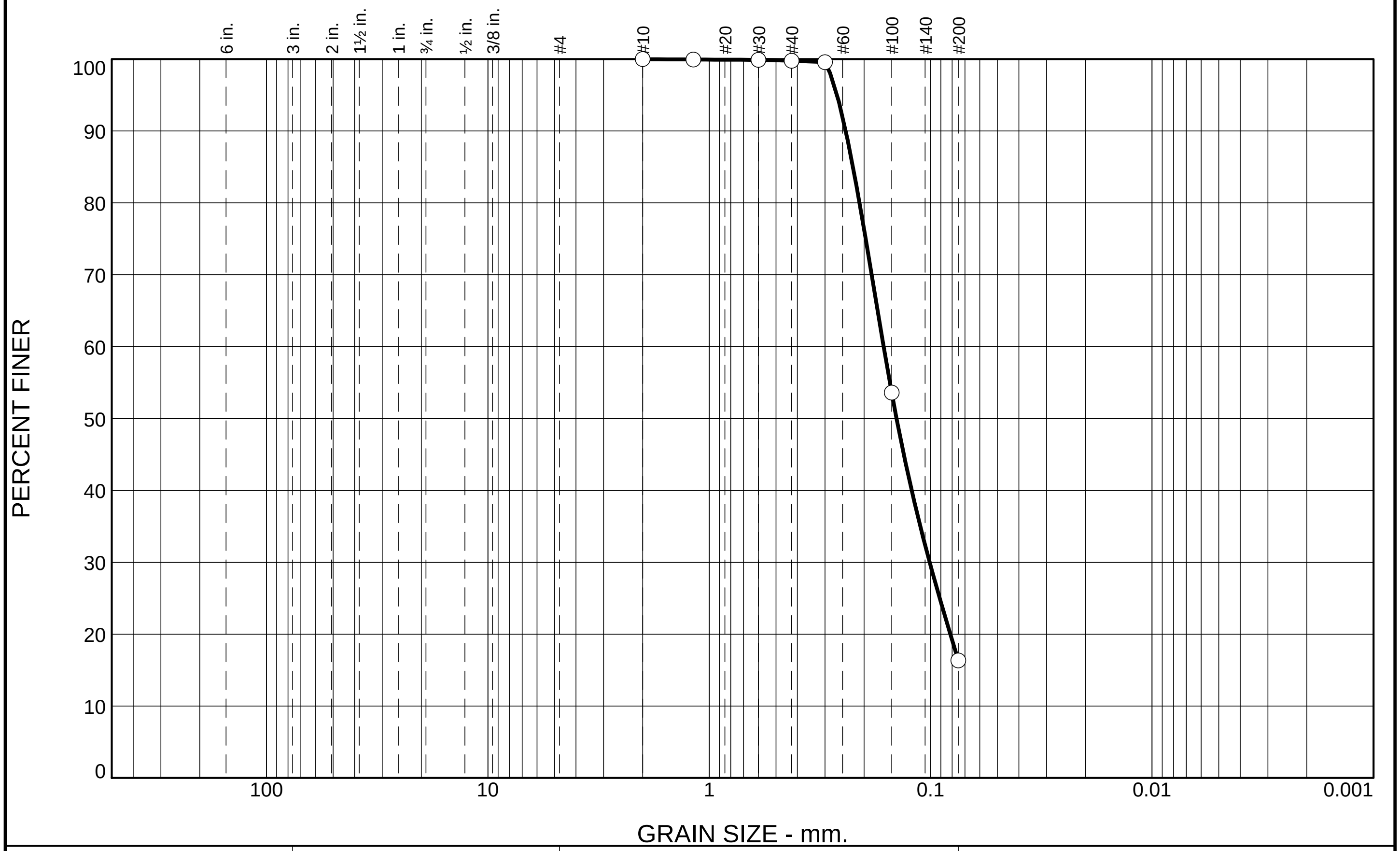
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: tw **Checked By:** ____________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.2	83.5	16.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.9		
#30	99.9		
#40	99.8		
#50	99.6		
#100	53.6		
#200	16.3		

* (no specification provided)

Soil Description

Gray silty fine sand (SM), slightly clayey

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2419 D_{85}= 0.2244 D_{60}= 0.1632
D_{50}= 0.1426 D_{30}= 0.1010 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: S1-3 **Depth:** 23.5'
Sample Number: 7
Date: 09-14-20

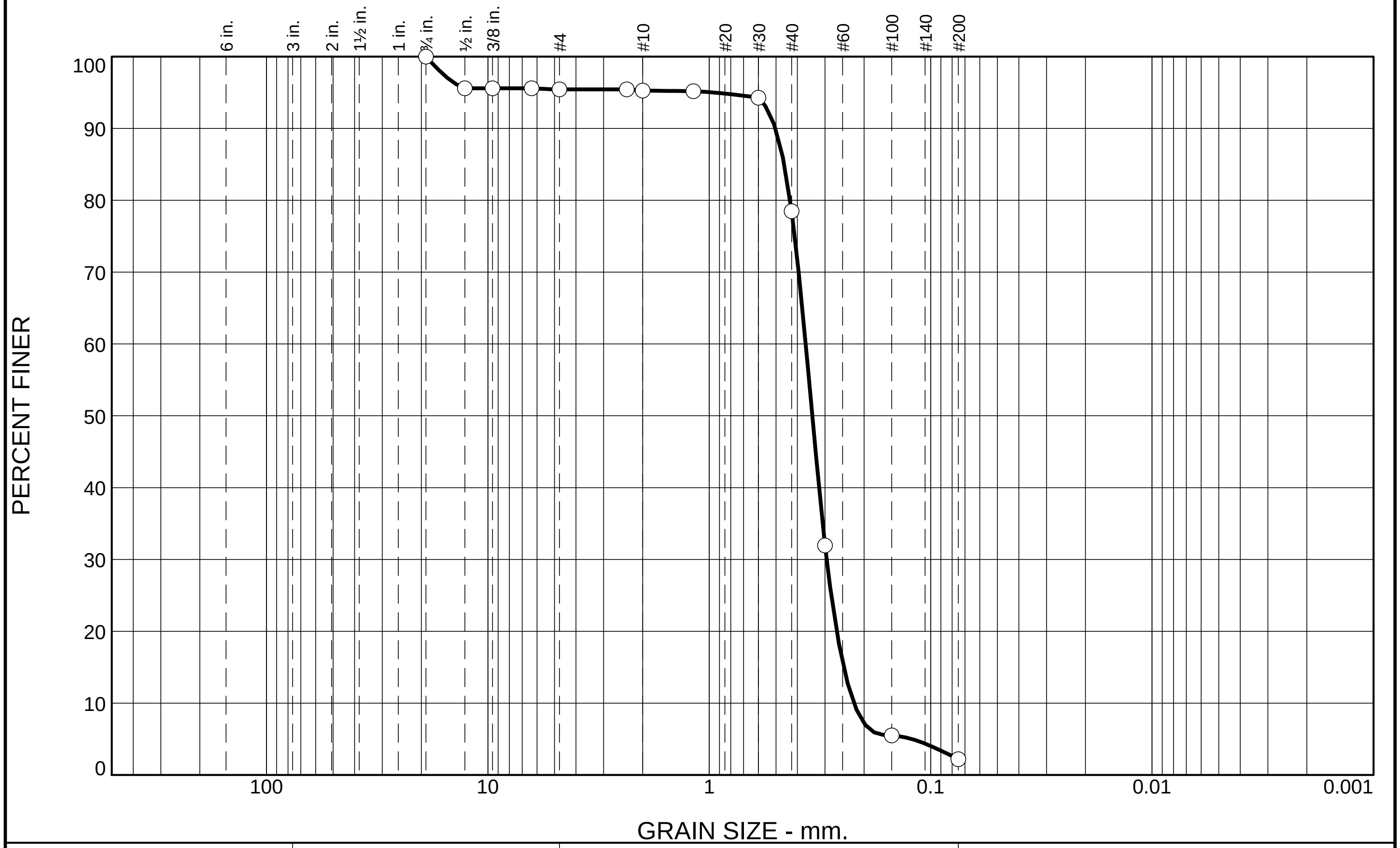
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: tw **Checked By:** ______

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.6	0.1	16.8	76.3	2.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	95.6		
3/8	95.6		
1/4	95.6		
#4	95.4		
#8	95.4		
#10	95.3		
#16	95.2		
#30	94.3		
#40	78.5		
#50	31.9		
#100	5.5		
#200	2.2		

* (no specification provided)

Soil Description

Gray fine to medium sand (SP) with trace of gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5025	D_{85}= 0.4588	D_{60}= 0.3672
D_{50}= 0.3428	D_{30}= 0.2950	D_{15}= 0.2472
D_{10}= 0.2223	C_u= 1.65	C_c= 1.07

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S1-4 **Depth:** 48.5'
Sample Number: 12

Date: 09-14-20

BURNS COOLEY DENNIS, INC.

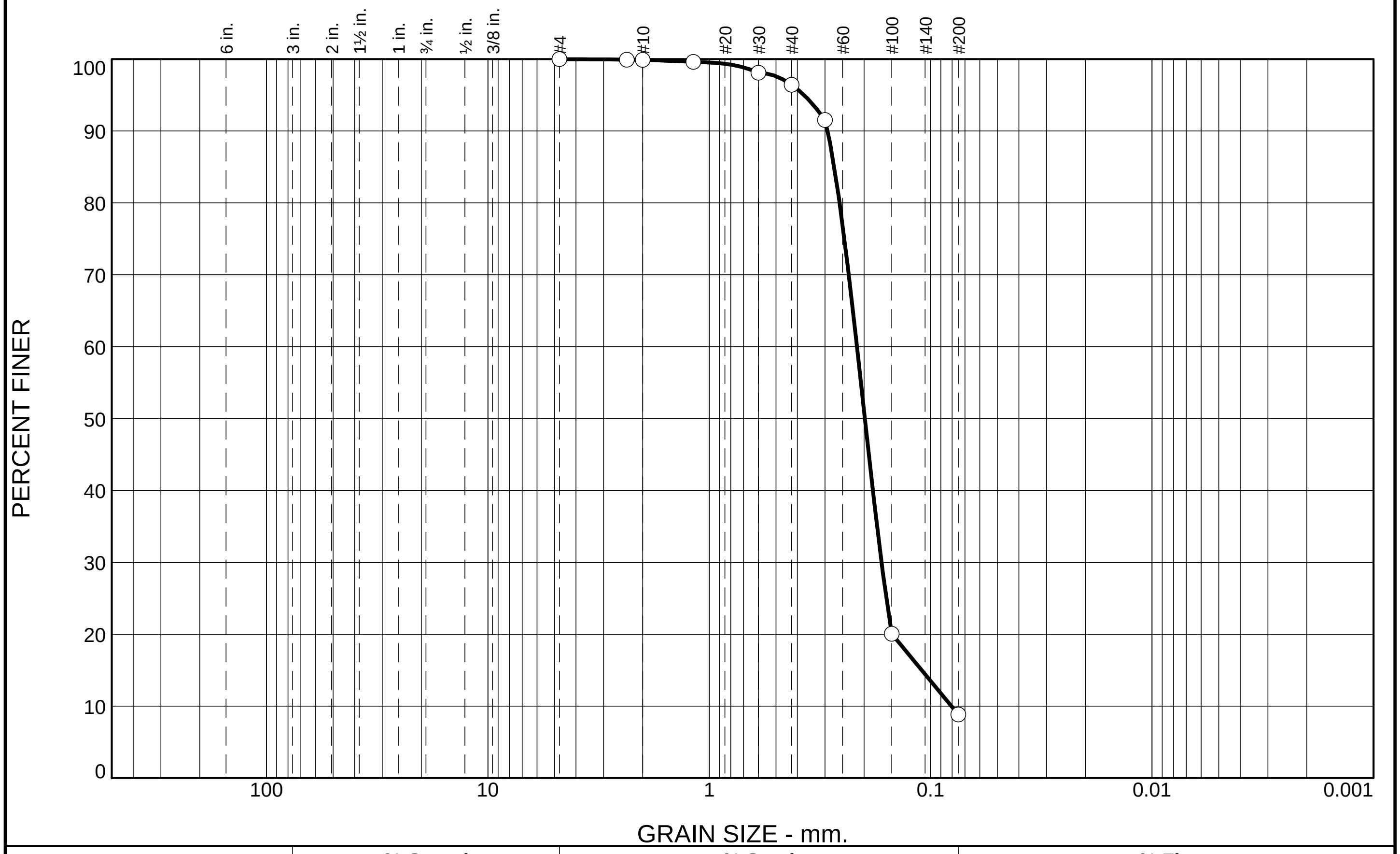
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: gw **Checked By:** ____________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	3.5	87.6	8.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#10	99.9		
#16	99.6		
#30	98.1		
#40	96.4		
#50	91.5		
#100	20.1		
#200	8.8		

* (no specification provided)

Soil Description

Gray and light gray fine sand (SP-SM), slightly silty

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2921 D_{85}= 0.2726 D_{60}= 0.2153
D_{50}= 0.1983 D_{30}= 0.1669 D_{15}= 0.1098
D_{10}= 0.0806 C_u= 2.67 C_c= 1.60

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: S1-4 **Depth:** 23.5'
Sample Number: 7

Date: 09-14-20

BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: tw **Checked By:**

APPENDIX B

Liquefaction Triggering Workbook

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]
Input Data Worksheet

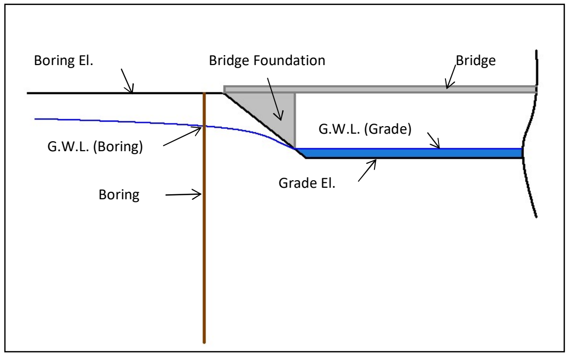
Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.89425	-91.07298
Logged By :	Christian Jackson	
Boring No:	S1-1	
Date:	20-Aug-20	
Type of Drilling:	HSA to 55', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: Date:
Checked by: Date:

All cells highlighted in this color should be input
All other cells should not be altered!

Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.369	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	249	ft
Ground Water Level (depth below boring surface) =	7	ft
Grade Surface Elevation =	236	ft
Ground Water Level (depth below or above grade surface) =	-6	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.



		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	246.5	2.5	CH	6						
2	244	5	CH	1			14	67		
3	239	10	CL	1	90.5		15	38	27	
4	234	15	CL	1			13	41		
5	229	20	SP	1	1.5					
6	224	25	SP	13	4.9					
7	219	30	SP	9	3.6					
8	214	35	SP	20	2.2					
9	209	40	SP	13	1.1					
10	204	45	SP	15	2.3					
11	199	50	SP	30	4.7					
12	194	55	SP	29	3.4					
13	189	60	SP	15	1.9					
14	184	65	SP	18	2.4					
15	179	70	SP	29	4.1					
16	174	75	SP	10	2.5					
17	169	80	SP	14	2.6					
18	164	85	SP	12	4.8					
19	159	90	SP	45	4.5					
20	154	95	SP	22	2.4					
21	149	100	SP	19	2.7					

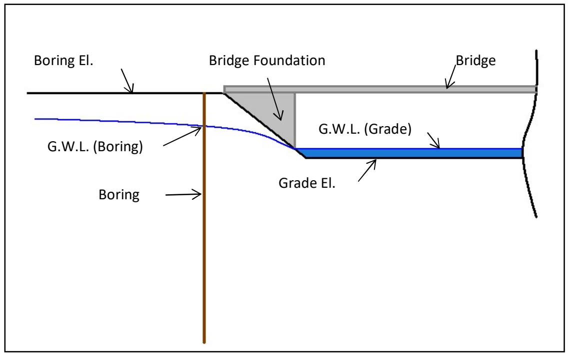
SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]
Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.89423	-91.07278
Logged By :	Christian Jackson	
Boring No:	S1-2	
Date:	24-Aug-20	
Type of Drilling:	HSA to 45', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by:		Date:	
Checked by:		Date:	

All cells highlighted in this color should be input

All other cells should not be altered!



Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.369	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	249	ft
Ground Water Level (depth below boring surface) =	7	ft
Grade Surface Elevation =	236	ft
Ground Water Level (depth below or above grade surface) =	-6	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.

		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	246.5	2.5	CL	5	68.4		14	41	21	
2	244	5	CL	1		127.7	13	33	24	1310
3	239	10	CL	11	86.8				33.6	
4	234	15	CH	1		116.3	18	74	25	770
5	229	20	CH	1		121	18	62	30.1	960
6	224	25	SP	8	3.8					
7	219	30	SM	4	24.8					
8	214	35	SP-SM	26	5.8					
9	209	40	SP	32	2.3					
10	204	45	SP	18	2.2					
11	199	50	SP	37	3.9					
12	194	55	SP	16	2.3					
13	189	60	SP	27	3.2					
14	184	65	SP	19	3.7					
15	179	70	SP-SM	41	5.3					
16	174	75	SP	9	2.5					
17	169	80	SP	13	2.5					
18	164	85	SP	14	3.3					
19	159	90	SP	49	3.4					
20	154	95	SP	11	2.5					
21	149	100	SP	35	3.5					

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]

Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.89421	-91.07223
Logged By :	Christian Jackson	
Boring No:	S1-3	
Date:	26-Aug-20	
Type of Drilling:	HSA to 30', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: ______ Date: ______

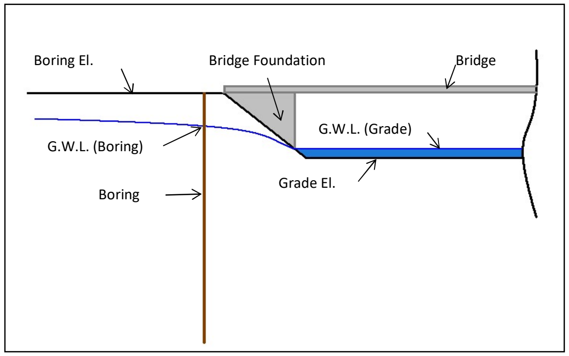
Checked by: ______ Date: ______

All cells highlighted in this color should be input

All other cells should not be altered!

Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.369	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	249	ft
Ground Water Level (depth below boring surface) =	7	ft
Grade Surface Elevation =	236	ft
Ground Water Level (depth below or above grade surface) =	-6	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.



		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	246.5	2.5	SC	7	42.6		14	38	15	
2	244	5	CL	1			17	36	12.8	
3	239	10	CL	1	81.4	125	15	41	30.2	260
4	234	15	CL	1			16	39	33.6	
5	229	20	SP-SM	1	5.9					
6	224	25	SM	5	16.3					
7	219	30	SP-SM	19	6.4					
8	214	35	SP-SM	25	7.4					
9	209	40	SP	25	2.6					
10	204	45	SP	13	3.5					
11	199	50	SP	16	4.6					
12	194	55	SP	19	3.1					
13	189	60	SP	15	3.1					
14	184	65	SP	8	1.8					
15	179	70	SP	48	4					
16	174	75	SP	21	4.2					
17	169	80	SP	15	4.5					
18	164	85	SP	23	4.4					
19	159	90	SP-SM	10	5.7					
20	154	95	SP	23	4.5					
21	149	100	SP	27	2.8					

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]

Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.89421	-91.07189
Logged By :	Christian Jackson	
Boring No:	S1-4	
Date:	26-Aug-20	
Type of Drilling:	HSA to 50', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: ____ Date: ____

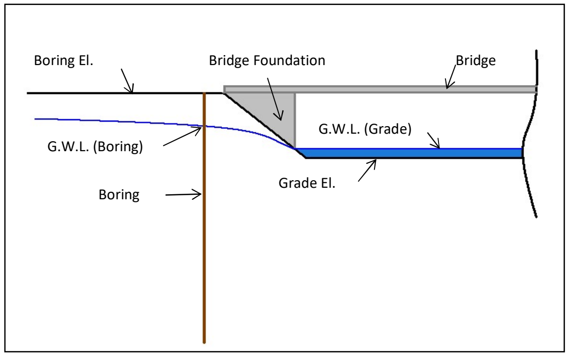
Checked by: ____ Date: ____

All cells highlighted in this color should be input

All other cells should not be altered!

Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.369	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	249	ft
Ground Water Level (depth below boring surface) =	7	ft
Grade Surface Elevation =	236	ft
Ground Water Level (depth below or above grade surface) =	-6	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.



		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	246.5	2.5	CL	12	54.6		15	37	15.4	
2	244	5	CL	1		128.9	14	37	15.1	1140
3	239	10	CL	1	84.7	128.9	15	35	20.5	1310
4	234	15	CL	1		126.1	16	37	27.4	1130
5	229	20	SP-SM	1	8.1					
6	224	25	SP-SM	5	8.8					
7	219	30	SP	38	4.5					
8	214	35	SP	9	0					
9	209	40	SP	18	3.3					
10	204	45	SP	13	3.6					
11	199	50	SP	7	2.2					
12	194	55	SP	11	2.4					
13	189	60	SP	16	2.9					
14	184	65	SP	29	3.7					
15	179	70	SP	19	3.7					
16	174	75	SP	30	7.5					
17	169	80	SP	17	2.8					
18	164	85	SP	17	2.8					
19	159	90	SP	12	1.7					
20	154	95	SP	25	3					
21	149	100	SP	32	4.4					

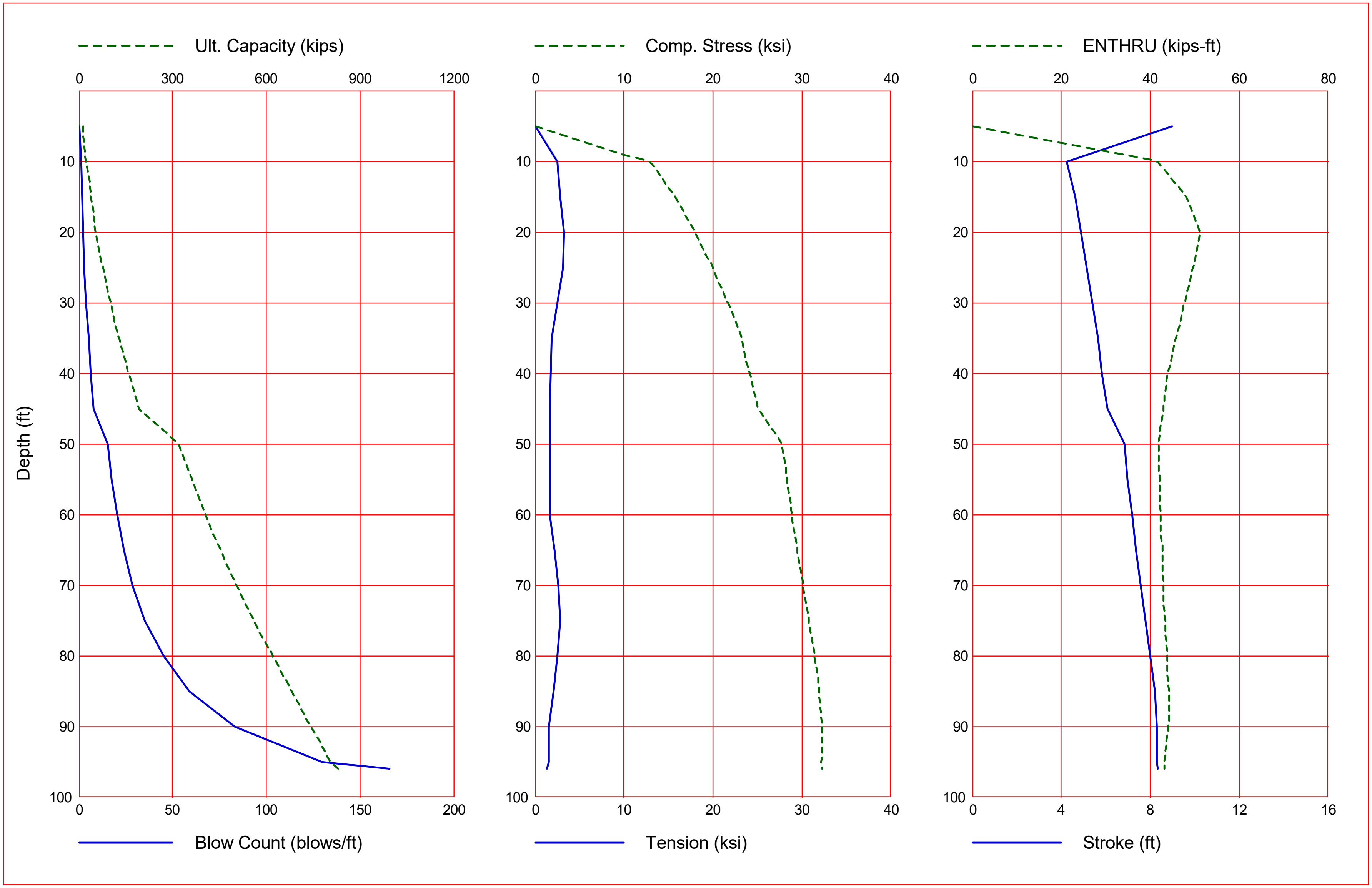
APPENDIX C

Pile Drivability Analysis Results

Burns Cooley Dennis, Inc.
Site 1- West Abutment - DELMAG D36

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

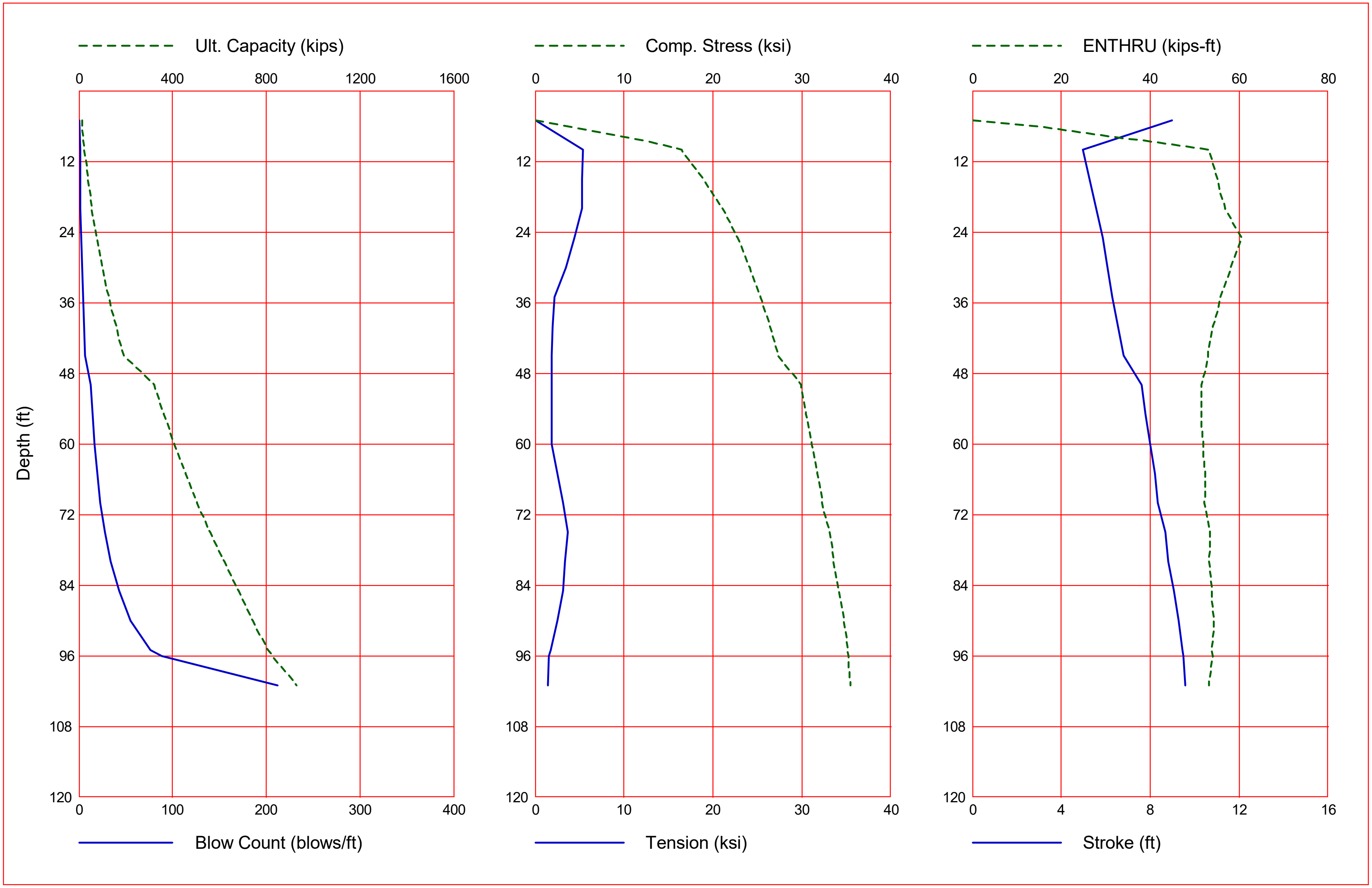
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	14.5	3.9	10.6	0.0	0.000	0.000	9.00	0.0
10.0	23.4	12.8	10.6	1.5	12.914	-2.557	4.23	41.8
15.0	38.4	26.5	11.9	1.8	15.696	-2.787	4.61	48.1
20.0	54.6	40.8	13.8	2.1	17.932	-3.280	4.87	51.4
25.0	76.2	58.5	17.7	2.7	20.027	-3.156	5.14	49.7
30.0	100.4	80.1	20.3	3.7	21.683	-2.544	5.40	47.7
35.0	128.6	105.6	23.0	5.1	23.176	-1.817	5.65	45.8
40.0	158.0	135.0	23.0	6.4	24.128	-1.746	5.82	43.9
45.0	191.4	168.4	23.0	8.0	25.149	-1.612	6.07	43.0
50.0	320.5	205.7	114.9	15.4	27.736	-1.687	6.84	42.0
55.0	361.7	246.9	114.9	17.5	28.311	-1.621	6.99	42.2
60.0	406.9	292.0	114.9	20.3	28.904	-1.652	7.18	42.4
65.0	455.0	340.1	114.9	23.9	29.488	-2.134	7.36	42.7
70.0	506.0	391.1	114.9	28.6	30.133	-2.624	7.57	43.0
75.0	560.9	446.1	114.9	35.4	30.744	-2.826	7.78	43.5
80.0	619.8	505.0	114.9	45.2	31.384	-2.467	8.00	43.9
85.0	680.7	565.8	114.9	58.9	31.968	-2.065	8.20	44.2
90.0	742.5	627.6	114.9	83.1	32.284	-1.581	8.31	44.0
95.0	806.3	691.4	114.9	130.1	32.193	-1.555	8.29	43.2
96.0	831.6	704.4	127.2	166.0	32.263	-1.281	8.34	43.1

Total Continuous Driving Time 54.00 minutes; Total Number of Blows 2272 (starting at penetration 5.0 ft)

Burns Cooley Dennis, Inc.
Site 1- East Abutment - DELMAG D36

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Burns Cooley Dennis, Inc.
Site 1- East Abutment - DELMAG D36

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

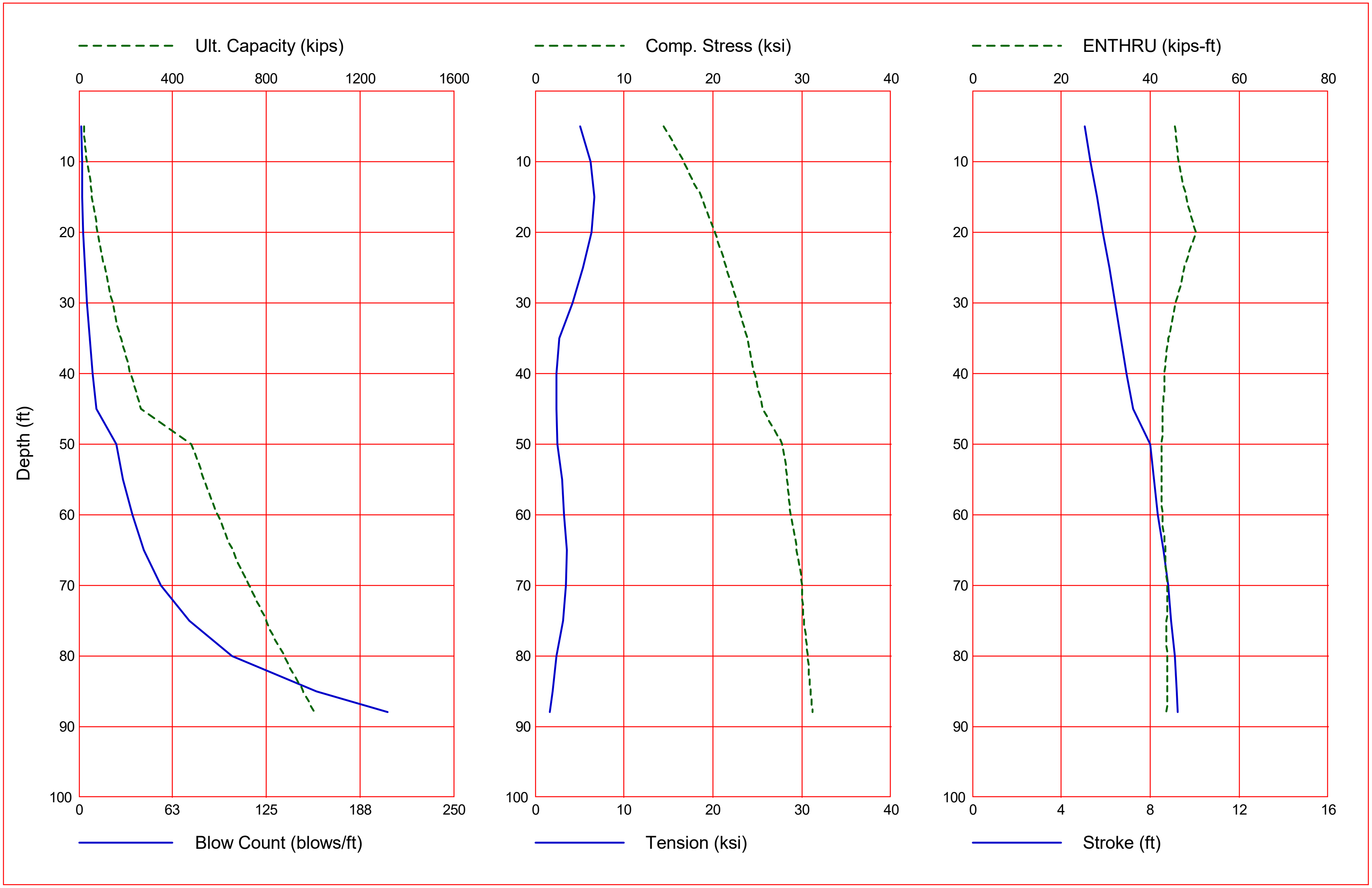
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	14.5	3.9	10.6	0.0	0.000	0.000	9.00	0.0
10.0	23.4	12.8	10.6	1.5	16.514	-5.346	4.99	53.2
15.0	38.4	26.5	11.9	1.7	18.929	-5.307	5.29	55.2
20.0	54.6	40.8	13.8	2.0	21.062	-5.249	5.58	56.9
25.0	76.2	58.5	17.7	2.5	22.751	-4.429	5.85	60.5
30.0	100.4	80.1	20.3	3.2	24.122	-3.464	6.10	58.2
35.0	128.6	105.6	23.0	4.4	25.375	-2.217	6.30	55.8
40.0	158.0	135.0	23.0	5.5	26.447	-1.967	6.54	54.1
45.0	191.4	168.4	23.0	6.8	27.398	-1.857	6.79	53.1
50.0	320.5	205.7	114.9	12.9	29.947	-1.877	7.62	51.5
55.0	361.7	246.9	114.9	14.6	30.487	-1.862	7.79	51.5
60.0	406.9	292.0	114.9	16.7	31.097	-1.888	7.98	52.0
65.0	455.0	340.1	114.9	19.4	31.763	-2.505	8.20	52.4
70.0	506.0	391.1	114.9	23.0	32.224	-3.160	8.36	52.2
75.0	560.9	446.1	114.9	27.4	33.116	-3.728	8.67	53.4
80.0	619.8	505.0	114.9	34.4	33.538	-3.409	8.83	53.3
85.0	680.7	565.8	114.9	43.3	34.206	-3.109	9.07	53.9
90.0	742.5	627.6	114.9	55.1	34.771	-2.511	9.30	54.2
95.0	806.3	691.4	114.9	76.6	35.137	-1.748	9.43	53.9
96.0	828.1	704.4	123.7	88.7	35.242	-1.557	9.48	54.0
101.0	929.6	770.5	159.0	212.6	35.430	-1.493	9.59	53.3

Total Continuous Driving Time 60.00 minutes; Total Number of Blows 2399 (starting at penetration 5.0 ft)

Burns Cooley Dennis, Inc.
Site 1- Interior Bents - DELMAG D36

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Burns Cooley Dennis, Inc. Feb 19 2021
Site 1- Interior Bents - DELMAG D36 GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	24.1	5.2	18.9	1.7	14.468	-5.078	5.07	45.6
10.0	35.9	17.0	18.9	1.9	16.643	-6.206	5.32	46.5
15.0	56.4	35.3	21.1	2.3	18.744	-6.713	5.61	48.1
20.0	78.9	54.4	24.5	2.8	20.212	-6.373	5.86	50.2
25.0	109.4	78.0	31.4	4.0	21.525	-5.406	6.15	47.7
30.0	142.9	106.8	36.1	5.6	22.794	-4.238	6.42	45.7
35.0	181.6	140.8	40.8	7.4	23.876	-2.769	6.69	44.1
40.0	220.9	180.0	40.8	9.3	24.724	-2.358	6.93	43.2
45.0	265.4	224.5	40.8	11.7	25.649	-2.374	7.24	42.7
50.0	478.4	274.2	204.2	25.2	27.782	-2.522	7.99	42.5
55.0	533.4	329.2	204.2	29.7	28.286	-3.070	8.19	42.6
60.0	593.6	389.4	204.2	36.0	28.749	-3.228	8.33	42.8
65.0	657.7	453.5	204.2	43.4	29.366	-3.554	8.58	43.5
70.0	725.7	521.5	204.2	54.6	29.975	-3.461	8.79	43.9
75.0	799.0	594.8	204.2	73.6	30.232	-3.108	8.92	43.7
80.0	877.5	673.3	204.2	102.4	30.717	-2.423	9.09	43.9
85.0	958.6	754.4	204.2	158.6	31.005	-1.974	9.19	43.8
88.0	1007.8	803.6	204.2	205.7	31.167	-1.623	9.25	43.7

Total Continuous Driving Time 75.00 minutes; Total Number of Blows 2995 (starting at penetration 5.0 ft)

APPENDIX D

AHTD Special Provision for Embankment Construction

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

DESCRIPTION: This Special Provision shall be supplementary to Section 210, Excavation and Embankment, of the Standard Specifications, Edition of 2003 and shall apply to the construction of embankments being built over existing borrow ditches as shown in the plans or where directed by the Engineer.

MATERIALS: Stone Backfill shall meet the requirements of Section 207 of the Standard Specifications, Edition of 2003.

Select Material (Class SM-2) shall meet the requirements of Section 302 of the Standard Specifications, Edition of 2003.

Dumped Riprap and Filter Blanket shall comply with Section 816 of the Standard Specifications except that synthetic geotextile fabric complying with requirements of Subsection 625.02, Type 5 must be used as a filter blanket under dumped riprap in lieu of a granular filter blanket material.

Clay plating shall consist of material having a minimum plasticity index of 10 and a maximum plasticity index of 25, which will support vegetation and not be highly susceptible to erosion.

CONSTRUCTION: When the embankment is to be built over existing borrow ditches, the ditches shall be undercut 2 feet below the existing flow line to remove all highly organic, wet material prior to embankment construction. The ditches shall then be filled using Stone Backfill. The top 4" to 6" of Stone Backfill shall be material complying with Section 303 of the Standard Specifications, Edition of 2003 for Class 7 Aggregate Base Course in accordance with Section 207. Excavation for the placement of Stone Backfill shall be considered part of the item in accordance with subsection 207.01 of the Standard Specifications.

The remaining embankment shall be constructed of Selected Material (Class SM-2). Synthetic Filter Blanket and Dumped Riprap shall be placed on the slopes of embankments constructed of Select Material (Class SM-2) from the top of the Stone Backfill to 2 feet above the high water elevation or as directed by the Engineer. The remainder of embankments constructed of Select Material (Class SM-2) or other material which is susceptible to erosion shall have a minimum 18 inch clay plating (measured

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

perpendicular to the finished slopes).

All embankment materials, including Selected Material (Class SM-2) and Clay Plating, shall be placed and compacted in accordance with Subsections 210.07, 210.09, and 210.10 of the Standard Specifications.

QUALTIY CONTROL AND ACCEPTANCE: The Contractor shall perform quality control and acceptance sampling and testing of the clay plating for plasticity index; Selected Material (Class SM-2) for gradation and plasticity index in accordance with Section 306 except that the size of the standard lot will be 3000 cubic yards. The Contractor shall perform quality control and acceptance sampling and testing of the Selected Material (Class SM-2) for density and moisture content in accordance with Subsection 210.02 of the Standard Specifications for Highway Construction. Selected Material (Class SM-2) shall meet the density requirements of Subsection 210.10.

METHOD OF MEASUREMENT: Embankments consisting of Selected Material (Class SM-2) and Clay Plating material and as shown on the plans, will be measured as Compacted Embankment in accordance with Subsection 210.12 of the Standard Specifications.

Stone Backfill will be measured in accordance with Section 207 of the Standard Specifications.

Filter Blanket and Dumped Riprap will be measured in accordance with Section 816 of the Standard Specifications.

BASIS OF PAYMENT: All accepted embankments; including Selected Material (Class SM-2) and Clay Plating material measured as provided above will be paid for as Compacted Embankment in accordance with Subsection 210.13 of the Standard Specifications.

Stone Backfill shall be paid in accordance with Section 207 of the Standard Specifications.

Filter Blanket and Dumped Riprap will be paid in accordance with Section 816 of the Standard Specifications.

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

Payment will be made under:

Pay Item	Pay Unit
Compacted Embankment	Cubic Yard
Stone Backfill	Ton
Filter Blanket	Square Yard
Dumped Riprap	Cubic Yard

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

Corporate Office
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Ridgeland, MS 39157
Phone: (601) 856-9911
Fax: (601) 853-2077

Mailing Address
Post Office Box 12828
Jackson, MS 39236

www.bcdgeo.com

Materials Laboratory
278 Commerce Park Drive
Ridgeland, MS 39157
Phone: (601) 856-2332
Fax: (601) 856-3552

March 16, 2021

Cindy Rich, P.E.
Neel-Schaffer, Inc.
125 South Congress Street, Suite 1100
Post Office Box 22625
Jackson, Mississippi 39201

Report No. 200518 – Site 2

Geotechnical Exploration
Site 2
ARDOT SR230 Bridge Replacements
Craighead and Lawrence Counties, Arkansas

Dear Ms. Rich:

Submitted here is the report of our geotechnical exploration for the above-captioned project. This exploration was authorized by Task Order 108 to the Subconsultant Agreement between Neel-Schaffer, Inc. and Burns Cooley Dennis, Inc. dated September 17, 2020.

We appreciate the opportunity to be of service. If you should have any questions concerning this report, please do not hesitate to call us.

Very truly yours,

BURNS COOLEY DENNIS, INC.

Alexander B. Reeb, Ph.D., P.E.

A. E. (Eddie) Templeton, P.E.

ABR/AET/khb
Copy Submitted: (via e-mail)

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 Project Description

Plans are being made for the construction of replacement bridges and box culverts at ten sites along Highway 230 between Alicia and Bono in Craighead and Lawrence Counties, Arkansas. Site 2 is located in Lawrence County where Highway 230 crosses Lick Pond Slough. At this site, a new bridge will be constructed on a new alignment just north of the existing bridge.

The new bridge will be 120 ft long and consist of three spans of approximately equal spacing. It is our understanding that new fill will be placed to raise the grade at the new abutments above the grade of the existing bridge. The abutment spill-through slopes will be constructed as 2H:1V slopes, and the abutment side slopes will be constructed as 3H:1V slopes. The abutment bents are to be supported by 18-in. diameter, closed-ended steel pipe piles, and the interior bents are to be supported by 24-in. diameter, closed-ended steel pipe piles. A preliminary layout showing the proposed construction is presented on Figure 1 of this report.

1.2 Purposes

The specific purposes of this exploration were:

1) to review the exploratory soil borings made within the area planned for construction of the new bridge;
2) to verify field classifications and to evaluate pertinent physical properties of the soils encountered in the borings by means of visual examination of the soil samples in the laboratory and routine tests performed on the samples;
3) to perform analyses to investigate liquefaction, slope stability, settlement, pile capacity, and downdrag; and
4) to provide geotechnical recommendations for design and construction of the bridge.

Our scope of work for the bridge does not include providing recommendations for roadway subgrades and pavements. Discussion and recommendations pertaining to roadway subgrades and pavements are provided under separate cover.

2.0 FIELD EXPLORATION

2.1 General

Subsurface soil conditions within the area planned for construction of the bridge were explored by means of four deep borings. Borings S2-1, S2-2, S2-3, and S2-4 were performed by McCray Drilling under contract to SoilTech Consultants, Inc. The approximate locations of the borings are shown on Figure 1.

All soils were classified in general accordance with the Unified Soil Classification System. A synopsis of the Unified Soil Classification System (USCS) is presented on Figure 2 along with symbols and terminology typically utilized on graphical soil boring logs. Graphical logs of the borings are presented on Figures 3 through 6. The graphical logs illustrate the types of soil and stratification encountered with depth below the existing ground surface at the individual boring locations. Approximate GPS coordinates for the boring locations are shown at the bottom of the graphical boring logs within the “Comments” section.

2.2 Drilling Methods and Groundwater Observations

Borings S2-1, S2-2, and S2-3 were made to an exploration depth of 100 ft and boring S2-4 was made to an exploration depth of 101.5 ft. The borings were made using a CME-750X buggy-mounted drill rig. Borings S2-1, S2-2, S2-3, and S2-4 were initially advanced to a depth of 28 ft, 35 ft, 25 ft and 50 ft, respectively, by dry augering and then were extended to completion using rotary wash drilling procedures. Groundwater was encountered at a depth of 24 ft, 3 ft, 12.5 ft, and 48 ft in Borings S2-1, S2-2, S2-3, and S2-4, respectively.

2.3 Sampling Methods

Disturbed samples of soils were obtained by driving a standard 2-in. OD split-spoon sampler 18 in. into the soil with a 140-lb hammer falling freely a distance of 30 in. The depths at which the split-spoon samples were taken are illustrated as crossed rectangular symbols under the "Samples" column of the graphic logs. Standard penetration test (SPT) blow counts resulting from split-spoon sampling are recorded under the "Blows Per Ft" column of the graphic logs. Where the full penetration of the sampler occurred under merely the weight of the sampler and sampling rod alone, the abbreviation “WOH” is recorded in the column. The SPT blow counts are the “raw” field values. The recommended hammer energy correction factor is indicated in the “Comments” section of the logs. Relatively undisturbed samples of the soils encountered in the borings were obtained by pushing a 3-in. OD Shelby tube sampler approximately 2 ft into the soil. The Shelby tube samples were obtained within the depth intervals illustrated as shaded portions of the "Samples" column of the graphic logs. The Shelby tube and/or split-spoon samples were generally obtained at approximate 3-ft to 5-ft intervals of depth. Disturbed auger cutting samples were taken near the ground surface in the borings. The depths at which the auger cutting samples were taken are illustrated as small I-shaped symbols under the "Samples" column of the graphic boring logs.

2.4 Field Classification, Sample Preservation and Borehole Abandonment

All soils encountered during drilling were examined and classified in the field by a geotechnical engineering technician. Representative portions of the split-spoon samples and the auger cutting samples were sealed in jars to provide material for visual examination and testing in the laboratory. The Shelby tubes were capped and the ends sealed with wax in the field to prevent moisture loss and structural disturbance while they were transported to the testing laboratory. At the testing laboratory, the Shelby tube samples were extruded, and an approximate 6-in. long portion of each sample was temporarily sealed in plastic wrap to prevent moisture loss during the period between sample extrusion and testing. Additional portions of each Shelby tube sample were sealed in jars to provide additional material for visual examination and testing. The boreholes were grouted after completion of drilling and sampling.

3.0 LABORATORY TESTING

3.1 General

All of the soil samples were examined in the laboratory and tests were performed on selected samples to verify field classifications and to assist in evaluating the strength and volume change properties of the soils encountered. The types of laboratory tests performed are described in the following paragraphs.

3.2 Strength Properties

The undrained shear strength characteristics of the fine-grained soils encountered in the borings were investigated by means of visual estimates of consistency and from the results of unconfined compression tests and unconsolidated undrained (UU) triaxial compression tests performed on selected undisturbed Shelby tube samples. The results of the unconfined compression tests in terms of cohesion are plotted as small open circles in the data sections of the graphic logs. The cohesions resulting from the UU triaxial compressions test are plotted as small open triangles in the data section of the graphic boring logs. The water content and dry density were also determined for each unconfined and UU triaxial compression test specimen. The water contents are plotted as small shaded circles in the data section of the graphic logs. The dry densities are tabulated to the nearest lb per cu ft under the "Dry Density" column of the graphic boring logs.

3.3 Consolidation Tests

The compressibility characteristics of the fine-grained soils encountered in the borings were investigated by means of a one-dimensional consolidation test performed on a representative undisturbed Shelby tube sample. The results of the consolidation test, including a plot of void ratio versus effective vertical stress, are presented in Appendix A.

3.4 Classification Tests

The classifications and volume change properties of the fine-grained soils encountered in the borings were investigated by means of Atterberg liquid and plastic limit tests performed on selected representative samples. The results of the liquid and plastic limit tests are plotted as small crosses interconnected by dashed lines in the data section of the graphic boring logs. In accordance with the Unified Soil Classification System, fine-grained soils are classified as either clays or silts of low or high plasticity based on the results of Atterberg limit tests. The numerical difference between the liquid limit and plastic limit is defined as the plasticity index (PI). The magnitudes of the liquid limit and plasticity index and the proximity of the natural water content to the plastic limit are indicators of the potential for a fine-grained soil to shrink or swell upon changes in moisture content or to consolidate under loading. The proximity of the natural water content to the plastic limit is also an indicator of soil strength.

The classifications of some samples were investigated by means of minus No. 200 sieve tests. The percentages of fines resulting from the minus No. 200 sieve tests are tabulated at the appropriate depths under the "% Passing No. 200 Sieve" column of the graphic boring logs.

The classifications of some samples were investigated by means of sieve and hydrometer analyses. Particle size distribution curves from these tests are presented in Appendix A. The percentages of fines resulting from the sieve tests are also tabulated at the appropriate depths under the "% Passing No. 200 Sieve" column of the graphic boring logs

3.5 Water Content Tests

Water content tests were performed on samples to corroborate field classifications and to extend the usefulness of the strength, plasticity, and field SPT blow count data. The results of the water content tests are plotted as small shaded circles in the data section of the graphic boring logs. The water content data have been interconnected on the logs to illustrate a continuous profile with depth.

3.6 Soluble Sulfates, pH, and Resistivity Tests

Laboratory testing was performed on selected samples from the borings to determine the percent of soluble sulfate by mass, soil pH, and soil resistivity. Sulfate testing was performed on four samples, and soil pH and resistivity testing was performed on a separate set of four samples. Results of the tests are presented in Table 1.

Table 1 - Soluble Sulfates, pH, and Resistivity Test Results

Boring	Sample Depth (ft)	USCS	Sulfate (SO_4), % by mass	Average pH	Resistance (ohm-cm)
S2-1	3	CL	-	7.77	1600
S2-1	23.5	ML	0.014	-	-
S2-2	4	CH	-	7.86	1300
S2-2	53.5	SP	0.014	-	-
S2-3	4	CL	-	6.59	1800
S2-3	38.5	SP-SM	0.019	-	-
S2-4	8	CL	-	6.85	2600
S2-4	14	CL-ML	0.011	-	-

4.0 GENERAL SUBSURFACE CONDITIONS

4.1 General

A general description of subsurface soil and groundwater conditions revealed by the borings made for this exploration is provided in the following paragraphs. The graphical logs shown on Figures 3 through 6 should be referred to for specific soil and groundwater conditions encountered at each boring location. Stick logs of the borings are shown in profile with the proposed bridge section on Figure 7 to aid in visualizing subsurface soil conditions. Tabulated adjacent to the stick logs are Atterberg liquid and plastic limits, water contents, dry densities, cohesions, percentages of fines passing the No. 200 sieve and field SPT blow counts.

4.2 Geology

The project site is located within the physiographic province known as the Mississippi River Alluvial Plain. Geological maps indicate Quaternary age deposits are continuous throughout the project area. The Quaternary deposits at the site include alluvial sediments from both the Holocene and Pleistocene series. Sediments typically include a substratum zone of sands and gravels overlain by a top stratum of clays and silts.

Tertiary deposits are present below the Quaternary deposits. Tertiary deposits within the project vicinity are expected to consist of hard clays, sandy clays and silty clays containing organics and lignite interbedded with very dense sand strata. Geological maps suggest that the elevation of top of the Tertiary deposits may be at about El 100 to 125 ft MSL.

4.3 Soil Stratification

As shown on the Figure 7 profile, the soils encountered at the site were grouped into the zones outlined below. The zones were generally based on the soil classifications and interpreted strengths used in design. The borings generally indicate fill materials and fine-grained top stratum soils overlying alluvial sands.

- Zone 1 – Medium stiff silty clay (CL) and stiff sandy clay (CL)
- Zone 2 – Very soft to soft sandy clay (CL), silty clay (CL) and clay (CH), slightly silty
- Zone 3 – Medium stiff sandy clay (CL), silty clay (CL) and very silty clay (CL-ML) and stiff clay (CH), slightly silty
- Zone 4 – Very loose silt (ML), medium dense silty sand (SM), loose to very dense sand (SP-SM), slightly silty, and loose to very dense sand (SP) with trace of gravel

Zones 1, 2 or 3 soils were encountered at the ground surface down to depths ranging from about 10 ft to 30 ft. Zone 4 soils were encountered beneath the Zone 3 soils and extend to the boring termination depths.

Zone 4 was further divided into Zones 4A, 4B, 4C and 4D based on the estimated likelihood of liquefaction and potential for strength loss due to an earthquake. The soils encountered in Zones 4A and 4C were generally identified as having a moderate likelihood of liquefaction but no significant strength loss.

We understand that new fill materials will be placed along the new alignment to create the approach embankments. The thickness of the proposed new fill at abutments along the bridge centerline is illustrated on the profile.

4.4 Groundwater

Groundwater was encountered during auger drilling at a depth of 24 ft, 3 ft, 12.5 ft, and 48 ft in Borings S2-1, S2-2, S2-3, and S2-4, respectively. Groundwater cannot be observed during rotary wash drilling. In our opinion, groundwater conditions at the site will be influenced by rainfall, surface drainage, and by the rise and fall of water levels in the nearby ditches, creeks, ponds or other bodies of water. The regional groundwater is primarily influenced by the Mississippi River. Groundwater conditions at the site can also be influenced by man-made changes. Surficial soils can become saturated and weak to relatively shallow depths during periods of prolonged and heavy rainfall.

5.0 ENGINEERING ANALYSES AND DISCUSSION

5.1 General

The purposes of this study were to perform analyses and develop geotechnical recommendations for: 1) seismic design including site classification, liquefaction, and seismic compression; 2) slope stability including proposed slope grading and configuration to provide acceptable factors of safety; and 3) deep foundation design including axial capacity curves, downdrag, lateral analysis parameters, and drivability analysis. A discussion of our analyses is provided in the following subsections.

5.2 Seismic

Seismic evaluations and analyses were generally performed based on the guidance provided by ARDOT and the recommendations discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual and in Idriss and Boulanger (2008).

5.2.1 Site Classification. Soil shear wave velocity data are not available for the bridge site. The site class was determined from SPT blow counts and undrained shear strength data in accordance with definitions provided in Table 3.10.3.1-1 of the AASHTO LRFD 2017 Bridge Design Specifications. We recommend that a site class E be utilized to determine the site coefficient and spectral response acceleration for this bridge site. The site is classified as within Seismic Zone 4 per Table 3.10.6 1.

The acceleration design response spectrum was developed using the computer program “AASHTO Seismic Design Parameters” version 2.10 developed by the U.S. Geological Survey.

The recommended design values are presented subsequently in tabular format. Plots of the design spectrum are included as Figures 8 and 9.

```
Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
  Latitude     =    35.894070
  Longitude  =   -91.069620
Site Class B
  Data are based on a 0.05 deg grid spacing.
     Period         Sa
      (sec)          (g)
        0.0          0.349     PGA - Site Class B
        0.2          0.669     Ss     - Site Class B
        1.0          0.172     S1    - Site Class B

Spectral Response Accelerations SDs and SD1
  As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class E  -  Fpga =  1.05,  Fa =  1.36,  Fv =  3.28
  Data are based on a 0.05 deg grid spacing.
     Period         Sa
      (sec)          (g)
        0.0          0.367     As   - Site Class E
        0.2          0.910     SDs - Site Class E
        1.0          0.563     SD1 - Site Class E: Seismic Zone 4
Data are based on a 0.05 deg grid spacing.
     Period         Sa         Sd
      (sec)          (g)          in.
      0.000         0.367     0.000     T = 0.0, Sa = As
      0.124         0.910     0.136
      0.200         0.910     0.356     T = 0.2, Sa = SDs
      0.619         0.910     3.403     T = Ts,  Sa = SDs
      0.700         0.804     3.850
      0.800         0.704     4.400
      1.000         0.563     5.501     T = 1.0, Sa = SD1
      1.200         0.469     6.601
      1.400         0.402     7.701
      1.600         0.352     8.801
      1.800         0.313     9.901
      2.000         0.282     11.001
      2.200         0.256     12.101
      2.400         0.235     13.201
      2.600         0.217     14.301
      2.800         0.201     15.402
      3.000         0.188     16.502
```

3.200	0.176	17.602
3.400	0.166	18.702
3.600	0.156	19.802
3.800	0.148	20.902
4.000	0.141	22.002

5.2.1 Liquefaction Triggering. Liquefaction triggering evaluations were performed using the Microsoft Excel workbook developed by Cox and Griffiths (2011)[1] and provided by ARDOT. The liquefaction evaluations were performed using all three procedures available in the workbook: Youd et al. (2001)[2], Cetin et al. (2004)[3], Idriss and Boulanger (2008)[4].

The design earthquake magnitude (M_w) was estimated using the Unified Hazard Tool on the U.S. Geological Survey (USGS) website. Deaggregations were computed using the 2008 (v3.3.3) edition of the National Seismic Hazard Mapping Project (NSHMP). A return period of 5% in 50 years (i.e., 975 years) was used in the deaggregation. The resulting modal earthquake magnitude of 7.7 was input in the liquefaction triggering workbook.

The liquefaction triggering evaluation was performed for each of the borings. The liquefaction triggering workbook input is provided for each boring in Appendix B. As recommended by Cox and Griffiths (2011), a blow count N-value of 1 was input in the workbook at sample depths where SPT blow counts were not measured. For these cases, the Factor of Safety (FS) against liquefaction was not calculated. Comparison plots that show the resulting liquefaction FS values vs. elevation for each of the three evaluation procedures are provided as Figures 10, 11, 12, and 13 for Borings S2-1, S2-2, S2-3, and S2-4, respectively.

5.2.2 Seismic Compression. Potential seismic compression was calculated for all soil layers that were identified as likely to liquefy (i.e., FS<=1.0) based on the Idriss and Boulanger (2008) liquefaction triggering criteria. The seismic compression calculations were performed

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[1] Cox, B. R., and Griffiths, S. C. (2011). *Practical Recommendations for Evaluation and Mitigation of Soil Liquefaction in Arkansas*, MBTC 3017, Mack-Blackwell Rural Trans. Center at the U. of Arkansas.

[2] Youd, T. L., Idriss, I.M., et al. (2001). "Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops of evaluation of liquefaction resistance of soils." *J. of Geotech. and Geoevir. Engrg*., Vol. 127(4): 297-313.

[3] Cetin, K.O., Seed, R.B., Kiureghain, A.D., Tokimatsu, K., Harder, L.F., Kayen, R.E., Moss, R.E.S. (2004). "Standard Penetration Test-Based Probabilistic and Deterministic Assesment of Seismic Soil Liquefaction Potential." *J.of Geotech. and Geoevir. Engrg.*, Vol. 130(12): 1314-1340.

[4] Idriss, I. M., and Boulanger, R. W. (2008). "Soil Liquefaction during Earthquakes." *MNO-12*, Earthquake Engineering Research Institute.

following two different procedures: Tomkimatsu & Seed (1987)[5] and Idriss and Boulanger (2008). The Tomkimatsu & Seed (1987) procedure for calculating seismic compression is discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

Plots that show the distribution of estimated seismic compression vs. elevation for the two procedures are provided as Figures 14, 15, 16, and 17 for Borings S2-1, S2-2, S2-3, and S2-4, respectively. For reference, the top and bottom elevation of the boring is indicated by a horizontal dashed line on each plot. As shown in these figures, the total estimated settlements at the boring locations due to seismic compression range from about 2 to 10 inches depending on the analysis method.

5.2.3 Residual Strengths of Liquefied Soils. Residual strengths for post-earthquake stability analyses were estimated for soils that were identified as likely to liquefy (i.e., FS<=1.0) based on the Idriss and Boulanger (2008) liquefaction triggering criteria. The residual strengths were estimated using the procedures outlined in Idriss and Boulanger (2008) and based on the correlation proposed by Olson and Johnson (2008)[6]. The correlations proposed by Olson and Johnson (2008) are included in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

5.3 Slope Stability

Slope stability analyses were performed for the proposed conditions using the SLOPE/W computer program and the Spencer Method. The stability analyses were performed for end of construction, long term, pseudo-static, and post-earthquake conditions. We understand that the target factors of safety are 1.5 for end of construction and long-term conditions, and 1.1 for pseudo-static and post-earthquake conditions. Analyses were performed for the spill-though slopes and for the embankment side slopes. A traffic surcharge load of 250 psf was applied in pavement areas in the analyses.

The end of construction analyses use undrained strengths for cohesive soils and drained strengths for cohesionless soils. The long-term analyses use drained strengths for all soils. The

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[5] Tokimatsu, K. and Seed, H.B. (1987). “Evaluation of settlements in sand due to earthquake shaking.” *J. of Geotech. Engrg.*, Vol. 113(8): 861-878.

[6] Olson, S. M. and Johnson, C. I. (2008). “Analyzing Liquefaction-induced Lateral Spreads Using Strength Ratios.” *J. of Geotech. and Geoenviron. Engrg.*, 134(8): 1035–1049.

pseudo-static analyses use undrained strengths for cohesive soils, drained strengths for cohesionless soils, and include a seismic coefficient equal to 0.5 times the site class specific PGA (i.e., $0.5*F_{PGA}*PGA$) as suggested in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual. The post-earthquake analyses use undrained strengths for cohesive soils, residual strengths for cohesionless soils that were identified as likely to liquefy, and drained strengths for cohesionless soils that were not identified as likely to liquefy. For cohesive soils that were estimated to have peak undrained strengths of approximately 1,500 psf or less, undrained strengths equal to 0.8 times the peak undrained strengths were used in the post-earthquake analyses to account for possible cyclic softening.

Due to issues both with stability and with settlements, we recommend that timber piles be used in combination with a biaxial geogrid load transfer platform to improve slope stability factors of safety and to mitigate large settlements. Based on our analyses, we recommend that 40-ft long timber piles be installed in a square grid arrangement at a center-to-center spacing of 4.25 ft in both directions. The timber piles should be 1-ft minimum in diameter at the top. At the west approach embankment, the grid of timber piles should extend from the mid-point of the spill-through slope back about 120 ft to at least Sta. 119+79. At the east approach embankment, the grid of timber piles should extend from the mid-point of the spill-through slope back about 50 ft to at least Sta. 122+70. A 2-ft thick bridging layer of granular material with 3 layers of geogrid should be constructed above the timber piles. The bottom layer of geogrid should be 6 inch above the top of timber piles/base of bridging layer. The geogrid layers should be at 6-in vertical spacing. Adjacent rolls of geogrid should be placed such that overlaps extend completely across pile tops (i.e., minimum 1 ft of overlap). The biaxial geogrid should have a minimum tensile strength at 5% strain equal to 1,350 lbs per ft in both directions. BaseLok BX3030 geogrid manufactured by Industrial Fabrics, Inc. is an example of a biaxial geogrid that satisfies these criteria. For reference, an example cross section of a timber-pile-supported embankment is shown in the following figure. Additional discussion of the timber-pile-supported embankment is provided in Section 5.4.

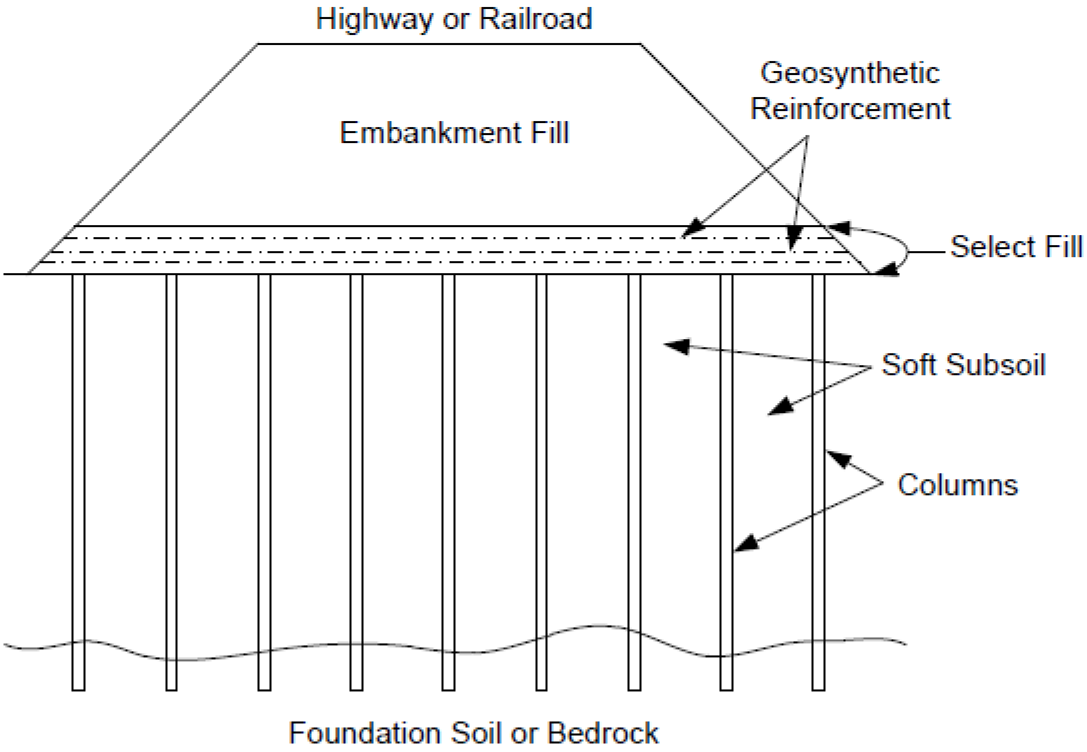

Diagram of a geosynthetic-reinforced column-supported embankment (Sloan et al.[7])

A summary of the slope stability Factor of Safety (FS) values is provided in Table 2. The analyzed geometries, soil properties, and critical failure surfaces are shown in Figures 18 to 25. The timber piles are accounted for in these analyses by restricting the slip surfaces to a minimum depth of 40 ft below the existing ground surface. Considering this and the close timber pile spacing, the slope stability FS results are probably conservative. For comparison, the factors of safety without timber piles for the pseudostatic case are 0.31 and 0.82 for the west and east abutments, respectively.

Table 2 - Slope Stability FS Results Summary

Conditions	Req'd	West Abutment Spill-Through	East Abutment Spill-Through
End of	1.5	6.14	4.79

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[7] Sloan, J.A., Filz, G.M., Collin, J.G., and Kumar, K. (2014). Column-Supported Embankments: Field Tests and Design Recommendations (2nd Edition), CGPR #77, Center for Geotechnical Practice and Research, Virginia Tech, Blacksburg, VA.

Construction			
Long Term	1.5	5.10	4.38
Pseudostatic	1.1	2.07	2.02
Post-Earthquake	1.1	6.06	4.66

5.4 Pile-Supported Embankment Design and Estimated Consolidation Settlement

The analysis of the timber-pile-supported embankment was performed using the GeogridBridge 2.0 spreadsheet (Sloan et al., 2014, and Filz and Smith, 2006[8]). A 10-step design method is integrated into the spreadsheet. The design method is based on rigorous numerical stress-strain analyses that were verified against closed-form solutions, pilot-scale laboratory tests, and field case histories. In general, the design method uses the Adapted Terzaghi Method in combination with the stiffnesses of the embankment, geosynthetic, and the foundation system to rationally evaluate the net load on the geosynthetic reinforcement. The net load is then used to rationally evaluate the strain and tension in the geosynthetic for design. In addition to geosynthetic strain and tension, the embankment settlement is also calculated by the spreadsheet. A copy of the geogrid spreadsheet input and output is presented as Figure 26.

For the design proposed in Section 5.3, the spreadsheet calculates a maximum geogrid strain of 0.047, geogrid tension of 1,350 lbs/ft (per geogrid layer), and a total embankment settlement of 2.5 inches. Approximately 50 percent of the settlement is expected to occur during bridge construction. No settlement problems due to consolidation settlement are anticipated if the proposed timber-pile-supported embankment is used. For comparison, settlements greater than 1 ft were calculated for the west approach embankment for the case without the timber-pile-supported embankment.

5.5 Deep Foundations

We understand that driven 18-in. and 24-in. diameter, closed-ended steel pipe piles are proposed for the abutment bents and interior bents, respectively. Analyses were performed to evaluate the abutment bents and interior bents pile capacities based on the guidance provided by

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[8] Filz, G.M. and Smith, M.E. (2006). Design of Bridging layers in Geosynthetic-Reinforced, Column-Supported Embankments, VTRC 06-CR12, Virginia Transportation Research Council, Charlottesville, VA.

ARDOT and the recommendations discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

5.5.1 Axial Pile Capacity. Axial pile capacity curves were computed based on the pile type shown on the provided plans and the subsurface soil conditions encountered in the borings. Scour was not considered in our analyses. If significant scour is anticipated, we should be contacted to provide revised capacity curves.

The pile capacities were estimated based on the FHWA design procedure using the ENSOFT computer program APile v2015. The compression capacity of an individual pile consists of a combination of skin friction around the perimeter of the pile shaft and end bearing at the tip. The skin friction in the upper 5 ft of soil was neglected. Separate calculations were performed to determine pile capacities with and without consideration of seismic effects. For the calculations that consider seismic effects, the pile skin friction was reduced by 90% for liquefiable soil layers between the ground surface and a depth of 50 ft and the pile skin friction was reduced by 50% for liquefiable soil layers below a depth of 50 ft.

The pile capacity curves are presented in Figures 27, 28, and 29, for the west abutment, east abutment, and interior bents, respectively. The pile capacity curves are presented as nominal (ultimate) values that do not include a resistance factor. An appropriate resistance factor should be applied to the nominal values presented on the pile capacity curves. Guidance on resistance factors is provided in Section 6.2. We recommend that the piles extend at least 10 feet into Zone 4D (see Figure 7 profile) to ensure that the piles are tipped below the deepest soil layer with a moderate likelihood of liquefaction (i.e., Zone 4C).

5.5.2 Downdrag. The seismic compression of the liquefiable soil layers can result in drag loads and increased pile settlement. Pile drag loads occur when the soils surrounding a pile settle more than the pile and apply negative skin friction to the pile. These drag loads increase the compressive loads in the pile that should be considered as part of the pile structural design. Structural capacity determination of the piles is not in our scope for this investigation.

The depth at which the pile and the soils settle the same amount is referred to as the neutral plane. Below the neutral plane, the pile settles more than the surrounding soils. The depth of the neutral plane depends on the soil settlement profile, the pile length, the distribution of pile skin friction and end bearing, and the load applied to the top of the pile. The soil settlement profiles were based on the distributions of seismic compression. The distributions of pile skin friction and end bearing were based on the axial pile capacity curves that consider reduced skin friction in the liquefiable soil layers. We used unfactored dead loads provided by Neel Schaffer, Inc. as the loads applied to the tops of the piles. For the interior bent piles, we added the self-weight of the pile stick-up (between the ground surface and the bottom of the pile cap) to the unfactored deadloads.

The downdrag analysis results are summarized in the following tables. Table 3 and Table 4 present the results for the west abutment bent for loads of 65 kips and 80 kips, respectively. Table 5 and Table 6 present the results for the east abutment bent for loads of 65 kips and 80 kips, respectively. Table 7 presents the results for the interior bents for a load of 87 kips. For each case, results are provided for a range of possible pile lengths.

Table 3 - Downdrag Analysis Results for West Abutment with Load of 65 kips

	Pile Length (ft) below El 248 ft				
	90	95	100	110	120
Maximum Drag Load (kips)	339	384	431	532	607
Top of Pile Settlement (in.)	3.2	3.2	3.2	1.2	0.2
Neutral Plane Depth (ft)	63.3	66.8	70.7	84.1	89.0

Table 4 - Downdrag Analysis Results for West Abutment with Load of 80 kips

	Pile Length (ft) below El 248 ft				
	90	95	100	110	120
Maximum Drag Load (kips)	332	377	422	524	607
Top of Pile Settlement (in.)	3.2	3.2	3.2	1.3	0.2
Neutral Plane Depth (ft)	62.7	66.2	69.8	83.2	89.0

Table 5 - Downdrag Analysis Results for East Abutment with Load of 65 kips

	Pile Length (ft) below El 246 ft				
	80	85	90	100	110
Maximum Drag Load (kips)	239	275	318	369	369
Top of Pile Settlement (in.)	4.4	2.9	1.6	0.1	0.1
Neutral Plane Depth (ft)	59.0	65.6	71.9	77.0	77.0

Table 6 - Downdrag Analysis Results for East Abutment with Load of 80 kips

	Pile Length (ft) below El 246 ft				
	80	85	90	100	110
Maximum Drag Load (kips)	229	267	309	369	369
Top of Pile Settlement (in.)	5.1	2.9	1.9	0.1	0.1
Neutral Plane Depth (ft)	57.6	64.4	70.8	77.0	77.0

Table 7 - Downdrag Analysis Results for Interior Bents with Load of 87 kips

	Pile Length (ft) below El 236 ft				
	75	80	85	90	100
Maximum Drag Load (kips)	359	410	468	518	518
Top of Pile Settlement (in.)	3.3	2.7	1.0	0.1	0.1
Neutral Plane Depth (ft)	56.2	62.7	68.9	72.0	72.0

5.5.3 Lateral Analysis Parameters. If lateral loads applied to the piles are substantial, a lateral load analysis should be performed. The piles should be designed so that angular rotation and deflection at the tops of the piles are maintained within structurally tolerable limits. We recommend that the response of the piles to applied moment and lateral loading be analyzed utilizing the method developed by Dr. Lymon C. Reese of the University of Texas or a similar analysis procedure. Computer programs (e.g., LPILE) are available for this method of analysis. The analysis method utilizes finite difference approximations to solve for deflection, moment, soil modulus and soil reaction for a single pile. Soil response to the laterally loaded pile is represented in the analysis by a set of nonlinear “p-y" curves that are developed for various depths along the pile and for the different soil types. The "p-y" curves essentially indicate the soil reaction in force per unit length of pile versus deflection for a given pile diameter. A tabulation of recommended soil parameters that can be used in the lateral pile analysis are presented in Table 8. The LPILE default values of E_{50} and k, which are correlated based on the cohesion and friction angle, can be used in the lateral pile analysis.

Table 8 - Recommended Soil Parameters for Lateral Pile Analysis

Soil Zone	p-y Curve Type	Effective Unit Weight (pcf)	Cohesion (psf)	Internal Friction Angle (degrees)
New Fill	Stiff Clay w/o Free Water (Reese)	57.6	1500	-
1	Soft Clay (Matlock)	60.6	830	-
2	Soft Clay (Matlock)	62.6	200	-
3	Stiff Clay w/o Free Water (Reese)	62.6	1300	-
4A, 4B, 4C, 4D	Sand (Reese)	57.6	-	34

Liquefaction of sands and cyclic softening of clay soils can result in significant short-term strength losses that can reduce lateral pile capacity. Accordingly, Table 9 provides a separate set of soil parameters that should be used instead of the values in Table 8 in the lateral pile analysis for seismic conditions.

Table 9 - Recommended Post-Earthquake Soil Parameters for Lateral Pile Analysis

Soil Zone	p-y Curve Type	Effective Unit Weight (pcf)	Cohesion (psf)	Internal Friction Angle (degrees)
New Fill	Stiff Clay w/o Free Water (Reese)	57.6	1200	-
1	Soft Clay (Matlock)	60.6	664	-
2	Soft Clay (Matlock)	62.6	160	-
3	Stiff Clay w/o Free Water (Reese)	62.6	1040	-
4A, 4B, 4C, 4D	Sand (Reese)	57.6	-	34

5.5.4 Drivability Analysis. A "drivability" type wave equation analysis relating blow counts to pile penetration, ultimate static pile capacities, dynamic pile driving stresses, minimum recommended hammer energy and hammer strokes was performed using the program GRLWEAP v.2010. The unit skin friction and end-bearing values in each soil layer were developed based on the results of unconsolidated undrained (UU) triaxial compression tests, supplemented by the results of the field standard penetration tests and visual estimates of consistency and the static analysis program in GRLWEAP. A 72% pile hammer efficiency and a shaft gain/loss factor of 0.833 and a toe gain/loss factor of 1.0 were used in the analysis. A maximum driving stress of 90% of the steel yield strength was considered for these analyses.

Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. An open-ended D36 diesel hammer was utilized for the drivability analyses of both pile sizes. Hammer and pile cushion information was based on manufacturer-recommended values. Both the 18-in. and 24-in. diameter steel pipe piles were assumed to be installed close-ended. In the analyses, the piles at the abutments and interior bents are assumed to be driven from the plan pile cap bottom elevations to the recommended tip elevations. Graphical and tabulated results of the drivability analyses are provided in Appendix C. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed prior to hammer acceptance and beginning of driving. The resulting minimum hammer energy to drive the piles at the abutment and interior bents is provided in Table 10.

Table 10 - Results of Drivability Analyses

Location	Hammer Type	Minimum Hammer Energy (kip-ft.)
Abutment Bents	D36	80
Interior Bents	D36	80

The parameters used in the wave equation analysis were based on general information available at the time of the analysis; however, actual field conditions may be different. We recommend prudent use of the wave equation analysis results. Soil response, hammer performance, and pile stresses and drivability should be verified by dynamic measurements using the Pile Driving Analyzer (PDA) on site and subsequent data analysis with the CAPWAP program. The actual suitability and final acceptance of a hammer system for a given project can only be determined after demonstration of satisfactory field performance, which is typically evaluated during the Test Pile Driving Program with PDA dynamic pile measurements and related data analyses.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Pile Design and Installation

Driving refusal for the steel pipe piles may occur in the dense to very dense sands encountered in Zone 4 (see Figure 7 profile). If refusal occurs at depths shallower than the required minimum depth, then jetting will be required to achieve additional penetration. However, the final 5 ft of pile penetration must be achieved by driving. Driven piles should be installed in accordance with AHTD Standard Specification Section 805 PILING.

The pile capacity curves presented in this report do not reflect the effects of jetting. As described in FHWA-NHI-16-009, Design and Construction of Driven Pile Foundations, the use of jetting will result in greater soil disturbance than considered in standard static pile capacity calculations. Some field studies have reported that the pile side resistance may be reduced by about 50 percent over the jetted depth. If jetting is necessary, we should be contracted to provide revised axial capacities. Dynamic load testing should be performed during construction to more accurately determine the ultimate capacity of the piles after jetting.

6.2 Test Piles, Dynamic Load Testing, and Resistance Factors

Based on Table 10.5.5.2.3-1 of the AASHTO LRFD 2017 Bridge Design Specifications and considering that the soil profiles consist predominantly of sand, a resistance factor of 0.45 should generally be applied for axial compression and a resistance factor of 0.35 should generally be applied for tension. A higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 11.

Table 11 - Pile Resistance Factors based on Condition/Resistance Determination Method

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 condition, but no less than 2% of the 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 the production piles.	0.65
	Wave equation analysis, without pile dynamic measurements or load test by with field confirmation of hammer performance.	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only).	0.40

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

As discussed in Section 10.5.5.3.3 of the Bridge Design Specifications, a resistance factor of 1.0 should be applied for axial compression and a resistance factor of 0.80 should be applied for tension when designing the foundations to resist earthquake loading.

We recommend a minimum of two test piles (one at an abutment bent and one at an interior bent) be driven to evaluate pile capacities and drivability, prior to ordering the production piles. The test pile lengths should be selected considering the estimated pile capacities, minimum penetration requirements, and the anticipated driving resistance. The test piles can be driven at permanent pile locations.

We recommend that dynamic pile load testing be performed on the test piles in accordance with ASTM D 4945. The results of the dynamic pile load test should be used to establish driving criteria for the production piles. The embedment length of the piles may be increased based on the

PDA evaluation. All testing should be performed prior to ordering production piles in case the design lengths change due to the testing.

The dynamic pile load testing data collection should be performed by an engineer with a minimum of one year of dynamic pile testing field 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/or Foundation QA. Pile driving modeling and analysis of PDA data should be performed by an engineer with a minimum of five years of 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/or Foundation QA.

6.3 Embankment Construction

Embankment construction shall conform with Section 210 and all other applicable requirements of the latest AHTD Standard Specification for Highway Construction. The fill material for embankment construction should classify as AASHTO A-6, A-5, or A-4 with a liquid limit less than 45 and a plasticity index less than or equal to 25. The fill materials should be compacted to not less than 95 percent of standard Proctor maximum dry density (AASHTO T99) at moisture contents within 3 percentage points of the optimum moisture content. Fill material with a plasticity index less than 10 or that is susceptible to erosion shall have a minimum 18-inch clay plating (measured perpendicular to the finished slopes). Clay plating shall consist of material having a plasticity index in the range of 10 to 25 that supports vegetation and that is not highly susceptible to erosion.

As an initial site preparation step, existing utilities or pipes and any other subsurface obstructions that might interfere with earthwork, bridge, and/or drainage ditch construction should be removed and/or relocated. Stripping should then be performed within the construction areas to remove organic-laden surficial soils, vegetation, debris, brush or roots. Temporary excavation slopes should not be steeper than 1H:1V. We recommend that excavations be left open for the shortest possible duration to minimize exposure of the bearing soils to rainfall. Drainage should be maintained away from the excavations during construction.

Prior to placement of any fill materials, the soils exposed after excavation should be inspected. Any obviously weak soils should be excavated and replaced with properly compacted backfill. The effort required to mitigate any unstable soils will be influenced by the season of the year when earthwork is performed. The soils may be drier during the hot late summer and could weaken during heavy rain events. We recommend that earthwork be performed during a dry summer or fall season, if the schedule permits. The vertical and lateral extent of excavation required to remove any weak soils must be determined in the field during earthwork construction. In order to minimize the amount of excavation, we recommend that a representative of Burns Cooley Dennis, Inc. be present to observe excavation operations and assist in evaluating the depth and lateral extent of any excavation required.

In areas where embankments are to be constructed over existing ditches, we understand that the work will conform with the requirements presented in the AHTD Special Provision for Embankment Construction, which is provided in Appendix D. This special provision requires that the ditches shall be undercut 2 feet to remove all highly organic, wet material and backfilled with Stone Backfill prior to embankment construction. The remaining embankment shall be constructed of Select Material (Class SM-2). Synthetic Filter Blanket and Dumped Riprap shall be placed on the slopes of embankments constructed of SM-2 from the top of the Stone Backfill to at least 2 feet above the high-water elevation. The remainder of embankments construction of SM-2 or other material that is susceptible to erosion shall have a minimum 18-inch clay plating (measured perpendicular to the finished slopes). Clay plating shall consist of material having a plasticity index in the range of 10 to 25 that supports vegetation and that is not highly susceptible to erosion.

As discussed in Section 210.09 of the AHTD Standard Specification, where fill materials are to be placed and compacted against a slope, the slope shall be continuously benched as the fill lifts are placed and compacted.

Laboratory classification tests, including grain size analyses and Atterberg limit determinations, should be performed on the backfill soils initially and routinely during earthwork operations to check for compliance with the recommendations provided herein. Field moisture and density tests should be performed at frequencies that satisfy the requirements specified in Section 210.02 of the AHTD Standard Specification.

7.0 REPORT LIMITATIONS

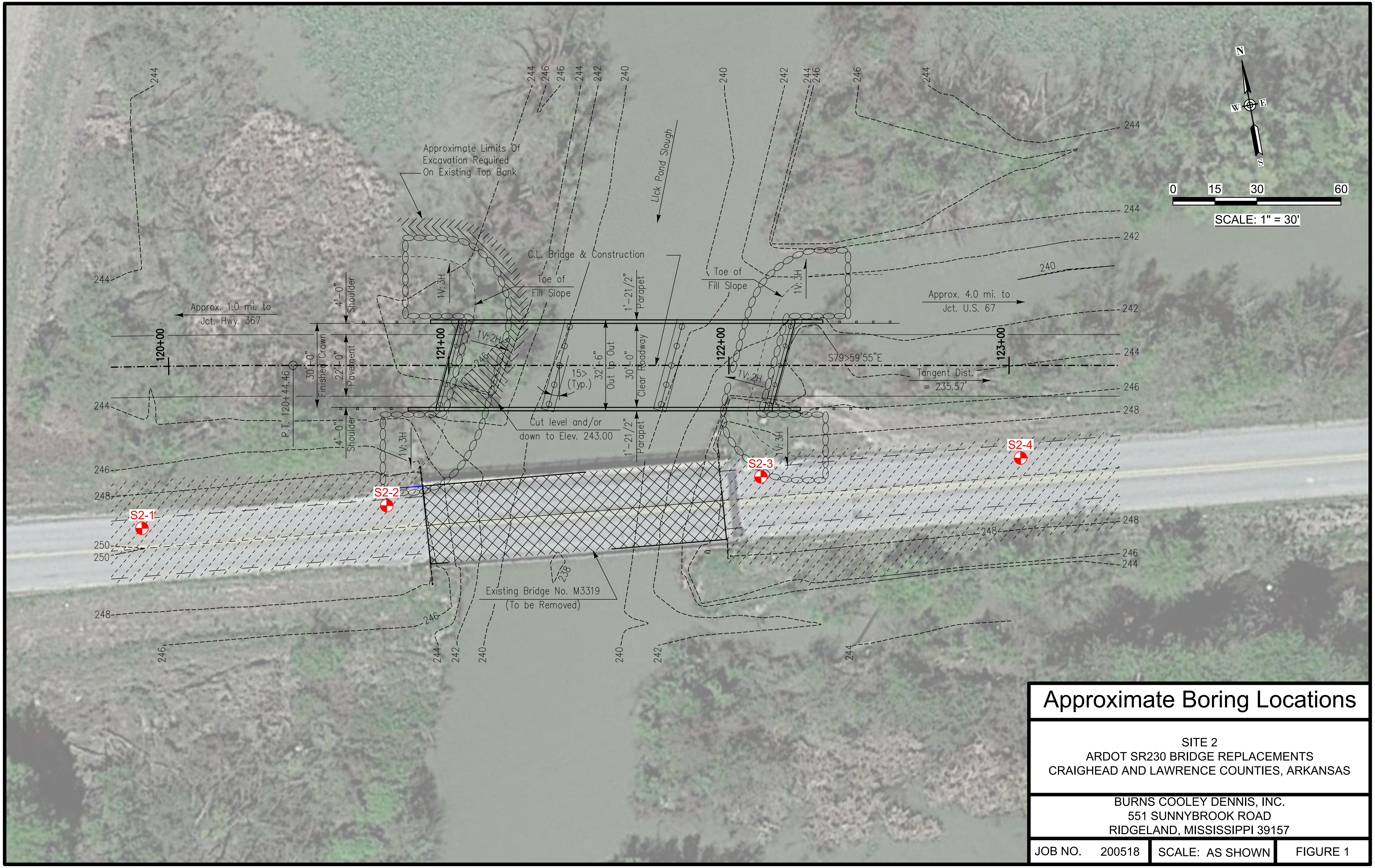
The analyses, conclusions, and recommendations discussed in this report are based on conditions as they existed at the time of the exploration and further on the assumption that the exploratory borings are representative of subsurface conditions throughout the areas investigated.

It should be noted that actual subsurface conditions between and beyond the borings might differ from those encountered at the boring locations. If subsurface conditions are encountered during construction that vary from those discussed in this report, Burns Cooley Dennis, Inc. should be notified immediately in order that we may evaluate the effects, if any, on earthwork and foundation design and construction.

Burns Cooley Dennis, Inc. should be retained for a general review of final design drawings and specifications. It is advised that we also be retained to observe earthwork for the project, to perform and observe the pile testing, and to develop the pile driving criteria. Our involvement during construction would give opportunity for us to help confirm that our recommendations are valid or to modify them accordingly. Burns Cooley Dennis, Inc. cannot assume responsibility or liability for the adequacy of recommendations if we do not observe construction.

This report has been prepared for the exclusive use of Neel-Schaffer, Inc. for specific application to the geotechnical-related aspects of design and construction of the ARDOT SR230 Bridge Replacements in Craighead and Lawrence Counties, Arkansas. The only warranty made by us in connection with the services provided is we have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing in the same or similar locality. No other warranty, express or implied, is made or intended.

FIGURES



Approximate Boring Locations

SITE 2
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

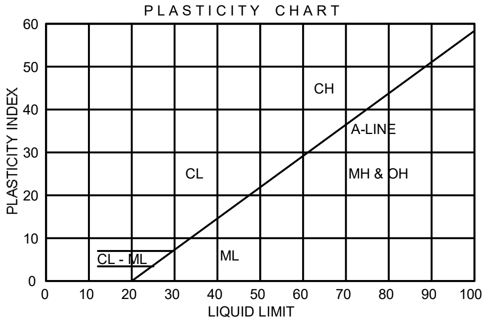
JOB NO.	200518	SCALE: AS SHOWN	FIGURE 1

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL & LETTER	DESCRIPTION
COARSE-GRAINED SOILS More than half of material larger than No. 200 sieve size	GRAVELS More than half of coarse fraction larger than No.4 sieve size	Clean Gravels (Little or no fines)	GW	WELL GRADED GRAVEL, GRAVEL-SAND MIXTURE
			GP	POORLY GRADED GRAVEL, GRAVEL-SAND MIXTURE
		Gravels with fines (Appreciable amount of fines)	GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURE
			GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY MIXTURE
	SANDS More than half of coarse fraction smaller than No.4 sieve size	Clean Sands (Little or no fines)	SW	WELL GRADED SAND, GRAVELLY SAND
			SP	POORLY GRADED SAND, GRAVELLY SAND
		Sands with fines (Appreciable amount of fines)	SM	SILTY SAND, SAND-SILT MIXTURE
			SC	CLAYEY SAND, SAND-CLAY MIXTURE
FINE-GRAINED SOILS More than half of material smaller than No. 200 sieve size	SILTS AND CLAYS	Liquid limit less than 50	ML	SILT WITH LITTLE OR NO PLASTICITY
			ML	CLAYEY SILT, SILT WITH SLIGHT TO MEDIUM PLASTICITY
			ML	SANDY SILT
			CL	SILTY CLAY, LOW TO MEDIUM PLASTICITY
			CL	SANDY CLAY, LOW TO MEDIUM PLASTICITY (30% TO 50% SAND)
	SILTS AND CLAYS	Liquid limit greater than 50	MH	SILT, HIGH PLASTICITY
			CH	CLAY, HIGH PLASTICITY
			OH	ORGANIC CLAY OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOIL

TERMS CHARACTERIZING SOIL STRUCTURE

Slickensided	- Clays with polished and striated planes created as a result of volume changes related to shrinking, swelling and/orchanges in overburden pressure.
Fissured	- Clays with a blocky or jointed structure generally created by seasonal shrinking and swelling.
Laminated	- Composed of thin alternating layers of varying color and texture.
Calcareous	- Containing appreciable quantities of calcium carbonate.
Parting	- Paper thin (less than 1/8 inch).
Seam	- 1/8 inch to 3 inch thickness.
Layer	- Greater than 3 inches in thickness.



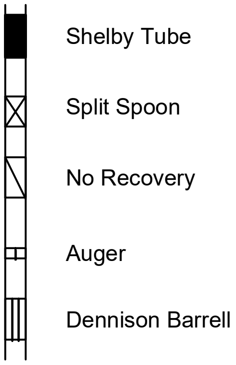
FOR CLASSIFICATION OF FINE GRAINED SOILS

TERMS CHARACTERIZING SOIL STRUCTURE

COARSE-GRAINED SOILS		FINE-GRAINED SOILS		
DENSITY	PENETRATION RESISTANCE, N Blows per Foot	Consistancy	COHESION Kips/Sq.Ft	PENETRATION RESISTANCE, N Blows per Foot
Very loose	0 - 4	Very Soft	<0.25	0 - 1
Loose	5 - 10	Soft	0.25 - 0.50	2 - 4
Medium Dense	11 - 30	Medium Stiff	0.50 - 1.00	5 - 8
Dense	31 - 50	Stiff	1.00 - 2.00	9 - 15
Very Dense	>4.00	Very Stiff	2.00 - 4.00	16 - 30
		Hard	>4.00	>30

SAMPLE TYPES

(Shown in Sample Column)



PARTICLE SIZE IDENTIFICATION

Cobbles	- Greater than 3 inches
Gravel	- Coarse-3/4 inch to 3 inches Fine-4.76 mm to 3/4 inch
Sand	- Coarse-2 mm to 4.76 mm Medium-0.42 mm to 2 mm Fine-0.074 mm to 0.42 mm
Silt & Clay	- Less than 0.074 mm

RELATIVE COMPOSITION

Slightly	5 - 15%
With	16 - 29%
Sandy (or gravelly)	30 - 50%

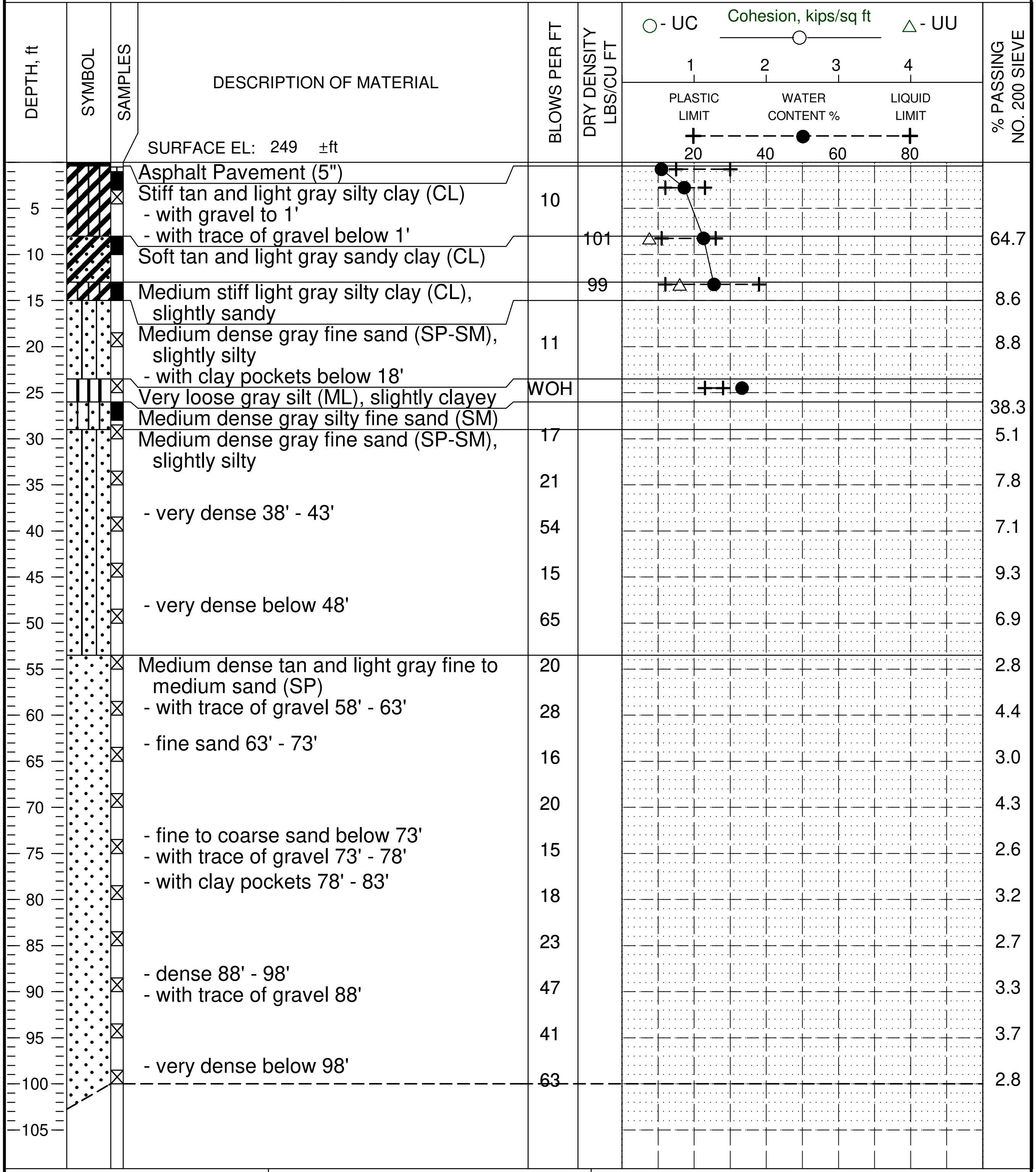
CLASSIFICATION, SYMBOLS AND
TERMS USED ON GRAPHICAL
BORING LOGS

LOG OF BORING NO. S2-1

ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 28', then rotary wash to completion.

LOCATION: Sta. 119+90 (Approximate) +/- 58' Right of Construction C/L



SURFACE EL: 249 ±ft

DEPTH, ft	DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
	Asphalt Pavement (5")			
5	Stiff tan and light gray silty clay (CL) - with gravel to 1' - with trace of gravel below 1'	10	101	64.7
10	Soft tan and light gray sandy clay (CL)			
15	Medium stiff light gray silty clay (CL), slightly sandy		99	8.6
20	Medium dense gray fine sand (SP-SM), slightly silty - with clay pockets below 18'	11		8.8
25	Very loose gray silt (ML), slightly clayey	WOH		
	Medium dense gray silty fine sand (SM)			38.3
30	Medium dense gray fine sand (SP-SM), slightly silty	17		5.1
35		21		7.8
40	- very dense 38' - 43'	54		7.1
45		15		9.3
50	- very dense below 48'	65		6.9
55	Medium dense tan and light gray fine to medium sand (SP)	20		2.8
60	- with trace of gravel 58' - 63'	28		4.4
65	- fine sand 63' - 73'	16		3.0
70		20		4.3
75	- fine to coarse sand below 73' - with trace of gravel 73' - 78'	15		2.6
80	- with clay pockets 78' - 83'	18		3.2
85		23		2.7
90	- dense 88' - 98' - with trace of gravel 88'	47		3.3
95		41		3.7
100	- very dense below 98'	63		2.8
105				

BORING DEPTH: 100 ft

DATE: 08/31/20

COMMENTS: Borehole filled with cement-bentonite grout. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 53' 38.65" - W 91° 4' 10.62"

GROUNDWATER DATA: Free water encountered at an approximate depth of 24' during auger drilling. Water level at an approximate depth of 23.5' after about 60 minutes.

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FIGURE 3

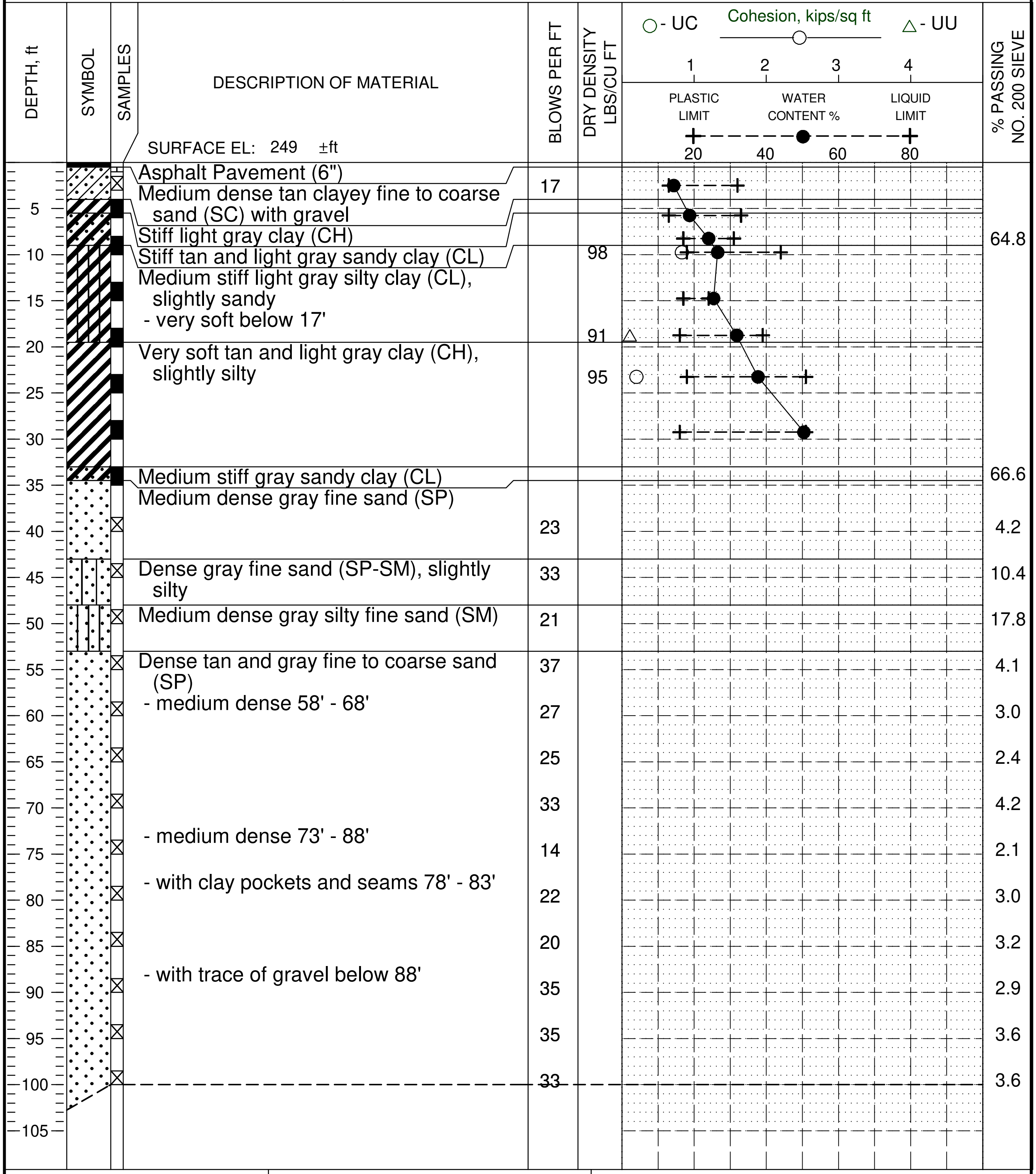
LOG OF BORING NO. S2-2

ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 35', then rotary wash to completion.

LOCATION: Sta. 120+78 (Approximate) +/- 50' Right of Construction C/L

SURFACE EL: 249 ±ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
Asphalt Pavement (6")	17		
Medium dense tan clayey fine to coarse sand (SC) with gravel			
Stiff light gray clay (CH)			64.8
Stiff tan and light gray sandy clay (CL)		98	
Medium stiff light gray silty clay (CL), slightly sandy - very soft below 17'		91	
Very soft tan and light gray clay (CH), slightly silty		95	
Medium stiff gray sandy clay (CL)			66.6
Medium dense gray fine sand (SP)	23		4.2
Dense gray fine sand (SP-SM), slightly silty	33		10.4
Medium dense gray silty fine sand (SM)	21		17.8
Dense tan and gray fine to coarse sand (SP) - medium dense 58' - 68' - medium dense 73' - 88' - with clay pockets and seams 78' - 83' - with trace of gravel below 88'	37 27 25 33 14 22 20 35 35 33		4.1 3.0 2.4 4.2 2.1 3.0 3.2 2.9 3.6 3.6

BORING DEPTH: 100 ft

DATE: 09/03/20

COMMENTS: Borehole filled with cement-bentonite grout. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 53' 38.57" - W 91° 4' 9.56"

GROUNDWATER DATA: Free water encountered at an approximate depth of 3' during auger drilling. Water level at an approximate depth of 1' after about 15 minutes.

FIGURE 4

LOG OF BORING NO. S2-3

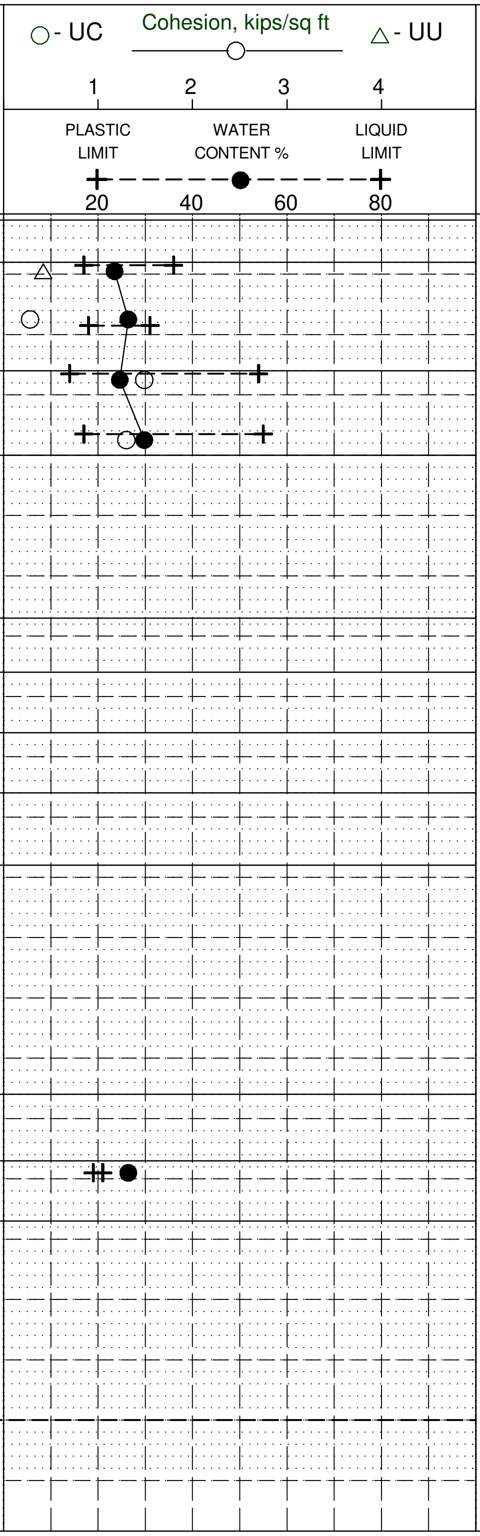
ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 25', then rotary wash to completion.

LOCATION: Sta. 122+11 (Approximate)
+/- 40' Right of Construction C/L

SURFACE EL: 249 ±ft

DEPTH, ft	DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
	Asphalt Pavement (6")			
	Medium dense tan clayey fine to coarse sand (SC) with gravel	17		
5			107	
10	Soft tan and light gray silty clay (CL)		104	
15	Stiff light gray clay (CH), slightly silty		103	
20			96	
25	Medium dense light gray fine sand (SP-SM), slighlty silty	11		7.4
30		15		9.4
35	Medium dense light gray fine sand (SP)	13		3.9
40	Medium dense light gray fine sand (SP-SM), slighlty silty	26		7.9
45	Dense light gray fine sand (SP)	42		4.6
50	Dense light gray fine sand (SP-SM), slightly silty	34		6.3
55	Medium dense light gray fine to coarse sand (SP) with trace of gravel	23		2.5
60	- loose 58' - 63'	9		2.7
65	- dense 63' - 68'	33		4.0
70		18		4.0
75	Medium dense light gray fine to coarse sand (SP-SM), slightly silty, with trace of gravel	17		5.8
80	Medium dense gray sandy silt (ML)	13		57.2
85	Dense tan fine sand (SP)	46		4.7
90	- medium dense 88' - 93' - fine to medium sand, with trace of gravel below 88'	26		3.7
95		43		4.3
100	- very dense below 98'	51		4.1
105				



BORING DEPTH: 100 ft

DATE: 09/08/20 & 09/09/20

COMMENTS: Borehole filled with cement-bentonite grout. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 53' 38.45" - W 91° 4' 7.94"

GROUNDWATER DATA: Free water encountered at an approximate depth of 12.5' during auger drilling. Water level at an approximate depth of 11.5' after about 15 minutes. Water level at an approximate depth of 4.5' after about 10 hours.

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FIGURE 5

LOG OF BORING NO. S2-4

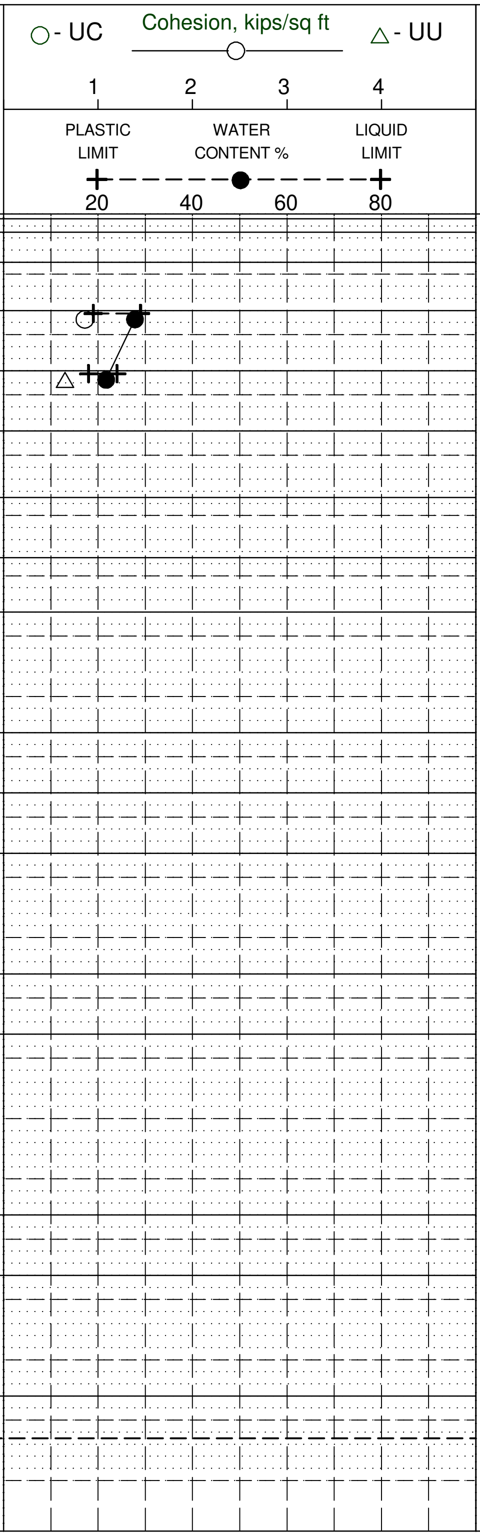
ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 50', then rotary wash to completion.

LOCATION: Sta. 123+04 (Approximate)
+/- 33' Right of Construction C/L

SURFACE EL: 249 ±ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
Asphalt Pavement (5")			
Medium dense tan and light gray clayey fine to coarse sand (SC) with gravel	19		
Very stiff tan and gray clay (CH)			70.2
Medium dense light gray sandy silt (ML)			
Medium stiff tan silty clay (CL), slightly sandy		101	
Medium stiff light gray and tan very silty clay (CL-ML)		103	
Medium dense light gray silty fine sand (SM)			19.0
Medium dense tan fine sand (SP-SM), slightly silty	13		5.1
Medium dense tan fine sand (SP)	14		4.9
Medium dense light gray fine sand (SP-SM), slightly silty	12		9.3
	28		6.3
Dense tan fine sand (SP)	40		4.6
Medium dense light gray fine sand (SP-SM), slightly silty	29		9.9
Medium dense tan fine sand (SP) with trace of gravel	22		3.0
	22		2.7
Medium dense gray fine sand (SP-SM), slighlty silty	18		5.0
Medium dense gray fine sand (SP)	27		4.1
- fine to coarse sand below 73'	28		2.9
	19		2.6
Loose gray fine to coarse sand (SP-SM), slighlty silty	5		5.2
Medium dense gray fine to coarse sand (SP) with trace of gravel	23		2.7
	24		2.3
Very dense tan fine to medium sand (SP-SM), slightly silty	55		
- medium dense below 100'	28		6.8



DEPTH, ft: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80, 85, 90, 95, 100, 105

BORING DEPTH: 101.5 ft

DATE: 09/09/20 &
09/10/20

COMMENTS: Borehole filled with cement-bentonite grout. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 53' 38.33" - W 91° 4' 6.81"

GROUNDWATER DATA: Free water encountered at an approximate depth of 48' during auger drilling. Water level at an approximate depth of 45.5' after about 30 minutes.

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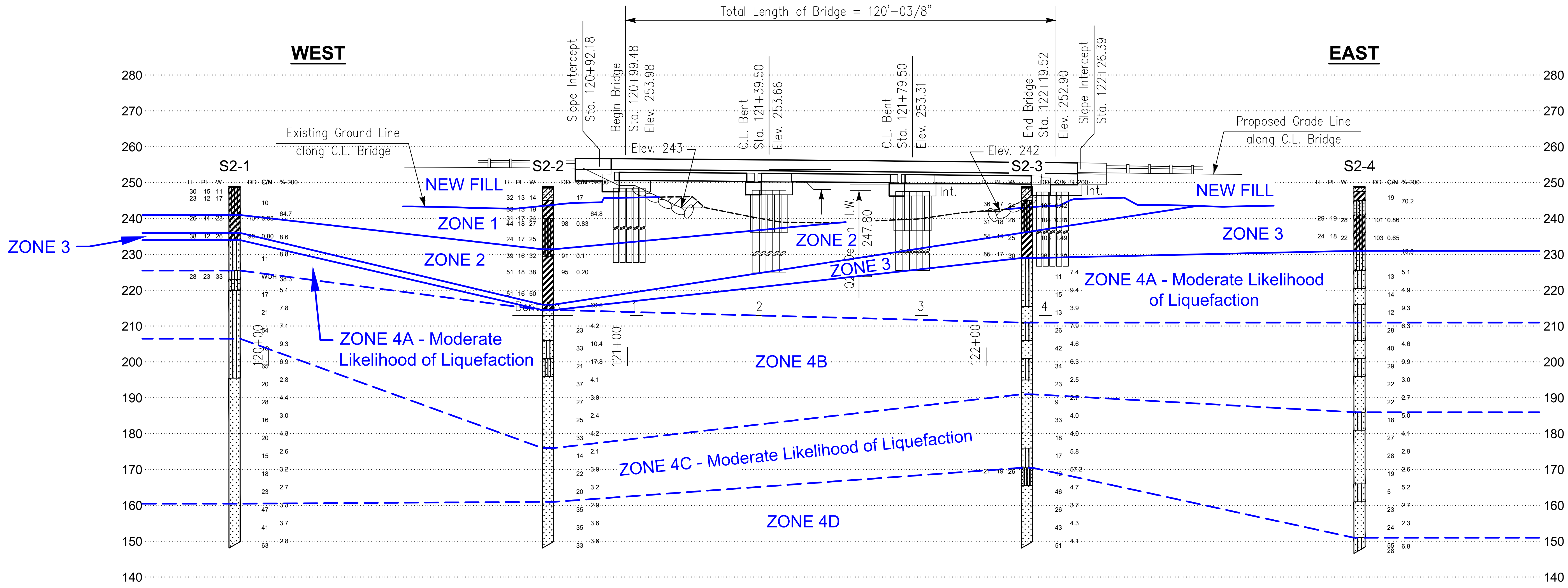
FIGURE 6



ZONE 1

Medium stiff silty clay (CL) and stiff sandy clay (CL)

ZONE 2

Very soft to soft sandy clay (CL), silty clay (CL), & clay (CH), slightly silty

ZONE 3

Medium stiff sandy clay (CL), silty clay (CL) & very silty clay (CL-ML), & stiff clay (CH), slightly silty

ZONE 4

Very loose silt (ML), medium dense silty sand (SM), loose to very dense sand (SP-SM), slightly silty, & loose to very dense sand (SP) with trace of gravel

0 15 30 60

SCALE: 1" = 30'

Note: The SPT blow count "N" values are raw values. They have not been corrected for hammer energy. A hammer energy correction factor of 1.36 applies to borings S2-1, S2-2, S2-3 & S2-4.

Soil Profile

SITE 2
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO.	200518	SCALE: AS SHOWN	FIGURE 7

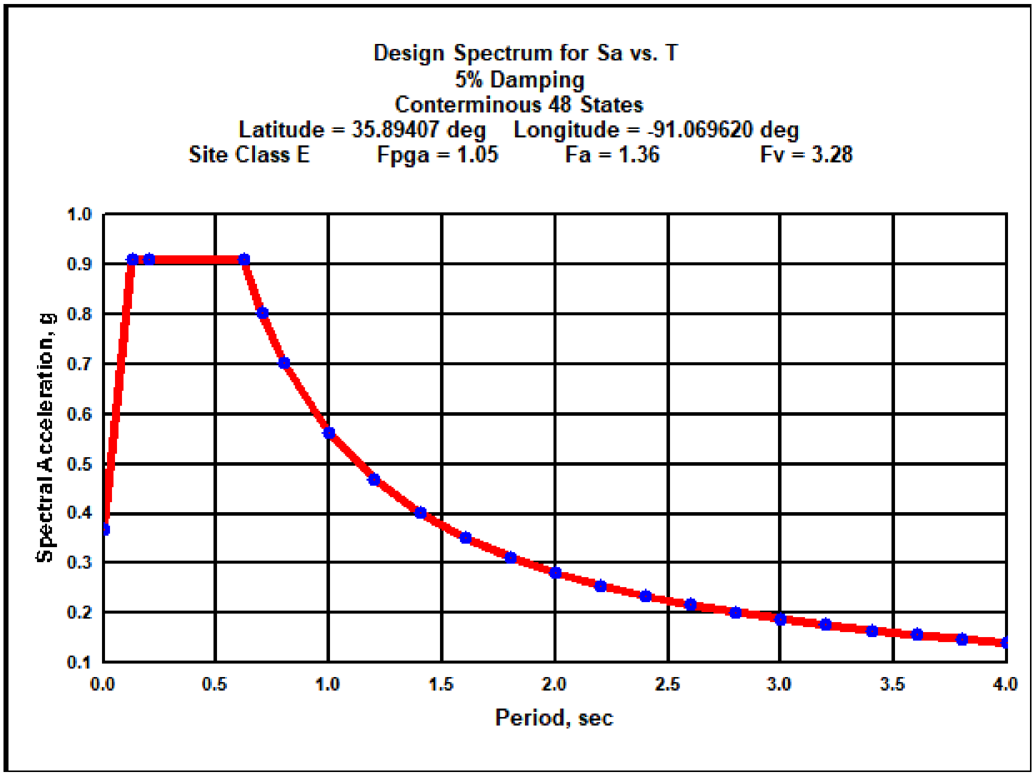


Figure 8 - Seismic Design Spectrum for Sa vs. T

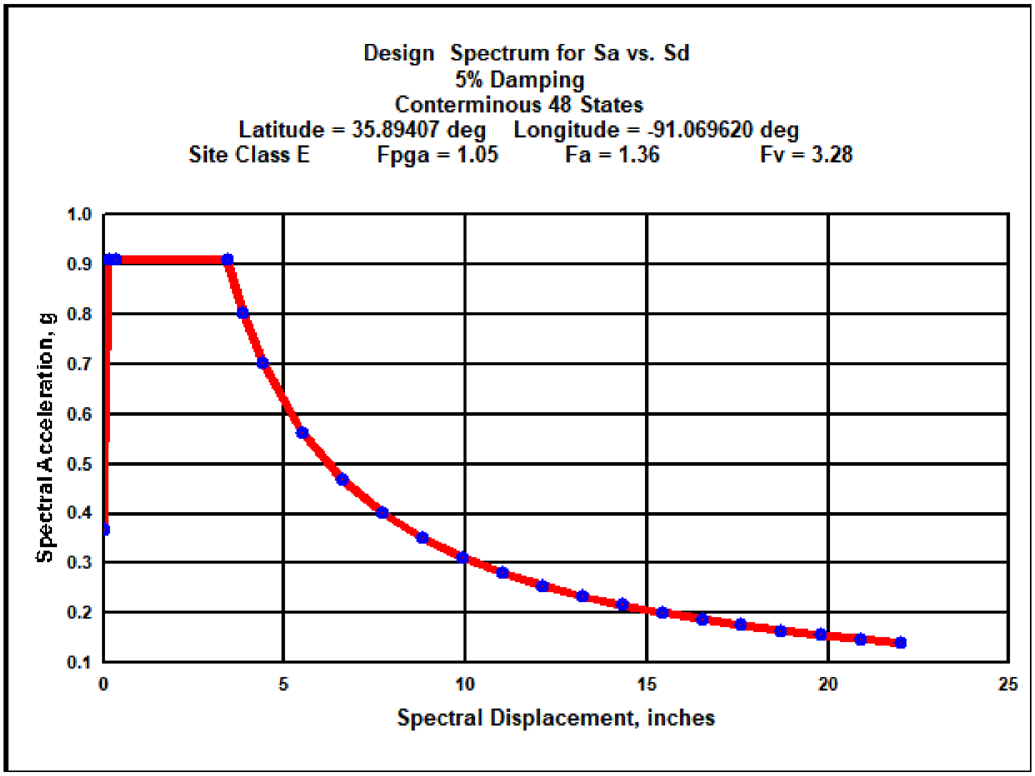


Figure 9 - Seismic Design Spectrum for Sa vs. Sd

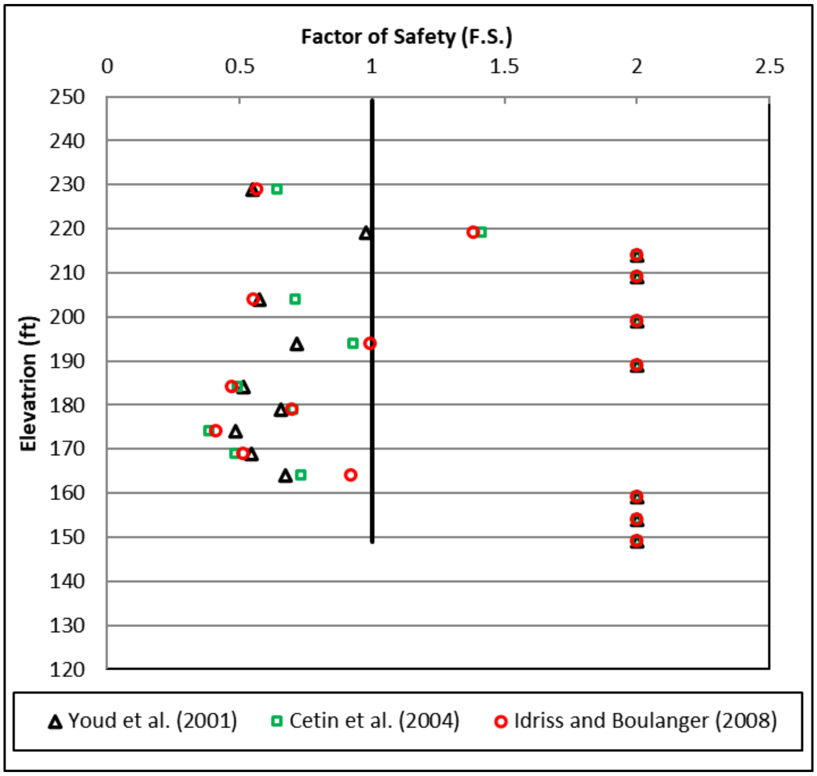


Figure 10 - Liquefaction Triggering FS Values for S2-1 (Top of Boring at EL 249 ft)

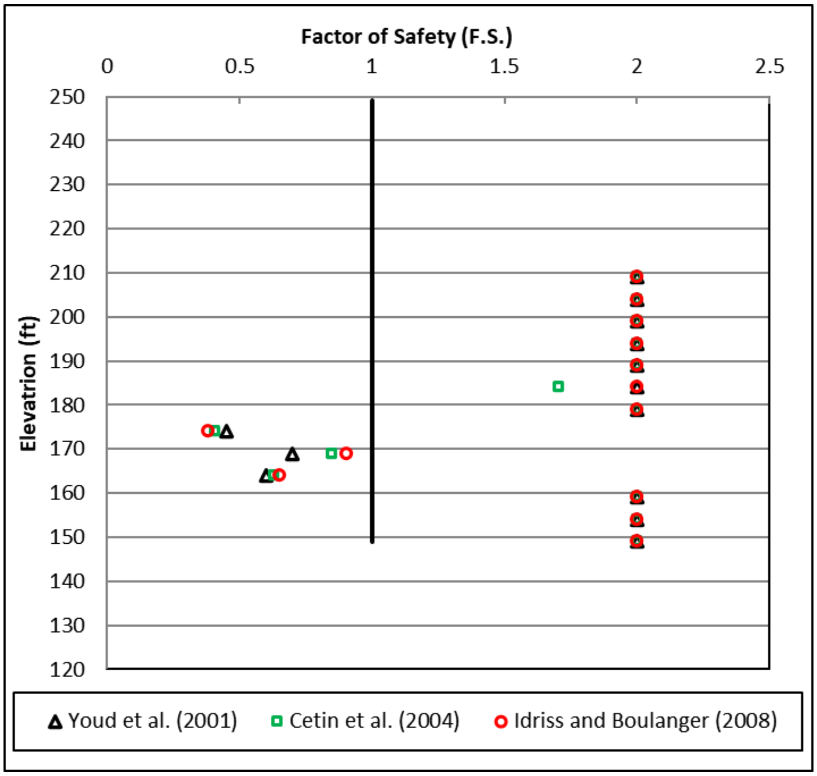


Figure 11 - Liquefaction Triggering FS Values for S2-2 (Top of Boring at EL 249 ft)

Figure 12 - Liquefaction Triggering FS Values for S2-3 (Top of Boring at EL 249 ft)

Figure 13 - Liquefaction Triggering FS Values for S2-4 (Top of Boring at EL 249 ft)

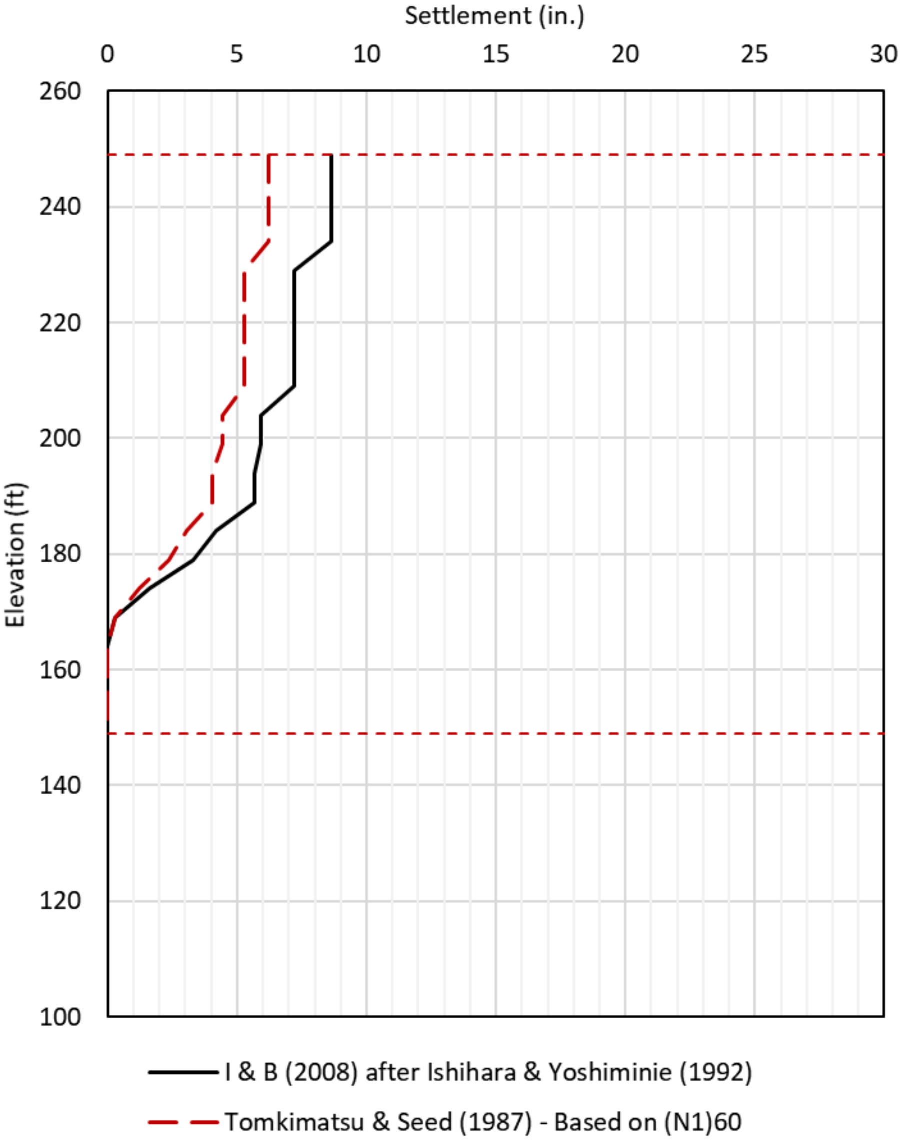


Figure 14 – Seismic Compression for S2-1 (Top of Boring at EL 249 ft)

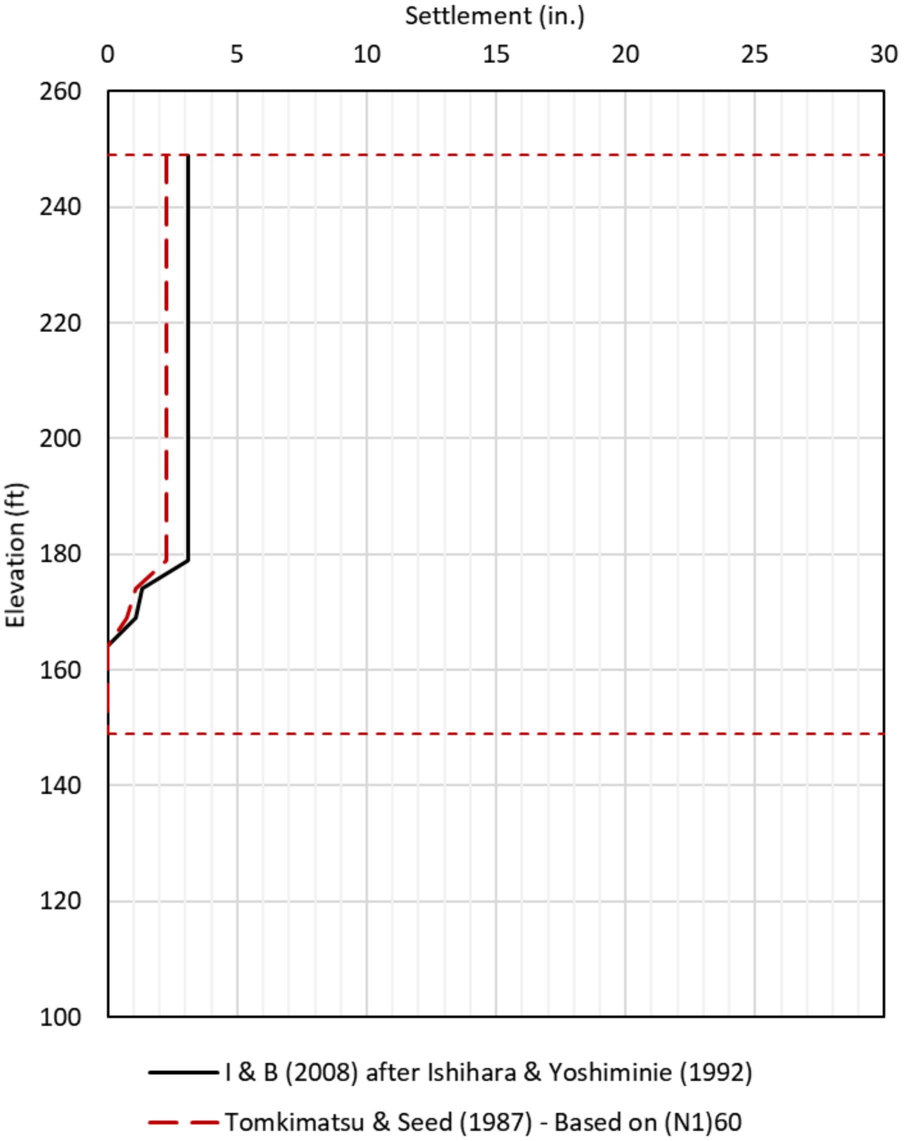


Figure 15 - Seismic Compression for S2-2 (Top of Boring at EL 249 ft)

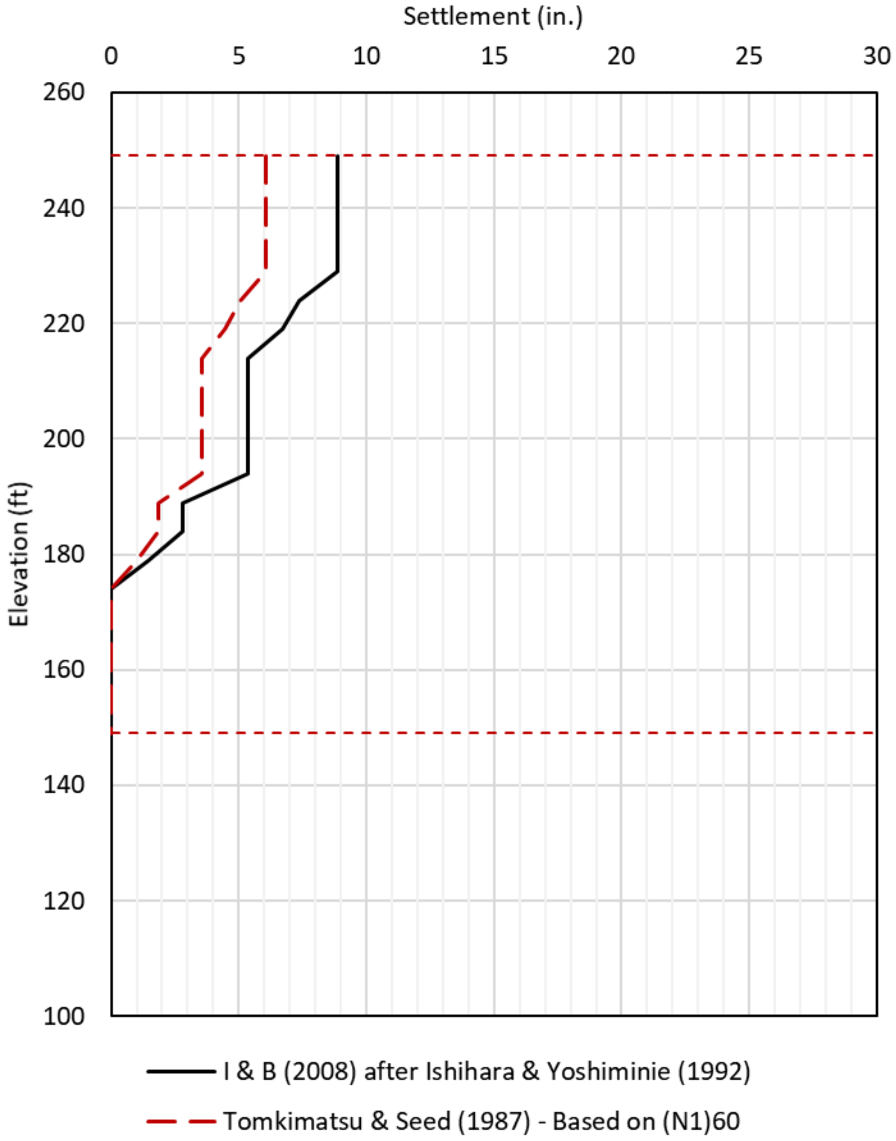


Figure 16 - Seismic Compression for S2-3 (Top of Boring at EL 249 ft)

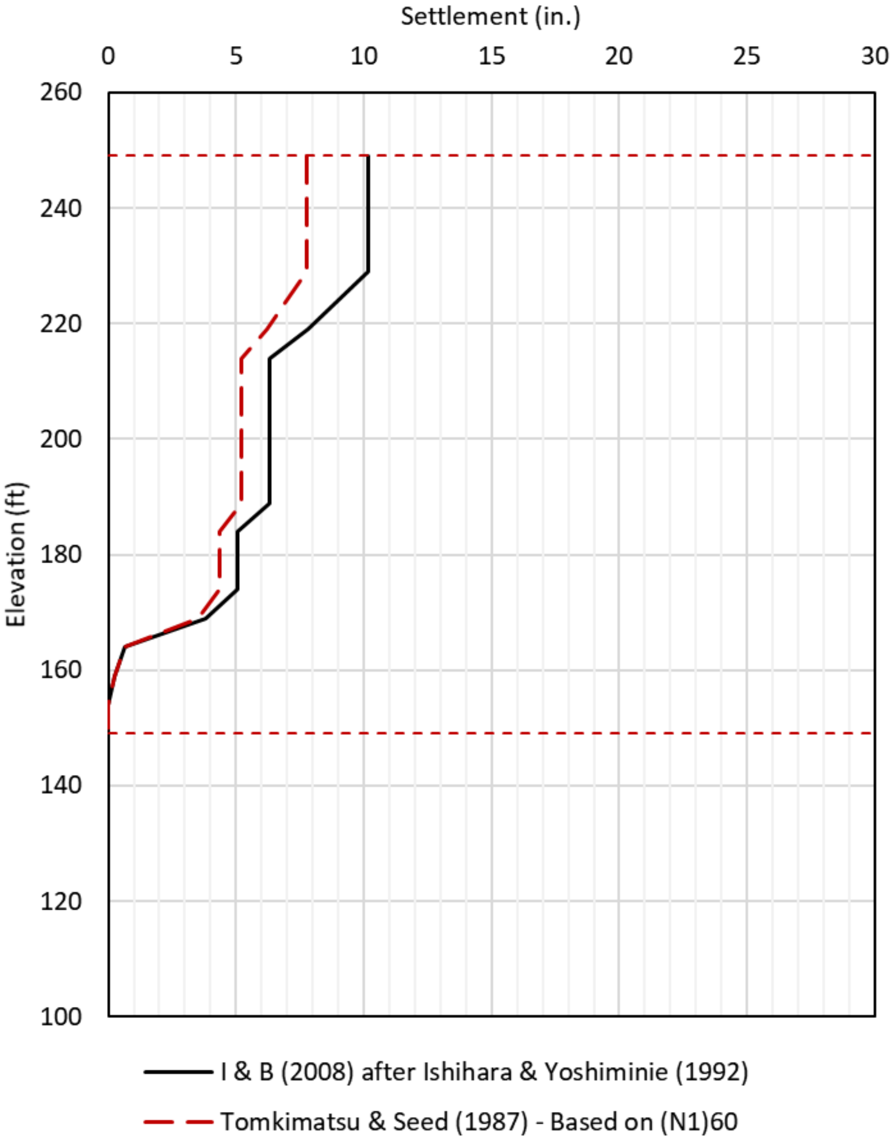
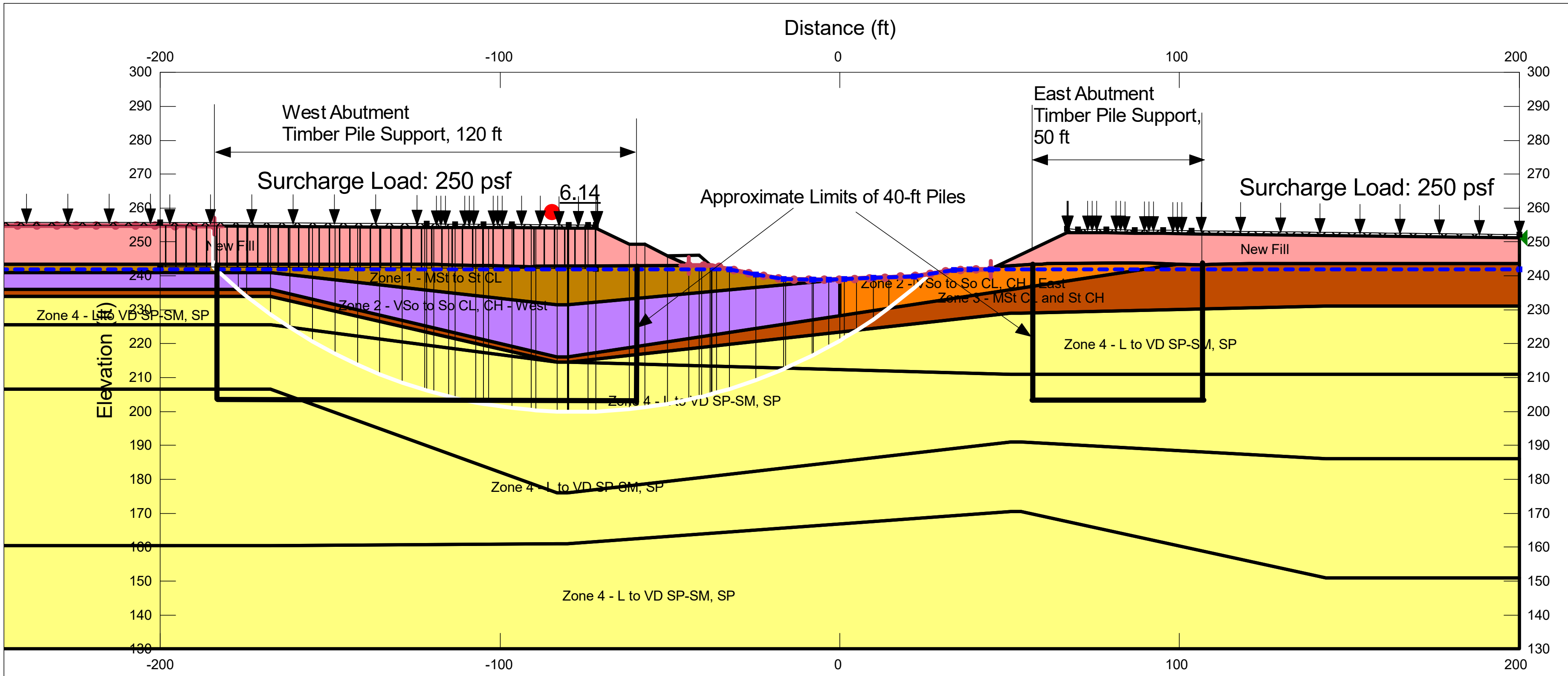


Figure 17 - Seismic Compression for S2-4 (Top of Boring at EL 249 ft)



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - MSt to St CL	Mohr-Coulomb	123	830	0
🟧	Zone 2 - VSo to So CL, CH - East	Mohr-Coulomb	125	280	0
🟪	Zone 2 - VSo to So CL, CH - West	Mohr-Coulomb	125	200	0
🟫	Zone 3 - MSt CL and St CH	Mohr-Coulomb	125	1,300	0
🟨	Zone 4 - L to VD SP-SM, SP	Mohr-Coulomb	120	0	35

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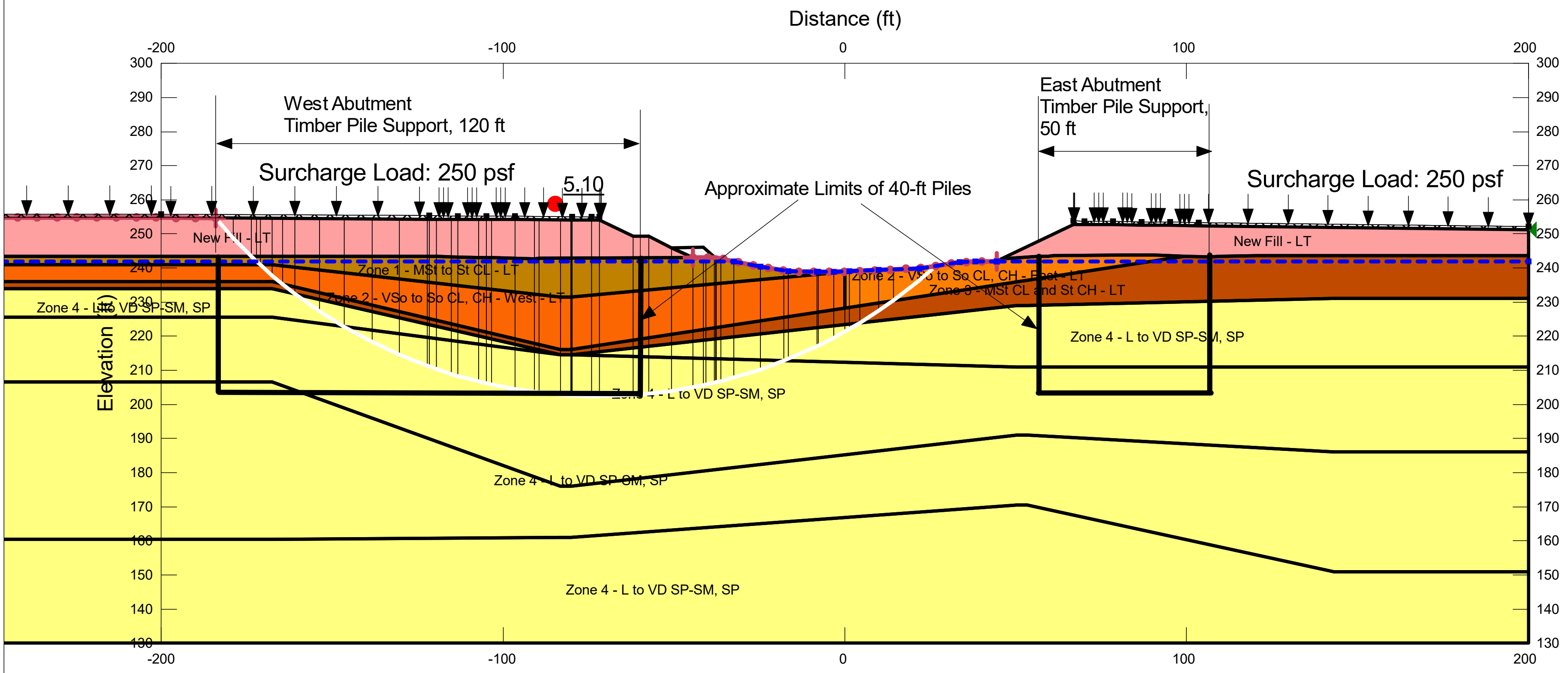
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

West Abutment Spill-Through
End of Construction

SR 230 Site 2
Craighead County, AR

Figure 18



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
🟫	Zone 1 - MSt to St CL - LT	Mohr-Coulomb	123	50	27
🟧	Zone 2 - VSo to So CL, CH - East - LT	Mohr-Coulomb	125	0	25
🟧	Zone 2 - VSo to So CL, CH - West - LT	Mohr-Coulomb	125	0	25
🟫	Zone 3 - MSt CL and St CH - LT	Mohr-Coulomb	125	0	26
🟨	Zone 4 - L to VD SP-SM, SP	Mohr-Coulomb	120	0	35

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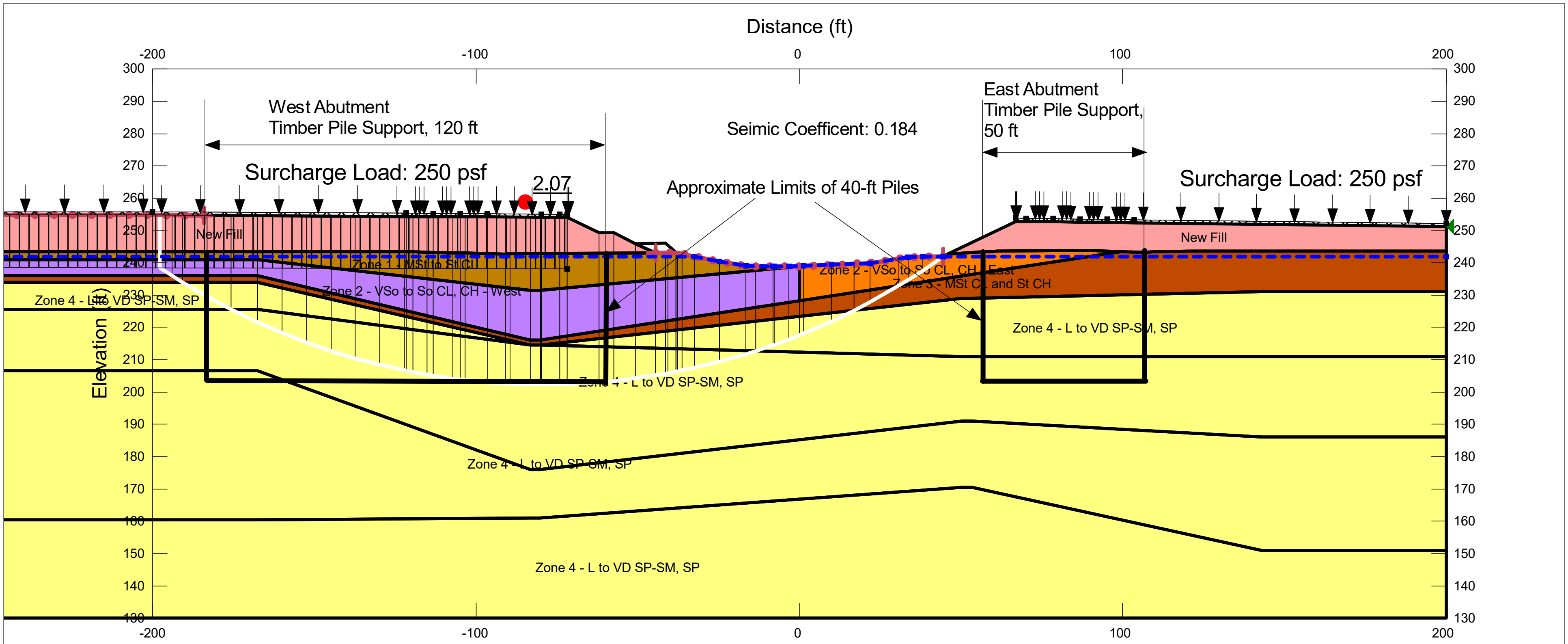
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

West Abutment Spill-Through
Long Term

SR 230 Site 2
Craighead County, AR

Figure 19



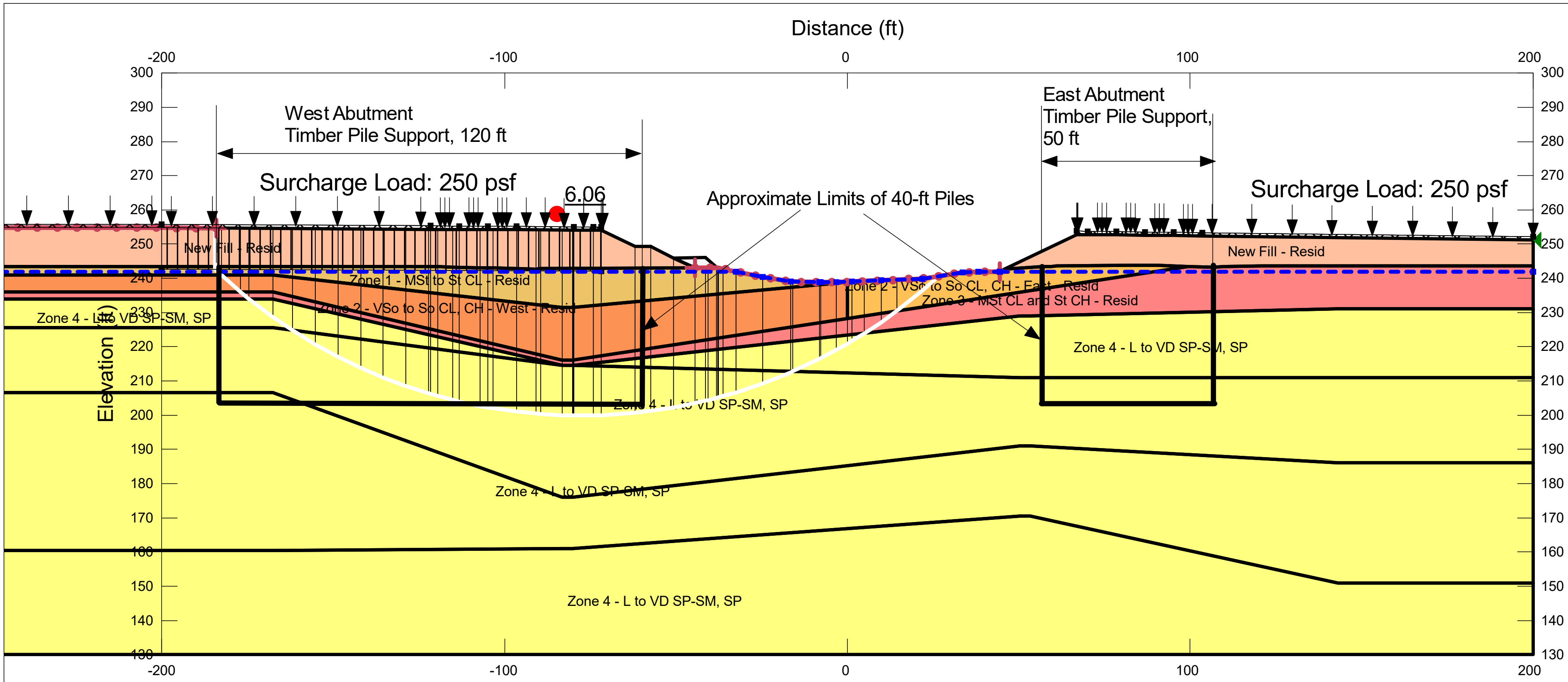
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - MSt to St CL	Mohr-Coulomb	123	830	0
🟧	Zone 2 - VSo to So CL, CH - East	Mohr-Coulomb	125	280	0
🟪	Zone 2 - VSo to So CL, CH - West	Mohr-Coulomb	125	200	0
🟫	Zone 3 - MSt CL and St CH	Mohr-Coulomb	125	1,300	0
🟨	Zone 4 - L to VD SP-SM, SP	Mohr-Coulomb	120	0	35

West Abutment Spill-Through Pseudostatic

SR 230 Site 2
Craighead County, AR

Figure 20

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



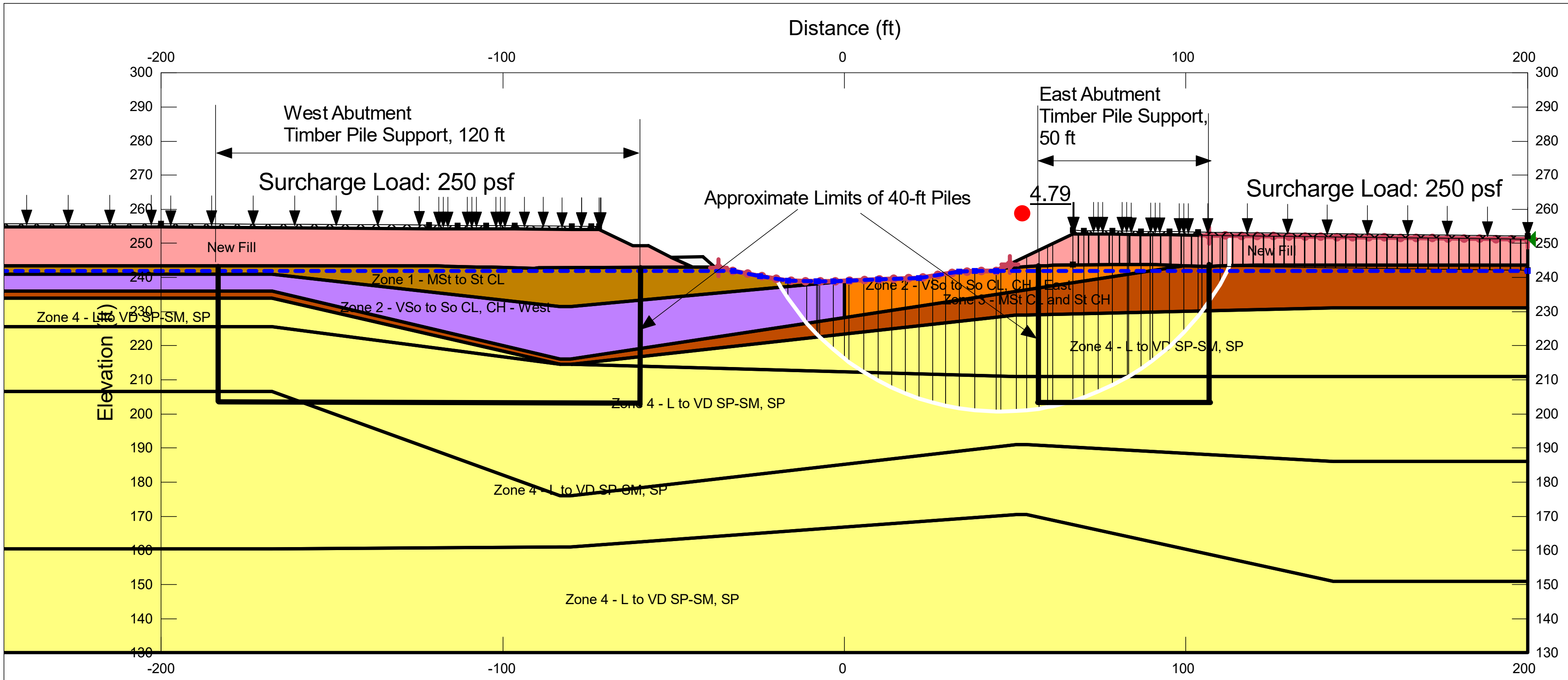
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
🟫	Zone 1 - MSt to St CL - Resid	Mohr-Coulomb	123	664	0
🟧	Zone 2 - VSo to So CL, CH - East - Resid	Mohr-Coulomb	125	224	0
🟧	Zone 2 - VSo to So CL, CH - West - Resid	Mohr-Coulomb	125	88	0
🟥	Zone 3 - MSt CL and St CH - Resid	Mohr-Coulomb	125	1,040	0
🟨	Zone 4 - L to VD SP-SM, SP	Mohr-Coulomb	120	0	35

West Abutment Spill-Through
Post-Seismic

SR 230 Site 2
Craighead County, AR

Figure 21

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



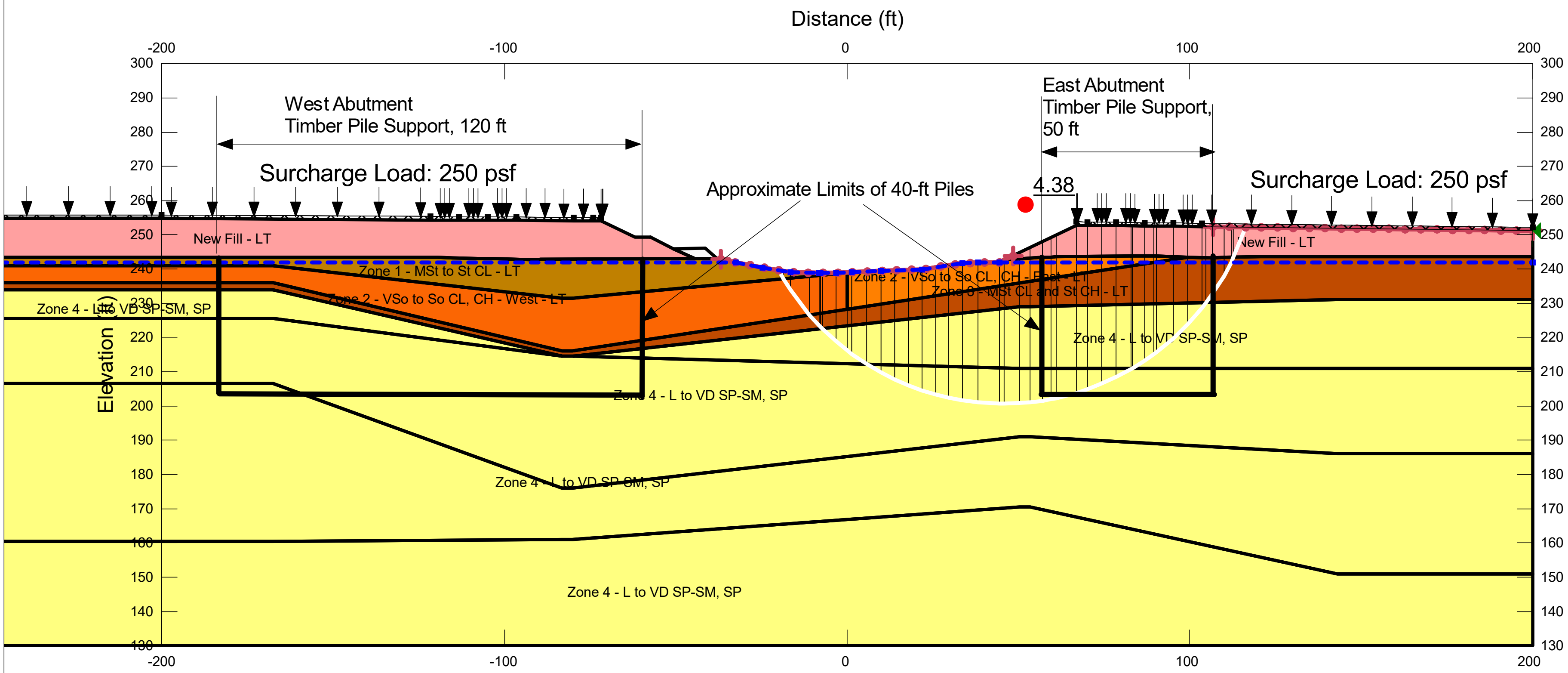
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - MSt to St CL	Mohr-Coulomb	123	830	0
🟧	Zone 2 - VSo to So CL, CH - East	Mohr-Coulomb	125	280	0
🟪	Zone 2 - VSo to So CL, CH - West	Mohr-Coulomb	125	200	0
🟫	Zone 3 - MSt CL and St CH	Mohr-Coulomb	125	1,300	0
🟨	Zone 4 - L to VD SP-SM, SP	Mohr-Coulomb	120	0	35

East Abutment Spill-Through
End of Construction

SR 230 Site 2
Craighead County, AR

Figure 22

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



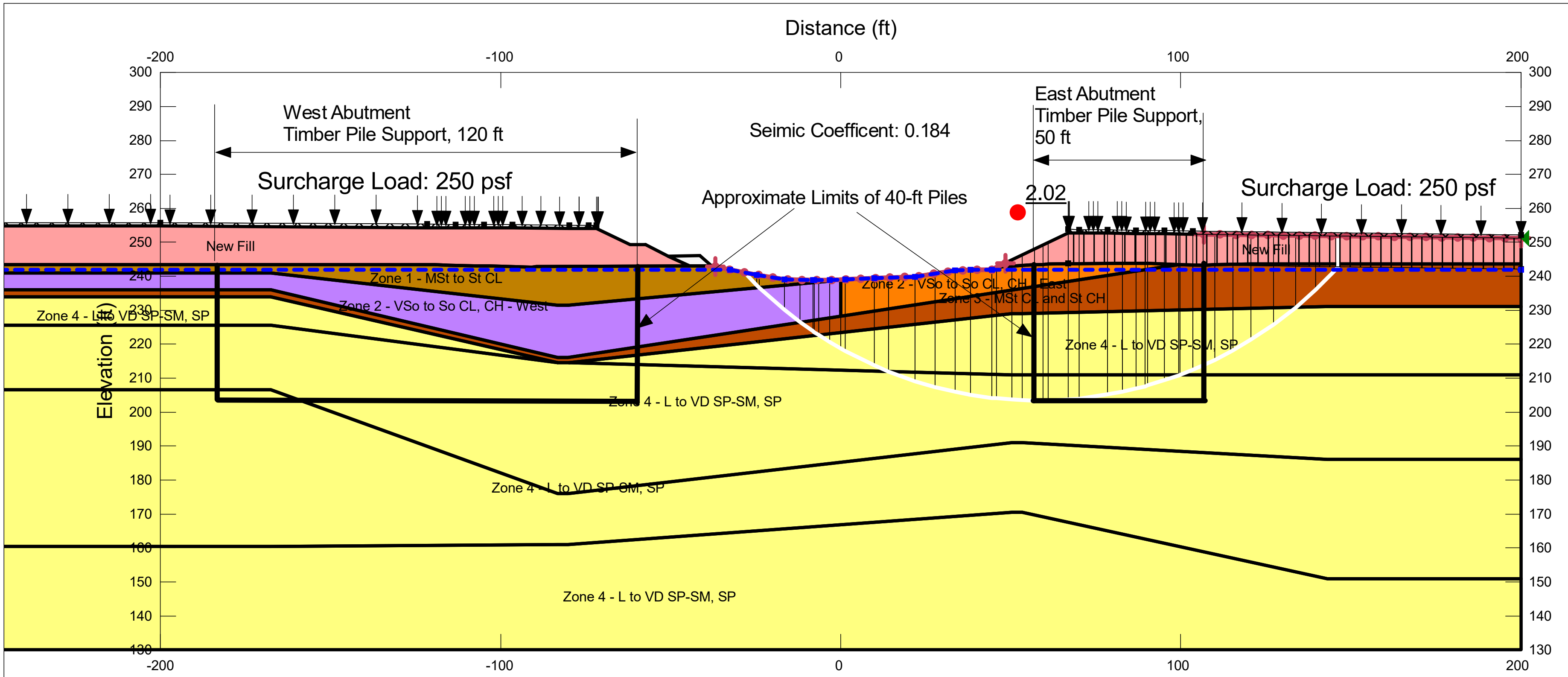
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
🟫	Zone 1 - MSt to St CL - LT	Mohr-Coulomb	123	50	27
🟧	Zone 2 - VSo to So CL, CH - East - LT	Mohr-Coulomb	125	0	25
🟧	Zone 2 - VSo to So CL, CH - West - LT	Mohr-Coulomb	125	0	25
🟫	Zone 3 - MSt CL and St CH - LT	Mohr-Coulomb	125	0	26
🟨	Zone 4 - L to VD SP-SM, SP	Mohr-Coulomb	120	0	35

East Abutment Spill-Through
Long Term

SR 230 Site 2
Craighead County, AR

Figure 23

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



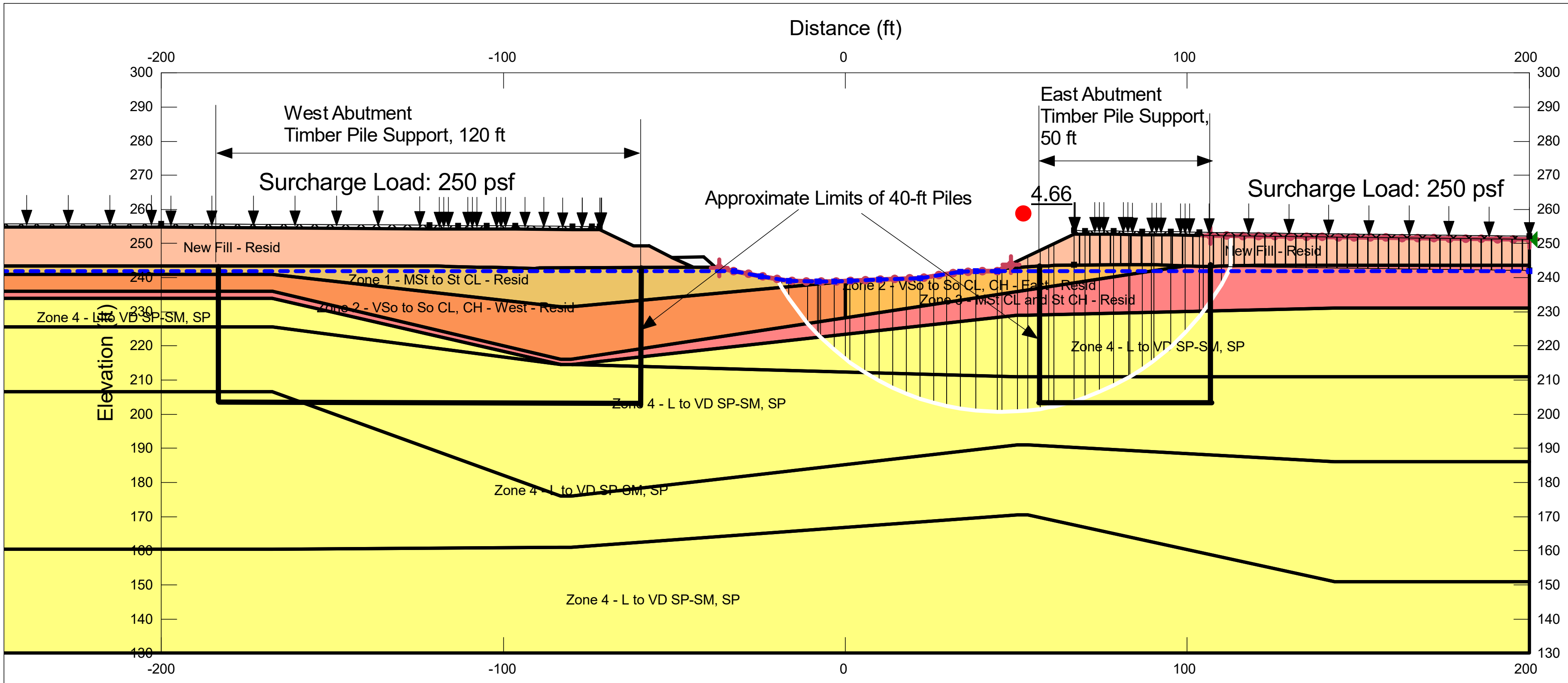
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - MSt to St CL	Mohr-Coulomb	123	830	0
🟧	Zone 2 - VSo to So CL, CH - East	Mohr-Coulomb	125	280	0
🟪	Zone 2 - VSo to So CL, CH - West	Mohr-Coulomb	125	200	0
🟫	Zone 3 - MSt CL and St CH	Mohr-Coulomb	125	1,300	0
🟨	Zone 4 - L to VD SP-SM, SP	Mohr-Coulomb	120	0	35

East Abutment Spill-Through
Pseudostatic

SR 230 Site 2
Craighead County, AR

Figure 24

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
🟫	Zone 1 - MSt to St CL - Resid	Mohr-Coulomb	123	664	0
🟧	Zone 2 - VSo to So CL, CH - East - Resid	Mohr-Coulomb	125	224	0
🟧	Zone 2 - VSo to So CL, CH - West - Resid	Mohr-Coulomb	125	88	0
🟥	Zone 3 - MSt CL and St CH - Resid	Mohr-Coulomb	125	1,040	0
🟨	Zone 4 - L to VD SP-SM, SP	Mohr-Coulomb	120	0	35

East Abutment Spill-Through
Post-Seismic

SR 230 Site 2
Craighead County, AR

Figure 25

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

GeogridBridge2.0 analyzes column-supported embankments with geosynthetic-reinforced bridging layers.
The complete report by Filz and Smith (2006), plus all Main sheet comments, as well as the CGPR report by
Sloan et al. (2014) should be read before using this workbook.
Provide the input data in the cells with red text. The cells in blue text are the calculated results based on the input data.
Definition sketches are provided in Figs. 1 through 6, which are located to the right.
Guidance information for material property values is provided in the pdf document to the right.
After providing all the proper input data, use the "Solve" button located at Cell B74.

Materials Properties

ARDOT Site 2

	Bridging Layer Fill	Embankment Fill #2	Preload
Layer Thickness, H (ft)	2.0	10.0	0.0
Total Unit Weight, γ (pcf)	135	120	110
Friction Angle, ϕ (deg)	38	28	N/A
Lateral Earth Pressure Coefficient, K	0.75	0.75	N/A
Young's Modulus, E (psf)	750,000	300,000	N/A
Poisson's Ratio, ν	0.30	0.33	N/A

Pavement Plus Traffic Surcharge Pressure, q (psf)	250

Time Available for Consolidation, t (days)	0
Allowable Post-Construction Settlement, S_A (in.)	5.0

Depth to Groundwater, d_w (ft)	1.0
Unit Weight of Groundwater, γ_w (pcf)	62.4

	Exist Sand #1	Exist Sand #2	Clay #1	Clay #2
Layer Thickness, H (ft)	0.0	0.0	11.0	15.0
Total Unit Weight, γ (pcf)	125	125	122	125
Young's Modulus, E (psf)	250,000	250,000	N/A	N/A
Poisson's Ratio, ν	0.33	0.30	0.35	0.35
Lat. Earth Press. Coeff., K_0	0.50	0.50	0.60	0.60
Interface Frict. Angle btwn Soil and Column, δ (deg)	32	32	13	13
Compression Ratio, $C_{\varepsilon c}$	N/A	N/A	0.169	0.129
Recompression Ratio, $C_{\varepsilon r}$	N/A	N/A	0.017	0.013
Coeff. of Consol., c_v (ft^2/day)	N/A	N/A	0.10	
Initial Eff. Vert. Stress at Top of Layer, $\sigma'_{v,top}$ (psf)	N/A	N/A	0	718
Preconsol. Press. at Top of Layer, $p_{p,top}$ (psf)	N/A	N/A	2000	718
Initial Eff. Vert. Stress at Bottom of Layer, $\sigma'_{v,bot}$ (psf)	N/A	N/A	718	1657
Preconsol. Press. at Bottom of Layer, $p_{p,bot}$ (psf)	N/A	N/A	2655.6	1657

	Biaxial Goegrid		Triaxial Geogrid
	Machine Direction	Cross-Machine Direction	
Type of Geosynthetic (use B for biaxial or T for triaxial)	B		
Stiffness of a Single Geogrid Layer (lb/ft)	28,800	28,800	16,000
Allowable Strength of a Single Geogrid Layer (lb/ft)	1,440	1,440	667
Number of Geogrid Layers	3		
Combined Geogrid Stiffness, J (lb/ft)	86,400	86,400	48,000
Combined Allowable Geogrid Strength, S_g (lb/ft)	4,320	4,320	2,001

	Pile Cap	Column
Vertical Distance from Top to Bottom of Element, H (ft)	0.0	26.0
Column Shape (use R for round and S for square)	R	R
Column Diameter or Width, d_c or a (ft)	1.0	1.0
Young's Modulus, E (psf)	216,000,000	216,000,000
Poisson's Ratio, ν	0.30	0.30
Column/Pile Cap Arrangement (use S for square/rectangular as in Fig. 5, or T for triangular as in Fig. 6)	s	
Center-to-Center Spacing, s_1 (ft)	4.3	
Center-to-Center Spacing, s_2 (ft)	4.3	

	Calc. Values	Criteria
Clear Spacing, $s - a$ (ft)	3.4	≤ 8.0
Area Replacement Ratio at Ground Surface, a_s	0.043	≥ 0.10
Bridging Layer Thickness, H_b (ft)	2.0	≥ 2.0
Total Embankment Height, $H_b + H_{emb\#2} \geq H_{crit}$ (ft)	12.0	≥ 4.3
Maximum Differential Settlement of Geogrid, d (in.)	5.2	N/A
Geogrid Strain, ε_g	0.047	≤ 0.05
Tension in a Single Geogrid Layer (lb/ft)	1,347	≤ 1,440
Combined Tension in the Geogrid Layers, T_g (lb/ft)	4,041	≤ 4,320
Post-Construction Embankment Settlement, S (in.)	2.5	≤ 5.0

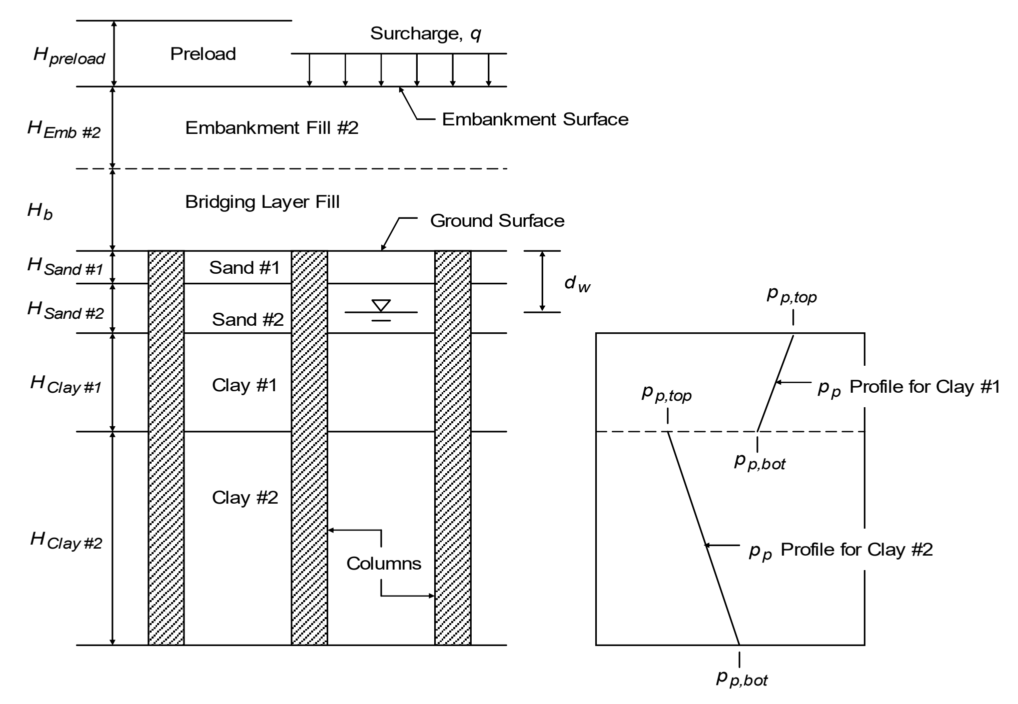


Fig. 1. Elevation View of Embankment, Columns, and Foundation Soil

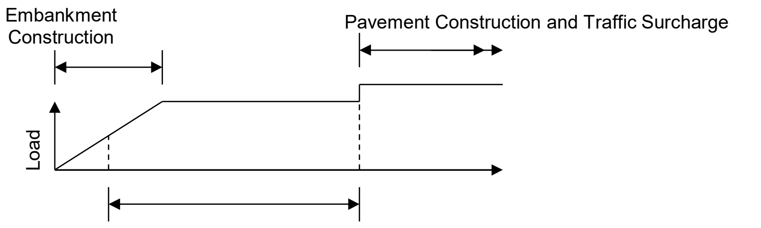


Fig. 2. Consolidation Time for Embankment Construction without Preload

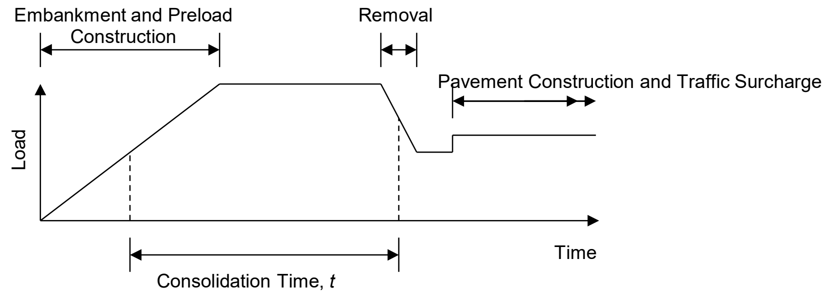


Fig. 3. Consolidation Time for Embankment Construction with Preload

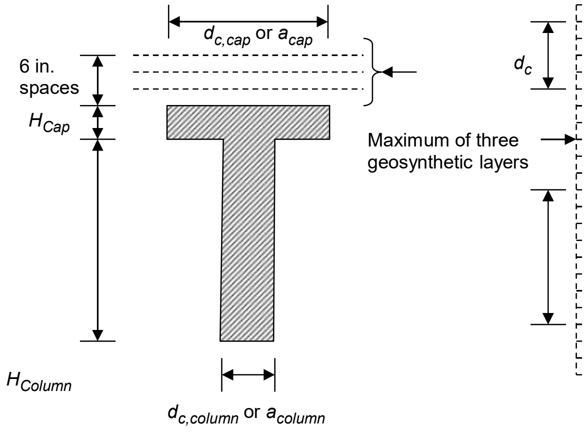


Fig. 4. Elevation View of Column with Pile Cap

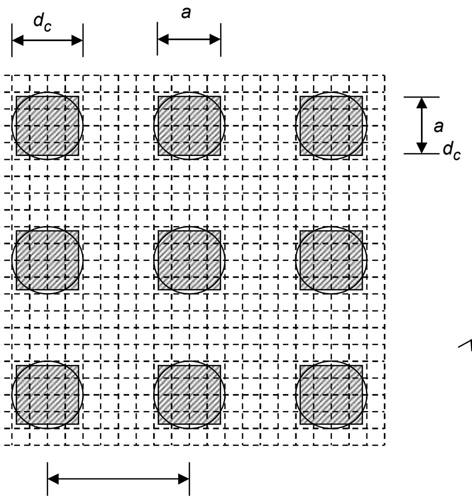


Fig. 5. Plan View of Columns or Pile Caps
(Square Column Arrangement)

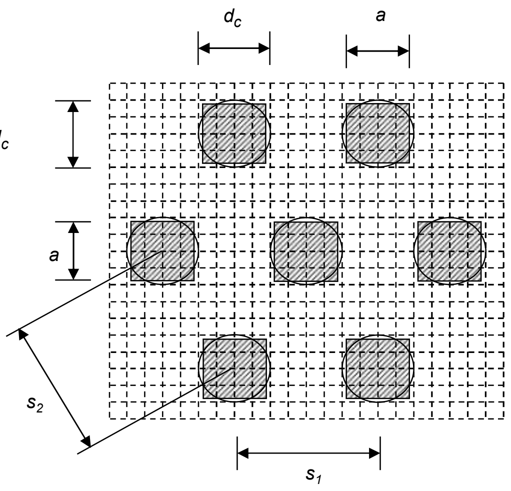
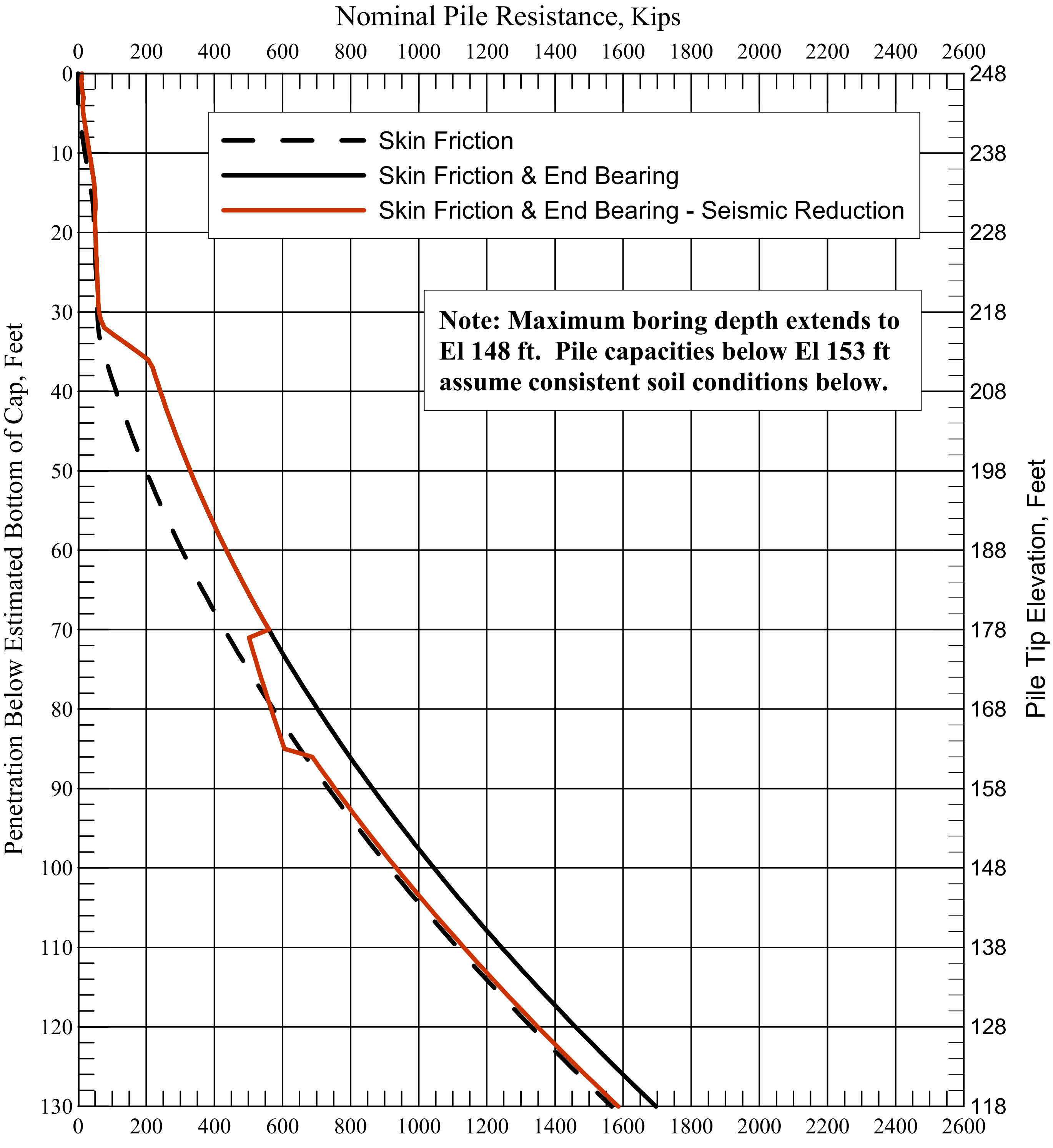
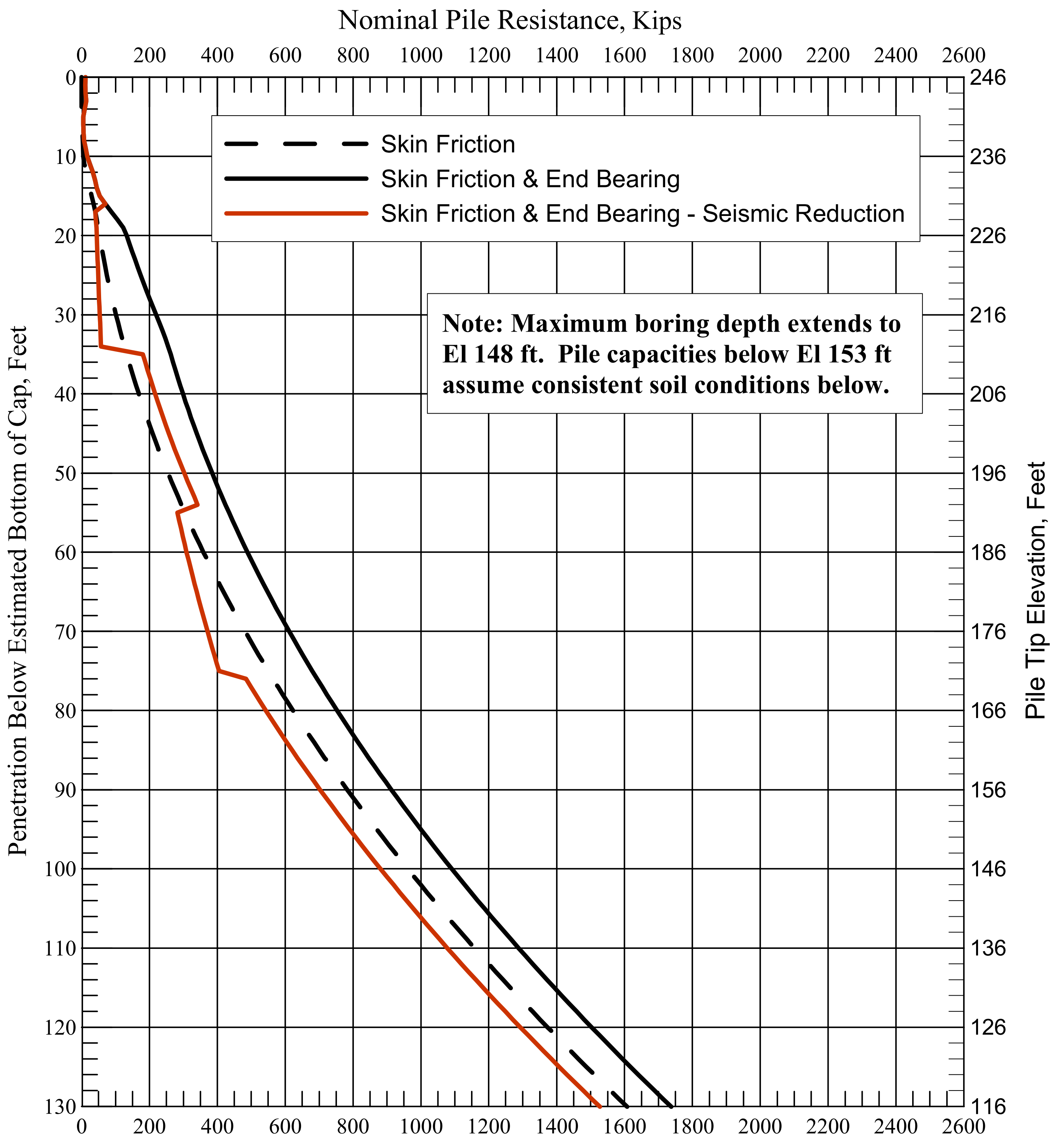


Fig. 6. Plan View of Columns or Pile Caps
(Triangular Column Arrangement)

Figure 26



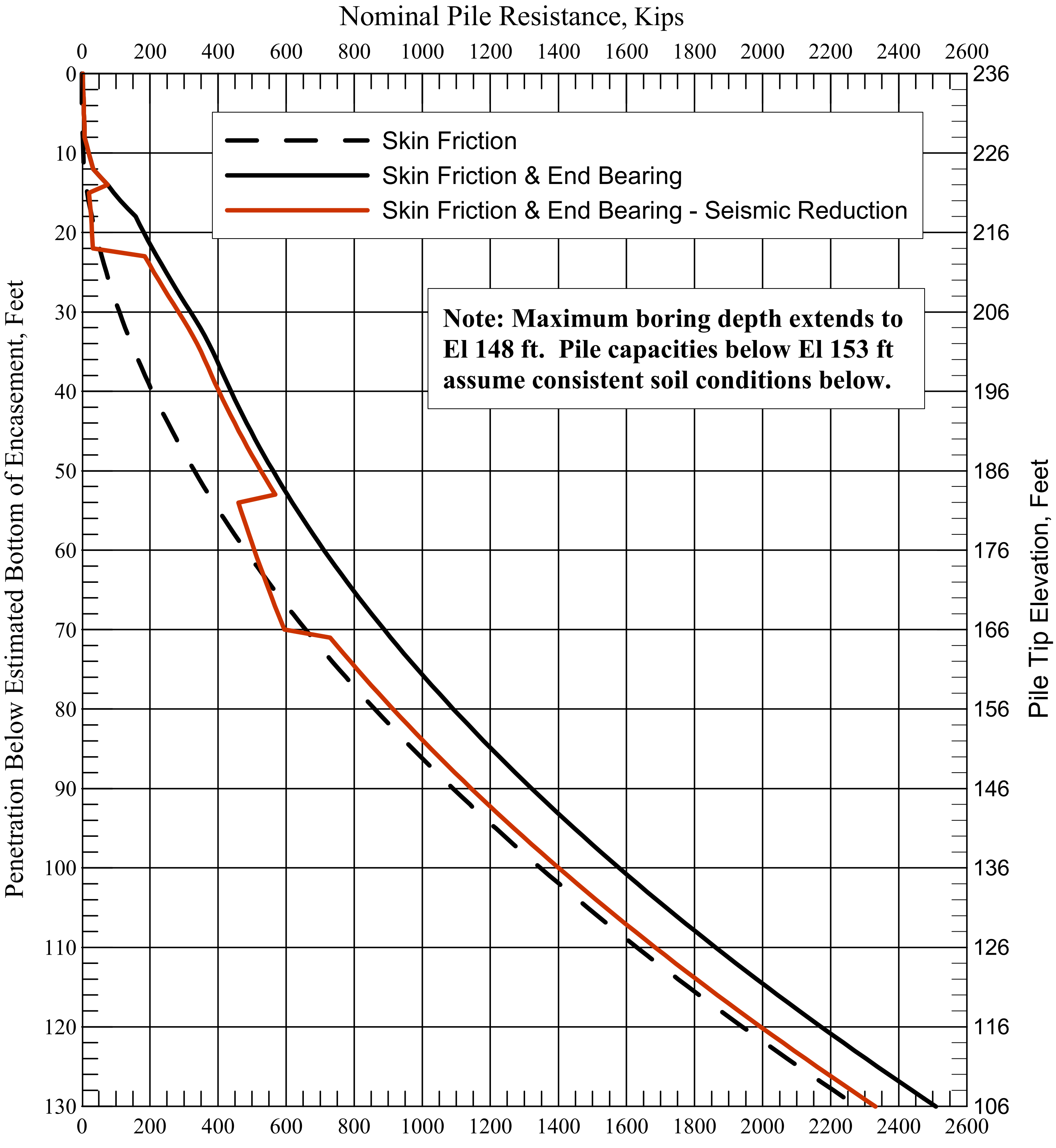
NOMINAL PILE RESISTANCE CURVES
SITE 2
WEST ABUTMENT
18" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS



Note: Maximum boring depth extends to El 148 ft. Pile capacities below El 153 ft assume consistent soil conditions below.

NOMINAL PILE RESISTANCE CURVES
SITE 2
EAST ABUTMENT
18" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

FIGURE 28

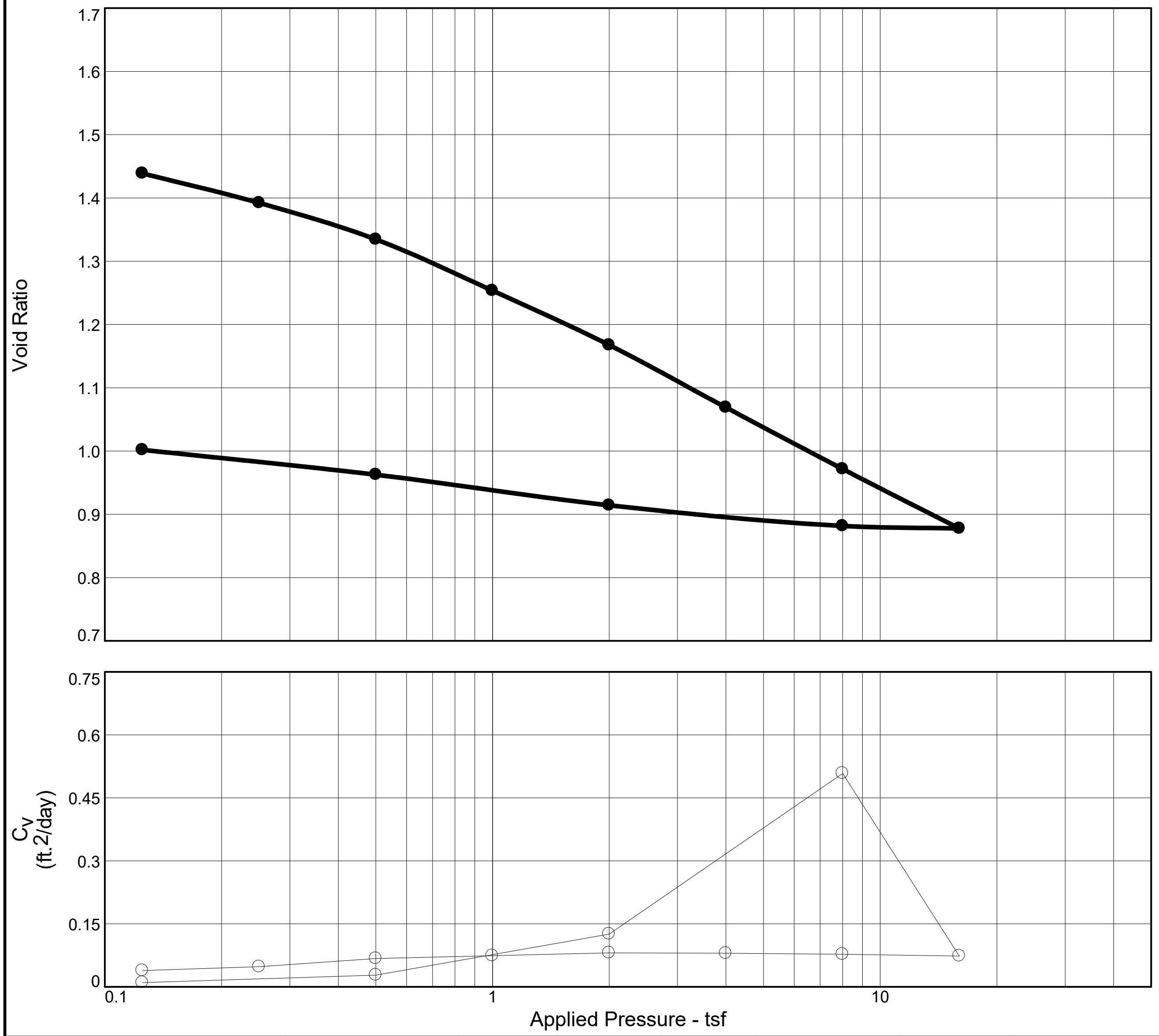


NOMINAL PILE RESISTANCE CURVES
SITE 2
INTERIOR BENTS
24" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

APPENDIX A

Consolidation Test Results
and
Particle Size Distribution Curves

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (tsf)	C_c	Initial Void Ratio
Saturation	Moisture							
74.3 %	40.5 %	68.4	51	35	2.72	0.7	0.32	1.483

MATERIAL DESCRIPTION	USCS	AASHTO
Very soft tan and light gray clay (CH), slightly silty	CH	

Project No. 200518 **Client:** ARDOT

Project: SR-230 Alicia to Bono, AR

Source of Sample: S2-2 **Depth:** 28.0' **Sample Number:** 9A

BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Remarks:

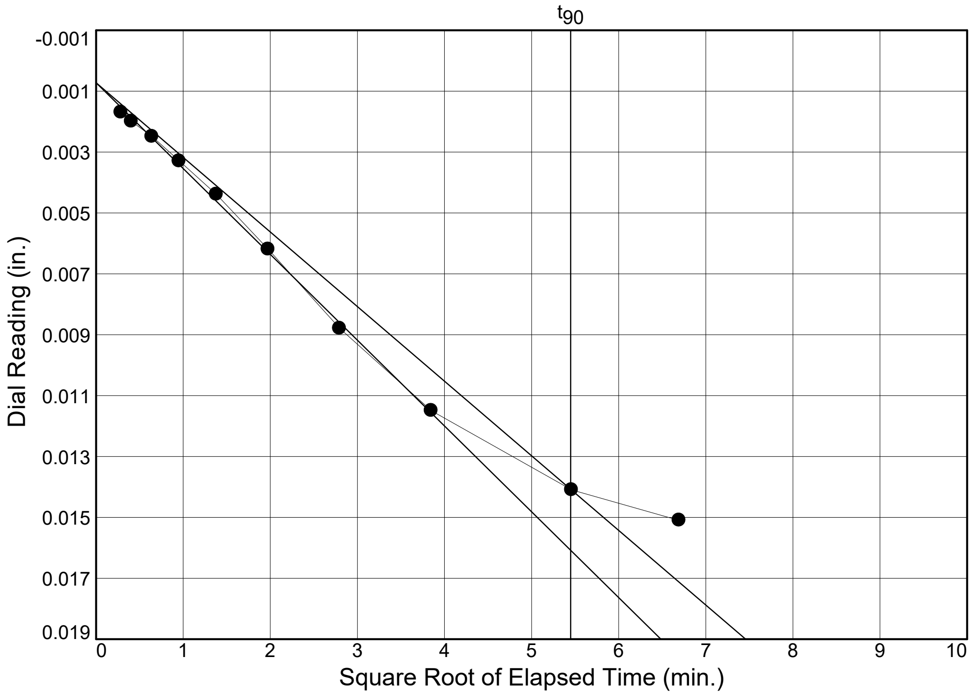
Figure

Checked By: ____________________

Dial Reading vs. Time

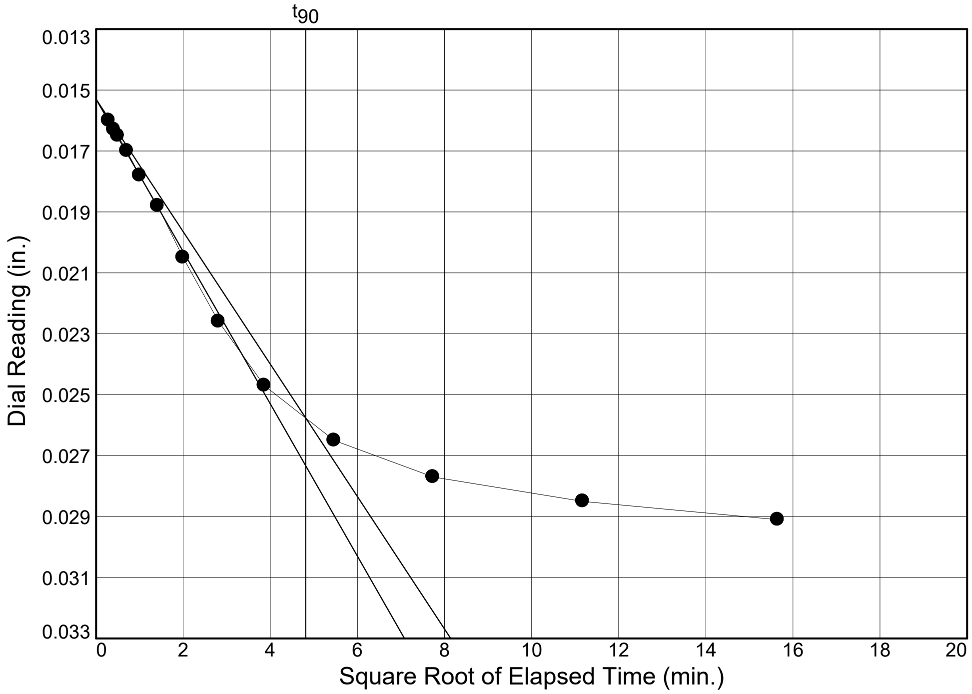
Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S2-2 Depth: 28.0' Sample Number: 9A



Load No.= 1
Load= 0.13 tsf
D_0 = 0.0007
D_{90} = 0.0141
D_{100} = 0.0156
T_{90} = 29.68 min.

C_v @ T_{90}
0.039 ft.^{2}/day



Load No.= 2
Load= 0.25 tsf
D_0 = 0.0153
D_{90} = 0.0258
D_{100} = 0.0269
T_{90} = 23.17 min.

C_v @ T_{90}
0.048 ft.^{2}/day

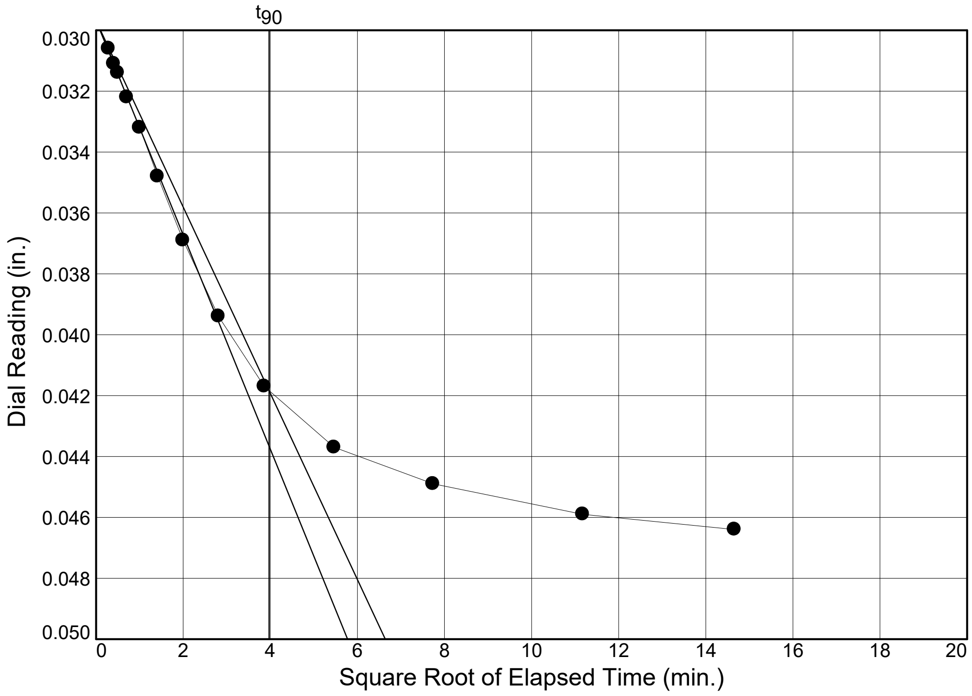
Figure

BURNS COOLEY DENNIS, INC.

Dial Reading vs. Time

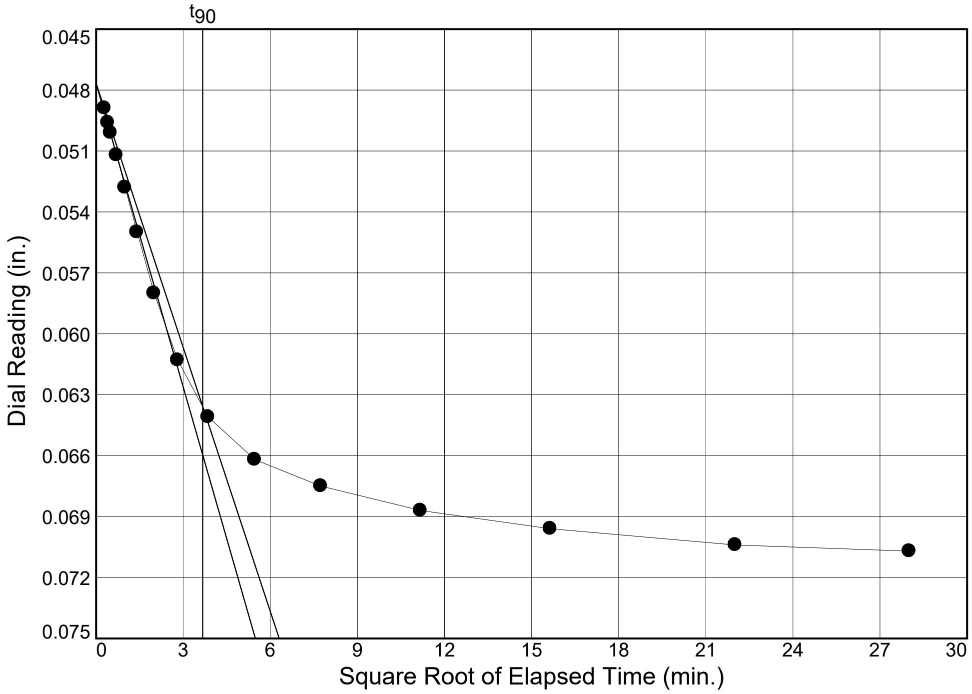
Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S2-2 Depth: 28.0' Sample Number: 9A



Load No.= 3
Load= 0.50 tsf
D_0 = 0.0297
D_{90} = 0.0418
D_{100} = 0.0432
T_{90} = 15.79 min.

C_v @ T_{90}
0.068 ft.^{2}/day



Load No.= 4
Load= 1.00 tsf
D_0 = 0.0477
D_{90} = 0.0636
D_{100} = 0.0654
T_{90} = 13.51 min.

C_v @ T_{90}
0.074 ft.^{2}/day

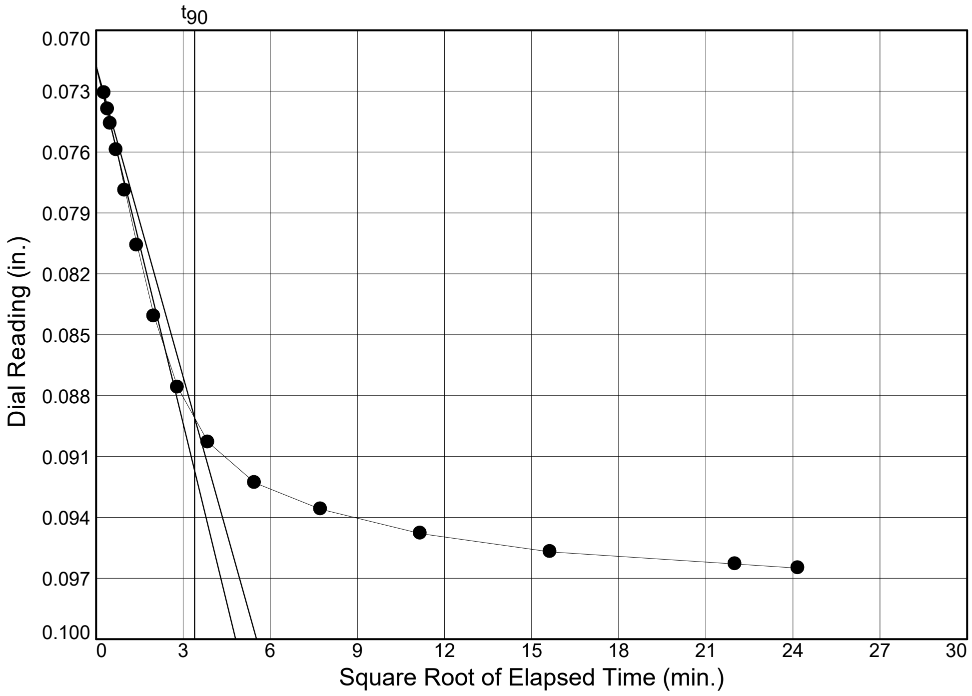
Figure

BURNS COOLEY DENNIS, INC.

Dial Reading vs. Time

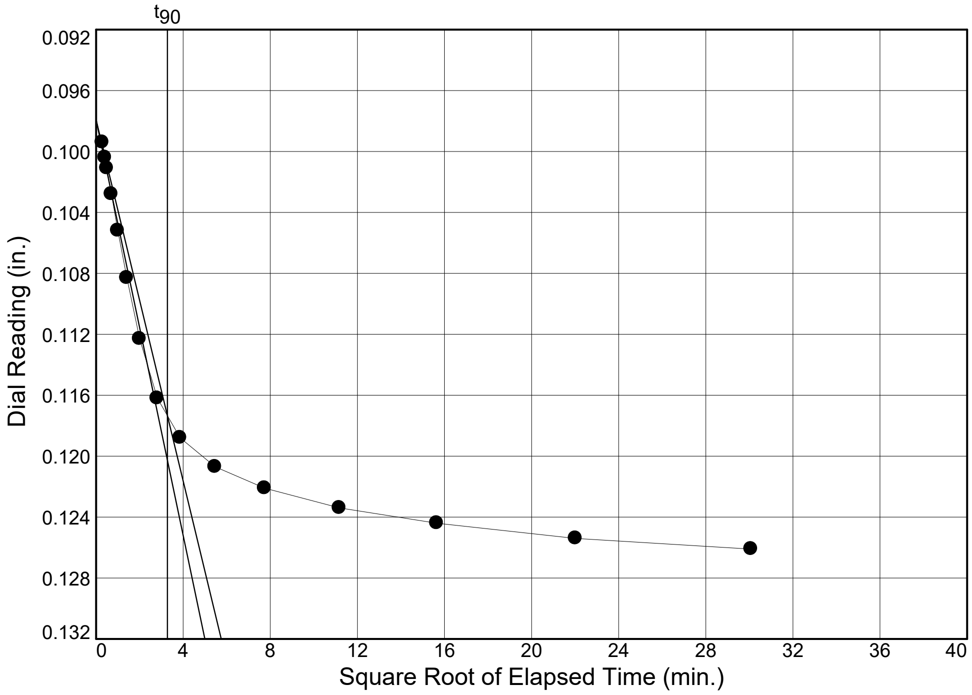
Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S2-2 Depth: 28.0' Sample Number: 9A



Load No.= 5
Load= 2.00 tsf
D_0 = 0.0718
D_{90} = 0.0891
D_{100} = 0.0910
T_{90} = 11.50 min.

C_v @ T_{90}
0.081 ft.^{2}/day



Load No.= 6
Load= 4.00 tsf
D_0 = 0.0979
D_{90} = 0.1173
D_{100} = 0.1195
T_{90} = 10.71 min.

C_v @ T_{90}
0.080 ft.^{2}/day

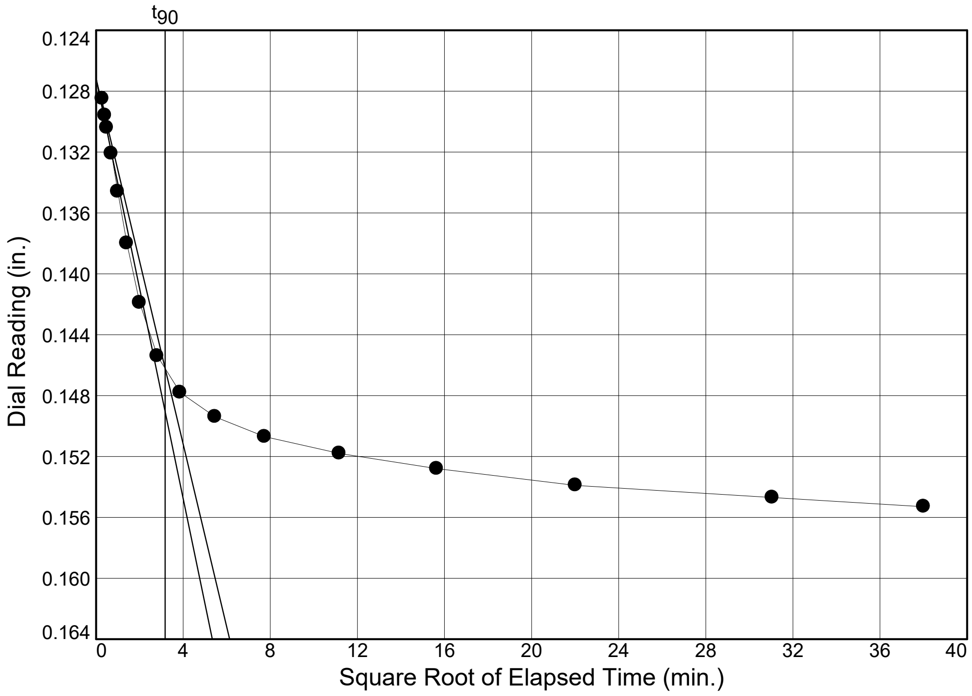
Figure

BURNS COOLEY DENNIS, INC.

Dial Reading vs. Time

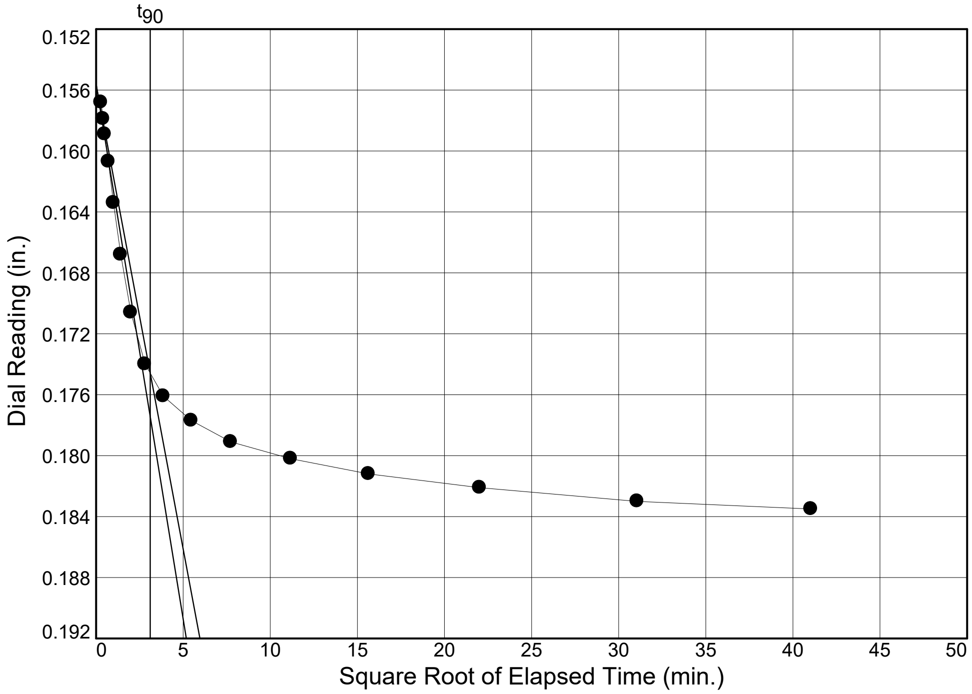
Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S2-2 Depth: 28.0' Sample Number: 9A



Load No.= 7
Load= 8.00 tsf
D_0 = 0.1272
D_{90} = 0.1462
D_{100} = 0.1483
T_{90} = 10.07 min.

C_v @ T_{90}
0.077 ft.^{2}/day



Load No.= 8
Load= 16.00 tsf
D_0 = 0.1556
D_{90} = 0.1746
D_{100} = 0.1767
T_{90} = 9.65 min.

C_v @ T_{90}
0.073 ft.^{2}/day

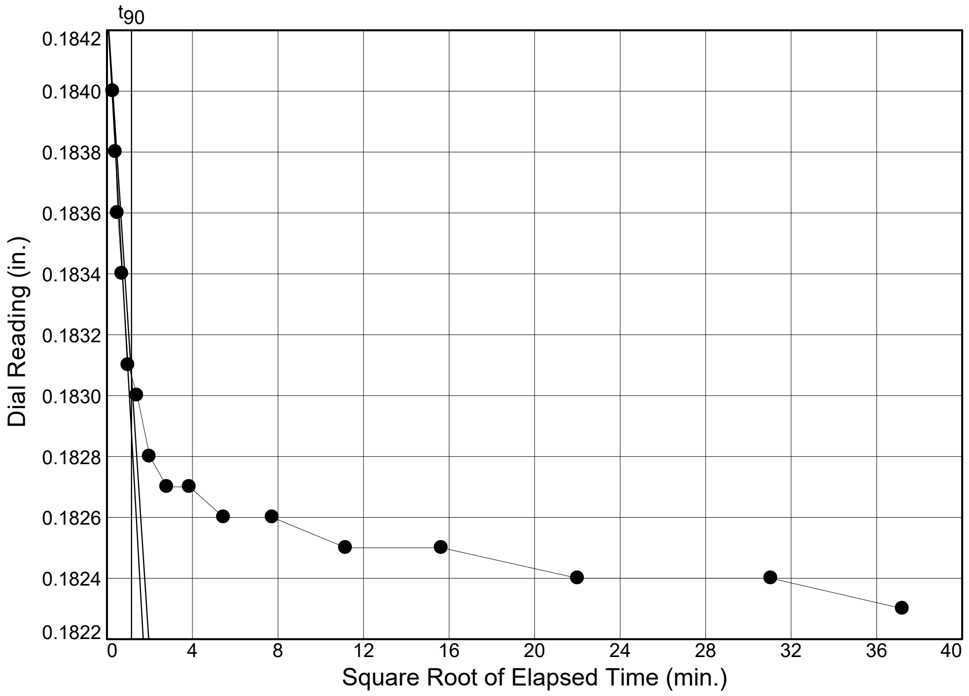
Figure

BURNS COOLEY DENNIS, INC.

Dial Reading vs. Time

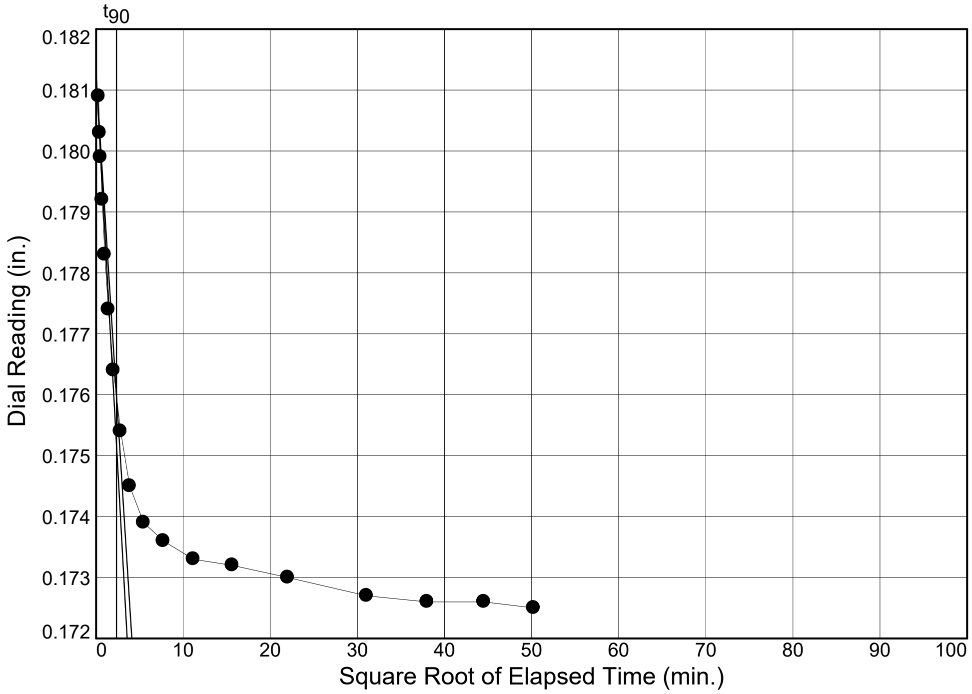
Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S2-2 Depth: 28.0' Sample Number: 9A



Load No.= 9
Load= 8.00 tsf
D_0 = 0.1843
D_{90} = 0.1831
D_{100} = 0.1829
T_{90} = 1.32 min.

C_v @ T_{90}
0.508 ft.^{2}/day



Load No.= 10
Load= 2.00 tsf
D_0 = 0.1813
D_{90} = 0.1760
D_{100} = 0.1754
T_{90} = 5.47 min.

C_v @ T_{90}
0.126 ft.^{2}/day

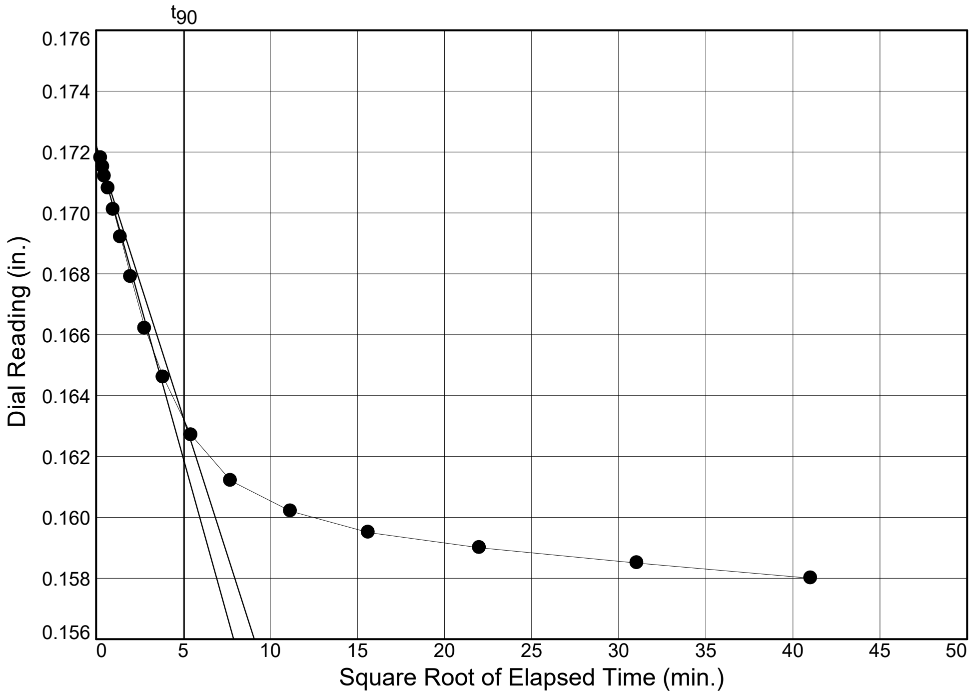
Figure

BURNS COOLEY DENNIS, INC.

Dial Reading vs. Time

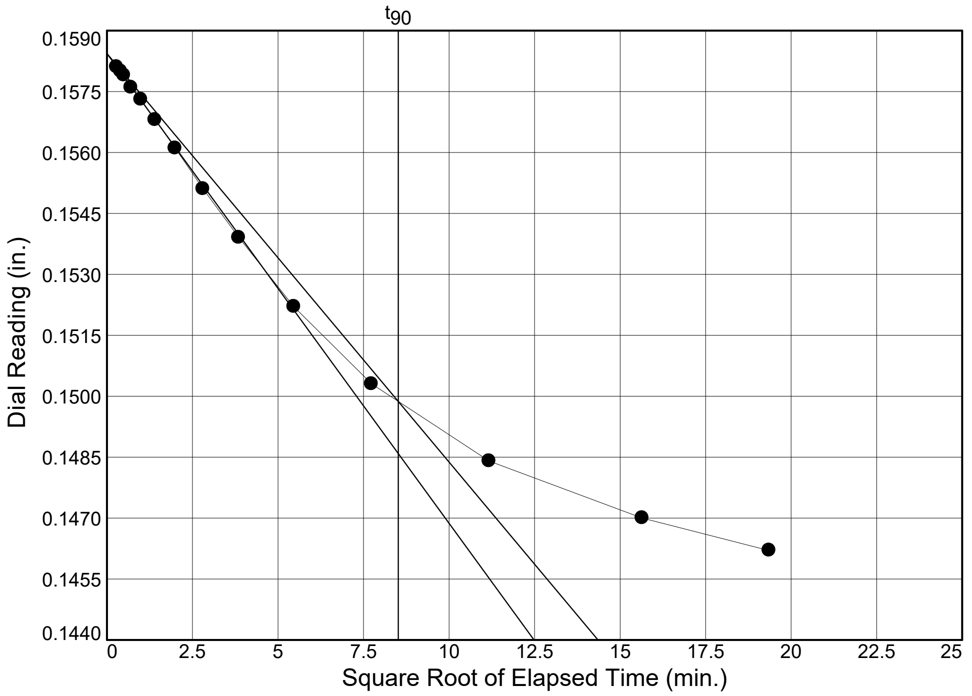
Project No.: 200518
Project: SR-230 Alicia to Bono, AR

Source of Sample: S2-2 Depth: 28.0' Sample Number: 9A



Load No.= 11
Load= 0.50 tsf
D_0 = 0.1722
D_{90} = 0.1632
D_{100} = 0.1622
T_{90} = 25.56 min.

C_v @ T_{90}
0.028 ft.^{2}/day



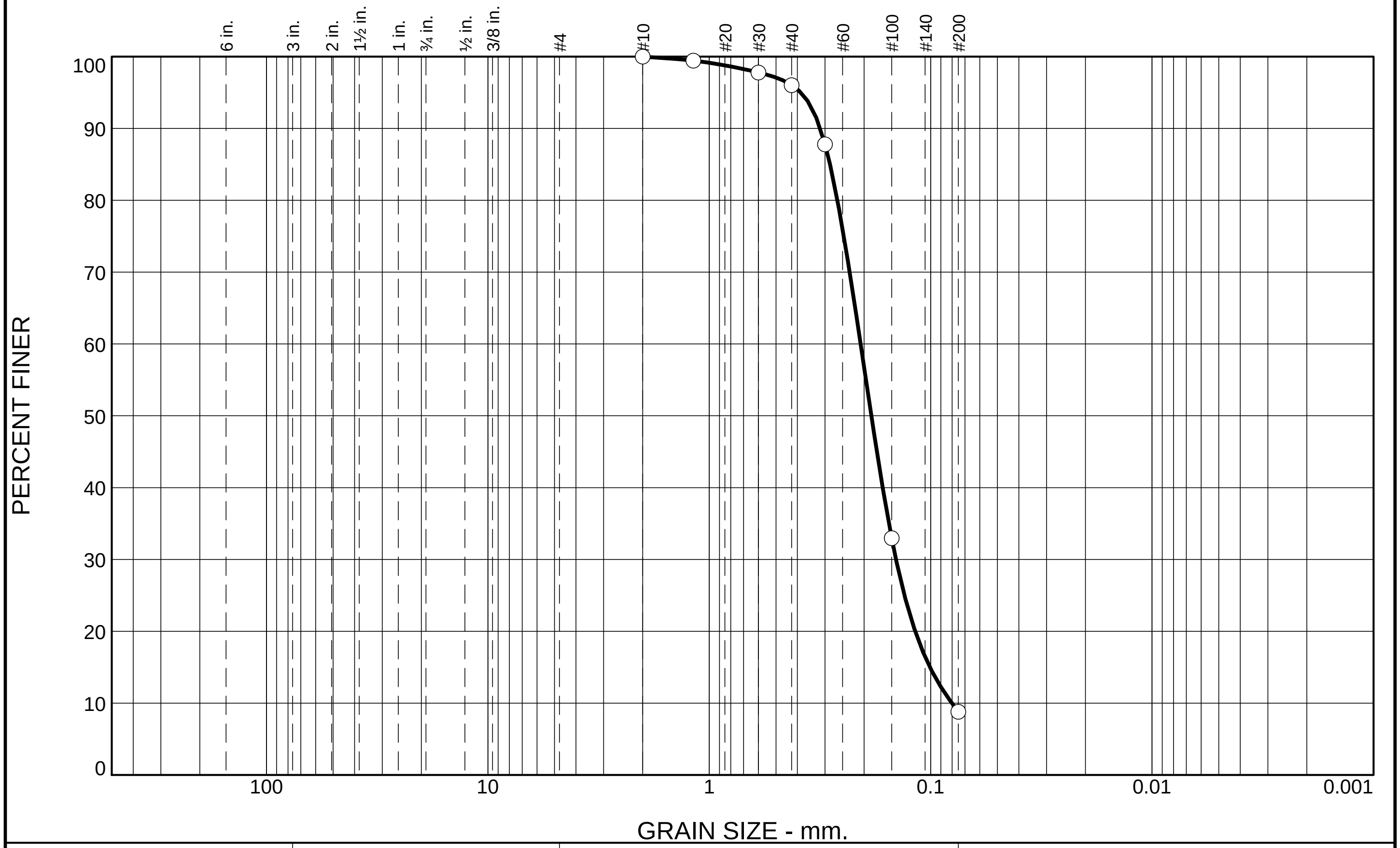
Load No.= 12
Load= 0.13 tsf
D_0 = 0.1584
D_{90} = 0.1499
D_{100} = 0.1489
T_{90} = 72.54 min.

C_v @ T_{90}
0.010 ft.^{2}/day

Figure

BURNS COOLEY DENNIS, INC.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	4.0	87.2	8.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.4		
#30	97.8		
#40	96.0		
#50	87.8		
#100	33.0		
#200	8.8		

* (no specification provided)

Soil Description

Gray fine to medium sand (SP-SM), slightly silty with organics

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3154 D_{85}= 0.2849 D_{60}= 0.2072

D_{50}= 0.1854 D_{30}= 0.1434 D_{15}= 0.1008

D_{10}= 0.0802 C_u= 2.58 C_c= 1.24

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: S2-1 **Depth:** 18.5'
Sample Number: 6 **Date:** 10-12-20

BURNS COOLEY DENNIS, INC.

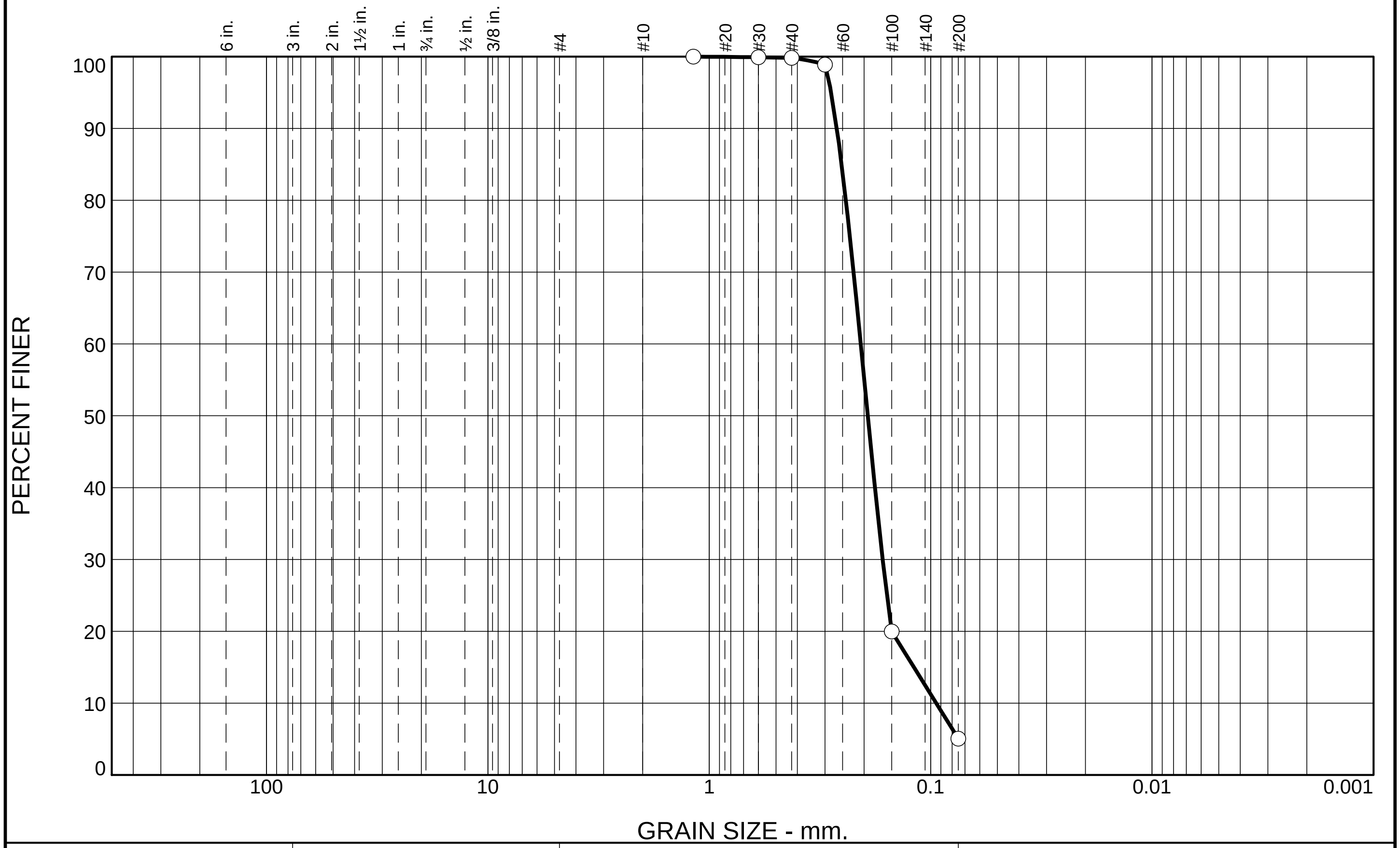
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: bb **Checked By:**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.2	94.8	5.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#16	100.0		
#30	99.9		
#40	99.8		
#50	98.9		
#100	20.0		
#200	5.0		

* (no specification provided)

Soil Description

Gray fine sand (SP-SM), slightly silty

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2652 D_{85}= 0.2524 D_{60}= 0.2070
D_{50}= 0.1924 D_{30}= 0.1649 D_{15}= 0.1191
D_{10}= 0.0944 C_u= 2.19 C_c= 1.39

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: S2-1 **Depth:** 28.5'
Sample Number: 9 **Date:** 10-12-20

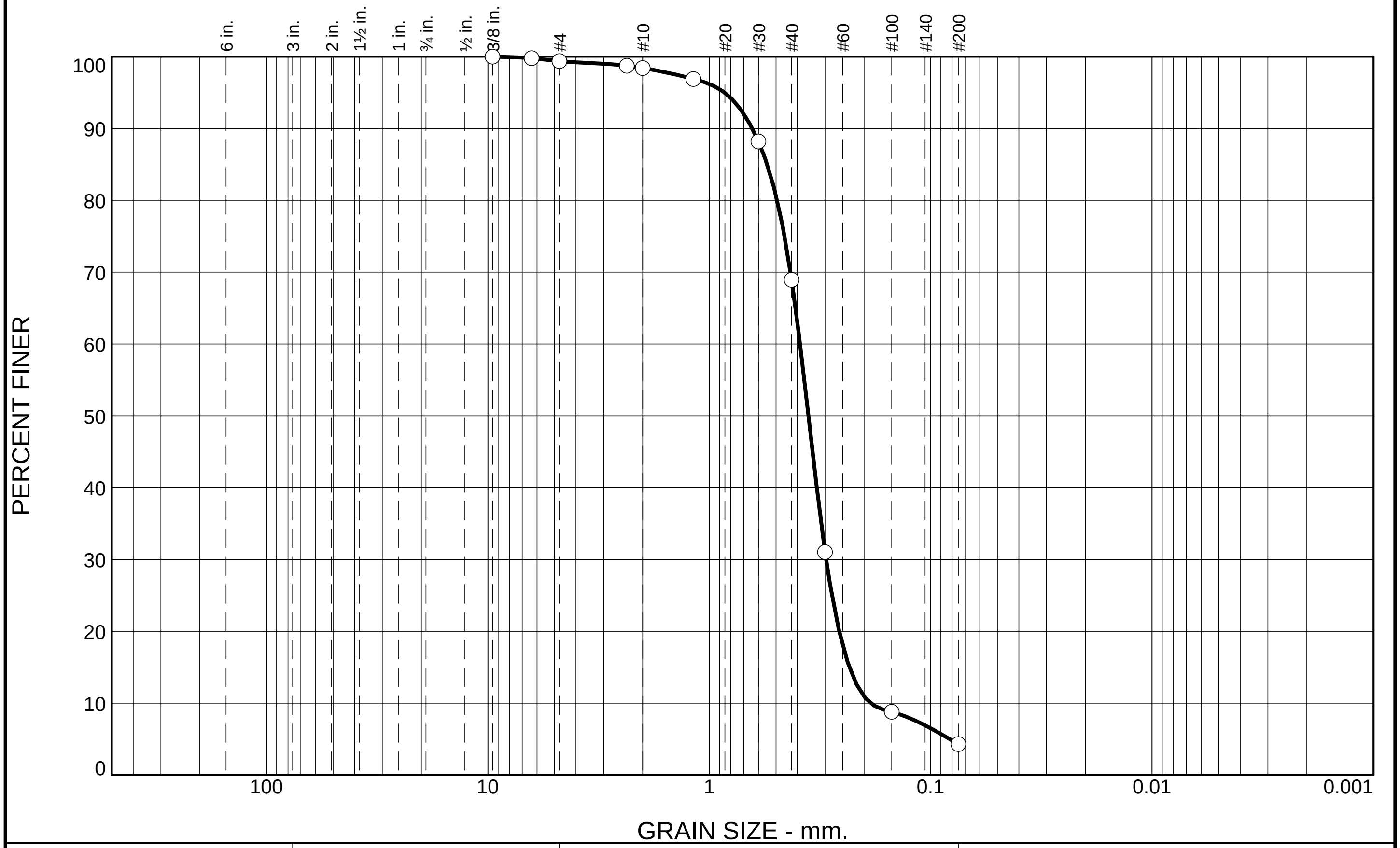
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: bb **Checked By:** ______

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.6	1.0	29.5	64.6	4.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100.0		
1/4	99.8		
#4	99.4		
#8	98.7		
#10	98.4		
#16	96.9		
#30	88.2		
#40	68.9		
#50	31.0		
#100	8.8		
#200	4.3		

* (no specification provided)

Soil Description

Gray and tan fine to medium sand (SP)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6394 D_{85}= 0.5479 D_{60}= 0.3895
D_{50}= 0.3570 D_{30}= 0.2967 D_{15}= 0.2331
D_{10}= 0.1870 C_u= 2.08 C_c= 1.21

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S2-1 **Depth:** 68.5'
Sample Number: 17 **Date:** 10-12-20

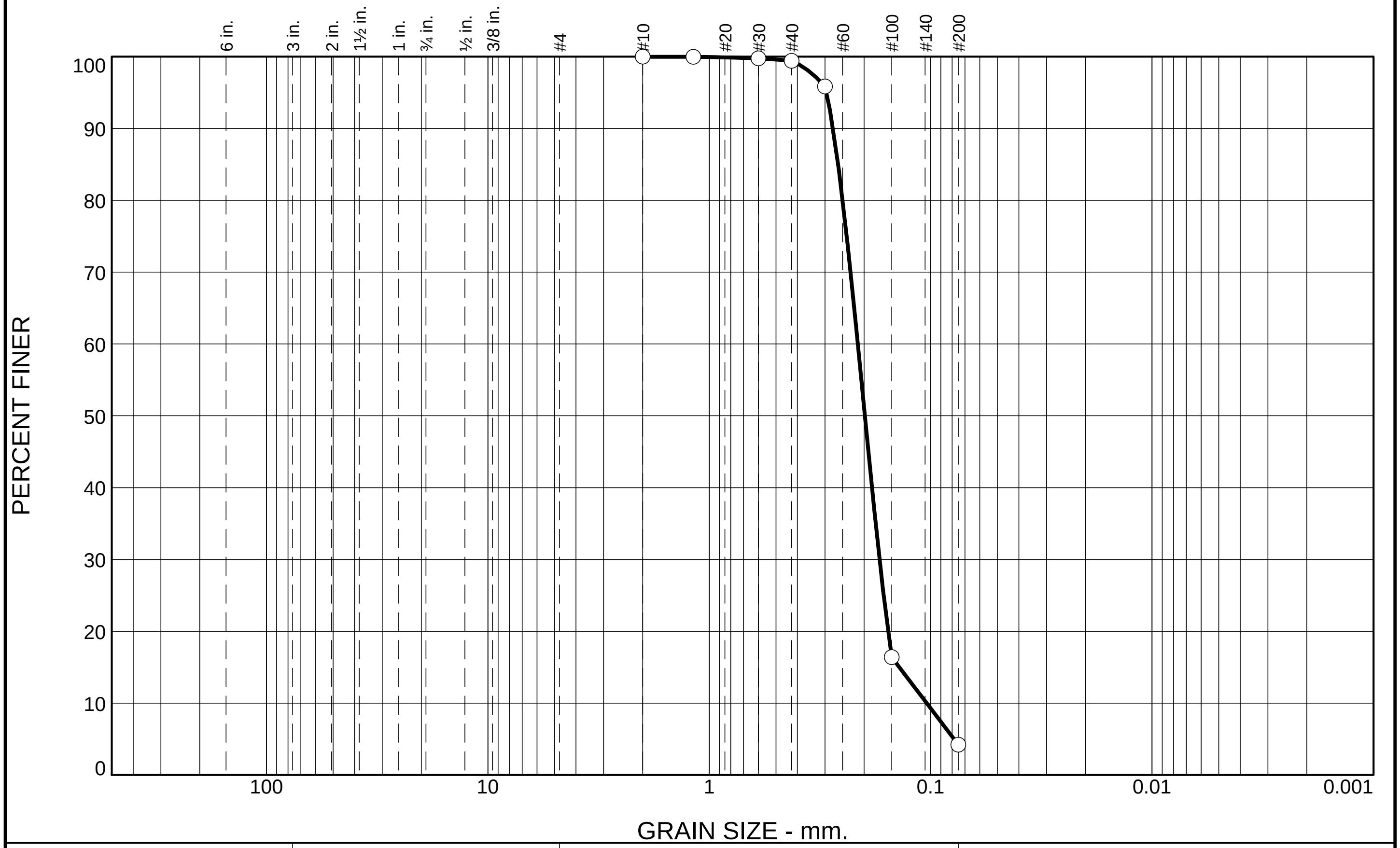
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: bb **Checked By:** ____________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.6	95.2	4.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	99.8		
#40	99.4		
#50	95.9		
#100	16.4		
#200	4.2		

* (no specification provided)

Soil Description

Gray fine sand (SP)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2761	D_{85}= 0.2618	D_{60}= 0.2134
D_{50}= 0.1983	D_{30}= 0.1703	D_{15}= 0.1385
D_{10}= 0.1042	C_u= 2.05	C_c= 1.30

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S2-2 **Depth:** 38.5'
Sample Number: 11
Date: 10-12-20

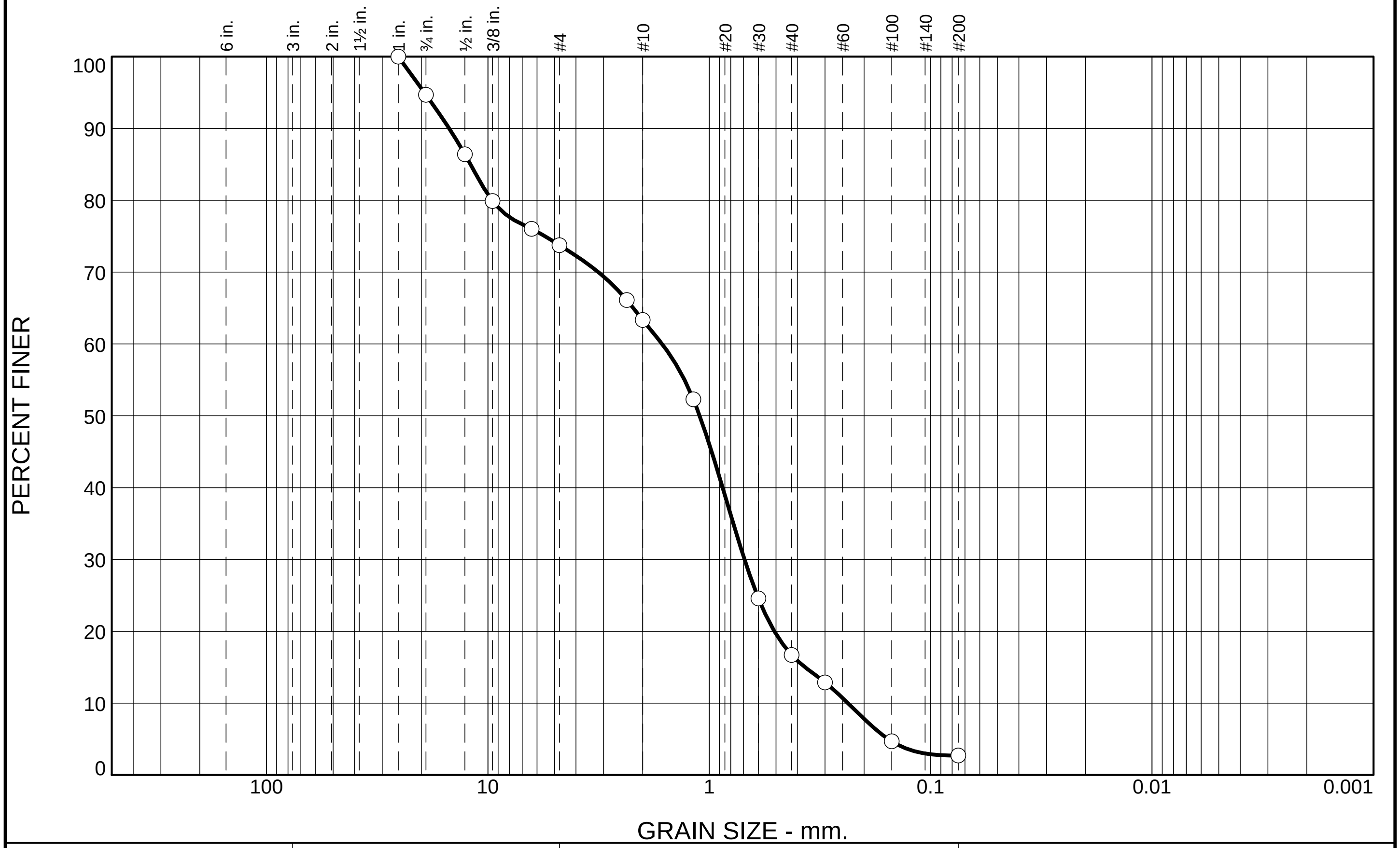
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: bb **Checked By:**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.3	21.0	10.4	46.6	14.0	2.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	94.7		
1/2	86.4		
3/8	79.9		
1/4	76.0		
#4	73.7		
#8	66.1		
#10	63.3		
#16	52.3		
#30	24.6		
#40	16.7		
#50	12.9		
#100	4.7		
#200	2.7		

* (no specification provided)

Soil Description

Light gray and gray fine to coarse sand (SW) with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 14.9463 D_{85}= 11.9752 D_{60}= 1.6342
D_{50}= 1.1050 D_{30}= 0.6934 D_{15}= 0.3697
D_{10}= 0.2367 C_u= 6.90 C_c= 1.24

Classification

USCS= SW AASHTO=

Remarks

Source of Sample: S2-3 **Depth:** 58.5'
Sample Number: 14 **Date:** 10-12-20

BURNS COOLEY DENNIS, INC.

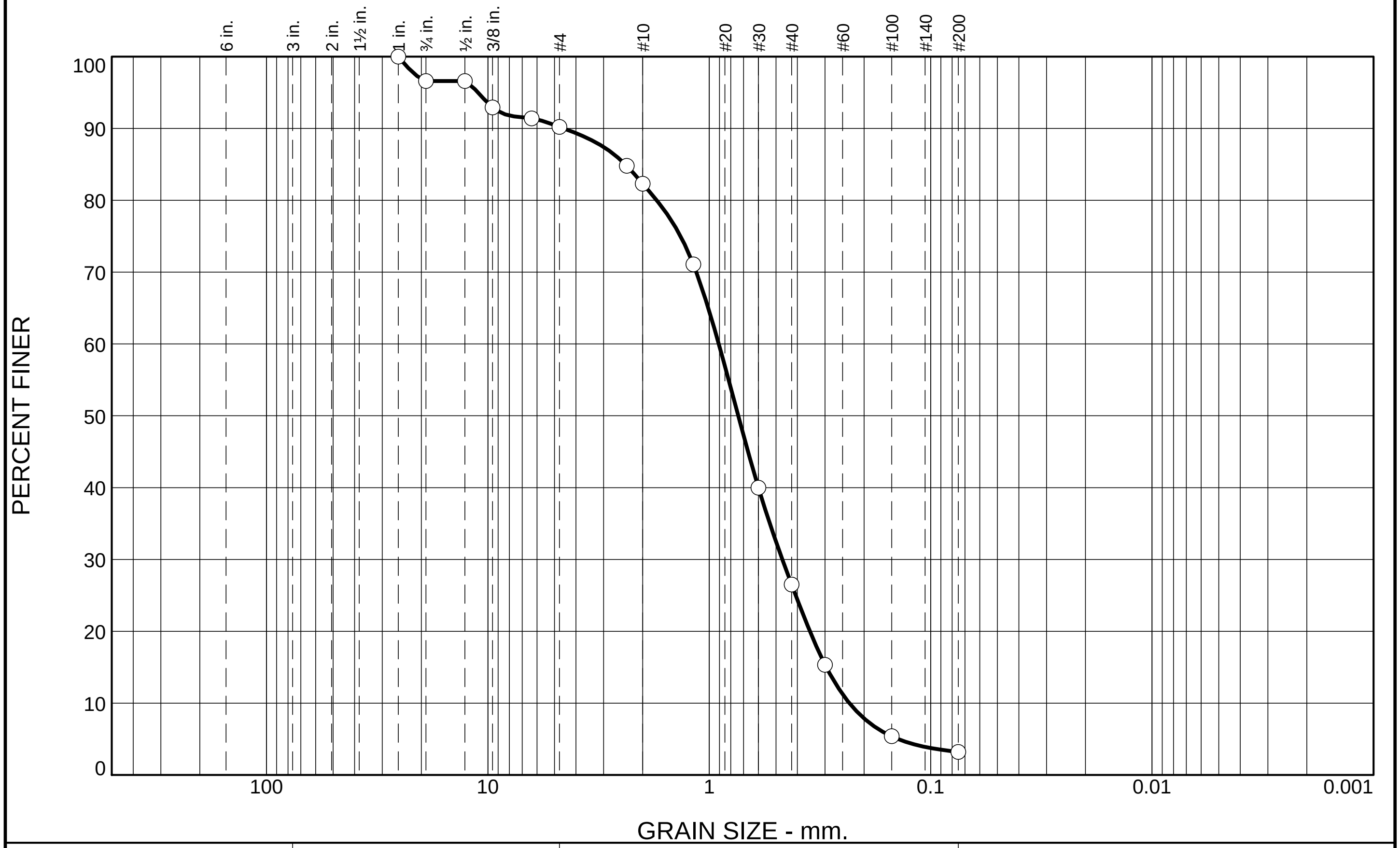
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: bb **Checked By:** ____________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.4	6.4	7.9	55.8	23.3	3.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	96.6		
1/2	96.6		
3/8	92.9		
1/4	91.4		
#4	90.2		
#8	84.8		
#10	82.3		
#16	71.1		
#30	40.0		
#40	26.5		
#50	15.3		
#100	5.4		
#200	3.2		

* (no specification provided)

Soil Description

Gray fine to coarse sand (SP) with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 4.5467 D_{85}= 2.3957 D_{60}= 0.9058

D_{50}= 0.7398 D_{30}= 0.4681 D_{15}= 0.2963

D_{10}= 0.2331 C_u= 3.89 C_c= 1.04

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S2-2 **Depth:** 83.5'
Sample Number: 20

Date: 10-12-20

BURNS COOLEY DENNIS, INC.

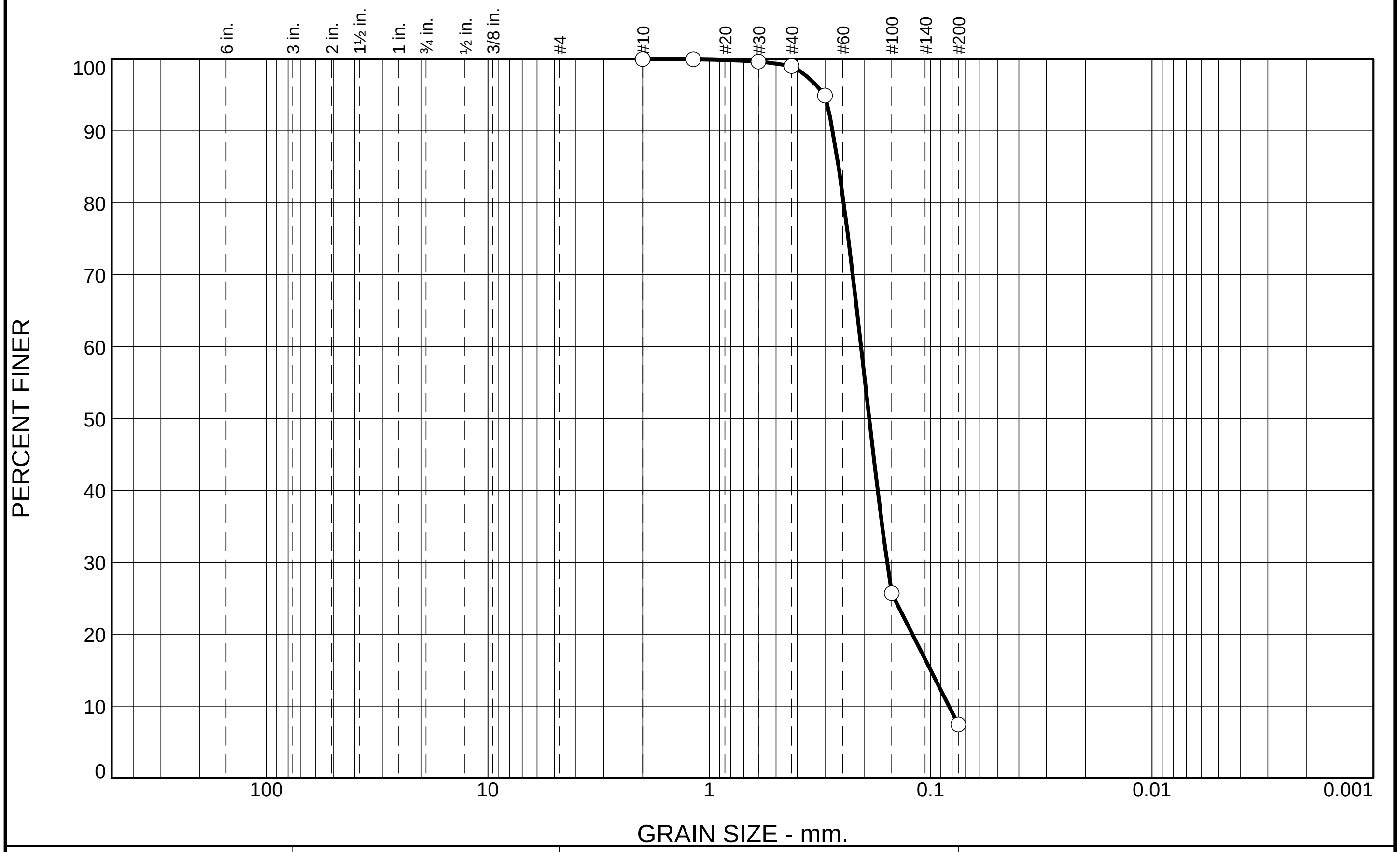
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: bb **Checked By:** ____________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.0	91.6	7.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	99.7		
#40	99.0		
#50	94.9		
#100	25.7		
#200	7.4		

* (no specification provided)

Soil Description

Gray fine sand (SP-SM), slightly silty

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2770 D_{85}= 0.2605 D_{60}= 0.2063
D_{50}= 0.1896 D_{30}= 0.1575 D_{15}= 0.1000
D_{10}= 0.0827 C_u= 2.49 C_c= 1.45

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: S2-3 **Depth:** 23.5'
Sample Number: 7 **Date:** 10-12-20

BURNS COOLEY DENNIS, INC.

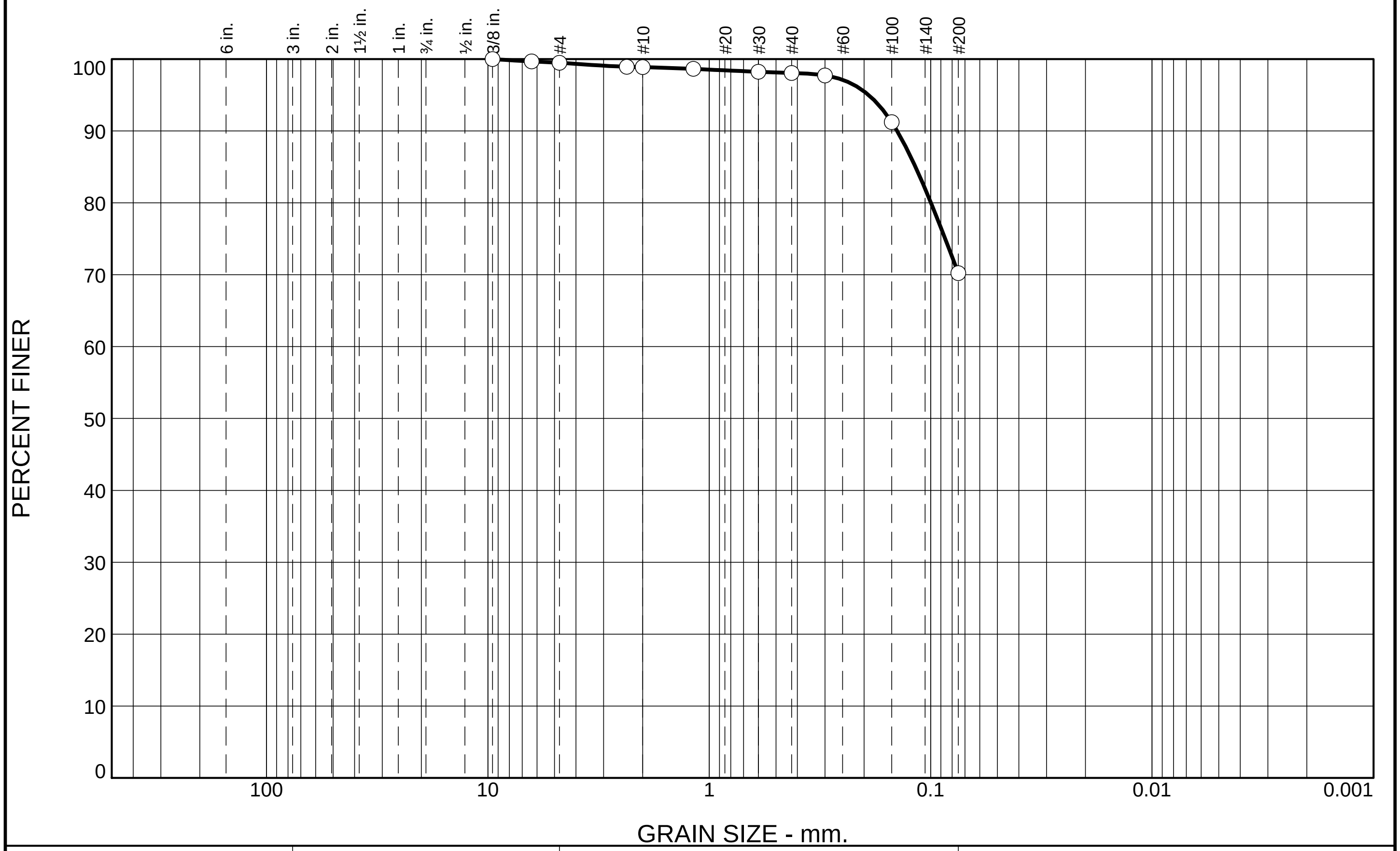
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: bb **Checked By:** ______

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.5	0.6	0.8	27.9	70.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100.0		
1/4	99.7		
#4	99.5		
#8	98.9		
#10	98.9		
#16	98.6		
#30	98.2		
#40	98.1		
#50	97.7		
#100	91.2		
#200	70.2		

* (no specification provided)

Soil Description

Light gray clay (CH) with silty fine sand seams

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1417 D_{85}= 0.1170 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO=

Remarks

Source of Sample: S2-4 **Depth:** 4.0'
Sample Number: 3

Date: 10-12-20

BURNS COOLEY DENNIS, INC.

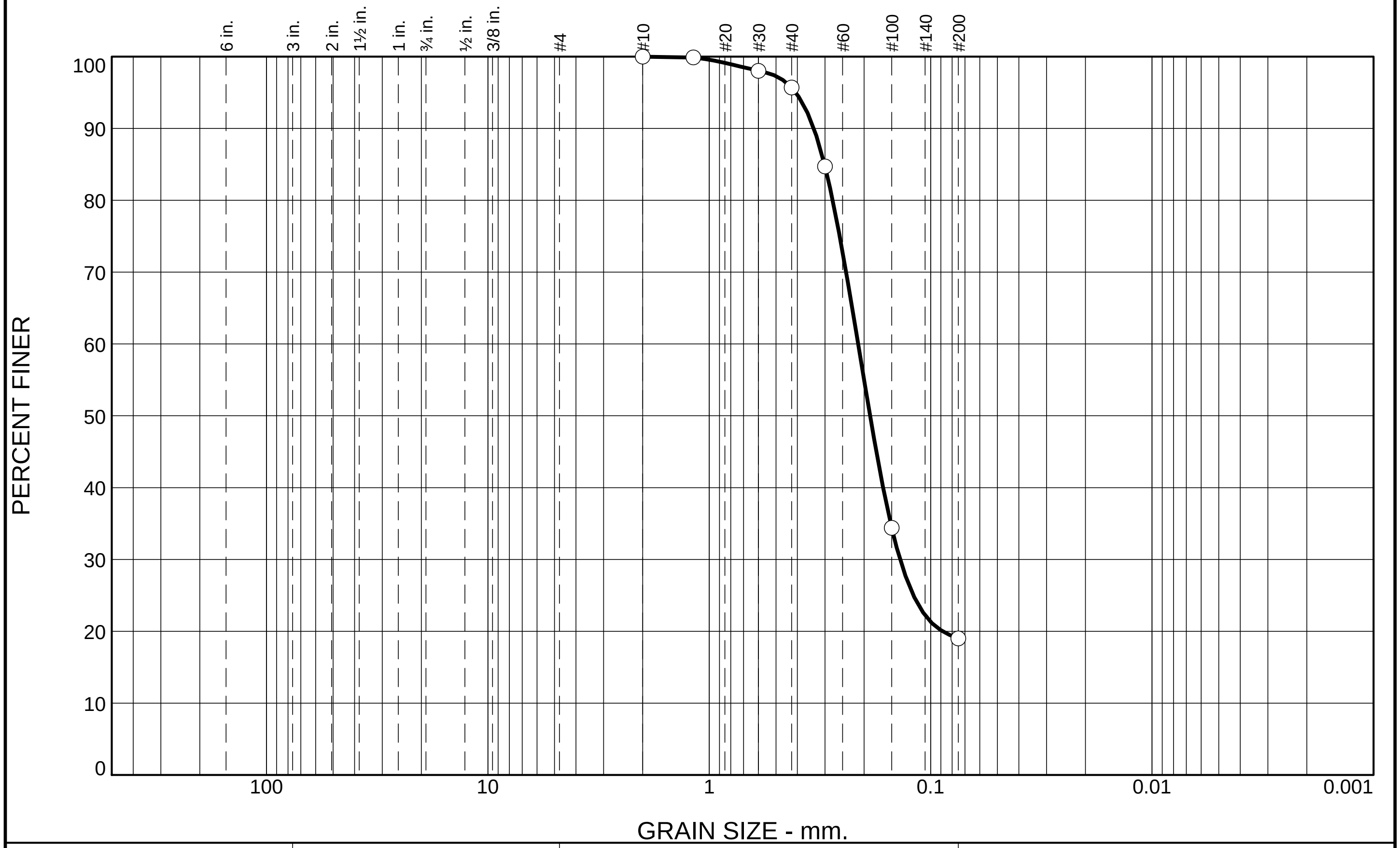
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: bb **Checked By:**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	4.3	76.7	19.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.9		
#30	98.0		
#40	95.7		
#50	84.7		
#100	34.4		
#200	19.0		

* (no specification provided)

Soil Description

Gray silty fine sand (SM) with clay partings

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3373 D_{85}= 0.3016 D_{60}= 0.2126
D_{50}= 0.1878 D_{30}= 0.1374 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: S2-4 **Depth:** 18.0'
Sample Number: 6
Date: 10-12-20

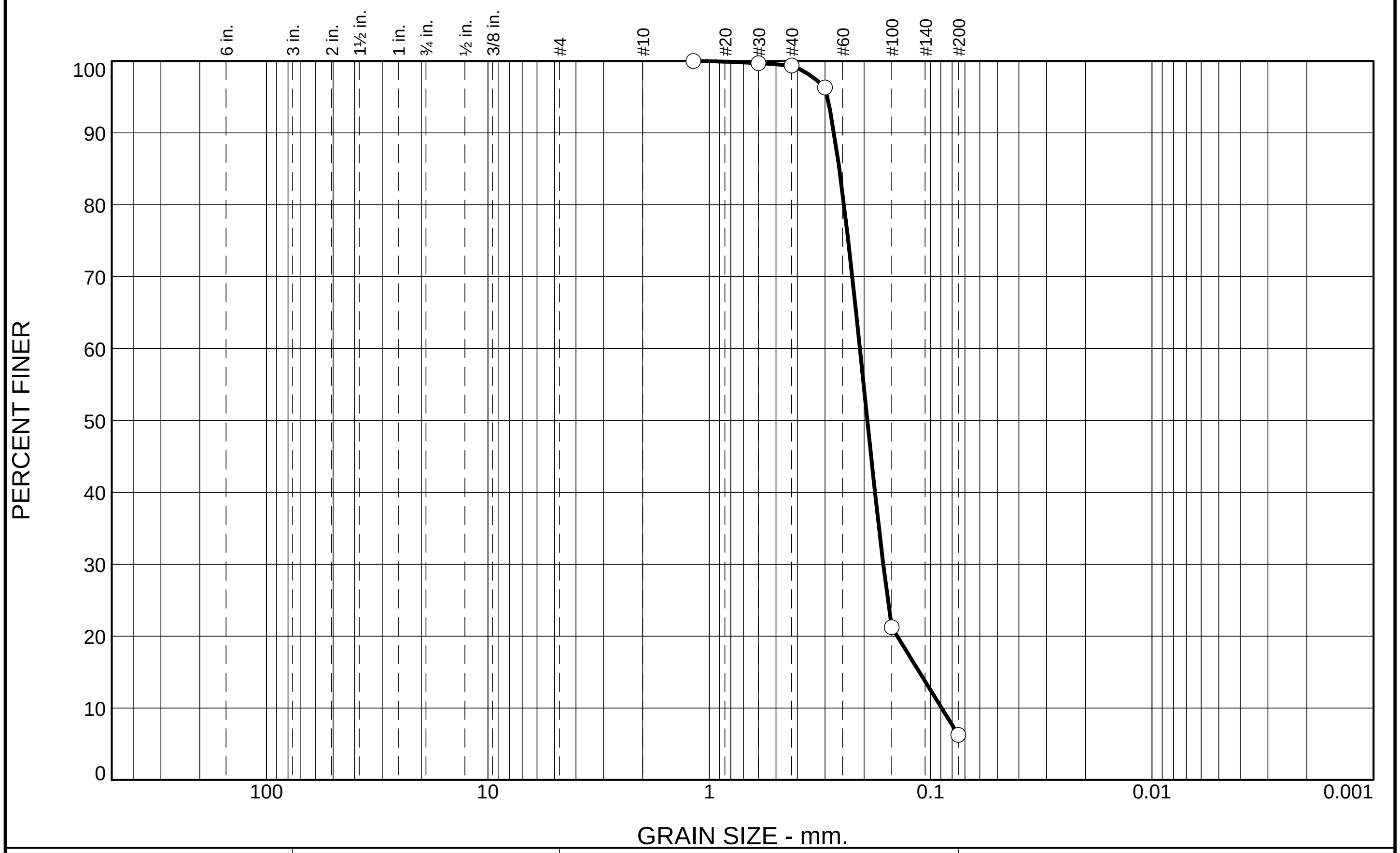
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: bb **Checked By:** ____________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.6	93.1	6.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#16	100.0		
#30	99.7		
#40	99.4		
#50	96.3		
#100	21.2		
#200	6.3		

* (no specification provided)

Soil Description

Gray and tan fine sand (SP-SM), slightly silty

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2732 D_{85}= 0.2585 D_{60}= 0.2088
D_{50}= 0.1933 D_{30}= 0.1639 D_{15}= 0.1124
D_{10}= 0.0892 C_u= 2.34 C_c= 1.44

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: S2-4 **Depth:** 38.5'
Sample Number: 10
Date: 10-12-20

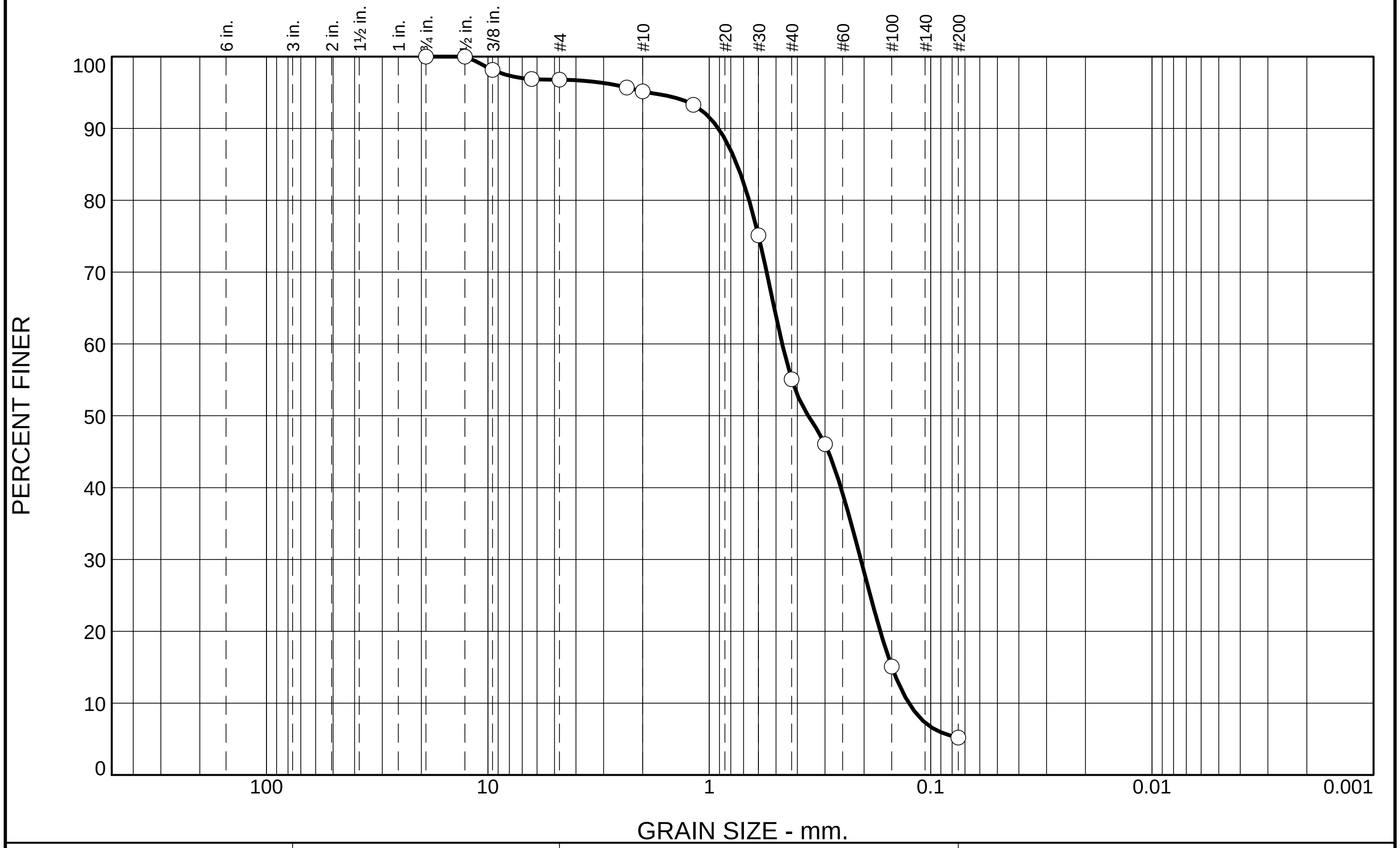
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: bb **Checked By:** ____________________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.2	1.6	40.1	49.9	5.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	100.0		
3/8	98.2		
1/4	96.9		
#4	96.8		
#8	95.7		
#10	95.2		
#16	93.3		
#30	75.1		
#40	55.1		
#50	46.0		
#100	15.1		
#200	5.2		

* (no specification provided)

Soil Description

Gray and light gray fine to coarse sand (SP-SM), slightly silty

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.9093 D_{85}= 0.7499 D_{60}= 0.4686

D_{50}= 0.3576 D_{30}= 0.2069 D_{15}= 0.1496

D_{10}= 0.1255 C_u= 3.73 C_c= 0.73

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: S2-4 **Depth:** 83.5'

Sample Number: 19

Date: 10-12-20

BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT

Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: bb **Checked By:** ____________

APPENDIX B

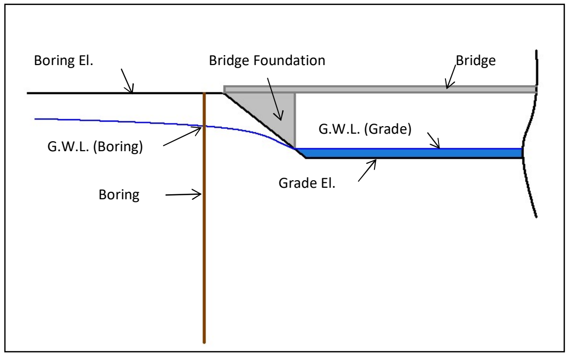
Liquefaction Triggering Workbook

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]
Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.89407	-91.06962
Logged By :	Christian Jackson	
Boring No:	S2-1	
Date:	31-Aug-20	
Type of Drilling:	HSA to 28', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: ______ Date: ______
Checked by: ______ Date: ______

All cells highlighted in this color should be input
All other cells should not be altered!



Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.367	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	249	ft
Ground Water Level (depth below boring surface) =	7	ft
Grade Surface Elevation =	240	ft
Ground Water Level (depth below or above grade surface) =	-2	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.

		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	246.5	2.5	CL	1						
2	244	5	CL	10						
3	239	10	CL	1	64.67					
4	234	15	CL	1	8.61					
5	229	20	SP-SM	11	8.79					
6	224	25	ML	1	38.33					
7	219	30	SP-SM	17	5.05					
8	214	35	SP-SM	21	7.82					
9	209	40	SP-SM	54	7.08					
10	204	45	SP-SM	15	9.3					
11	199	50	SP-SM	65	6.85					
12	194	55	SP	20	2.8					
13	189	60	SP	28	4.37					
14	184	65	SP	16	2.97					
15	179	70	SP	20	4.29					
16	174	75	SP	15	2.56					
17	169	80	SP	18	3.22					
18	164	85	SP	23	2.71					
19	159	90	SP	47	3.25					
20	154	95	SP	41	3.65					
21	149	100	SP	63	2.83					

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]

Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.89405	-91.06932
Logged By :	Christian Jackson	
Boring No:	S2-2	
Date:	3-Sep-20	
Type of Drilling:	HSA to 35', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: ____________ Date: ____________

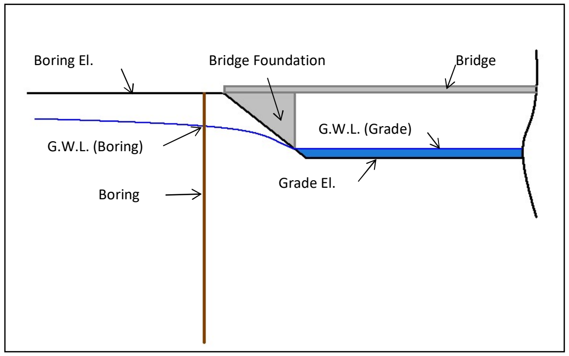
Checked by: ____________ Date: ____________

All cells highlighted in this color should be input

All other cells should not be altered!

Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.367	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	249	ft
Ground Water Level (depth below boring surface) =	7	ft
Grade Surface Elevation =	240	ft
Ground Water Level (depth below or above grade surface) =	-2	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.



		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	246.5	2.5	SC	17						
2	244	5	CH	1						
3	239	10	CL	1	64.83					
4	234	15	CL	1						
5	229	20	CL	1						
6	224	25	CL	1						
7	219	30	CL	1						
8	214	35	CL	1	66.57					
9	209	40	SP	23	4.21					
10	204	45	SP-SM	33	10.39					
11	199	50	SM	21	17.81					
12	194	55	SP	37	4.07					
13	189	60	SP	27	3.02					
14	184	65	SP	25	2.42					
15	179	70	SP	33	4.24					
16	174	75	SP	14	2.12					
17	169	80	SP	22	3					
18	164	85	SP	20	3.19					
19	159	90	SP	35	2.93					
20	154	95	SP	35	3.63					
21	149	100	SP	33	3.64					

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]

Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.89401	-91.06887
Logged By :	Christian Jackson	
Boring No:	S2-3	
Date:	9/8/2020 to 9/9/2020	
Type of Drilling:	HSA to 25', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: ______ Date: ______

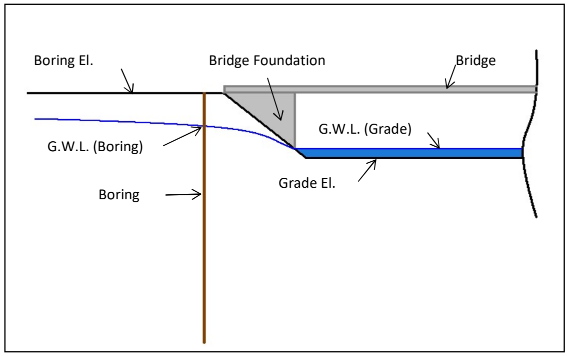
Checked by: ______ Date: ______

All cells highlighted in this color should be input

All other cells should not be altered!

Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.367	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	249	ft
Ground Water Level (depth below boring surface) =	7	ft
Grade Surface Elevation =	240	ft
Ground Water Level (depth below or above grade surface) =	-2	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.



		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	246.5	2.5	SC	17						
2	244	5	CL	1						
3	239	10	CL	1						
4	234	15	CH	1						
5	229	20	SP-SM	1						
6	224	25	SP-SM	11	7.43					
7	219	30	SP-SM	15	9.4					
8	214	35	SP	13	3.92					
9	209	40	SP-SM	26	7.92					
10	204	45	SP	42	4.58					
11	199	50	SP-SM	34	6.25					
12	194	55	SP	23	2.5					
13	189	60	SP	9	2.7					
14	184	65	SP	33	4.03					
15	179	70	SP	18	3.97					
16	174	75	SP-SM	17	5.76					
17	169	80	CL	13	57.18					
18	164	85	SP	46	4.66					
19	159	90	SP	26	3.69					
20	154	95	SP	43	4.31					
21	149	100	SP	51	4.06					

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]

Input Data Worksheet

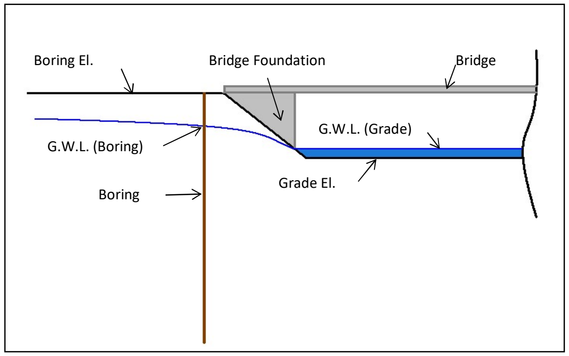
Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.89398	-91.06856
Logged By :	Christian Jackson	
Boring No:	S2-4	
Date:	9/9/2020 to 9/10/2020	
Type of Drilling:	HSA to 50', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: ____ Date: ____

Checked by: ____ Date: ____

All cells highlighted in this color should be input

All other cells should not be altered!



Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.367	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	249	ft
Ground Water Level (depth below boring surface) =	7	ft
Grade Surface Elevation =	240	ft
Ground Water Level (depth below or above grade surface) =	-2	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.

		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	246.5	2.5	CH	19						
2	244	5	ML	1	70.22					
3	239	10	CL	1						
4	234	15	CL-ML	1						
5	229	20	SM	1	19					
6	224	25	SP-SM	13	5.1					
7	219	30	SP	14	4.91					
8	214	35	SP-SM	12	9.29					
9	209	40	SP-SM	28	6.25					
10	204	45	SP	40	4.55					
11	199	50	SP-SM	29	9.88					
12	194	55	SP	22	2.99					
13	189	60	SP	22	2.73					
14	184	65	SP-SM	18	5					
15	179	70	SP	27	4.07					
16	174	75	SP	28	2.9					
17	169	80	SP	19	2.55					
18	164	85	SP-SM	5	5.21					
19	159	90	SP	23	2.65					
20	154	95	SP	24	2.27					
21	149	100	SP-SM	28	6.77					

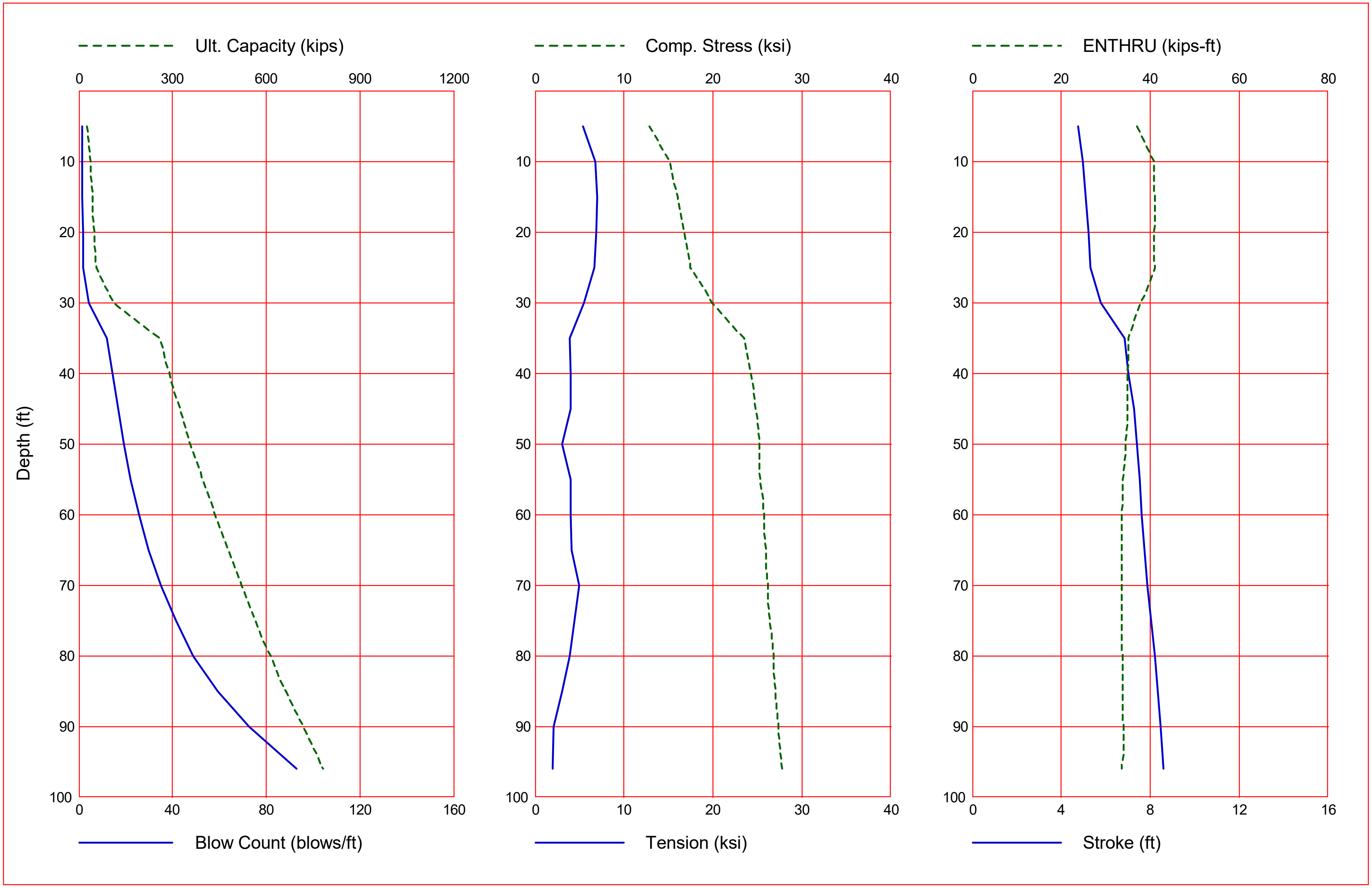
APPENDIX C

Pile Drivability Analysis Results

Burns Cooley Dennis, Inc.
Site 2- West Abutment - DELMAG D30

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Burns Cooley Dennis, Inc.
Site 2- West Abutment - DELMAG D30

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

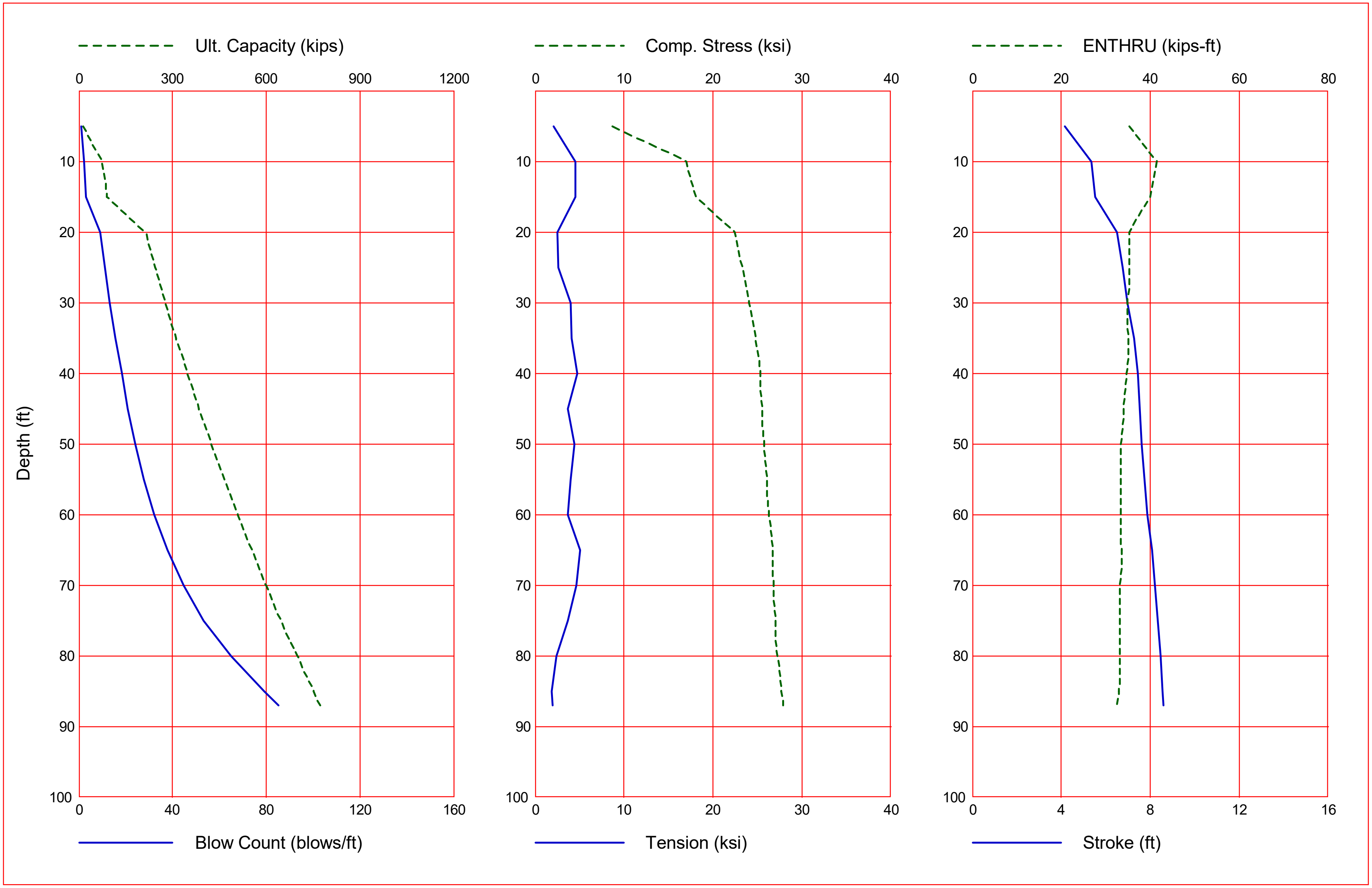
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	25.6	12.4	13.1	1.4	12.838	-5.426	4.75	37.0
10.0	38.3	25.1	13.1	1.6	15.044	-6.727	4.99	40.9
15.0	43.9	36.8	7.1	1.6	16.076	-6.975	5.11	41.0
20.0	50.8	43.7	7.1	1.8	16.791	-6.836	5.21	40.8
25.0	57.7	50.6	7.1	1.9	17.402	-6.684	5.30	41.1
30.0	110.4	62.7	47.7	4.1	19.920	-5.491	5.79	37.8
35.0	258.5	90.7	167.9	12.1	23.524	-3.906	6.84	35.0
40.0	289.7	121.8	167.9	14.4	24.227	-3.950	7.03	34.8
45.0	323.0	155.1	167.9	16.8	24.815	-4.031	7.28	34.9
50.0	358.4	190.5	167.9	19.3	25.243	-3.081	7.41	34.4
55.0	395.9	228.0	167.9	22.2	25.379	-4.007	7.51	33.9
60.0	435.5	267.6	167.9	25.6	25.750	-3.950	7.63	33.6
65.0	477.2	309.3	167.9	29.7	25.953	-4.116	7.76	33.7
70.0	521.0	353.1	167.9	34.9	26.235	-4.916	7.89	33.7
75.0	566.9	399.0	167.9	41.5	26.430	-4.450	8.03	33.6
80.0	614.9	447.0	167.9	48.9	26.830	-3.889	8.22	33.8
85.0	665.0	497.2	167.9	59.4	27.002	-3.087	8.36	33.9
90.0	717.2	549.4	167.9	72.9	27.341	-2.117	8.48	34.0
96.0	782.7	614.8	167.9	93.1	27.804	-1.966	8.60	33.6

Total Continuous Driving Time 56.00 minutes; Total Number of Blows 2363 (starting at penetration 5.0 ft)

Burns Cooley Dennis, Inc.
Site 2- East Abutment - DELMAG D30

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

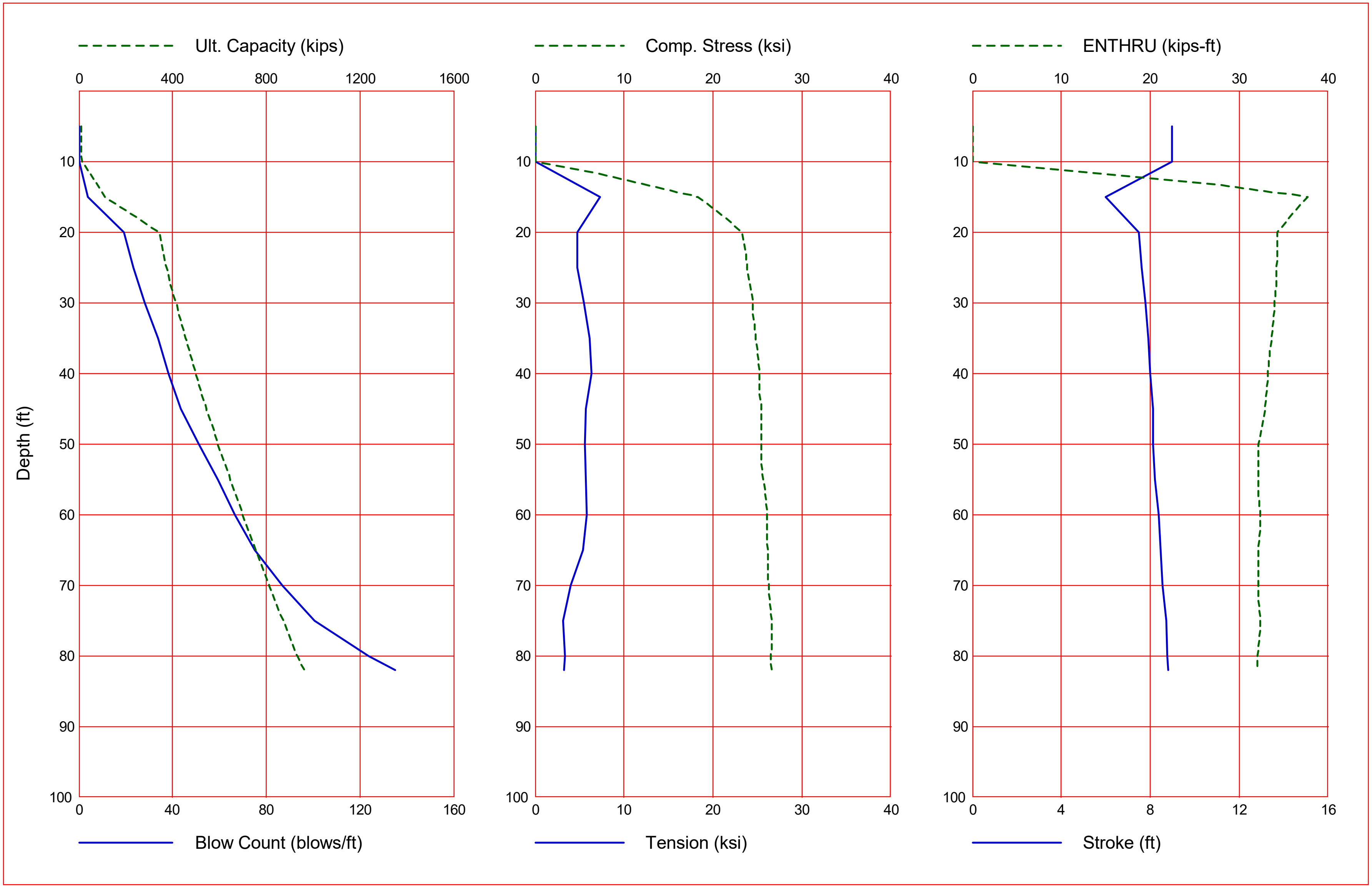
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	14.2	9.8	4.4	1.1	8.733	-2.050	4.16	35.3
10.0	75.7	17.4	58.3	2.4	16.992	-4.537	5.35	41.5
15.0	90.7	32.4	58.3	3.0	18.112	-4.500	5.53	40.0
20.0	213.7	56.4	157.3	9.2	22.431	-2.539	6.52	35.3
25.0	244.6	87.3	157.3	11.0	23.296	-2.593	6.75	35.2
30.0	277.2	119.9	157.3	13.2	24.028	-4.043	6.97	34.9
35.0	311.5	154.2	157.3	15.6	24.788	-4.152	7.26	35.0
40.0	347.6	190.3	157.3	18.3	25.306	-4.714	7.43	34.7
45.0	385.4	228.1	157.3	20.9	25.507	-3.676	7.52	34.0
50.0	425.0	267.7	157.3	24.0	25.773	-4.452	7.61	33.4
55.0	466.3	309.0	157.3	27.8	26.056	-4.031	7.76	33.3
60.0	509.3	352.0	157.3	32.4	26.287	-3.678	7.89	33.3
65.0	554.1	396.8	157.3	37.7	26.694	-5.038	8.07	33.5
70.0	600.6	443.3	157.3	44.9	26.871	-4.669	8.20	33.1
75.0	648.9	491.6	157.3	53.4	27.086	-3.643	8.32	33.1
80.0	698.9	541.6	157.3	64.9	27.281	-2.353	8.45	33.1
85.0	750.6	593.4	157.3	79.3	27.644	-1.881	8.55	32.9
87.0	771.8	614.5	157.3	85.4	27.851	-1.970	8.59	32.6

Total Continuous Driving Time 54.00 minutes; Total Number of Blows 2261 (starting at penetration 5.0 ft)

Burns Cooley Dennis, Inc.
Site 2- Interior Bents - DELMAG D30

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	9.5	3.8	5.7	0.0	0.000	0.000	9.00	0.0
10.0	13.4	7.8	5.7	0.0	0.000	0.000	9.00	0.0
15.0	111.7	26.9	84.8	4.0	18.273	-7.291	6.00	37.7
20.0	340.4	60.8	279.6	19.3	23.262	-4.756	7.47	34.3
25.0	376.7	97.1	279.6	23.2	23.837	-4.702	7.63	34.2
30.0	415.4	135.8	279.6	28.3	24.503	-5.494	7.80	34.0
35.0	456.6	177.0	279.6	33.7	24.817	-6.153	7.91	33.6
40.0	500.1	220.5	279.6	38.4	25.187	-6.298	8.01	33.2
45.0	546.1	266.5	279.6	43.7	25.480	-5.662	8.14	32.9
50.0	594.5	314.9	279.6	51.4	25.463	-5.601	8.12	32.2
55.0	645.2	365.6	279.6	59.2	25.636	-5.660	8.21	32.2
60.0	698.4	418.8	279.6	66.8	26.033	-5.843	8.39	32.4
65.0	754.0	474.4	279.6	75.3	26.192	-5.361	8.48	32.2
70.0	812.0	532.4	279.6	86.9	26.312	-3.972	8.56	32.2
75.0	872.4	592.8	279.6	100.8	26.581	-3.096	8.73	32.4
80.0	935.2	655.6	279.6	123.9	26.543	-3.381	8.75	32.1
82.0	961.0	681.4	279.6	135.2	26.595	-3.265	8.79	32.1

Total Continuous Driving Time 91.00 minutes; Total Number of Blows 3722 (starting at penetration 5.0 ft)

APPENDIX D

AHTD Special Provision for Embankment Construction

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

DESCRIPTION: This Special Provision shall be supplementary to Section 210, Excavation and Embankment, of the Standard Specifications, Edition of 2003 and shall apply to the construction of embankments being built over existing borrow ditches as shown in the plans or where directed by the Engineer.

MATERIALS: Stone Backfill shall meet the requirements of Section 207 of the Standard Specifications, Edition of 2003.

Select Material (Class SM-2) shall meet the requirements of Section 302 of the Standard Specifications, Edition of 2003.

Dumped Riprap and Filter Blanket shall comply with Section 816 of the Standard Specifications except that synthetic geotextile fabric complying with requirements of Subsection 625.02, Type 5 must be used as a filter blanket under dumped riprap in lieu of a granular filter blanket material.

Clay plating shall consist of material having a minimum plasticity index of 10 and a maximum plasticity index of 25, which will support vegetation and not be highly susceptible to erosion.

CONSTRUCTION: When the embankment is to be built over existing borrow ditches, the ditches shall be undercut 2 feet below the existing flow line to remove all highly organic, wet material prior to embankment construction. The ditches shall then be filled using Stone Backfill. The top 4" to 6" of Stone Backfill shall be material complying with Section 303 of the Standard Specifications, Edition of 2003 for Class 7 Aggregate Base Course in accordance with Section 207. Excavation for the placement of Stone Backfill shall be considered part of the item in accordance with subsection 207.01 of the Standard Specifications.

The remaining embankment shall be constructed of Selected Material (Class SM-2). Synthetic Filter Blanket and Dumped Riprap shall be placed on the slopes of embankments constructed of Select Material (Class SM-2) from the top of the Stone Backfill to 2 feet above the high water elevation or as directed by the Engineer. The remainder of embankments constructed of Select Material (Class SM-2) or other material which is susceptible to erosion shall have a minimum 18 inch clay plating (measured

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

perpendicular to the finished slopes).

All embankment materials, including Selected Material (Class SM-2) and Clay Plating, shall be placed and compacted in accordance with Subsections 210.07, 210.09, and 210.10 of the Standard Specifications.

QUALTIY CONTROL AND ACCEPTANCE: The Contractor shall perform quality control and acceptance sampling and testing of the clay plating for plasticity index; Selected Material (Class SM-2) for gradation and plasticity index in accordance with Section 306 except that the size of the standard lot will be 3000 cubic yards. The Contractor shall perform quality control and acceptance sampling and testing of the Selected Material (Class SM-2) for density and moisture content in accordance with Subsection 210.02 of the Standard Specifications for Highway Construction. Selected Material (Class SM-2) shall meet the density requirements of Subsection 210.10.

METHOD OF MEASUREMENT: Embankments consisting of Selected Material (Class SM-2) and Clay Plating material and as shown on the plans, will be measured as Compacted Embankment in accordance with Subsection 210.12 of the Standard Specifications.

Stone Backfill will be measured in accordance with Section 207 of the Standard Specifications.

Filter Blanket and Dumped Riprap will be measured in accordance with Section 816 of the Standard Specifications.

BASIS OF PAYMENT: All accepted embankments; including Selected Material (Class SM-2) and Clay Plating material measured as provided above will be paid for as Compacted Embankment in accordance with Subsection 210.13 of the Standard Specifications.

Stone Backfill shall be paid in accordance with Section 207 of the Standard Specifications.

Filter Blanket and Dumped Riprap will be paid in accordance with Section 816 of the Standard Specifications.

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

Payment will be made under:

Pay Item	Pay Unit
Compacted Embankment	Cubic Yard
Stone Backfill	Ton
Filter Blanket	Square Yard
Dumped Riprap	Cubic Yard

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

Corporate Office
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Phone: (601) 856-9911
Fax: (601) 853-2077

Mailing Address
Post Office Box 12828
Jackson, MS 39236

www.bcdgeo.com

Materials Laboratory
278 Commerce Park Drive
Ridgeland, MS 39157
Phone: (601) 856-2332
Fax: (601) 856-3552

March 1, 2021

Cindy Rich, P.E.
Neel-Schaffer, Inc.
125 South Congress Street, Suite 1100
Post Office Box 22625
Jackson, Mississippi 39201

Report No. 200518 – Site 5

Geotechnical Exploration
Site 5
ARDOT SR230 Bridge Replacements
Craighead and Lawrence Counties, Arkansas

Dear Ms. Rich:

Submitted here is the report of our geotechnical exploration for the above-captioned project. This exploration was authorized by Task Order 108 to the Subconsultant Agreement between Neel-Schaffer, Inc. and Burns Cooley Dennis, Inc. dated September 17, 2020.

We appreciate the opportunity to be of service. If you should have any questions concerning this report, please do not hesitate to call us.

Very truly yours,

BURNS COOLEY DENNIS, INC.

Alexander B. Reeb, Ph.D., P.E.

A. E. (Eddie) Templeton, P.E.

ABR/AET/khb
Copy Submitted: (via e-mail)

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 Project Description

Plans are being made for the construction of replacement bridges and box culverts at ten sites along Highway 230 between Alicia and Bono in Craighead and Lawrence Counties, Arkansas. Site 5 is located in Lawrence County where Highway 230 crosses Lick Pond Slough. At this site, a new bridge will be constructed on a new alignment just north of the existing bridge.

The new bridge will be about 227 ft long and consist of three spans of approximately equal spacing. It is our understanding that new fill will be placed to raise the grade at the new abutments above the grade of the existing bridge. The abutment spill-through slopes will be constructed as 2H:1V slopes, and the abutment side slopes will be constructed as 3H:1V slopes. The abutment bents are to be supported by 18-in. diameter, closed-ended steel pipe piles, and the interior bents are to be supported by 24-in. diameter, closed-ended steel pipe piles. A preliminary layout showing the proposed construction is presented on Figure 1 of this report.

1.2 Purposes

The specific purposes of this exploration were:

1) to review the exploratory soil borings made within the area planned for construction of the new bridge;

2) to verify field classifications and to evaluate pertinent physical properties of the soils encountered in the borings by means of visual examination of the soil samples in the laboratory and routine tests performed on the samples;

3) to perform analyses to investigate liquefaction, slope stability, settlement, pile capacity, and downdrag; and

4) to provide geotechnical recommendations for design and construction of the bridge.

Our scope of work for the bridge does not include providing recommendations for roadway subgrades and pavements. Discussion and recommendations pertaining to roadway subgrades and pavements are provided under separate cover.

2.0 FIELD EXPLORATION

2.1 General

Subsurface soil conditions within the area planned for construction of the bridge were explored by means of four deep borings. Borings S5-1, S5-2, S5-3, and S5-4 were performed by McCray Drilling under contract to SoilTech Consultants, Inc. The approximate locations of the borings are shown on Figure 1.

All soils were classified in general accordance with the Unified Soil Classification System. A synopsis of the Unified Soil Classification System (USCS) is presented on Figure 2 along with symbols and terminology typically utilized on graphical soil boring logs. Graphical logs of the borings are presented on Figures 3 through 6. The graphical logs illustrate the types of soil and stratification encountered with depth below the existing ground surface at the individual boring locations. Approximate GPS coordinates for the boring locations are shown at the bottom of the graphical boring logs within the “Comments” section.

2.2 Drilling Methods and Groundwater Observations

Borings S5-1, S5-2, S5-3, and S5-4 were made to an exploration depth of 100 ft. The borings were made using a CME-750X buggy-mounted drill rig. Borings S5-1, S5-2, S5-3, and S5-4 were initially advanced to a depth of 55 ft, 60 ft, 60 ft and 55 ft, respectively, by dry augering and then were extended to completion using rotary wash drilling procedures. Groundwater was encountered at a depth of 52 ft, 54 ft, 39 ft, and 52.5 ft in Borings S5-1, S5-2, S5-3, and S5-4, respectively.

2.3 Sampling Methods

Disturbed samples of soils were obtained by driving a standard 2-in. OD split-spoon sampler 18 in. into the soil with a 140-lb hammer falling freely a distance of 30 in. The depths at which the split-spoon samples were taken are illustrated as crossed rectangular symbols under the "Samples" column of the graphic logs. Standard penetration test (SPT) blow counts resulting from split-spoon sampling are recorded under the "Blows Per Ft" column of the graphic logs. The SPT blow counts are the “raw” field values. The recommended hammer energy correction factor is indicated in the “Comments” section of the logs. Relatively undisturbed samples of the soils encountered in the borings were obtained by pushing a 3-in. OD Shelby tube sampler approximately 2 ft into the soil. The Shelby tube samples were obtained within the depth intervals illustrated as shaded portions of the "Samples" column of the graphic logs. The Shelby tube and/or split-spoon samples were generally obtained at approximate 3-ft to 5-ft intervals of depth. Disturbed auger cutting samples were taken near the ground surface in the borings. The depths at which the auger cutting samples were taken are illustrated as small I-shaped symbols under the "Samples" column of the graphic boring logs.

2.4 Field Classification, Sample Preservation and Borehole Abandonment

All soils encountered during drilling were examined and classified in the field by a geotechnical engineering technician. Representative portions of the split-spoon samples and the auger cutting samples were sealed in jars to provide material for visual examination and testing in the laboratory. The Shelby tubes were capped and the ends sealed with wax in the field to prevent moisture loss and structural disturbance while they were transported to the testing laboratory. At the testing laboratory, the Shelby tube samples were extruded, and an approximate 6-in. long portion of each sample was temporarily sealed in plastic wrap to prevent moisture loss during the period between sample extrusion and testing. Additional portions of each Shelby tube sample were sealed in jars to provide additional material for visual examination and testing. The borehole for Boring S5-3 was grouted and the other boreholes were plugged with soil cuttings after completion of drilling and sampling.

3.0 LABORATORY TESTING

3.1 General

All of the soil samples were examined in the laboratory and tests were performed on selected samples to verify field classifications and to assist in evaluating the strength and volume change properties of the soils encountered. The types of laboratory tests performed are described in the following paragraphs.

3.2 Strength Properties

The undrained shear strength characteristics of the fine-grained soils encountered in the borings were investigated by means of visual estimates of consistency and from the results of unconfined compression tests and unconsolidated undrained (UU) triaxial compression tests performed on selected undisturbed Shelby tube samples. The results of the unconfined compression tests in terms of cohesion are plotted as small open circles in the data sections of the graphic logs. The cohesions resulting from the UU triaxial compressions test are plotted as small open triangles in the data section of the graphic boring logs. The water content and dry density were also determined for each unconfined and UU triaxial compression test specimen. The water contents are plotted as small shaded circles in the data section of the graphic logs. The dry densities are tabulated to the nearest lb per cu ft under the "Dry Density" column of the graphic boring logs.

3.3 Classification Tests

The classifications and volume change properties of the fine-grained soils encountered in the borings were investigated by means of Atterberg liquid and plastic limit tests performed on selected representative samples. The results of the liquid and plastic limit tests are plotted as small crosses interconnected by dashed lines in the data section of the graphic boring logs. In accordance with the Unified Soil Classification System, fine-grained soils are classified as either clays or silts of low or high plasticity based on the results of Atterberg limit tests. The numerical difference between the liquid limit and plastic limit is defined as the plasticity index (PI). The magnitudes of the liquid limit and plasticity index and the proximity of the natural water content to the plastic limit are indicators of the potential for a fine-grained soil to shrink or swell upon changes in moisture content or to consolidate under loading. The proximity of the natural water content to the plastic limit is also an indicator of soil strength.

The classifications of some samples were investigated by means of minus No. 200 sieve tests. The percentages of fines resulting from the minus No. 200 sieve tests are tabulated at the appropriate depths under the "% Passing No. 200 Sieve" column of the graphic boring logs.

The classifications of some samples were investigated by means of sieve and hydrometer analyses. Particle size distribution curves from these tests are presented in Appendix A. The percentages of fines resulting from the sieve tests are also tabulated at the appropriate depths under the "% Passing No. 200 Sieve" column of the graphic boring logs

3.4 Water Content Tests

Water content tests were performed on samples to corroborate field classifications and to extend the usefulness of the strength, plasticity, and field SPT blow count data. The results of the water content tests are plotted as small shaded circles in the data section of the graphic boring logs. The water content data have been interconnected on the logs to illustrate a continuous profile with depth.

3.5 Soluble Sulfates, pH, and Resistivity Tests

Laboratory testing was performed on selected samples from the borings to determine the percent of soluble sulfate by mass, soil pH, and soil resistivity. Sulfate testing was performed on one sample, and soil pH and resistivity testing was performed on a different sample. Results of the tests are presented in Table 1.

Table 1 - Soluble Sulfates, pH, and Resistivity Test Results

Boring	Sample Depth (ft)	USCS	Sulfate (SO_4), % by mass	Average pH	Resistance (ohm-cm)
S5-3	48	CL	0.013	-	-
S5-4	4	SC	-	7.69	1800

4.0 GENERAL SUBSURFACE CONDITIONS

4.1 General

A general description of subsurface soil and groundwater conditions revealed by the borings made for this exploration is provided in the following paragraphs. The graphical logs shown on Figures 3 through 6 should be referred to for specific soil and groundwater conditions encountered at each boring location. Stick logs of the borings are shown in profile with the proposed bridge section on Figure 7 to aid in visualizing subsurface soil conditions. Tabulated adjacent to the stick logs are Atterberg liquid and plastic limits, water contents, dry densities, cohesions, percentages of fines passing the No. 200 sieve and field SPT blow counts.

4.2 Geology

The project site is located within the physiographic province known as the Mississippi River Alluvial Plain. Geological maps indicate Quaternary age deposits are continuous throughout the project area. The Quaternary deposits at the site include alluvial sediments from both the Holocene and Pleistocene series. Sediments typically include a substratum zone of sands and gravels overlain by a top stratum of clays and silts.

Tertiary deposits are present below the Quaternary deposits. Tertiary deposits within the project vicinity are expected to consist of hard clays, sandy clays and silty clays containing organics and lignite interbedded with very dense sand strata. Geological maps suggest that the elevation of top of the Tertiary deposits may be at about El 125 to 150 ft MSL.

4.3 Soil Stratification

As shown on the Figure 7 profile, the soils encountered at the site were grouped into the zones outlined below. The zones were generally based on the soil classifications and interpreted strengths used in design. The borings generally indicate fill materials and fine-grained top stratum soils overlying alluvial sands.

- Zone 1 – Medium dense silty sand (SM) and clayey sand (SC) with gravel, medium stiff to stiff silty clay (CL) and sandy clay (CL), and stiff clay (CH)
- Zone 2 – Loose to medium dense sand (SP) and slightly silty sand (SP-SM)
- Zone 3 – Medium dense clayey sand (SC), soft to stiff candy clay (CL), and medium stiff clay (CH)
- Zone 4 – Dense to very dense slightly silty sand (SP-SM), and loose to very dense sand (SP) with trace of gravel

Zone 1 soils were generally encountered from the ground surface down to depths ranging from about 8 to 15 ft. Zone 2 soils were encountered beneath the Zone 1 soils down to a depth of about 43 ft. Zone 3 soils were encountered beneath the Zone 2 soils down to depths ranging from about 49 to 53.5 ft. Zone 4 soils were encountered beneath the Zone 3 soils and extend to the boring termination depths.

Zone 4 was further divided into Zones 4A, 4B, 4C, and 4D based on the estimated likelihood of liquefaction and potential for strength loss due to an earthquake. The soils encountered in Zones 4A and 4C were generally identified as having a high likelihood of liquefaction and significant strength loss. The soils encountered in Zones 4B and 4D were generally identified as not being likely to liquefy. The soils in Zone 2 were identified as having a moderate likelihood of liquefaction but no significant strength loss.

We understand that new fill materials will be placed along the new alignment to create the approach embankments. The thickness of the proposed new fill at abutments along the bridge centerline is illustrated on the profile.

4.4 Groundwater

Groundwater was encountered during auger drilling at a depth of 52 ft, 54 ft, 39 ft, and 52.5 ft in Borings S5-1, S5-2, S5-3, and S5-4, respectively. Groundwater cannot be observed during rotary wash drilling. In our opinion, groundwater conditions at the site will be influenced by rainfall, surface drainage, and by the rise and fall of water levels in the nearby ditches, creeks, ponds or other bodies of water. The regional groundwater is primarily influenced by the Mississippi River. Groundwater conditions at the site can also be influenced by man-made changes. Surficial soils can become saturated and weak to relatively shallow depths during periods of prolonged and heavy rainfall.

5.0 ENGINEERING ANALYSES AND DISCUSSION

5.1 General

The purposes of this study were to perform analyses and develop geotechnical recommendations for: 1) seismic design including site classification, liquefaction, and seismic compression; 2) slope stability including proposed slope grading and configuration to provide acceptable factors of safety; and 3) deep foundation design including axial capacity curves, downdrag, lateral analysis parameters, and drivability analysis. A discussion of our analyses is provided in the following subsections.

5.2 Seismic

Seismic evaluations and analyses were generally performed based on the guidance provided by ARDOT and the recommendations discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual and in Idriss and Boulanger (2008).

5.2.1 Site Classification. Soil shear wave velocity data are not available for the bridge site. The site class was determined from SPT blow counts and undrained shear strength data in accordance with definitions provided in Table 3.10.3.1-1 of the AASHTO LRFD 2017 Bridge Design Specifications. We recommend that a site class D be utilized to determine the site coefficient and spectral response acceleration for this bridge site. The site is classified as within Seismic Zone 3 per Table 3.10.6 1.

The acceleration design response spectrum was developed using the computer program "AASHTO Seismic Design Parameters" version 2.10 developed by the U.S. Geological Survey. The recommended design values are presented subsequently in tabular format. Plots of the design spectrum are included as Figures 8 and 9.

ARDOT SR230 – Site 5

Conterminous 48 States

2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 35.910640
Longitude = -90.891930
Site Class B
Data are based on a 0.05 deg grid spacing.

Period	Sa	
(sec)	(g)	
0.0	0.416	PGA - Site Class B
0.2	0.771	Ss - Site Class B
1.0	0.197	S1 - Site Class B

Spectral Response Accelerations SDs and SD1
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1

Site Class D - Fpga = 1.08, Fa = 1.19, Fv = 2.01
Data are based on a 0.05 deg grid spacing.

Period	Sa	
(sec)	(g)	
0.0	0.450	As - Site Class D
0.2	0.917	SDs - Site Class D
1.0	**0.396**	**SD1 - Site Class D: Seismic Zone 3**

Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	Sd in.	
0.000	0.450	0.000	T = 0.0, Sa = As
0.086	0.917	0.067	
0.200	0.917	0.359	T = 0.2, Sa = SDs
0.432	0.917	1.674	T = Ts, Sa = SDs
0.500	0.793	1.937	
0.600	0.661	2.324	
0.800	0.496	3.098	
1.000	0.396	3.873	T = 1.0, Sa = SD1
1.200	0.330	4.648	
1.400	0.283	5.422	
1.600	0.248	6.197	
1.800	0.220	6.972	
2.000	0.198	7.746	
2.200	0.180	8.521	
2.400	0.165	9.295	
2.600	0.152	10.070	
2.800	0.142	10.845	
3.000	0.132	11.619	
3.200	0.124	12.394	
3.400	0.117	13.169	
3.600	0.110	13.943	
3.800	0.104	14.718	
4.000	0.099	15.002	

5.2.1 Liquefaction Triggering. Liquefaction triggering evaluations were performed using the Microsoft Excel workbook developed by Cox and Griffiths (2011)[1] and provided by ARDOT. The liquefaction evaluations were performed using all three procedures available in the workbook: Youd et al. (2001)[2], Cetin et al. (2004)[3], Idriss and Boulanger (2008)[4].

The design earthquake magnitude (M_w) was estimated using the Unified Hazard Tool on the U.S. Geological Survey (USGS) website. Deaggregations were computed using the 2008

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[1] Cox, B. R., and Griffiths, S. C. (2011). *Practical Recommendations for Evaluation and Mitigation of Soil Liquefaction in Arkansas*, MBTC 3017, Mack-Blackwell Rural Trans. Center at the U. of Arkansas.

[2] Youd, T. L., Idriss, I.M., et al. (2001). "Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops of evaluation of liquefaction resistance of soils." *J. of Geotech. and Geoevir. Engrg*., Vol. 127(4): 297-313.

[3] Cetin, K.O., Seed, R.B., Kiureghain, A.D., Tokimatsu, K., Harder, L.F., Kayen, R.E., Moss, R.E.S. (2004). “Standard Penetration Test-Based Probabilistic and Deterministic Assesment of Seismic Soil Liquefaction Potential.” *J.of Geotech. and Geoevir. Engrg.*, Vol. 130(12): 1314-1340.

[4] Idriss, I. M., and Boulanger, R. W. (2008). "Soil Liquefaction during Earthquakes." *MNO-12*, Earthquake Engineering Research Institute.

(v3.3.3) edition of the National Seismic Hazard Mapping Project (NSHMP). A return period of 5% in 50 years (i.e., 975 years) was used in the deaggregation. The resulting modal earthquake magnitude of 7.7 was input in the liquefaction triggering workbook.

The liquefaction triggering evaluation was performed for each of the borings. The liquefaction triggering workbook input is provided for each boring in Appendix B. As recommended by Cox and Griffiths (2011), a blow count N-value of 1 was input in the workbook at sample depths where SPT blow counts were not measured. For these cases, the Factor of Safety (FS) against liquefaction was not calculated. Comparison plots that show the resulting liquefaction FS values vs. elevation for each of the three evaluation procedures are provided as Figures 10, 11, 12, and 13 for Borings S5-1, S5-2, S5-3, and S5-4, respectively.

5.2.2 Seismic Compression. Potential seismic compression was calculated for all soil layers that were identified as likely to liquefy (i.e., FS<=1.0) based on the Idriss and Boulanger (2008) liquefaction triggering criteria. The seismic compression calculations were performed following two different procedures: Tomkimatsu & Seed (1987)[5] and Idriss and Boulanger (2008). The Tomkimatsu & Seed (1987) procedure for calculating seismic compression is discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

Plots that show the distribution of estimated seismic compression vs. elevation for the two procedures are provided as Figures 14, 15, 16, and 17 for Borings S5-1, S5-2, S5-3, and S5-4, respectively. For reference, the top and bottom elevation of the boring is indicated by a horizontal dashed line on each plot. As shown in these figures, the total estimated settlements at the boring locations due to seismic compression range from about 4 to 9 inches depending on the analysis method.

5.2.3 Residual Strengths of Liquefied Soils. Residual strengths for post-earthquake stability analyses were estimated for soils that were identified as likely to liquefy (i.e., FS<=1.0) based on the Idriss and Boulanger (2008) liquefaction triggering criteria. The residual strengths were estimated using the procedures outlined in Idriss and Boulanger (2008) and based on the

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[5] Tokimatsu, K. and Seed, H.B. (1987). “Evaluation of settlements in sand due to earthquake shaking.” *J. of Geotech. Engrg.*, Vol. 113(8): 861-878.

correlation proposed by Olson and Johnson (2008)[6]. The correlations proposed by Olson and Johnson (2008) are included in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

5.3 Slope Stability

Slope stability analyses were performed for the proposed conditions using the SLOPE/W computer program and the Spencer Method. The stability analyses were performed for end of construction, long term, pseudo-static, and post-earthquake conditions. We understand that the target factors of safety are 1.5 for end of construction and long-term conditions, and 1.1 for pseudo-static and post-earthquake conditions. Analyses were performed for the spill-though slopes and for the embankment side slopes. A traffic surcharge load of 250 psf was applied in pavement areas in the analyses.

The end of construction analyses use undrained strengths for cohesive soils and drained strengths for cohesionless soils. The long-term analyses use drained strengths for all soils. The pseudo-static analyses use undrained strengths for cohesive soils, drained strengths for cohesionless soils, and include a seismic coefficient equal to 0.5 times the site class specific PGA (i.e., $0.5*F_{PGA}*PGA$) as suggested in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual. The post-earthquake analyses use undrained strengths for cohesive soils, residual strengths for cohesionless soils that were identified as likely to liquefy, and drained strengths for cohesionless soils that were not identified as likely to liquefy. For cohesive soils that were estimated to have peak undrained strengths of approximately 1,500 psf or less, undrained strengths equal to 0.8 times the peak undrained strengths were used in the post-earthquake analyses to account for possible cyclic softening.

The stability analyses indicate that slope stabilization measures are required to achieve acceptable factors of safety for pseudostatic and post-seismic conditions, and the slope stabilization could be accomplished with multiple layers of geosynthetic reinforcement. In our analyses, we assumed that the geosynthetic reinforcement would have an allowable tensile strength of 20,000 lbs/ft. Each geosynthetic layer shall be continuous along its length, and it shall be placed

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[6] Olson, S. M. and Johnson, C. I. (2008). “Analyzing Liquefaction-induced Lateral Spreads Using Strength Ratios.” *J. of Geotech. and Geoenviron. Engrg.*, 134(8): 1035–1049.

to lay flat, pulled tight and pinned or weighted down to its position until the subsequent soil layer can be placed.

At the west approach embankment, 1 layer of geosynthetic reinforcement that is oriented parallel to the roadway alignment is required to stabilize the west abutment spill-through slope. The geosynthetic should extend from the mid-point of the spill-through slope back about 120 ft to at least Sta. 514+71. The geosynthetic should be placed such that the full width of the embankment is covered between the top edges of the side slopes, the distance between which measures about 40 ft. The geosynthetic should be placed at the bottom of the embankment.

At the east approach embankment, 2 layers of geosynthetic reinforcement that are oriented parallel to the roadway alignment are required to stabilize the east abutment spill-through slope. The stability analyses for pseudostatic conditions indicate that the geosynthetic should extend from the mid-point of the spill-through slope back about 260 ft to at least Sta. 520+81. However, we understand from our conversations with ARDOT, that ARDOT typically only considers failure surfaces that extend back up to 120 ft behind the top edge of the bridge abutment for pseudostatic stability analyses. In this case, the geosynthetic only needs to extend from the mid-point of the spill-through slope back about 150 ft to at least Sta. 519+71. The geosynthetic should be placed such that the full width of the embankment is covered between the top edges of the side slopes, the distance between which measures about 40 ft. The layers of geosynthetic should be placed at 1-ft vertical spacing, and the bottom layer should be placed at the bottom of the embankment.

Additional layers of geosynthetic reinforcement that are oriented perpendicular to the roadway alignment are not required.

A summary of the slope stability Factor of Safety (FS) values is provided in Table 2. The analyzed geometries, soil properties, and critical failure surfaces are shown in Figures 18 to 29. Based on our review of the soil conditions and the proposed abutment grading, we judge that the north side slope of the east abutment is the critical side slope for stability. Since the resulting FS values are acceptable, we did not perform stability analyses for the other side slopes.

Table 2 - Slope Stability FS Results Summary

Conditions	Req'd	West Abutment Spill-Through	East Abutment Spill-Through	East Abutment North Side Slope (518+29)
End of Construction	1.5	3.38	3.65	3.67
Long Term	1.5	3.44	3.60	1.62
Pseudostatic	1.1	1.14	1.10	1.30
Post-Earthquake	1.1	2.71	3.98	3.38

5.4 Consolidation Settlement

Considering the height of fill to be placed for the approach embankments and the compressibility of the soils encountered in the borings, it is our opinion that consolidation settlement of the bridge embankments will be less than 2 in. Approximately 50 percent of the settlement is expected to occur during bridge construction. No settlement problems due to consolidation settlement are anticipated at this site, and no special mitigation will be required.

5.5 Deep Foundations

We understand that driven 18-in. and 24-in. diameter, closed-ended steel pipe piles are proposed for the abutment bents and interior bents, respectively. Analyses were performed to evaluate the abutment bents and interior bents pile capacities based on the guidance provided by ARDOT and the recommendations discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

5.5.1 Axial Pile Capacity. Axial pile capacity curves were computed based on the pile type shown on the provided plans and the subsurface soil conditions encountered in the borings. Scour was not considered in our analyses. If significant scour is anticipated, we should be contacted to provide revised capacity curves.

The pile capacities were estimated based on the FHWA design procedure using the ENSOFT computer program APile v2015. The compression capacity of an individual pile consists of a combination of skin friction around the perimeter of the pile shaft and end bearing at the tip. The skin friction in the upper 5 ft of soil was neglected. Separate calculations were performed to determine pile capacities with and without consideration of seismic effects. For the calculations that consider seismic effects, the pile skin friction was reduced by 90% for liquefiable soil layers between the ground surface and a depth of 50 ft and the pile skin friction was reduced by 50% for liquefiable soil layers below a depth of 50 ft.

The pile capacity curves are presented in Figures 30, 31, and 32, for the west abutment, east abutment, and interior bents, respectively. The pile capacity curves are presented as nominal (ultimate) values that do not include a resistance factor. An appropriate resistance factor should be applied to the nominal values presented on the pile capacity curves. Guidance on resistance factors is provided in Section 6.2. We recommend that the piles extend at least 10 feet into Zone 4D (see Figure 7 profile) to ensure that the piles are tipped below the deepest soil layer with a high likelihood of liquefaction (i.e., Zone 4C).

5.5.2 Downdrag. The seismic compression of the liquefiable soil layers can result in drag loads and increased pile settlement. Pile drag loads occur when the soils surrounding a pile settle more than the pile and apply negative skin friction to the pile. These drag loads increase the compressive loads in the pile that should be considered as part of the pile structural design. Structural capacity determination of the piles is not in our scope for this investigation.

The depth at which the pile and the soils settle the same amount is referred to as the neutral plane. Below the neutral plane, the pile settles more than the surrounding soils. The depth of the neutral plane depends on the soil settlement profile, the pile length, the distribution of pile skin friction and end bearing, and the load applied to the top of the pile. The soil settlement profiles were based on the distributions of seismic compression. The distributions of pile skin friction and end bearing were based on the axial pile capacity curves that consider reduced skin friction in the liquefiable soil layers. We used unfactored dead loads provided by Neel Schaffer, Inc. as the loads applied to the tops of the piles. For the interior bent piles, we added the self-weight of the pile stick-up (between the ground surface and the bottom of the pile cap) to the unfactored deadloads.

The downdrag analysis results are summarized in the following tables. Table 3 and Table 4 present the results for the west abutment bent for loads of 105 kips and 130 kips, respectively. Table 5 and Table 6 present the results for the east abutment bent for loads of 105 kips and 130 kips, respectively. Table 7 presents the results for the interior bents for a load of 158 kips. For each case, results are provided for a range of possible pile lengths.

Table 3 - Downdrag Analysis Results for West Abutment with Load of 105 kips

	Pile Length (ft) below El 246.5 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	331	380	472	579	610
Top of Pile Settlement (in.)	2.8	2.2	1.9	0.5	0.2
Neutral Plane Depth (ft)	72.8	76.1	82.6	91.7	93.0

Table 4 - Downdrag Analysis Results for West Abutment with Load of 130 kips

	Pile Length (ft) below El 246.5 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	320	364	462	572	610
Top of Pile Settlement (in.)	2.9	2.4	1.9	0.7	0.2
Neutral Plane Depth (ft)	71.9	75.2	81.8	90.9	93.0

Table 5 - Downdrag Analysis Results for East Abutment with Load of 105 kips

	Pile Length (ft) below El 246.5 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	309	356	456	520	520
Top of Pile Settlement (in.)	5.3	5.3	2.0	0.2	0.2
Neutral Plane Depth (ft)	71.4	74.8	85.7	90.0	90.0

Table 6 - Downdrag Analysis Results for East Abutment with Load of 130 kips

	Pile Length (ft) below El 246.5 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	297	346	445	520	520
Top of Pile Settlement (in.)	5.3	5.3	2.8	0.2	0.2
Neutral Plane Depth (ft)	70.5	73.9	84.1	90.0	90.0

Table 7 - Downdrag Analysis Results for Interior Bents with Load of 158 kips

	Pile Length (ft) below El 234 ft				
	80	85	90	100	110
Maximum Drag Load (kips)	349	408	467	597	640
Top of Pile Settlement (in.)	2.3	1.8	1.8	0.8	0.1
Neutral Plane Depth (ft)	62.7	65.9	69.1	78.0	80.0

5.5.3 Lateral Analysis Parameters. If lateral loads applied to the piles are substantial, a lateral load analysis should be performed. The piles should be designed so that angular rotation and deflection at the tops of the piles are maintained within structurally tolerable limits. We recommend that the response of the piles to applied moment and lateral loading be analyzed utilizing the method developed by Dr. Lymon C. Reese of the University of Texas or a similar analysis procedure. Computer programs (e.g., LPILE) are available for this method of analysis. The analysis method utilizes finite difference approximations to solve for deflection, moment, soil modulus and soil reaction for a single pile. Soil response to the laterally loaded pile is represented in the analysis by a set of nonlinear “p-y" curves that are developed for various depths along the pile and for the different soil types. The "p-y" curves essentially indicate the soil reaction in force per unit length of pile versus deflection for a given pile diameter. A tabulation of recommended soil parameters that can be used in the lateral pile analysis are presented in Table 8. The LPILE default values of E_{50} and k, which are correlated based on the cohesion and friction angle, can be used in the lateral pile analysis.

Table 8 - Recommended Soil Parameters for Lateral Pile Analysis

Soil Zone	p-y Curve Type	Effective Unit Weight (pcf)	Cohesion (psf)	Internal Friction Angle (degrees)
New Fill	Stiff Clay w/o Free Water (Reese)	120	1500	-
1	Stiff Clay w/o Free Water (Reese)	61.6	1200	-
2	Sand (Reese)	57.6	-	34
3	Soft Clay (Matlock)	63.6	700	-
4A, 4B, 4C, 4D	Sand (Reese)	57.6	-	34

Liquefaction of sands and cyclic softening of clay soils can result in significant short-term strength losses that can reduce lateral pile capacity. Accordingly, Table 9 provides a separate set of soil parameters that should be used instead of the values in Table 8 in the lateral pile analysis for seismic conditions.

Table 9 - Recommended Post-Earthquake Soil Parameters for Lateral Pile Analysis

Soil Zone	p-y Curve Type	Effective Unit Weight (pcf)	Cohesion (psf)	Internal Friction Angle (degrees)
New Fill	Stiff Clay w/o Free Water (Reese)	120	1200	-
1	Soft Clay (Matlock)	61.6	960	-
2	Sand (Reese)	57.6	-	34
3	Soft Clay (Matlock)	63.6	560	-
4A	Soft Clay (Matlock)	57.6	440	-
4B	Sand (Reese)	57.6	-	34
4C	Soft Clay (Matlock)	57.6	420	-
4D	Sand (Reese)	57.6	-	34

5.5.4 Drivability Analysis. A "drivability" type wave equation analysis relating blow counts to pile penetration, ultimate static pile capacities, dynamic pile driving stresses, minimum recommended hammer energy and hammer strokes was performed using the program GRLWEAP v.2010. The unit skin friction and end-bearing values in each soil layer were developed based on the results of unconsolidated undrained (UU) triaxial compression tests, supplemented by the results of the field standard penetration tests and visual estimates of consistency and the static analysis program in GRLWEAP. A 72% pile hammer efficiency and a shaft gain/loss factor of 0.833 and a toe gain/loss factor of 1.0 were used in the analysis. A maximum driving stress of 90% of the steel yield strength was considered for these analyses.

Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. An open-ended D30 diesel hammer was utilized for the drivability analyses of both pile sizes. Hammer and pile cushion information was based on manufacturer-recommended values. Both the 18-in. and 24-in. diameter steel pipe piles were assumed to be installed close-ended. In the analyses, the piles at the abutments and interior bents are assumed to be driven from the plan pile cap bottom elevations to the recommended tip elevations. Graphical and tabulated results of the drivability analyses are provided in Appendix C. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed prior to hammer acceptance and beginning of driving. The resulting minimum hammer energy to drive the piles at the abutment and interior bents is provided in Table 10.

Table 10 - Results of Drivability Analyses

Location	Hammer Type	Minimum Hammer Energy (kip-ft.)
Abutment Bents	D30	70
Interior Bents	D30	70

The parameters used in the wave equation analysis were based on general information available at the time of the analysis; however, actual field conditions may be different. We recommend prudent use of the wave equation analysis results. Soil response, hammer performance, and pile stresses and drivability should be verified by dynamic measurements using the Pile Driving Analyzer (PDA) on site and subsequent data analysis with the CAPWAP program. The actual suitability and final acceptance of a hammer system for a given project can only be determined after demonstration of satisfactory field performance, which is typically evaluated during the Test Pile Driving Program with PDA dynamic pile measurements and related data analyses.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Pile Design and Installation

Driving refusal for the steel pipe piles may occur in the dense to very dense sands encountered in Zone 4 (see Figure 7 profile). If refusal occurs at depths shallower than the required minimum depth, then jetting will be required to achieve additional penetration. However, the final 5 ft of pile penetration must be achieved by driving. Driven piles should be installed in accordance with AHTD Standard Specification Section 805 PILING.

The pile capacity curves presented in this report do not reflect the effects of jetting. As described in FHWA-NHI-16-009, Design and Construction of Driven Pile Foundations, the use of jetting will result in greater soil disturbance than considered in standard static pile capacity calculations. Some field studies have reported that the pile side resistance may be reduced by about 50 percent over the jetted depth. If jetting is necessary, we should be contracted to provide revised

axial capacities. Dynamic load testing should be performed during construction to more accurately determine the ultimate capacity of the piles after jetting.

6.2 Test Piles, Dynamic Load Testing, and Resistance Factors

Based on Table 10.5.5.2.3-1 of the AASHTO LRFD 2017 Bridge Design Specifications and considering that the soil profiles consist predominantly of sand, a resistance factor of 0.45 should generally be applied for axial compression and a resistance factor of 0.35 should generally be applied for tension. A higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 11.

Table 11 - Pile Resistance Factors based on Condition/Resistance Determination Method

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 condition, but no less than 2% of the 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 the production piles.	0.65
	Wave equation analysis, without pile dynamic measurements or load test by with field confirmation of hammer performance.	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only).	0.40

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

As discussed in Section 10.5.5.3.3 of the Bridge Design Specifications, a resistance factor of 1.0 should be applied for axial compression and a resistance factor of 0.80 should be applied for tension when designing the foundations to resist earthquake loading.

We recommend a minimum of two test piles (one at an abutment bent and one at an interior bent) be driven to evaluate pile capacities and drivability, prior to ordering the production piles. The test pile lengths should be selected considering the estimated pile capacities, minimum penetration requirements, and the anticipated driving resistance. The test piles can be driven at permanent pile locations.

We recommend that dynamic pile load testing be performed on the test piles in accordance with ASTM D 4945. The results of the dynamic pile load test should be used to establish driving criteria for the production piles. The embedment length of the piles may be increased based on the PDA evaluation. All testing should be performed prior to ordering production piles in case the design lengths change due to the testing.

The dynamic pile load testing data collection should be performed by an engineer with a minimum of one year of dynamic pile testing field 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/or Foundation QA. Pile driving modeling and analysis of PDA data should be performed by an engineer with a minimum of five years of 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/or Foundation QA.

6.3 Embankment Construction

Embankment construction shall conform with Section 210 and all other applicable requirements of the latest AHTD Standard Specification for Highway Construction. The fill material for embankment construction should classify as AASHTO A-6, A-5, or A-4 with a liquid limit less than 45 and a plasticity index less than or equal to 25. The fill materials should be compacted to not less than 95 percent of standard Proctor maximum dry density (AASHTO T99) at moisture contents within 3 percentage points of the optimum moisture content. Fill material with a plasticity index less than 10 or that is susceptible to erosion shall have a minimum 18-inch clay plating (measured perpendicular to the finished slopes). Clay plating shall consist of material having a plasticity index in the range of 10 to 25 that supports vegetation and that is not highly susceptible to erosion.

As an initial site preparation step, existing utilities or pipes and any other subsurface obstructions that might interfere with earthwork, bridge, and/or drainage ditch construction should

be removed and/or relocated. Stripping should then be performed within the construction areas to remove organic-laden surficial soils, vegetation, debris, brush or roots. Temporary excavation slopes should not be steeper than 1H:1V. We recommend that excavations be left open for the shortest possible duration to minimize exposure of the bearing soils to rainfall. Drainage should be maintained away from the excavations during construction.

Prior to placement of any fill materials, the soils exposed after excavation should be inspected. Any obviously weak soils should be excavated and replaced with properly compacted backfill. The effort required to mitigate any unstable soils will be influenced by the season of the year when earthwork is performed. The soils may be drier during the hot late summer and could weaken during heavy rain events. We recommend that earthwork be performed during a dry summer or fall season, if the schedule permits. The vertical and lateral extent of excavation required to remove any weak soils must be determined in the field during earthwork construction. In order to minimize the amount of excavation, we recommend that a representative of Burns Cooley Dennis, Inc. be present to observe excavation operations and assist in evaluating the depth and lateral extent of any excavation required.

In areas where embankments are to be constructed over existing ditches, we understand that the work will conform with the requirements presented in the AHTD Special Provision for Embankment Construction, which is provided in Appendix D. This special provision requires that the ditches shall be undercut 2 feet to remove all highly organic, wet material and backfilled with Stone Backfill prior to embankment construction. The remaining embankment shall be constructed of Select Material (Class SM-2). Synthetic Filter Blanket and Dumped Riprap shall be placed on the slopes of embankments constructed of SM-2 from the top of the Stone Backfill to at least 2 feet above the high-water elevation. The remainder of embankments construction of SM-2 or other material that is susceptible to erosion shall have a minimum 18-inch clay plating (measured perpendicular to the finished slopes). Clay plating shall consist of material having a plasticity index in the range of 10 to 25 that supports vegetation and that is not highly susceptible to erosion.

As discussed in Section 210.09 of the AHTD Standard Specification, where fill materials are to be placed and compacted against a slope, the slope shall be continuously benched as the fill lifts are placed and compacted.

Laboratory classification tests, including grain size analyses and Atterberg limit determinations, should be performed on the backfill soils initially and routinely during earthwork

operations to check for compliance with the recommendations provided herein. Field moisture and density tests should be performed at frequencies that satisfy the requirements specified in Section 210.02 of the AHTD Standard Specification.

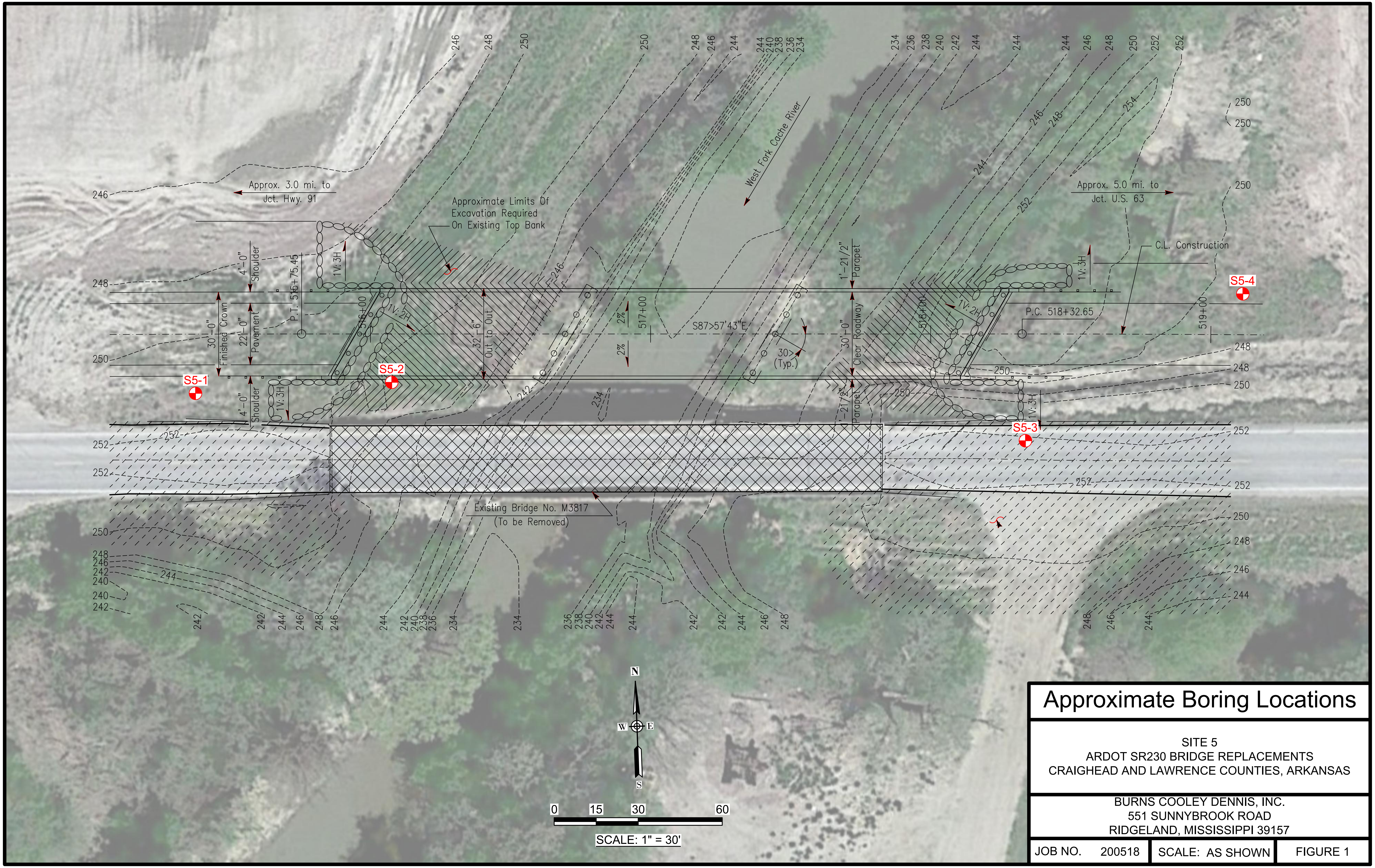
7.0 REPORT LIMITATIONS

The analyses, conclusions, and recommendations discussed in this report are based on conditions as they existed at the time of the exploration and further on the assumption that the exploratory borings are representative of subsurface conditions throughout the areas investigated. It should be noted that actual subsurface conditions between and beyond the borings might differ from those encountered at the boring locations. If subsurface conditions are encountered during construction that vary from those discussed in this report, Burns Cooley Dennis, Inc. should be notified immediately in order that we may evaluate the effects, if any, on earthwork and foundation design and construction.

Burns Cooley Dennis, Inc. should be retained for a general review of final design drawings and specifications. It is advised that we also be retained to observe earthwork for the project, to perform and observe the pile testing, and to develop the pile driving criteria. Our involvement during construction would give opportunity for us to help confirm that our recommendations are valid or to modify them accordingly. Burns Cooley Dennis, Inc. cannot assume responsibility or liability for the adequacy of recommendations if we do not observe construction.

This report has been prepared for the exclusive use of Neel-Schaffer, Inc. for specific application to the geotechnical-related aspects of design and construction of the ARDOT SR230 Bridge Replacements in Craighead and Lawrence Counties, Arkansas. The only warranty made by us in connection with the services provided is we have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing in the same or similar locality. No other warranty, express or implied, is made or intended.

FIGURES



Approximate Boring Locations

SITE 5
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO.	200518	SCALE: AS SHOWN	FIGURE 1

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL & LETTER		DESCRIPTION
COARSE-GRAINED SOILS More than half of material larger than No. 200 sieve size	GRAVELS More than half of coarse fraction larger than No.4 sieve size	Clean Gravels (Little or no fines)	▨	GW	WELL GRADED GRAVEL, GRAVEL-SAND MIXTURE
			▨	GP	POORLY GRADED GRAVEL, GRAVEL-SAND MIXTURE
		Gravels with fines (Appreciable amount of fines)	▨	GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURE
			▨	GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY MIXTURE
	SANDS More than half of coarse fraction smaller than No.4 sieve size	Clean Sands (Little or no fines)	▨	SW	WELL GRADED SAND, GRAVELLY SAND
			▨	SP	POORLY GRADED SAND, GRAVELLY SAND
		Sands with fines (Appreciable amount of fines)	▨	SM	SILTY SAND, SAND-SILT MIXTURE
			▨	SC	CLAYEY SAND, SAND-CLAY MIXTURE
FINE-GRAINED SOILS More than half of material smaller than No. 200 sieve size	SILTS AND CLAYS	Liquid limit less than 50	▨	ML	SILT WITH LITTLE OR NO PLASTICITY
			▨	ML	CLAYEY SILT, SILT WITH SLIGHT TO MEDIUM PLASTICITY
			▨	ML	SANDY SILT
			▨	CL	SILTY CLAY, LOW TO MEDIUM PLASTICITY
			▨	CL	SANDY CLAY, LOW TO MEDIUM PLASTICITY (30% TO 50% SAND)
	SILTS AND CLAYS	Liquid limit greater than 50	▨	MH	SILT, HIGH PLASTICITY
			▨	CH	CLAY, HIGH PLASTICITY
			▨	OH	ORGANIC CLAY OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			▨	PT	PEAT, HUMUS, SWAMP SOIL

TERMS CHARACTERIZING SOIL STRUCTURE

Slickensided	- Clays with polished and striated planes created as a result of volume changes related to shrinking, swelling and/orchanges in overburden pressure.
Fissured	- Clays with a blocky or jointed structure generally created by seasonal shrinking and swelling.
Laminated	- Composed of thin alternating layers of varying color and texture.
Calcareous	- Containing appreciable quantities of calcium carbonate.
Parting	- Paper thin (less than 1/8 inch).
Seam	- 1/8 inch to 3 inch thickness.
Layer	- Greater than 3 inches in thickness.

TERMS CHARACTERIZING SOIL STRUCTURE

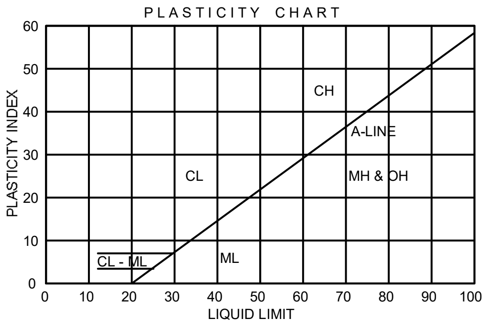
COARSE-GRAINED SOILS		FINE-GRAINED SOILS		
	PENETRATION RESISTANCE, N		PENETRATION COHESION	RESISTANCE, N
DENSITY	Blows per Foot	Consistancy	Kips/Sq.Ft	Blows per Foot
Very loose	0 - 4	Very Soft	<0.25	0 - 1
Loose	5 - 10	Soft	0.25 - 0.50	2 - 4
Medium Dense	11 - 30	Medium Stiff	0.50 - 1.00	5 - 8
Dense	31 - 50	Stiff	1.00 - 2.00	9 - 15
Very Dense	>4.00	Very Stiff	2.00 - 4.00	16 - 30
		Hard	>4.00	>30

PARTICLE SIZE IDENTIFICATION

Cobbles	- Greater than 3 inches
Gravel	- Coarse-3/4 inch to 3 inches Fine-4.76 mm to 3/4 inch
Sand	- Coarse-2 mm to 4.76 mm Medium-0.42 mm to 2 mm Fine-0.074 mm to 0.42 mm
Silt & Clay	- Less than 0.074 mm

RELATIVE COMPOSITION

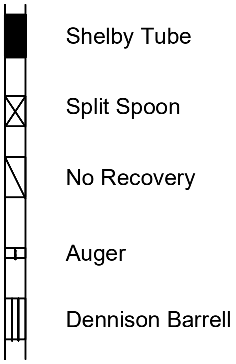
Slightly	5 - 15%
With	16 - 29%
Sandy (or gravelly)	30 - 50%



FOR CLASSIFICATION OF FINE GRAINED SOILS

SAMPLE TYPES

(Shown in Sample Column)



CLASSIFICATION, SYMBOLS AND TERMS USED ON GRAPHICAL BORING LOGS

BURNS COOLEY DENNIS, INC.

FIGURE 2

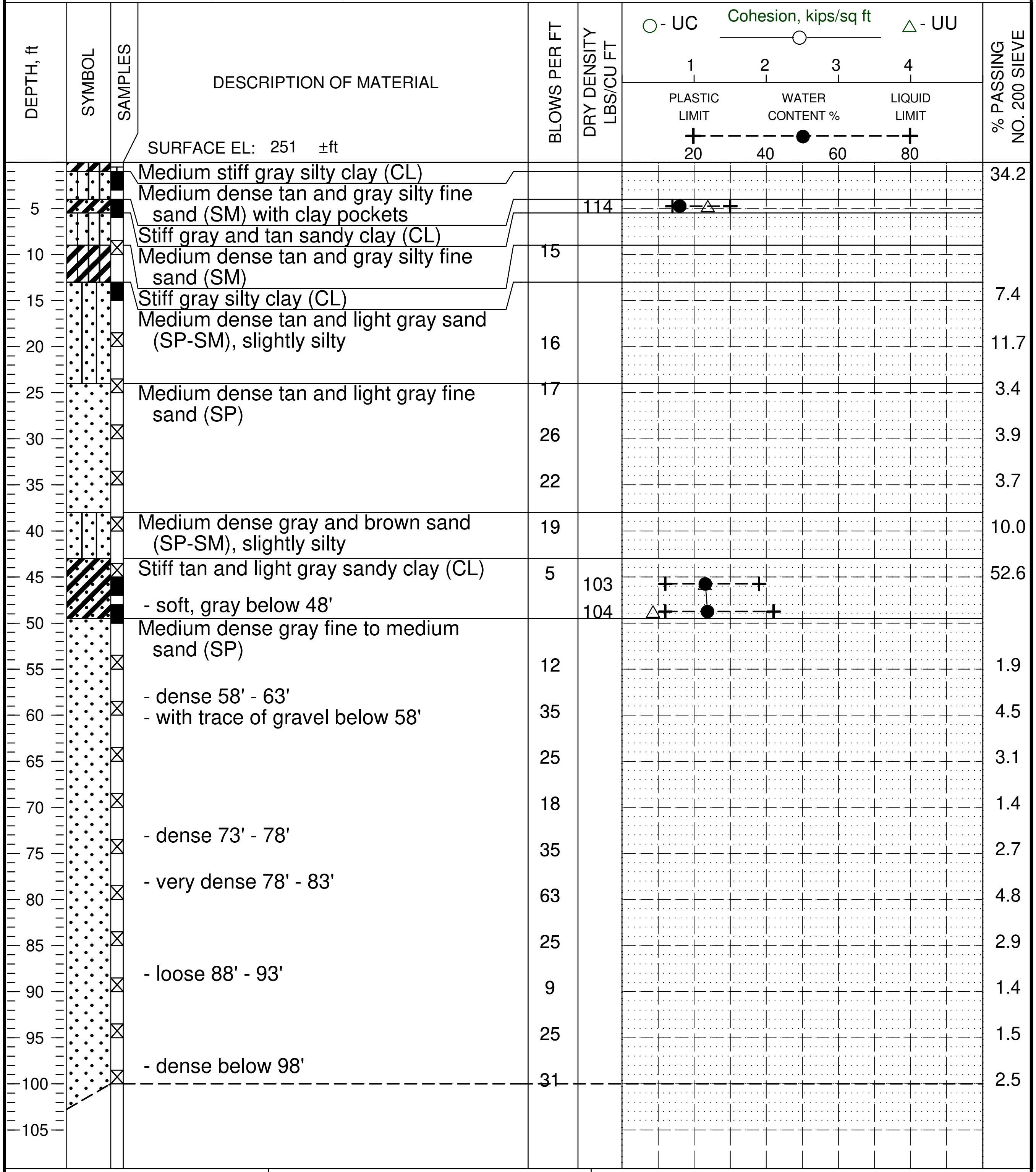
LOG OF BORING NO. S5-1

ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 55', then rotary wash to completion.

LOCATION: Sta. 515+38 (Approximate)
+/- 21' Right of Construction C/L

SURFACE EL: 251 ±ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
Medium stiff gray silty clay (CL)			34.2
Medium dense tan and gray silty fine sand (SM) with clay pockets		114	
Stiff gray and tan sandy clay (CL)			
Medium dense tan and gray silty fine sand (SM)	15		
Stiff gray silty clay (CL)			7.4
Medium dense tan and light gray sand (SP-SM), slightly silty	16		11.7
Medium dense tan and light gray fine sand (SP)	17		3.4
	26		3.9
	22		3.7
Medium dense gray and brown sand (SP-SM), slightly silty	19		10.0
Stiff tan and light gray sandy clay (CL)	5		52.6
		103	
- soft, gray below 48'		104	
Medium dense gray fine to medium sand (SP)			
	12		1.9
- dense 58' - 63' - with trace of gravel below 58'	35		4.5
	25		3.1
	18		1.4
- dense 73' - 78'	35		2.7
- very dense 78' - 83'	63		4.8
	25		2.9
- loose 88' - 93'	9		1.4
	25		1.5
- dense below 98'	31		2.5

BORING DEPTH: 100 ft

DATE: 09/11/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 38.32" - W 90° 53' 30.95"

GROUNDWATER DATA: Free water encountered at an approximate depth of 52' during auger drilling. Water level remained at an approximate depth of 52' after about 15 minutes.

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FIGURE 3

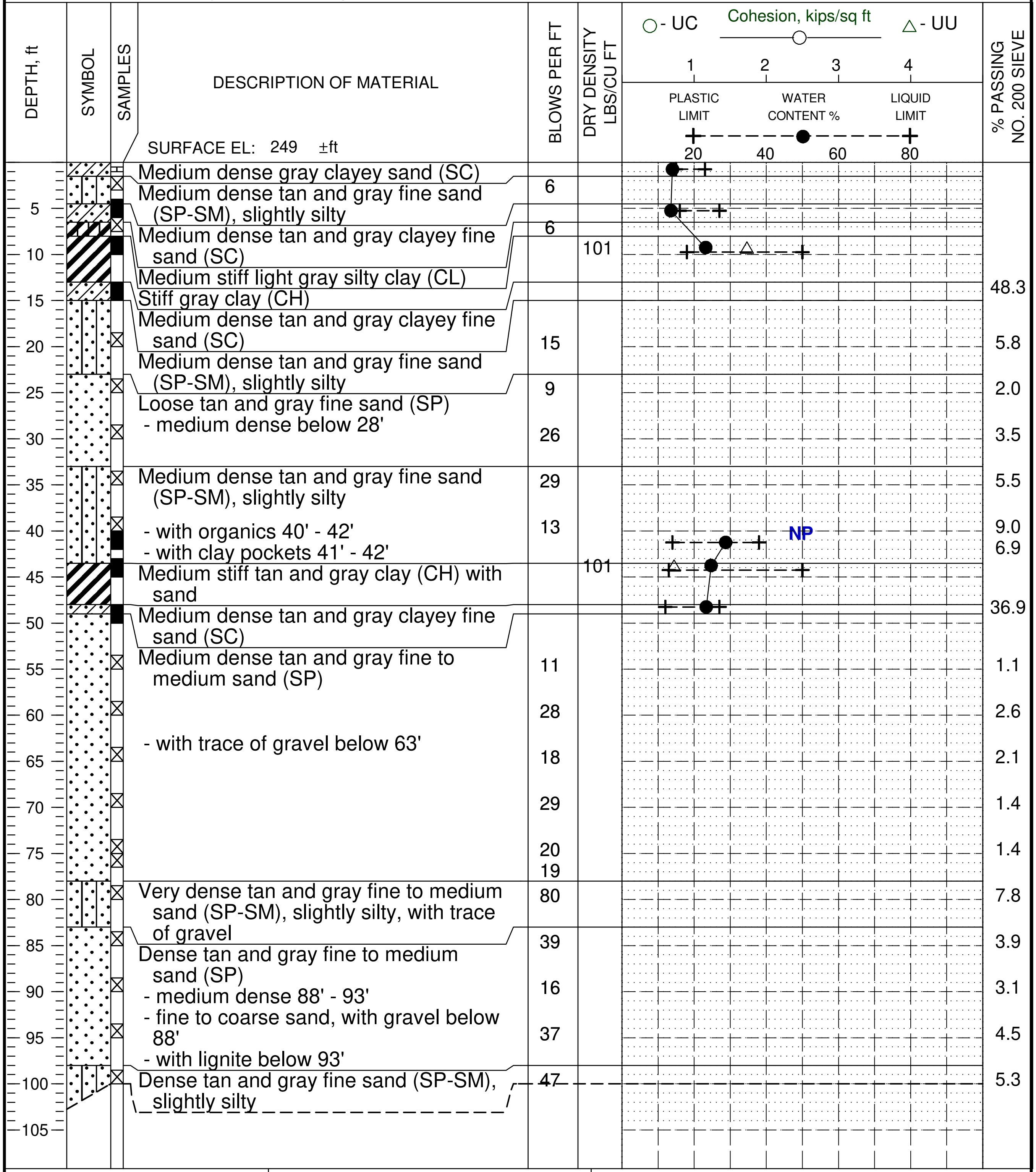
LOG OF BORING NO. S5-2

ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 60', then rotary wash to completion.

LOCATION: Sta. 516+08 (Approximate)
+/- 17' Right of Construction C/L

SURFACE EL: 249 ±ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
Medium dense gray clayey sand (SC)			
Medium dense tan and gray fine sand (SP-SM), slightly silty	6		
	6		
Medium dense tan and gray clayey fine sand (SC)		101	
Medium stiff light gray silty clay (CL)			48.3
Stiff gray clay (CH)			
Medium dense tan and gray clayey fine sand (SC)	15		5.8
Medium dense tan and gray fine sand (SP-SM), slightly silty	9		2.0
Loose tan and gray fine sand (SP) - medium dense below 28'	26		3.5
Medium dense tan and gray fine sand (SP-SM), slightly silty	29		5.5
- with organics 40' - 42'	13		9.0
- with clay pockets 41' - 42'			6.9
Medium stiff tan and gray clay (CH) with sand		101	
Medium dense tan and gray clayey fine sand (SC)			36.9
Medium dense tan and gray fine to medium sand (SP)	11		1.1
	28		2.6
- with trace of gravel below 63'	18		2.1
	29		1.4
	20		1.4
	19		
Very dense tan and gray fine to medium sand (SP-SM), slightly silty, with trace of gravel	80		7.8
Dense tan and gray fine to medium sand (SP)	39		3.9
- medium dense 88' - 93'	16		3.1
- fine to coarse sand, with gravel below 88' - with lignite below 93'	37		4.5
Dense tan and gray fine sand (SP-SM), slightly silty	47		5.3

BORING DEPTH: 100 ft

DATE: 09/14/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 38.35" - W 90° 53' 30.09"

GROUNDWATER DATA: Free water encountered at an approximate depth of 54' during auger drilling. Water level remained at an approximate depth of 54' after about 15 minutes.

FIGURE 4

LOG OF BORING NO. S5-3

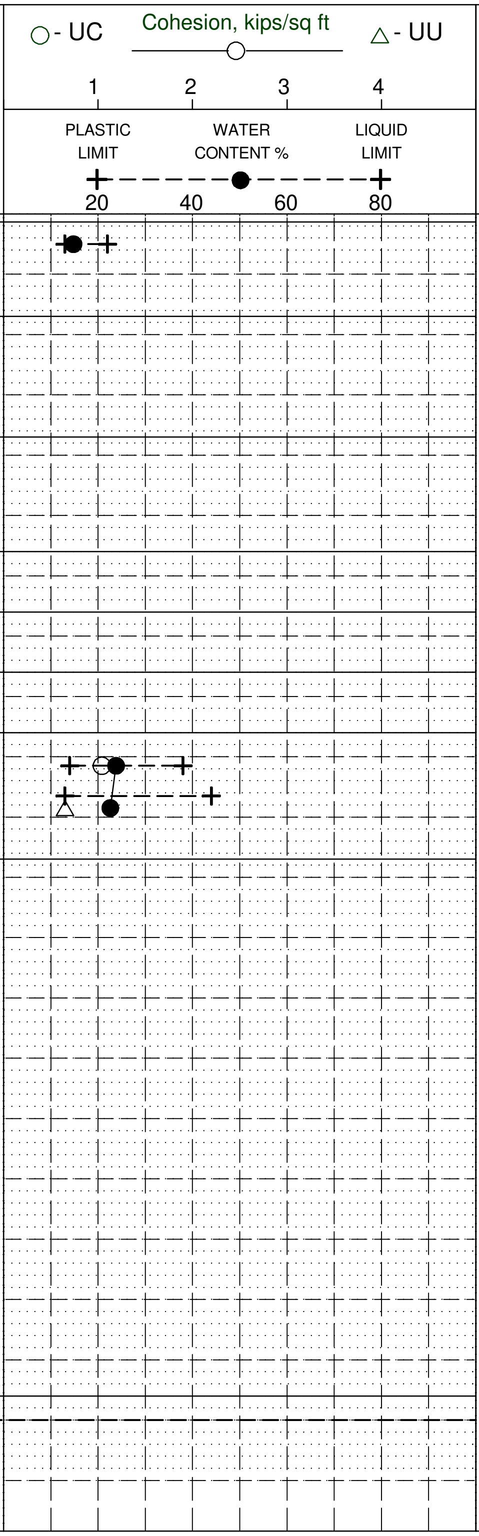
ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 60', then rotary wash to completion.

LOCATION: Sta. 518+34 (Approximate)
+/- 38' Right of Construction C/L

SURFACE EL: 252 ±ft

DEPTH, ft	DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
	Asphalt Pavement (7")			
	Medium dense tan and gray clayey fine sand (SC) with gravel	16		
5				38.8
				47.1
10	Medium dense brown and tan fine sand (SP-SM), slightly silty - with wood fragments to 10'	15		
15		13		6.9
20	Medium dense tan and light gray fine sand (SP)	19		3.0
25		18		3.3
30	Medium dense tan and light gray fine sand (SP-SM), slightly silty	17		9.2
35	Medium dense tan and light gray fine sand (SP)	22		3.5
40	Medium dense tan and gray fine to medium sand (SP-SM), slightly silty	15		8.3
45	Stiff gray sandy clay (CL)	4		49.9
			107	
50	- medium stiff below 48'		101	
55	Dense tan and gray fine to medium sand (SP)	33		4.2
60	- medium dense 58' - 68' - fine sand below 63'	19		2.0
65		28		3.1
70		31		3.5
75	- medium dense 73' - 78' - with trace of gravel 73' - 95'	26		2.2
80		40		3.2
85	- loose 83' - 88'	9		2.4
90	- medium dense 88' - 93'	12		2.9
95		45		4.5
100	Very dense tan and gray fine sand (SP-SM), slightly silty	76		8.9
105				



BORING DEPTH: 100 ft

DATE: 09/16/20

COMMENTS: Borehole filled with cement-bentonite grout. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 38.02" - W 90° 53' 27.38"

GROUNDWATER DATA: Free water encountered at an approximate depth of 39' during auger drilling. Water level remained at an approximate depth of 39' after about 15 minutes.

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BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

FIGURE 5

LOG OF BORING NO. S5-4

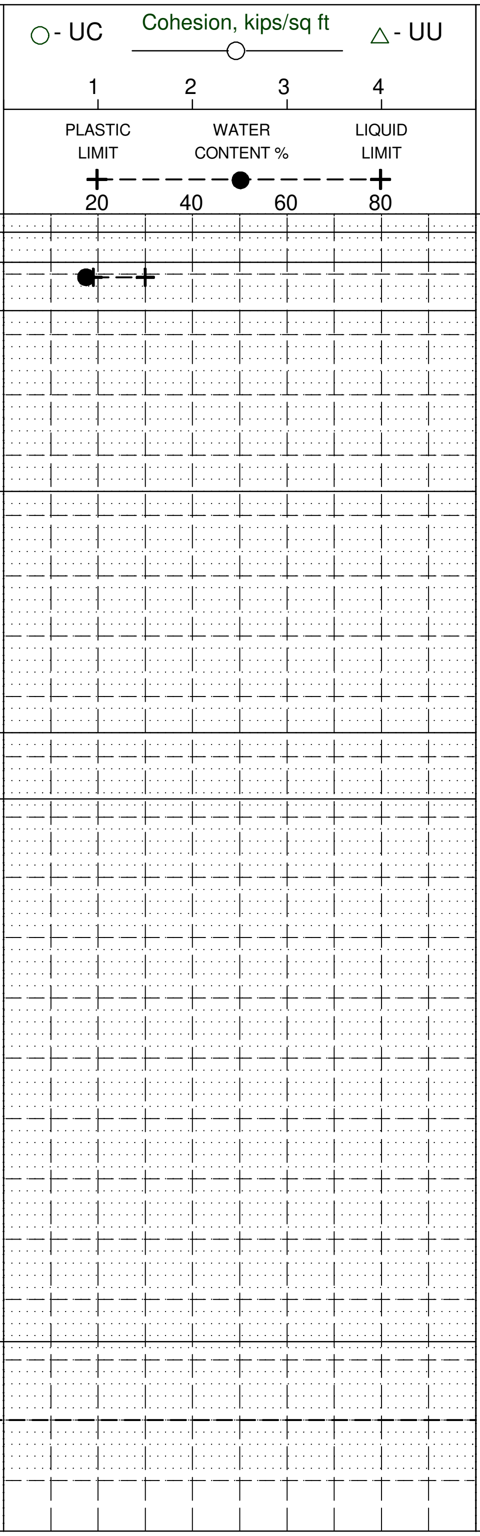
ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 55', then rotary wash to completion.

LOCATION: Sta. 519+11 (Approximate)
+/- 15' Left of Construction C/L

SURFACE EL: 250 ±ft

DEPTH, ft	DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
	Medium dense brown silty fine sand (SM)	4		82.5
5	Medium stiff light gray and tan silty clay (CL) with sand			38.0
10	Medium dense tan and brown clayey sand (SC)			7.1
15	Medium dense tan and gray fine sand (SP-SM), slightly silty	9		6.4
20	- loose 13' - 18'	13		6.6
25	Medium dense tan and light gray fine sand (SP)	12		2.0
30		15		3.0
35		17		2.6
40		22		3.4
45	Stiff tan and light gray sandy clay (CL)	6		57.6
	- medium stiff below 48'			2.5
50	Medium dense tan and light gray fine sand (SP)			
55	- fine to medium sand 53' - 88'	16		.6
	- with trace of gravel 58' - 63'			
60		23		2.0
65		30		3.5
	- dense 68' - 73'			
70		31		2.5
	- with trace of gravel 73' - 88'			
75		23		2.8
80		22		2.7
	- very dense 83' - 88'			
85		52		3.0
	- fine sand below 88'			
90		28		3.5
95	Very dense tan and gray fine sand (SP-SM), slightly silty	100		5.6
100		87		5.3
105				



BORING DEPTH: 100 ft

DATE: 09/15/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 38.49" - W 90° 53' 26.37"

GROUNDWATER DATA: Free water encountered at an approximate depth of 52.5' during auger drilling. Water level remained at an approximate depth of 52.5' after about 15 minutes.

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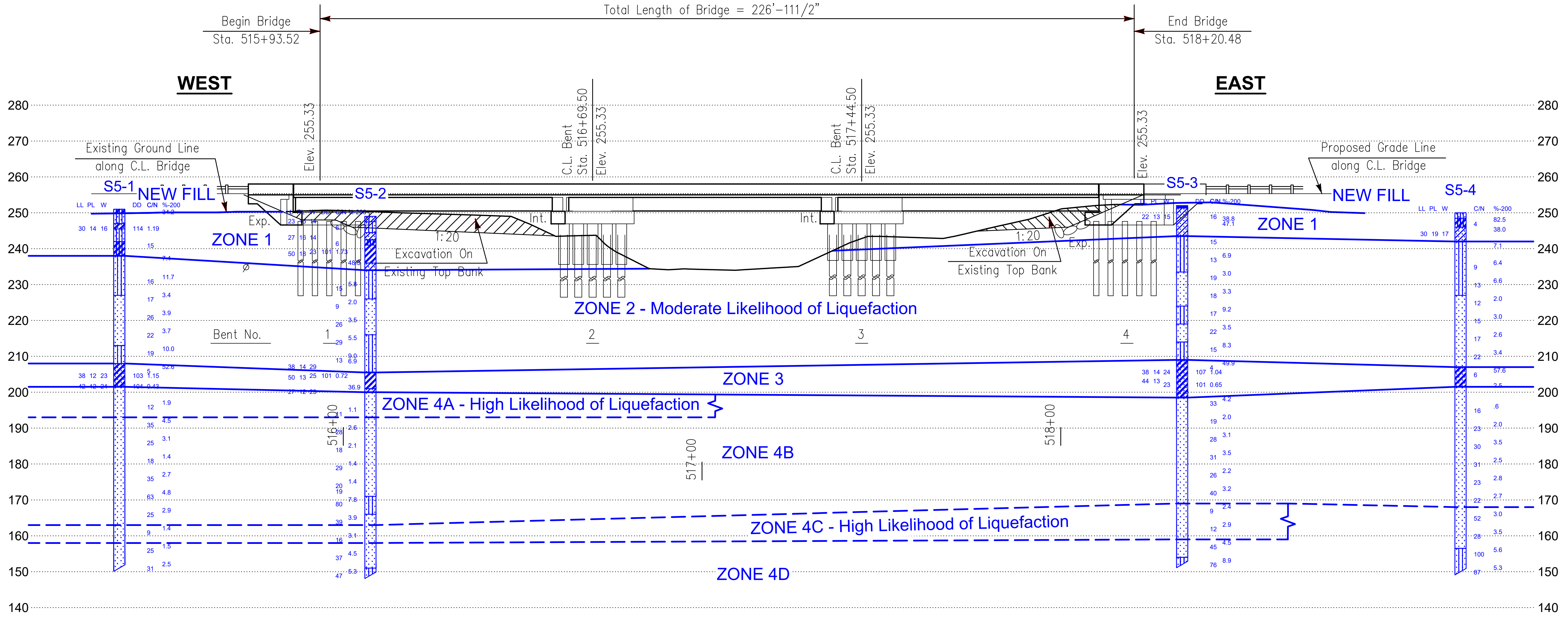
BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

FIGURE 6



ZONE 1

Medium dense silty sand (SM) & clayey sand (SC) with gravel,
medium stiff to stiff silty clay (CL) & sandy clay (CL), & stiff clay (CH)

ZONE 2

Loose to medium dense sand (SP) & sand (SP-SM), slightly silty

ZONE 3

Medium dense clayey sand (SC), soft to stiff sandy clay (CL), & medium stiff clay (CH)

ZONE 4

Dense to very dense sand (SP-SM), slightly silty, & loose to very dense sand (SP) with trace of gravel

0 15 30 60

SCALE: 1" = 30'

Note: The SPT blow count "N" values are raw values. They have not been corrected for hammer energy. A hammer energy correction factor of 1.36 applies to borings S5-1, S5-2, S5-3 & S5-4.

Soil Profile

SITE 5
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO.	200518	SCALE: AS SHOWN	FIGURE 7

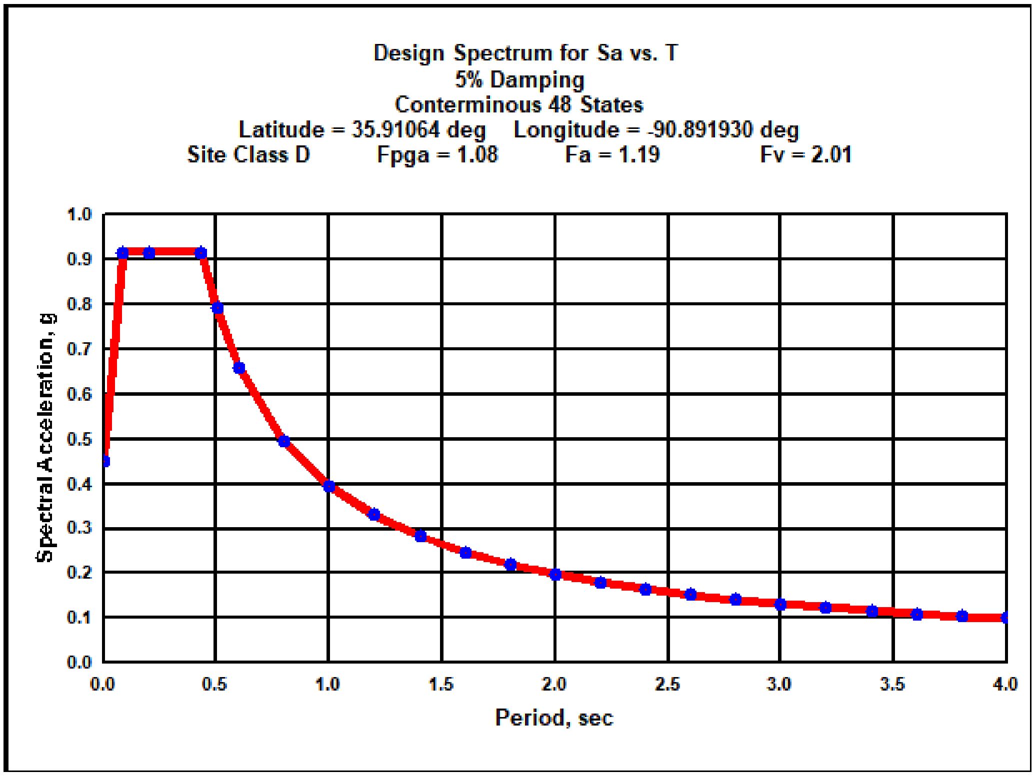


Figure 8 - Seismic Design Spectrum for Sa vs. T

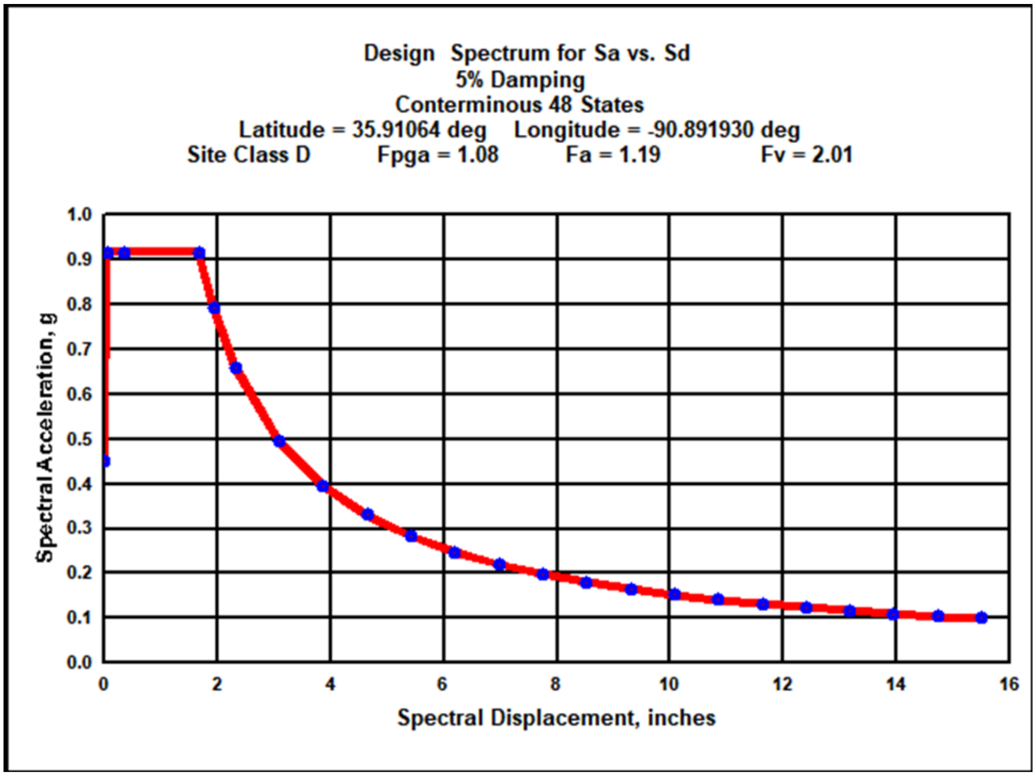


Figure 9 - Seismic Design Spectrum for Sa vs. Sd

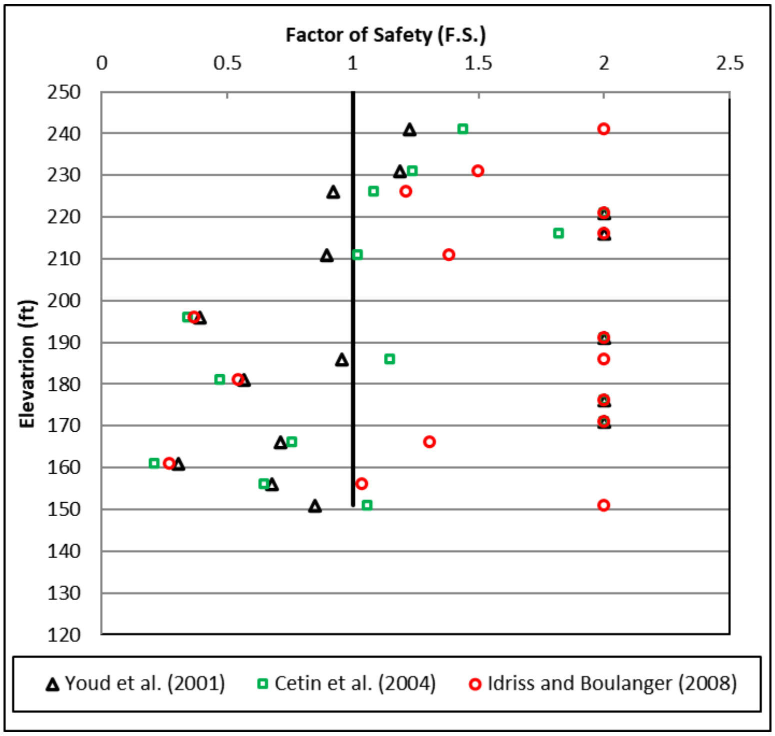


Figure 10 - Liquefaction Triggering FS Values for S5-1 (Top of Boring at EL 251 ft)

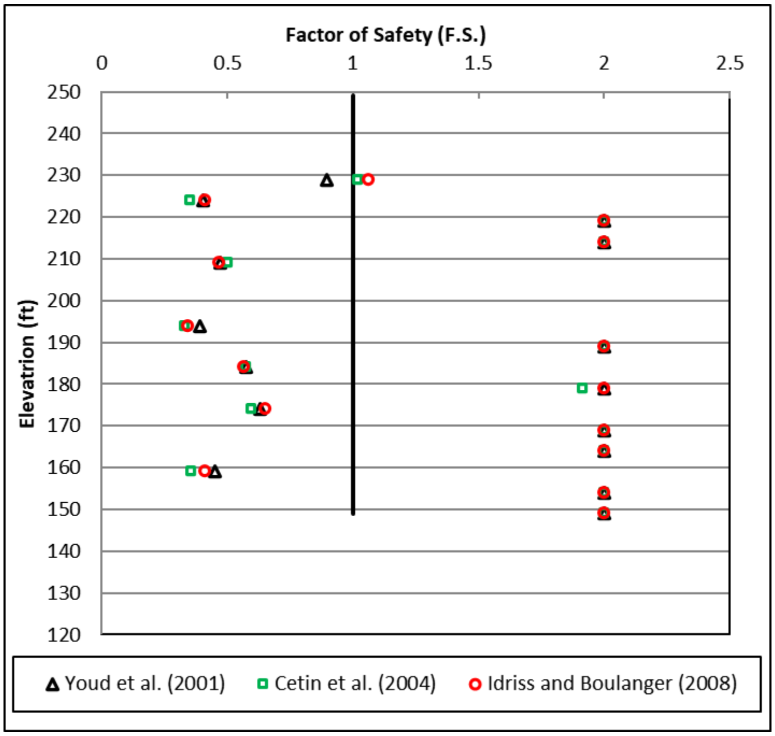


Figure 11 - Liquefaction Triggering FS Values for S5-2 (Top of Boring at EL 249 ft)

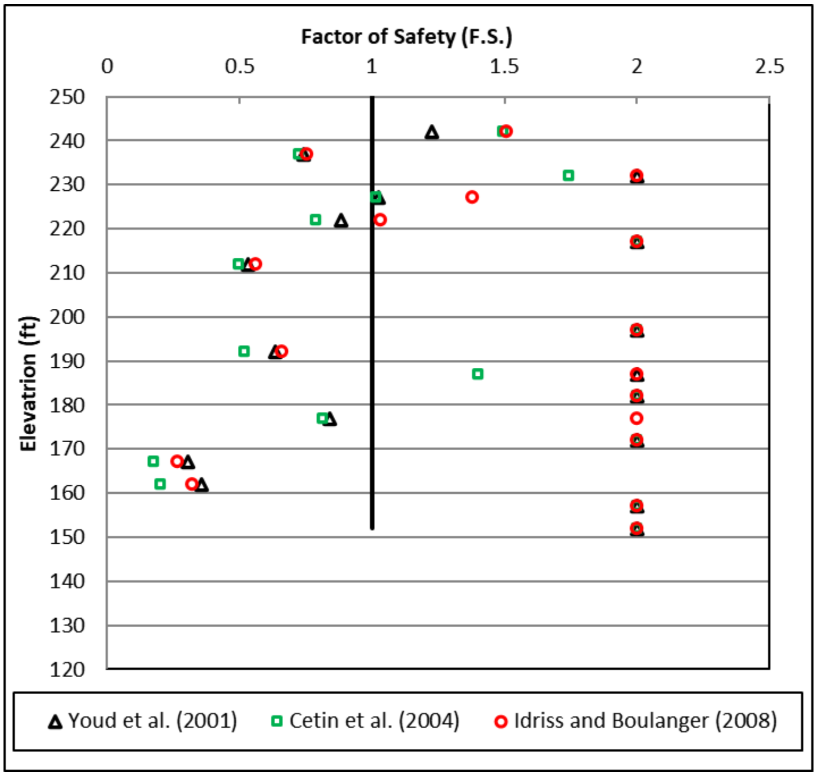


Figure 12 - Liquefaction Triggering FS Values for S5-3 (Top of Boring at EL 252 ft)

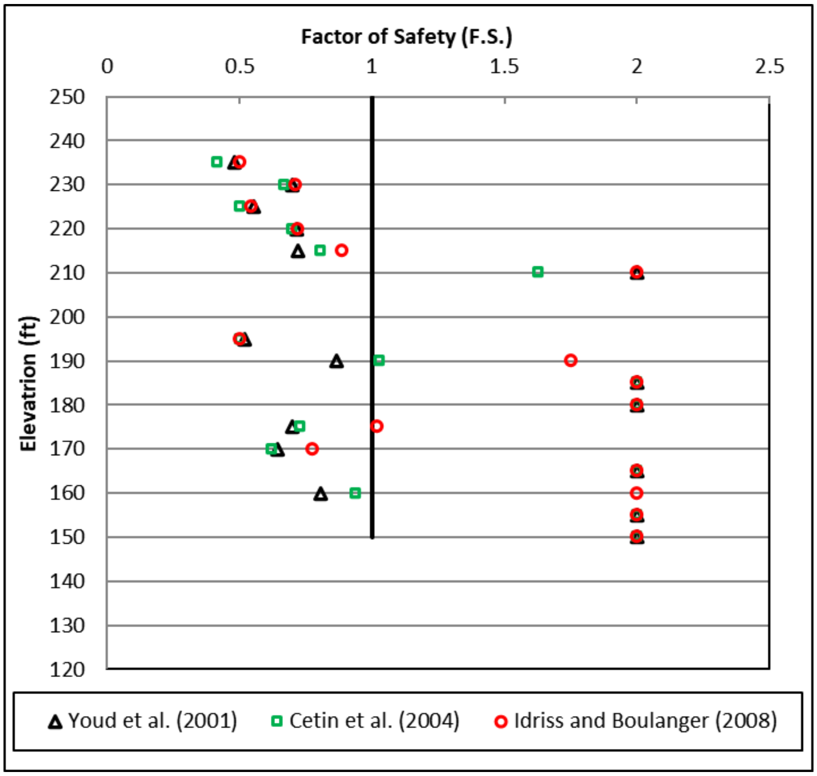


Figure 13 - Liquefaction Triggering FS Values for S5-4 (Top of Boring at EL 250 ft)

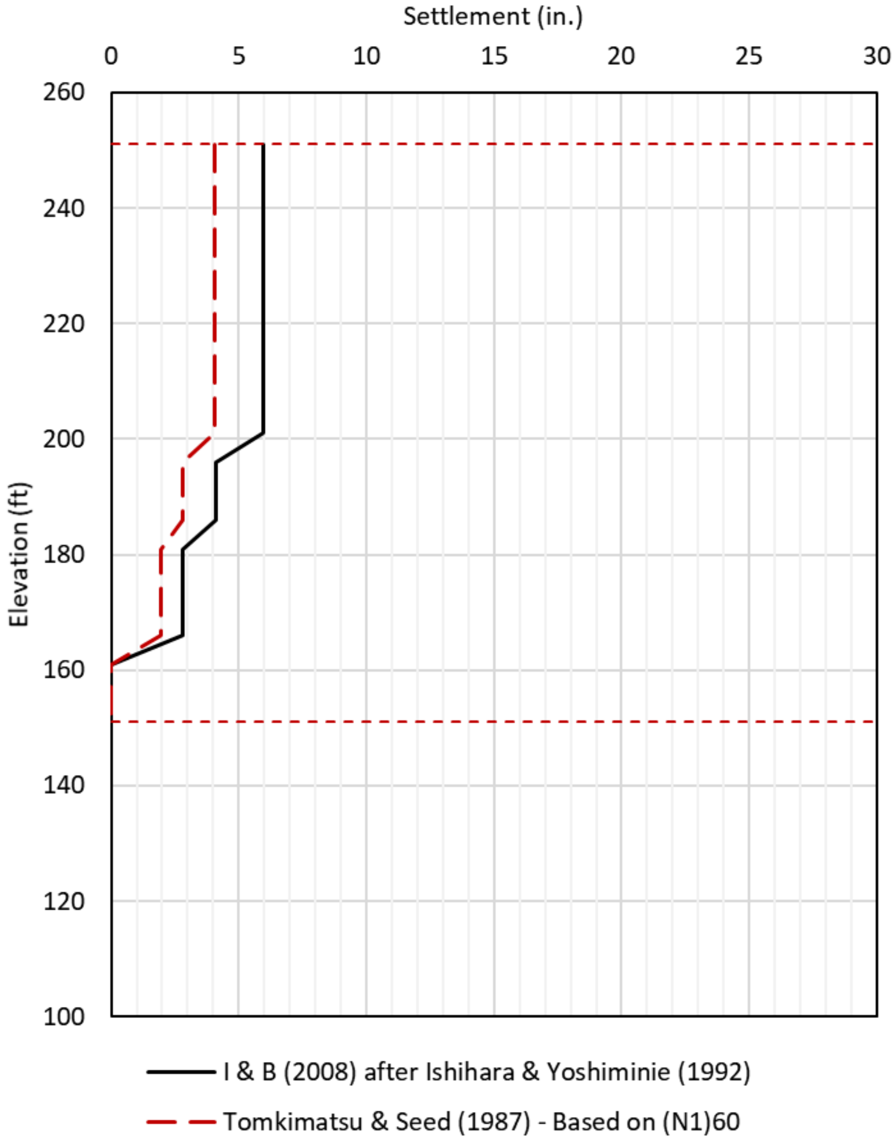


Figure 14 – Seismic Compression for S5-1 (Top of Boring at EL 251 ft)

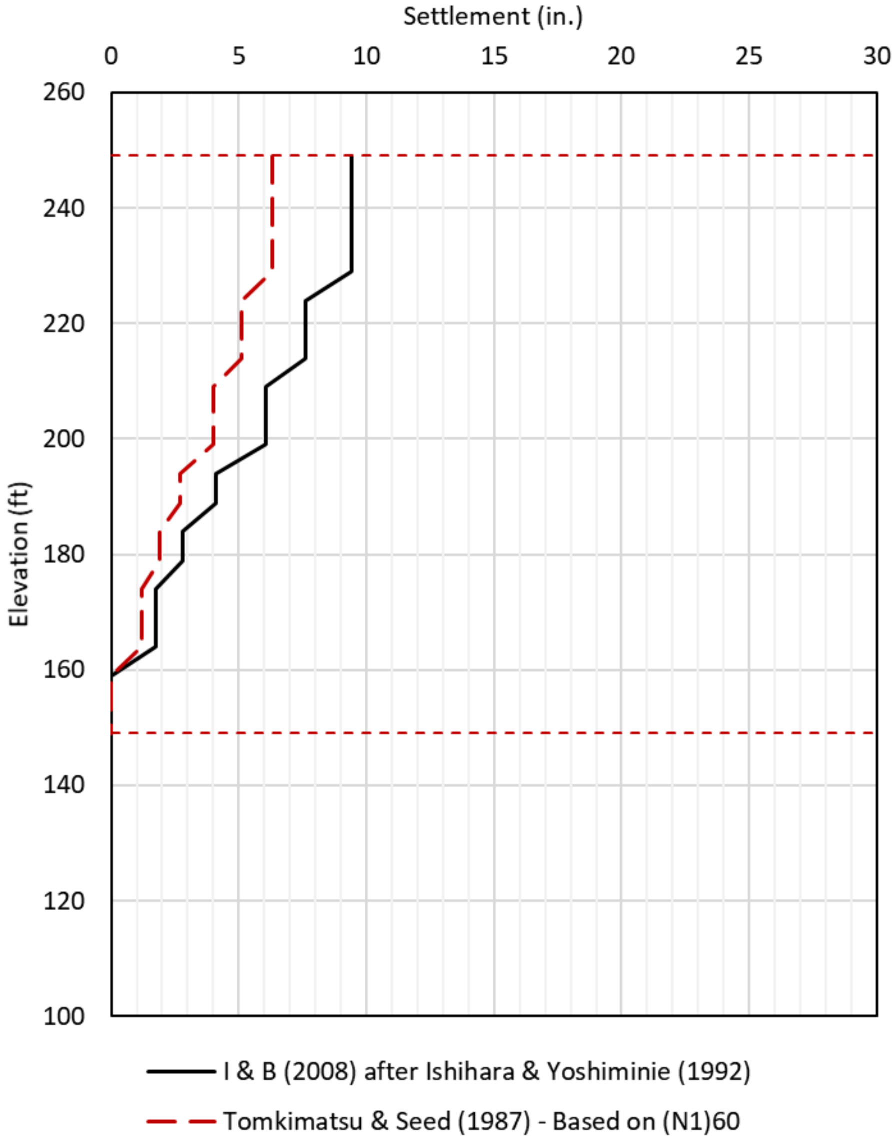


Figure 15 - Seismic Compression for S5-2 (Top of Boring at EL 249 ft)

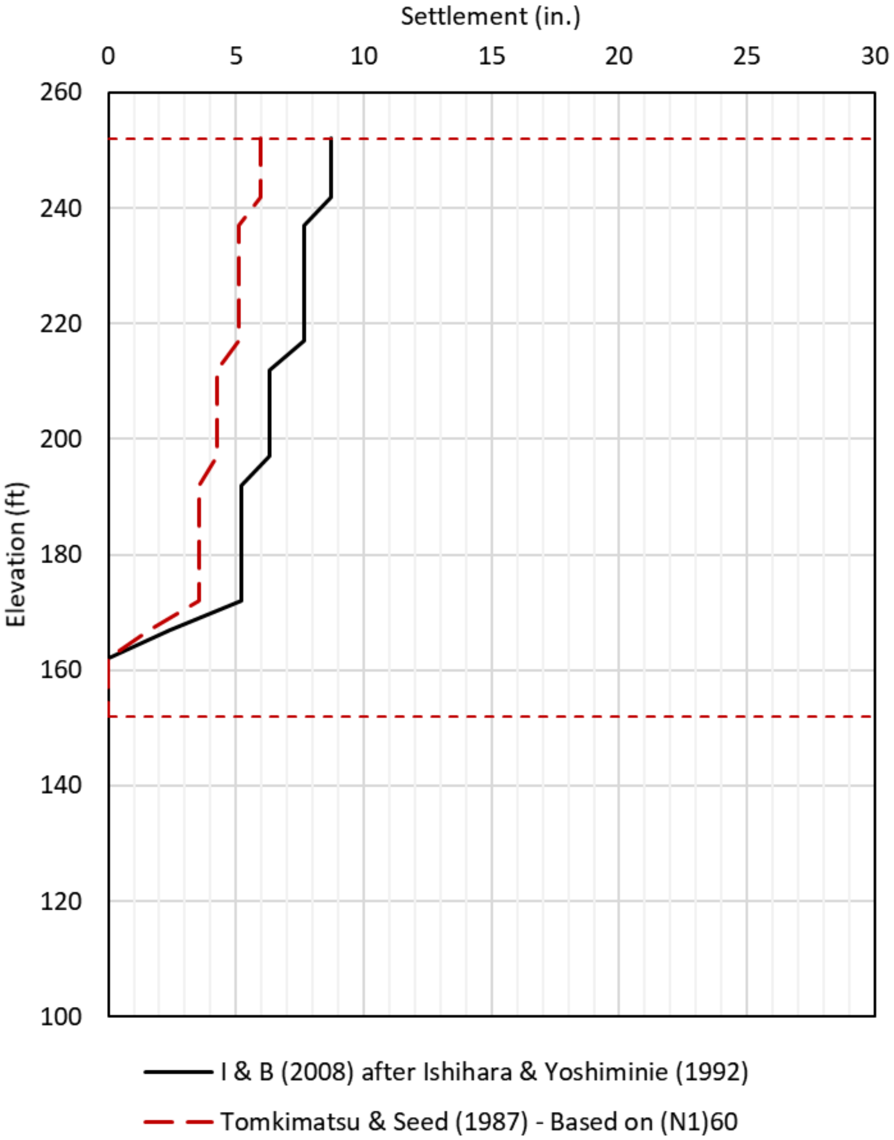


Figure 16 - Seismic Compression for S5-3 (Top of Boring at EL 252 ft)

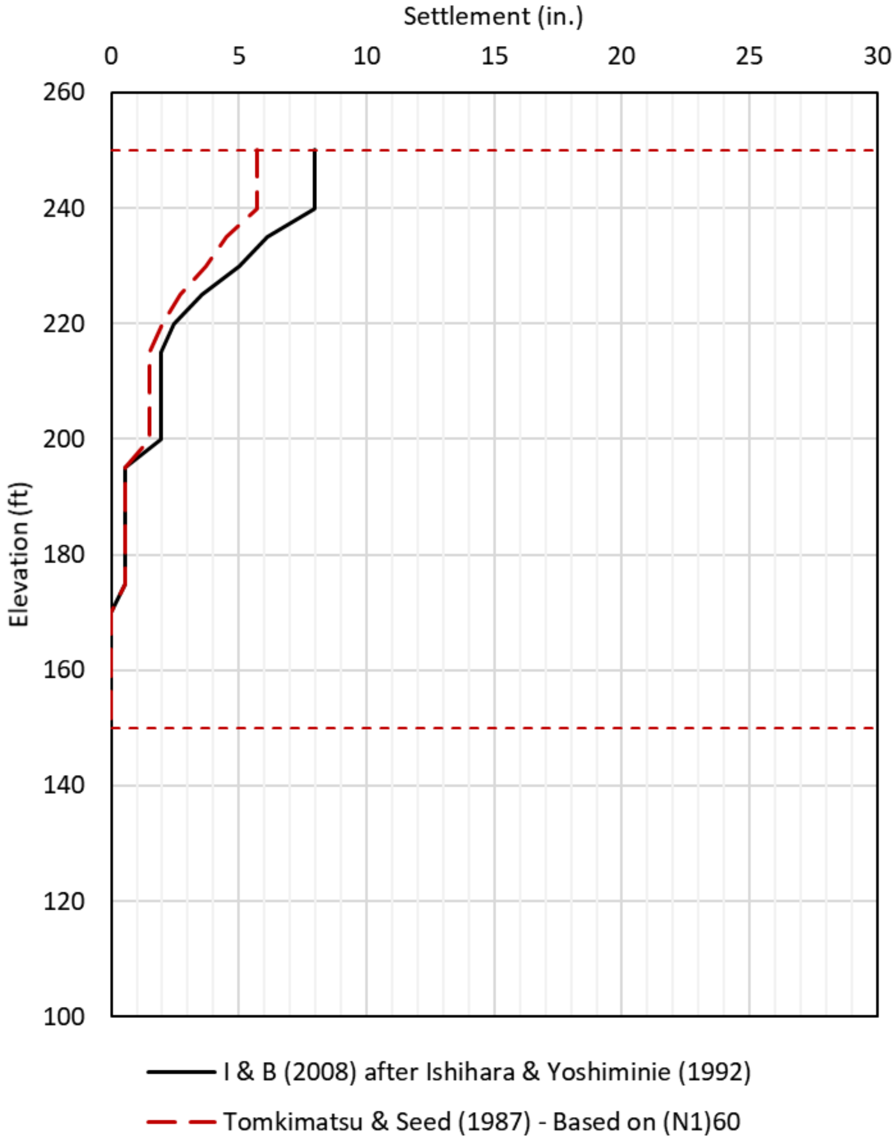
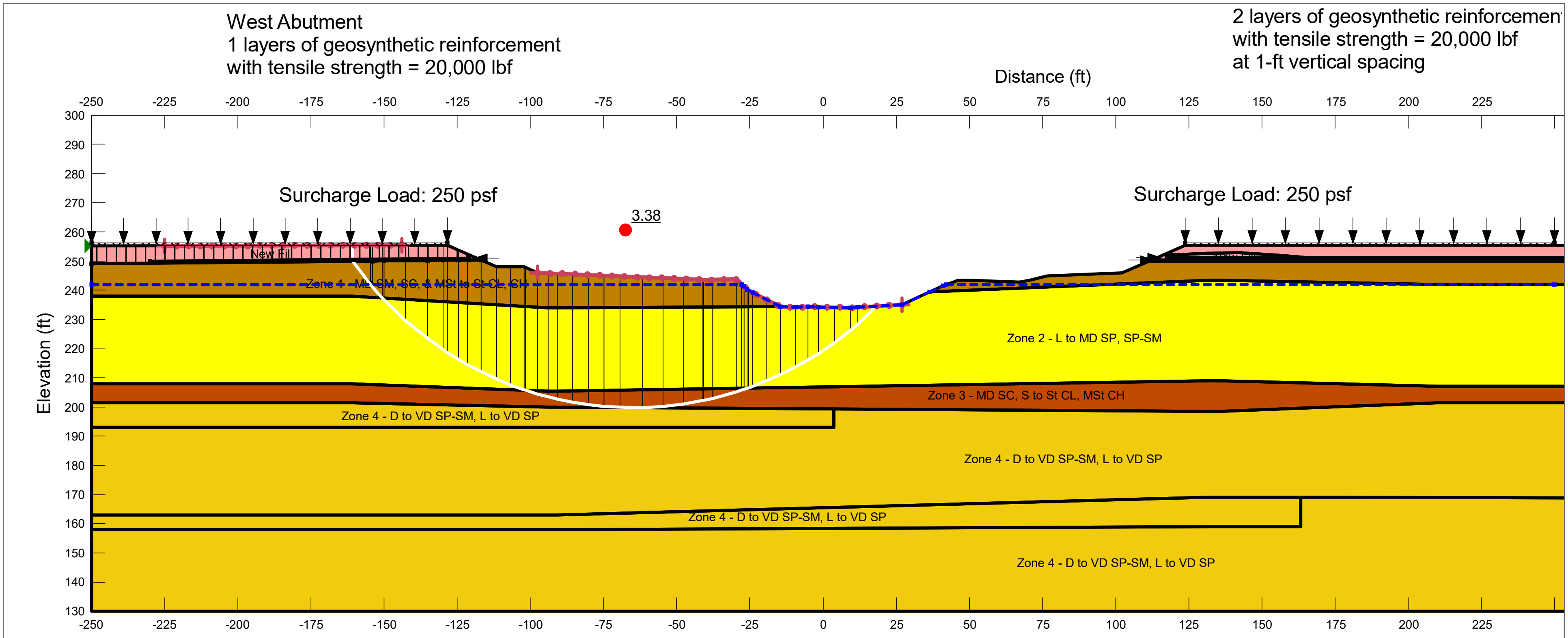


Figure 17 - Seismic Compression for S5-4 (Top of Boring at EL 250 ft)



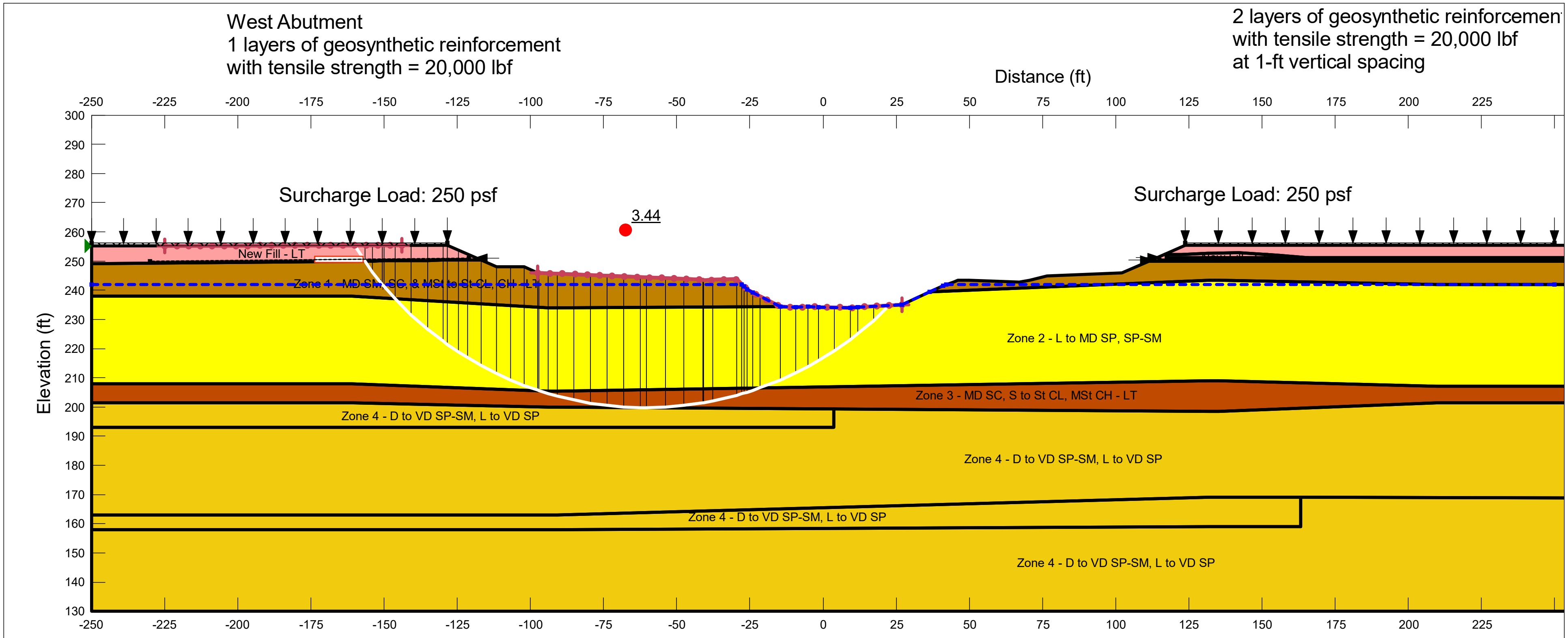
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH	Mohr-Coulomb	124	1,200	0
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH	Mohr-Coulomb	126	840	0
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34

West Abutment Spill-Through
End of Construction

SR 230 Site 5
Craighead County, AR

Figure 18

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



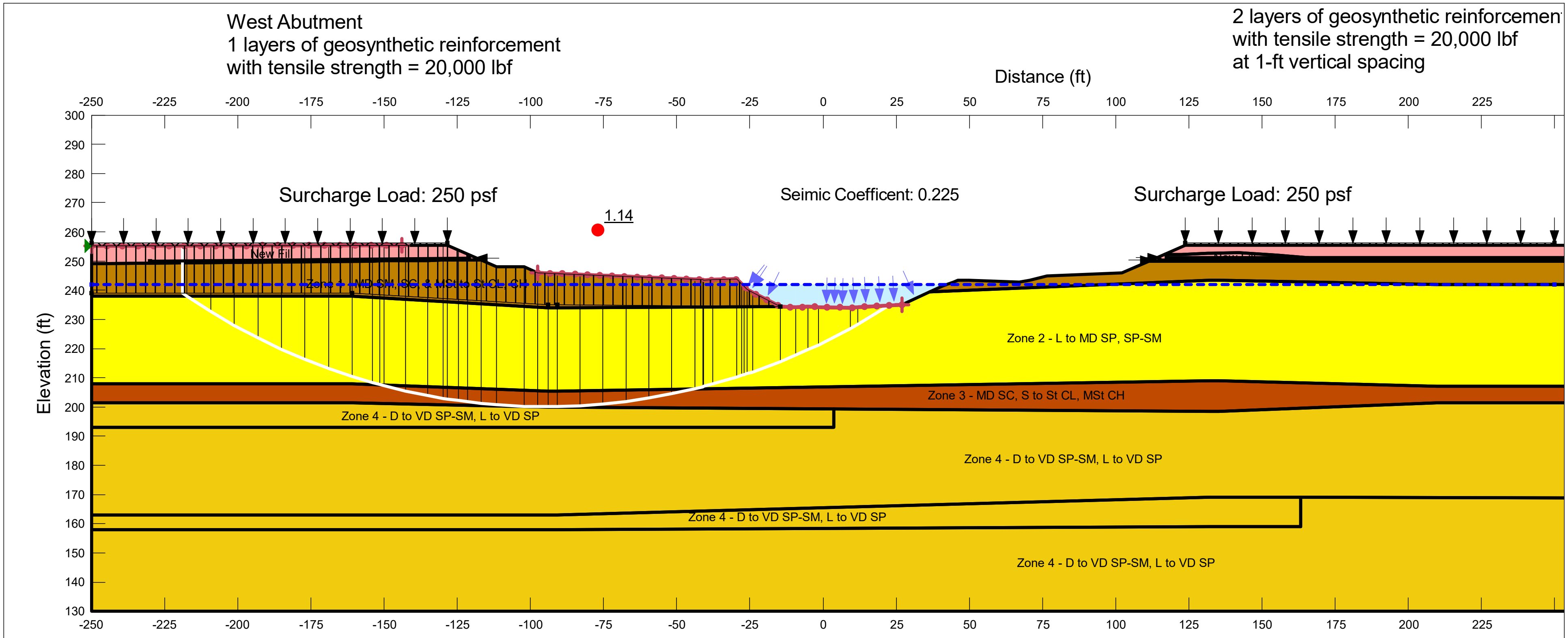
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH - LT	Mohr-Coulomb	124	50	21
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH - LT	Mohr-Coulomb	126	50	21
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34

West Abutment Spill-Through
Long Term

SR 230 Site 5
Craighead County, AR

Figure 19

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH	Mohr-Coulomb	124	1,200	0
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH	Mohr-Coulomb	126	840	0
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34

West Abutment Spill-Through
Pseudostatic

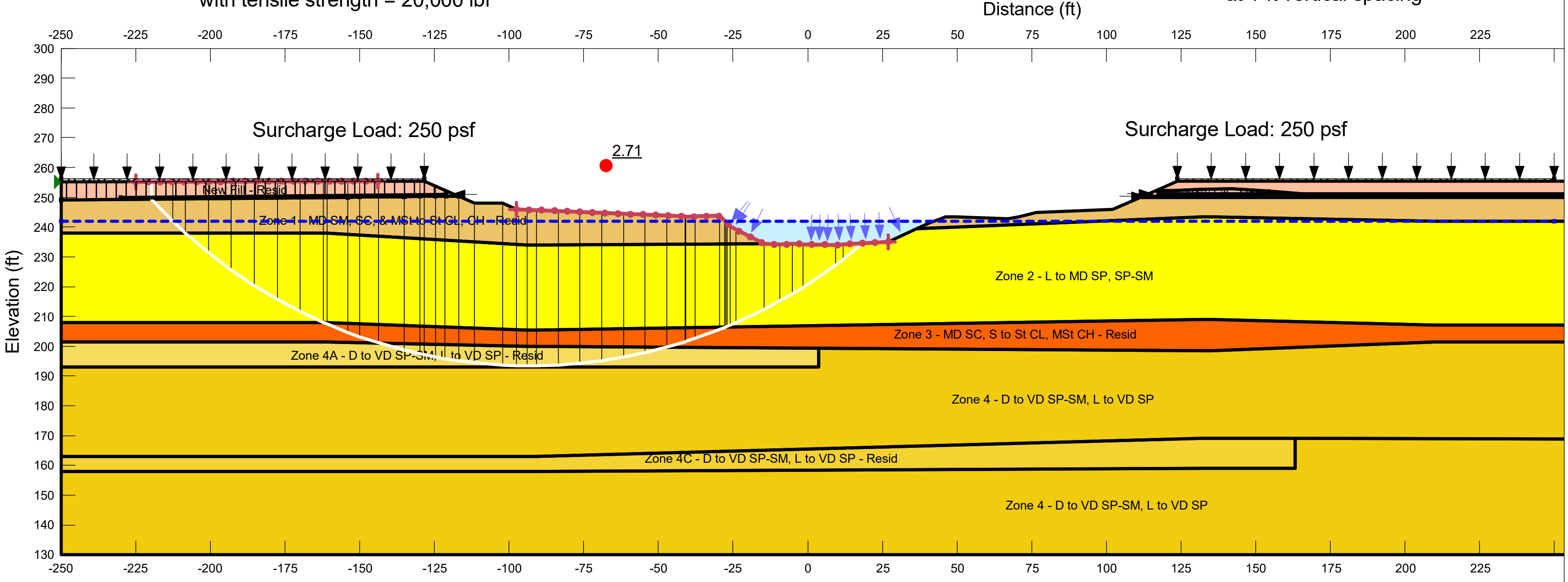
SR 230 Site 5
Craighead County, AR

Figure 20

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

West Abutment
1 layers of geosynthetic reinforcement
with tensile strength = 20,000 lbf

2 layers of geosynthetic reinforcemen
with tensile strength = 20,000 lbf
at 1-ft vertical spacing



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH - Resid	Mohr-Coulomb	124	960	0
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH - Resid	Mohr-Coulomb	126	672	0
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34
🟨	Zone 4A - D to VD SP-SM, L to VD SP - Resid	Mohr-Coulomb	120	440	0
🟨	Zone 4C - D to VD SP-SM, L to VD SP - Resid	Mohr-Coulomb	120	420	0

West Abutment Spill-Through
Post-Seismic

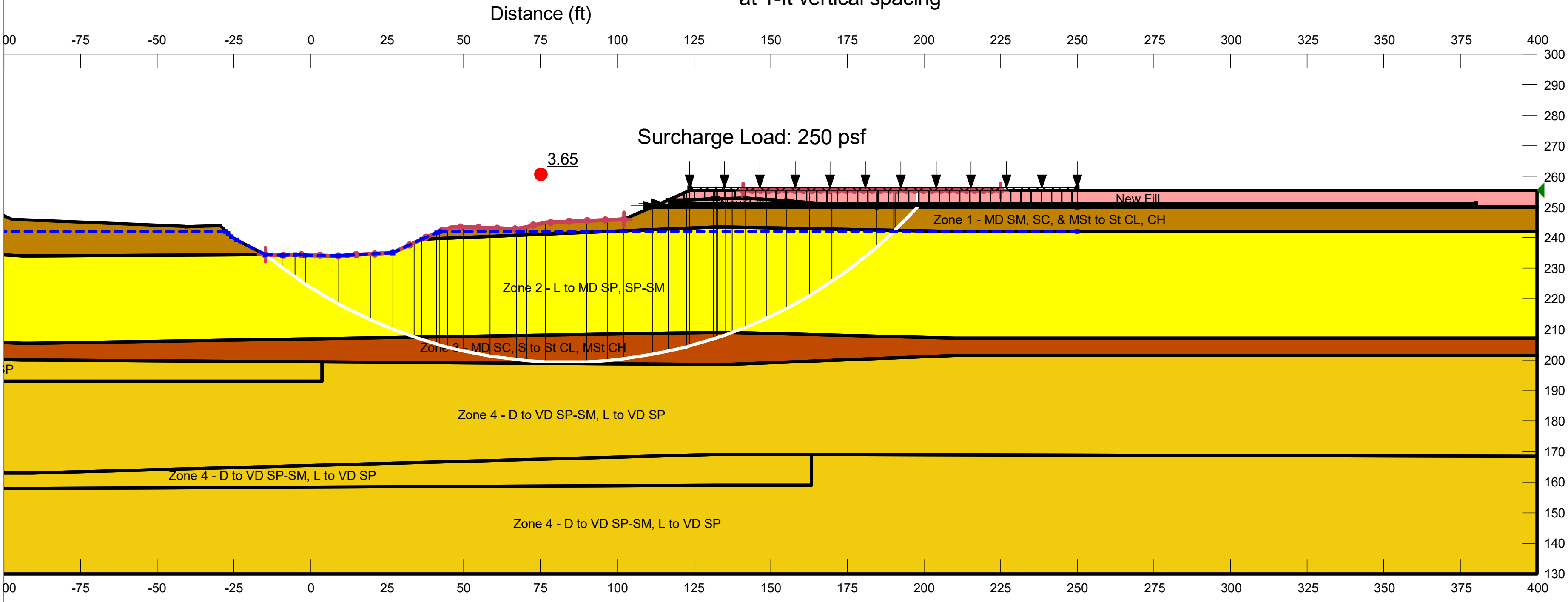
SR 230 Site 5
Craighead County, AR

Figure 21

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

ent

2 layers of geosynthetic reinforcement
with tensile strength = 20,000 lbf
at 1-ft vertical spacing



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH	Mohr-Coulomb	124	1,200	0
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH	Mohr-Coulomb	126	840	0
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34

East Abutment Spill-Through
End of Construction

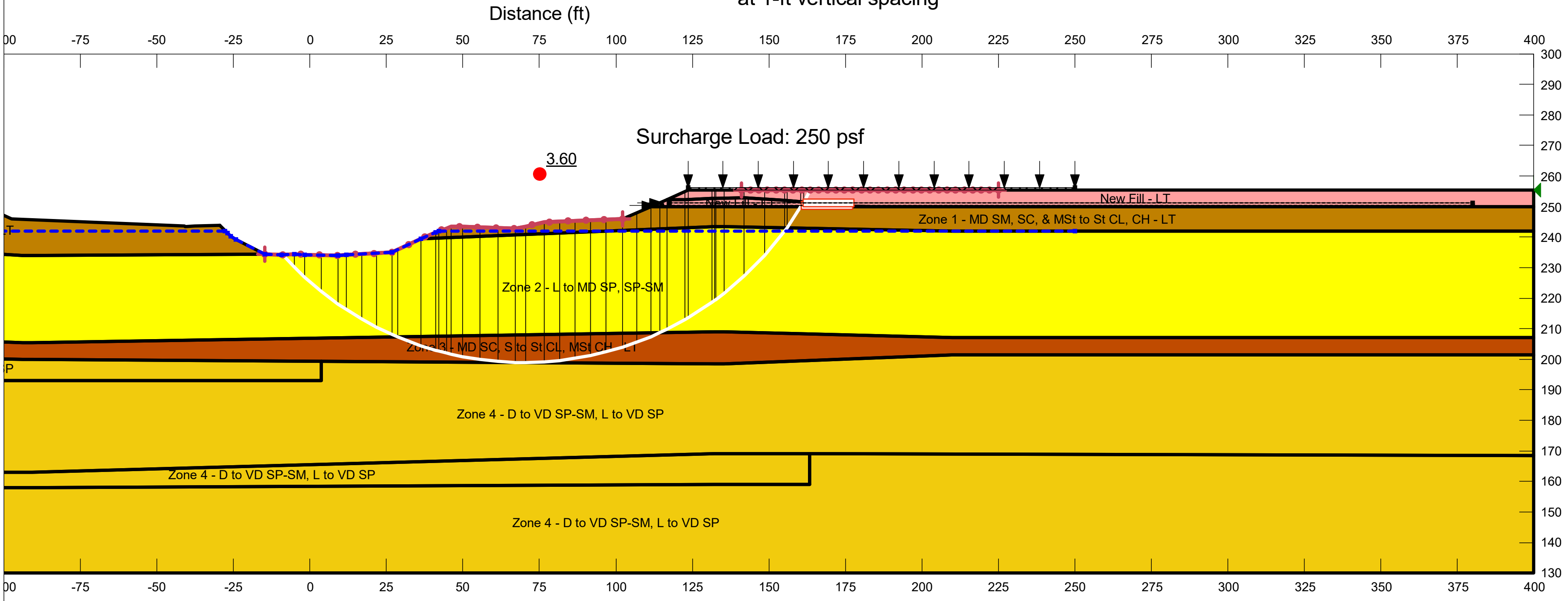
SR 230 Site 5
Craighead County, AR

Figure 22

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

ent

2 layers of geosynthetic reinforcement
with tensile strength = 20,000 lbf
at 1-ft vertical spacing



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH - LT	Mohr-Coulomb	124	50	21
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH - LT	Mohr-Coulomb	126	50	21
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34

East Abutment Spill-Through
Long Term

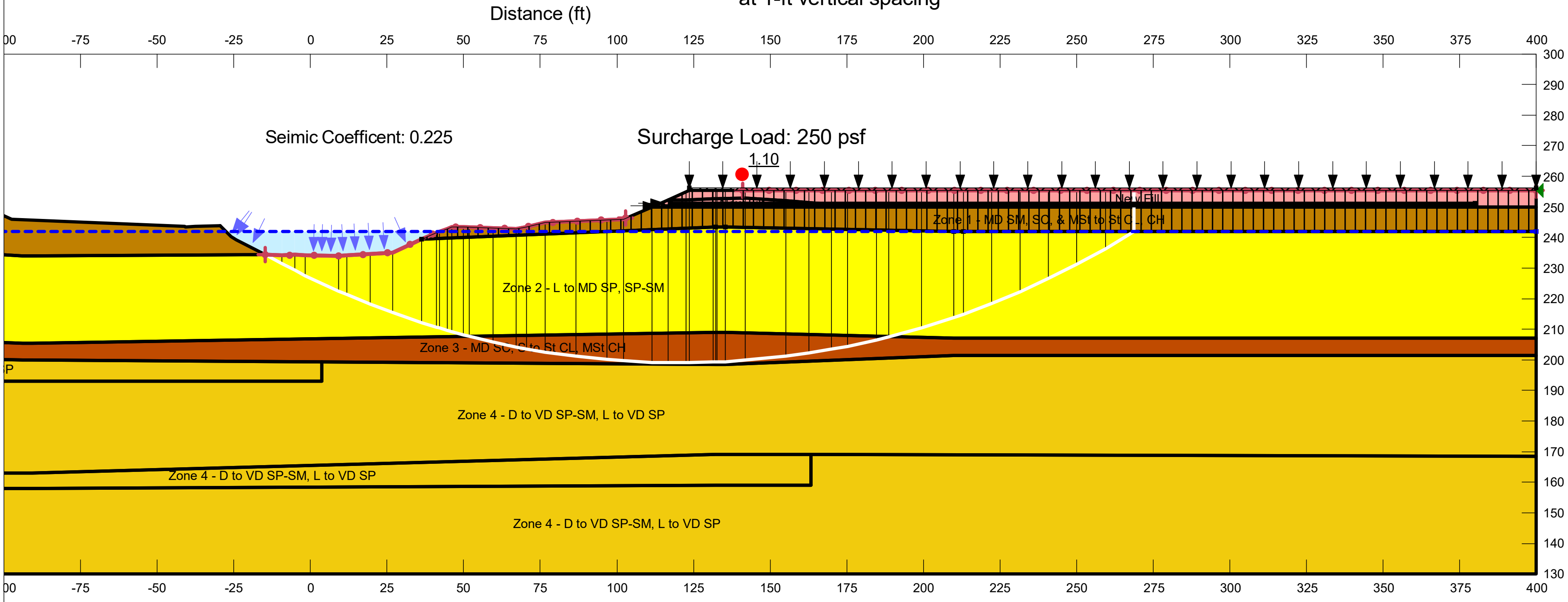
SR 230 Site 5
Craighead County, AR

Figure 23

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

ient

2 layers of geosynthetic reinforcement
with tensile strength = 20,000 lbf
at 1-ft vertical spacing



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH	Mohr-Coulomb	124	1,200	0
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH	Mohr-Coulomb	126	840	0
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34

East Abutment Spill-Through
Pseudostatic

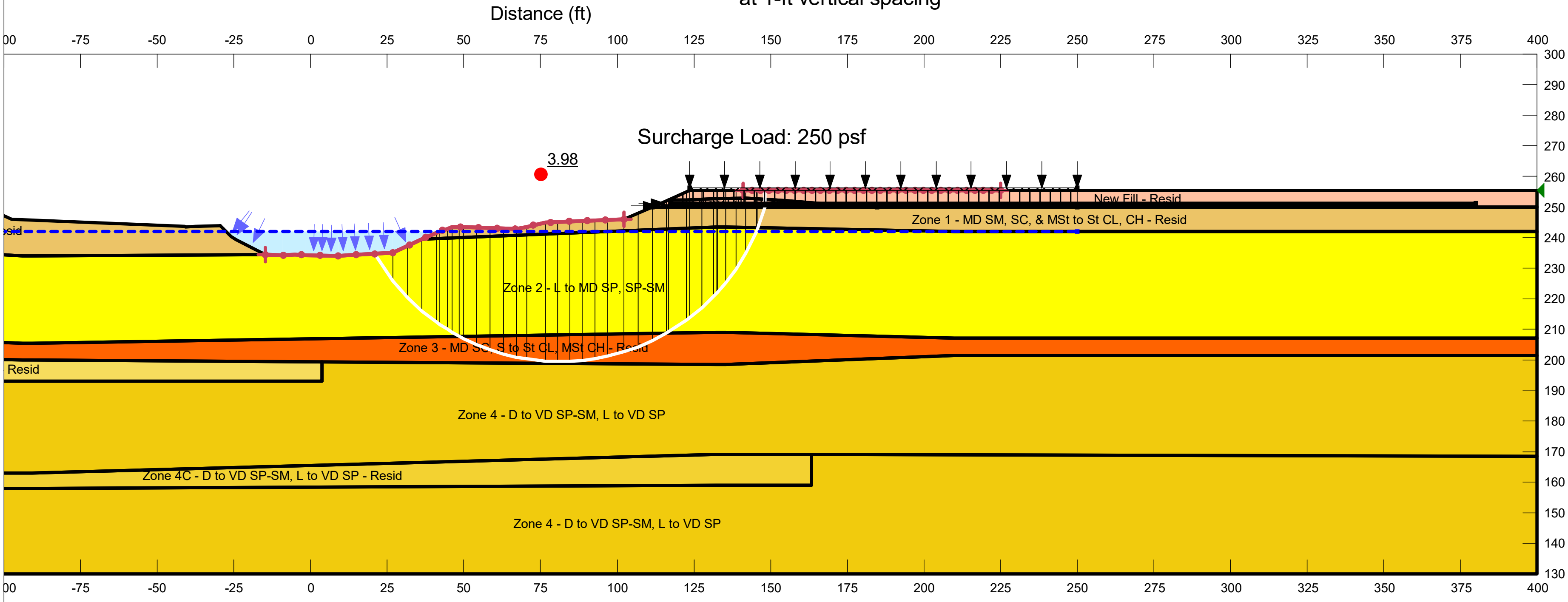
SR 230 Site 5
Craighead County, AR

Figure 24

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

ent

2 layers of geosynthetic reinforcement
with tensile strength = 20,000 lbf
at 1-ft vertical spacing



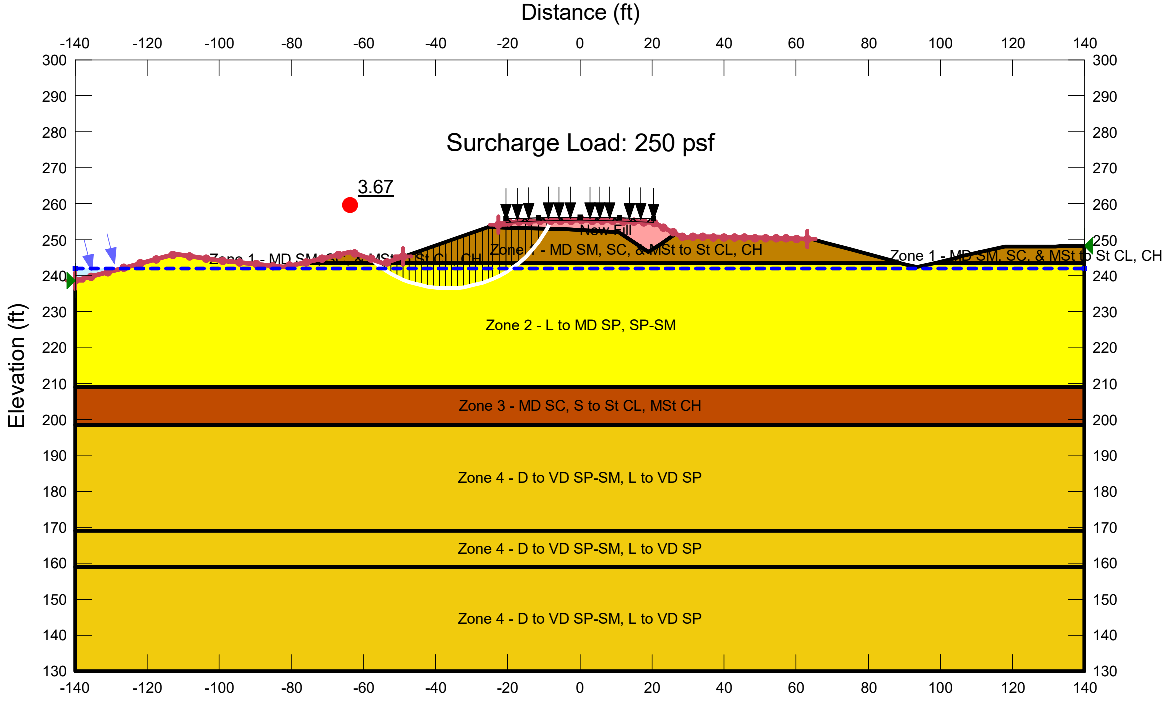
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH - Resid	Mohr-Coulomb	124	960	0
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH - Resid	Mohr-Coulomb	126	672	0
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34
🟨	Zone 4A - D to VD SP-SM, L to VD SP - Resid	Mohr-Coulomb	120	440	0
🟨	Zone 4C - D to VD SP-SM, L to VD SP - Resid	Mohr-Coulomb	120	420	0

East Abutment Spill-Through
Post-Seismic

SR 230 Site 5
Craighead County, AR

Figure 25

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

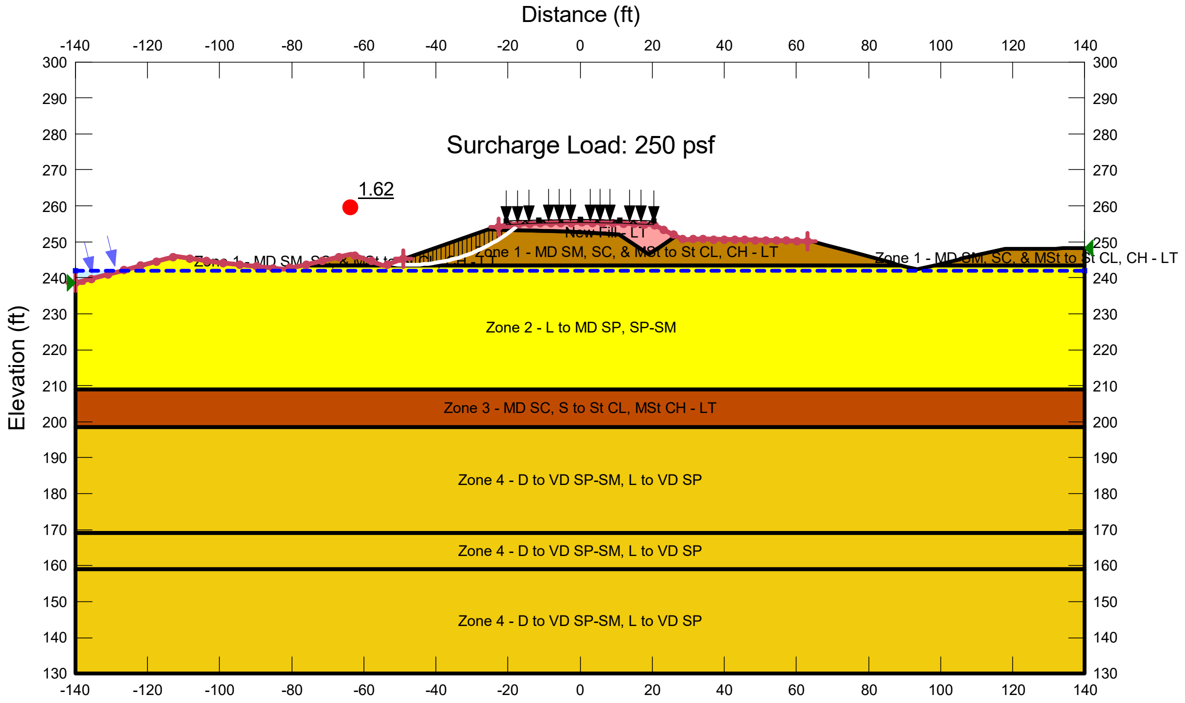


Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH	Mohr-Coulomb	124	1,200	0
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH	Mohr-Coulomb	126	840	0
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34

East Abutment North Side Slope
518+29
End of Construction

SR 230 Site 5
Craighead County, AR

Figure 26



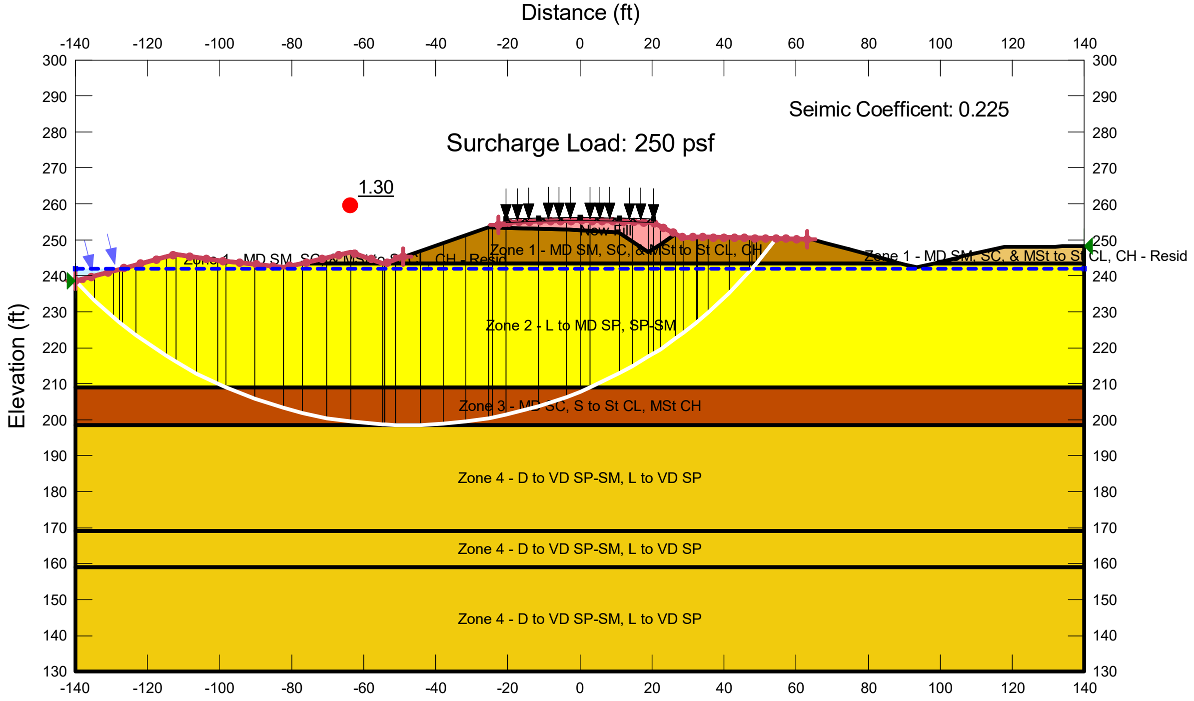
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH - LT	Mohr-Coulomb	124	50	21
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH - LT	Mohr-Coulomb	126	50	21
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34

East Abutment North Side Slope
518+29
Long Term

SR 230 Site 5
Craighead County, AR

Figure 27

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



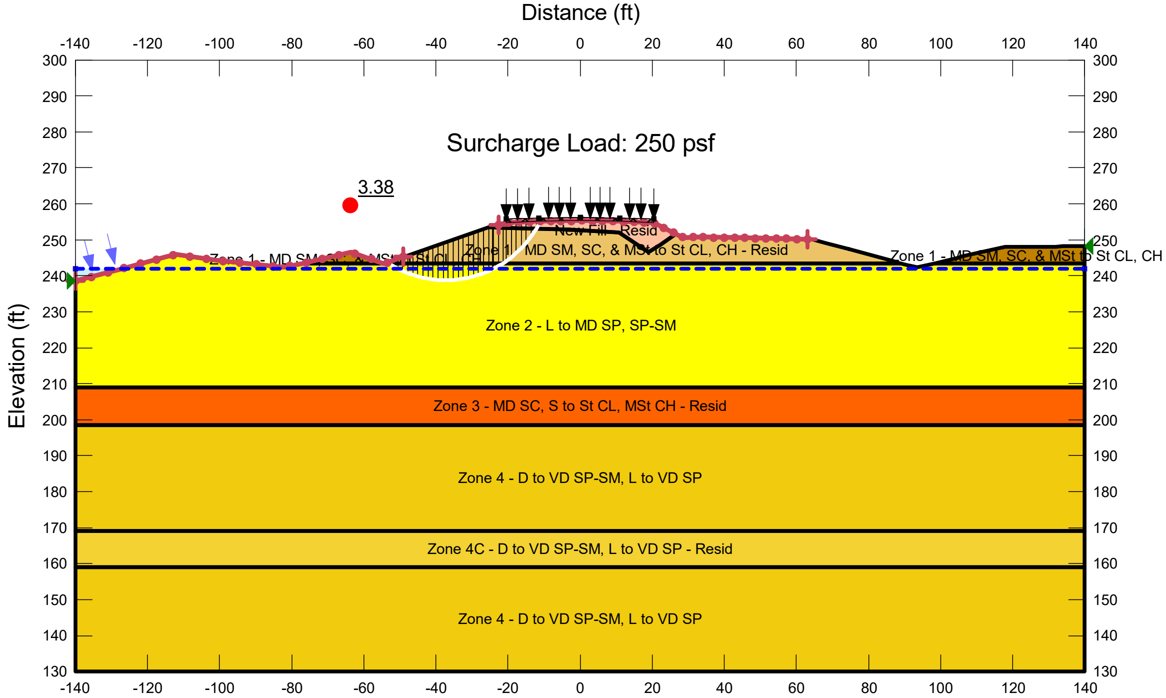
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH	Mohr-Coulomb	124	1,200	0
🟧	Zone 1 - MD SM, SC, & MSt to St CL, CH - Resid	Mohr-Coulomb	124	960	0
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟥	Zone 3 - MD SC, S to St CL, MSt CH	Mohr-Coulomb	126	840	0
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34

East Abutment North Side Slope
518+29
Pseudostatic

SR 230 Site 5
Craighead County, AR

Figure 28

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



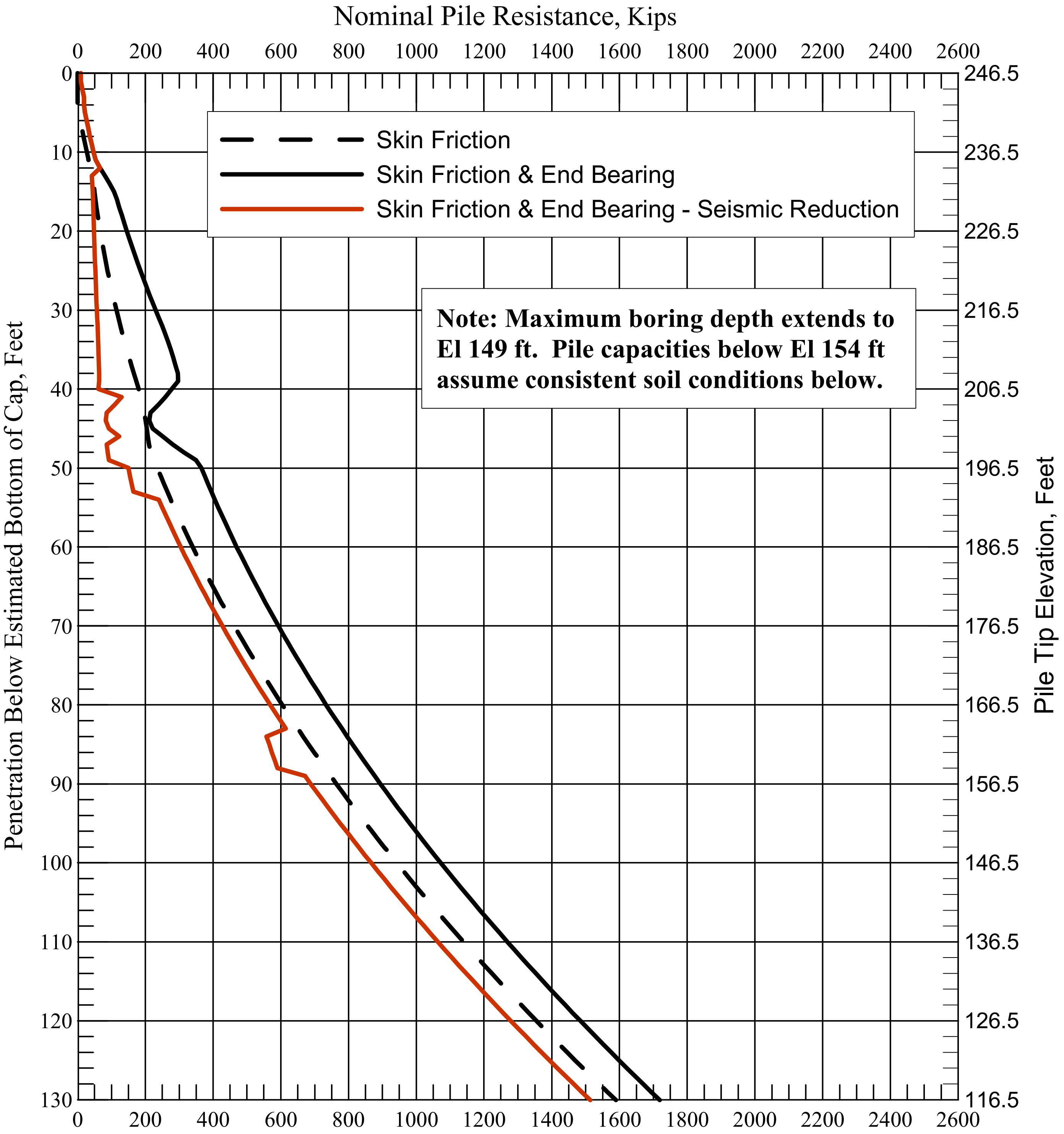
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH	Mohr-Coulomb	124	1,200	0
🟫	Zone 1 - MD SM, SC, & MSt to St CL, CH - Resid	Mohr-Coulomb	124	960	0
🟨	Zone 2 - L to MD SP, SP-SM	Mohr-Coulomb	120	0	34
🟧	Zone 3 - MD SC, S to St CL, MSt CH - Resid	Mohr-Coulomb	126	672	0
🟨	Zone 4 - D to VD SP-SM, L to VD SP	Mohr-Coulomb	120	0	34
🟨	Zone 4C - D to VD SP-SM, L to VD SP - Resid	Mohr-Coulomb	120	420	0

East Abutment North Side Slope
518+29
Post-Seismic

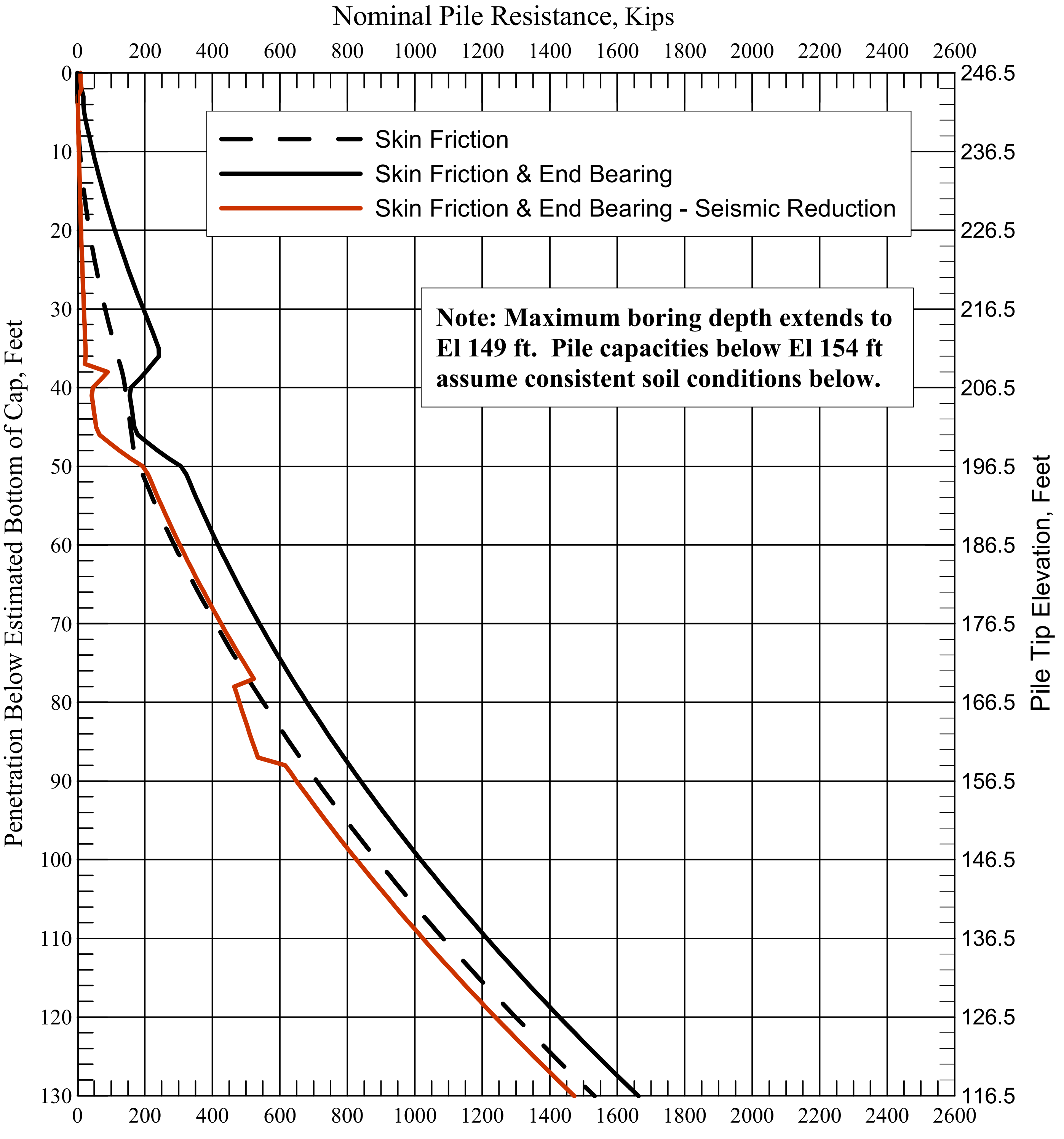
SR 230 Site 5
Craighead County, AR

Figure 29

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



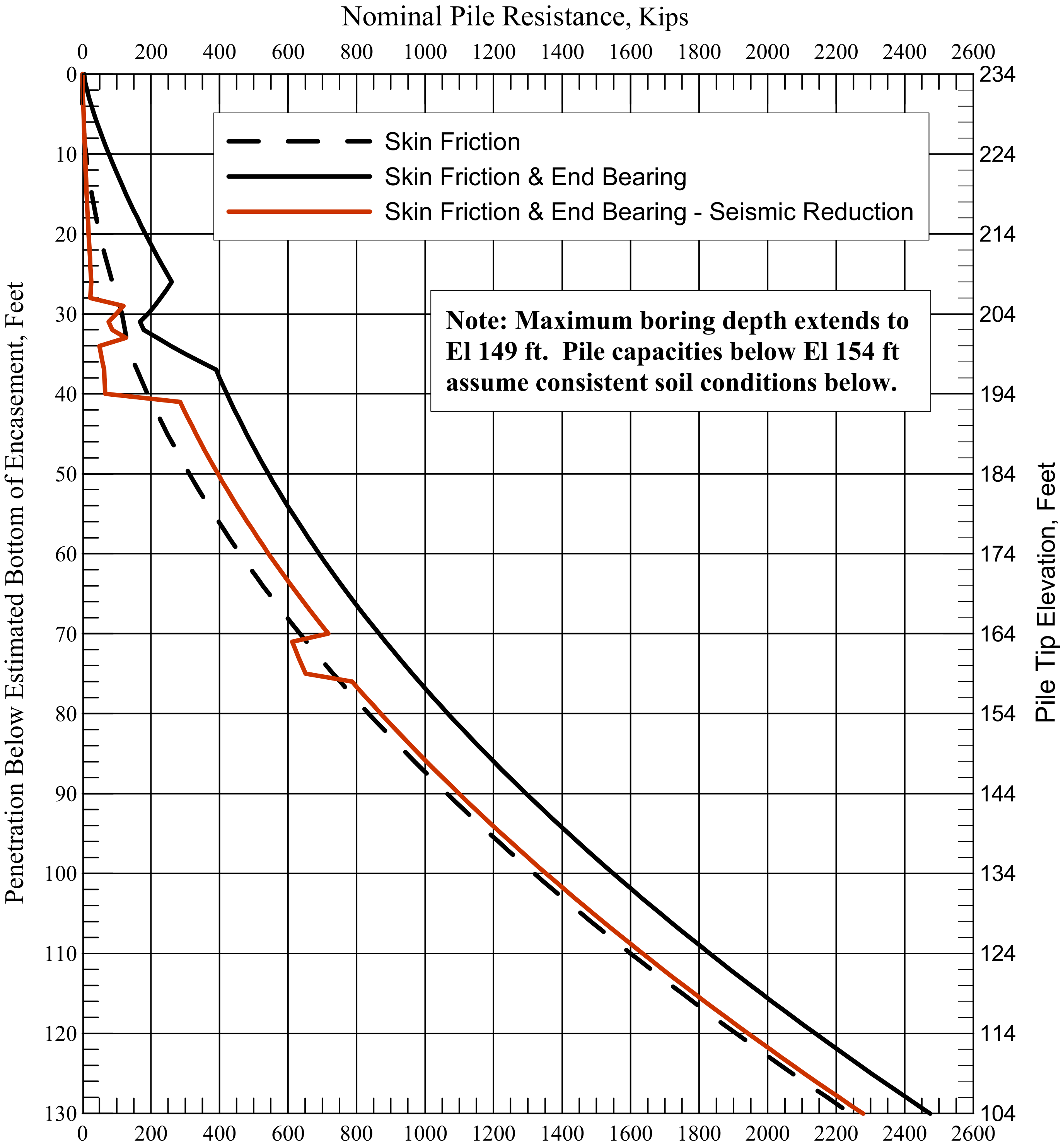
NOMINAL PILE RESISTANCE CURVES
SITE 5
WEST ABUTMENT
18" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS



Note: Maximum boring depth extends to El 149 ft. Pile capacities below El 154 ft assume consistent soil conditions below.

NOMINAL PILE RESISTANCE CURVES
SITE 5
EAST ABUTMENT
18" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

FIGURE 31

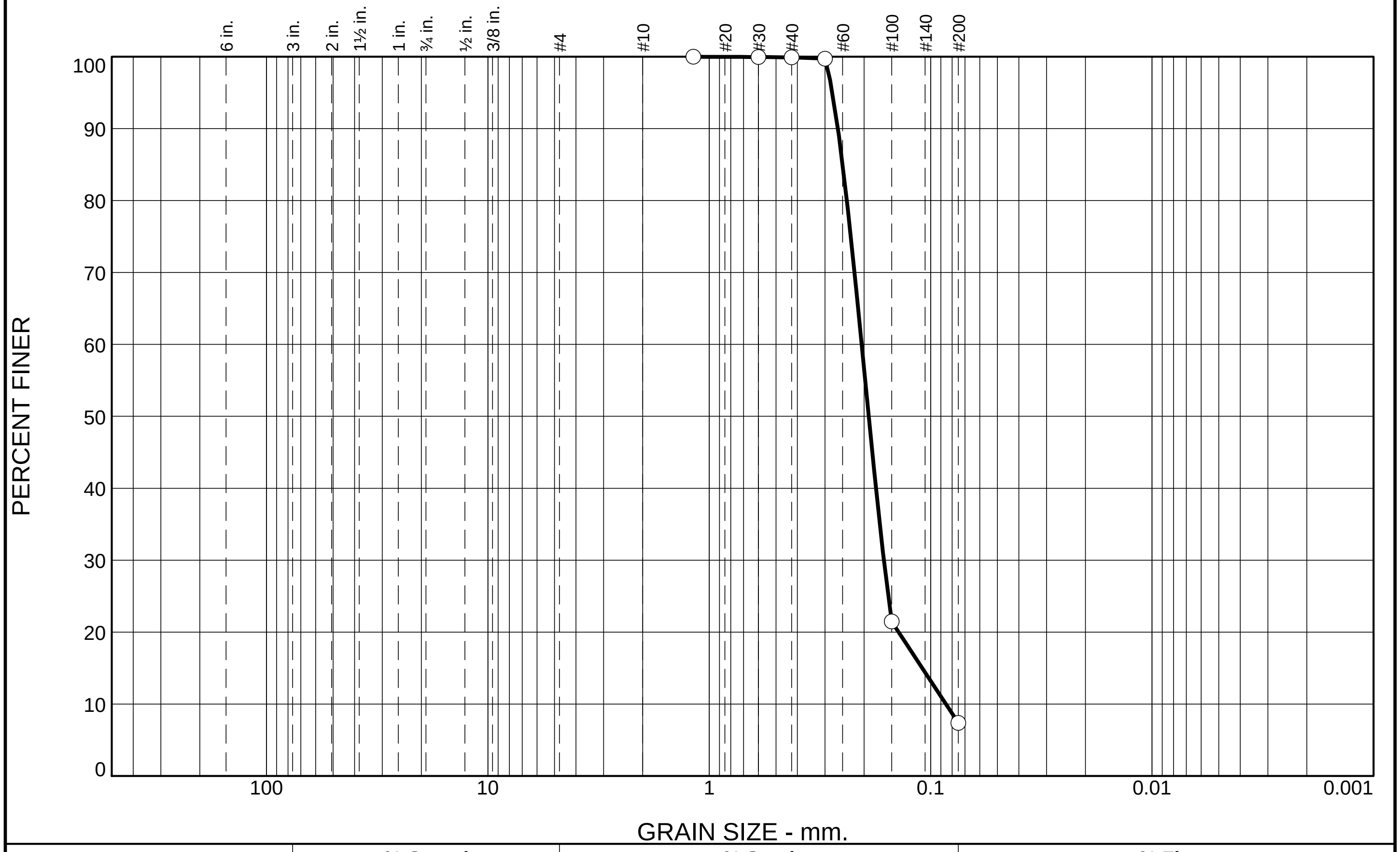


NOMINAL PILE RESISTANCE CURVES
SITE 5
INTERIOR BENTS
24" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

APPENDIX A

Particle Size Distribution Curves

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.1	92.5	7.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#16	100.0		
#30	100.0		
#40	99.9		
#50	99.7		
#100	21.5		
#200	7.4		

* (no specification provided)

Soil Description

Tan and brown fine sand (SP-SM), slightly silty

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2623	D_{85}= 0.2498	D_{60}= 0.2050
D_{50}= 0.1905	D_{30}= 0.1628	D_{15}= 0.1091
D_{10}= 0.0853	C_u= 2.40	C_c= 1.52

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: S5-1 **Depth:** 13.0'
Sample Number: 5

Date: 10-14-20

BURNS COOLEY DENNIS, INC.

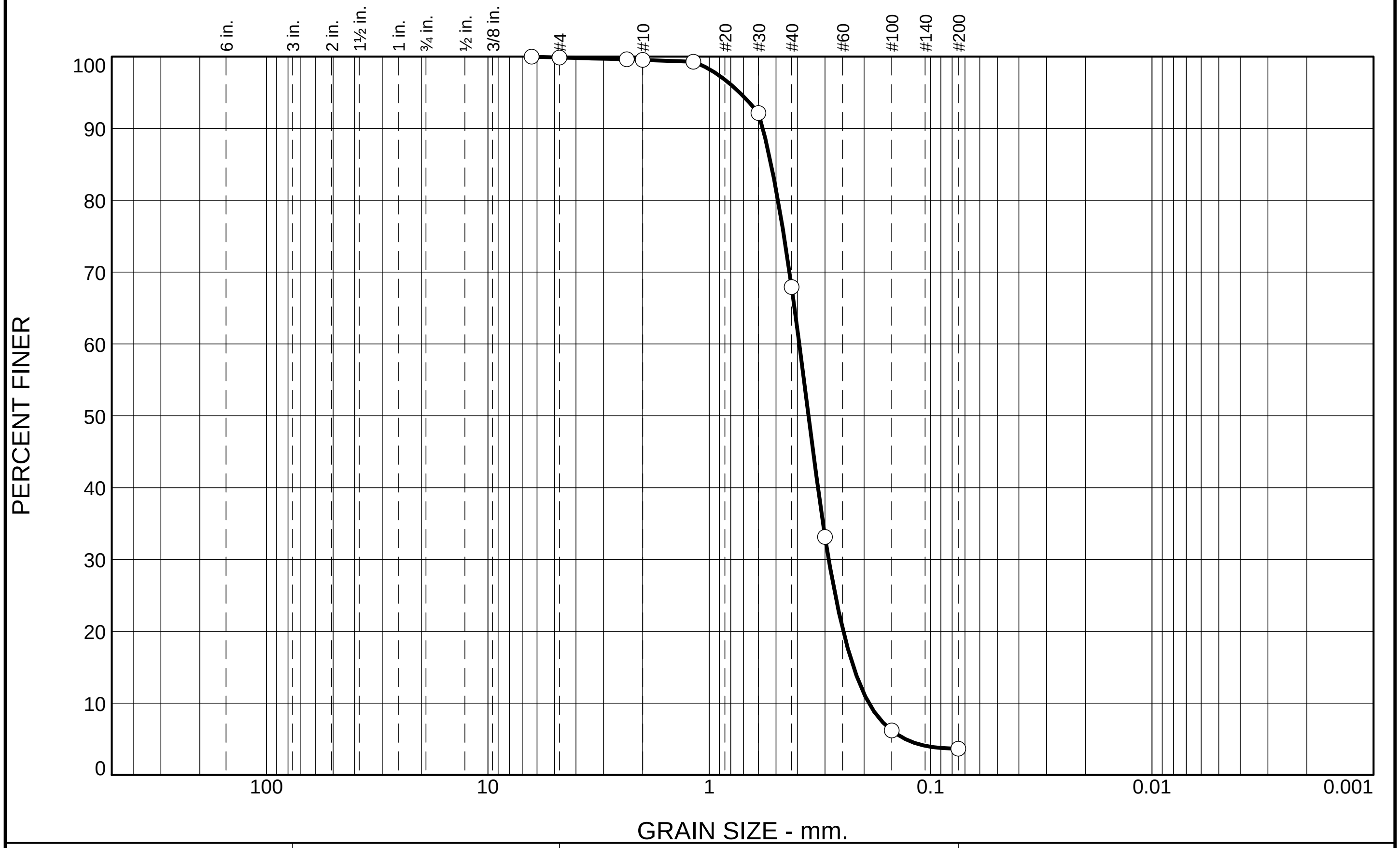
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: bb **Checked By:** ____________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.1	0.3	31.7	64.3	3.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/4	100.0		
#4	99.9		
#8	99.6		
#10	99.6		
#16	99.3		
#30	92.2		
#40	67.9		
#50	33.1		
#100	6.2		
#200	3.6		

* (no specification provided)

Soil Description

Tan and brown fine to medium sand (SP)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5734 D_{85}= 0.5260 D_{60}= 0.3925
D_{50}= 0.3566 D_{30}= 0.2888 D_{15}= 0.2232
D_{10}= 0.1903 C_u= 2.06 C_c= 1.12

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S5-1 **Depth:** 33.5'
Sample Number: 9 **Date:** 10-14-20

BURNS COOLEY DENNIS, INC.

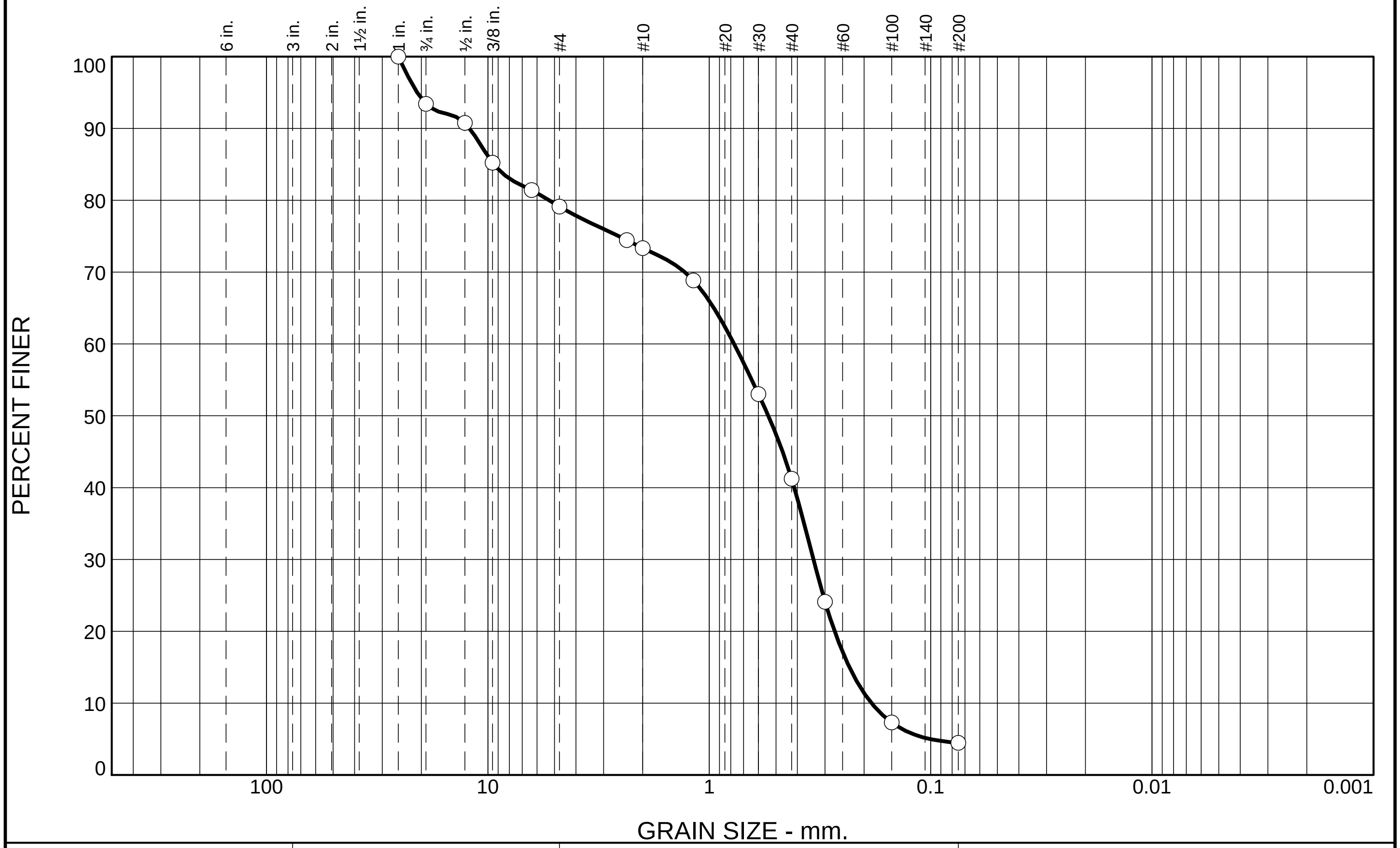
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: bb **Checked By:** ____

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	6.6	14.3	5.8	32.1	36.7	4.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	93.4		
1/2	90.8		
3/8	85.2		
1/4	81.4		
#4	79.1		
#8	74.5		
#10	73.3		
#16	68.9		
#30	53.0		
#40	41.2		
#50	24.1		
#100	7.3		
#200	4.5		

* (no specification provided)

Soil Description

Gray and tan fine to coarse sand (SP) with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 12.0711 D_{85}= 9.3937 D_{60}= 0.7725
D_{50}= 0.5416 D_{30}= 0.3388 D_{15}= 0.2330
D_{10}= 0.1853 C_u= 4.17 C_c= 0.80

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S5-1 **Depth:** 58.5'
Sample Number: 15

Date: 10-14-20

BURNS COOLEY DENNIS, INC.

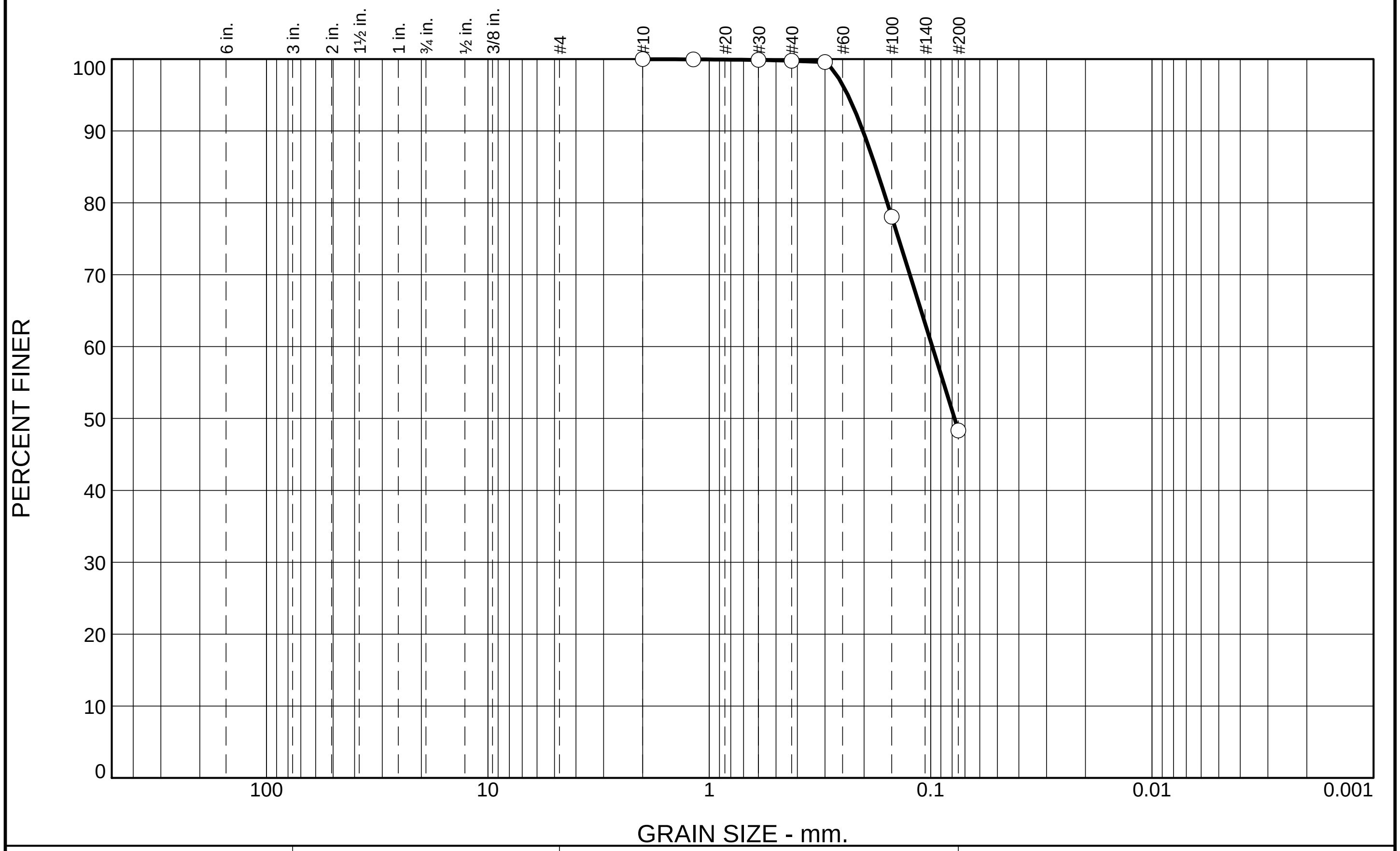
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: bb **Checked By:** ____________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.2	51.5	48.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	99.9		
#40	99.8		
#50	99.6		
#100	78.1		
#200	48.3		

* (no specification provided)

Soil Description

Tan and light gray silty fine sand (SM) with clay partings

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2023 D_{85}= 0.1773 D_{60}= 0.0983
D_{50}= 0.0779 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: S5-2 **Depth:** 13.0'
Sample Number: 6A

Date: 10-19-20

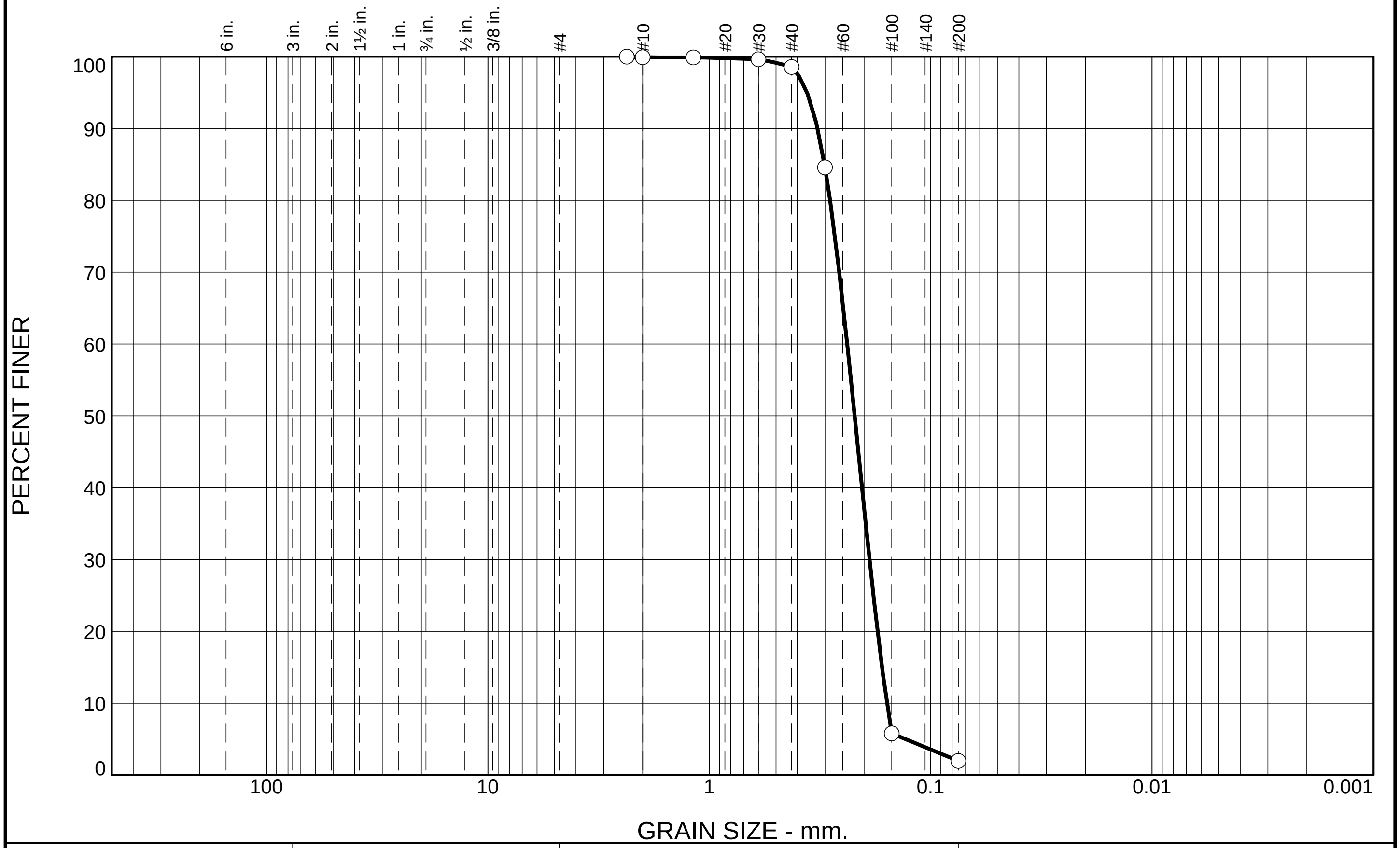
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: cr **Checked By:** ______

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	1.3	96.6	2.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#8	100.0		
#10	99.9		
#16	99.9		
#30	99.6		
#40	98.6		
#50	84.6		
#100	5.8		
#200	2.0		

* (no specification provided)

Soil Description

Tan and brown fine sand (SP)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3245 D_{85}= 0.3016 D_{60}= 0.2383
D_{50}= 0.2205 D_{30}= 0.1890 D_{15}= 0.1660
D_{10}= 0.1577 C_u= 1.51 C_c= 0.95

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S5-2 **Depth:** 23.5'
Sample Number: 8 **Date:** 10-19-20

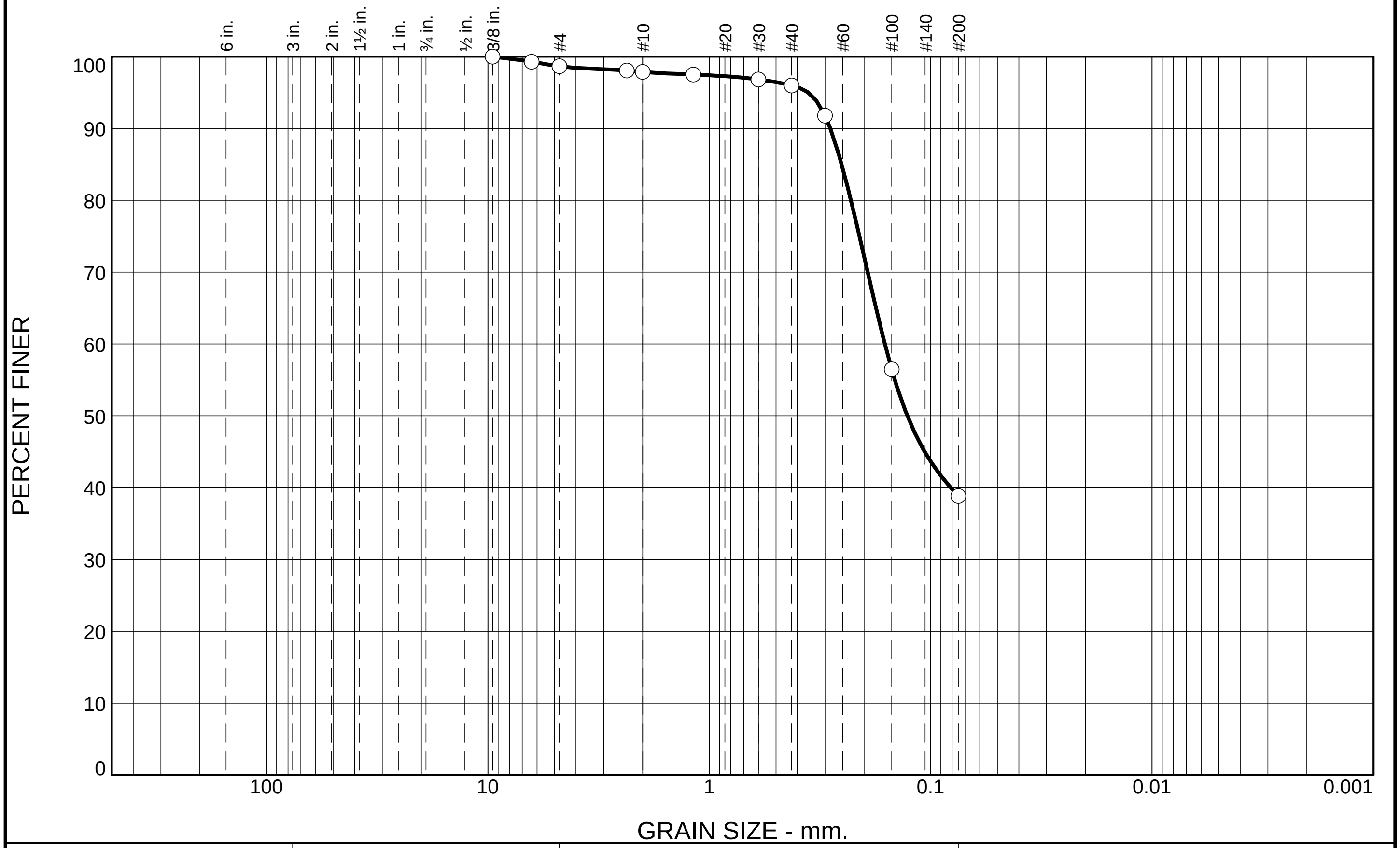
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: cr **Checked By:**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.3	0.8	1.9	57.2	38.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100.0		
1/4	99.3		
#4	98.7		
#8	98.1		
#10	97.9		
#16	97.5		
#30	96.8		
#40	96.0		
#50	91.8		
#100	56.5		
#200	38.8		

* (no specification provided)

Soil Description

Brown and tan silty fine sand (SM) with clay partings

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2839 D_{85}= 0.2524 D_{60}= 0.1612
D_{50}= 0.1274 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: S5-3 **Depth:** 4.0'
Sample Number: 3A

Date: 10-19-20

BURNS COOLEY DENNIS, INC.

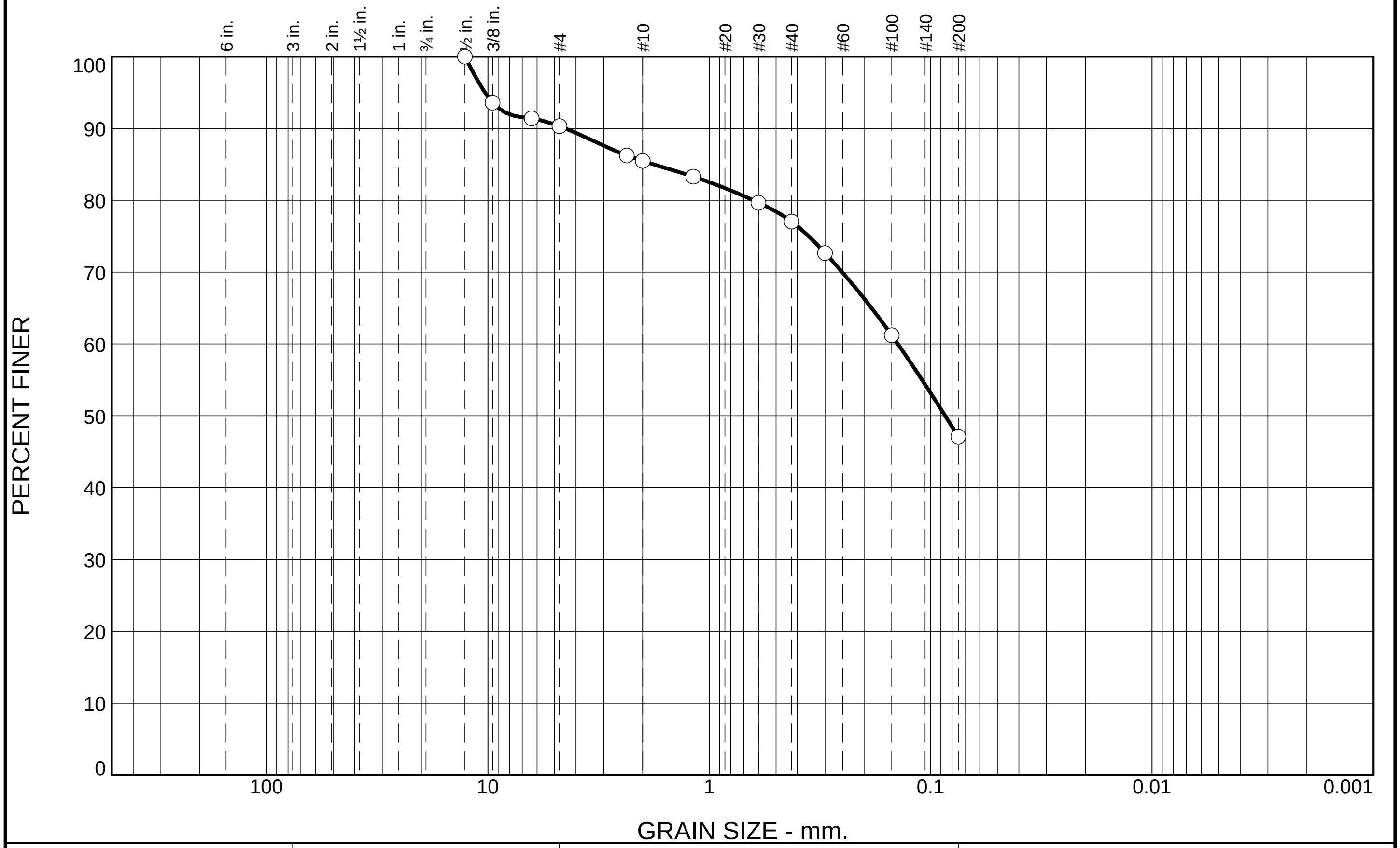
Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: cr **Checked By:** ____________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	9.7	4.8	8.5	29.9	47.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2	100.0		
3/8	93.6		
1/4	91.4		
#4	90.3		
#8	86.3		
#10	85.5		
#16	83.3		
#30	79.7		
#40	77.0		
#50	72.7		
#100	61.2		
#200	47.1		

* (no specification provided)

Soil Description

Gray clayey fine to coarse sand (SC) with trace of gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 4.4652 D_{85}= 1.7820 D_{60}= 0.1406
D_{50}= 0.0860 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Source of Sample: S5-3 **Depth:** 4.0'
Sample Number: 3B
Date: 10-19-20

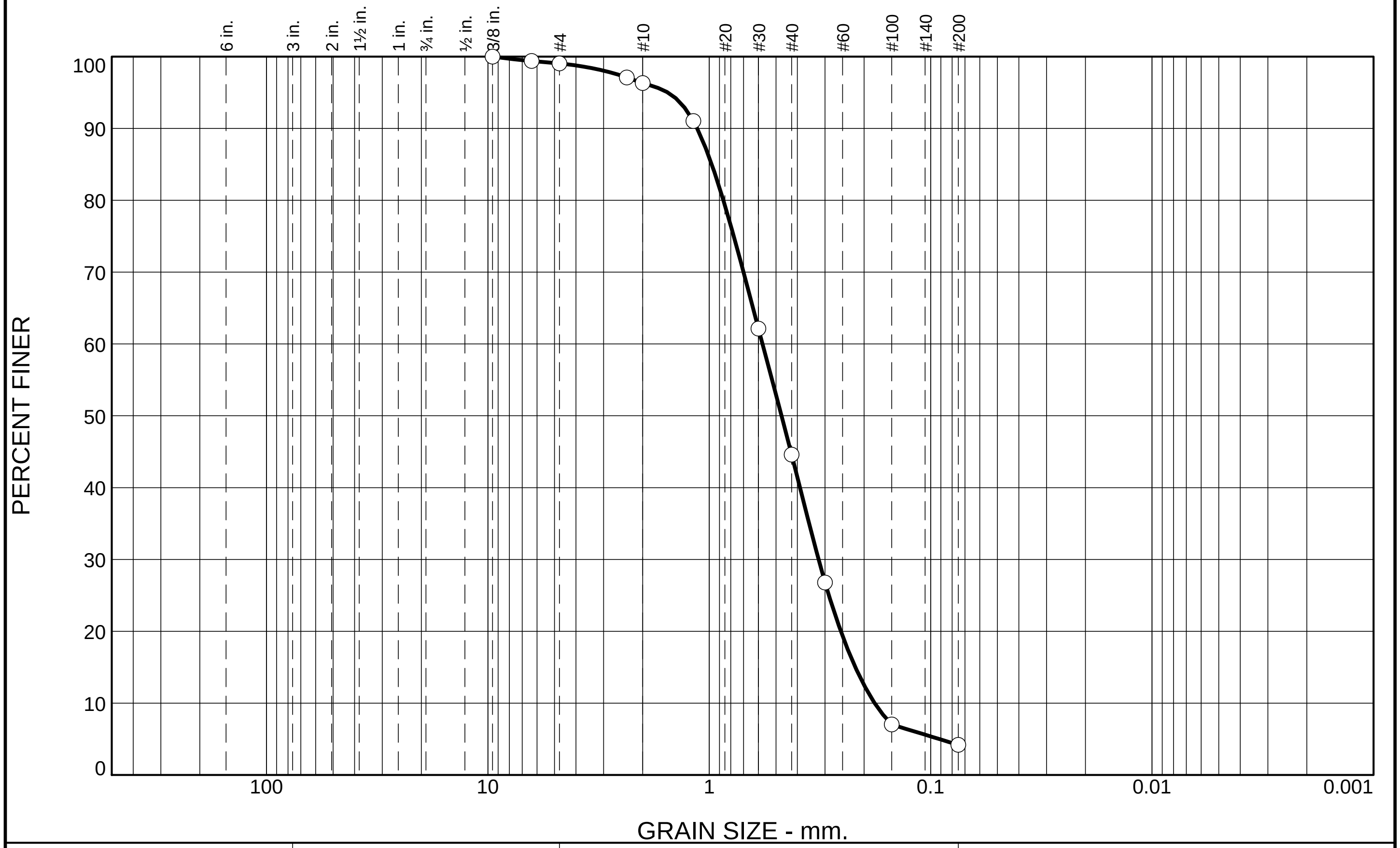
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: cr **Checked By:** ____________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.9	2.8	51.7	40.4	4.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100.0		
1/4	99.4		
#4	99.1		
#8	97.1		
#10	96.3		
#16	91.0		
#30	62.1		
#40	44.6		
#50	26.8		
#100	7.0		
#200	4.2		

* (no specification provided)

Soil Description

Gray and light gray fine to coarse sand (SP) with clay partings and trace of gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.1334 D_{85}= 0.9744 D_{60}= 0.5751
D_{50}= 0.4719 D_{30}= 0.3211 D_{15}= 0.2191
D_{10}= 0.1793 C_u= 3.21 C_c= 1.00

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S5-3 **Depth:** 53.5'
Sample Number: 14

Date: 10-19-20

BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR

Project No: 200518 **Figure**

Tested By: cr **Checked By:** ____________

APPENDIX B

Liquefaction Triggering Workbook

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]

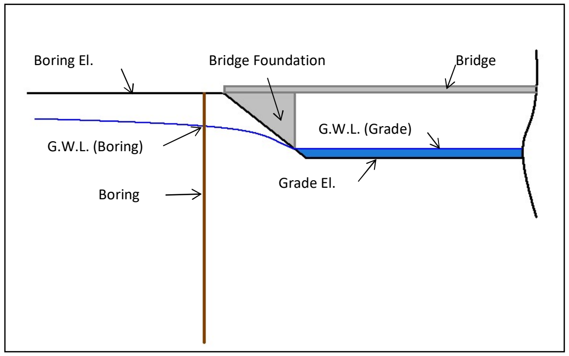
Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.91064	-90.89193
Logged By :	Christian Jackson	
Boring No:	S5-1	
Date:	11-Sep-20	
Type of Drilling:	HSA to 55', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by:		Date:	
Checked by:		Date:	

All cells highlighted in this color should be input

All other cells should not be altered!



Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.375	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	251	ft
Ground Water Level (depth below boring surface) =	10	ft
Grade Surface Elevation =	234	ft
Ground Water Level (depth below or above grade surface) =	-8	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.

		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	248.5	2.5	SM	1	34.2					
2	246	5	CL	1						
3	241	10	CL	15						
4	236	15	SP-SM	1	7.37					
5	231	20	SP-SM	16	11.7					
6	226	25	SP	17	3.4					
7	221	30	SP	26	3.9					
8	216	35	SP	22	3.65					
9	211	40	SP-SM	19	10					
10	206	45	CL	5	52.6					
11	201	50	CL	1						
12	196	55	SP	12	1.9					
13	191	60	SP	35	4.46					
14	186	65	SP	25	3.1					
15	181	70	SP	18	1.4					
16	176	75	SP	35	2.7					
17	171	80	SP	53	4.8					
18	166	85	SP	25	2.9					
19	161	90	SP	9	1.4					
20	156	95	SP	25	1.5					
21	151	100	SP	31	2.5					

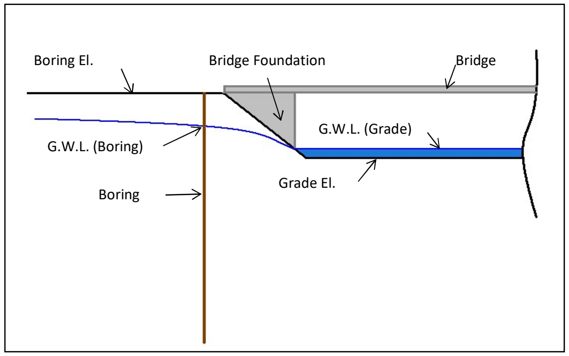
SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]
Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.91065	-90.89169
Logged By :	Christian Jackson	
Boring No:	S5-2	
Date:	14-Sep-20	
Type of Drilling:	HSA to 60', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by:		Date:	
Checked by:		Date:	

All cells highlighted in this color should be input

All other cells should not be altered!



Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.375	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	249	ft
Ground Water Level (depth below boring surface) =	10	ft
Grade Surface Elevation =	234	ft
Ground Water Level (depth below or above grade surface) =	-8	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.

		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	246.5	2.5	SP-SM	6						
2	244	5	SC	1						
3	241.5	7.5	CL	6						
4	239	10	CH	1						
5	234	15	SC	1	48.3					
6	229	20	SP-SM	15	5.8					
7	224	25	SP	9	2					
8	219	30	SP	26	3.5					
9	214	35	SP-SM	29	5.5					
10	209	40	SP-SM	13	9					
11	204	45	CH	1						
12	199	50	SC	1	36.9					
13	194	55	SP	11	1.1					
14	189	60	SP	28	2.6					
15	184	65	SP	18	2.1					
16	179	70	SP	29	1.4					
17	174	75	SP	20	1.4					
18	169	80	SP-SM	80	7.8					
19	164	85	SP	39	3.9					
20	159	90	SP	16	3.1					
21	154	95	SP	37	4.5					
22	149	100	SP-SM	47	5.3					

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]
Input Data Worksheet

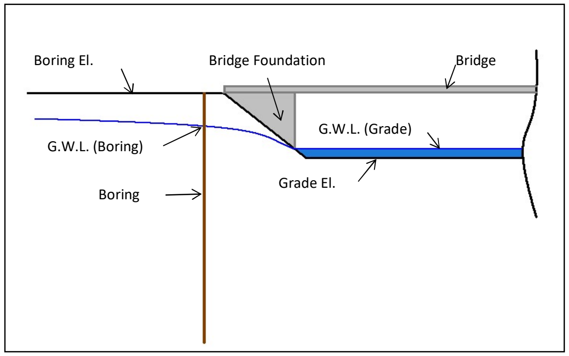
Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.91056	-90.89094
Logged By :	Christian Jackson	
Boring No:	S5-3	
Date:	14-Sep-20	
Type of Drilling:	HSA to 60', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: ________ Date: ________
Checked by: ________ Date: ________

All cells highlighted in this color should be input
All other cells should not be altered!

Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.375	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	252	ft
Ground Water Level (depth below boring surface) =	10	ft
Grade Surface Elevation =	234	ft
Ground Water Level (depth below or above grade surface) =	-8	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.



		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	249.5	2.5	SC	16						
2	247	5	SC	1	38.8					
3	242	10	SP-SM	15	7					
4	237	15	SP-SM	13	6.9					
5	232	20	SP	19	3					
6	227	25	SP	18	3.3					
7	222	30	SP-SM	17	9.2					
8	217	35	SP	36	3.5					
9	212	40	SP-SM	15	8.3					
10	207	45	CL	4	49.9					
11	202	50	CL	1						
12	197	55	SP	33	4.2					
13	192	60	SP	19	2					
14	187	65	SP	28	3.1					
15	182	70	SP	31	3.5					
16	177	75	SP	26	2.2					
17	172	80	SP	40	3.2					
18	167	85	SP	9	2.4					
19	162	90	SP	12	2.9					
20	157	95	SP	45	4.5					
21	152	100	SP-SM	76	8.9					

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]

Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Lawrence County	
Latitude and Longitude (decimal degrees)	35.91069	-90.89066
Logged By :	Christian Jackson	
Boring No:	S5-4	
Date:	15-Sep-20	
Type of Drilling:	HSA to 55', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: 　　　　Date:

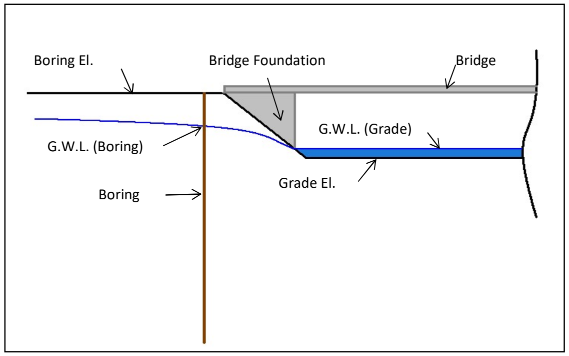
Checked by: 　　　　Date:

All cells highlighted in this color should be input

All other cells should not be altered!

Design Peak Horizontal Ground Acceleration (a_{max}, or A_s) =	0.375	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	250	ft
Ground Water Level (depth below boring surface) =	10	ft
Grade Surface Elevation =	234	ft
Ground Water Level (depth below or above grade surface) =	-8	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.



		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	247.5	2.5	CL	4	82.5					
2	245	5	SC	1	38					
3	240	10	SP-SM	1	7.1					
4	235	15	SP-SM	9	6.4					
5	230	20	SP-SM	13	6.6					
6	225	25	SP	12	2					
7	220	30	SP	15	3					
8	215	35	SP	17	2.6					
9	210	40	SP	22	3.4					
10	205	45	CL	6	57.6					
11	200	50	SP	1	2.5					
12	195	55	SP	16	0.6					
13	190	60	SP	23	2					
14	185	65	SP	30	3.5					
15	180	70	SP	31	2.5					
16	175	75	SP	23	2.8					
17	170	80	SP	22	2.7					
18	165	85	SP	52	3					
19	160	90	SP	28	3.5					
20	155	95	SP-SM	100	5.6					
21	150	100	SP-SM	87	5.3					

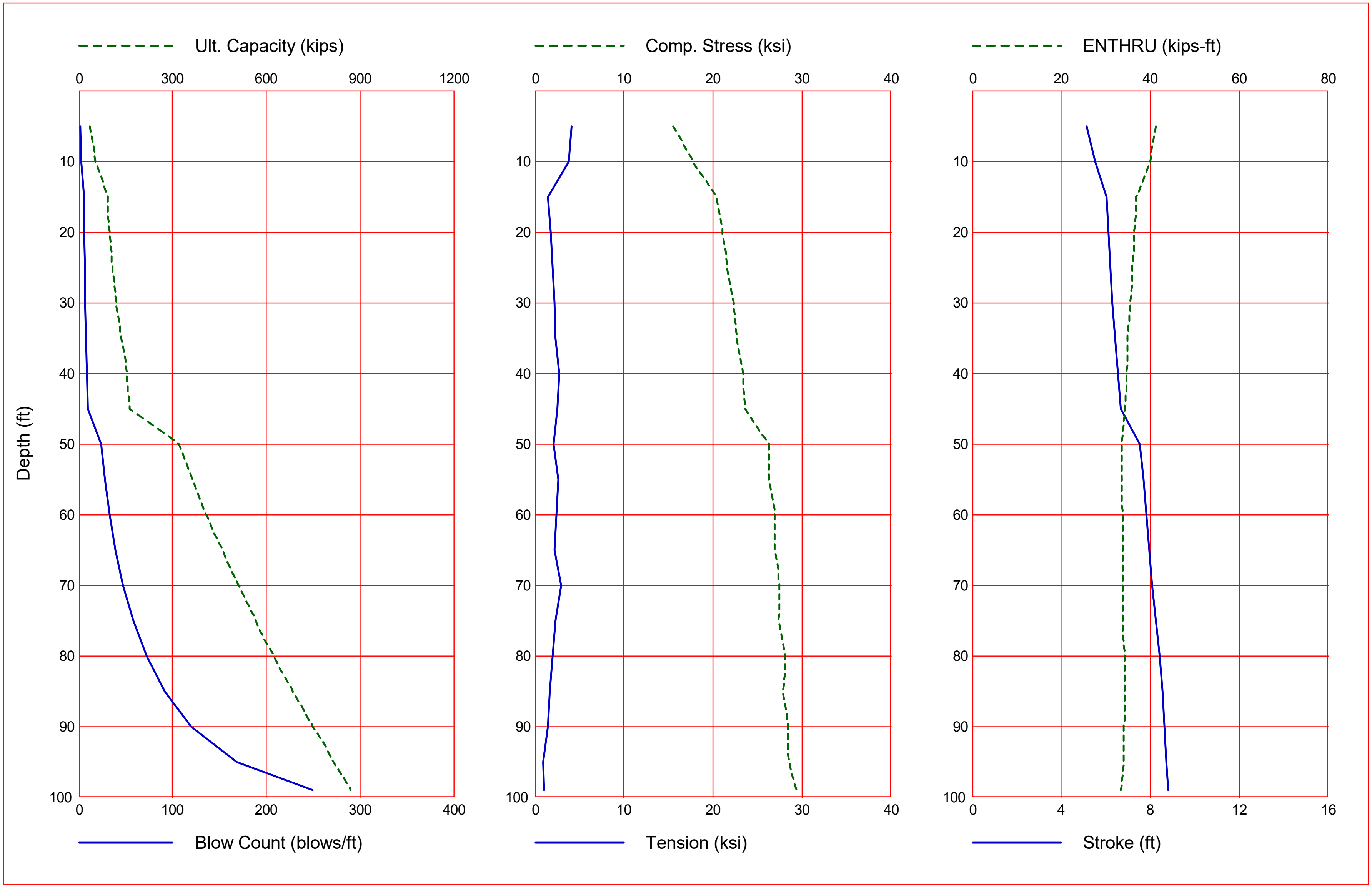
APPENDIX C

Pile Drivability Analysis Results

Burns Cooley Dennis, Inc.
Site 5 - West Abutment - DELMAG D30

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

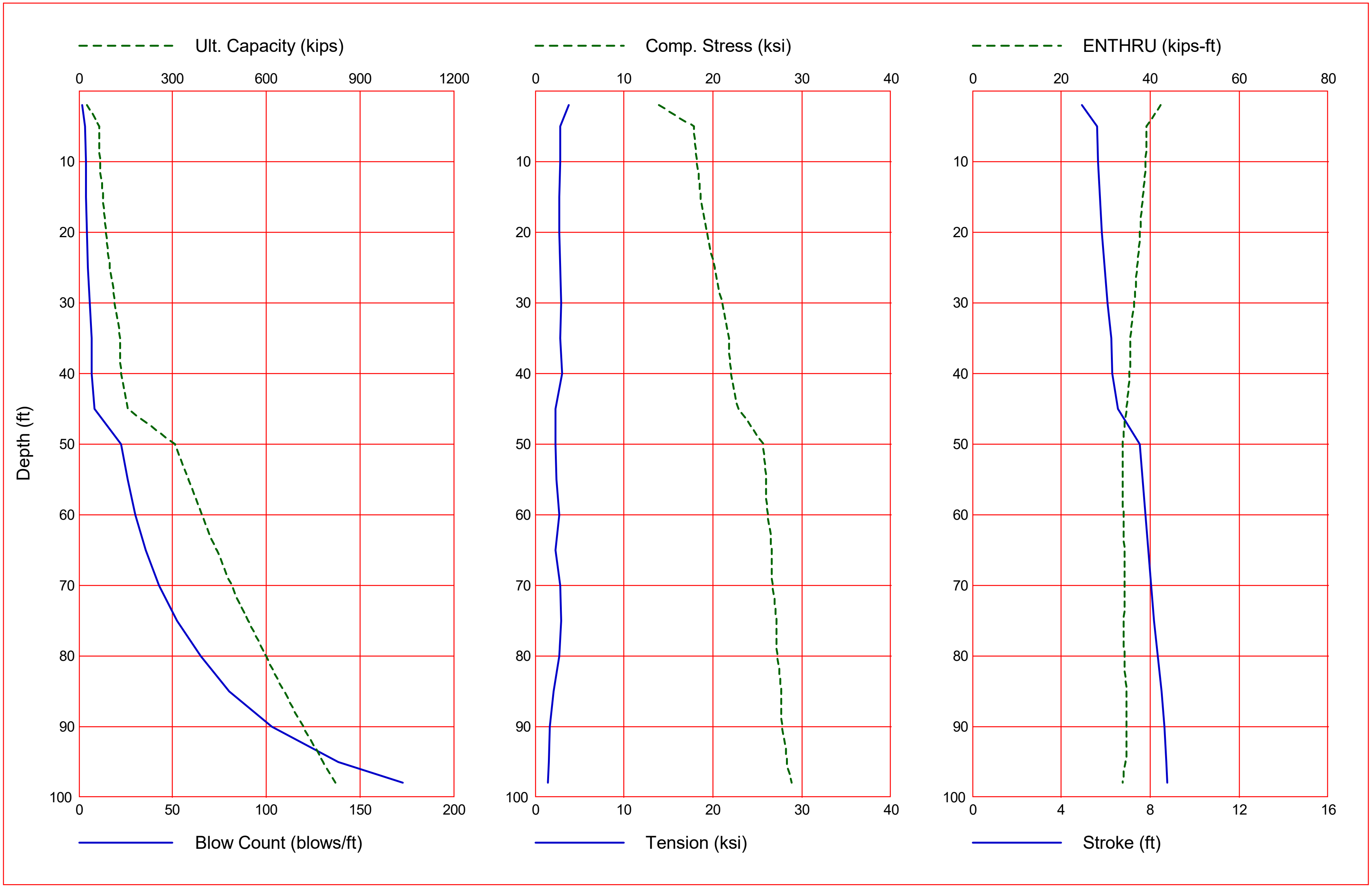
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	35.5	16.4	19.1	1.9	15.516	-4.130	5.16	41.3
10.0	52.1	33.0	19.1	2.7	17.796	-3.811	5.52	40.1
15.0	92.8	41.5	51.3	5.5	20.415	-1.491	6.05	36.7
20.0	98.5	47.3	51.3	5.8	21.096	-1.720	6.11	36.4
25.0	107.5	56.2	51.3	6.3	21.577	-2.015	6.20	36.0
30.0	119.7	68.4	51.3	6.9	22.322	-2.222	6.30	35.5
35.0	135.1	83.8	51.3	7.6	22.686	-2.282	6.42	34.9
40.0	153.7	102.4	51.3	8.6	23.407	-2.667	6.56	34.6
45.0	162.0	123.9	38.2	9.4	23.609	-2.547	6.67	34.3
50.0	319.0	161.7	157.3	24.0	26.272	-2.093	7.55	33.6
55.0	362.8	205.6	157.3	27.8	26.318	-2.613	7.68	33.6
60.0	409.5	252.3	157.3	32.7	26.917	-2.355	7.82	33.8
65.0	459.1	301.8	157.3	38.9	26.890	-2.204	7.95	33.8
70.0	511.5	354.2	157.3	46.9	27.463	-2.924	8.08	33.8
75.0	566.7	409.5	157.3	57.9	27.401	-2.261	8.24	33.9
80.0	624.8	467.5	157.3	71.9	28.074	-2.006	8.42	34.3
85.0	685.7	528.5	157.3	91.5	27.903	-1.652	8.54	34.2
90.0	749.5	592.2	157.3	119.7	28.439	-1.444	8.63	34.1
95.0	816.1	658.8	157.3	168.5	28.580	-0.930	8.72	34.0
99.0	871.5	714.2	157.3	249.3	29.348	-1.057	8.80	33.3

Total Continuous Driving Time 99.00 minutes; Total Number of Blows 4079 (starting at penetration 5.0 ft)

Burns Cooley Dennis, Inc.
Site 5 - East Abutment - DELMAG D30

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

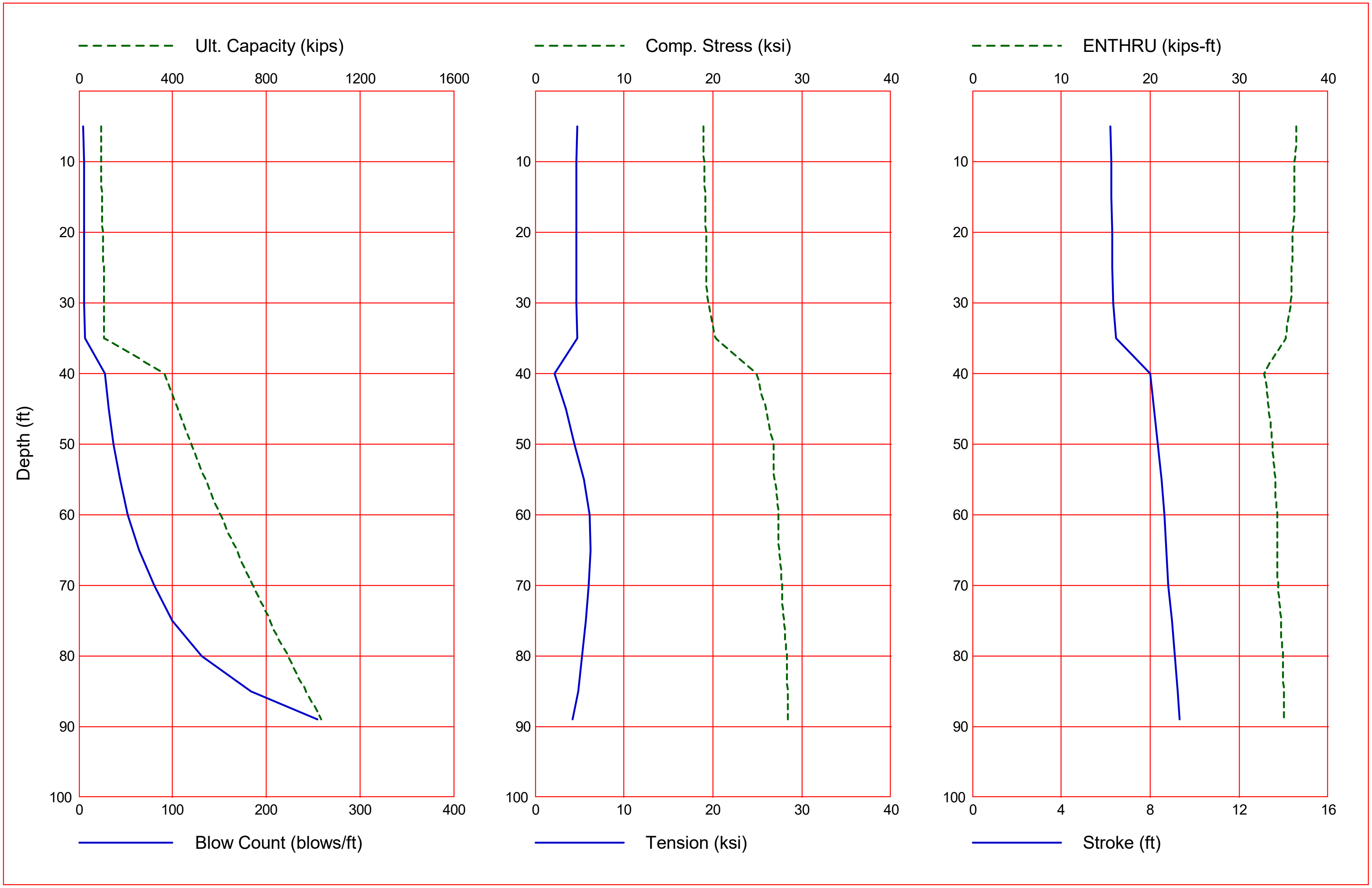
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
2.0	25.7	6.6	19.1	1.6	13.915	-3.835	4.93	42.3
5.0	65.0	13.7	51.3	3.4	17.871	-2.848	5.59	39.2
10.0	69.1	17.8	51.3	3.6	18.138	-2.842	5.64	38.9
15.0	76.0	24.7	51.3	3.9	18.651	-2.756	5.72	38.3
20.0	85.6	34.3	51.3	4.3	19.344	-2.737	5.83	37.7
25.0	97.9	46.6	51.3	4.9	20.227	-2.860	5.96	37.1
30.0	113.0	61.8	51.3	5.7	21.075	-2.913	6.10	36.4
35.0	130.9	79.6	51.3	6.6	21.788	-2.809	6.24	35.6
40.0	135.0	99.7	35.3	6.7	22.028	-3.064	6.29	35.3
45.0	156.4	121.1	35.3	8.5	22.904	-2.243	6.55	34.6
50.0	307.9	150.6	157.3	22.6	25.612	-2.327	7.52	33.9
55.0	349.4	192.1	157.3	25.9	25.996	-2.447	7.64	33.9
60.0	393.7	236.4	157.3	30.3	26.207	-2.691	7.77	34.0
65.0	440.8	283.5	157.3	35.7	26.600	-2.296	7.91	34.2
70.0	490.7	333.4	157.3	42.7	26.768	-2.776	8.05	34.2
75.0	543.4	386.1	157.3	52.6	27.105	-2.898	8.18	34.1
80.0	599.0	441.7	157.3	65.0	27.236	-2.757	8.34	34.3
85.0	657.3	500.1	157.3	80.4	27.688	-2.092	8.51	34.6
90.0	718.5	561.2	157.3	102.8	27.821	-1.637	8.62	34.6
95.0	782.5	625.2	157.3	138.3	28.349	-1.521	8.72	34.5
98.0	822.2	665.0	157.3	172.9	28.811	-1.409	8.78	33.9

Total Continuous Driving Time 81.00 minutes; Total Number of Blows 3338 (starting at penetration 2.0 ft)

Burns Cooley Dennis, Inc.
Site 5 - Interior Bents - DELMAG D30

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Burns Cooley Dennis, Inc.
Site 5 - Interior Bents - DELMAG D30

Feb 19 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	94.1	2.9	91.2	5.0	18.918	-4.762	6.22	36.4
10.0	97.0	5.8	91.2	5.2	19.000	-4.690	6.24	36.2
15.0	99.9	8.7	91.2	5.3	19.099	-4.654	6.27	36.2
20.0	102.8	11.6	91.2	5.5	19.204	-4.634	6.29	36.0
25.0	105.7	14.5	91.2	5.6	19.291	-4.608	6.31	35.9
30.0	108.6	17.4	91.2	5.8	19.433	-4.641	6.34	35.8
35.0	108.5	45.7	62.8	6.1	20.342	-4.791	6.47	35.2
40.0	365.9	86.3	279.6	27.5	24.893	-2.165	7.98	32.8
45.0	422.0	142.4	279.6	32.0	25.992	-3.440	8.17	33.3
50.0	480.9	201.3	279.6	37.3	26.809	-4.406	8.34	33.8
55.0	542.6	263.0	279.6	44.1	26.906	-5.478	8.50	34.1
60.0	607.1	327.5	279.6	52.5	27.389	-6.118	8.63	34.3
65.0	674.4	394.8	279.6	64.3	27.426	-6.193	8.74	34.3
70.0	744.6	465.0	279.6	81.0	27.743	-5.986	8.80	34.4
75.0	817.6	538.0	279.6	100.1	27.966	-5.739	9.00	34.7
80.0	893.3	613.7	279.6	130.9	28.293	-5.254	9.12	34.9
85.0	971.9	692.3	279.6	183.4	28.391	-4.845	9.23	35.0
89.0	1036.8	757.2	279.6	255.1	28.381	-4.253	9.32	35.0

Total Continuous Driving Time 110.00 minutes; Total Number of Blows 4366 (starting at penetration 5.0 ft)

APPENDIX D

AHTD Special Provision for Embankment Construction

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

DESCRIPTION: This Special Provision shall be supplementary to Section 210, Excavation and Embankment, of the Standard Specifications, Edition of 2003 and shall apply to the construction of embankments being built over existing borrow ditches as shown in the plans or where directed by the Engineer.

MATERIALS: Stone Backfill shall meet the requirements of Section 207 of the Standard Specifications, Edition of 2003.

Select Material (Class SM-2) shall meet the requirements of Section 302 of the Standard Specifications, Edition of 2003.

Dumped Riprap and Filter Blanket shall comply with Section 816 of the Standard Specifications except that synthetic geotextile fabric complying with requirements of Subsection 625.02, Type 5 must be used as a filter blanket under dumped riprap in lieu of a granular filter blanket material.

Clay plating shall consist of material having a minimum plasticity index of 10 and a maximum plasticity index of 25, which will support vegetation and not be highly susceptible to erosion.

CONSTRUCTION: When the embankment is to be built over existing borrow ditches, the ditches shall be undercut 2 feet below the existing flow line to remove all highly organic, wet material prior to embankment construction. The ditches shall then be filled using Stone Backfill. The top 4" to 6" of Stone Backfill shall be material complying with Section 303 of the Standard Specifications, Edition of 2003 for Class 7 Aggregate Base Course in accordance with Section 207. Excavation for the placement of Stone Backfill shall be considered part of the item in accordance with subsection 207.01 of the Standard Specifications.

The remaining embankment shall be constructed of Selected Material (Class SM-2). Synthetic Filter Blanket and Dumped Riprap shall be placed on the slopes of embankments constructed of Select Material (Class SM-2) from the top of the Stone Backfill to 2 feet above the high water elevation or as directed by the Engineer. The remainder of embankments constructed of Select Material (Class SM-2) or other material which is susceptible to erosion shall have a minimum 18 inch clay plating (measured

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

perpendicular to the finished slopes).

All embankment materials, including Selected Material (Class SM-2) and Clay Plating, shall be placed and compacted in accordance with Subsections 210.07, 210.09, and 210.10 of the Standard Specifications.

QUALTIY CONTROL AND ACCEPTANCE: The Contractor shall perform quality control and acceptance sampling and testing of the clay plating for plasticity index; Selected Material (Class SM-2) for gradation and plasticity index in accordance with Section 306 except that the size of the standard lot will be 3000 cubic yards. The Contractor shall perform quality control and acceptance sampling and testing of the Selected Material (Class SM-2) for density and moisture content in accordance with Subsection 210.02 of the Standard Specifications for Highway Construction. Selected Material (Class SM-2) shall meet the density requirements of Subsection 210.10.

METHOD OF MEASUREMENT: Embankments consisting of Selected Material (Class SM-2) and Clay Plating material and as shown on the plans, will be measured as Compacted Embankment in accordance with Subsection 210.12 of the Standard Specifications.

Stone Backfill will be measured in accordance with Section 207 of the Standard Specifications.

Filter Blanket and Dumped Riprap will be measured in accordance with Section 816 of the Standard Specifications.

BASIS OF PAYMENT: All accepted embankments; including Selected Material (Class SM-2) and Clay Plating material measured as provided above will be paid for as Compacted Embankment in accordance with Subsection 210.13 of the Standard Specifications.

Stone Backfill shall be paid in accordance with Section 207 of the Standard Specifications.

Filter Blanket and Dumped Riprap will be paid in accordance with Section 816 of the Standard Specifications.

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

Payment will be made under:

Pay Item	Pay Unit
Compacted Embankment	Cubic Yard
Stone Backfill	Ton
Filter Blanket	Square Yard
Dumped Riprap	Cubic Yard

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

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January 28, 2021

Cindy Rich, P.E.
Neel-Schaffer, Inc.
125 South Congress Street, Suite 1100
Post Office Box 22625
Jackson, Mississippi 39201

Report No. 200518 – Site 8

Geotechnical Exploration
Site 8
ARDOT SR230 Bridge Replacements
Craighead and Lawrence Counties, Arkansas

Dear Ms. Rich:

Submitted here is the report of our geotechnical exploration for the above-captioned project. This exploration was authorized by Task Order 108 to the Subconsultant Agreement between Neel-Schaffer, Inc. and Burns Cooley Dennis, Inc. dated September 17, 2020.

We appreciate the opportunity to be of service. If you should have any questions concerning this report, please do not hesitate to call us.

Very truly yours,

BURNS COOLEY DENNIS, INC.

Alexander B. Reeb, Ph.D., P.E.

A. E. (Eddie) Templeton, P.E.

ABR/AET/khb
Copy Submitted: (via e-mail)

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 Project Description

Plans are being made for the construction of replacement bridges and box culverts at ten sites along Highway 230 between Alicia and Bono in Craighead and Lawrence Counties, Arkansas. Site 8 is located in Craighead County where Highway 230 crosses the East Cache River Ditch. At this site, a new bridge will be constructed on a new alignment just north of the existing bridge.

The new bridge will be 225 ft long and consist of three spans of approximately equal spacing. It is our understanding that new fill will be placed to raise the grade at the new abutments above the grade of the existing bridge. The abutment spill-through slopes will be constructed as 2H:1V slopes, and the abutment side slopes will be constructed as 3H:1V slopes. The abutment bents are to be supported by 18-in. diameter, closed-ended steel pipe piles, and the interior bents are to be supported by 24-in. diameter, closed-ended steel pipe piles. A preliminary layout showing the proposed construction is presented on Figure 1 of this report.

1.2 Purposes

The specific purposes of this exploration were:

1) to review the exploratory soil borings made within the area planned for construction of the new bridge;

2) to verify field classifications and to evaluate pertinent physical properties of the soils encountered in the borings by means of visual examination of the soil samples in the laboratory and routine tests performed on the samples;

3) to perform analyses to investigate liquefaction, slope stability, settlement, pile capacity, and downdrag; and

4) to provide geotechnical recommendations for design and construction of the bridge.

Our scope of work for the bridge does not include providing recommendations for roadway subgrades and pavements. Discussion and recommendations pertaining to roadway subgrades and pavements are provided under separate cover.

2.0 FIELD EXPLORATION

2.1 General

Subsurface soil conditions within the area planned for construction of the bridge were explored by means of four deep borings. Borings ARDOT-1 and ARDOT-2 were performed by ARDOT. Borings S8-1 and S8-2 were performed by McCray Drilling under contract to SoilTech Consultants, Inc. The approximate locations of the borings are shown on Figure 1.

All soils were classified in general accordance with the Unified Soil Classification System. A synopsis of the Unified Soil Classification System (USCS) is presented on Figure 2 along with symbols and terminology typically utilized on graphical soil boring logs. Graphical logs of the borings are presented on Figures 3 through 6. The graphical logs illustrate the types of soil and stratification encountered with depth below the existing ground surface at the individual boring locations. Approximate GPS coordinates for the boring locations are shown at the bottom of the graphical boring logs within the “Comments” section.

2.2 Drilling Methods and Groundwater Observations

Boring ARDOT-1 was made to an exploration depth of 101.5 ft, Boring ARDOT-2 was made to an exploration depth of 111.5 ft, and Borings S8-1 and S8-2 were made to an exploration depth of 100 ft. The ARDOT borings were made using the ARDOT Acker AR2094 drill rig, and the McCray Drilling borings were made using a CME-750X buggy-mounted drill rig. Boring S8-1 and S8-2 were initially advanced to a depth of 50 ft and 55 ft, respectively, by dry augering and then were extended to completion using rotary wash drilling procedures. Groundwater was encountered at a depth of 48.5 ft and 49 ft in Borings S8-1 and S8-2, respectively.

2.3 Sampling Methods

Disturbed samples of soils were obtained by driving a standard 2-in. OD split-spoon sampler 18 in. into the soil with a 140-lb hammer falling freely a distance of 30 in. The depths at which the split-spoon samples were taken are illustrated as crossed rectangular symbols under the "Samples" column of the graphic logs. Standard penetration test (SPT) blow counts resulting from split-spoon sampling are recorded under the "Blows Per Ft" column of the graphic logs. The SPT blow counts are the “raw” field values. The recommended hammer energy correction factors are indicated in the “Comments” section of the logs. Relatively undisturbed samples of the soils encountered in Borings S8-1 and S8-2 were obtained by pushing a 3-in. OD Shelby tube sampler approximately 2 ft into the soil. The Shelby tube samples were obtained within the depth intervals illustrated as shaded portions of the "Samples" column of the graphic logs. The Shelby tube and/or split-spoon samples were generally obtained at approximate 3-ft to 5-ft intervals of depth. Disturbed auger cutting samples were taken near the ground surface in Borings S8-1 and S8-2. The depths at which the auger cutting samples were taken are illustrated as small I-shaped symbols under the "Samples" column of the graphic boring logs.

2.4 Field Classification, Sample Preservation and Borehole Abandonment

All soils encountered during drilling were examined and classified in the field by a geotechnical engineering technician. Representative portions of the split-spoon samples and the auger cutting samples were sealed in jars to provide material for visual examination and testing in the laboratory. The Shelby tubes obtained from Borings S8-1 and S8-2 were capped and the ends sealed with wax in the field to prevent moisture loss and structural disturbance while they were transported to the testing laboratory. At the testing laboratory, the Shelby tube samples were extruded, and an approximate 6-in. long portion of each sample was temporarily sealed in plastic wrap to prevent moisture loss during the period between sample extrusion and testing. Additional portions of each Shelby tube sample were sealed in jars to provide additional material for visual examination and testing. The boreholes for Boring S8-1 and S8-2 were plugged with soil cuttings after completion of drilling and sampling.

3.0 LABORATORY TESTING

3.1 General

All of the soil samples were examined in the laboratory and tests were performed on selected samples to verify field classifications and to assist in evaluating the strength and volume change properties of the soils encountered. The types of laboratory tests performed are described in the following paragraphs.

3.2 Strength Properties

The undrained shear strength characteristics of the fine-grained soils encountered in the borings were investigated by means of visual estimates of consistency and from the results of unconsolidated undrained (UU) triaxial compression tests performed on selected undisturbed Shelby tube samples. The cohesions resulting from the UU triaxial compressions test are plotted as small open triangles in the data section of the graphic boring logs. The water content and dry density were also determined for each UU triaxial compression test specimen. The water contents are plotted as small shaded circles in the data section of the graphic logs. The dry densities are tabulated to the nearest lb per cu ft under the "Dry Density" column of the graphic boring logs.

3.3 Classification Tests

The classifications and volume change properties of the fine-grained soils encountered in the borings were investigated by means of Atterberg liquid and plastic limit tests performed on selected representative samples. The results of the liquid and plastic limit tests are plotted as small crosses interconnected by dashed lines in the data section of the graphic boring logs. In accordance with the Unified Soil Classification System, fine-grained soils are classified as either clays or silts of low or high plasticity based on the results of Atterberg limit tests. The numerical difference between the liquid limit and plastic limit is defined as the plasticity index (PI). The magnitudes of the liquid limit and plasticity index and the proximity of the natural water content to the plastic limit are indicators of the potential for a fine-grained soil to shrink or swell upon changes in moisture content or to consolidate under loading. The proximity of the natural water content to the plastic limit is also an indicator of soil strength.

The classifications of some samples were investigated by means of minus No. 200 sieve tests. The percentages of fines resulting from the minus No. 200 sieve tests are tabulated at the appropriate depths under the "% Passing No. 200 Sieve" column of the graphic boring logs.

The classifications of some samples were investigated by means of sieve and hydrometer analyses. Particle size distribution curves from these tests are presented in Appendix A. The percentages of fines resulting from the sieve tests are also tabulated at the appropriate depths under the "% Passing No. 200 Sieve" column of the graphic boring logs

3.4 Water Content Tests

Water content tests were performed on samples to corroborate field classifications and to extend the usefulness of the strength, plasticity, and field SPT blow count data. The results of the water content tests are plotted as small shaded circles in the data section of the graphic boring logs. The water content data have been interconnected on the logs to illustrate a continuous profile with depth.

3.5 Soluble Sulfates, pH, and Resistivity Tests

Laboratory testing was performed on selected samples from the borings to determine the percent of soluble sulfate by mass, soil pH, and soil resistivity. Sulfate testing was performed on all three samples, and soil pH and resistivity testing was performed on two of the three samples. Results of the tests are presented in Table 1.

Table 1 - Soluble Sulfates, pH, and Resistivity Test Results

Boring	Sample Depth (ft)	USCS	Sulfate (SO_4), % by mass	Average pH	Resistance (ohm-cm)
S8-1	5	SC	<0.010	7.74	3600
S8-1	53.5	SP	0.014	-	-
S8-2	8	CH	0.022	7.06	1100

4.0 GENERAL SUBSURFACE CONDITIONS

4.1 General

A general description of subsurface soil and groundwater conditions revealed by the borings made for this exploration is provided in the following paragraphs. The graphical logs shown on Figures 3 through 6 should be referred to for specific soil and groundwater conditions encountered at each boring location. Stick logs of the borings are shown in profile with the proposed bridge section on Figure 7 to aid in visualizing subsurface soil conditions. Tabulated adjacent to the stick logs are Atterberg liquid and plastic limits, water contents, dry densities, cohesions, percentages of fines passing the No. 200 sieve and field SPT blow counts.

4.2 Geology

The project site is located within the physiographic province known as the Mississippi River Alluvial Plain. Geological maps indicate Quaternary age deposits are continuous throughout the project area. The Quaternary deposits at the site include alluvial sediments from both the Holocene and Pleistocene series. Sediments typically include a substratum zone of sands and gravels overlain by a top stratum of clays and silts.

Tertiary deposits are present below the Quaternary deposits. Tertiary deposits within the project vicinity are expected to consist of hard clays, sandy clays and silty clays containing organics and lignite interbedded with very dense sand strata. Geological maps suggest that the elevation of top of the Tertiary deposits range from about El 125 to 150 ft MSL.

4.3 Soil Stratification

As shown on the Figure 7 profile, the soils encountered at the site were grouped into the zones outlined below. The zones were generally based on the soil classifications and interpreted strengths used in design. The borings generally indicate fill materials and fine-grained top stratum soils overlying alluvial sands.

- Zone 1 – Loose to medium dense clayey sand (SC), silty sand (SM), and sand (SP) with silt
- Zone 2 – Medium stiff to stiff sandy clay (CL) and clay (CH) with sand
- Zone 3 – Loose to dense sand (SP), slightly silty sand (SP-SM), and sand (SP) with gravel
- Zone 4 – Medium dense to very dense sand (SP) and sand (SP) with gravel

Zone 1 and 2 soils were generally encountered from the ground surface to a depth of about 20 ft. Zone 3 soils were encountered from a depth of about 20 ft down to depths ranging from about 70 to 80 ft. Zone 4 soils were encountered beneath the Zone 3 soils and extend to the boring termination depths.

Zone 3 was further divided into Zones 3A, 3B, and 3C based on the estimated likelihood of liquefaction and potential for strength loss due to an earthquake. The soils encountered in Zones 3A and 3C were generally identified as having a high likelihood of liquefaction and significant strength loss. The soils encountered in Zone 3B were generally identified as having a moderate likelihood of liquefaction but no significant strength loss.

We understand that new fill materials will be placed along the new alignment to create the approach embankments. The thickness of the proposed new fill at abutments along the bridge centerline is illustrated on the profile.

4.4 Groundwater

Groundwater was encountered during auger drilling at a depth of 48.5 ft and 49 ft in Borings S8-1 and S8-2, respectively. Groundwater observations were not reported on the ARDOT boring logs. Groundwater cannot be observed during rotary wash drilling. In our opinion, groundwater conditions at the site will be influenced by rainfall, surface drainage, and by the rise and fall of water levels in the nearby ditches, creeks, ponds or other bodies of water. The regional groundwater is primarily influenced by the Mississippi River. Groundwater conditions at the site can also be influenced by man-made changes. Surficial soils can become saturated and weak to relatively shallow depths during periods of prolonged and heavy rainfall.

5.0 ENGINEERING ANALYSES AND DISCUSSION

5.1 General

The purposes of this study were to perform analyses and develop geotechnical recommendations for: 1) seismic design including site classification, liquefaction, and seismic compression; 2) slope stability including proposed slope grading and configuration to provide acceptable factors of safety; and 3) deep foundation design including axial capacity curves, downdrag, lateral analysis parameters, and drivability analysis. A discussion of our analyses is provided in the following subsections.

5.2 Seismic

Seismic evaluations and analyses were generally performed based on the guidance provided by ARDOT and based on the recommendations discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual and in Idriss and Boulanger (2008).

5.2.1 Site Classification. Soil shear wave velocity data are not available for the bridge site. The site class was determined from SPT blow counts and undrained shear strength data in accordance with definitions provided in Table 3.10.3.1-1 of the AASHTO LRFD 2017 Bridge Design Specifications. We recommend that a site class D be utilized to determine the site coefficient and spectral response acceleration for this bridge site. The site is classified as within Seismic Zone 3 per Table 3.10.6-1.

The acceleration design response spectrum was developed using the computer program "AASHTO Seismic Design Parameters" version 2.10 developed by the U.S. Geological Survey. The recommended design values are presented subsequently in tabular format. Plots of the design spectrum are included as Figures 8 and 9.

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 35.909722
Longitude = -90.863056

Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.432	PGA - Site Class B
0.2	0.799	Ss - Site Class B
1.0	0.204	S1 - Site Class B

Spectral Response Accelerations SDs and SD1
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1

Site Class D - Fpga = 1.07, Fa = 1.18, Fv = 1.99
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.463	As - Site Class D
0.2	0.943	SDs - Site Class D
1.0	**0.407**	**SD1 - Site Class D: Seismic Zone 3**

Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	Sd in.	
0.000	0.463	0.000	T = 0.0, Sa = As
0.086	0.943	0.069	
0.200	0.943	0.368	T = 0.2, Sa = SDs
0.432	0.943	1.716	T = Ts, Sa = SDs
0.500	0.814	1.988	
0.600	0.678	2.385	
0.800	0.509	3.180	
1.000	0.407	3.976	T = 1.0, Sa = SD1
1.200	0.339	4.771	
1.400	0.291	5.566	
1.600	0.254	6.361	
1.800	0.226	7.156	
2.000	0.203	7.951	
2.200	0.185	8.746	
2.400	0.170	9.541	
2.600	0.157	10.336	
2.800	0.145	11.131	
3.000	0.136	11.927	
3.200	0.127	12.722	
3.400	0.120	13.517	
3.600	0.113	14.312	
3.800	0.107	15.107	
4.000	0.102	15.902	

5.2.1 Liquefaction Triggering. Liquefaction triggering evaluations were performed using the Microsoft Excel workbook developed by Cox and Griffiths (2011)[1] and provided by ARDOT. The liquefaction evaluations were performed using all three procedures available in the workbook: Youd et al. (2001)[2], Cetin et al. (2004)[3], and Idriss and Boulanger (2008)[4].

The design earthquake magnitude (M_W) was estimated using the Unified Hazard Tool on the U.S. Geological Survey (USGS) website. Deaggregations were computed using the 2008

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[1] Cox, B. R., and Griffiths, S. C. (2011). *Practical Recommendations for Evaluation and Mitigation of Soil Liquefaction in Arkansas*, MBTC 3017, Mack-Blackwell Rural Trans. Center at the U. of Arkansas.

[2] Youd, T. L., Idriss, I.M., et al. (2001). "Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops of evaluation of liquefaction resistance of soils." *J. of Geotech. and Geoevir. Engrg*., Vol. 127(4): 297-313.

[3] Cetin, K.O., Seed, R.B., Kiureghain, A.D., Tokimatsu, K., Harder, L.F., Kayen, R.E., Moss, R.E.S. (2004). "Standard Penetration Test-Based Probabilistic and Deterministic Assesment of Seismic Soil Liquefaction Potential." *J.of Geotech. and Geoevir. Engrg.*, Vol. 130(12): 1314-1340.

[4] Idriss, I. M., and Boulanger, R. W. (2008). "Soil Liquefaction during Earthquakes." *MNO-12*, Earthquake Engineering Research Institute.

(v3.3.3) edition of the National Seismic Hazard Mapping Project (NSHMP). A return period of 5% in 50 years (i.e., 975 years) was used in the deaggregation. The resulting modal earthquake magnitude of 7.7 was used as input in the liquefaction triggering workbook.

The liquefaction triggering evaluation was performed for Borings ARDOT-2, S8-1, and S8-2. A liquefaction triggering evaluation was not performed for Boring ARDOT-1 because the laboratory tests to determine the percent fines content were not run for this boring. The liquefaction triggering workbook input is provided for each boring in Appendix B. As recommended by Cox and Griffiths (2011), a blow count N-value of 1 was input in the workbook at sample depths where SPT blow counts were not measured in Borings S8-1 and S8-2. For these borings and depths, the Factor of Safety (FS) against liquefaction was not calculated. In Boring S8-2, there was not enough sample recovered at a depth of 25 ft to perform a minus No. 200 sieve test. For the liquefaction evaluation, it was assumed that this sample has a fines content of 6.6%, which is the average of the fines contents of the samples above and below. Comparison plots that show the resulting liquefaction FS values vs. elevation for each of the three evaluation procedures are provided as Figures 10, 11, and 12 for Borings ARDOT-2, S8-1, and S8-2, respectively.

5.2.2 Seismic Compression. Potential seismic compression was calculated for all soil layers that were identified as likely to liquefy (i.e., FS<=1.0) based on the Idriss and Boulanger (2008) liquefaction triggering criteria. The seismic compression calculations were performed following two different procedures: Tomkimatsu & Seed (1987)[5] and Idriss and Boulanger (2008). The Tomkimatsu & Seed (1987) procedure for calculating seismic compression is discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

Plots that show the distribution of estimated seismic compression vs. elevation for the two procedures are provided as Figures 13, 14, and 15 for Borings ARDOT-2, S8-1, and S8-2, respectively. For reference, the top and bottom elevation of the boring is indicated by a horizontal dashed line on each plot. As shown in these figures, the total estimated settlements at the boring locations due to seismic compression range from about 8 to 13 inches depending on the analysis method.

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[5] Tokimatsu, K. and Seed, H.B. (1987). "Evaluation of settlements in sand due to earthquake shaking." *J. of Geotech. Engrg.*, Vol. 113(8): 861-878.

5.2.3 Residual Strengths of Liquefied Soils. Residual strengths for post-earthquake stability analyses were estimated for soils that were identified as likely to liquefy (i.e., FS<=1.0) based on the Idriss and Boulanger (2008) liquefaction triggering criteria. The residual strengths were estimated using the procedures outlined in Idriss and Boulanger (2008) and based on the correlation proposed by Olson and Johnson (2008)[6]. The correlations proposed by Olson and Johnson (2008) are included in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

5.3 Slope Stability

Slope stability analyses were performed for the proposed conditions using the SLOPE/W computer program and the Spencer Method. The stability analyses were performed for end of construction, long term, pseudo-static, and post-earthquake conditions. We understand that the target factors of safety are 1.5 for end of construction and long-term conditions, and 1.1 for pseudo-static and post-earthquake conditions. Analyses were performed for the spill-though slopes and for the embankment side slopes. A traffic surcharge load of 250 psf was applied in pavement areas in the analyses.

The end-of-construction analyses use undrained strengths for cohesive soils and drained strengths for cohesionless soils. The long-term analyses use drained strengths for all soils. The pseudo-static analyses use undrained strengths for cohesive soils, drained strengths for cohesionless soils, and include a seismic coefficient equal to 0.5 times the site class specific PGA (i.e., $0.5*F_{PGA}*PGA$) as suggested in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual. The post-earthquake analyses use undrained strengths for cohesive soils, residual strengths for cohesionless soils that were identified as likely to liquefy, and drained strengths for cohesionless soils that were not identified as likely to liquefy. For cohesive soils that were estimated to have peak undrained strengths of approximately 1,500 psf or less, undrained strengths equal to 0.8 times the peak undrained strengths were used in the post-earthquake analyses to account for possible cyclic softening.

A summary of the slope stability Factor of Safety (FS) values is provided in Table 2. The analyzed geometries, soil properties, and critical failure surfaces are shown in Figures 16 to 27.

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[6] Olson, S. M. and Johnson, C. I. (2008). "Analyzing Liquefaction-induced Lateral Spreads Using Strength Ratios." *J. of Geotech. and Geoenviron. Engrg.*, 134(8): 1035–1049.

Based on our review of the soil conditions and the proposed abutment grading, we judge that the north side slope of the west abutment is the critical side slope for stability. Since the resulting FS values are acceptable, we did not perform stability analyses for the other side slopes.

Table 2 - Slope Stability FS Results Summary

		Calculated FS Values		
Conditions	Required FS	West Abutment Spill-Through	East Abutment Spill-Through	West Abutment North Side Slope (815+00)
End of Construction	1.5	2.63	2.50	3.05
Long Term	1.5	1.95	1.85	1.67
Pseudostatic	1.1	1.25	1.31	1.59
Post-Earthquake	1.1	1.22	1.13	1.31

5.4 Embankment Consolidation Settlement

Considering the height of fill to be placed for the approach embankments and the compressibility of the soils encountered in the borings, it is our opinion that consolidation settlement of the bridge embankments will be less than 2 in. Approximately 50 percent of the settlement is expected to occur during bridge construction. No settlement problems due to consolidation settlement are anticipated at these sites, and no special mitigation will be required.

5.5 Deep Foundations

We understand that driven 18-in. and 24-in. diameter, closed-ended steel pipe piles are proposed for the abutment bents and interior bents, respectively. Analyses were performed to evaluate the abutment bents and interior bents pile capacities based on the guidance provided by ARDOT and the recommendations discussed in the FHWA (2014) LRFD Seismic Analysis and Design of Bridges Reference Manual.

5.5.1 Axial Pile Capacity. Axial pile capacity curves were computed based on the pile type shown on the provided plans and the subsurface soil conditions encountered in the borings. Scour was not considered in our analyses. If significant scour is anticipated, we should be contacted to provide revised capacity curves.

The pile capacities were estimated based on the FHWA design procedure using the ENSOFT computer program APile v2015. The compression capacity of an individual pile consists of a combination of skin friction around the perimeter of the pile shaft and end bearing at the tip. The skin friction in the upper 5 ft of soil was neglected. Separate calculations were performed to determine pile capacities with and without consideration of seismic effects. For the calculations that consider seismic effects, the pile skin friction was reduced by 90% for liquefiable soil layers between the ground surface and a depth of 50 ft and the pile skin friction was reduced by 50% for liquefiable soil layers below a depth of 50 ft.

The pile capacity curves are presented in Figures 28, 29, and 30, for the west abutment, east abutment, and interior bents, respectively. The pile capacity curves are presented as nominal (ultimate) values that do not include a resistance factor. An appropriate resistance factor should be applied to the nominal values presented on the pile capacity curves. Guidance on resistance factors is provided in Section 6.2. We recommend that the piles extend at least 10 feet into Zone 4 (see Figure 7 profile) to ensure that the piles are tipped below the deepest soil layer with a high likelihood of liquefaction (i.e., Zone 3C).

5.5.2 Downdrag. The seismic compression of the liquefiable soil layers can result in drag loads and increased pile settlement. Pile drag loads occur when the soils surrounding a pile settle more than the pile and apply negative skin friction to the pile. These drag loads increase the compressive loads in the pile that should be considered as part of the pile structural design. Structural capacity determination of the piles is not in our scope for this investigation.

The depth at which the pile and the soils settle the same amount is referred to as the neutral plane. Below the neutral plane, the pile settles more than the surrounding soils. The depth of the neutral plane depends on the soil settlement profile, the pile length, the distribution of pile skin friction and end bearing, and the load applied to the top of the pile. The soil settlement profiles were based on the distributions of seismic compression. The distributions of pile skin friction and end bearing were based on the axial pile capacity curves that consider reduced skin friction in the liquefiable soil layers. We used unfactored dead loads provided by Neel Schaffer, Inc. as the loads applied to the tops of the piles. For the interior bent piles, we added the self-weight of the pile stick-up (between the ground surface and the bottom of the pile cap) to the unfactored dead loads.

The downdrag analysis results are summarized in the following tables. Table 3 and Table 4 present the results for the west abutment bent for loads of 105 kips and 130 kips, respectively. Table 5 and Table 6 present the results for the east abutment bent for loads of 105 kips and 130 kips, respectively. Table 7 presents the results for the interior bents for a load of 159 kips. For each case, results are provided for a range of possible pile lengths.

Table 3 - Downdrag Analysis Results for West Abutment with Load of 105 kips

	Pile Length (ft) below El 247 ft				
	70	75	80	90	100
Maximum Drag Load (kips)	115	145	171	171	171
Top of Pile Settlement (in.)	2.5	0.9	0.1	0.1	0.1
Neutral Plane Depth (ft)	61.9	65.3	67.0	67.0	67.0

Table 4 - Downdrag Analysis Results for West Abutment with Load of 130 kips

	Pile Length (ft) below El 247 ft				
	70	75	80	90	100
Maximum Drag Load (kips)	102	134	171	171	171
Top of Pile Settlement (in.)	3.5	1.4	0.1	0.1	0.1
Neutral Plane Depth (ft)	59.6	64.2	67.0	67.0	67.0

Table 5 - Downdrag Analysis Results for East Abutment with Load of 105 kips

	Pile Length (ft) below El 247 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	237	283	315	315	315
Top of Pile Settlement (in.)	3.6	1.0	0.1	0.1	0.1
Neutral Plane Depth (ft)	79.1	85.0	87.0	87.0	87.0

Table 6 - Downdrag Analysis Results for East Abutment with Load of 130 kips

	Pile Length (ft) below El 247 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	222	267	315	315	315
Top of Pile Settlement (in.)	4.4	1.8	0.1	0.1	0.1
Neutral Plane Depth (ft)	77.3	83.4	87.0	87.0	87.0

Table 7 - Downdrag Analysis Results for Interior Bents with Load of 159 kips

	Pile Length (ft) below El 232 ft				
	70	75	80	90	100
Maximum Drag Load (kips)	151	171	171	171	171
Top of Pile Settlement (in.)	0.4	0.1	0.1	0.1	0.1
Neutral Plane Depth (ft)	65.2	66.0	66.0	66.0	66.0

5.5.3 Lateral Analysis Parameters. If lateral loads applied to the piles are substantial, a lateral load analysis should be performed. The piles should be designed so that angular rotation and deflection at the tops of the piles are maintained within structurally tolerable limits. We recommend that the response of the piles to applied moment and lateral loading be analyzed utilizing the method developed by Dr. Lymon C. Reese of the University of Texas or a similar analysis procedure. Computer programs (e.g., LPILE) are available for this method of analysis. The analysis method utilizes finite difference approximations to solve for deflection, moment, soil modulus and soil reaction for a single pile. Soil response to the laterally loaded pile is represented in the analysis by a set of nonlinear “p-y" curves that are developed for various depths along the pile and for the different soil types. The "p-y" curves essentially indicate the soil reaction in force per unit length of pile versus deflection for a given pile diameter. A tabulation of recommended soil parameters that can be used in the lateral pile analysis are presented in Table 8. The LPILE default values of E_{50} and k, which are correlated based on the cohesion and friction angle, can be used in the lateral pile analysis.

Table 8 - Recommended Soil Parameters for Lateral Pile Analysis

Soil Zone	p-y Curve Type	Effective Unit Weight (pcf)	Cohesion (psf)	Internal Friction Angle (degrees)
New Fill	Stiff Clay w/o Free Water (Reese)	120	1500	-
1	Sand (Reese)	52.6	-	32
2	Soft Clay (Matlock)	62.6	1000	-
3A, 3B, 3C	Sand (Reese)	57.6	-	34
4	Sand (Reese)	57.6	-	37

Liquefaction of sands and cyclic softening of clay soils can result in significant short-term strength losses that can reduce lateral pile capacity. Accordingly, Table 9 provides a separate set of soil parameters that should be used instead of the values in Table 8 in the lateral pile analysis for seismic conditions.

Table 9 - Recommended Post-Earthquake Soil Parameters for Lateral Pile Analysis

Soil Zone	p-y Curve Type	Effective Unit Weight (pcf)	Cohesion (psf)	Internal Friction Angle (degrees)
New Fill	Stiff Clay w/o Free Water (Reese)	120	1200	-
1	Sand (Reese)	52.6	-	32
2	Soft Clay (Matlock)	62.6	800	-
3A	Soft Clay (Matlock)	57.6	300	-
3B	Sand (Reese)	57.6	-	34
3C	Soft Clay (Matlock)	57.6	300	-
4	Sand (Reese)	57.6	-	37

5.5.4 Drivability Analysis. A "drivability" type wave equation analysis relating pile penetration, ultimate static pile capacities, dynamic pile driving stresses, minimum recommended open-ended, diesel hammer energy and hammer strokes to blow counts was performed using the program GRLWEAP v.2010. The unit skin friction and end-bearing values in each soil layer were developed based on the results of unconsolidated undrained (UU) triaxial compression tests, supplemented by the results of the field standard penetration tests and visual estimates of consistency and the static analysis program in GRLWEAP. A 72% pile hammer efficiency and a shaft gain/loss factor of 0.833 and a toe gain/loss factor of 1.0 were used in the analysis. A maximum driving stress of 90% of the steel yield strength was considered for these analyses.

Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. An open-ended D36 diesel hammer was utilized for the drivability analyses of both pile sizes. Hammer and pile cushion information was based on manufacturer-recommended values. Both the 18-in. and 24-in. diameter steel pipe piles were assumed to be installed close-ended. In the analyses, the piles at the abutments and interior bents are assumed to be driven from the plan pile cap bottom elevations to the recommended tip elevations. Graphical and tabulated results of the drivability analyses are provided in Appendix C. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed prior to hammer acceptance and beginning of driving. The resulting minimum hammer energy to drive the piles at the abutment and interior bents is provided in Table 10.

Table 10 - Results of Drivability Analyses

Location	Hammer Type	Minimum Hammer Energy (kip-ft.)
Abutment Bents	D36	80
Interior Bents	D36	80

The parameters used in the wave equation analysis were based on general information available at the time of the analysis; however, actual field conditions may be different. We recommend prudent use of the wave equation analysis results. Soil response, hammer performance, and pile stresses and drivability should be verified by dynamic measurements using the Pile Driving Analyzer (PDA) on site and subsequent data analysis with the CAPWAP program. The actual suitability and final acceptance of a hammer system for a given project can only be determined after demonstration of satisfactory field performance, which is typically evaluated during the Test Pile Driving Program with PDA dynamic pile measurements and related data analyses.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Pile Design and Installation

Driving refusal for the steel pipe piles may occur in the dense to very dense sands encountered in Zone 4 (see Figure 7 profile). If refusal occurs at depths shallower than the required minimum depth, then jetting will be required to achieve additional penetration. However, the final 5 ft of pile penetration must be achieved by driving. Driven piles should be installed in accordance with AHTD Standard Specification Section 805 PILING.

The pile capacity curves presented in this report do not reflect the effects of jetting. As described in FHWA-NHI-16-009, Design and Construction of Driven Pile Foundations, the use of jetting will result in greater soil disturbance than considered in standard static pile capacity calculations. Some field studies have reported that the pile side resistance may be reduced by about 50 percent over the jetted depth. If jetting is necessary, we should be contracted to provide revised

axial capacities. Dynamic load testing should be performed during construction to more accurately determine the ultimate capacity of the piles after jetting.

6.2 Test Piles, Dynamic Load Testing, and Resistance Factors

Based on Table 10.5.5.2.3-1 of the AASHTO LRFD 2017 Bridge Design Specifications and considering that the soil profiles consist predominantly of sand, a resistance factor of 0.45 should generally be applied for axial compression and a resistance factor of 0.35 should generally be applied for tension. A higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 11.

Table 11 - Pile Resistance Factors based on Condition/Resistance Determination Method

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 condition, but no less than 2% of the 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 the production piles.	0.65
	Wave equation analysis, without pile dynamic measurements or load test by with field confirmation of hammer performance.	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only).	0.40

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

As discussed in Section 10.5.5.3.3 of the Bridge Design Specifications, a resistance factor of 1.0 should be applied for axial compression and a resistance factor of 0.80 should be applied for tension when designing the foundations to resist earthquake loading.

We recommend a minimum of two test piles (one at an abutment bent and one at an interior bent) be driven to evaluate pile capacities and drivability, prior to ordering the production piles. The test pile lengths should be selected considering the estimated pile capacities, minimum penetration requirements, and the anticipated driving resistance. The test piles can be driven at permanent pile locations.

We recommend that dynamic pile load testing be performed on the test piles in accordance with ASTM D 4945. The results of the dynamic pile load test should be used to establish driving criteria for the production piles. The embedment length of the piles may be increased based on the PDA evaluation. All testing should be performed prior to ordering production piles in case the design lengths change due to the testing.

The dynamic pile load testing data collection should be performed by an engineer with a minimum of one year of dynamic pile testing field 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/or Foundation QA. Pile driving modeling and analysis of PDA data should be performed by an engineer with a minimum of five years of 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/or Foundation QA.

6.3 Embankment Construction

Embankment construction shall conform with Section 210 and all other applicable requirements of the latest AHTD Standard Specification for Highway Construction. The fill material for embankment construction should classify as AASHTO A-6, A-5, or A-4 with a liquid limit less than 45 and a plasticity index less than or equal to 25. The fill materials should be compacted to not less than 95 percent of standard Proctor maximum dry density (AASHTO T99) at moisture contents within 3 percentage points of the optimum moisture content. Fill material with a plasticity index less than 10 or that is susceptible to erosion shall have a minimum 18-inch clay plating (measured perpendicular to the finished slopes). Clay plating shall consist of material having a plasticity index in the range of 10 to 25 that supports vegetation and that is not highly susceptible to erosion.

As an initial site preparation step, existing utilities or pipes and any other subsurface obstructions that might interfere with earthwork, bridge, and/or drainage ditch construction should

be removed and/or relocated. Stripping should then be performed within the construction areas to remove organic-laden surficial soils, vegetation, debris, brush or roots. Temporary excavation slopes should not be steeper than 1H:1V. We recommend that excavations be left open for the shortest possible duration to minimize exposure of the bearing soils to rainfall. Drainage should be maintained away from the excavations during construction.

Prior to placement of any fill materials, the soils exposed after excavation should be inspected. Any obviously weak soils should be excavated and replaced with properly compacted backfill. The effort required to mitigate any unstable soils will be influenced by the season of the year when earthwork is performed. The soils may be drier during the hot late summer and could weaken during heavy rain events. We recommend that earthwork be performed during a dry summer or fall season, if the schedule permits. The vertical and lateral extent of excavation required to remove any weak soils must be determined in the field during earthwork construction. In order to minimize the amount of excavation, we recommend that a representative of Burns Cooley Dennis, Inc. be present to observe excavation operations and assist in evaluating the depth and lateral extent of any excavation required.

In areas where embankments are to be constructed over existing ditches, we understand that the work will conform with the requirements presented in the AHTD Special Provision for Embankment Construction, which is provided in Appendix D. This special provision requires that the ditches shall be undercut 2 feet to remove all highly organic, wet material and backfilled with Stone Backfill prior to embankment construction. The remaining embankment shall be constructed of Select Material (Class SM-2). Synthetic Filter Blanket and Dumped Riprap shall be placed on the slopes of embankments constructed of SM-2 from the top of the Stone Backfill to at least 2 feet above the high-water elevation. The remainder of embankments construction of SM-2 or other material that is susceptible to erosion shall have a minimum 18-inch clay plating (measured perpendicular to the finished slopes). Clay plating shall consist of material having a plasticity index in the range of 10 to 25 that supports vegetation and that is not highly susceptible to erosion.

As discussed in Section 210.09 of the AHTD Standard Specification, where fill materials are to be placed and compacted against a slope, the slope shall be continuously benched as the fill lifts are placed and compacted.

Laboratory classification tests, including grain size analyses and Atterberg limit determinations, should be performed on the backfill soils initially and routinely during earthwork

operations to check for compliance with the recommendations provided herein. Field moisture and density tests should be performed at frequencies that satisfy the requirements specified in Section 210.02 of the AHTD Standard Specification.

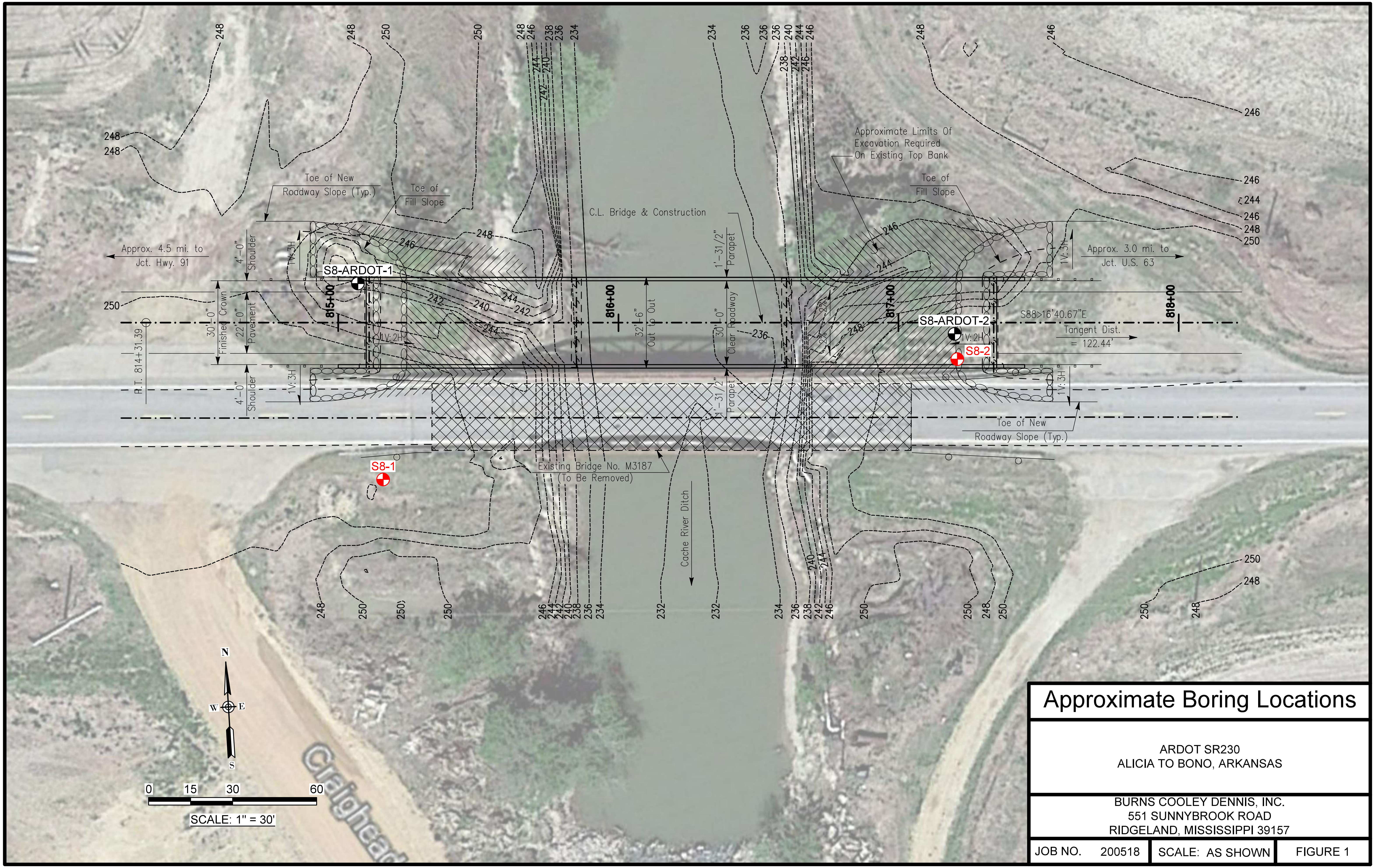
7.0 REPORT LIMITATIONS

The analyses, conclusions, and recommendations discussed in this report are based on conditions as they existed at the time of the exploration and further on the assumption that the exploratory borings are representative of subsurface conditions throughout the areas investigated. It should be noted that actual subsurface conditions between and beyond the borings might differ from those encountered at the boring locations. If subsurface conditions are encountered during construction that vary from those discussed in this report, Burns Cooley Dennis, Inc. should be notified immediately in order that we may evaluate the effects, if any, on earthwork and foundation design and construction.

Burns Cooley Dennis, Inc. should be retained for a general review of final design drawings and specifications. It is advised that we also be retained to observe earthwork for the project, to perform and observe the pile testing, and to develop the pile driving criteria. Our involvement during construction would give opportunity for us to help confirm that our recommendations are valid or to modify them accordingly. Burns Cooley Dennis, Inc. cannot assume responsibility or liability for the adequacy of recommendations if we do not observe construction.

This report has been prepared for the exclusive use of Neel-Schaffer, Inc. for specific application to the geotechnical-related aspects of design and construction of the ARDOT SR230 Bridge Replacements in Craighead and Lawrence Counties, Arkansas. The only warranty made by us in connection with the services provided is we have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing in the same or similar locality. No other warranty, express or implied, is made or intended.

FIGURES



Approximate Boring Locations

ARDOT SR230
ALICIA TO BONO, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO.	200518	SCALE: AS SHOWN	FIGURE 1

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL & LETTER	DESCRIPTION
COARSE-GRAINED SOILS More than half of material larger than No. 200 sieve size	GRAVELS More than half of coarse fraction larger than No.4 sieve size	Clean Gravels (Little or no fines)	GW	WELL GRADED GRAVEL, GRAVEL-SAND MIXTURE
			GP	POORLY GRADED GRAVEL, GRAVEL-SAND MIXTURE
		Gravels with fines (Appreciable amount of fines)	GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURE
			GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY MIXTURE
	SANDS More than half of coarse fraction smaller than No.4 sieve size	Clean Sands (Little or no fines)	SW	WELL GRADED SAND, GRAVELLY SAND
			SP	POORLY GRADED SAND, GRAVELLY SAND
		Sands with fines (Appreciable amount of fines)	SM	SILTY SAND, SAND-SILT MIXTURE
			SC	CLAYEY SAND, SAND-CLAY MIXTURE
FINE-GRAINED SOILS More than half of material smaller than No. 200 sieve size	SILTS AND CLAYS	Liquid limit less than 50	ML	SILT WITH LITTLE OR NO PLASTICITY
			ML	CLAYEY SILT, SILT WITH SLIGHT TO MEDIUM PLASTICITY
			ML	SANDY SILT
			CL	SILTY CLAY, LOW TO MEDIUM PLASTICITY
			CL	SANDY CLAY, LOW TO MEDIUM PLASTICITY (30% TO 50% SAND)
	SILTS AND CLAYS	Liquid limit greater than 50	MH	SILT, HIGH PLASTICITY
			CH	CLAY, HIGH PLASTICITY
			OH	ORGANIC CLAY OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOIL

TERMS CHARACTERIZING SOIL STRUCTURE

Slickensided - Clays with polished and striated planes created as a result of volume changes related to shrinking, swelling and/orchanges in overburden pressure.

Fissured - Clays with a blocky or jointed structure generally created by seasonal shrinking and swelling.

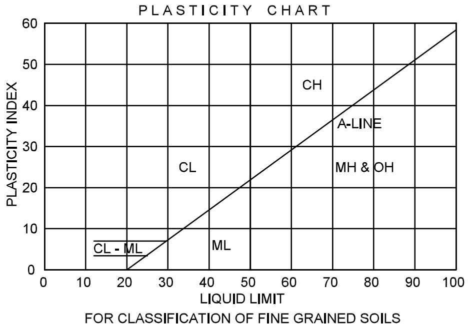
Laminated - Composed of thin alternating layers of varying color and texture.

Calcareous - Containing appreciable quantities of calcium carbonate.

Parting - Paper thin (less than 1/8 inch).

Seam - 1/8 inch to 3 inch thickness.

Layer - Greater than 3 inches in thickness.



FOR CLASSIFICATION OF FINE GRAINED SOILS

TERMS CHARACTERIZING SOIL STRUCTURE

COARSE-GRAINED SOILS		FINE-GRAINED SOILS		
	PENETRATION RESISTANCE, N		COHESION	PENETRATION RESISTANCE, N
DENSITY	Blows per Foot	Consistancy	Kips/Sq.Ft	Blows per Foot
Very loose	0 - 4	Very Soft	<0.25	0 - 1
Loose	5 - 10	Soft	0.25 - 0.50	2 - 4
Medium Dense	11 - 30	Medium Stiff	0.50 - 1.00	5 - 8
Dense	31 - 50	Stiff	1.00 - 2.00	9 - 15
Very Dense	>4.00	Very Stiff	2.00 - 4.00	16 - 30
		Hard	>4.00	>30

PARTICLE SIZE IDENTIFICATION

Cobbles - Greater than 3 inches

Gravel - Coarse-3/4 inch to 3 inches
Fine-4.76 mm to 3/4 inch

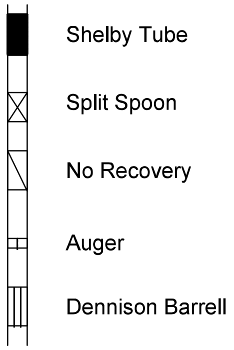
Sand - Coarse-2 mm to 4.76 mm
Medium-0.42 mm to 2 mm
Fine-0.074 mm to 0.42 mm

Silt & Clay - Less than 0.074 mm

RELATIVE COMPOSITION

Slightly	5 - 15%
With	16 - 29%
Sandy (or gravelly)	30 - 50%

SAMPLE TYPES
(Shown in Sample Column)



CLASSIFICATION, SYMBOLS AND
TERMS USED ON GRAPHICAL
BORING LOGS

BURNS COOLEY DENNIS, INC.

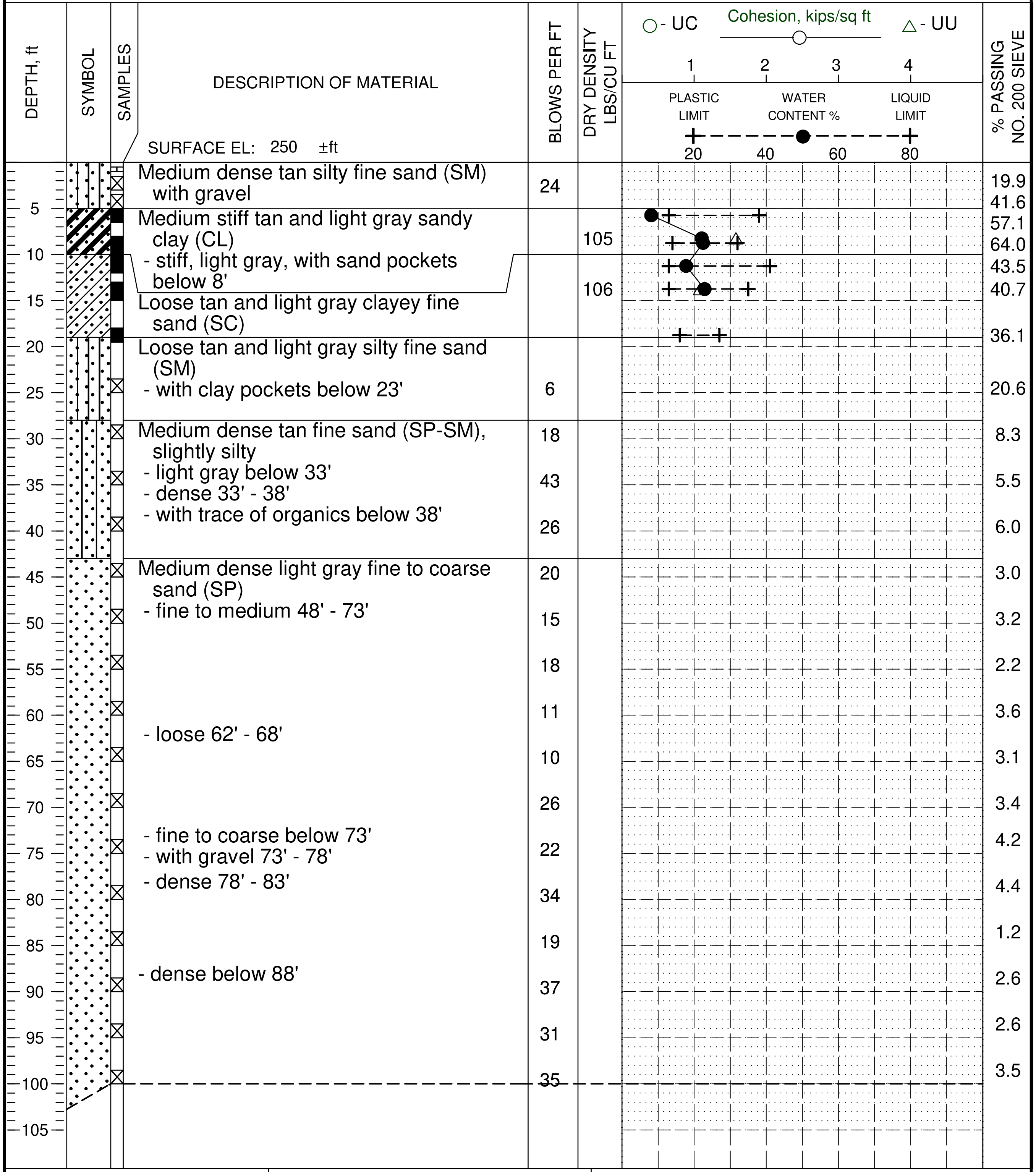
FIGURE 2

LOG OF BORING NO. S8-1

ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 50', then rotary wash to completion.

LOCATION: Sta. 815+16 (Approximate)
+/- 56' Right of Construction C/L



DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
Medium dense tan silty fine sand (SM) with gravel	24		19.9 41.6
Medium stiff tan and light gray sandy clay (CL) - stiff, light gray, with sand pockets below 8'		105	57.1 64.0
Loose tan and light gray clayey fine sand (SC)		106	43.5 40.7 36.1
Loose tan and light gray silty fine sand (SM) - with clay pockets below 23'	6		20.6
Medium dense tan fine sand (SP-SM), slightly silty - light gray below 33' - dense 33' - 38' - with trace of organics below 38'	18 43 26		8.3 5.5 6.0
Medium dense light gray fine to coarse sand (SP) - fine to medium 48' - 73' - loose 62' - 68' - fine to coarse below 73' - with gravel 73' - 78' - dense 78' - 83' - dense below 88'	20 15 18 11 10 26 22 34 19 37 31 35		3.0 3.2 2.2 3.6 3.1 3.4 4.2 4.4 1.2 2.6 2.6 3.5

BORING DEPTH: 100 ft

DATE: 08/11/20 &
08/18/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 34.88" - W 90° 51' 48.15"

GROUNDWATER DATA: Free water encountered at an approximate depth of 48.5' during auger drilling. Water level remained at an approximate depth of 48.5' after about 30 minutes.

200518 1/27/2021 10:29:31 AM

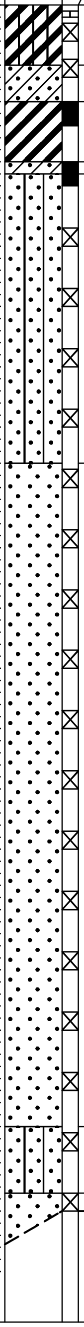
LOG OF BORING NO. S8-2

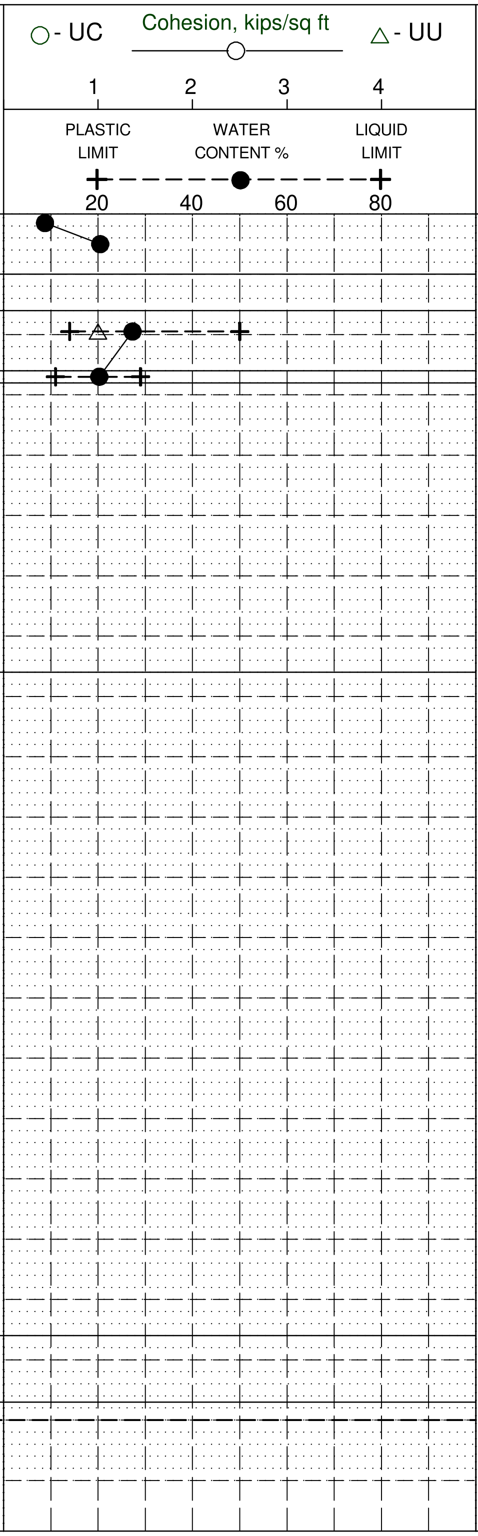
ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger to 55', then rotary wash to completion.

LOCATION: Sta. 817+21 (Approximate)
+/- 13' Right of Construction C/L

SURFACE EL: 251 ±ft





DEPTH, ft	DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
	Very stiff tan silty clay (CL), slightly sandy	9		
5	- medium stiff, with sand pockets below 1'	8		
10	Loose tan clayey fine sand (SC)		101	
	Stiff tan and light gray clay (CH), slightly sandy			35.7
15				
20	Medium dense tan and light gray clayey fine sand (SC)	22		6.8
25	Medium dense tan and light gray fine sand (SP-SM), slightly silty - with clay pockets to 20' - light gray below 23'	13		
30		14		6.4
35		20		6.9
40	Medium dense light gray fine sand (SP)	25		4.4
45	- fine to medium 43' - 48'	28		1.7
50	- fine to coarse, with trace of gravel below 48'	19		2.7
55	- dense 53' - 58'	33		1.0
60		22		2.4
65		22		2.8
70		20		2.7
75		17		2.2
80		20		3.2
85		11		2.3
90	- dense below 88'	33		3.4
95	Very dense tan fine to coarse sand (SP-SM), slighlty silty with gravel	88		5.7
100	Dense gray fine sand (SP) with trace of gravel	39		3.3
105				

BORING DEPTH: 100 ft

DATE: 08/19/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 35.01" - W 90° 51' 45.74"

GROUNDWATER DATA: Free water encountered at an approximate depth of 49' during auger drilling. Water level remained at an approximate depth of 49' after about 15 minutes.

LOG OF BORING NO. S8-ARDOT-1

ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger

LOCATION: Sta. 815+07 (Approximate)
14' Left of Construction C/L

SURFACE EL: 251.2 ±ft

○ - UC Cohesion, kips/sq ft △ - UU
1 2 3 4

PLASTIC LIMIT + WATER CONTENT % ● LIQUID LIMIT +
20 40 60 80

DEPTH, ft	DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
5	Loose brown sand (SP-SM), slightly silty, with trace of gravel	7		
10	Stiff gray sandy clay (CL)	9		
15	Loose brown sand (SP-SM), slightly silty	10		
20	- medium dense below 20'	17		
25		11		
30	Medium dense brown sand (SP)	26		
35	- very dense 35' - 40'	54		
40	- dense, with trace of gravel 40' - 45'	35		
45	- with gravel 45' - 50' - medium dense below 45'	24		
50		22		
55	- with trace of gravel below 55'	25		
60	Medium dense brown sand (SP-SM), slightly silty, with trace of gravel	20		
65	Medium dense brown sand (SP) with trace of gravel	29		
70		26		
75	- dense 75' - 90'	38		
80		31		
85		39		
90	- very dense 90' - 95'	66		
95	- dense below 95'	38		
100		47		
105				
110				

BORING DEPTH: 101.5 ft

DATE: 10/08/19

COMMENTS: SPT performed with automatic hammer. A hammer energy correction factor of 1.3 applies.

-

GROUNDWATER DATA: No free water encountered during auger drilling.

FIGURE 5

LOG OF BORING NO. S8-ARDOT-2

ARDOT SR230
ALICIA TO BONO, ARKANSAS

TYPE: Hollow-stem auger

LOCATION: Sta. 817+20 (Approximate)
4' Right of Construction C/L

SURFACE EL: 251.3 ±ft

○ - UC Cohesion, kips/sq ft △ - UU
1 2 3 4

PLASTIC LIMIT WATER CONTENT % LIQUID LIMIT
20 40 60 80

DEPTH, ft	DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
5	Medium stiff brown sandy clay (CL)	8		40.0
10	Medium stiff brown and gray clay (CH) with sand	5		59.0
15	Loose brown clayey sand (SC)	6		30.0
20	Medium dense brown poorly graded sand (SP-SM), slightly silty	18		9.0
25	Loose brown clayey sand (SC)	10		16.0
30	Medium dense brown sand (SP-SM), slightly silty	11		6.0
35		21		5.0
40	- well graded, with trace of gravel below 40'	18		5.0
45	Dense brown well graded sand (SW) with trace of gravel	39		3.0
50	Medium dense brown poorly graded sand (SP) with trace of gravel	29		3.0
55	Medium dense brown well graded sand (SW) with gravel	20		4.0
60	- with trace of gravel below 60'	23		4.0
65	Dense brown and gray clayey sand (SC)	33		42.0
70	Medium dense brown well graded sand (SW) with gravel	22		3.0
75	Dense brown poorly graded sand (SP) - with gravel to 80'	36		4.0
80	- with trace of gravel 80' - 85' - medium dense, poorly graded below 80'	14		2.0
85	- with gravel below 85'	13		2.0
90	Very dense brown poorly graded sand (SP-SM), slightly silty - with gravel to 95'	70		6.0
95		57		5.0
100	- dense 100' - 105' - with trace of gravel below 100'	39		4.0
105	- very dense 105' - 110'	76		6.0
110	- dense below 110'	41		5.0
115				

BORING DEPTH: 111.5 ft

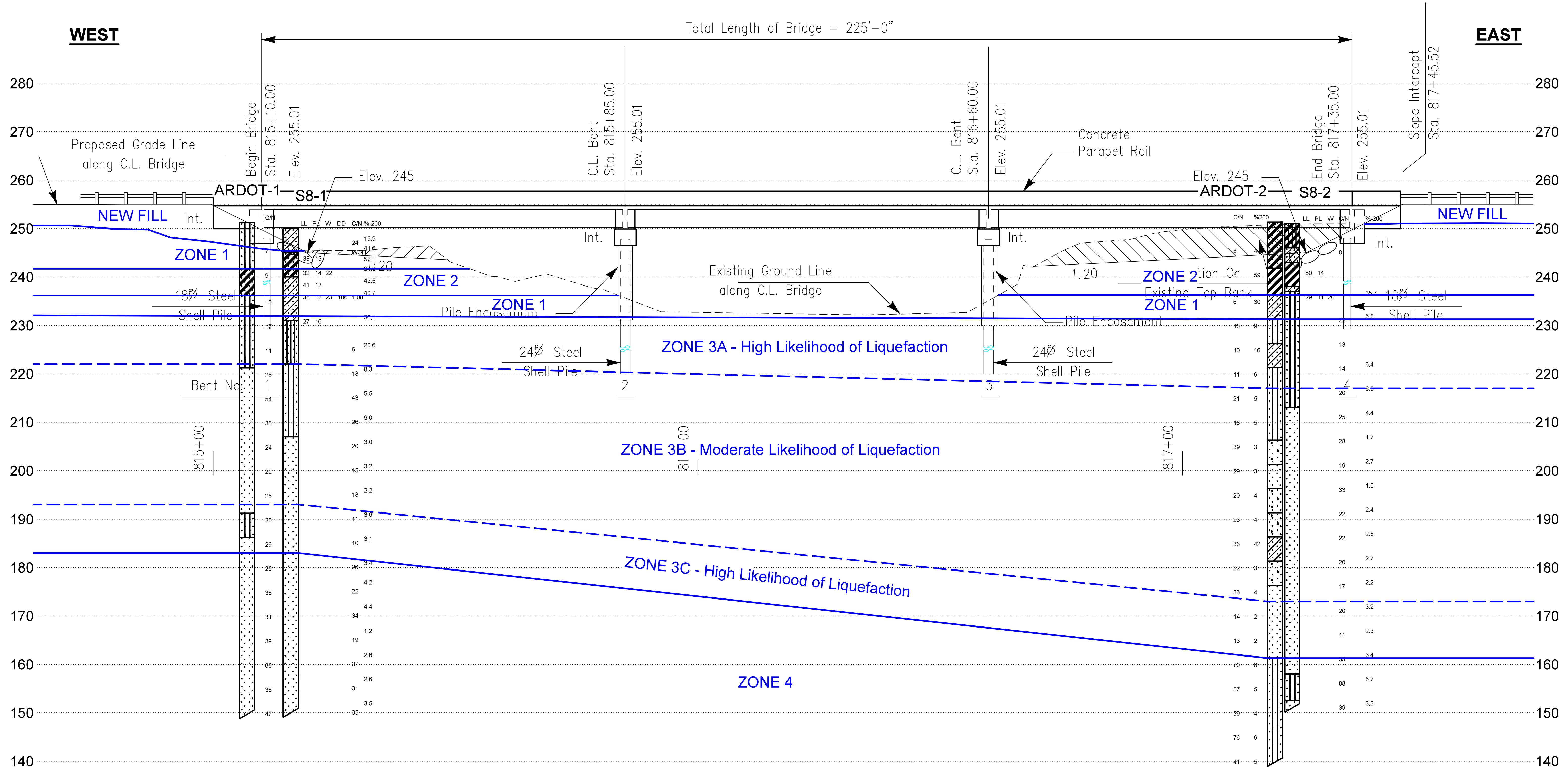
DATE: 10/15/19 & 10/16/19

COMMENTS: SPT performed with automatic hammer. A hammer energy correction factor of 1.3 applies.

-

GROUNDWATER DATA: No free water encountered during auger drilling.

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



0 10 20 40

SCALE: 1" = 20'

ZONE 1
Loose to medium dense clayey sand (SC), silty sand (SM), & sand (SP) with silt

ZONE 2
Medium stiff to stiff sandy clay (CL) & clay (CH) with sand

ZONE 3
Loose to dense sand (SP), sand (SP) with silt, and sand (SP) with gravel

ZONE 4
Medium dense to very dense sand (SP) & sand (SP) with gravel

Note: The SPT blow count "N" values are raw values. They have not been corrected for hammer energy. ARDOT indicated that a hammer energy correction factor of 1.3 applies to borings ARDOT-1 & ARDOT-2. A hammer energy correction factor of 1.36 applies to borings S8-1 & S8-2.

Soil Profile

SITE 8
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO.	200518	SCALE: AS SHOWN	FIGURE 7

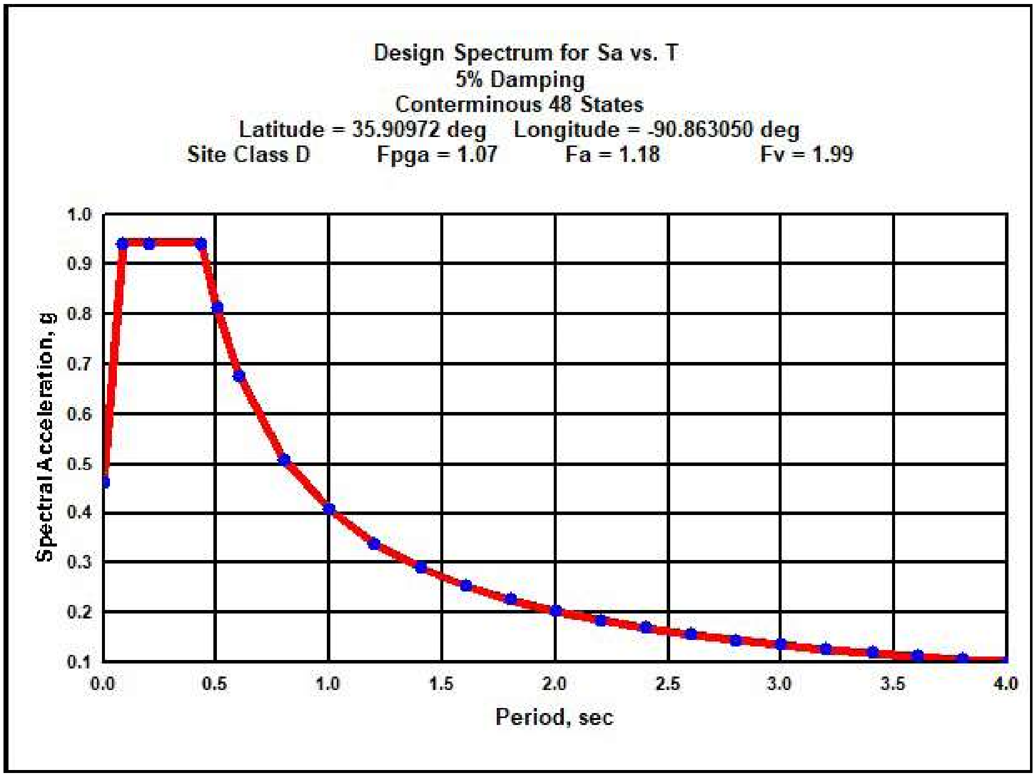


Figure 8 - Seismic Design Spectrum for Sa vs. T

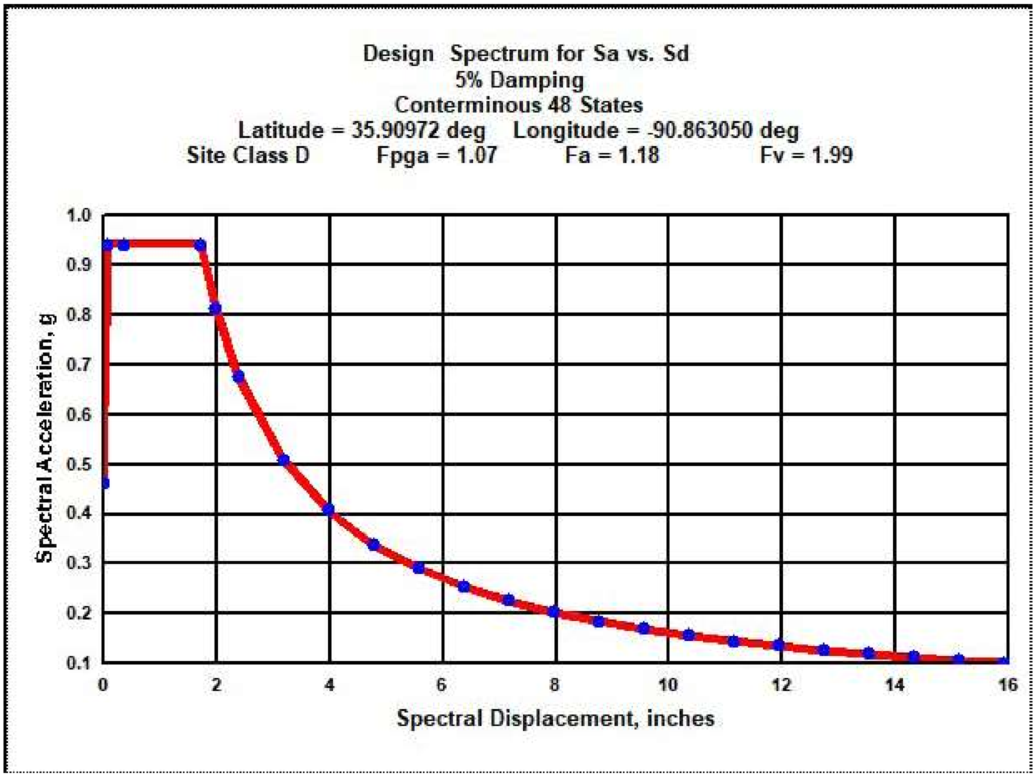


Figure 9 - Seismic Design Spectrum for Sa vs. Sd

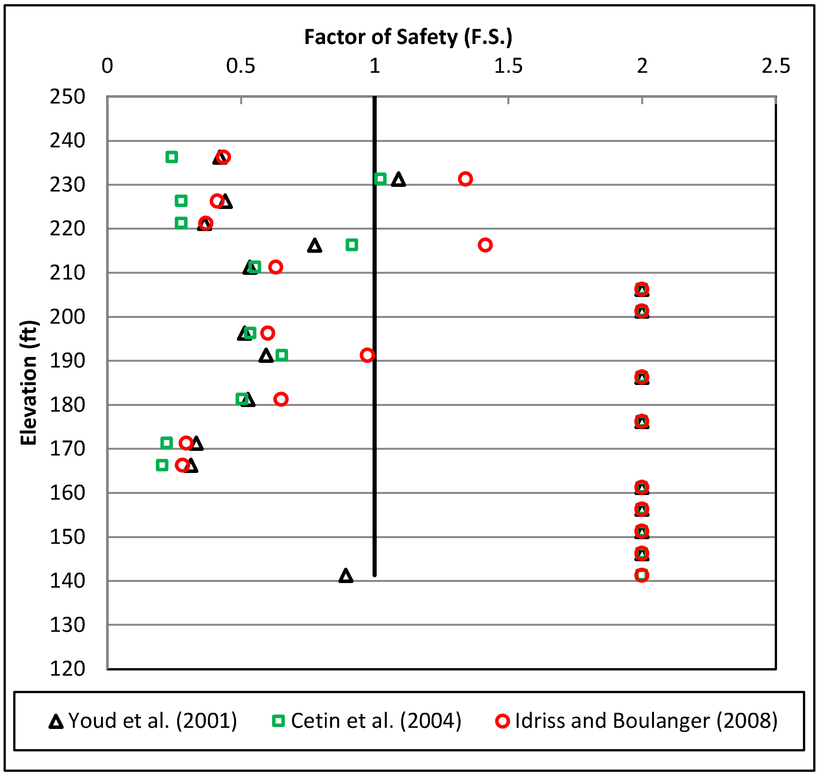


Figure 10 - Liquefaction Triggering FS Values for ARDOT-2 (Top of Boring at EL 251.3 ft)

Figure 11 - Liquefaction Triggering FS Values for S8-1 (Top of Boring at EL 250.0 ft)

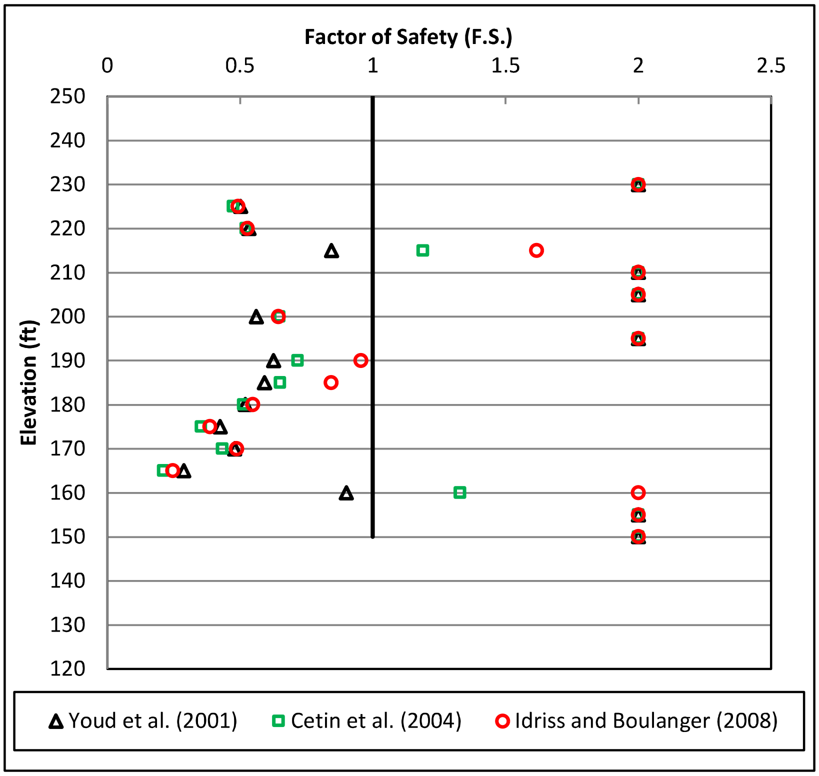


Figure 12 - Liquefaction Triggering FS Values for S8-2 (Top of Boring at EL 250.0 ft)

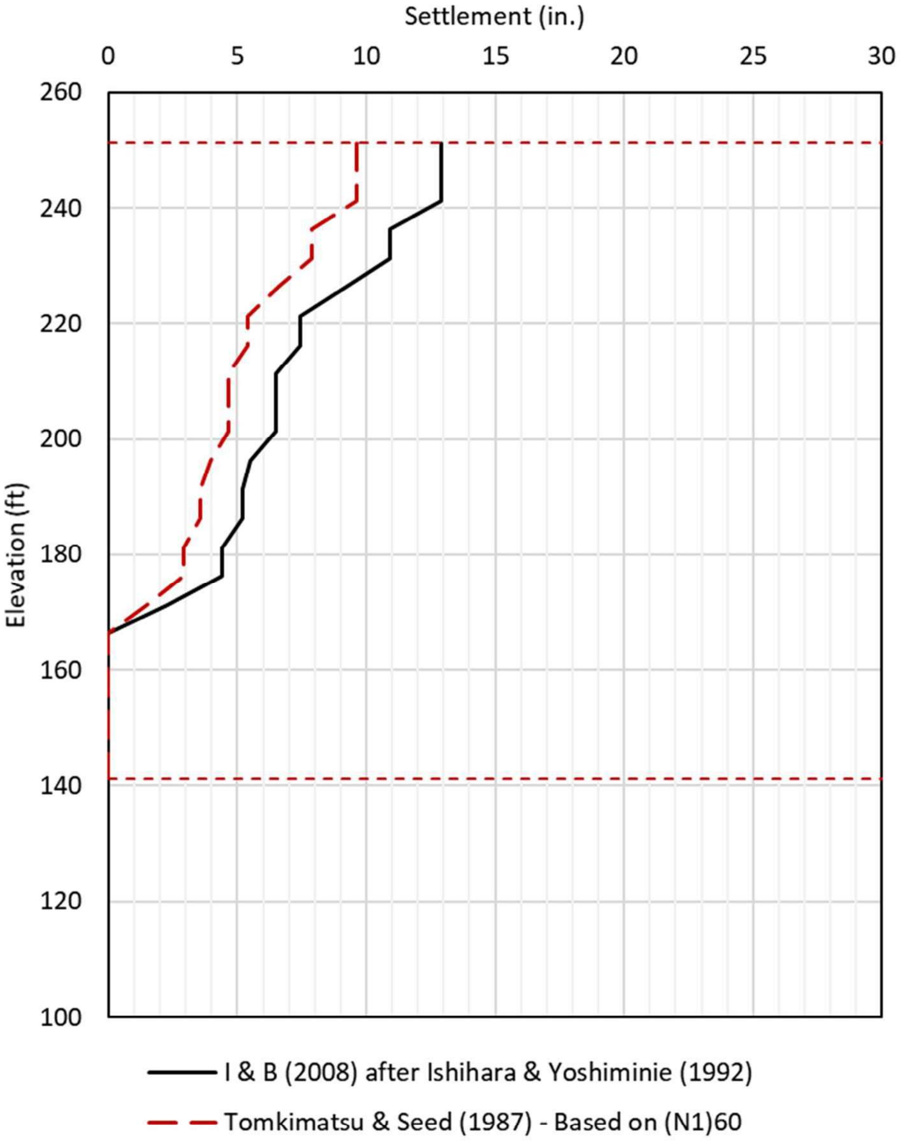


Figure 13 – Seismic Compression for ARDOT-2 (Top of Boring at EL 251.3 ft)

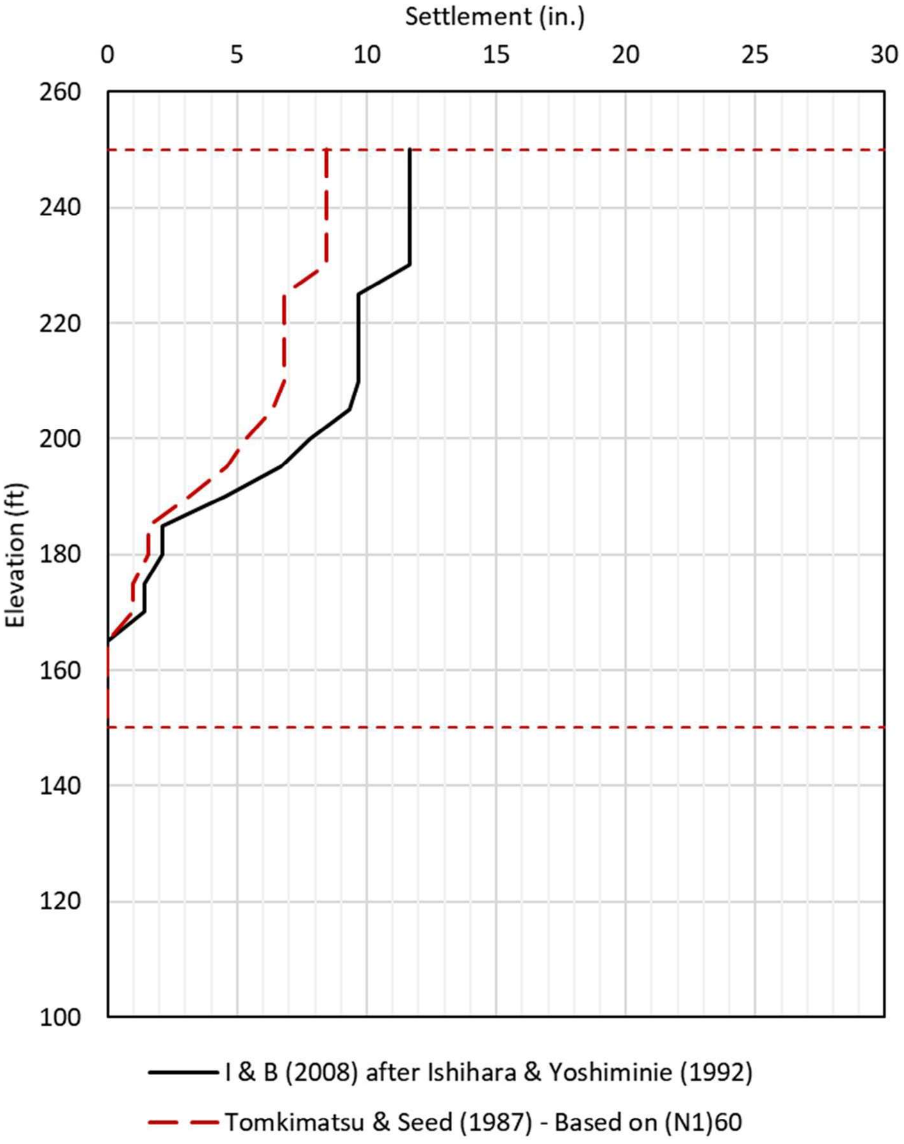


Figure 14 - Seismic Compression for S8-1 (Top of Boring at EL 250.0 ft)

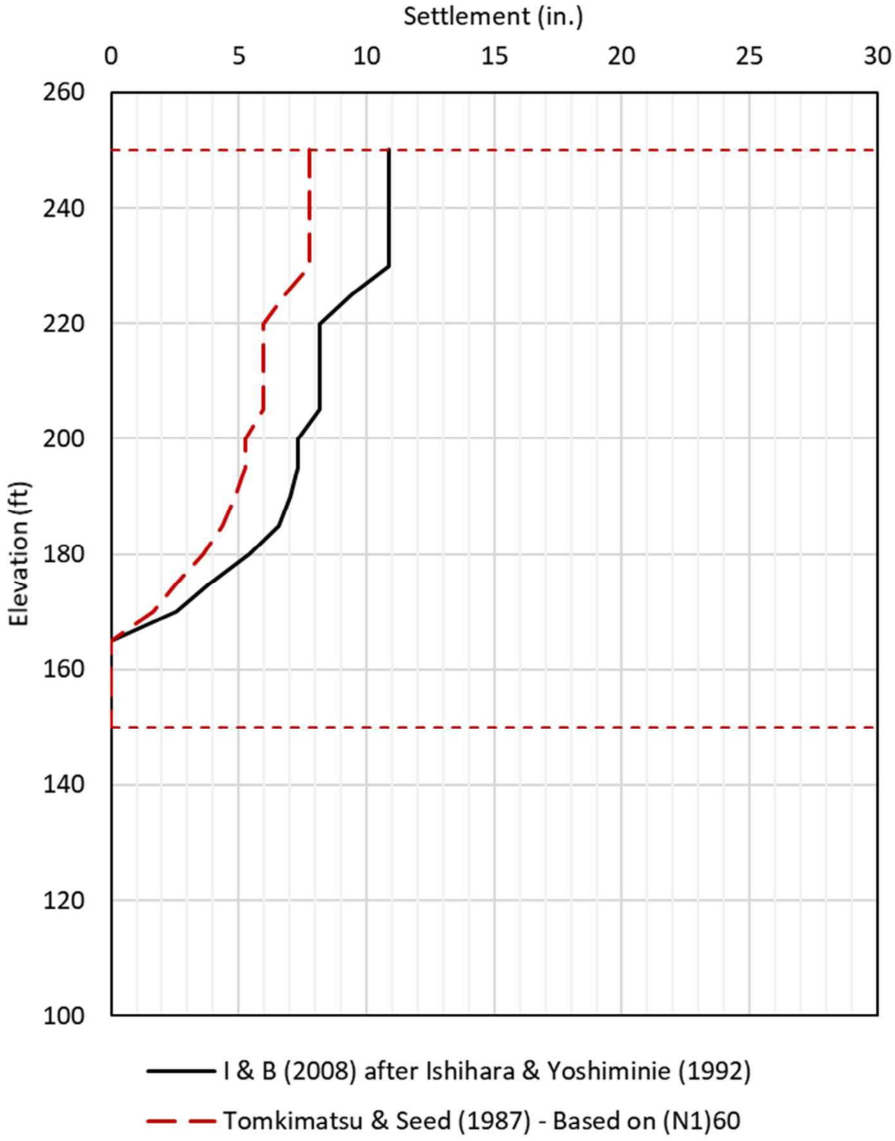
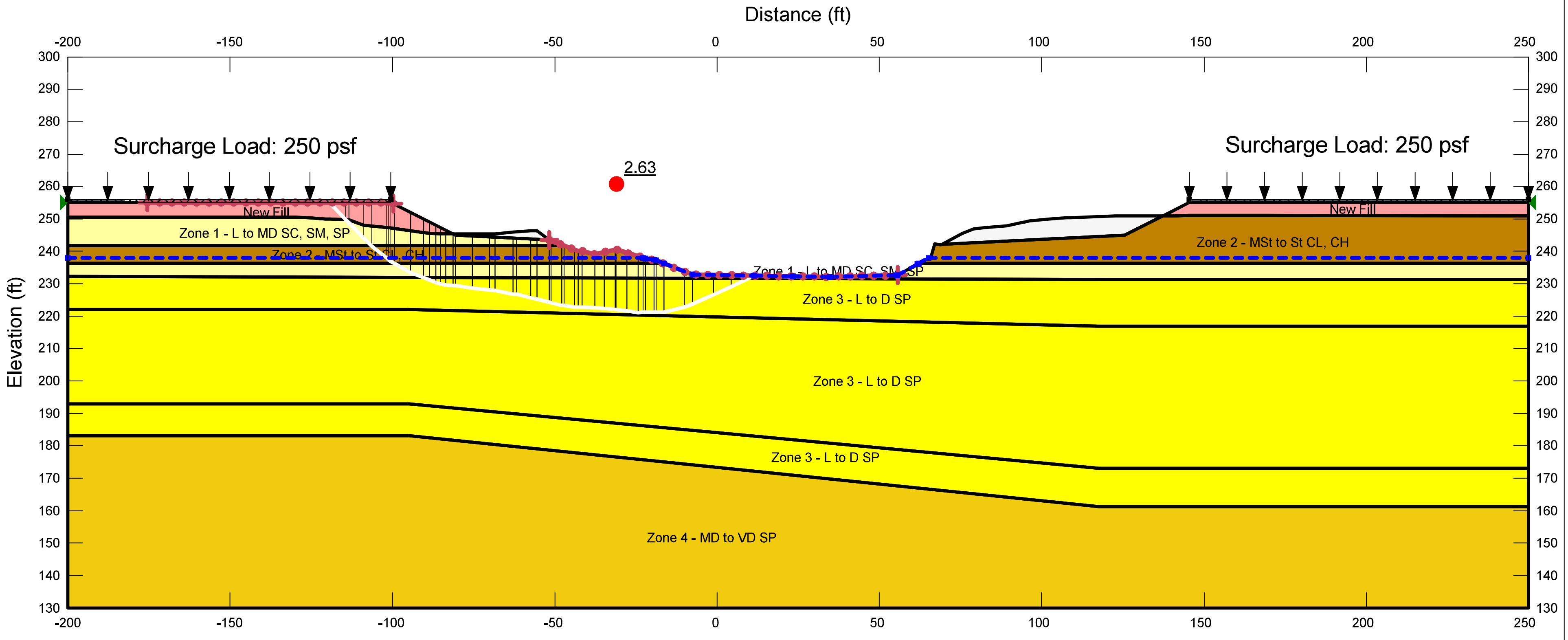


Figure 15 - Seismic Compression for S8-2 (Top of Boring at EL 250.0 ft)



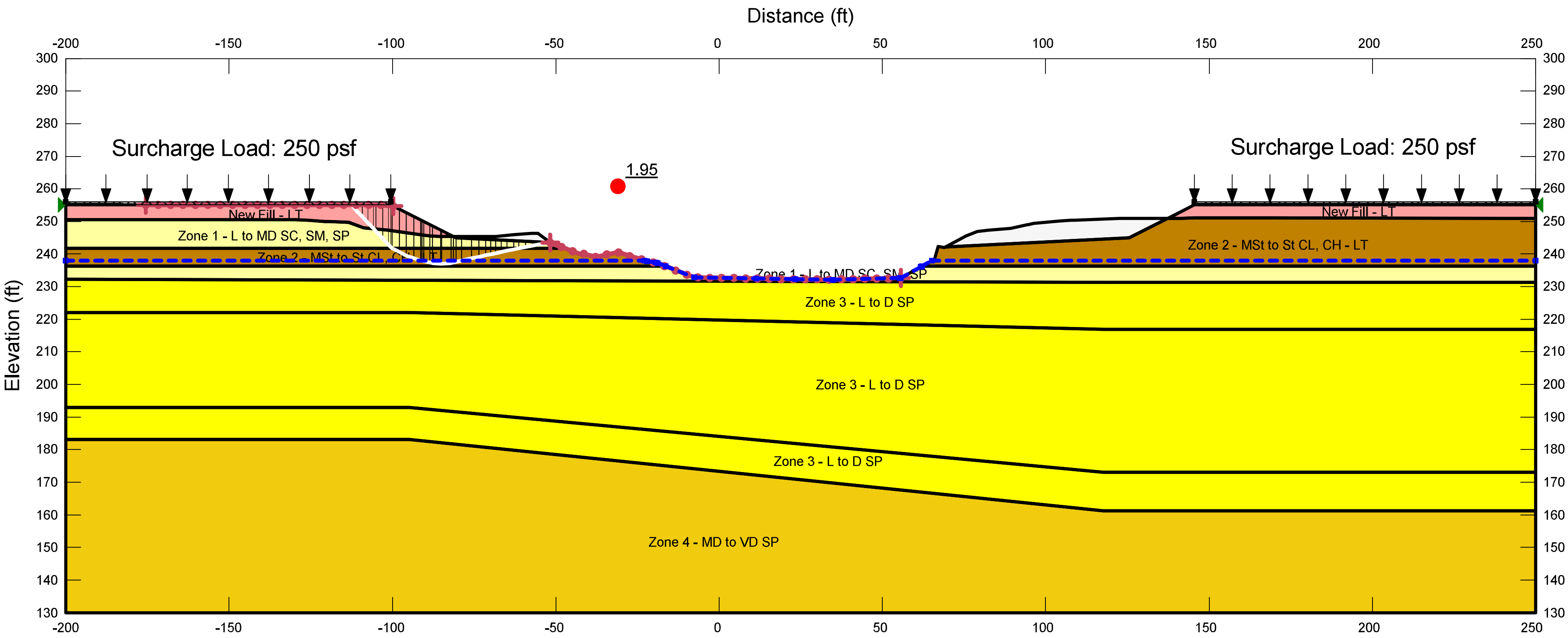
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH	Mohr-Coulomb	125	1,000	0
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟧	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

West Abutment Spill-Through
End of Construction

SR 230 Site 8
Craighead County, AR

Figure 16

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



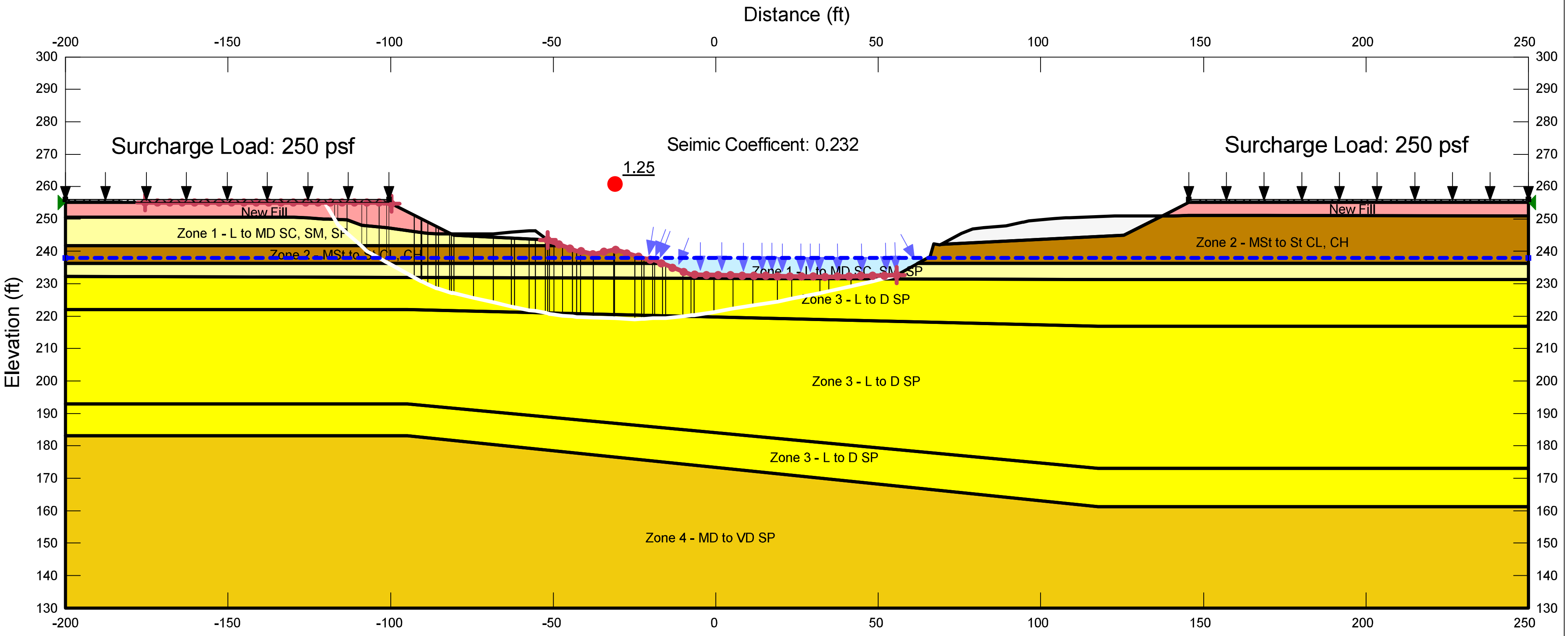
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH - LT	Mohr-Coulomb	125	50	21
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟧	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

West Abutment Spill-Through
Long Term

SR 230 Site 8
Craighead County, AR

Figure 17

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



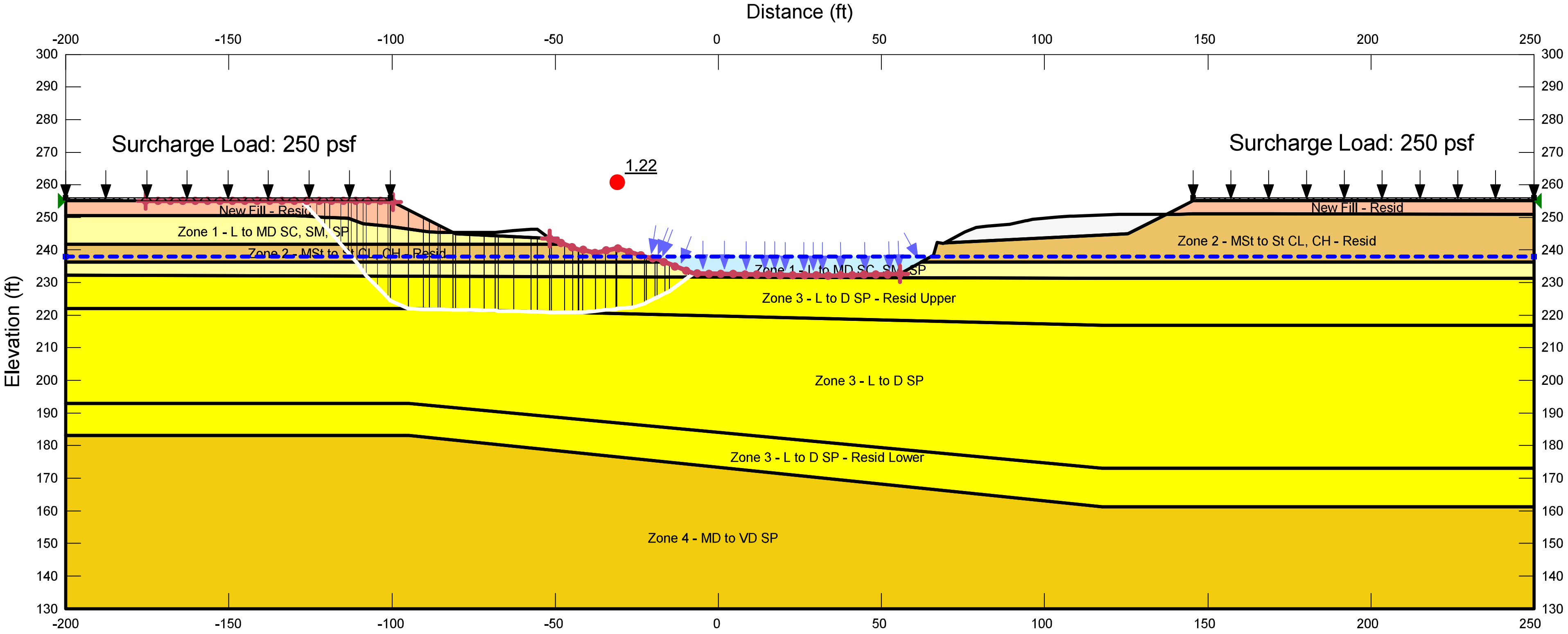
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
▨	New Fill	Mohr-Coulomb	120	1,500	0
▨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
▨	Zone 2 - MSt to St CL, CH	Mohr-Coulomb	125	1,000	0
▨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
▨	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

West Abutment Spill-Through
Pseudostatic

SR 230 Site 8
Craighead County, AR

Figure 18

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



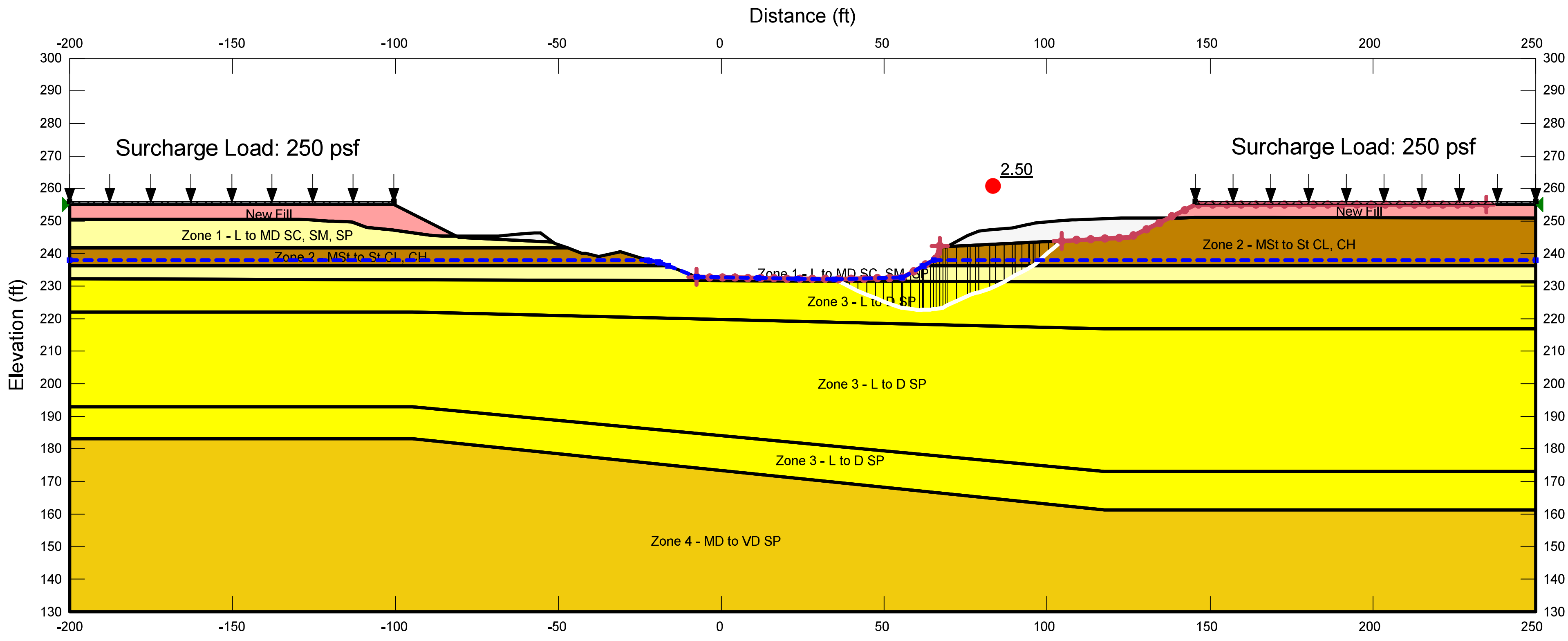
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH - Resid	Mohr-Coulomb	125	800	0
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟨	Zone 3 - L to D SP - Resid Lower	Mohr-Coulomb	120	300	0
🟨	Zone 3 - L to D SP - Resid Upper	Mohr-Coulomb	120	300	0
🟨	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

West Abutment Spill-Through
Post-Seismic

SR 230 Site 8
Craighead County, AR

Figure 19

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



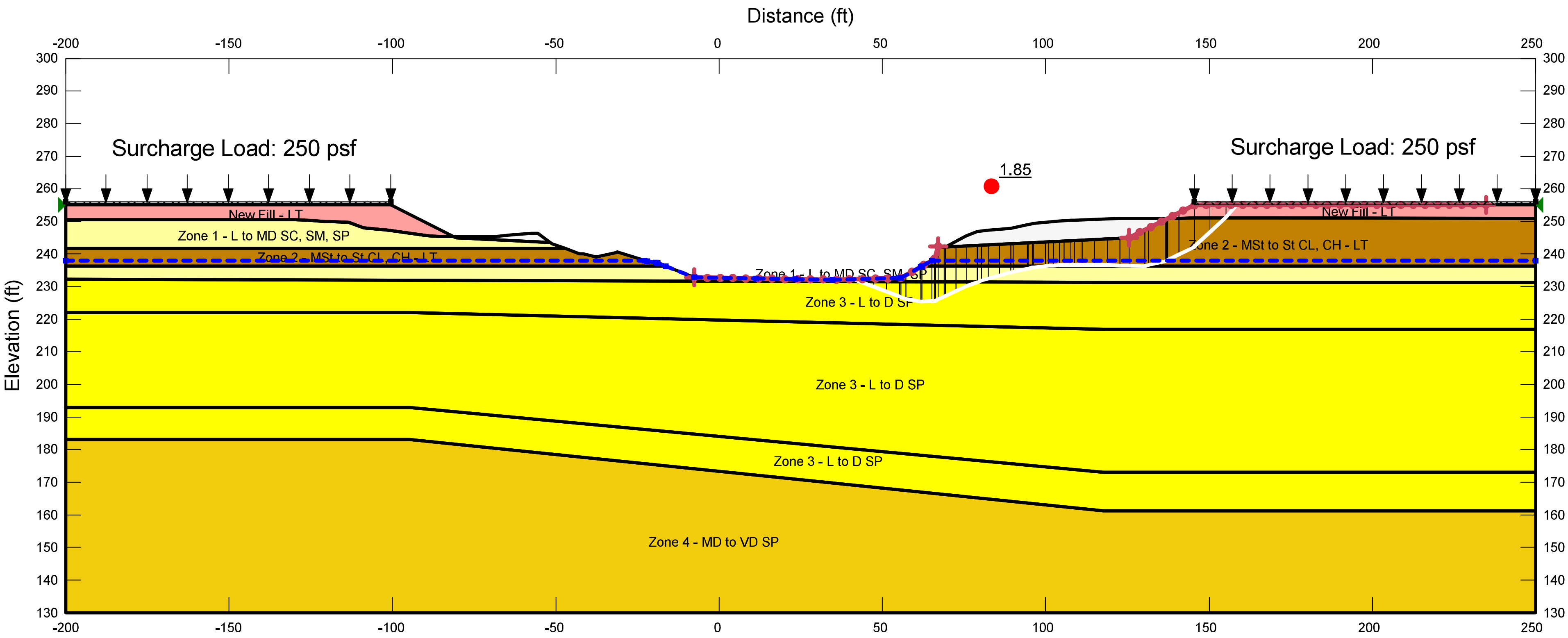
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH	Mohr-Coulomb	125	1,000	0
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟧	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

East Abutment Spill-Through
End of Construction

SR 230 Site 8
Craighead County, AR

Figure 20

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



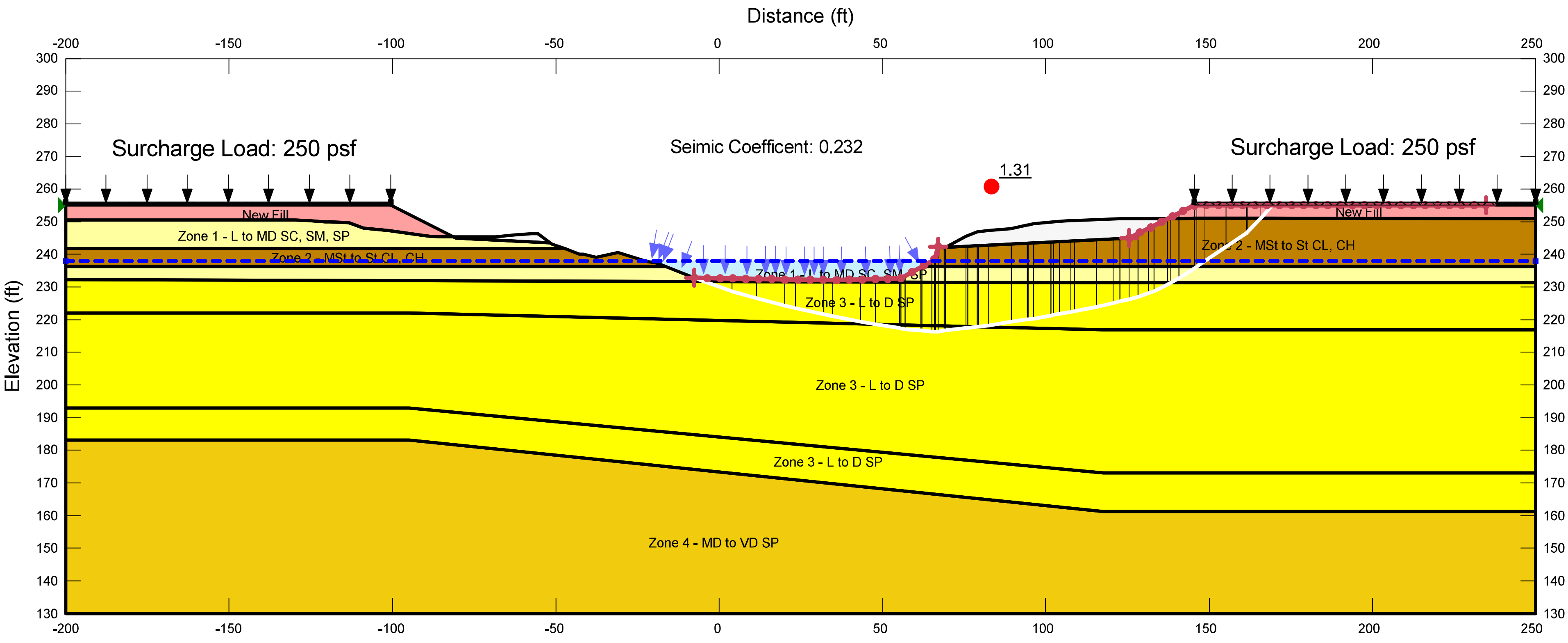
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH - LT	Mohr-Coulomb	125	50	21
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟧	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

East Abutment Spill-Through
Long Term

SR 230 Site 8
Craighead County, AR

Figure 21

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



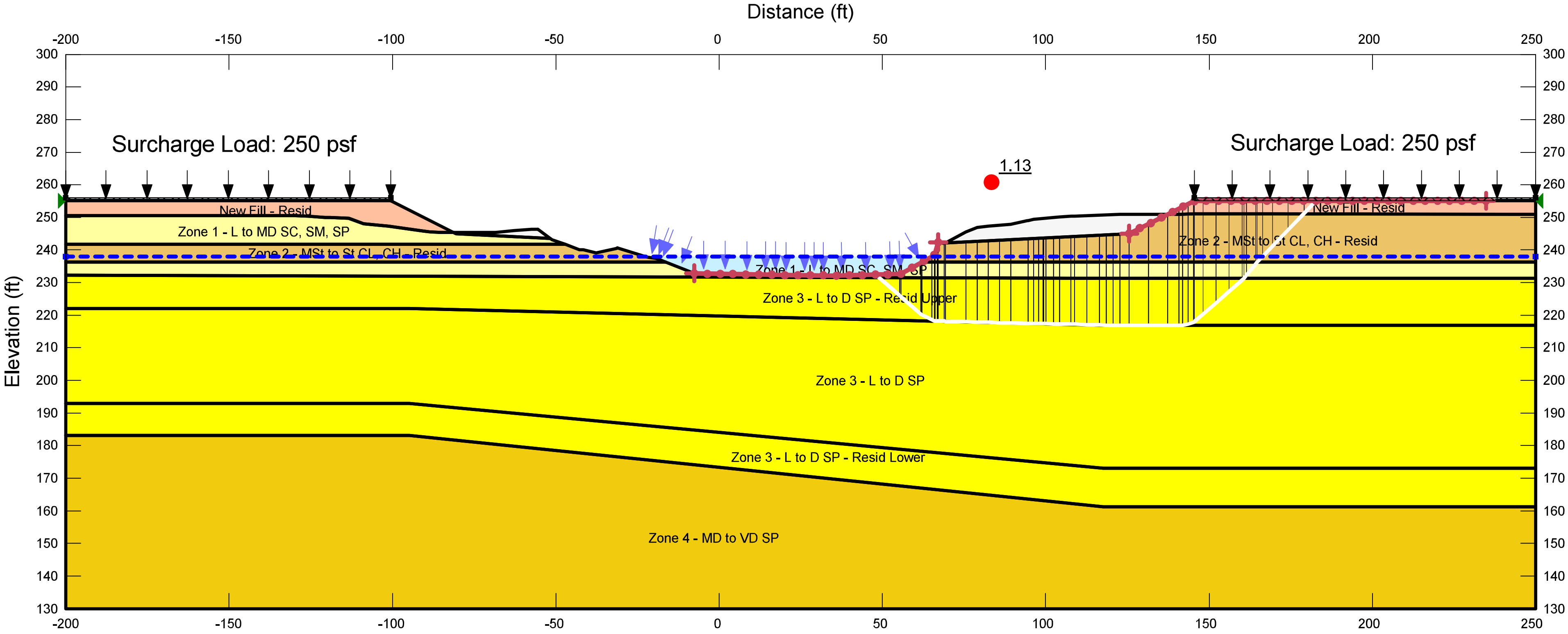
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH	Mohr-Coulomb	125	1,000	0
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟧	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

East Abutment Spill-Through
Pseudostatic

SR 230 Site 8
Craighead County, AR

Figure 22

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



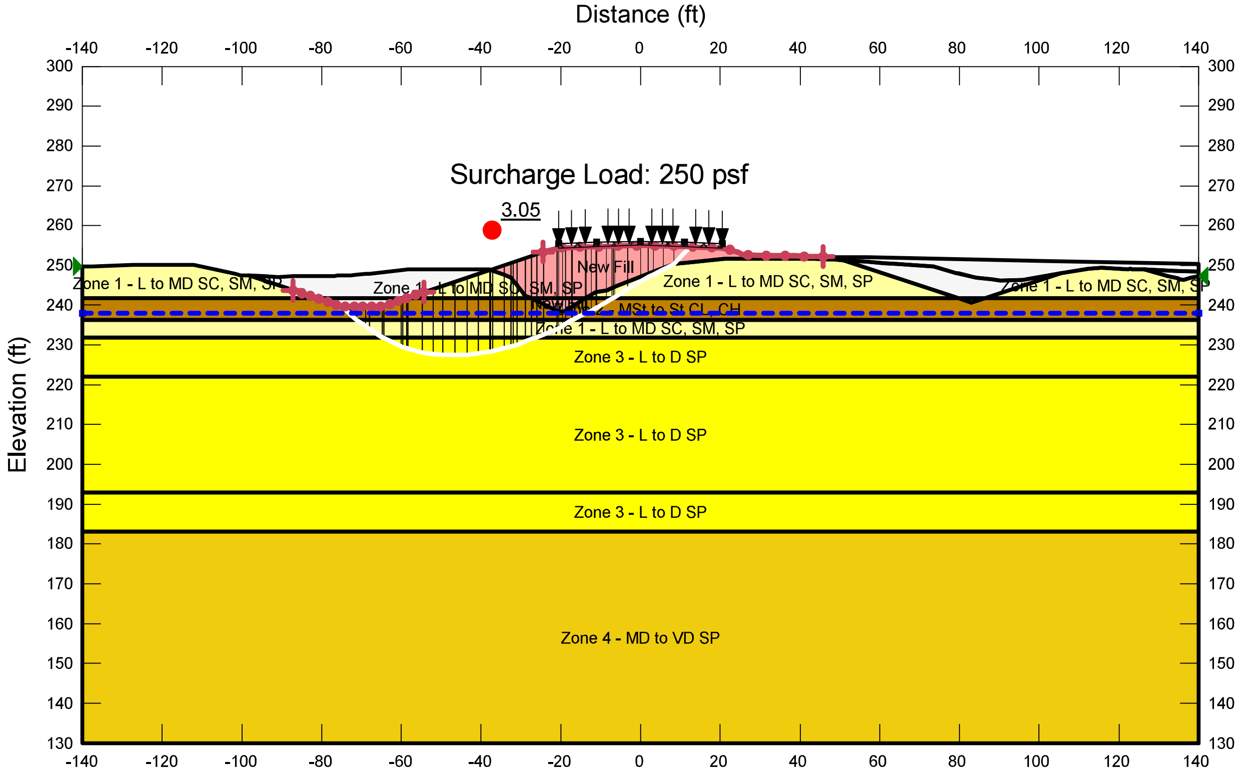
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH - Resid	Mohr-Coulomb	125	800	0
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟨	Zone 3 - L to D SP - Resid Lower	Mohr-Coulomb	120	300	0
🟨	Zone 3 - L to D SP - Resid Upper	Mohr-Coulomb	120	300	0
🟨	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

East Abutment Spill-Through
Post-Seismic

SR 230 Site 8
Craighead County, AR

Figure 23

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



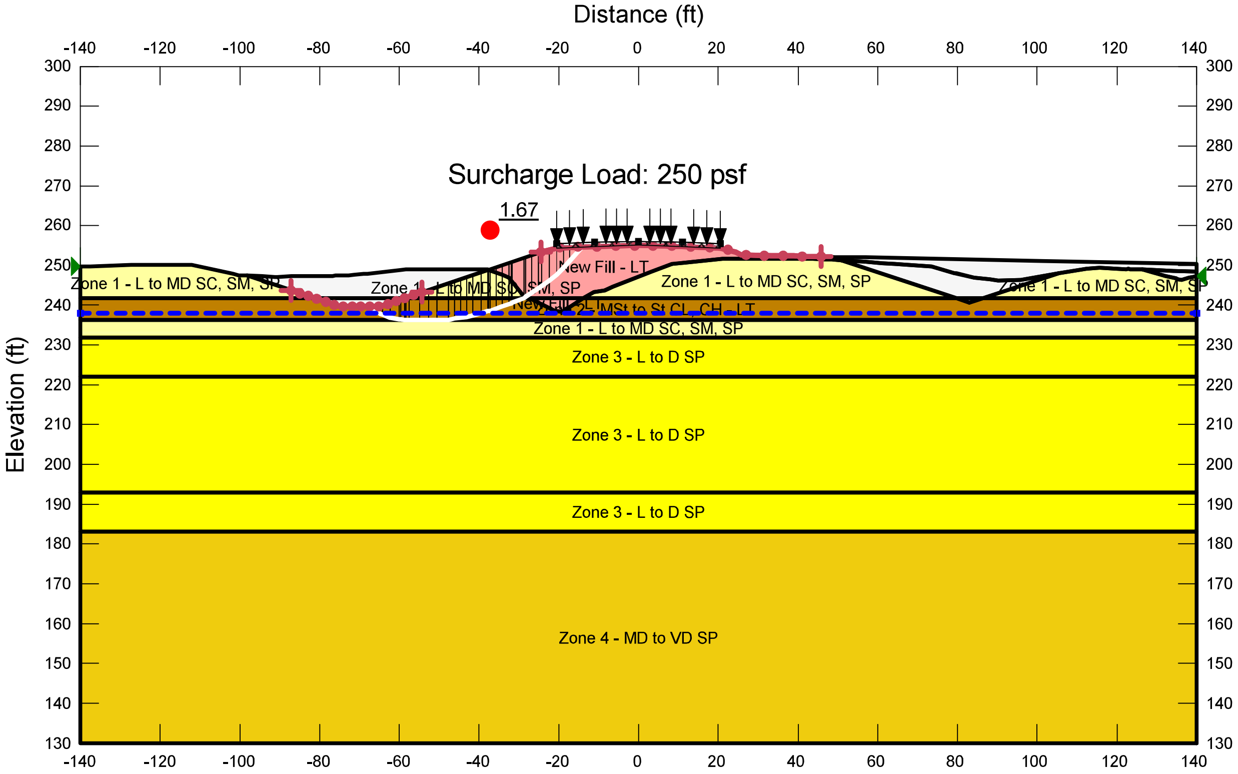
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH	Mohr-Coulomb	125	1,000	0
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟧	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

West Abutment North Side Slope
815+00
End of Construction

SR 230 Site 8
Craighead County, AR

Figure 24

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



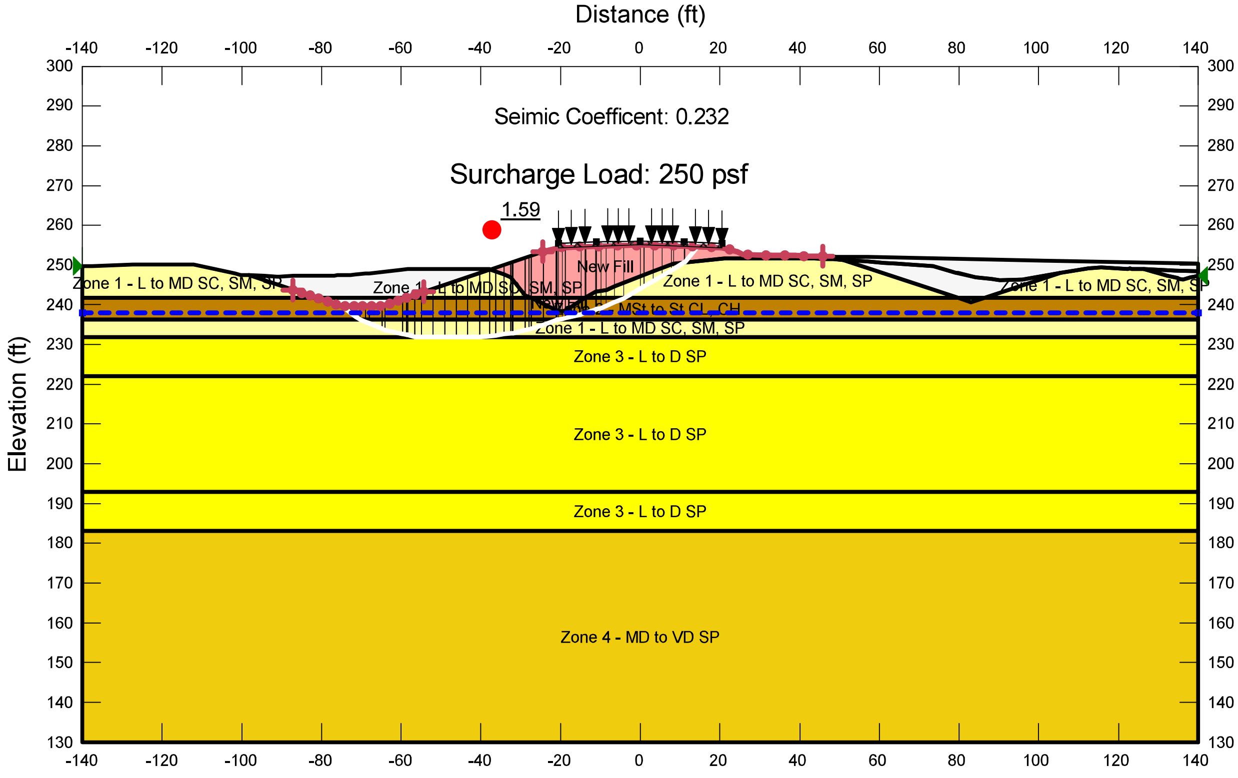
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill - LT	Mohr-Coulomb	120	50	28
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH - LT	Mohr-Coulomb	125	50	21
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟧	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

West Abutment North Side Slope
815+00
Long Term

SR 230 Site 8
Craighead County, AR

Figure 25

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



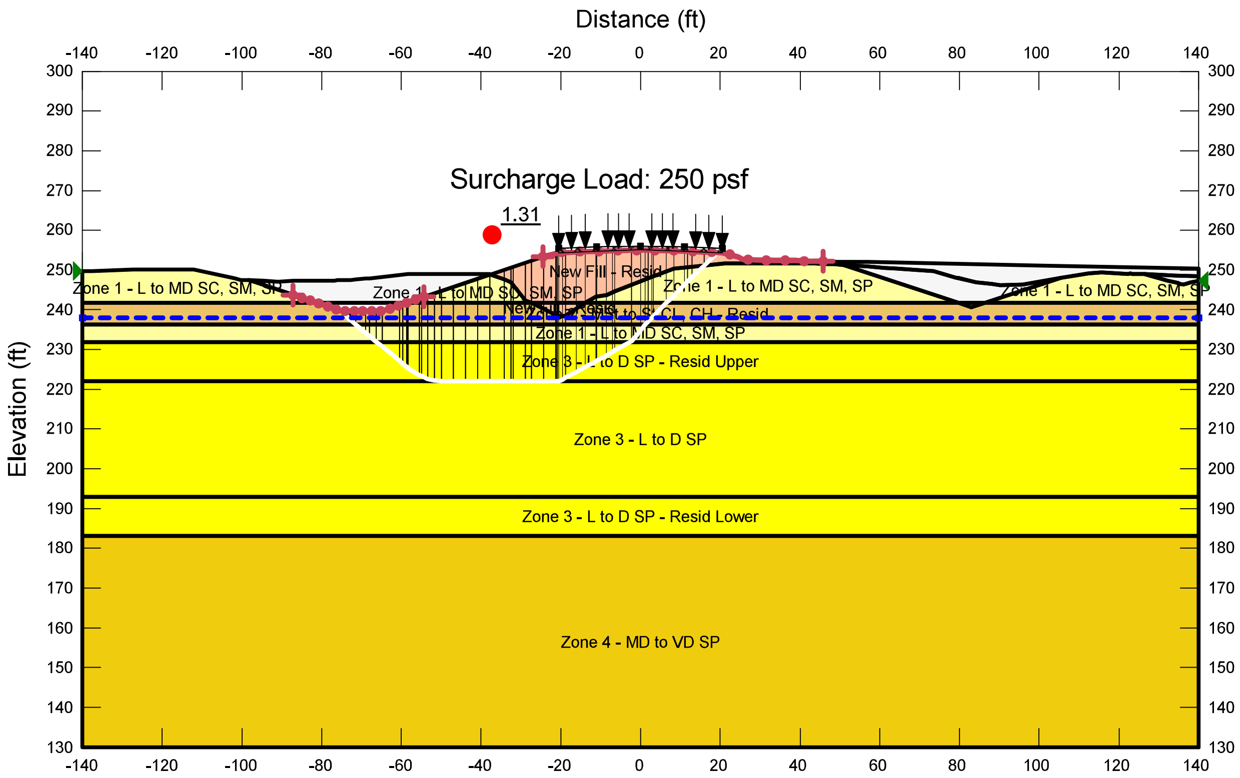
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	New Fill	Mohr-Coulomb	120	1,500	0
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH	Mohr-Coulomb	125	1,000	0
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟧	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

West Abutment North Side Slope
815+00
Pseudostatic

SR 230 Site 8
Craighead County, AR

Figure 26

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



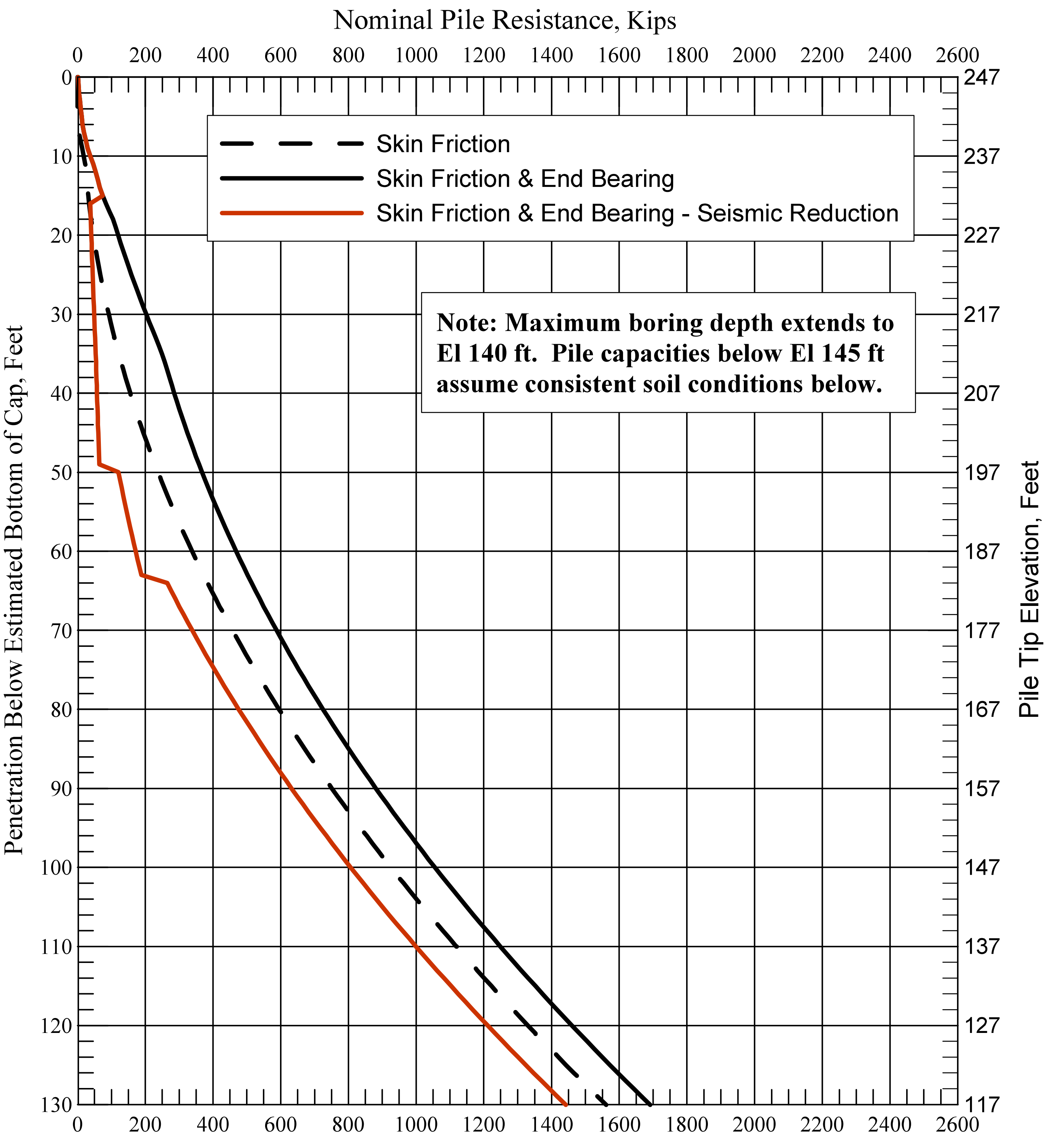
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	New Fill - Resid	Mohr-Coulomb	120	1,200	0
🟨	Zone 1 - L to MD SC, SM, SP	Mohr-Coulomb	115	0	32
🟫	Zone 2 - MSt to St CL, CH - Resid	Mohr-Coulomb	125	800	0
🟨	Zone 3 - L to D SP	Mohr-Coulomb	120	0	34
🟨	Zone 3 - L to D SP - Resid Lower	Mohr-Coulomb	120	300	0
🟨	Zone 3 - L to D SP - Resid Upper	Mohr-Coulomb	120	300	0
🟨	Zone 4 - MD to VD SP	Mohr-Coulomb	120	0	37

West Abutment North Side Slope
815+00
Post-Seismic

SR 230 Site 8
Craighead County, AR

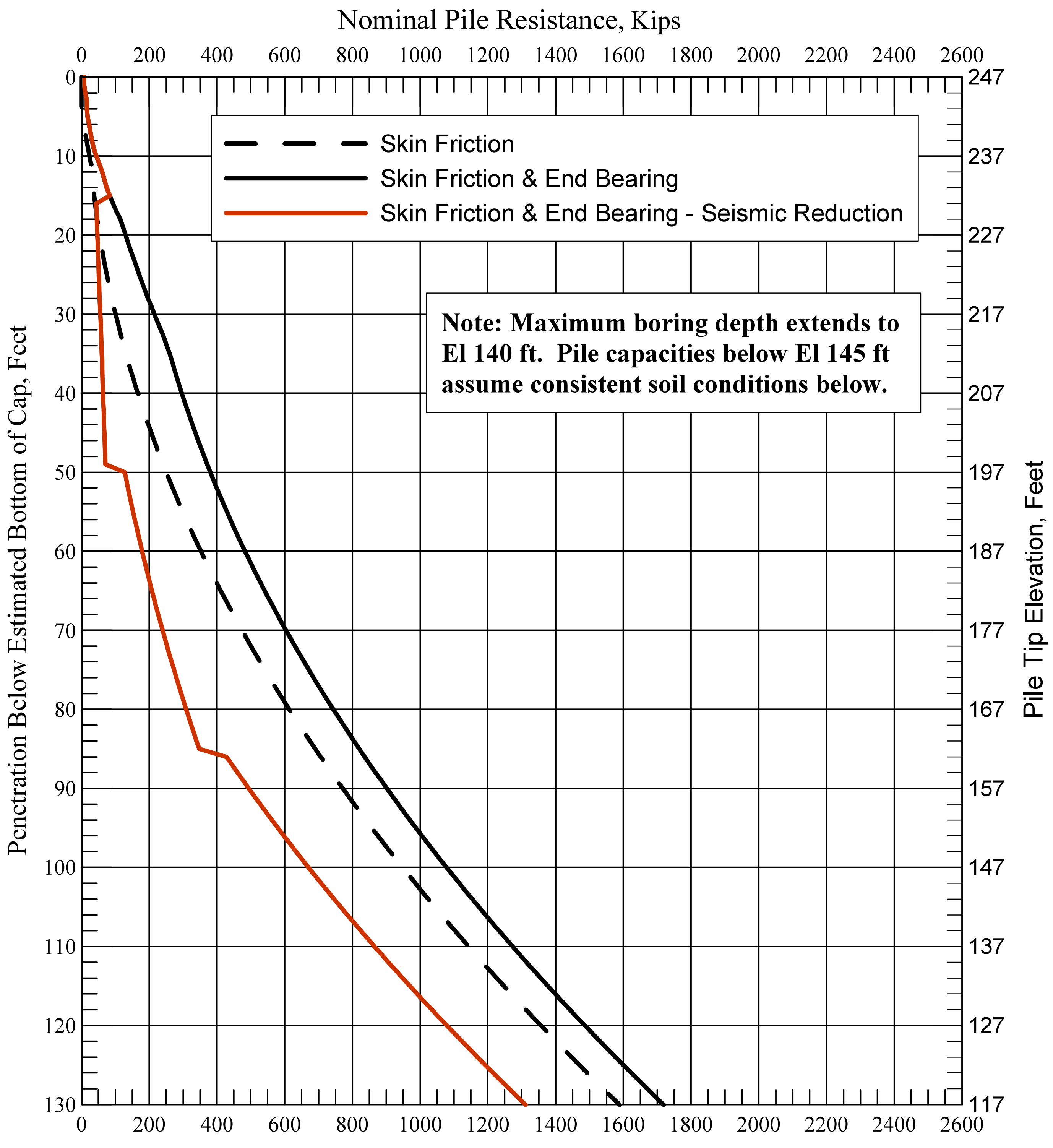
Figure 27

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS



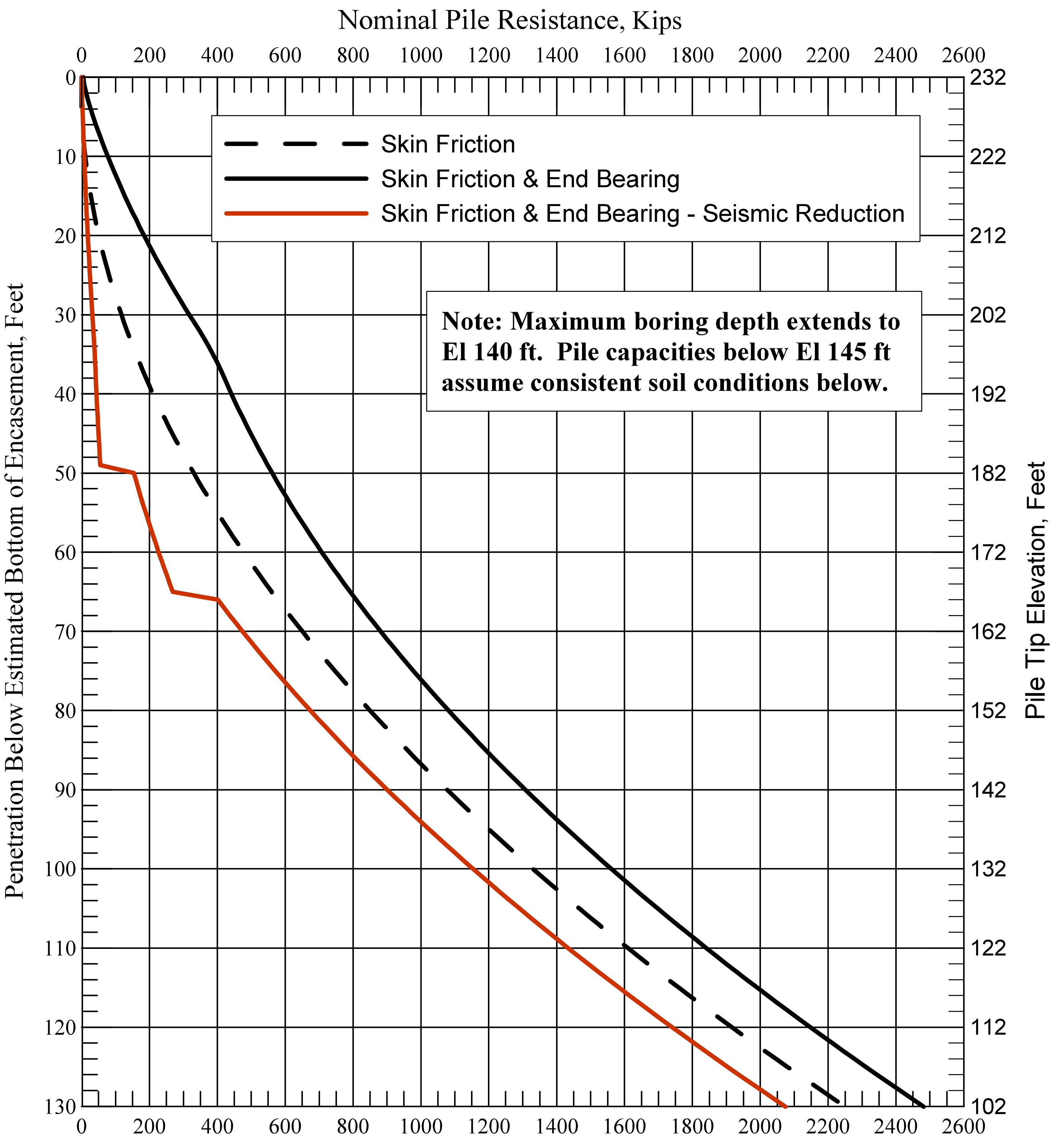
Note: Maximum boring depth extends to El 140 ft. Pile capacities below El 145 ft assume consistent soil conditions below.

NOMINAL PILE RESISTANCE CURVES
SITE 8
WEST ABUTMENT
18" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS



Note: Maximum boring depth extends to El 140 ft. Pile capacities below El 145 ft assume consistent soil conditions below.

NOMINAL PILE RESISTANCE CURVES
SITE 8
EAST ABUTMENT
18" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS



Note: Maximum boring depth extends to El 140 ft. Pile capacities below El 145 ft assume consistent soil conditions below.

NOTHING

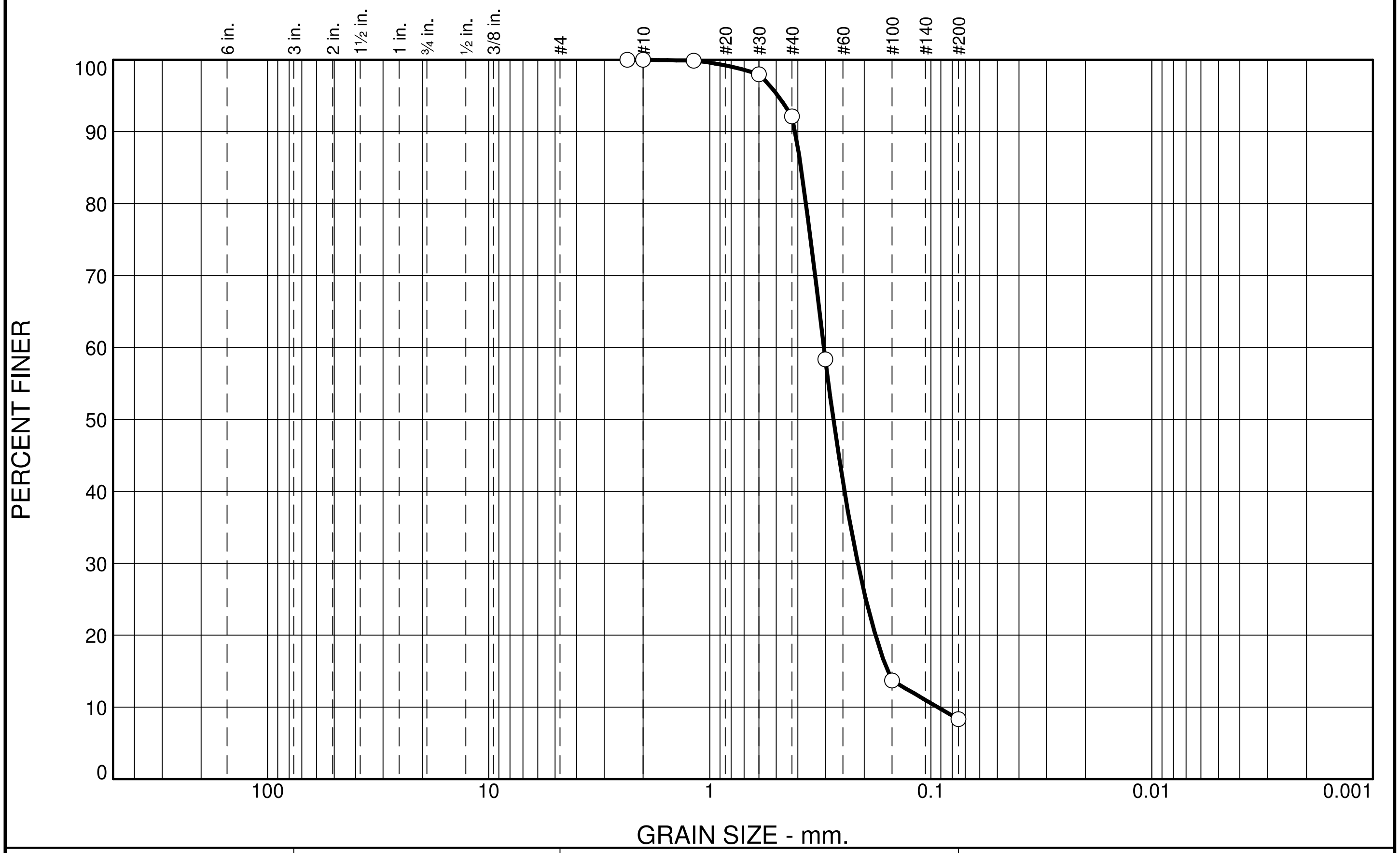
NOMINAL PILE RESISTANCE CURVES
SITE 8
INTERIOR BENTS
24" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

FIGURE 30

APPENDIX A

Particle Size Distribution Curves

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	7.9	83.8	8.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#8	100.0		
#10	100.0		
#16	99.9		
#30	98.0		
#40	92.1		
#50	58.3		
#100	13.7		
#200	8.3		

* (no specification provided)

Soil Description

Tan fine sand (SP-SM), slightly silty

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.4115 D_{85}= 0.3868 D_{60}= 0.3047
D_{50}= 0.2758 D_{30}= 0.2139 D_{15}= 0.1564
D_{10}= 0.0931 C_u= 3.27 C_c= 1.61

Classification

USCS= AASHTO=

Remarks

Source of Sample: S8-1 **Depth:** 28.5
Sample Number: 10

Date: 8/18/2020

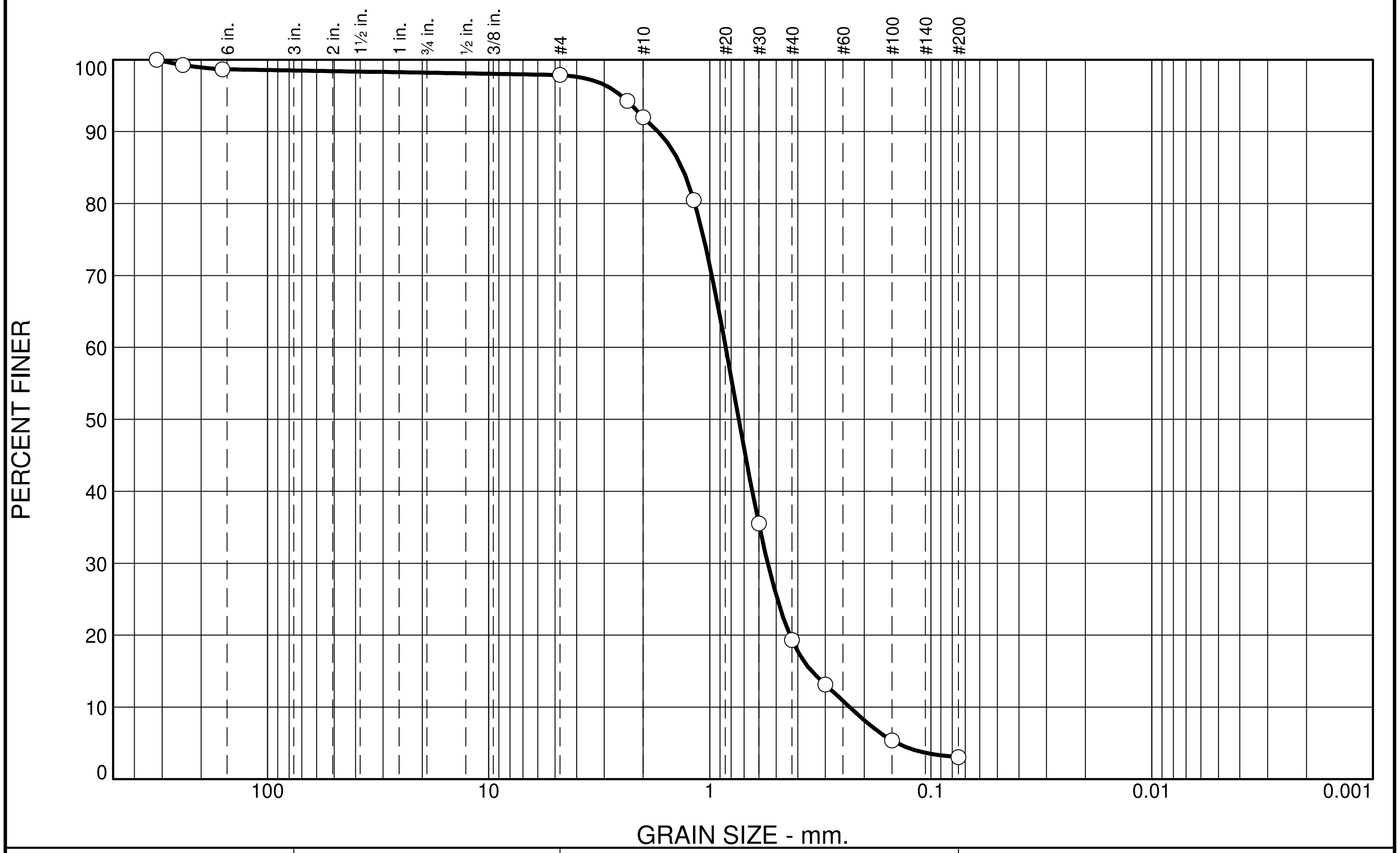
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: AW **Checked By:** ______________________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
1.5	0.3	0.3	5.9	72.7	16.3	3.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
12.5	100.0		
9.5	99.3		
6.3	98.7		
#4	97.9		
#8	94.3		
#10	92.0		
#16	80.5		
#30	35.5		
#40	19.3		
#50	13.2		
#100	5.4		
#200	3.0		

* (no specification provided)

Soil Description

Light gray fine to coarse sand (SP)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.7141 D_{85}= 1.3353 D_{60}= 0.8468
D_{50}= 0.7392 D_{30}= 0.5463 D_{15}= 0.3460
D_{10}= 0.2328 C_u= 3.64 C_c= 1.51

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S8-1 **Depth:** 43.5
Sample Number: 13

Date: 8/18/2020

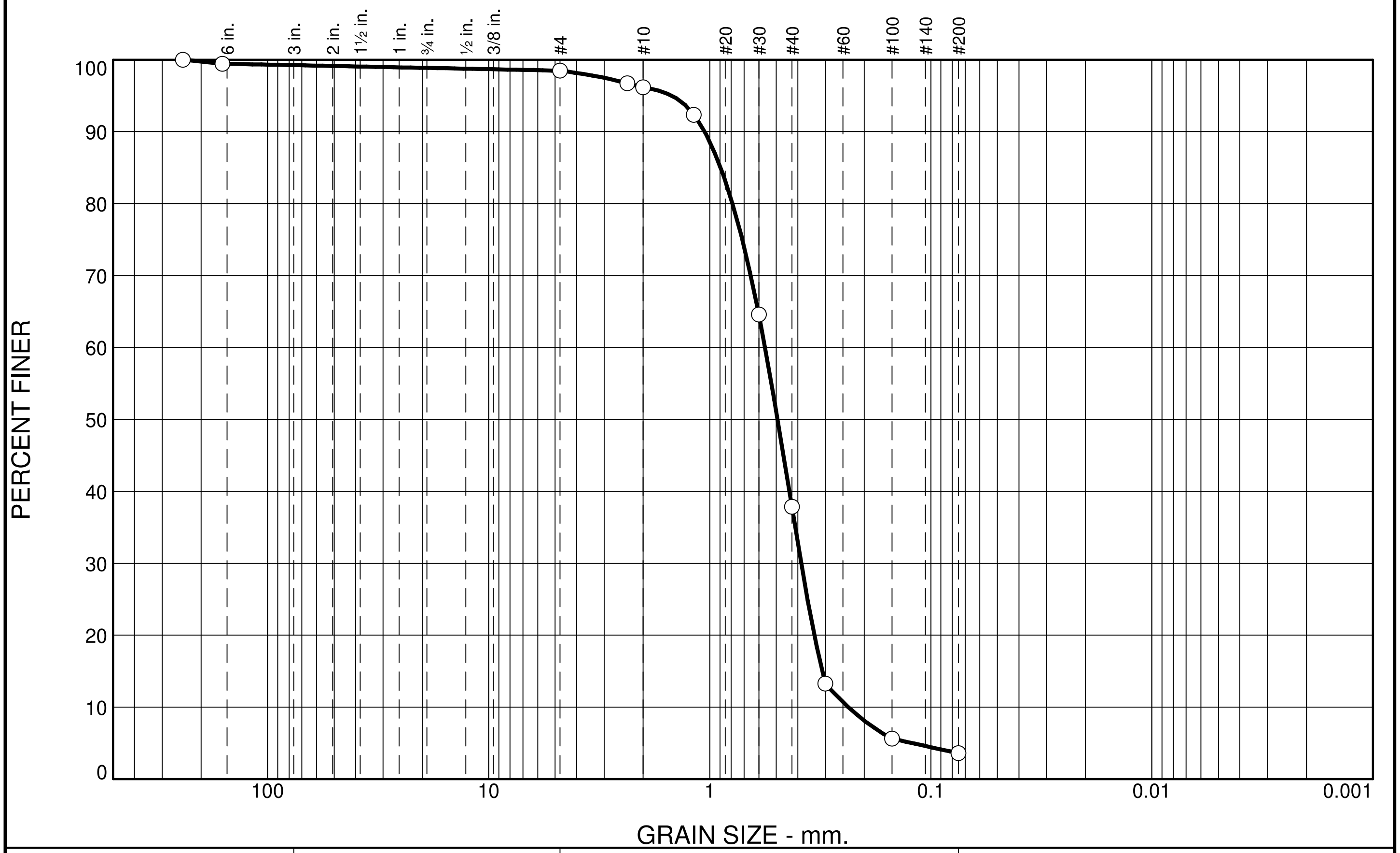
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: aw **Checked By:**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.8	0.3	0.4	2.3	58.4	34.2	3.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
9.5	100.0		
6.3	99.4		
#4	98.5		
#8	96.7		
#10	96.2		
#16	92.4		
#30	64.6		
#40	37.8		
#50	13.3		
#100	5.6		
#200	3.6		

* (no specification provided)

Soil Description

Light gray fine to medium sand (SP)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.0544 D_{85}= 0.8931 D_{60}= 0.5623
D_{50}= 0.4936 D_{30}= 0.3856 D_{15}= 0.3100
D_{10}= 0.2360 C_u= 2.38 C_c= 1.12

Classification

USCS= SP AASHTO=

Remarks

Source of Sample: S8-1 **Depth:** 58.5
Sample Number: 16 **Date:** 8/18/2020

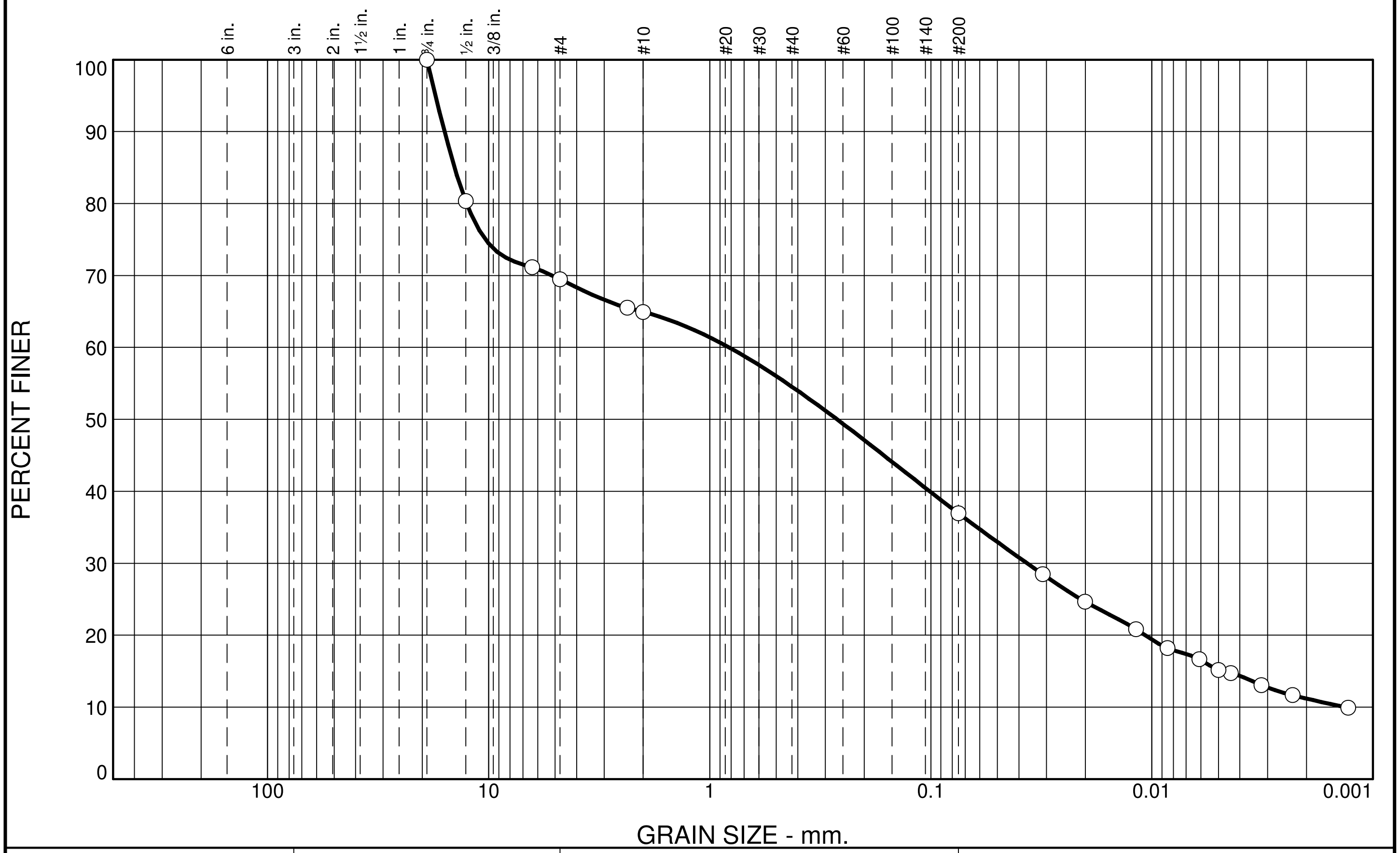
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: aw **Checked By:** ____________________

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	30.5	4.6	10.4	17.6	21.7	15.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	80.3		
1/4	71.1		
#4	69.5		
#8	65.5		
#10	64.9		
#200	36.9		
.001228	28.5		
.000791	24.7		
.000465	20.8		
.000335	18.2		
.000240	16.6		
.000197	15.2		
.000173	14.7		
.000126	13.1		
.000091	11.7		
.000051	9.9		

* (no specification provided)

Soil Description

Tan silty clay (CL), slightly sandy, with sand pockets

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 15.8205 D_{85}= 14.2610 D_{60}= 0.8177
D_{50}= 0.2658 D_{30}= 0.0367 D_{15}= 0.0048
D_{10}= 0.0013 C_u= 614.37 C_c= 1.24

Classification

USCS= AASHTO=

Remarks

Source of Sample: S8-2 **Depth:** 4.5'
Sample Number: 3 **Date:** 09-02-20

BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Client: ARDOT
Project: SR-230 Alicia to Bono, AR
Project No: 200518 **Figure**

Tested By: aw **Checked By:** ____________

APPENDIX B

Liquefaction Triggering Workbook

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]

Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:	111+10	
Location:	Craighead County	
Latitude and Longitude (decimal degrees)	35.909722	-90.863056
Logged By :	Don McCollum	
Boring No:	2	
Date:	Oct 15 and 16, 2019	
Type of Drilling:	HSA - Diamond Core	
Equipment:	Acker 2019	
Hammer Energy Correction Factor:	1.3	

Data Input by: Date:

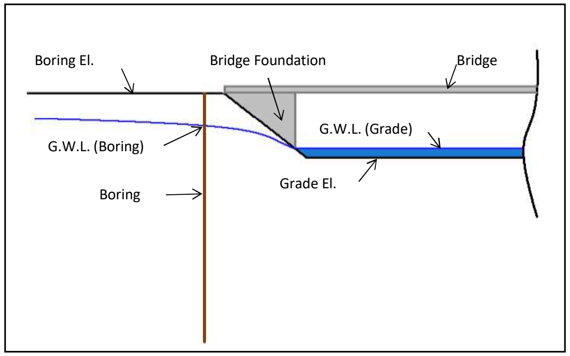
Checked by: Date:

All cells highlighted in this color should be input

All other cells should not be altered!

Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.463	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	251.3	ft
Ground Water Level (depth below boring surface) =	13.3	ft
Grade Surface Elevation =	233	ft
Ground Water Level (depth below or above grade surface) =	-5	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.



		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	246.3	5	SC	8	40					
2	241.3	10	CL	5	59					
3	236.3	15	SC	6	30					
4	231.3	20	SP-SM	18	9					
5	226.3	25	SC	10	16					
6	221.3	30	SP-SM	11	6					
7	216.3	35	SP-SM	21	5					
8	211.3	40	SW-SM	18	5					
9	206.3	45	SW	39	3					
10	201.3	50	SP	29	3					
11	196.3	55	SW	20	4					
12	191.3	60	SW	23	4					
13	186.3	65	SC	33	42					
14	181.3	70	SW	22	3					
15	176.3	75	SP	36	4					
16	171.3	80	SP	14	2					
17	166.3	85	SP	13	2					
18	161.3	90	SP-SM	70	6					
19	156.3	95	SP-SM	57	5					
20	151.3	100	SP-SM	39	4					
21	146.3	105	SP-SM	76	6					
22	141.3	110	SP-SM	41	5					

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]

Input Data Worksheet

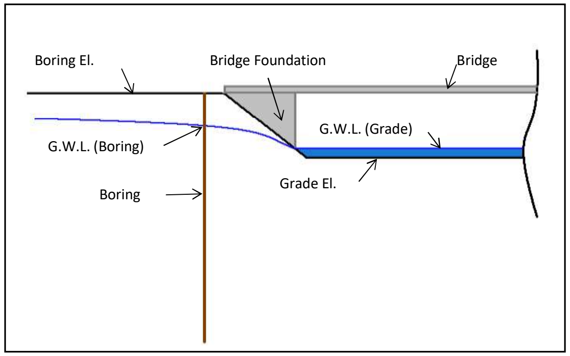
Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:	815+45	
Location:	Craighead County	
Latitude and Longitude (decimal degrees)	35.909689	-90.863056
Logged By :	B. Bates	
Boring No:	S8-1	
Date:	Aug 11, 12, and 18, 2020	
Type of Drilling:	HSA to 50', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: ______ Date: ______

Checked by: ______ Date: ______

All cells highlighted in this color should be input

All other cells should not be altered!



Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.463	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	250	ft
Ground Water Level (depth below boring surface) =	12	ft
Grade Surface Elevation =	233	ft
Ground Water Level (depth below or above grade surface) =	-5	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.

		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	247.5	2.5	SC	24	19.9					
2	245	5	SC	1	41.6					
3	242.5	7.5	CL	1	57.1		13	38		
4	240	10	CL	1	64		14	32	22.5	
5	237.5	12.5	SC	1	43.5		13	41		
6	235	15	SC	1	40.7	106	13	35	22.9	
7	230	20	SM	1	36.1		16	27		
8	225	25	SM	6	20.6					
9	220	30	SP-SM	18	8.3					
10	215	35	SP-SM	43	5.5					
11	210	40	SP-SM	26	6					
12	205	45	SP	20	3					
13	200	50	SP	15	3.2					
14	195	55	SP	18	2.2					
15	190	60	SP	11	3.6					
16	185	65	SP	10	3.1					
17	180	70	SP	26	3.4					
18	175	75	SP	22	4.2					
19	170	80	SP	34	4.4					
20	165	85	SP	19	1.2					
21	160	90	SP	37	2.6					
22	155	95	SP	31	2.6					
23	150	100	SP	35	3.5					

SPT Liquefaction Triggering Evaluation for all three procedures [Youd et al. (2001), Cetin et al. (2004), Idriss and Boulanger (2008)]

Input Data Worksheet

Job No:	101054	
Job Name:	Lawrence Co. Line - Bono Strs. & Apprs. (Hwy. 230) (S)	
Station:		
Location:	Craighead County	
Latitude and Longitude (decimal degrees)	35.909728	-90.862706
Logged By :	Christian Jackson	
Boring No:	S8-2	
Date:	19-Aug-20	
Type of Drilling:	HSA to 55', then rotary wash to comp.	
Equipment:		
Hammer Energy Correction Factor:	1.36	

Data Input by: 　　　　Date:

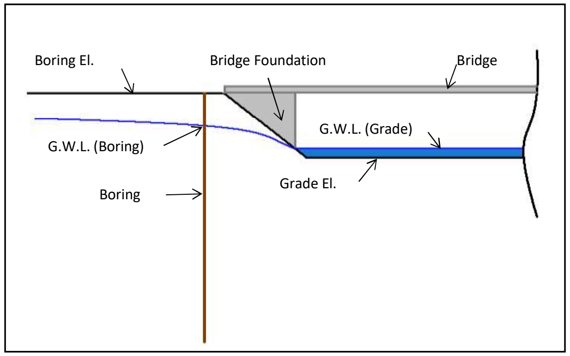
Checked by: 　　　　Date:

All cells highlighted in this color should be input

All other cells should not be altered!

Design Peak Horizontal Ground Acceleration (a_{max} , or A_s) =	0.463	g's
Earthquake Moment Magnitude (M_w) =	7.7	
Boring Surface Elevation =	250	ft
Ground Water Level (depth below boring surface) =	12	ft
Grade Surface Elevation =	233	ft
Ground Water Level (depth below or above grade surface) =	-5	ft
Sampler Type: Liner Space [Yes], or No Liner Space [No] =	Yes	
Liner Used [Yes], or no Liner Used [No]=	No	
Borehole Diameter =	4	in

The grade surface elevation input is useful in cases were a foundation location could not be bored due to access restrictions, but a nearby location was bored. This allows the user to "move" the soil boring to the location of the foundation by adjusting for the difference in elevation between the boring and the desired grade surface. Engineering judgment should be used in these situations as lateral variablility in the soil profile may be significant. See schematic to the right.



		Must Enter: Depth, USCS Classification (estimate if unknown) and N value								
Sample Number	**Elevation at Sample Location (ft)**	**Depth to Sample Location (ft)**	**USCS Classification**	**Raw SPT Blow Count, N**	**Fines Content (%)**	**Measured Unit Weight of Soil (pcf)**	**Plastic Limit, PL**	**Liquid Limit, LL**	**In-Situ Water Content, w_c**	**Undrained Shear Strength of the Soil, Su (psf)**
1	247.5	2.5	CL	9						
2	245	5	SC	8						
3	240	10	CH	1			14	50		
4	235	15	SC	1	35.7		11	29		
5	230	20	SP-SM	22	6.8					
6	225	25	SP-SM	13	6.6					
7	220	30	SP-SM	14	6.4					
8	215	35	SP-SM	20	6.9					
9	210	40	SP	25	4.4					
10	205	45	SP	28	1.7					
11	200	50	SP	19	2.7					
12	195	55	SP	33	1					
13	190	60	SP	22	2.4					
14	185	65	SP	22	2.8					
15	180	70	SP	20	2.7					
16	175	75	SP	17	2.2					
17	170	80	SP	20	3.2					
18	165	85	SP	11	2.3					
19	160	90	SP	33	3.4					
20	155	95	SP-SM	88	5.7					
21	150	100	SP	39	3.3					

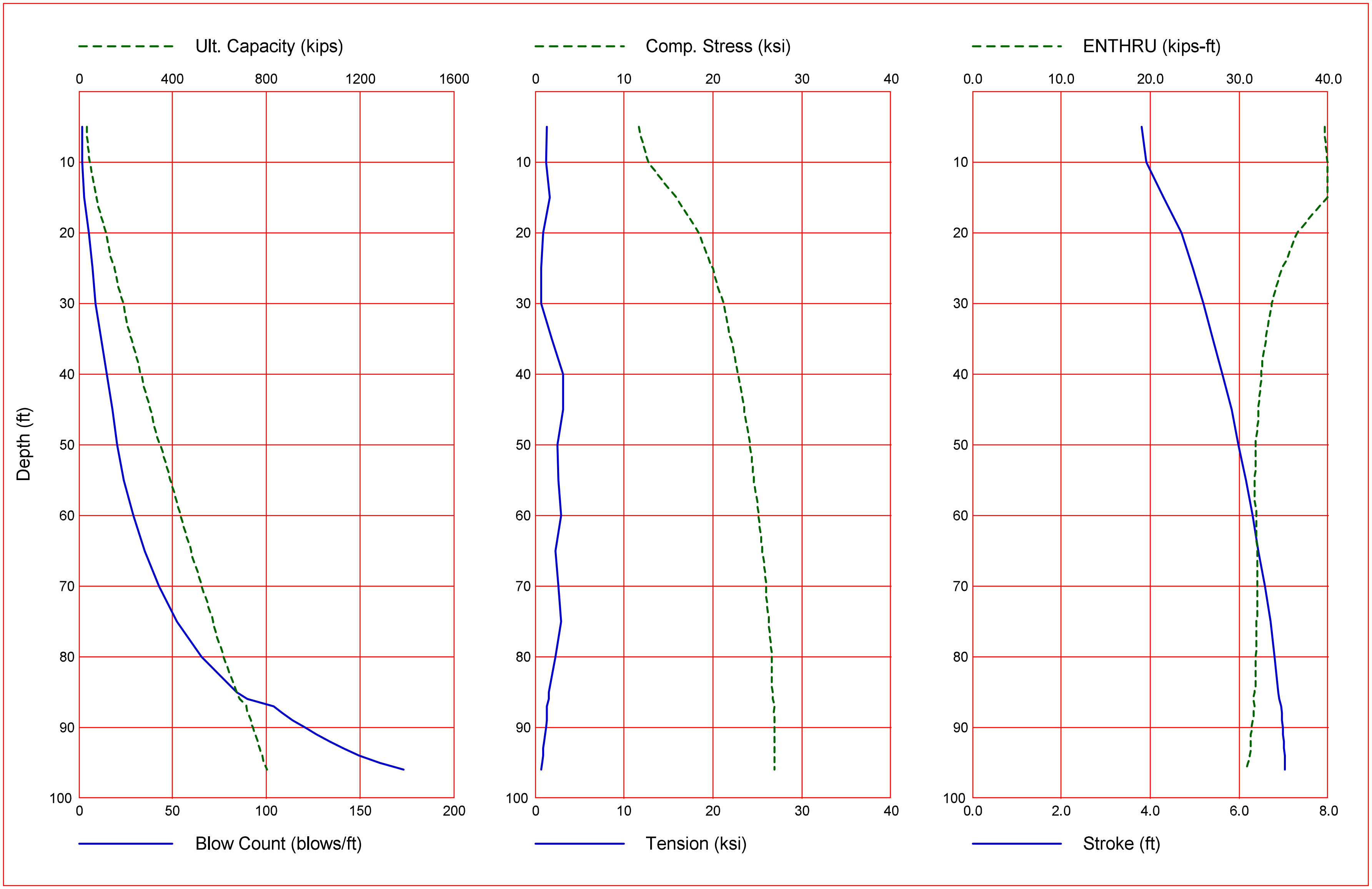
APPENDIX C

Pile Drivability Analysis Results

Burns Cooley Dennis, Inc.
Site 8 - East Abut - DELMAG D36

Jan 27 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Burns Cooley Dennis, Inc. Jan 27 2021
Site 8 - East Abut - DELMAG D36 GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

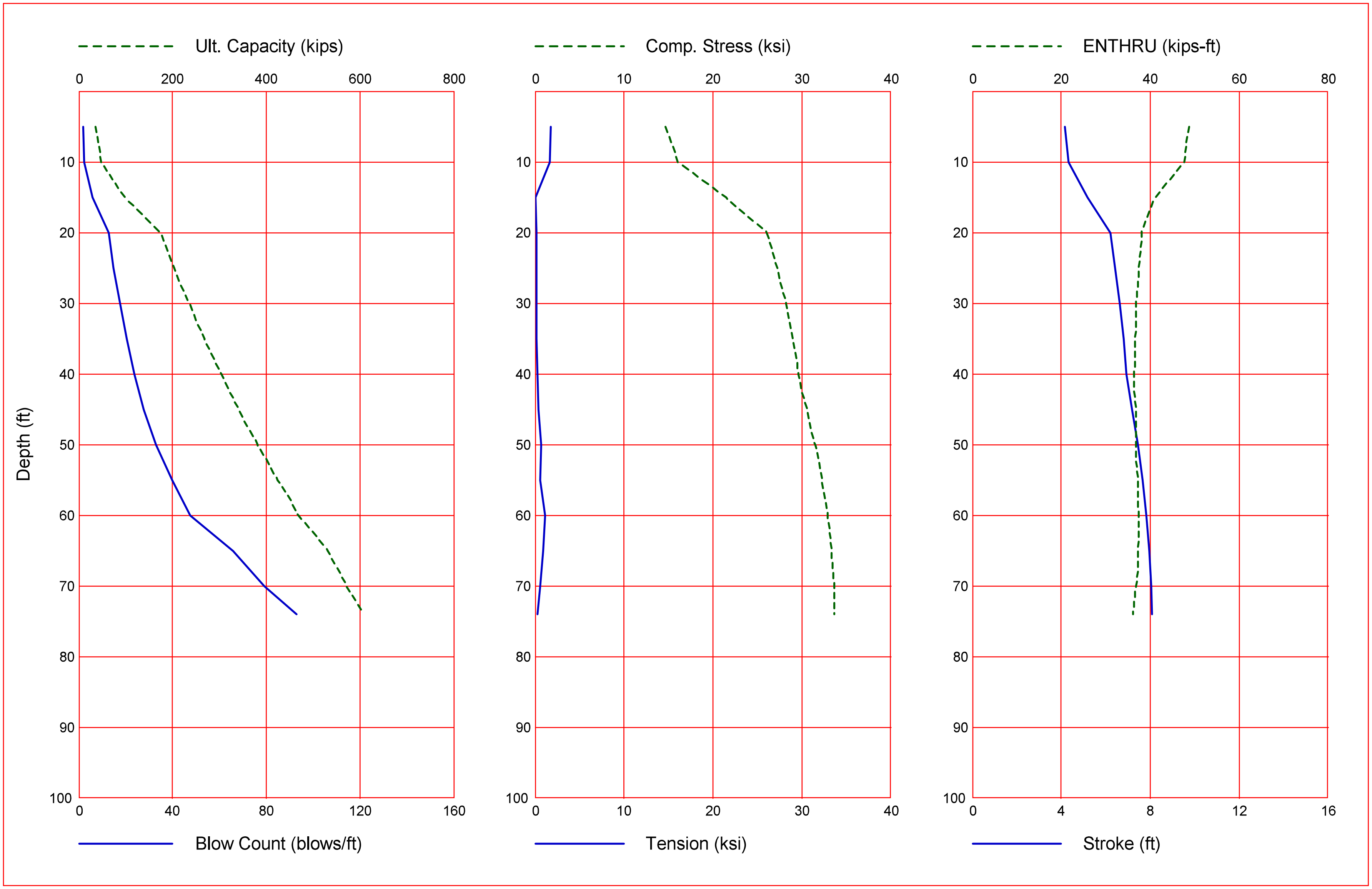
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	36.3	9.8	26.5	1.8	11.643	-1.322	3.81	39.6
10.0	46.1	19.6	26.5	2.0	12.725	-1.243	3.92	39.9
15.0	73.6	38.9	34.7	2.9	15.792	-1.683	4.29	39.9
20.0	116.4	63.4	53.0	5.2	18.423	-0.883	4.71	36.5
25.0	151.2	89.4	61.8	7.1	20.008	-0.717	4.97	34.8
30.0	187.1	116.4	70.7	9.1	21.154	-0.722	5.20	33.7
35.0	224.4	144.8	79.5	11.8	21.988	-1.814	5.41	33.0
40.0	263.6	175.3	88.4	14.7	22.849	-3.178	5.63	32.5
45.0	304.6	207.4	97.2	17.8	23.535	-3.197	5.84	32.2
50.0	347.0	241.0	106.0	20.7	24.149	-2.517	5.99	31.8
55.0	390.5	275.6	114.9	24.3	24.637	-2.642	6.16	31.7
60.0	434.4	310.7	123.7	28.9	25.168	-2.902	6.30	31.9
65.0	479.0	346.5	132.5	35.1	25.507	-2.339	6.43	32.0
70.0	524.7	383.3	141.4	42.6	25.968	-2.581	6.58	32.0
75.0	571.3	421.1	150.2	52.3	26.317	-2.976	6.71	31.9
80.0	618.9	459.8	159.0	65.5	26.594	-2.265	6.80	31.8
85.0	675.3	501.5	173.8	84.3	26.769	-1.595	6.89	31.7
86.0	687.2	510.5	176.7	89.7	26.746	-1.498	6.90	31.5
87.0	714.3	519.5	194.7	104.0	26.887	-1.380	6.94	31.7
88.0	723.8	528.7	195.1	108.6	26.865	-1.352	6.96	31.6
89.0	733.3	537.9	195.4	114.0	26.910	-1.305	6.97	31.5
90.0	743.0	547.2	195.8	120.6	26.940	-1.222	6.98	31.4
91.0	752.7	556.6	196.2	126.6	26.951	-1.153	6.99	31.3
92.0	762.5	566.0	196.5	133.9	26.957	-1.019	7.01	31.3
93.0	772.4	575.6	196.9	140.9	26.914	-0.952	7.01	31.3
94.0	782.4	585.2	197.2	149.8	26.974	-0.892	7.03	31.2
95.0	792.5	594.9	197.6	161.0	26.923	-0.806	7.03	31.0
96.0	802.6	604.7	197.9	173.6	26.895	-0.673	7.03	30.8

Total Continuous Driving Time 72.00 minutes; Total Number of Blows 3291 (starting at penetration 5.0 ft)

Burns Cooley Dennis, Inc.
Site 8- West Abut - DELMAG D36

Jan 27 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

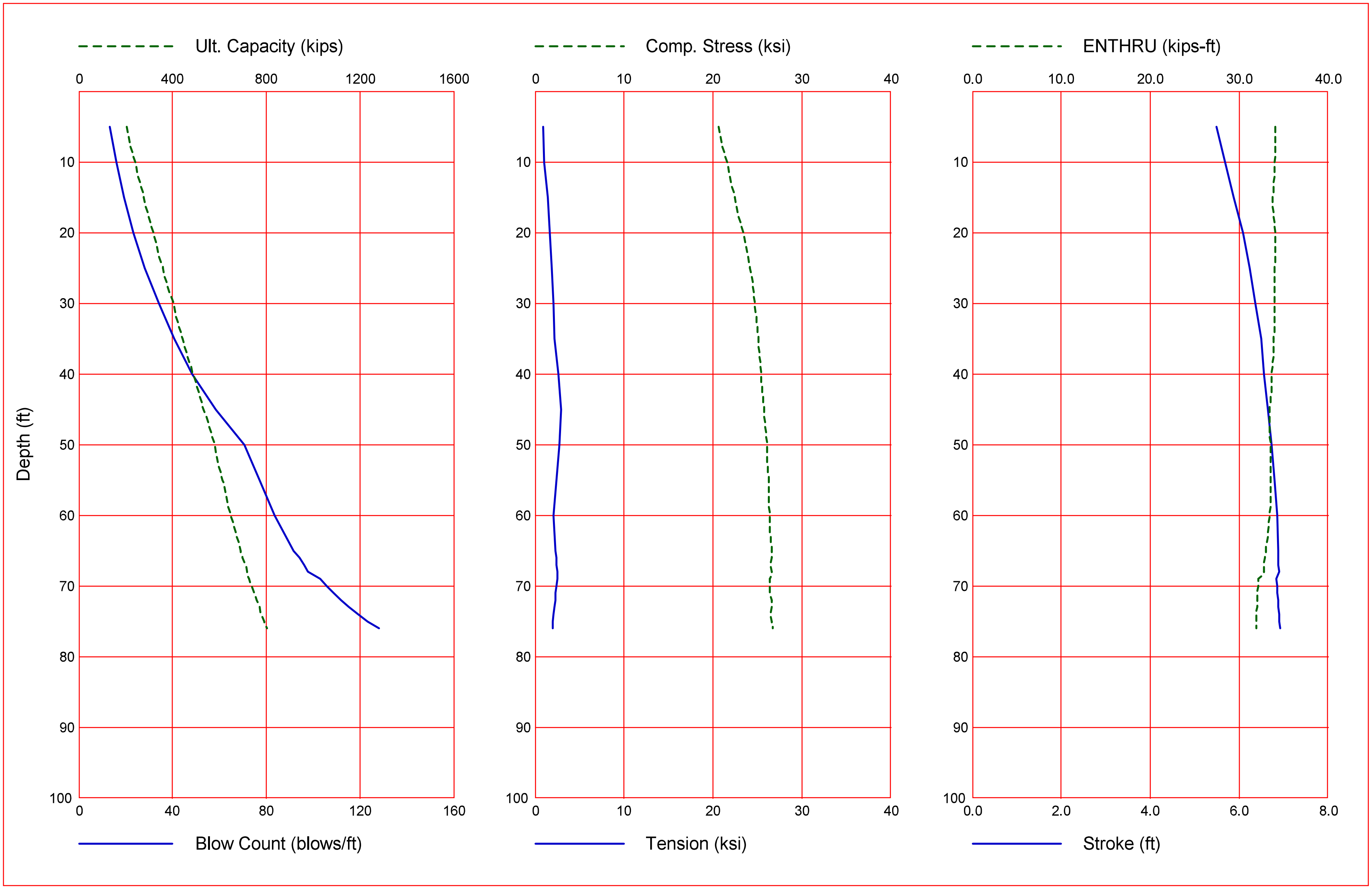
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	35.3	1.0	34.4	2.0	14.616	-1.766	4.14	48.7
10.0	47.7	13.3	34.4	2.4	16.084	-1.679	4.32	47.6
15.0	97.8	19.5	78.3	6.0	21.507	0.000	5.18	41.1
20.0	175.0	34.7	140.3	12.6	26.131	-0.168	6.22	38.1
25.0	203.9	54.3	149.6	14.9	27.232	-0.209	6.43	37.4
30.0	235.0	76.2	158.8	17.5	28.186	-0.165	6.62	36.8
35.0	268.4	100.3	168.1	20.3	28.920	-0.120	6.80	36.5
40.0	304.0	126.7	177.3	23.8	29.604	-0.258	6.93	36.3
45.0	341.9	155.4	186.5	27.7	30.578	-0.397	7.20	36.7
50.0	382.1	186.3	195.8	33.0	31.514	-0.648	7.45	36.8
55.0	424.5	219.4	205.0	39.8	32.248	-0.591	7.67	37.2
60.0	469.1	254.9	214.3	47.7	32.939	-1.102	7.84	37.4
65.0	531.1	292.6	238.6	65.7	33.306	-0.854	7.97	37.2
70.0	571.5	333.0	238.6	79.0	33.676	-0.575	8.04	36.7
74.0	605.9	367.4	238.6	93.0	33.692	-0.303	8.09	36.1

Total Continuous Driving Time 49.00 minutes; Total Number of Blows 2105 (starting at penetration 5.0 ft)

Burns Cooley Dennis, Inc.
Site 8 - Interior Bents - DELMAG D36

Jan 27 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Burns Cooley Dennis, Inc. Jan 27 2021
Site 8 - Interior Bents - DELMAG D36 GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	205.8	19.6	186.1	13.2	20.688	-0.885	5.50	34.1
10.0	241.1	39.3	201.8	16.0	21.575	-0.999	5.68	34.0
15.0	278.1	60.5	217.5	19.3	22.451	-1.485	5.87	33.8
20.0	318.3	85.0	233.3	23.2	23.460	-1.698	6.08	34.1
25.0	360.2	111.2	249.0	28.1	24.127	-1.820	6.24	34.0
30.0	402.1	137.4	264.7	34.3	24.649	-2.126	6.37	34.0
35.0	444.6	164.2	280.4	40.9	25.126	-2.153	6.49	33.9
40.0	488.4	192.3	296.1	48.6	25.460	-2.637	6.57	33.7
45.0	534.5	221.1	313.4	58.7	25.716	-2.884	6.65	33.4
50.0	580.6	249.9	330.6	70.7	26.027	-2.751	6.73	33.5
55.0	612.0	281.4	330.6	77.3	26.277	-2.443	6.80	33.5
60.0	648.8	318.1	330.6	83.7	26.453	-2.077	6.86	33.4
65.0	690.8	360.2	330.6	91.9	26.565	-2.311	6.88	33.0
66.0	699.9	369.2	330.6	94.1	26.578	-2.366	6.88	32.9
67.0	709.2	378.5	330.6	96.0	26.506	-2.410	6.89	32.8
68.0	718.7	388.1	330.6	98.0	26.585	-2.459	6.90	32.8
69.0	728.5	397.9	330.6	103.2	26.409	-2.511	6.83	32.2
70.0	738.6	408.0	330.6	105.8	26.357	-2.416	6.85	32.2
71.0	749.0	418.3	330.6	108.8	26.438	-2.340	6.86	32.1
72.0	759.6	428.9	330.6	111.9	26.569	-2.287	6.87	32.1
73.0	770.4	439.8	330.6	115.3	26.564	-2.156	6.88	32.0
74.0	781.5	450.9	330.6	119.3	26.523	-2.073	6.90	31.9
75.0	792.9	462.3	330.6	123.5	26.629	-2.021	6.91	31.9
76.0	804.6	473.9	330.6	128.2	26.669	-1.943	6.93	31.9

Total Continuous Driving Time 87.00 minutes; Total Number of Blows 3952 (starting at penetration 5.0 ft)

APPENDIX D

AHTD Special Provision for Embankment Construction

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

DESCRIPTION: This Special Provision shall be supplementary to Section 210, Excavation and Embankment, of the Standard Specifications, Edition of 2003 and shall apply to the construction of embankments being built over existing borrow ditches as shown in the plans or where directed by the Engineer.

MATERIALS: Stone Backfill shall meet the requirements of Section 207 of the Standard Specifications, Edition of 2003.

Select Material (Class SM-2) shall meet the requirements of Section 302 of the Standard Specifications, Edition of 2003.

Dumped Riprap and Filter Blanket shall comply with Section 816 of the Standard Specifications except that synthetic geotextile fabric complying with requirements of Subsection 625.02, Type 5 must be used as a filter blanket under dumped riprap in lieu of a granular filter blanket material.

Clay plating shall consist of material having a minimum plasticity index of 10 and a maximum plasticity index of 25, which will support vegetation and not be highly susceptible to erosion.

CONSTRUCTION: When the embankment is to be built over existing borrow ditches, the ditches shall be undercut 2 feet below the existing flow line to remove all highly organic, wet material prior to embankment construction. The ditches shall then be filled using Stone Backfill. The top 4" to 6" of Stone Backfill shall be material complying with Section 303 of the Standard Specifications, Edition of 2003 for Class 7 Aggregate Base Course in accordance with Section 207. Excavation for the placement of Stone Backfill shall be considered part of the item in accordance with subsection 207.01 of the Standard Specifications.

The remaining embankment shall be constructed of Selected Material (Class SM-2). Synthetic Filter Blanket and Dumped Riprap shall be placed on the slopes of embankments constructed of Select Material (Class SM-2) from the top of the Stone Backfill to 2 feet above the high water elevation or as directed by the Engineer. The remainder of embankments constructed of Select Material (Class SM-2) or other material which is susceptible to erosion shall have a minimum 18 inch clay plating (measured

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

perpendicular to the finished slopes).

All embankment materials, including Selected Material (Class SM-2) and Clay Plating, shall be placed and compacted in accordance with Subsections 210.07, 210.09, and 210.10 of the Standard Specifications.

QUALTIY CONTROL AND ACCEPTANCE: The Contractor shall perform quality control and acceptance sampling and testing of the clay plating for plasticity index; Selected Material (Class SM-2) for gradation and plasticity index in accordance with Section 306 except that the size of the standard lot will be 3000 cubic yards. The Contractor shall perform quality control and acceptance sampling and testing of the Selected Material (Class SM-2) for density and moisture content in accordance with Subsection 210.02 of the Standard Specifications for Highway Construction. Selected Material (Class SM-2) shall meet the density requirements of Subsection 210.10.

METHOD OF MEASUREMENT: Embankments consisting of Selected Material (Class SM-2) and Clay Plating material and as shown on the plans, will be measured as Compacted Embankment in accordance with Subsection 210.12 of the Standard Specifications.

Stone Backfill will be measured in accordance with Section 207 of the Standard Specifications.

Filter Blanket and Dumped Riprap will be measured in accordance with Section 816 of the Standard Specifications.

BASIS OF PAYMENT: All accepted embankments; including Selected Material (Class SM-2) and Clay Plating material measured as provided above will be paid for as Compacted Embankment in accordance with Subsection 210.13 of the Standard Specifications.

Stone Backfill shall be paid in accordance with Section 207 of the Standard Specifications.

Filter Blanket and Dumped Riprap will be paid in accordance with Section 816 of the Standard Specifications.

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 070291

EMBANKMENT CONSTRUCTION

Payment will be made under:

Pay Item	Pay Unit
Compacted Embankment	Cubic Yard
Stone Backfill	Ton
Filter Blanket	Square Yard
Dumped Riprap	Cubic Yard

BURNS COOLEY DENNIS, INC.

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July 2, 2021

Cindy Rich, P.E.
Neel-Schaffer, Inc.
125 South Congress Street, Suite 1100
Post Office Box 22625
Jackson, Mississippi 39201

Report No. 200518 – Sites 1, 2, 5, and 8 Addendum

Geotechnical Exploration
Sites 1, 2, 5, and 8
ARDOT SR230 Bridge Replacements
Craighead and Lawrence Counties, Arkansas

Dear Ms. Rich:

Submitted here is an addendum to the reports of our geotechnical explorations for the above-captioned project. This project was authorized by Task Order 108 to the Subconsultant Agreement between Neel-Schaffer, Inc. and Burns Cooley Dennis, Inc. dated September 17, 2020. This addendum presents analysis results for additional pile sizes.

We appreciate the opportunity to be of service. If you should have any questions concerning this addendum, please do not hesitate to call us.

Very truly yours,

BURNS COOLEY DENNIS, INC.

Alexander B. Reeb, Ph.D., P.E.

A. E. (Eddie) Templeton, P.E.

ABR/AET/khb
Copy Submitted: (via e-mail)

TABLE OF CONTENTS

1.0 SITE 1

1.1 Deep Foundations

We understand that driven 16-in. diameter, closed-ended steel pipe piles are now being proposed for the abutments. The supplemental pile analysis results are provided in brief below. Please see our main report for additional details.

1.1.1 Axial Pile Capacity. Updated axial pile capacity curves are presented in Appendix A for the west abutment and east abutment bents. The pile capacity curves are presented as nominal (ultimate) values that do not include a resistance factor. An appropriate resistance factor should be applied to the nominal values presented on the pile capacity curves. Guidance on resistance factors and other details related to our pile analyses and recommendations are provided in our main report.

1.1.2 Downdrag. The updated downdrag analysis results are summarized in the following tables. Table 1 presents the results for the west abutment bent for a load of 80 kips. Table 2 presents the results for the east abutment bent for a load of 80 kips. For each case, results are provided for a range of possible pile lengths.

Table 1 - Downdrag Analysis Results for West Abutment with Load of 80 kips

	Pile Length (ft) below El 247 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	195	226	276	276	276
Top of Pile Settlement (in.)	3.7	1.5	0.1	0.1	0.1
Neutral Plane Depth (ft)	79.2	84.6	88.0	88.0	88.0

Table 2 - Downdrag Analysis Results for East Abutment with Load of 80 kips

	Pile Length (ft) below El 247 ft				
	95	100	110	120	130
Maximum Drag Load (kips)	168	204	276	302	302
Top of Pile Settlement (in.)	5.2	4.1	0.9	0.1	0.1
Neutral Plane Depth (ft)	74.9	81.1	92.6	94.0	94.0

1.1.3 Drivability Analysis. Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. An open-ended D36 diesel hammer was utilized for the drivability analysis. Hammer and pile cushion information was based on manufacturer-recommended values. The 16-in. diameter steel pipe piles were assumed to be installed close-ended, and the piles were assumed to be driven from the plan pile cap bottom elevations to the recommended tip elevations. Graphical and tabulated results of the drivability analyses are provided in Appendix A. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed prior to hammer acceptance and beginning of driving. The resulting minimum hammer energy to drive the piles at the abutment bents is provided in Table 3.

Table 3 - Results of Drivability Analyses

Location	Hammer Type	Minimum Hammer Energy (kip-ft.)
Abutment Bents	D36	80

2.0 SITE 2

2.1 Deep Foundations

We understand that driven 16-in. diameter, closed-ended steel pipe piles are now being proposed for the abutments. The supplemental pile analysis results are provided in brief below. Please see our main report for additional details.

2.1.1 Axial Pile Capacity. Updated axial pile capacity curves are presented in Appendix B for the west abutment and east abutment bents. The pile capacity curves are presented as nominal (ultimate) values that do not include a resistance factor. An appropriate resistance factor should be applied to the nominal values presented on the pile capacity curves. Guidance on resistance factors and other details related to our pile analyses and recommendations are provided in our main report.

2.1.2 Downdrag. The updated downdrag analysis results are summarized in the following tables. Table 4 presents the results for the west abutment bent for a load of 80 kips. Table 5 presents the results for the east abutment bent for a load of 80 kips. For each case, results are provided for a range of possible pile lengths.

Table 4 - Downdrag Analysis Results for West Abutment with Load of 80 kips

	Pile Length (ft) below El 248 ft				
	90	95	100	110	120
Maximum Drag Load (kips)	257	296	328	409	487
Top of Pile Settlement (in.)	3.2	3.2	3.2	1.4	0.2
Neutral Plane Depth (ft)	61.4	65.0	68.6	81.2	89.0

Table 5 - Downdrag Analysis Results for East Abutment with Load of 80 kips

	Pile Length (ft) below El 246 ft				
	80	85	90	100	110
Maximum Drag Load (kips)	175	206	238	295	295
Top of Pile Settlement (in.)	5.4	2.9	2.4	0.1	0.1
Neutral Plane Depth (ft)	55.2	62.2	68.8	77.0	77.0

2.1.3 Drivability Analysis. Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. An open-ended D30 diesel hammer was utilized for the drivability analysis. Hammer and pile cushion information was based on manufacturer-recommended values. The 16-in. diameter steel pipe piles were assumed to be installed close-ended, and the piles were assumed to be driven from the plan pile cap bottom elevations to the recommended tip elevations. Graphical and tabulated results of the drivability analyses are provided in Appendix B. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed prior to hammer acceptance and beginning of driving. The resulting minimum hammer energy to drive the piles at the abutment bents is provided in Table 6.

Table 6 - Results of Drivability Analyses

Location	Hammer Type	Minimum Hammer Energy (kip-ft.)
Abutment Bents	D30	70

3.0 SITE 5

3.1 Deep Foundations

We understand that driven 30-in. diameter, closed-ended steel pipe piles are now being proposed for the interior bents. The supplemental pile analysis results are provided in brief below. Please see our main report for additional details.

3.1.1 Axial Pile Capacity. Updated axial pile capacity curves are presented in Appendix C for the interior bents. The pile capacity curves are presented as nominal (ultimate) values that do not include a resistance factor. An appropriate resistance factor should be applied to the nominal values presented on the pile capacity curves. Guidance on resistance factors and other details related to our pile analyses and recommendations are provided in our main report.

3.1.2 Downdrag. The updated downdrag analysis results are summarized in the following table. Table 7 presents the results for the interior bents for a load of 173 kips (i.e., 160 kips dead load plus 13 kips for pile stick-up). Results are provided for a range of possible pile lengths.

Table 7 - Downdrag Analysis Results for Interior Bents with Load of 173 kips

	Pile Length (ft) below El 234 ft				
	80	85	90	100	110
Maximum Drag Load (kips)	508	583	666	825	847
Top of Pile Settlement (in.)	1.9	1.8	1.8	0.2	0.1
Neutral Plane Depth (ft)	64.7	67.8	71.9	79.7	80.0

3.1.4 Drivability Analysis. Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. An open-ended D46 diesel hammer was utilized for the drivability analysis. Hammer and pile cushion information was based on manufacturer-recommended values. The 30-in. diameter steel pipe piles were assumed to be installed close-ended, and the piles were assumed to be driven from the plan pile cap bottom elevations to the recommended tip elevations. Graphical and tabulated results of the drivability analyses are provided in Appendix C. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed prior to hammer acceptance and beginning of driving. The resulting minimum hammer energy to drive the piles at the interior bents is provided in Table 8.

Table 8 - Results of Drivability Analyses

Location	Hammer Type	Minimum Hammer Energy (kip-ft.)
Interior Bents	D46	90

4.0 SITE 8

4.1 Deep Foundations

We understand that driven 30-in. diameter, closed-ended steel pipe piles are now being proposed for the interior bents. The supplemental pile analysis results are provided in brief below. Please see our main report for additional details.

4.1.1 Axial Pile Capacity. Updated axial pile capacity curves are presented in Appendix D for the interior bents. The pile capacity curves are presented as nominal (ultimate) values that do not include a resistance factor. An appropriate resistance factor should be applied to the nominal values presented on the pile capacity curves. Guidance on resistance factors and other details related to our pile analyses and recommendations are provided in our main report.

4.1.2 Downdrag. The updated downdrag analysis results are summarized in the following table. Table 9 presents the results for the interior bents for a load of 169 kips (i.e., 155 kips dead load plus 14 kips for pile stick-up). Results are provided for a range of possible pile lengths.

Table 9 - Downdrag Analysis Results for Interior Bents with Load of 169 kips

	Pile Length (ft) below El 232 ft				
	65	70	75	80	90
Maximum Drag Load (kips)	107	226	226	226	226
Top of Pile Settlement (in.)	4.4	0.04	0.04	0.04	0.04
Neutral Plane Depth (ft)	56.0	66.0	66.0	66.0	66.0

4.1.3 Drivability Analysis. Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. An open-ended D46 diesel hammer was utilized for the drivability analysis. Hammer and pile cushion information was based on manufacturer-recommended values. The 30-in. diameter steel pipe piles were assumed to be installed close-ended, and the piles were assumed to be driven from the plan pile cap bottom elevations to the recommended tip elevations. Graphical and tabulated results of the drivability analyses are provided in Appendix D. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed prior to hammer acceptance and beginning of driving. The resulting minimum hammer energy to drive the piles at the interior bents is provided in Table 10.

Table 10 - Results of Drivability Analyses

Location	Hammer Type	Minimum Hammer Energy (kip-ft.)
Interior Bents	D46	90

5.0 REPORT LIMITATIONS

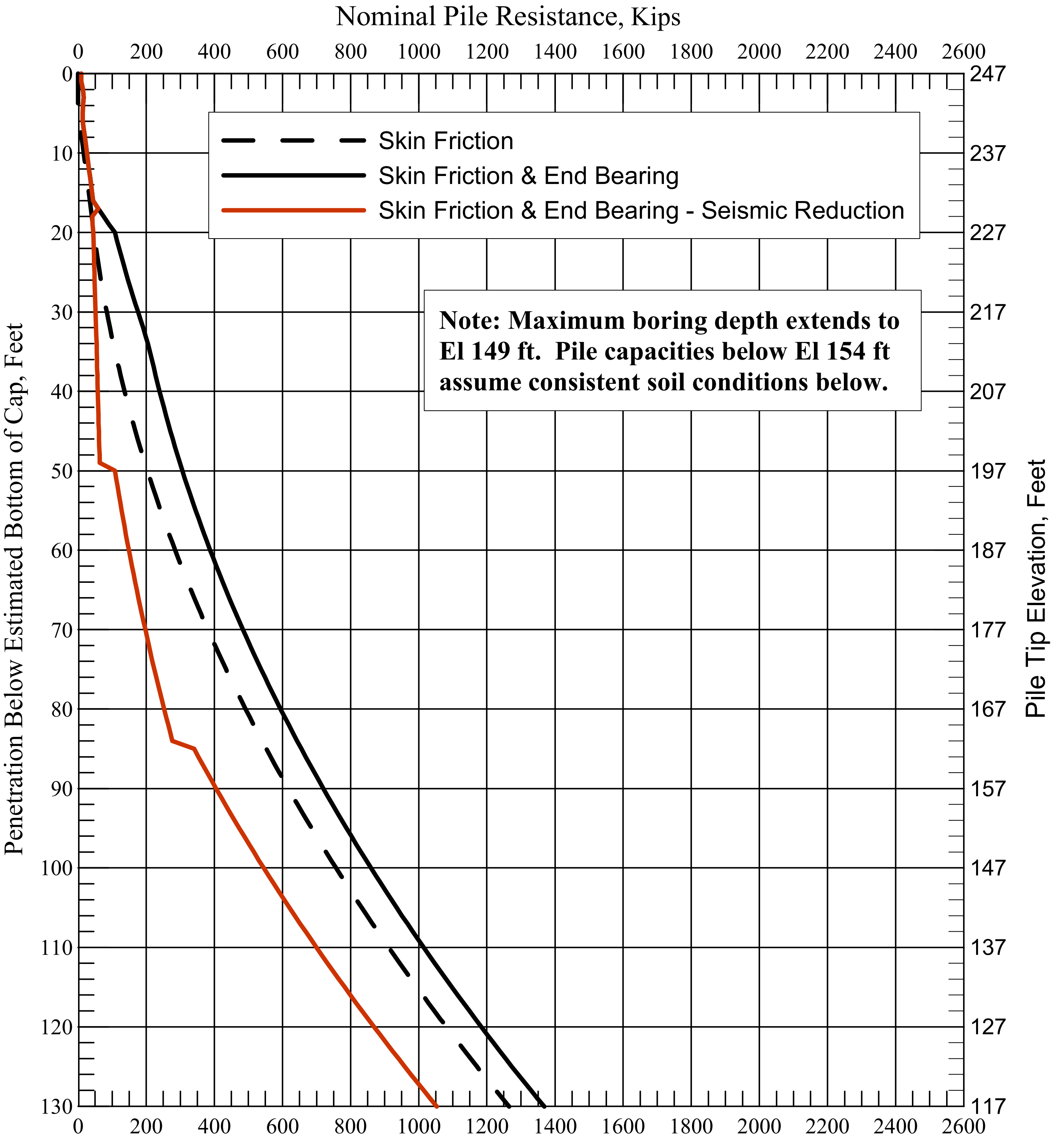
The analyses, conclusions, and recommendations discussed in this report are based on conditions as they existed at the time of the exploration and further on the assumption that the exploratory borings are representative of subsurface conditions throughout the areas investigated. It should be noted that actual subsurface conditions between and beyond the borings might differ from those encountered at the boring locations. If subsurface conditions are encountered during construction that vary from those discussed in this report, Burns Cooley Dennis, Inc. should be notified immediately in order that we may evaluate the effects, if any, on earthwork and foundation design and construction.

Burns Cooley Dennis, Inc. should be retained for a general review of final design drawings and specifications. It is advised that we also be retained to observe earthwork for the project, to perform and observe the pile testing, and to develop the pile driving criteria. Our involvement during construction would give opportunity for us to help confirm that our recommendations are valid or to modify them accordingly. Burns Cooley Dennis, Inc. cannot assume responsibility or liability for the adequacy of recommendations if we do not observe construction.

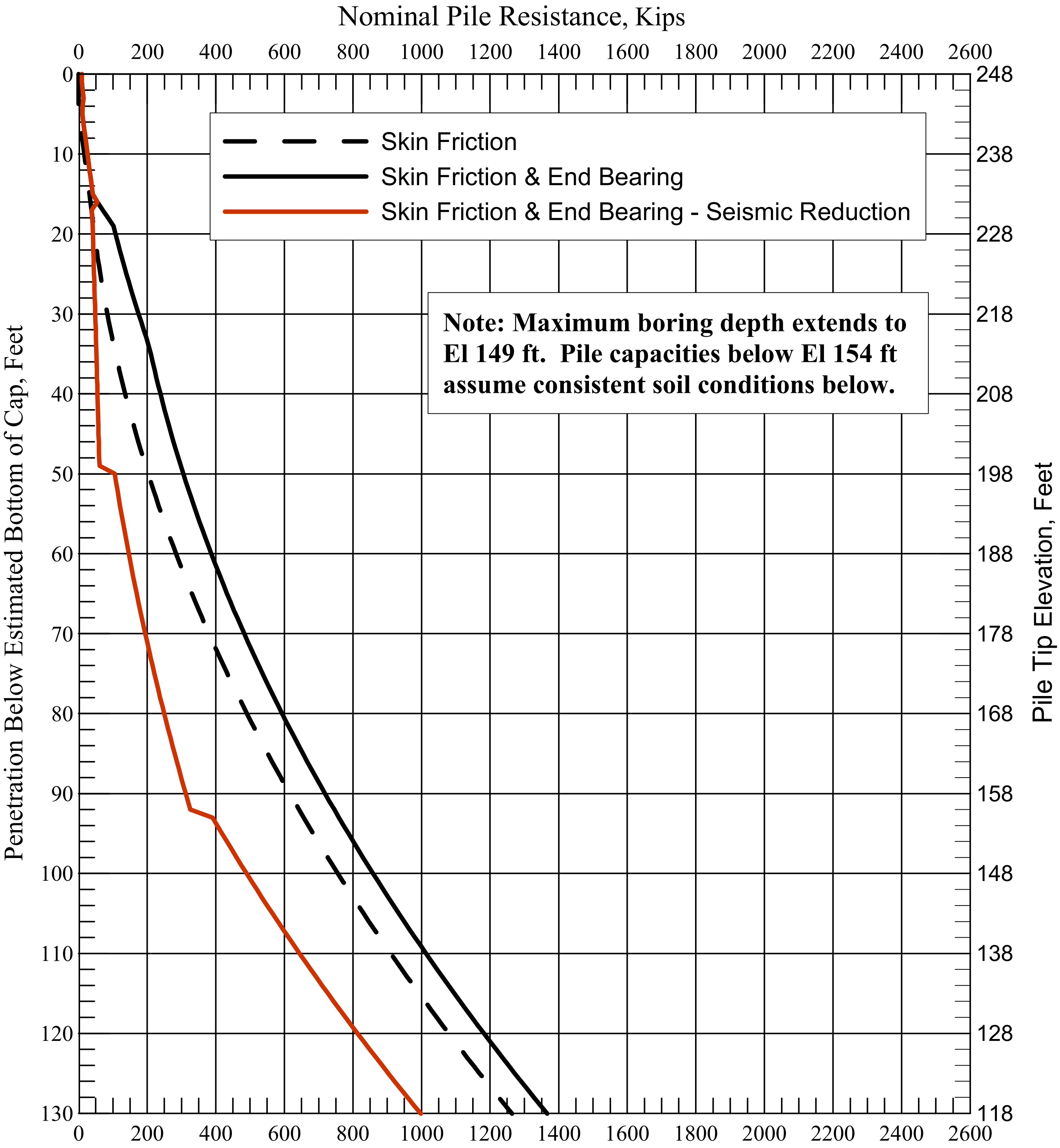
This report has been prepared for the exclusive use of Neel-Schaffer, Inc. for specific application to the geotechnical-related aspects of design and construction of the ARDOT SR230 Bridge Replacements in Craighead and Lawrence Counties, Arkansas. The only warranty made by us in connection with the services provided is we have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing in the same or similar locality. No other warranty, express or implied, is made or intended.

APPENDIX A

Site 1: Pile Axial Capacity Curves and Drivability Analysis Results



NOMINAL PILE RESISTANCE CURVES
SITE 1
WEST ABUTMENT
16" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

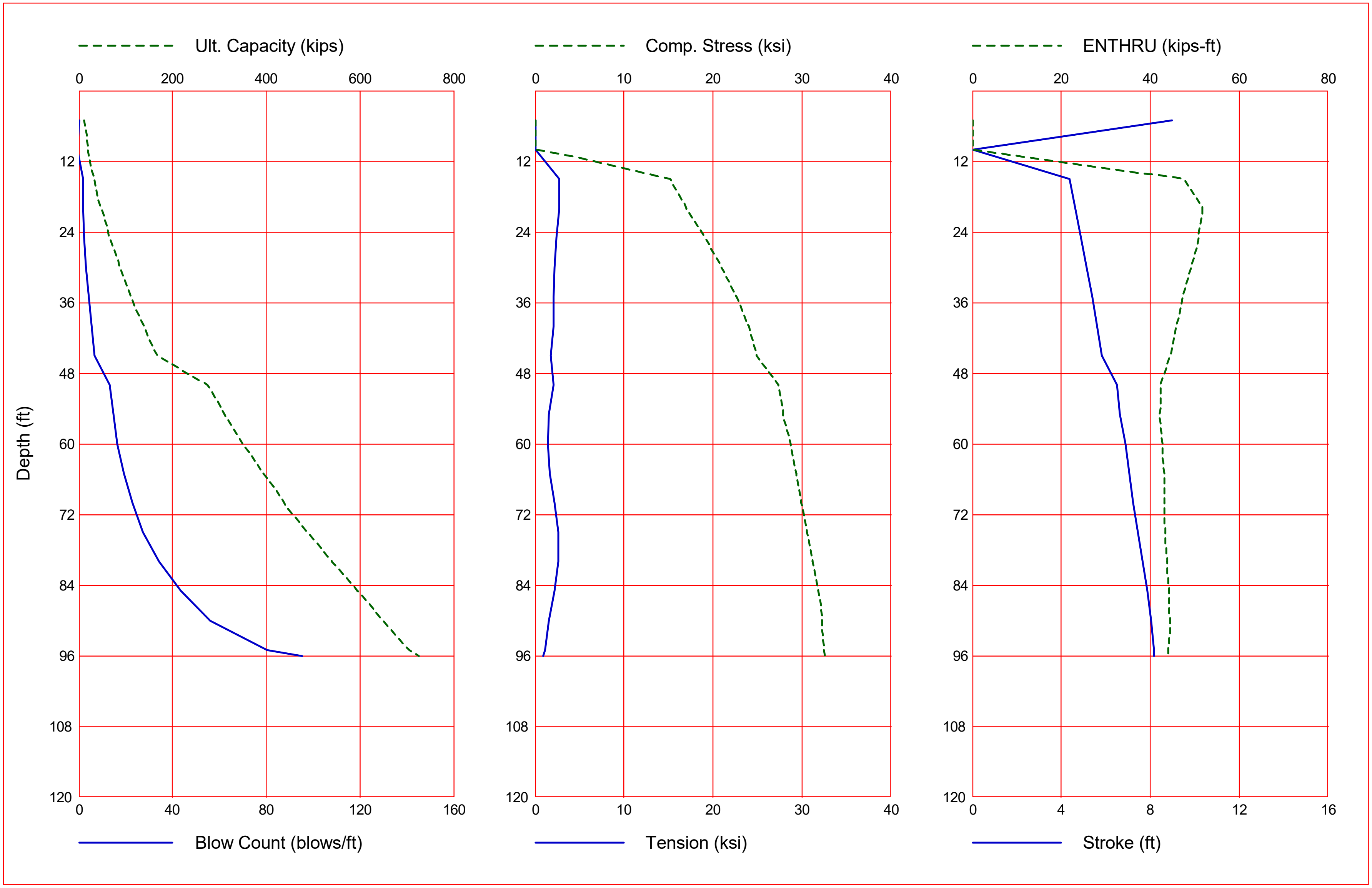


NOMINAL PILE RESISTANCE CURVES
SITE 1
EAST ABUTMENT
16" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

Burns Cooley Dennis, Inc.
Site 1- West Abutment - DELMAG D36

Jun 30 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Burns Cooley Dennis, Inc. Jun 30 2021
Site 1- West Abutment - DELMAG D36 GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

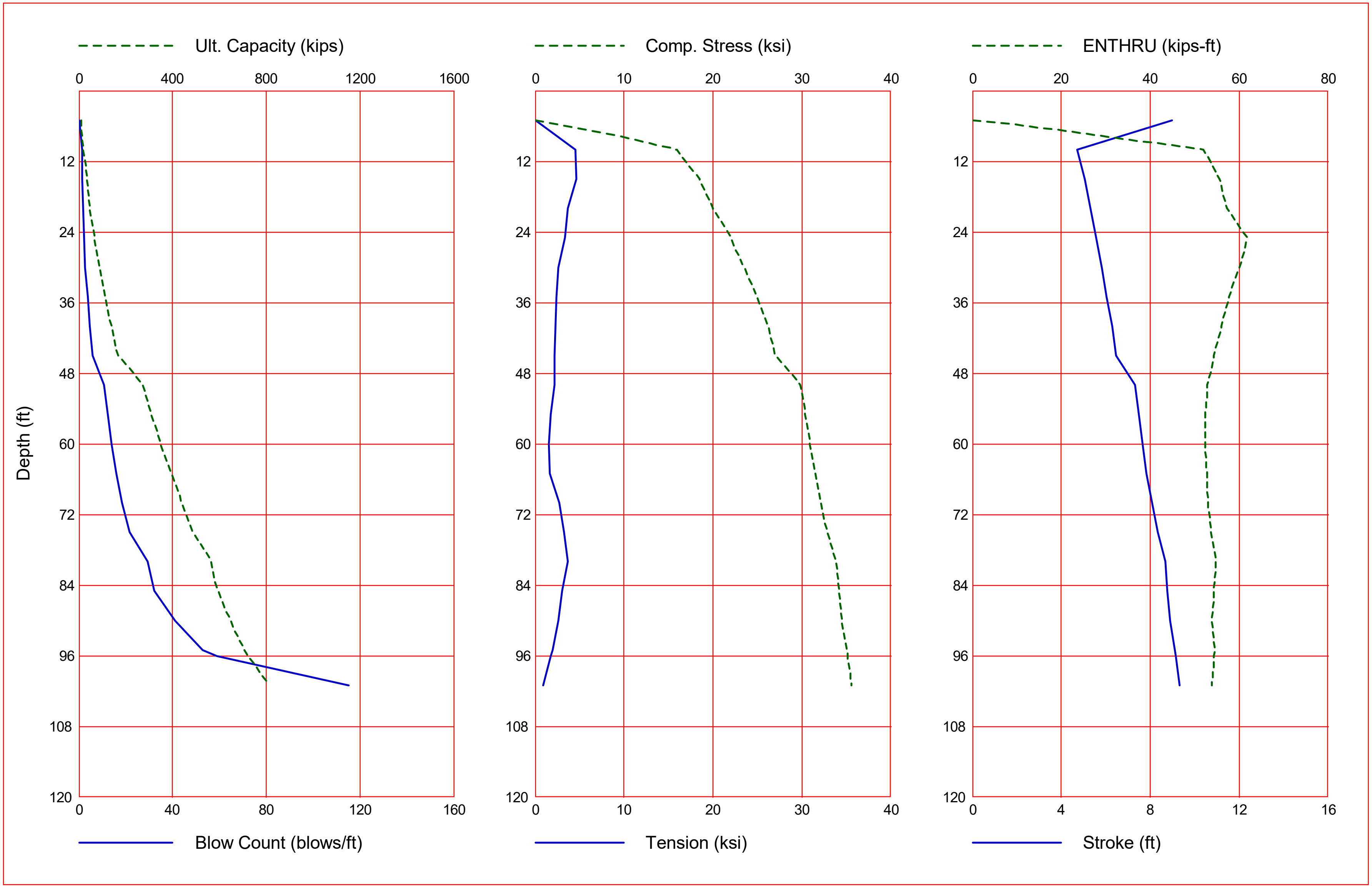
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	11.9	3.5	8.4	0.0	0.000	0.000	9.00	0.0
10.0	19.7	11.3	8.4	-1.0	0.000	0.000	0.00	0.0
15.0	32.9	23.5	9.4	1.7	15.249	-2.767	4.37	47.7
20.0	47.2	36.3	10.9	1.9	17.025	-2.710	4.63	51.9
25.0	65.9	52.0	14.0	2.3	19.109	-2.392	4.89	50.9
30.0	87.2	71.2	16.1	3.2	21.009	-2.239	5.15	49.2
35.0	112.0	93.8	18.1	4.3	22.671	-2.100	5.40	47.3
40.0	138.2	120.0	18.1	5.5	24.004	-2.037	5.62	45.7
45.0	167.8	149.7	18.1	6.8	24.958	-1.792	5.84	44.4
50.0	273.6	182.8	90.8	13.0	27.353	-2.053	6.51	42.4
55.0	310.2	219.4	90.8	14.7	27.903	-1.558	6.65	42.2
60.0	350.3	259.6	90.8	16.6	28.717	-1.486	6.89	42.7
65.0	393.0	302.3	90.8	19.2	29.334	-1.616	7.07	43.1
70.0	438.4	347.6	90.8	22.7	29.936	-2.195	7.25	43.2
75.0	487.2	396.5	90.8	27.4	30.562	-2.580	7.45	43.5
80.0	539.6	448.8	90.8	34.3	31.196	-2.642	7.67	43.9
85.0	593.6	502.9	90.8	43.5	31.840	-2.151	7.89	44.3
90.0	648.6	557.8	90.8	56.1	32.285	-1.594	8.06	44.5
95.0	705.3	614.5	90.8	80.6	32.498	-1.132	8.15	44.0
96.0	726.6	626.1	100.5	95.4	32.629	-0.946	8.19	44.1

Total Continuous Driving Time 38.00 minutes; Total Number of Blows 1658 (starting at penetration 5.0 ft)

Burns Cooley Dennis, Inc.
Site 1- East Abutment - DELMAG D36

Jun 30 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



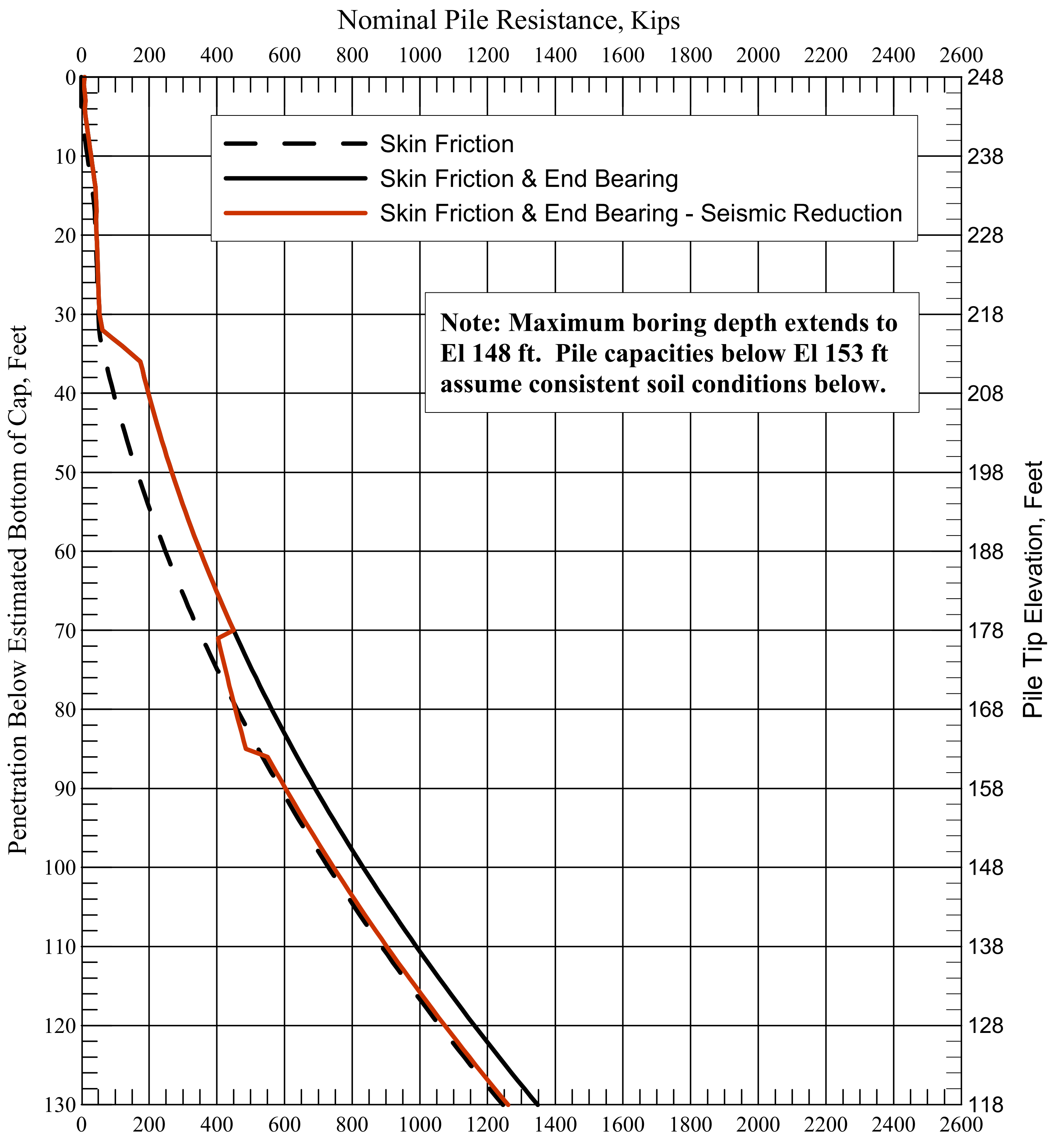
Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	11.9	3.5	8.4	0.0	0.000	0.000	9.00	0.0
10.0	19.7	11.3	8.4	1.4	15.939	-4.529	4.70	52.0
15.0	32.9	23.5	9.4	1.6	18.482	-4.643	5.07	55.6
20.0	47.2	36.3	10.9	1.8	20.027	-3.726	5.31	57.2
25.0	65.9	52.0	14.0	2.2	22.009	-3.358	5.57	61.7
30.0	87.2	71.2	16.1	2.8	23.527	-2.592	5.81	60.0
35.0	112.0	93.8	18.1	3.7	25.018	-2.416	6.05	57.8
40.0	138.2	120.0	18.1	4.8	26.191	-2.306	6.28	56.0
45.0	167.8	149.7	18.1	5.9	26.994	-2.181	6.47	54.2
50.0	273.6	182.8	90.8	10.9	29.766	-2.236	7.31	52.8
55.0	310.2	219.4	90.8	12.2	30.305	-1.747	7.47	52.3
60.0	350.3	259.6	90.8	13.9	30.883	-1.546	7.65	52.3
65.0	393.0	302.3	90.8	15.9	31.489	-1.679	7.85	52.7
70.0	438.4	347.6	90.8	18.4	32.145	-2.704	8.07	53.0
75.0	487.2	396.5	90.8	21.7	32.902	-3.204	8.32	53.6
80.0	563.7	448.8	114.9	29.3	33.905	-3.685	8.68	54.8
85.0	593.6	502.9	90.8	32.4	34.199	-3.010	8.77	54.3
90.0	648.6	557.8	90.8	41.2	34.462	-2.577	8.88	53.9
95.0	705.3	614.5	90.8	52.8	35.078	-1.986	9.11	54.4
96.0	723.8	626.1	97.7	59.0	35.123	-1.758	9.17	54.3
101.0	810.5	684.8	125.7	115.1	35.529	-0.889	9.33	53.9

Total Continuous Driving Time 42.00 minutes; Total Number of Blows 1724 (starting at penetration 5.0 ft)

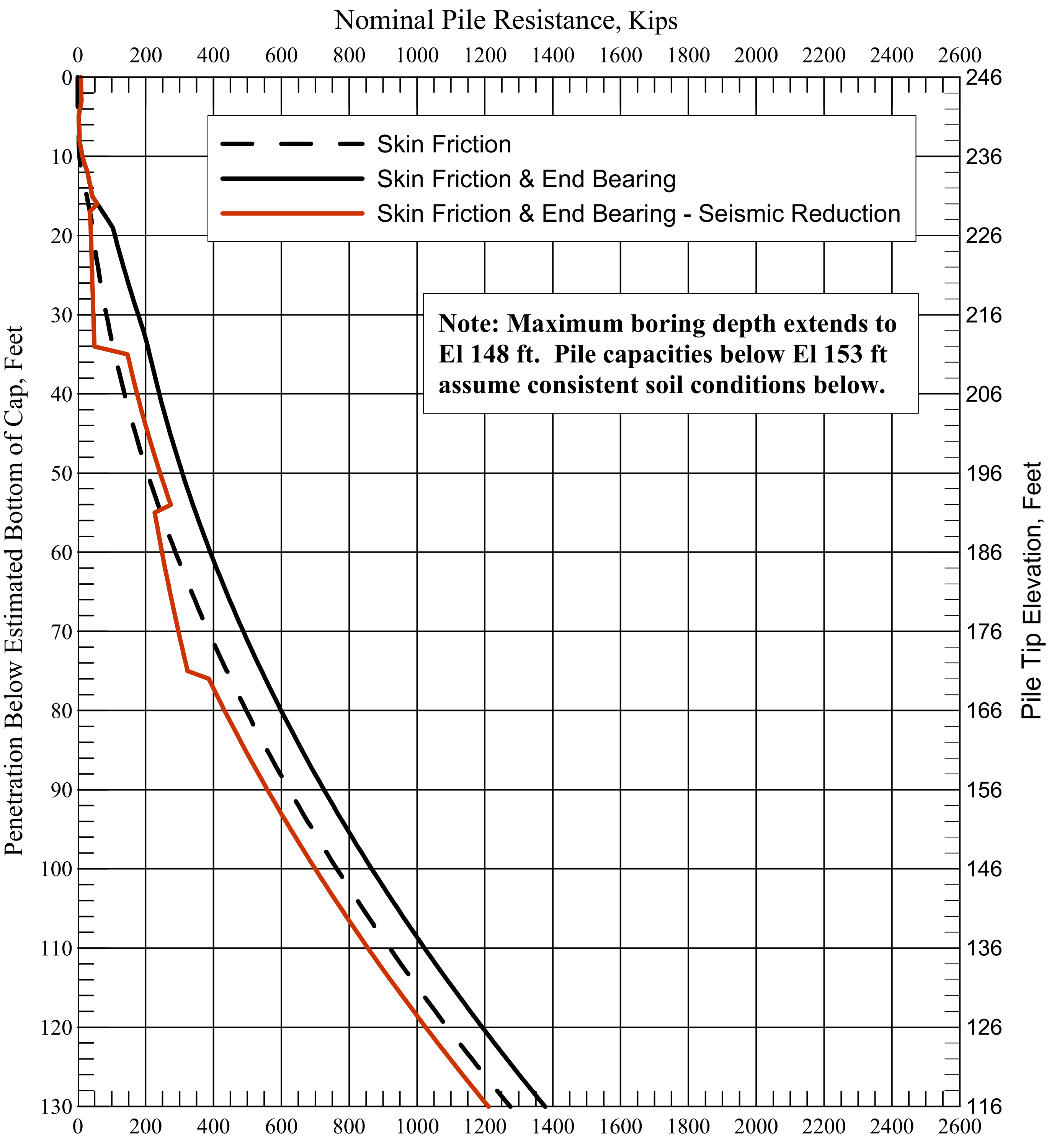
APPENDIX B

Site 2: Pile Axial Capacity Curves and Drivability Analysis Results



Note: Maximum boring depth extends to El 148 ft. Pile capacities below El 153 ft assume consistent soil conditions below.

NOMINAL PILE RESISTANCE CURVES
SITE 2
WEST ABUTMENT
16" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS



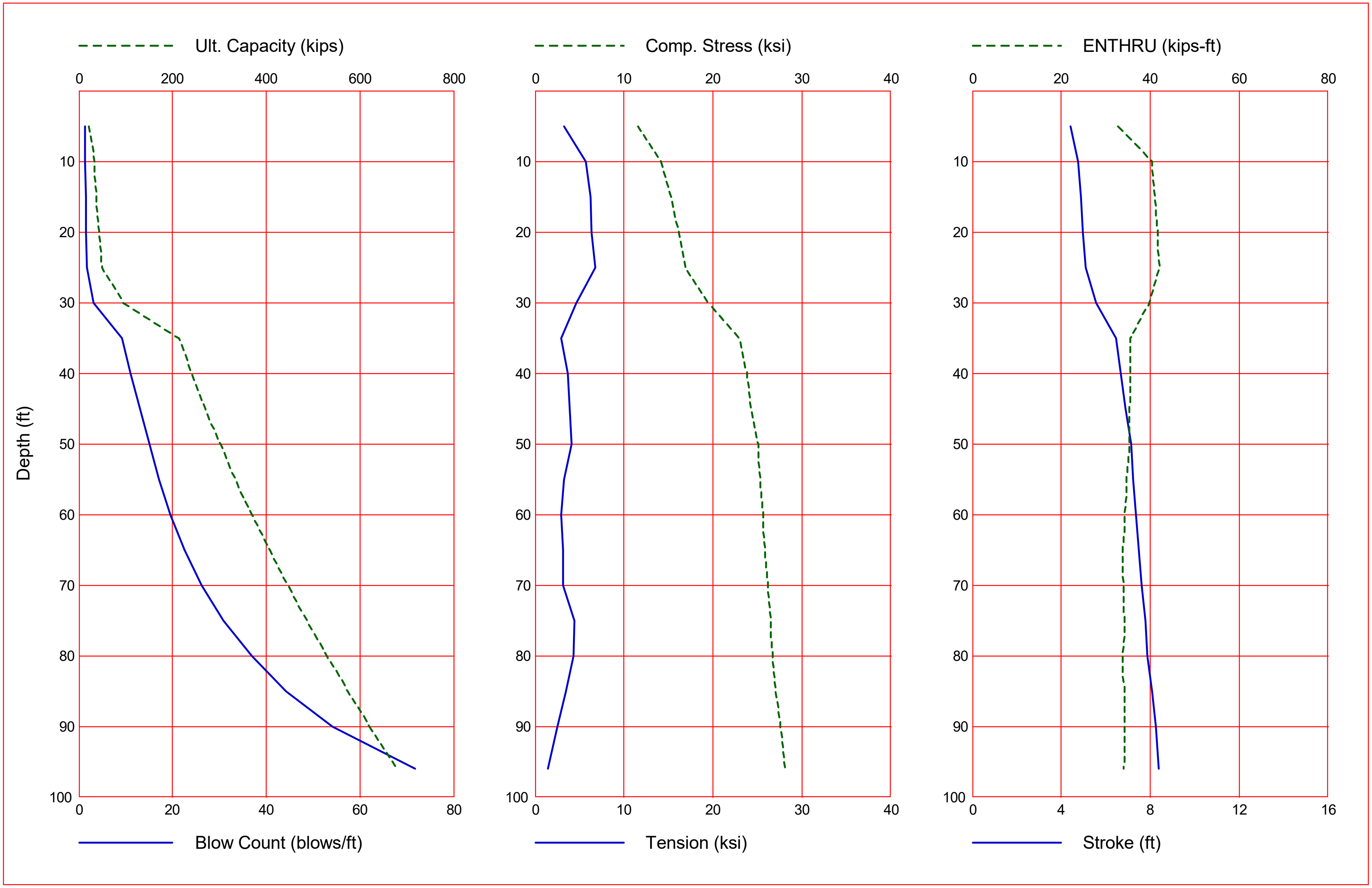
NOMINAL PILE RESISTANCE CURVES
SITE 2
EAST ABUTMENT
16" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

FIGURE 28

Burns Cooley Dennis, Inc.
Site 2- West Abutment - DELMAG D30

Jun 30 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

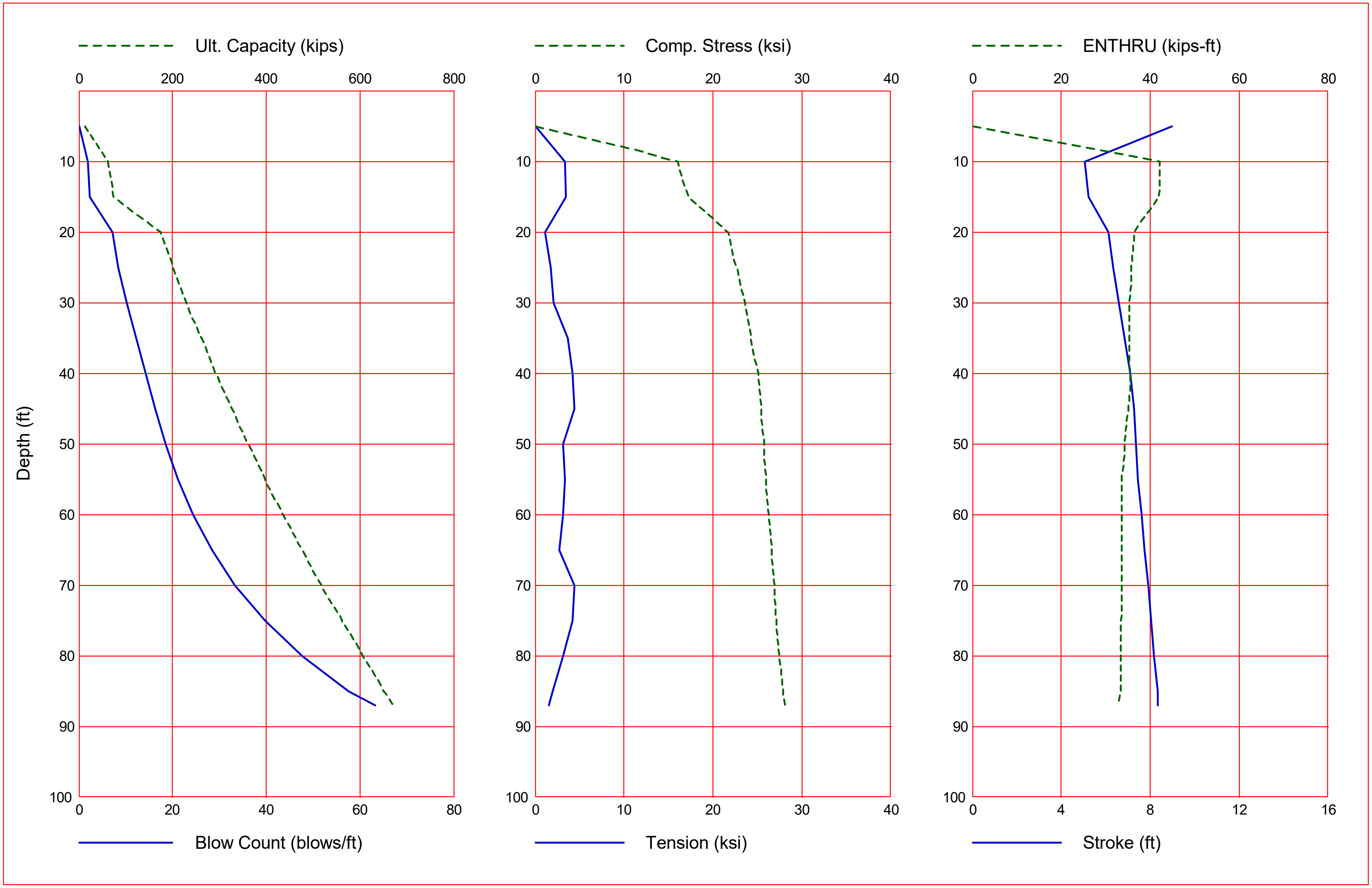
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	21.4	11.1	10.4	1.3	11.526	-3.264	4.42	32.8
10.0	32.7	22.3	10.4	1.4	14.184	-5.698	4.75	40.4
15.0	38.3	32.7	5.6	1.5	15.297	-6.203	4.87	41.1
20.0	44.4	38.9	5.6	1.6	16.110	-6.339	4.97	41.6
25.0	50.5	45.0	5.6	1.7	16.959	-6.728	5.11	42.1
30.0	93.5	55.8	37.7	3.2	19.427	-4.674	5.57	39.8
35.0	213.2	80.6	132.6	9.2	23.007	-2.946	6.46	35.6
40.0	240.9	108.3	132.6	11.0	23.813	-3.732	6.67	35.5
45.0	270.5	137.9	132.6	13.0	24.306	-3.886	6.87	35.2
50.0	302.0	169.3	132.6	15.1	25.138	-4.129	7.13	35.3
55.0	335.3	202.7	132.6	17.1	25.322	-3.230	7.25	34.7
60.0	370.5	237.9	132.6	19.5	25.680	-2.919	7.36	34.2
65.0	407.6	274.9	132.6	22.5	25.853	-3.166	7.48	33.8
70.0	446.5	313.9	132.6	26.3	26.231	-3.149	7.62	34.1
75.0	487.3	354.7	132.6	30.8	26.467	-4.450	7.77	34.2
80.0	530.0	397.3	132.6	37.0	26.740	-4.285	7.89	33.9
85.0	574.5	441.9	132.6	44.3	27.073	-3.480	8.09	34.2
90.0	620.9	488.3	132.6	54.1	27.553	-2.498	8.24	34.3
96.0	679.1	546.4	132.6	71.7	28.101	-1.435	8.39	34.0

Total Continuous Driving Time 42.00 minutes; Total Number of Blows 1791 (starting at penetration 5.0 ft)

Burns Cooley Dennis, Inc.
Site 2- East Abutment - DELMAG D30

Jun 30 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Burns Cooley Dennis, Inc.
Site 2- East Abutment - DELMAG D30

Jun 30 2021
GRLWEAP Version 2010

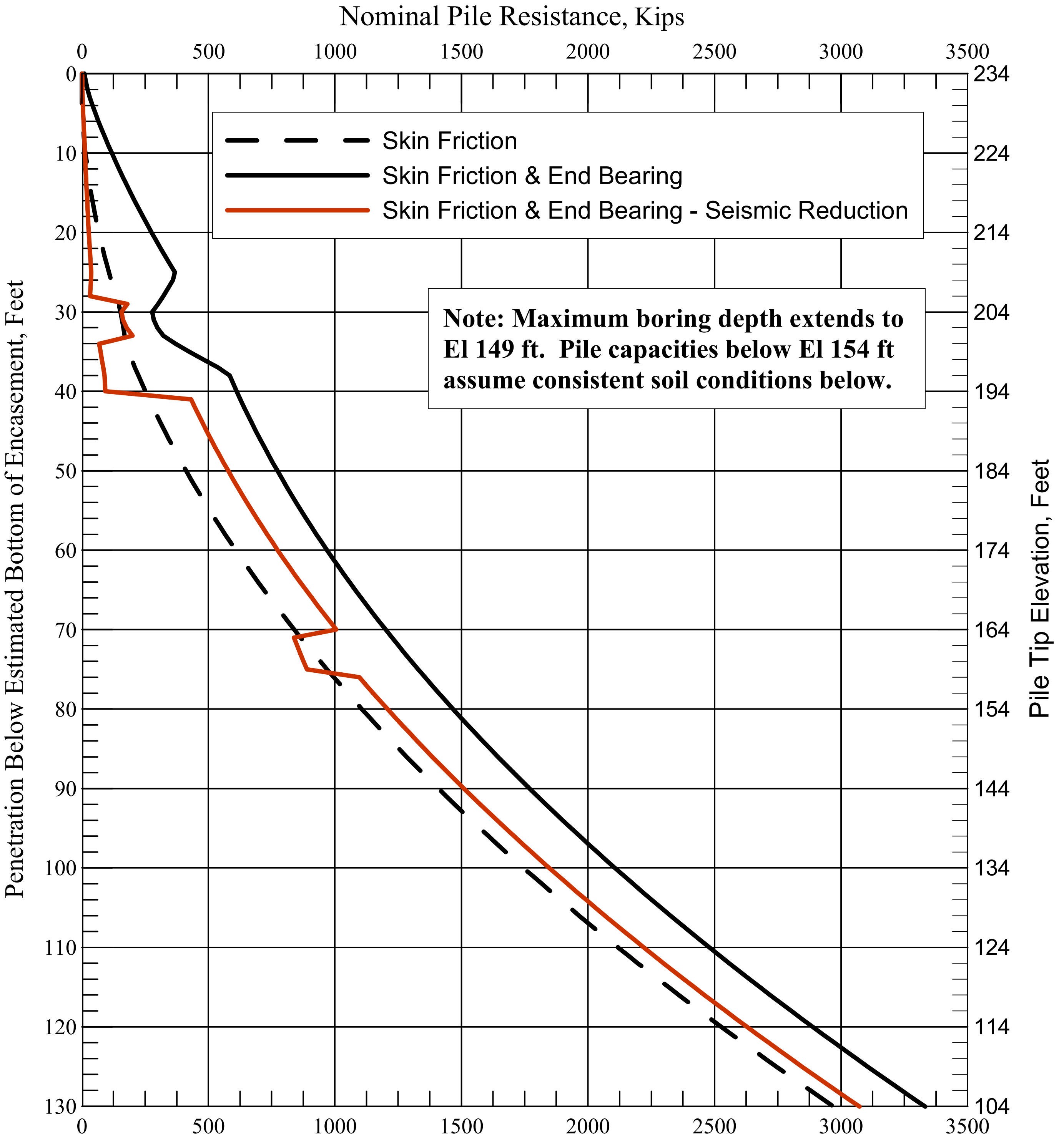
Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	12.2	8.7	3.5	0.0	0.000	0.000	9.00	0.0
10.0	61.5	15.4	46.1	1.9	16.011	-3.392	5.04	42.2
15.0	74.9	28.8	46.1	2.4	17.278	-3.494	5.24	41.9
20.0	174.4	50.2	124.3	7.1	21.735	-1.070	6.11	36.3
25.0	201.9	77.6	124.3	8.5	22.673	-1.754	6.35	35.7
30.0	230.8	106.6	124.3	10.2	23.587	-2.057	6.60	35.4
35.0	261.4	137.1	124.3	12.2	24.257	-3.710	6.83	35.4
40.0	293.4	169.2	124.3	14.3	25.123	-4.248	7.11	35.5
45.0	327.0	202.8	124.3	16.4	25.489	-4.415	7.27	35.0
50.0	362.2	237.9	124.3	18.6	25.757	-3.129	7.37	34.2
55.0	398.9	274.6	124.3	21.2	25.971	-3.365	7.46	33.7
60.0	437.2	312.9	124.3	24.5	26.250	-3.152	7.61	33.5
65.0	477.0	352.7	124.3	28.5	26.582	-2.690	7.76	33.7
70.0	518.3	394.0	124.3	33.3	26.897	-4.395	7.92	33.7
75.0	561.2	436.9	124.3	39.8	27.162	-4.252	8.06	33.4
80.0	605.7	481.4	124.3	47.6	27.449	-3.126	8.19	33.3
85.0	651.6	527.4	124.3	57.7	27.893	-1.985	8.32	33.3
87.0	670.5	546.2	124.3	63.2	28.103	-1.598	8.36	33.0

Total Continuous Driving Time 40.00 minutes; Total Number of Blows 1699 (starting at penetration 5.0 ft)

APPENDIX C

Site 5: Pile Axial Capacity Curves and Drivability Analysis Results



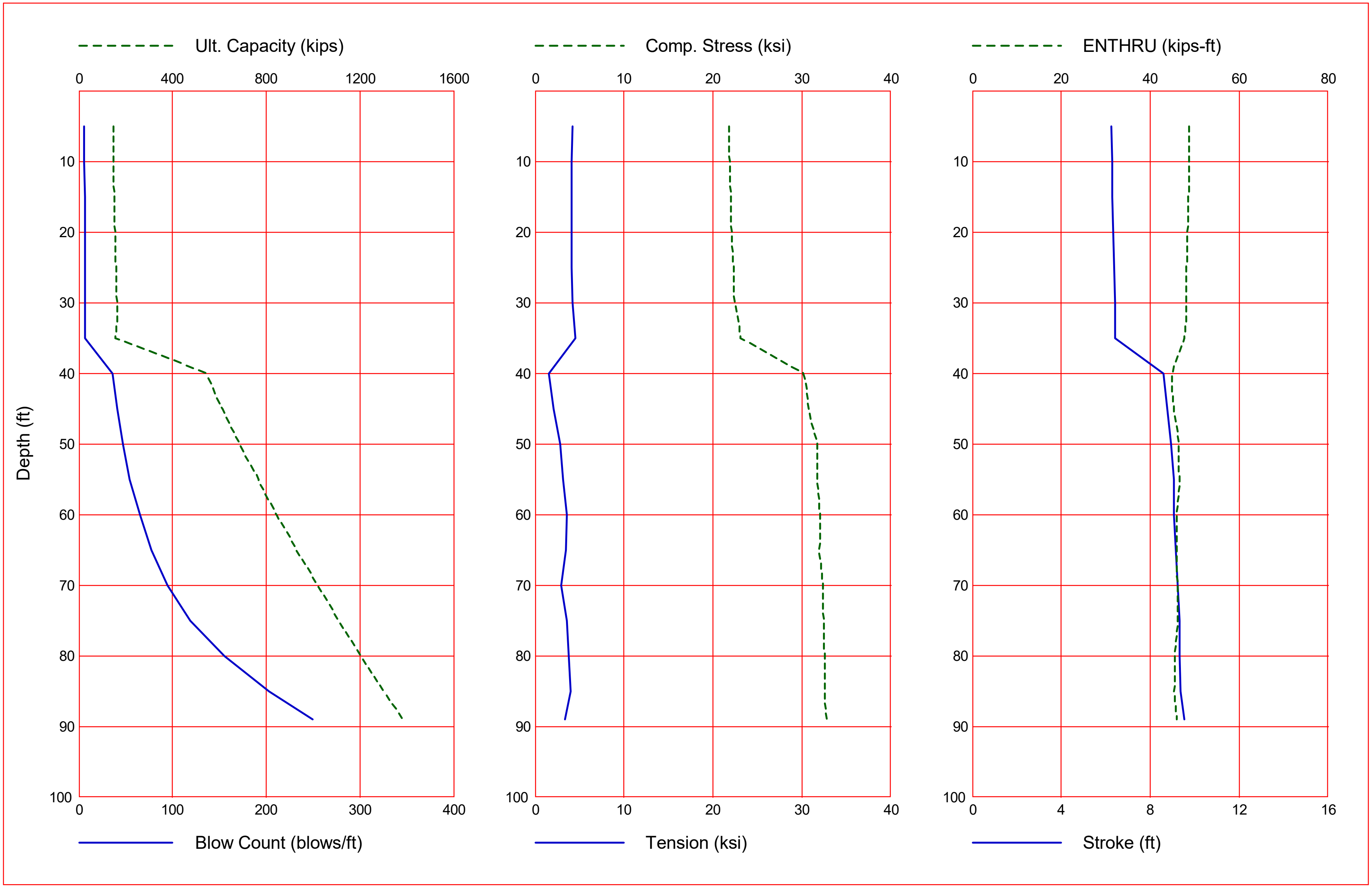
NOMINAL PILE RESISTANCE CURVES
SITE 5
INTERIOR BENTS
30" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

FIGURE 32

Burns Cooley Dennis, Inc.
Site 5 - Interior Bents - DELMAG D46

Jun 30 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



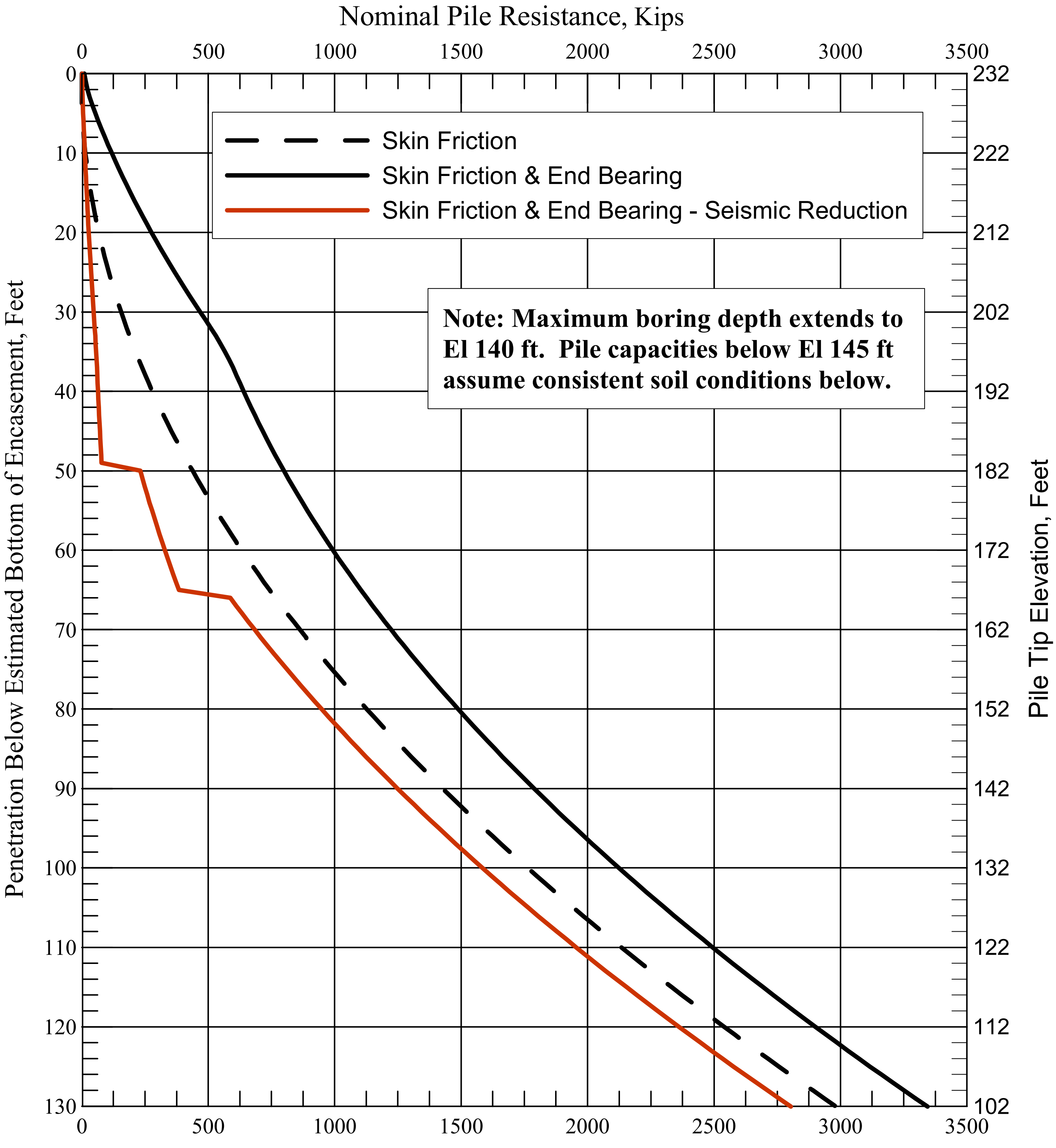
Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	146.1	3.6	142.5	5.9	21.842	-4.167	6.27	48.8
10.0	149.7	7.3	142.5	6.0	21.938	-4.129	6.29	48.7
15.0	153.4	10.9	142.5	6.2	22.053	-4.118	6.31	48.5
20.0	157.0	14.5	142.5	6.3	22.180	-4.126	6.34	48.4
25.0	160.7	18.2	142.5	6.5	22.314	-4.141	6.37	48.2
30.0	164.3	21.8	142.5	6.6	22.452	-4.183	6.40	48.0
35.0	155.3	57.1	98.2	6.4	23.088	-4.578	6.44	47.6
40.0	544.8	107.9	436.9	36.0	30.277	-1.596	8.61	44.9
45.0	614.9	178.0	436.9	41.3	30.787	-2.125	8.77	45.4
50.0	688.5	251.6	436.9	47.3	31.747	-2.780	8.95	46.3
55.0	765.6	328.8	436.9	54.6	31.734	-3.133	9.08	46.7
60.0	846.3	409.4	436.9	65.2	32.014	-3.561	9.07	45.9
65.0	930.4	493.6	436.9	77.6	31.974	-3.433	9.15	46.0
70.0	1018.1	581.3	436.9	94.8	32.423	-2.895	9.23	46.1
75.0	1109.3	672.5	436.9	119.3	32.454	-3.591	9.31	46.1
80.0	1204.1	767.2	436.9	155.9	32.614	-3.747	9.33	45.6
85.0	1302.3	865.4	436.9	203.0	32.589	-3.983	9.38	45.4
89.0	1383.4	946.5	436.9	249.9	32.810	-3.359	9.53	46.0

Total Continuous Driving Time 130.00 minutes; Total Number of Blows 5077 (starting at penetration 5.0 ft)

APPENDIX D

Site 8: Pile Axial Capacity Curves and Drivability Analysis Results



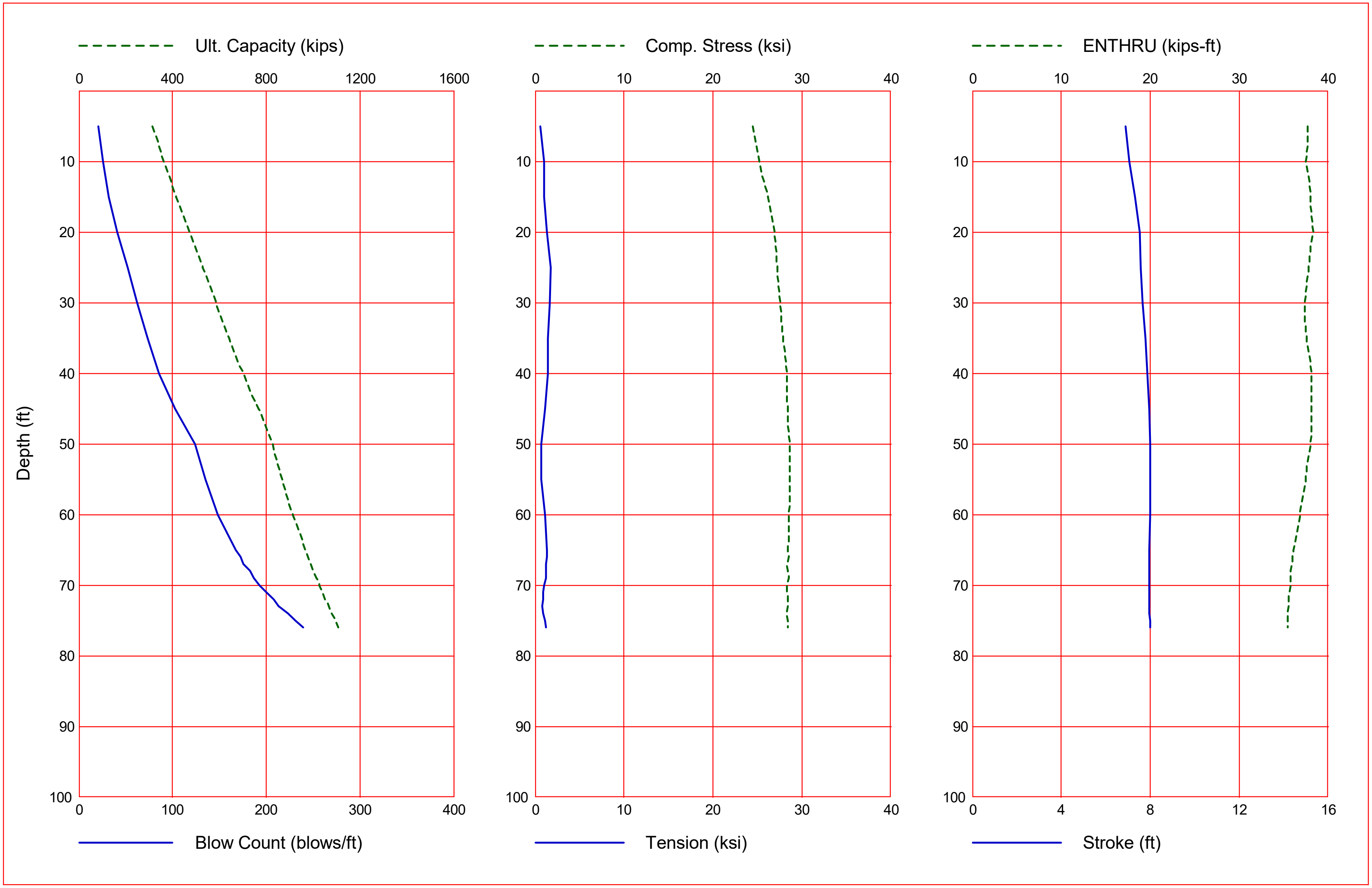
NOMINAL PILE RESISTANCE CURVES
SITE 8
INTERIOR BENTS
30" PIPE PILES, CLOSED-ENDED
ARDOT SR230 BRIDGE REPLACEMENTS
CRAIGHEAD AND LAWRENCE COUNTIES, ARKANSAS

FIGURE 30

Burns Cooley Dennis, Inc.
Site 8 - Interior Bents - DELMAG D46

Jun 30 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	315.4	24.5	290.8	20.6	24.524	-0.555	6.91	37.7
10.0	364.4	49.1	315.4	25.8	25.176	-1.043	7.08	37.5
15.0	415.6	75.6	339.9	31.9	26.185	-1.061	7.32	38.0
20.0	470.8	106.3	364.5	40.9	26.941	-1.316	7.52	38.3
25.0	528.0	139.0	389.0	52.1	27.224	-1.746	7.59	37.8
30.0	585.3	171.7	413.6	61.8	27.566	-1.621	7.67	37.4
35.0	643.4	205.3	438.1	73.5	27.873	-1.449	7.78	37.6
40.0	703.1	240.4	462.6	85.6	28.279	-1.478	7.89	38.1
45.0	766.1	276.4	489.6	102.3	28.428	-1.152	7.95	38.1
50.0	829.0	312.4	516.6	123.8	28.609	-0.642	8.00	38.0
55.0	868.3	351.7	516.6	135.1	28.611	-0.657	8.00	37.5
60.0	914.3	397.7	516.6	148.0	28.570	-1.147	7.99	36.9
65.0	966.9	450.2	516.6	167.1	28.475	-1.348	7.96	36.1
66.0	978.2	461.6	516.6	172.3	28.487	-1.328	7.95	36.0
67.0	989.8	473.2	516.6	175.9	28.354	-1.224	7.95	36.0
68.0	1001.8	485.1	516.6	182.5	28.407	-1.195	7.95	35.8
69.0	1014.0	497.4	516.6	186.7	28.500	-1.189	7.95	35.8
70.0	1026.6	510.0	516.6	192.9	28.356	-1.020	7.96	35.8
71.0	1039.5	522.9	516.6	199.8	28.346	-0.916	7.96	35.7
72.0	1052.8	536.1	516.6	207.7	28.404	-0.871	7.96	35.6
73.0	1066.4	549.7	516.6	213.4	28.397	-0.813	7.96	35.6
74.0	1080.3	563.6	516.6	222.8	28.349	-0.937	7.97	35.5
75.0	1094.5	577.9	516.6	230.9	28.447	-1.106	7.98	35.5
76.0	1109.1	592.4	516.6	239.5	28.443	-1.201	7.98	35.5

Total Continuous Driving Time 169.00 minutes; Total Number of Blows 7063 (starting at penetration 5.0 ft)

BURNS COOLEY DENNIS, INC.

GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

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Ridgeland, MS 39157
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■

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Ridgeland, MS 39157
Phone: (601) 856-2332
Fax: (601) 856-3552

June 11, 2021

Cindy Rich, P.E.
Neel-Schaffer
125 South Congress Street, Suite 1100
Post Office Box 22625
Jackson, Mississippi 39201

Project No. 200518

Re: Box Culvert Boring Summary
ARDOT SR230 – Alicia to Bono
Lawrence and Craighead Counties, Arkansas

Dear Ms. Rich:

Plans are being made for the construction of box culverts at six sites along Highway 230 between Alicia and Bono in Lawrence and Craighead Counties in Arkansas. The soil conditions near the planned box culverts at these six sites, which are designated Sites 3, 4, 6, 7, 9, and 10, were explored by means of six borings, i.e., one boring per site. The borings were performed by representatives of McCray Drilling, SoilTech Consultants, Inc. and Burns Cooley Dennis, Inc. The boring locations and depths are presented in the summary table below.

Site	Boring No.	Station	Offset	GPS Coordinates		Boring Depth (ft)
3	S3-1	311+61	19' LT	N 35° 53' 52.4"	W 90° 57' 40.9"	5
4	S4-1	410+67	11' LT	N 35° 54' 41.2"	W 90° 55' 2.2"	25
6	S6-1	614+10	9' LT	N 35° 54' 36.6"	W 90° 52' 37.4"	25
7	S7-1	705+63	18' LT	N 35° 54' 36.3"	W 90° 52' 24.5"	25
9	S9-1	906+43	8' LT	N 35° 54' 33.3"	W 90° 50' 51.7"	25
10	S10-1	1005+49	39' RT	N 35° 54' 32.0"	W 90° 48' 34.9"	25

Aerial images showing the boring location at each site are presented on Figures 1A through 1F attached to this letter. A synopsis of the Unified Soil Classification System (USCS) is presented on Figure 2 along with symbols and terminology typically utilized on graphical soil boring logs. Graphical logs of the borings are presented on Figures 3A through 3F. The aerial images also show the boring locations for additional roadway subgrade borings that were performed. Details of the roadway subgrade borings are presented in a previously sent report for this project dated June 1, 2021.

We appreciate the opportunity to be of service. If you should have any questions concerning this letter, please do not hesitate to call us.

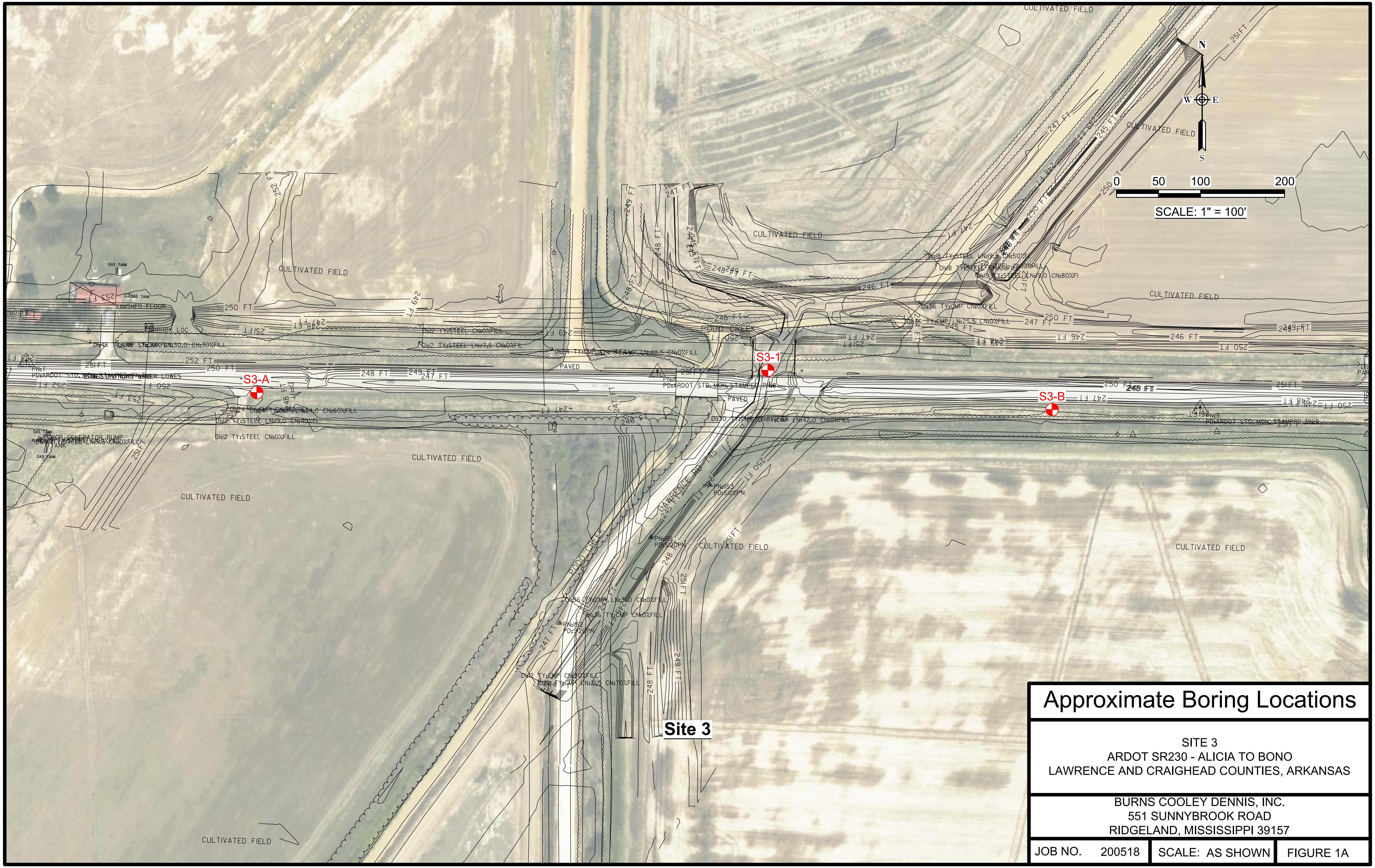
Very truly yours,

BURNS COOLEY DENNIS, INC.

Alexander B. Reeb, Ph.D., P.E.

A. E. (Eddie) Templeton, P.E.

AET/khb
Copy Submitted: (via e-mail)
Attachment

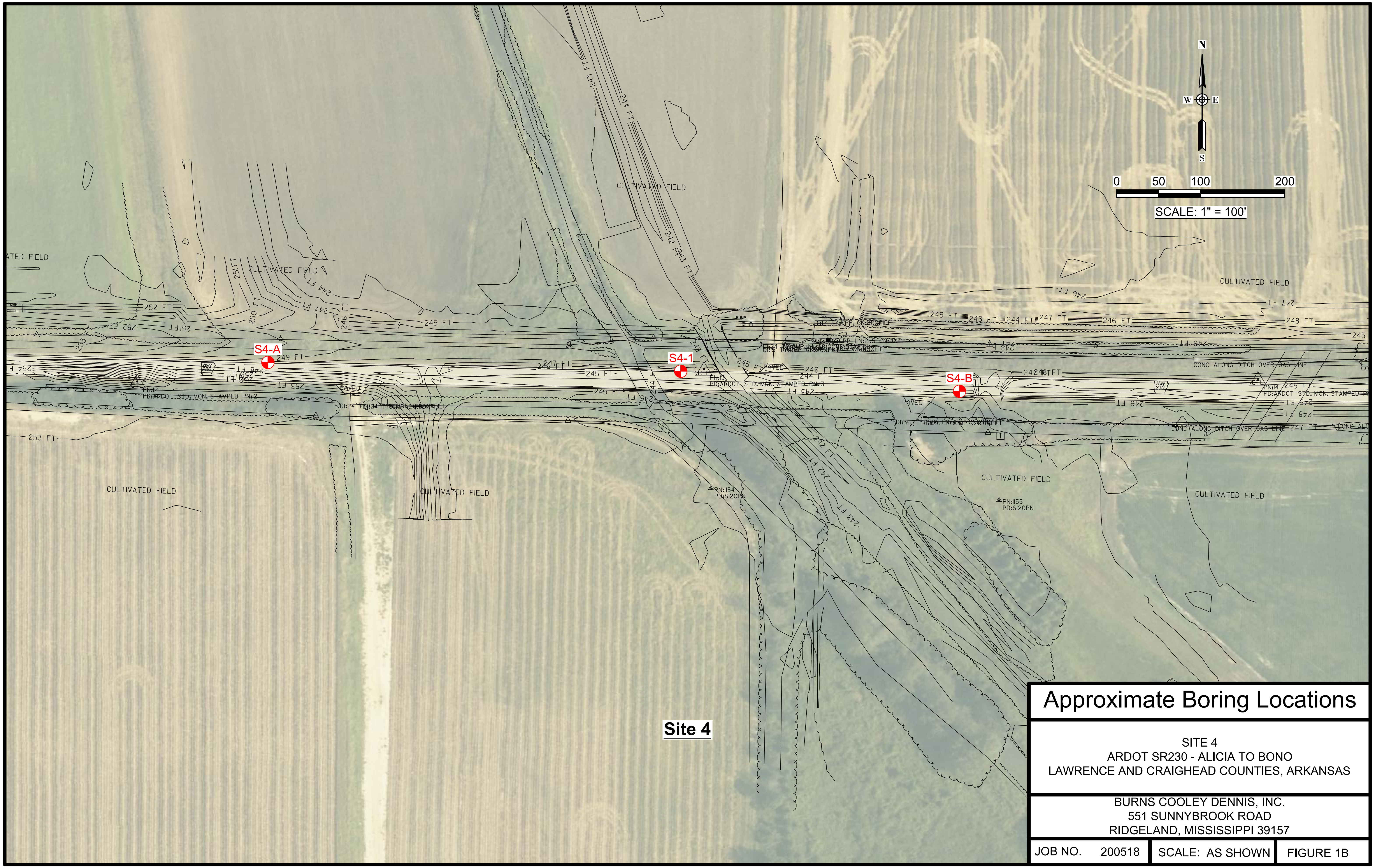


Approximate Boring Locations

SITE 3
ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO. 200518 | SCALE: AS SHOWN | FIGURE 1A

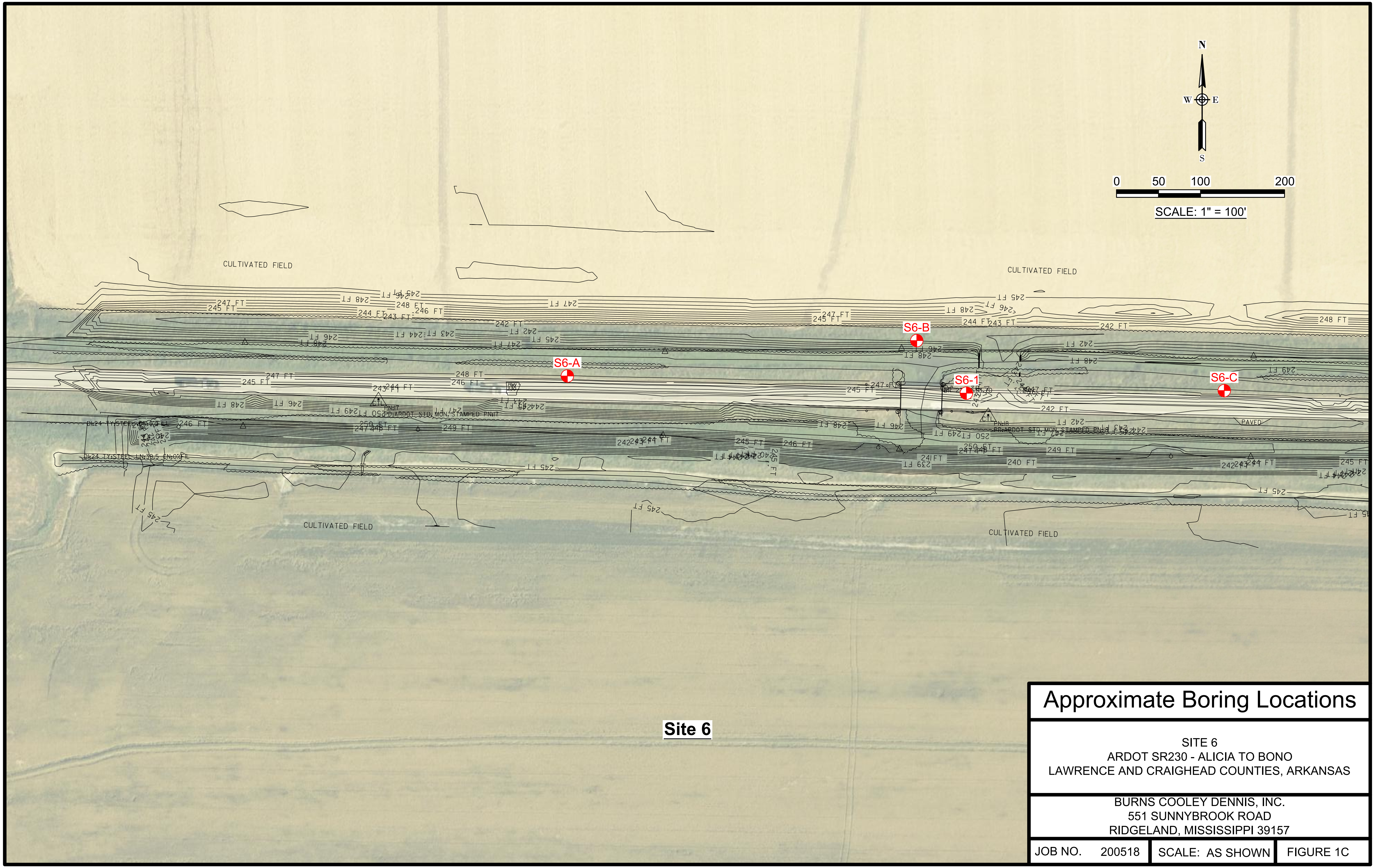


Approximate Boring Locations

SITE 4
ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO. 200518	SCALE: AS SHOWN	FIGURE 1B



Approximate Boring Locations

SITE 6
ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO. 200518	SCALE: AS SHOWN	FIGURE 1C

Approximate Boring Locations

SITE 7
ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO.	200518	SCALE: AS SHOWN	FIGURE 1D

Approximate Boring Locations

SITE 9
ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO. 200518	SCALE: AS SHOWN	FIGURE 1E

Approximate Boring Locations

SITE 10
ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO. 200518 | SCALE: AS SHOWN | FIGURE 1F

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL & LETTER	DESCRIPTION
COARSE-GRAINED SOILS More than half of material larger than No. 200 sieve size	GRAVELS More than half of coarse fraction larger than No.4 sieve size	Clean Gravels (Little or no fines)	GW	WELL GRADED GRAVEL, GRAVEL-SAND MIXTURE
			GP	POORLY GRADED GRAVEL, GRAVEL-SAND MIXTURE
		Gravels with fines (Appreciable amount of fines)	GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURE
			GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY MIXTURE
	SANDS More than half of coarse fraction smaller than No.4 sieve size	Clean Sands (Little or no fines)	SW	WELL GRADED SAND, GRAVELLY SAND
			SP	POORLY GRADED SAND, GRAVELLY SAND
		Sands with fines (Appreciable amount of fines)	SM	SILTY SAND, SAND-SILT MIXTURE
			SC	CLAYEY SAND, SAND-CLAY MIXTURE
FINE-GRAINED SOILS More than half of material smaller than No. 200 sieve size	SILTS AND CLAYS	Liquid limit less than 50	ML	SILT WITH LITTLE OR NO PLASTICITY
			ML	CLAYEY SILT, SILT WITH SLIGHT TO MEDIUM PLASTICITY
			ML	SANDY SILT
			CL	SILTY CLAY, LOW TO MEDIUM PLASTICITY
			CL	SANDY CLAY, LOW TO MEDIUM PLASTICITY (30% TO 50% SAND)
	SILTS AND CLAYS	Liquid limit greater than 50	MH	SILT, HIGH PLASTICITY
			CH	CLAY, HIGH PLASTICITY
			OH	ORGANIC CLAY OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOIL

TERMS CHARACTERIZING SOIL STRUCTURE

Slickensided	- Clays with polished and striated planes created as a result of volume changes related to shrinking, swelling and/orchanges in overburden pressure.
Fissured	- Clays with a blocky or jointed structure generally created by seasonal shrinking and swelling.
Laminated	- Composed of thin alternating layers of varying color and texture.
Calcareous	- Containing appreciable quantities of calcium carbonate.
Parting	- Paper thin (less than 1/8 inch).
Seam	- 1/8 inch to 3 inch thickness.
Layer	- Greater than 3 inches in thickness.

TERMS CHARACTERIZING SOIL STRUCTURE

COARSE-GRAINED SOILS		FINE-GRAINED SOILS		
DENSITY	PENETRATION RESISTANCE, N Blows per Foot	Consistancy	COHESION Kips/Sq.Ft	PENETRATION RESISTANCE, N Blows per Foot
Very loose	0 - 4	Very Soft	<0.25	0 - 1
Loose	5 - 10	Soft	0.25 - 0.50	2 - 4
Medium Dense	11 - 30	Medium Stiff	0.50 - 1.00	5 - 8
Dense	31 - 50	Stiff	1.00 - 2.00	9 - 15
Very Dense	>4.00	Very Stiff	2.00 - 4.00	16 - 30
		Hard	>4.00	>30

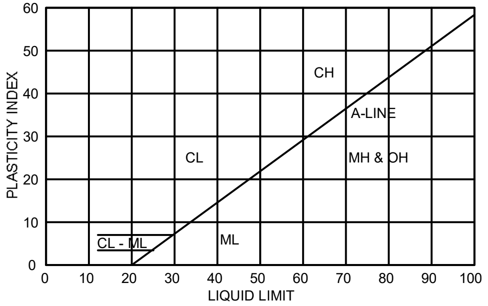
PARTICLE SIZE IDENTIFICATION

Cobbles	- Greater than 3 inches
Gravel	- Coarse-3/4 inch to 3 inches Fine-4.76 mm to 3/4 inch
Sand	- Coarse-2 mm to 4.76 mm Medium-0.42 mm to 2 mm Fine-0.074 mm to 0.42 mm
Silt & Clay	- Less than 0.074 mm

RELATIVE COMPOSITION

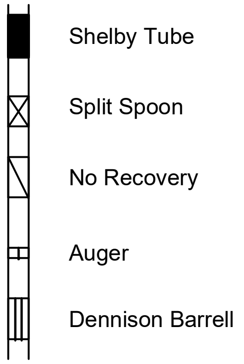
Slightly	5 - 15%
With	16 - 29%
Sandy (or gravelly)	30 - 50%

PLASTICITY CHART



FOR CLASSIFICATION OF FINE GRAINED SOILS

SAMPLE TYPES
(Shown in Sample Column)



CLASSIFICATION, SYMBOLS AND
TERMS USED ON GRAPHICAL
BORING LOGS

LOG OF BORING NO. S3-1

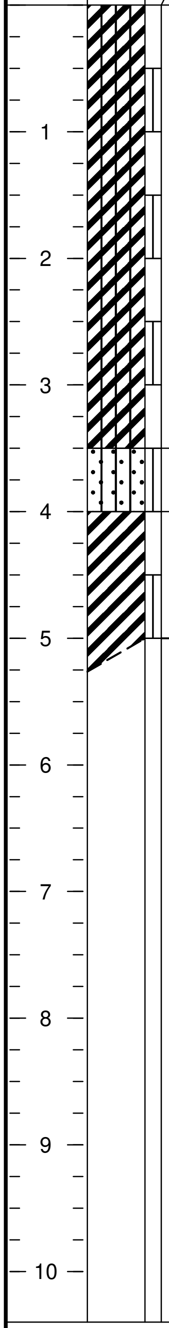
ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

TYPE: 4" Hand auger

LOCATION: Sta. 311+61
+/- 19' Left of Construction C/L

DEPTH, ft | SYMBOL | SAMPLES | DESCRIPTION OF MATERIAL | BLOWS PER FT | DRY DENSITY LBS/CU FT | ○ - UC Cohesion, kips/sq ft △ - UU (1, 2, 3, 4) | PLASTIC LIMIT, WATER CONTENT %, LIQUID LIMIT (20, 40, 60, 80) | % PASSING NO. 200 SIEVE

SURFACE EL: 243 ±ft



Stiff light gray and tan silty clay (CL), slightly sandy

- with rootlets below 1.5'

- with clay pockets below 2.5'

Loose tan silty fine sand (SM)

Stiff tan and light gray clay (CH) with fine sand pockets

BORING DEPTH: 5 ft

DATE: 03/01/21

COMMENTS: Borehole backfilled with cuttings.
GPS Coordinates
N 35° 53' 52.4"
W 90° 57' 40.9"

GROUNDWATER DATA: Free water encountered at an approximate depth of 2' during auger drilling. Water level remained at an approximate depth of 2' after about 15 minutes.

200518 6/11/2021 3:56:54 PM

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

FIGURE 3A

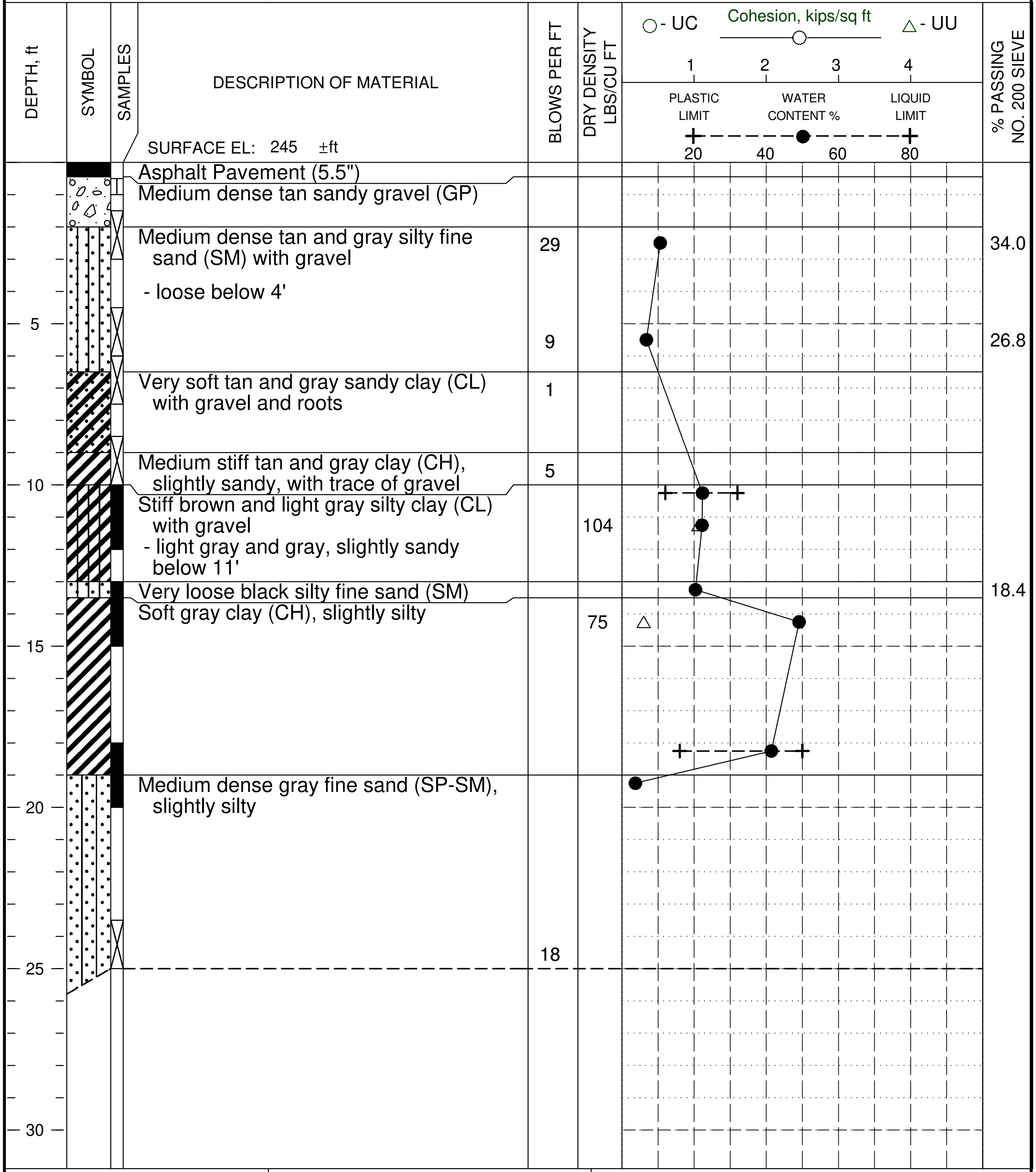
LOG OF BORING NO. S4-1

ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

TYPE: Hollow-stem auger

LOCATION: Sta. 410+76
+/- 11' Left of Construction C/L

SURFACE EL: 245 ±ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
Asphalt Pavement (5.5")			
Medium dense tan sandy gravel (GP)			
Medium dense tan and gray silty fine sand (SM) with gravel - loose below 4'	29 9		34.0 26.8
Very soft tan and gray sandy clay (CL) with gravel and roots	1		
Medium stiff tan and gray clay (CH), slightly sandy, with trace of gravel	5		
Stiff brown and light gray silty clay (CL) with gravel - light gray and gray, slightly sandy below 11'		104	
Very loose black silty fine sand (SM)			18.4
Soft gray clay (CH), slightly silty		75	
Medium dense gray fine sand (SP-SM), slightly silty	18		

BORING DEPTH: 25 ft

DATE: 09/21/20

COMMENTS: Borehole grouted. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 41.2"
W 90° 55' 2.2"

GROUNDWATER DATA: No free water encountered during auger drilling.

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GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

FIGURE 3B

LOG OF BORING NO. S6-1

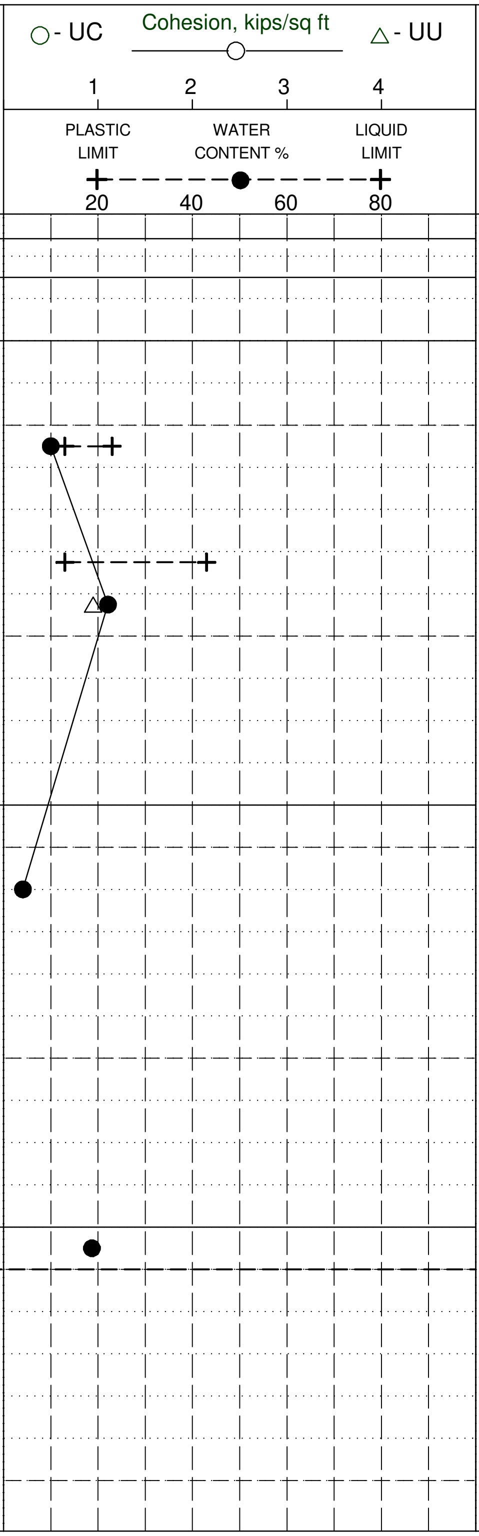
ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

TYPE: Hollow-stem auger

LOCATION: Sta. 614+10
+/- 9' Left of Construction C/L

SURFACE EL: 246 ±ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
Asphalt Pavement (7")			
Brown sandy gravel (GP), slightly clayey			
Medium dense gray and tan clayey fine to coarse sand (SC) with gravel	27		20.1
Medium stiff tan and brown sandy clay (CL) with gravel	6		
- tan and gray below 8.5'		102	
Loose gray, tan and red fine sand (SP) with clay seams	10		6.2
- medium dense below 18.5'	11		
- with trace of lignite at 20'			
Medium dense gray sand tan silty fine sand (SM)	13		21.5



BORING DEPTH: 25 ft

DATE: 09/21/20

COMMENTS: Borehole grouted. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 36.6"
W 90° 52' 37.4"

GROUNDWATER DATA: No free water encountered during auger drilling.

200518 6/11/2021 12:30:15 PM

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

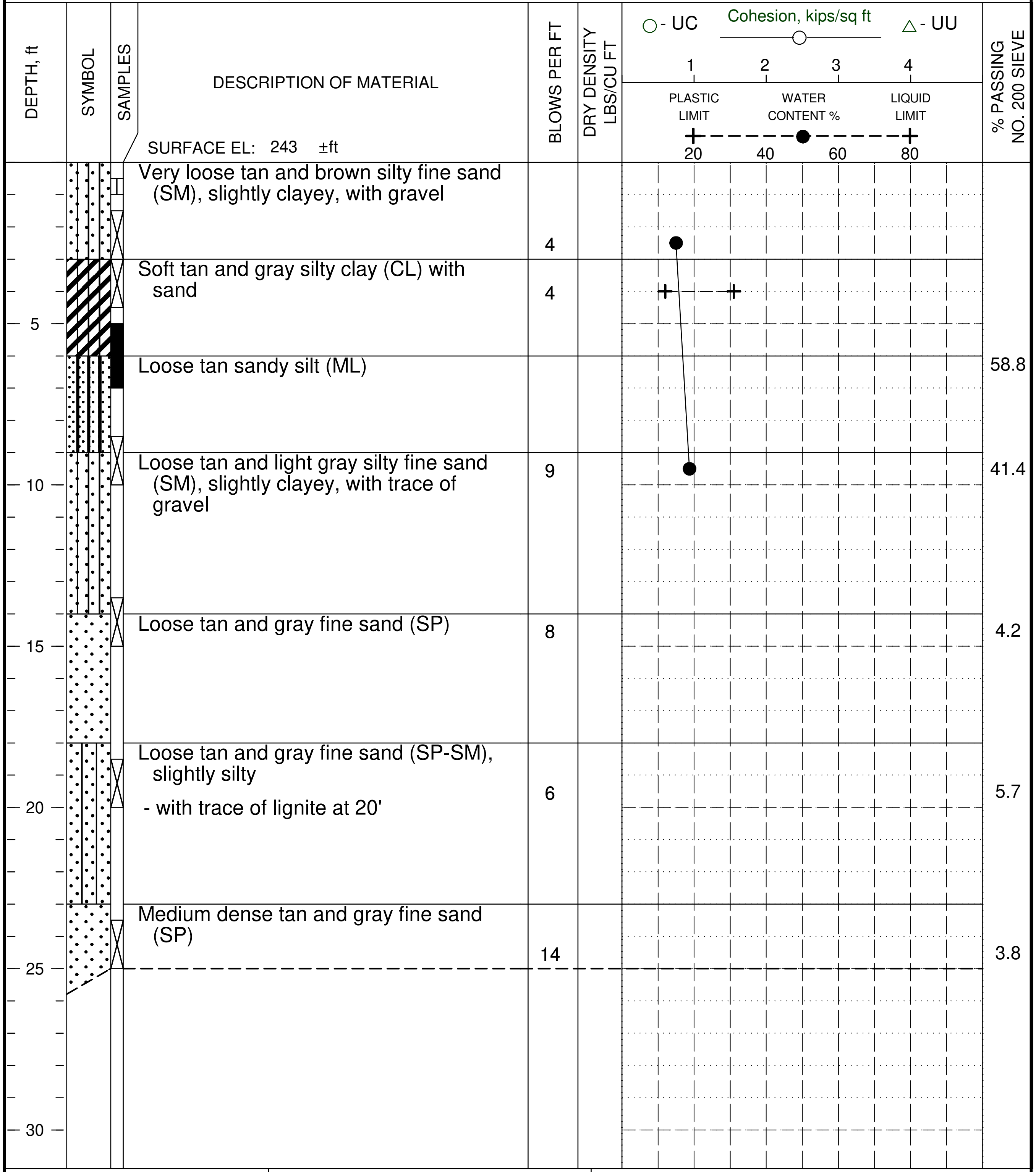
FIGURE 3C

LOG OF BORING NO. S7-1

ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

TYPE: Hollow-stem auger

LOCATION: Sta. 705+63
+/- 18' Left of Construction C/L



SURFACE EL: 243 ±ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	% PASSING NO. 200 SIEVE
Very loose tan and brown silty fine sand (SM), slightly clayey, with gravel	4	
Soft tan and gray silty clay (CL) with sand	4	
Loose tan sandy silt (ML)		58.8
Loose tan and light gray silty fine sand (SM), slightly clayey, with trace of gravel	9	41.4
Loose tan and gray fine sand (SP)	8	4.2
Loose tan and gray fine sand (SP-SM), slightly silty - with trace of lignite at 20'	6	5.7
Medium dense tan and gray fine sand (SP)	14	3.8

BORING DEPTH: 25 ft

DATE: 09/21/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 36.3"
W 90° 52' 24.5"

GROUNDWATER DATA: No free water encountered during auger drilling.

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FIGURE 3D

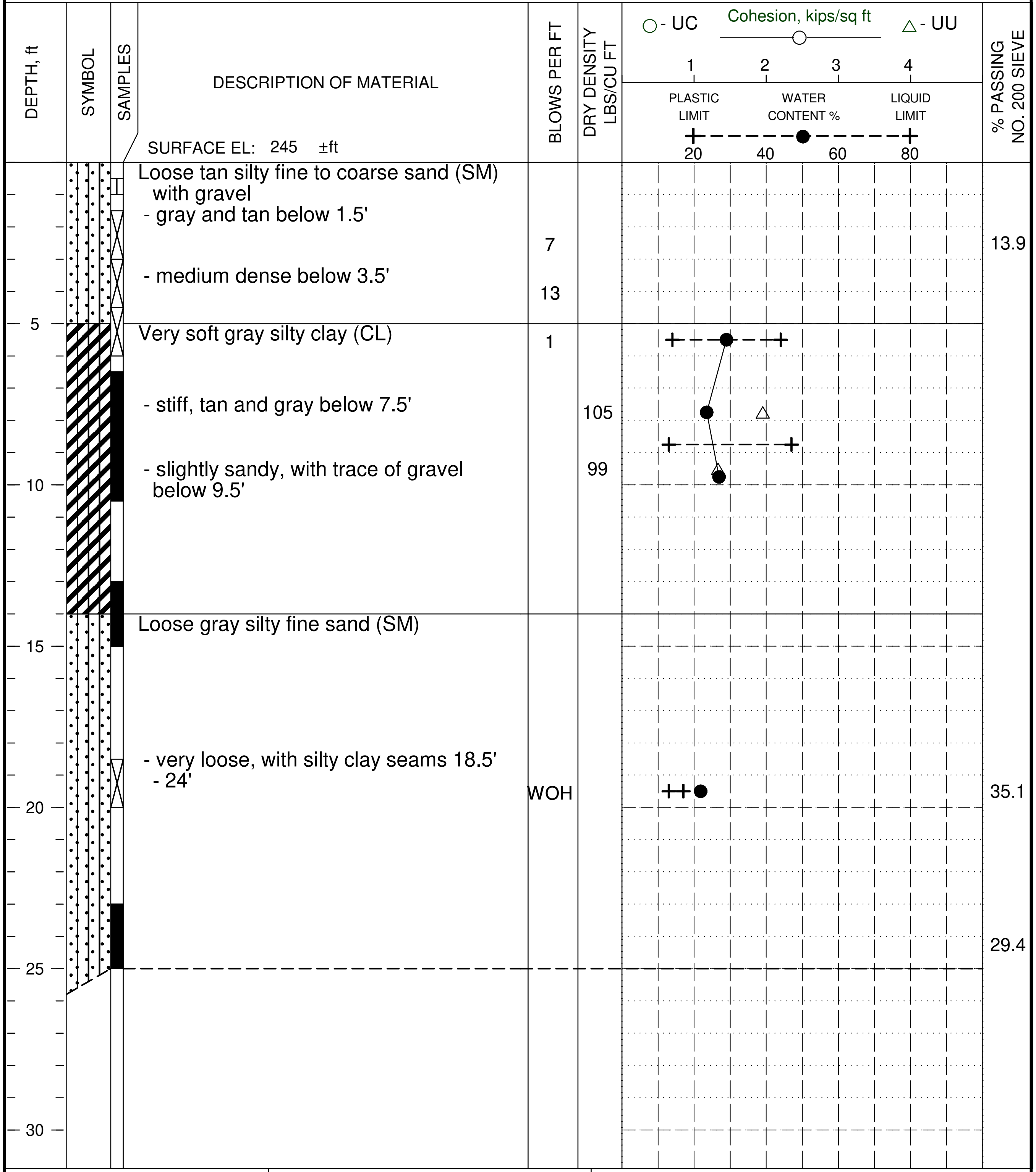
LOG OF BORING NO. S9-1

ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

TYPE: Hollow-stem auger

LOCATION: Sta. 906+43
+/- 8' Left of Construction C/L

SURFACE EL: 245 ±ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
Loose tan silty fine to coarse sand (SM) with gravel			
- gray and tan below 1.5'			
	7		13.9
- medium dense below 3.5'			
	13		
Very soft gray silty clay (CL)	1		
- stiff, tan and gray below 7.5'		105	
- slightly sandy, with trace of gravel below 9.5'		99	
Loose gray silty fine sand (SM)			
- very loose, with silty clay seams 18.5' - 24'			
	WOH		35.1
			29.4

BORING DEPTH: 25 ft

DATE: 09/22/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 33.3"
W 90° 50' 51.7"

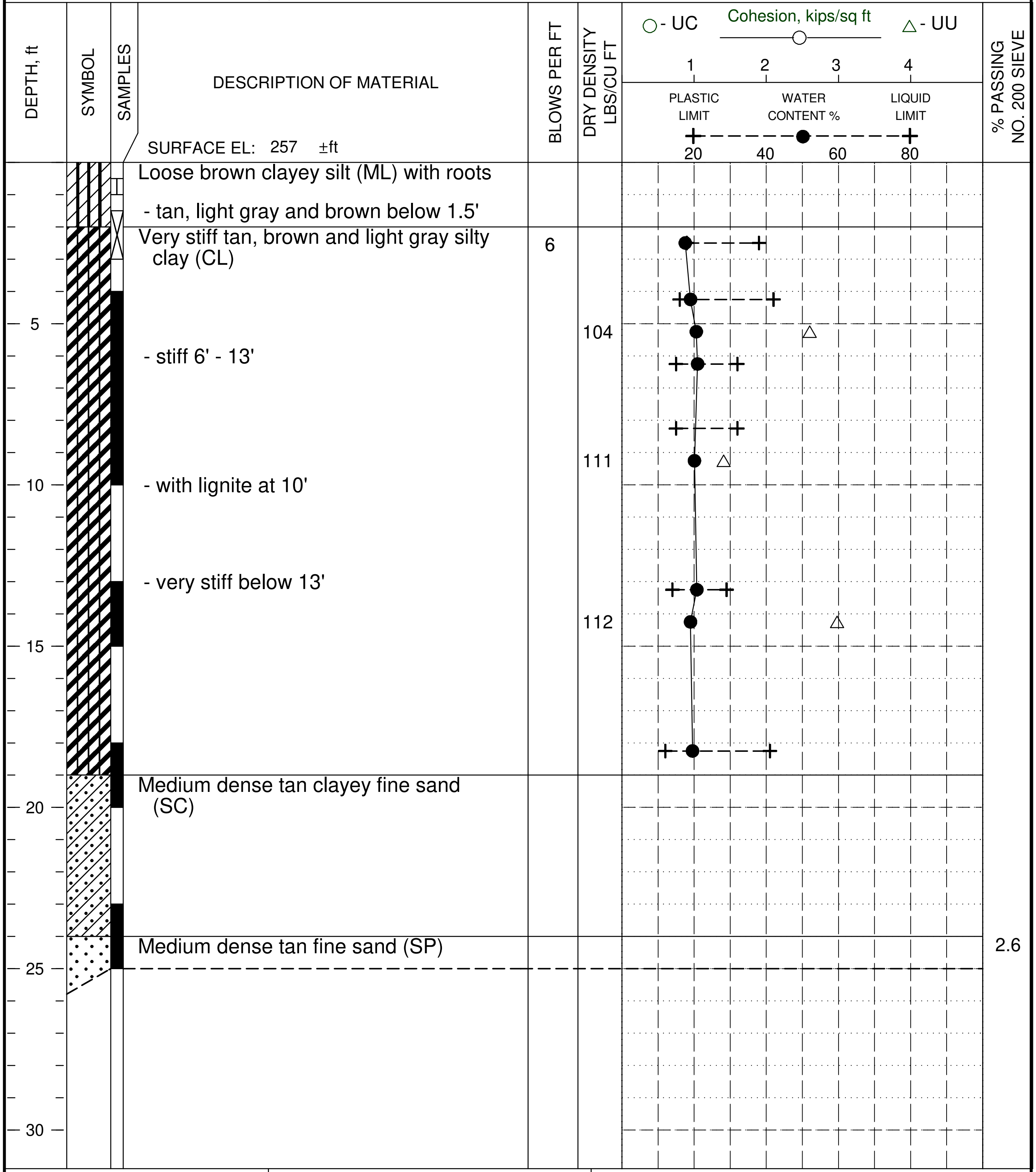
GROUNDWATER DATA: No free water encountered during auger drilling.

LOG OF BORING NO. S10-1

ARDOT SR230 - ALICIA TO BONO
LAWRENCE AND CRAIGHEAD COUNTIES, ARKANSAS

TYPE: Hollow-stem auger

LOCATION: Sta. 1005+49
+/- 39' Right of Construction C/L



SURFACE EL: 257 ±ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	DRY DENSITY LBS/CU FT	% PASSING NO. 200 SIEVE
Loose brown clayey silt (ML) with roots - tan, light gray and brown below 1.5'			
Very stiff tan, brown and light gray silty clay (CL) - stiff 6' - 13' - with lignite at 10' - very stiff below 13'	6	104 111 112	
Medium dense tan clayey fine sand (SC)			
Medium dense tan fine sand (SP)			2.6

BORING DEPTH: 25 ft

DATE: 09/22/20

COMMENTS: Borehole backfilled with cuttings. SPT performed with automatic hammer. A hammer energy correction factor of 1.36 applies.
GPS Coordinates
N 35° 54' 32.0"
W 90° 48' 34.9"

GROUNDWATER DATA: No free water encountered during auger drilling.

BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

FIGURE 3F

BURNS COOLEY DENNIS, INC.

GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

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Phone: (601) 856-2332
Fax: (601) 856-3552

May 28, 2021

Neel-Schaffer, Inc.
125 South Congress Street, Suite 1100
Jackson, Mississippi 39201

Attention: Keith Purvis, P.E.
Vice President, Transportation Department

BCD Project No. 200518

Re: Resilient Modulus Testing
Recompacted Subgrade Soils
ARDOT SR 230 – Alicia to Bono
Lawrence and Craighead Counties, Arkansas

Dear Mr. Purvis,

Submitted here is the BCD report presenting the laboratory test results of resilient modulus testing for typical representative subgrade soils encountered along the alignment of the referenced project. This laboratory testing was performed for Task Order 108 and was authorized by Subconsultant Agreement between Neel-Schaffer, Inc. and Burns Cooley Dennis, Inc.

We appreciate the opportunity to be of service. If you should have any questions, please do not hesitate to call us.

Very Truly Yours,
Burns Cooley Dennis, Inc.

L Allen Cooley Jr

L. Allen Cooley, Jr., Ph.D.

RC Ahlrich

R. C. Ahlrich, Ph.D., P.E.

A. E. (Eddie) Templeton, P.E.

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No.7833
ALEXIS TEMPLETON

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No.7833
ALEXIS TEMPLETON

Memorandum

To: Keith Purvis, P.E.
Neel-Schaffer, Inc.

From: Allen Cooley, Jr., Ph.D.
Randy Ahlrich, Ph.D., P.E.
Eddie Templeton, P.E.

Date: May 28, 2021

BCD Project No. 200518

Subject: Resilient Modulus Testing – AASHTO T307
Recompacted Subgrade Soils
ARDOT SR 230 – Alicia to Bono
Lawrence and Craighead Counties, Arkansas

Plans are being made for the construction of replacement bridges and box culverts along with adjacent roadway alignments at 10 sites along Highway 230 between Alicia and Bono in Lawrence and Craighead Counties in Arkansas. Representative bulk subgrade samples were obtained at various boring locations along the roadway alignment by representatives of McCray Drilling, SoilTech Consultants, Inc. and Burns Cooley Dennis, Inc. The sample locations (Station numbers with offset and GPS coordinates) are presented in the attached summary table (Table 1). The sample locations are also presented on the moisture-density relationship curves (AASHTO T99) (Appendix A) and the resilient modulus test results (AASHTO T307) (Appendix B).

The classification of each representative subgrade material was evaluated by Atterberg liquid and plastic limit tests (AASHTO T89 and T90) and sieve analyses (AASHTO T27 and T11). Based on these laboratory results the AASHTO and USCS classifications were determined for each representative subgrade material. The results of these tests are summarized in the attached Table 1. The subgrade soils along Highway 230 were typically found to be fine grained soils with AASHTO classifications of A-4, A-6 and A-7. The primary subgrade soil type along this roadway is A-6.

Each of the ten representative subgrade soil samples was tested and evaluated according to AASHTO T307 for recompacted samples. The results of this resilient modulus testing are presented in Appendix B. The determined resilient modulus values and the correlated R-values for each material are presented in Table 1. The resilient modulus values ranged from 8,443 psi to 14,764 psi. The correlated R values range from 12 to 24.

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TABLE 1

ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

RESILIENT MODULUS OF SUBGRADE SOILS
RECOMPACTED SAMPLES

Boring No.	S1-B	S2-A	S3-A	S4-B	S5-B
County	Lawrence	Lawrence	Lawrence	Lawrence	Lawrence
Station	105+43.5	117+40	305+10	413+99	511+14
Location	9' LT	26.5' LT	18' RT	8.5' RT	27.5' LT
Latitude	35° 53' 39.47"	35° 53' 39.03"	35° 53' 52.10"	35° 54' 40.92"	35° 54' 38.56"
Longitude	91° 4' 27.66"	91° 4' 13.13"	90° 57' 48.30"	90° 54' 58.15"	90° 53' 36.01"
% Passing					
¾ in.	100.0	100.0	100.0	100.0	100.0
3/8 in.	100.0	100.0	100.0	100.0	100.0
No. 4	100.0	100.0	99.9	100.0	99.9
No. 10	99.5	98.9	99.5	98.7	98.9
No. 40	98.8	96.8	98.1	96.7	97.8
No. 80	94.8	94.2	96.2	91.3	90.4
No. 200	63.2	76.9	94.0	74.0	55.8
Liquid Limit	28	34	47	30	25
Plasticity Index	13	20	32	15	9
AASHTO Classification	A-6(5)	A-6(13)	A-7-6(31)	A-6(9)	A-4(2)
USCS Classification	CL	CL	CL	CL	CL
Resilient Modulus (psi)	11,046	9,701	14,764	11,286	8,302
R-Value	18	15	24	18	12

TABLE 1 (continued)

ARDOT SR 230 – BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

RESILIENT MODULUS OF SUBGRADE SOILS
RECOMPACTED SAMPLES

Boring No.	S5-C	S6-B	S7-A	S7-B	S8-A
County	Lawrence	Lawrence	Lawrence	Lawrence	Craighead
Station	523+82.25	613+14	700+84.75	704+37	819+74.33
Location	34’ LT	68’ LT	9.5’ LT	66’ LT	29’ LT
Latitude	35° 54’ 38.11”	35° 54’ 37.25”	35° 54’ 36.43”	35° 54’ 36.86”	35° 54’ 35.08”
Longitude	90° 53’ 21.72”	90° 52’ 38.06”	90° 52’ 29.91”	90° 52’ 25.66”	90° 51’ 42.72”
% Passing					
¾ in.	100.0	100.0	100.0	100.0	100.0
3/8 in.	100.0	100.0	100.0	100.0	100.0
No. 4	100.0	100.0	99.9	99.9	100.0
No. 10	99.3	99.7	99.5	99.6	99.5
No. 40	98.0	98.9	98.6	99.1	98.2
No. 80	93.3	95.2	91.2	90.5	89.3
No. 200	71.6	60.7	64.7	49.8	62.7
Liquid Limit	33	34	29	32	31
Plasticity Index	19	21	17	20	17
AASHTO Classification	A-6(11)	A-6(9)	A-6(8)	A-6(6)	A-6(8)
USCS Classification	CL	CL	CL	SC	CL
Resilient Modulus (psi)	10,804	10,613	12,308	8,443	13,690
R-Value	17	17	20	13	22

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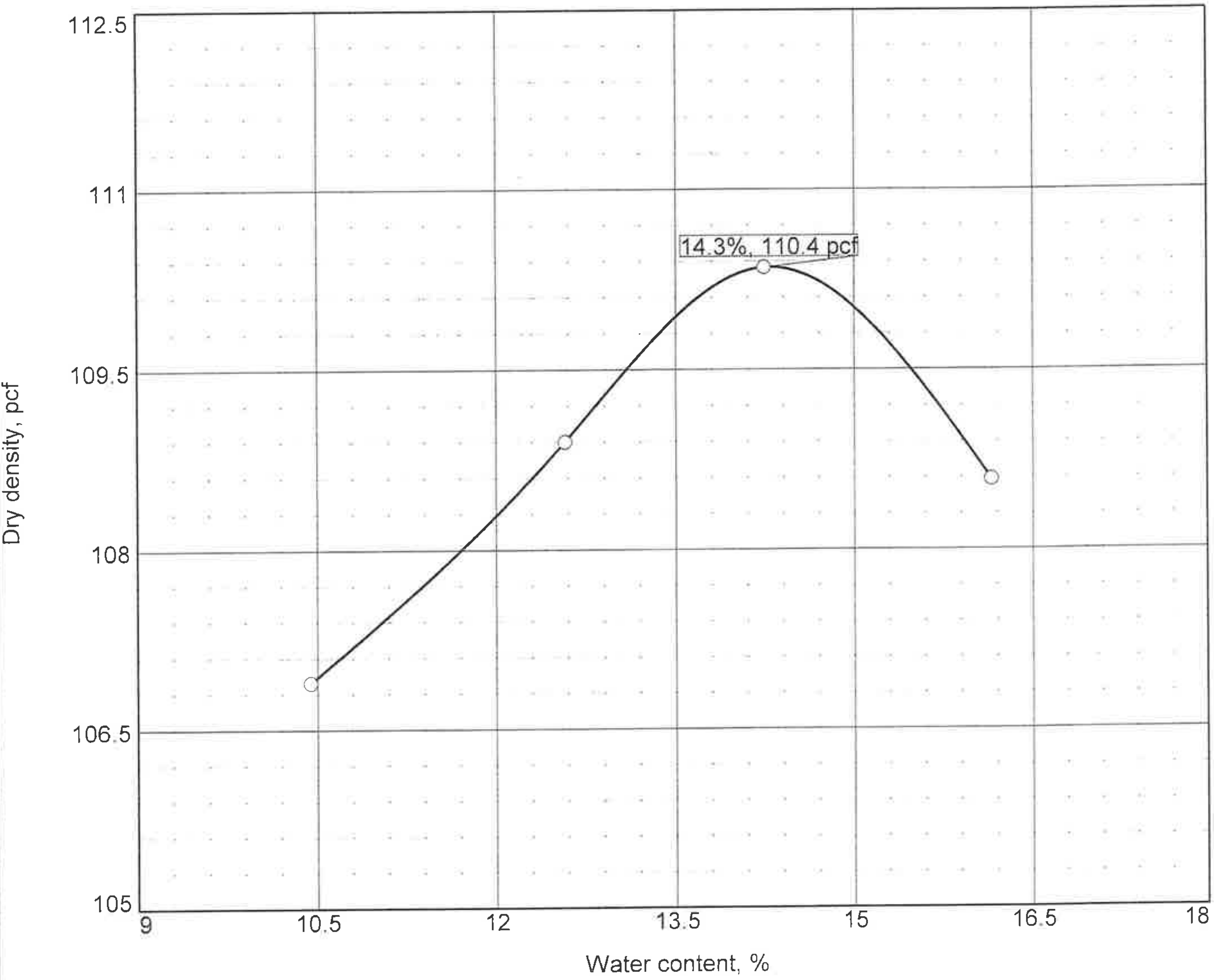
APPENDIX A

MOISTURE – DENSITY RELATIONSHIPS – AASHTO T99

ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

COMPACTION TEST REPORT



Test specification: AASHTO T 99-15 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL	A-6(5)			28	13		63.2

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 110.4 pcf Optimum moisture = 14.3 %	Tan sandy clay (CL)

Project No. 200518 **Client:** Neel-Schaffer

Project: ARDOT Highway 230 Alicia to Bono, Arkansas

Lawrence and Craighead Counties

○ **Location:** S1-B **Sample Number:** 8

Remarks:

Site 1

Station 105+43.5

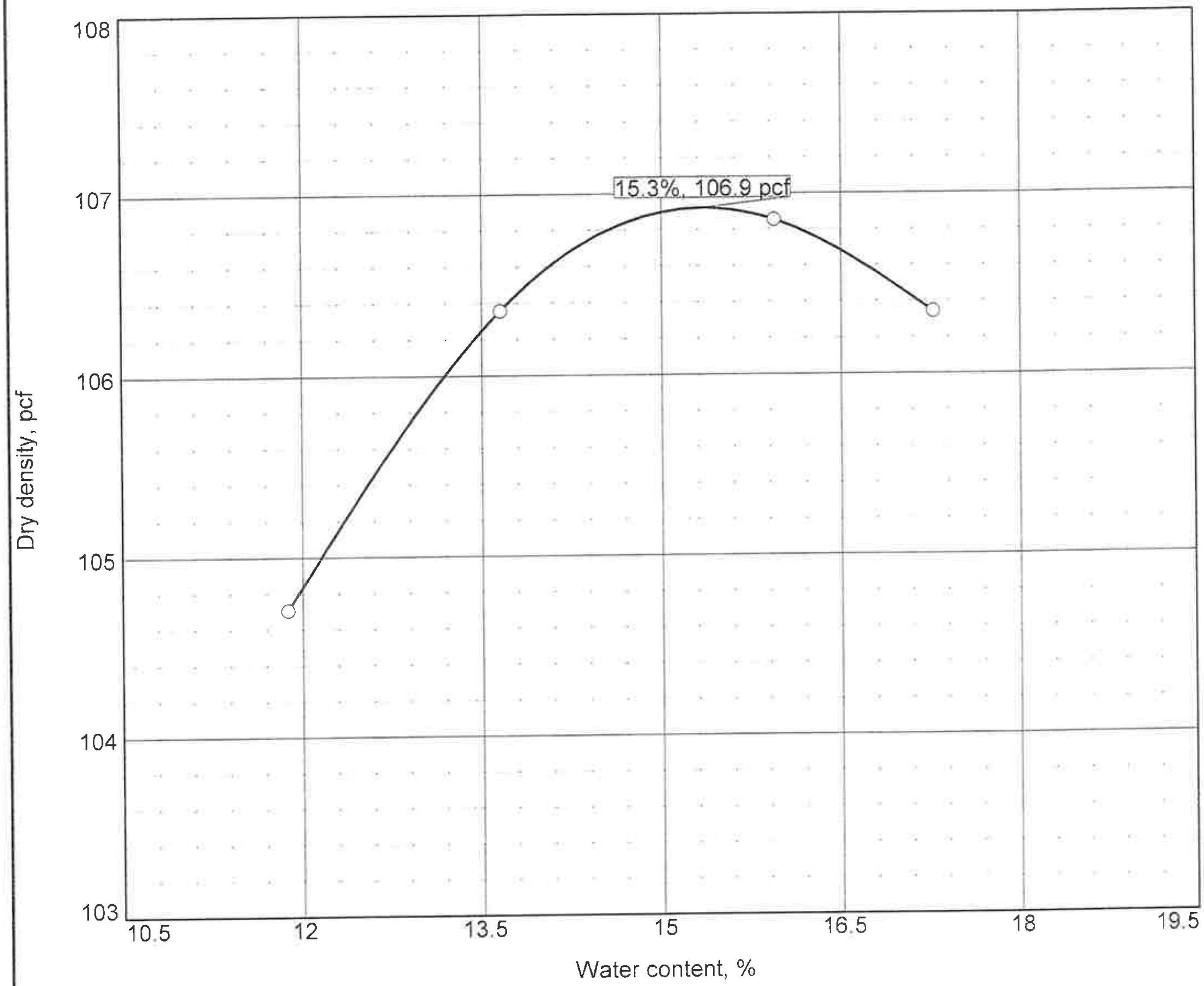
Location 9' Left

BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Figure 1

COMPACTION TEST REPORT



Test specification: AASHTO T 99-15 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL	A-6(13)			34	20		76.9

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 106.9 pcf Optimum moisture = 15.3 %	Tan silty clay (CL) with sand

Project No. 200518 **Client:** Neel-Schaffer

Project: ARDOT Highway 230 Alicia to Bono, Arkansas

Lawrence and Craighead Counties

○ **Location:** S2-A **Sample Number:** 9

BURNS COOLEY DENNIS, INC.

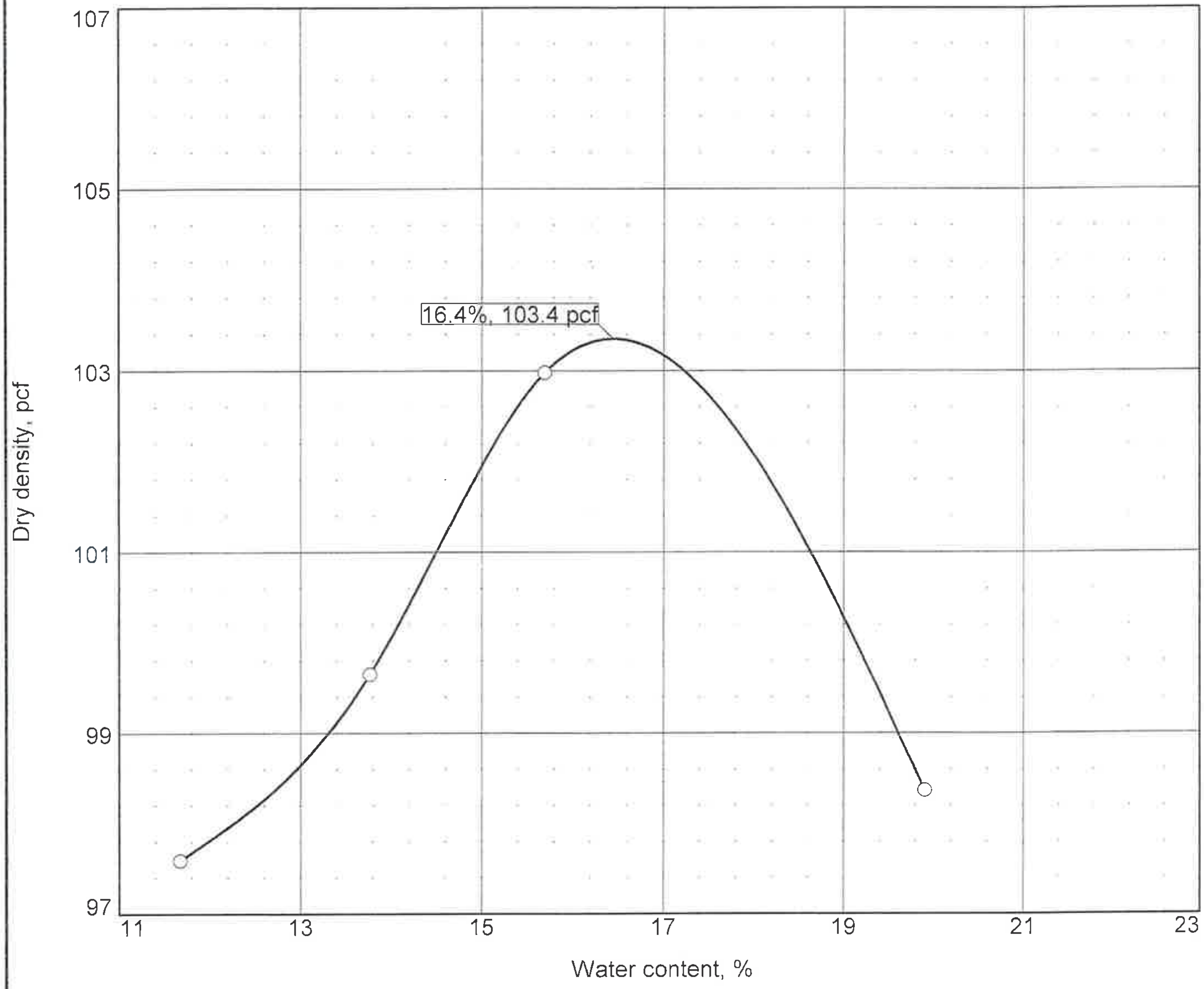
Ridgeland, Mississippi

Remarks:

Site 2
Station 117+40
Location 26.5' Left

Figure 2

COMPACTION TEST REPORT



Test specification: AASHTO T 99-15 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL	A-7-6(31)			47	32		94.0

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 103.4 pcf Optimum moisture = 16.4 %	Tan silty clay (CL), slightly sandy

Project No. 200518 **Client:** Neel-Schaffer
Project: ARDOT Highway 230 Alicia to Bono, Arkansas
Lawrence and Craighead Counties
○ **Location:** S3-A **Sample Number:** 12

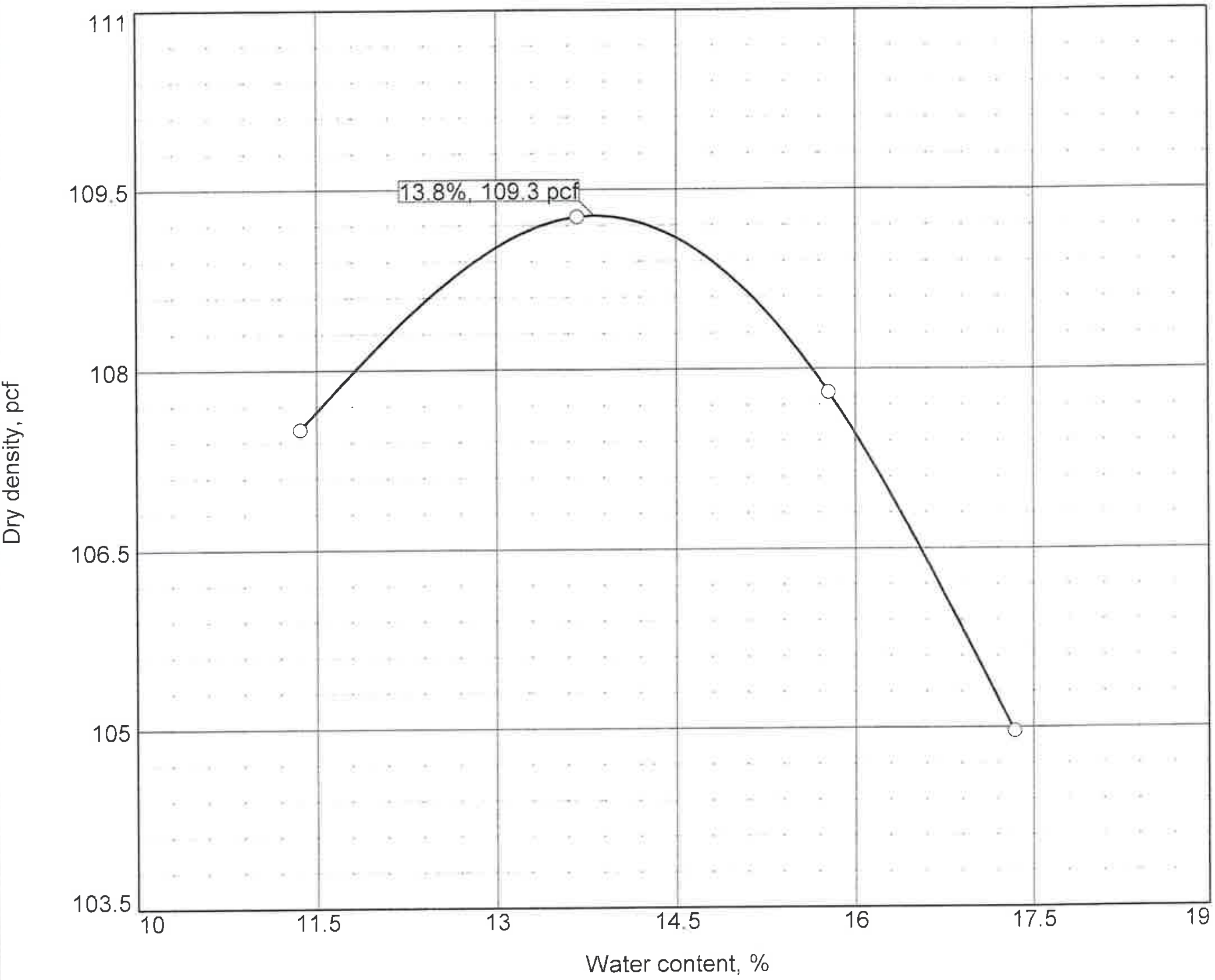
Remarks:
Site 3
Station 305+10
Location 18' Right

BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Figure 3

COMPACTION TEST REPORT



Test specification: AASHTO T 99-15 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL	A-6(9)			30	15		74.0

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 109.3 pcf Optimum moisture = 13.8 %	Tan sandy clay (CL)

Project No. 200518 **Client:** Neel-Schaffer

Project: ARDOT Highway 230 Alicia to Bono, Arkansas

Lawrence and Craighead Counties

○ **Location:** S4-B **Sample Number:** 10

BURNS COOLEY DENNIS, INC.

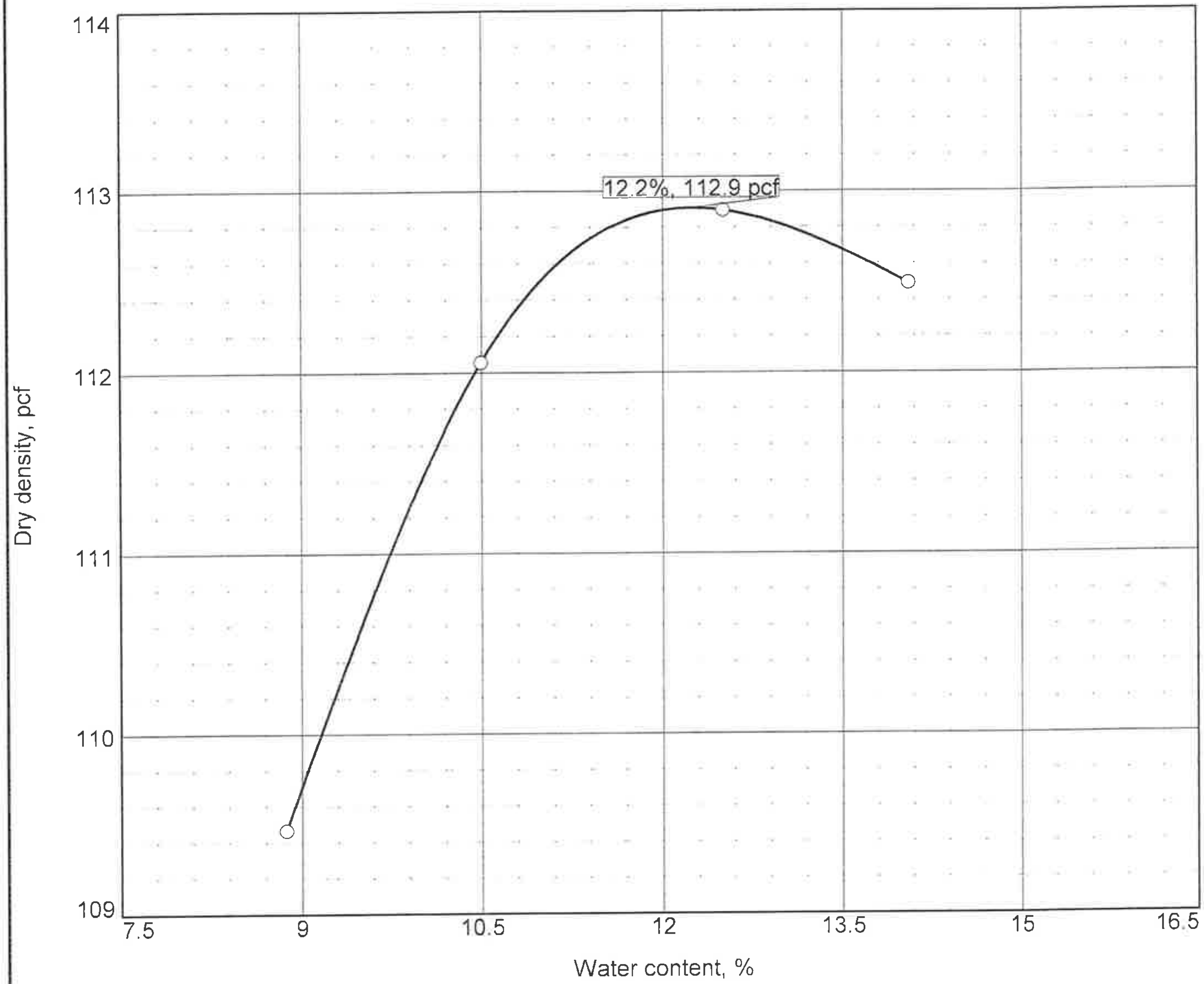
Ridgeland, Mississippi

Remarks:

Site 4
Station 413+99
Location 8.5' Right

Figure 4

COMPACTION TEST REPORT



Test specification: AASHTO T 99-15 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL	A-4(2)			25	9		55.8

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 112.9 pcf Optimum moisture = 12.2 %	Tan sandy clay (CL)

Project No. 200518 **Client:** Neel-Schaffer
Project: ARDOT Highway 230 Alicia to Bono, Arkansas
Lawrence and Craighead Counties
○ **Location:** S5-B **Sample Number:** 1

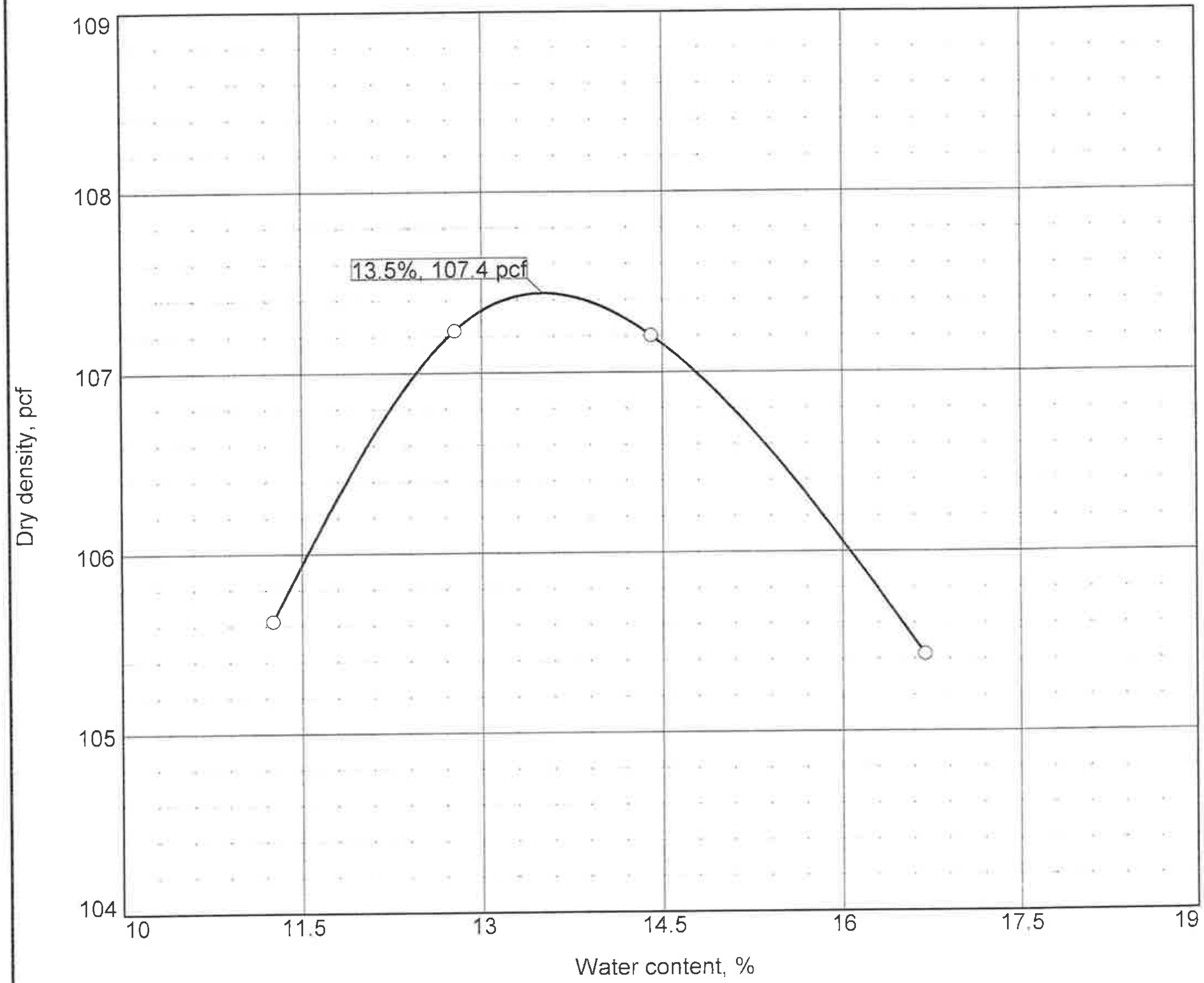
Remarks:
Site 5
Station 511+14
Location 27.5' Left

BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Figure 5

COMPACTION TEST REPORT



Test specification: AASHTO T 99-15 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL	A-6(11)			33	19		71.6

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 107.4 pcf Optimum moisture = 13.5 %	Tan silty clay (CL) with sand

Project No. 200518 **Client:** Neel-Schaffer
Project: ARDOT Highway 230 Alicia to Bono, Arkansas
Lawrence and Craighead Counties
○ **Location:** S5-C **Sample Number:** 2

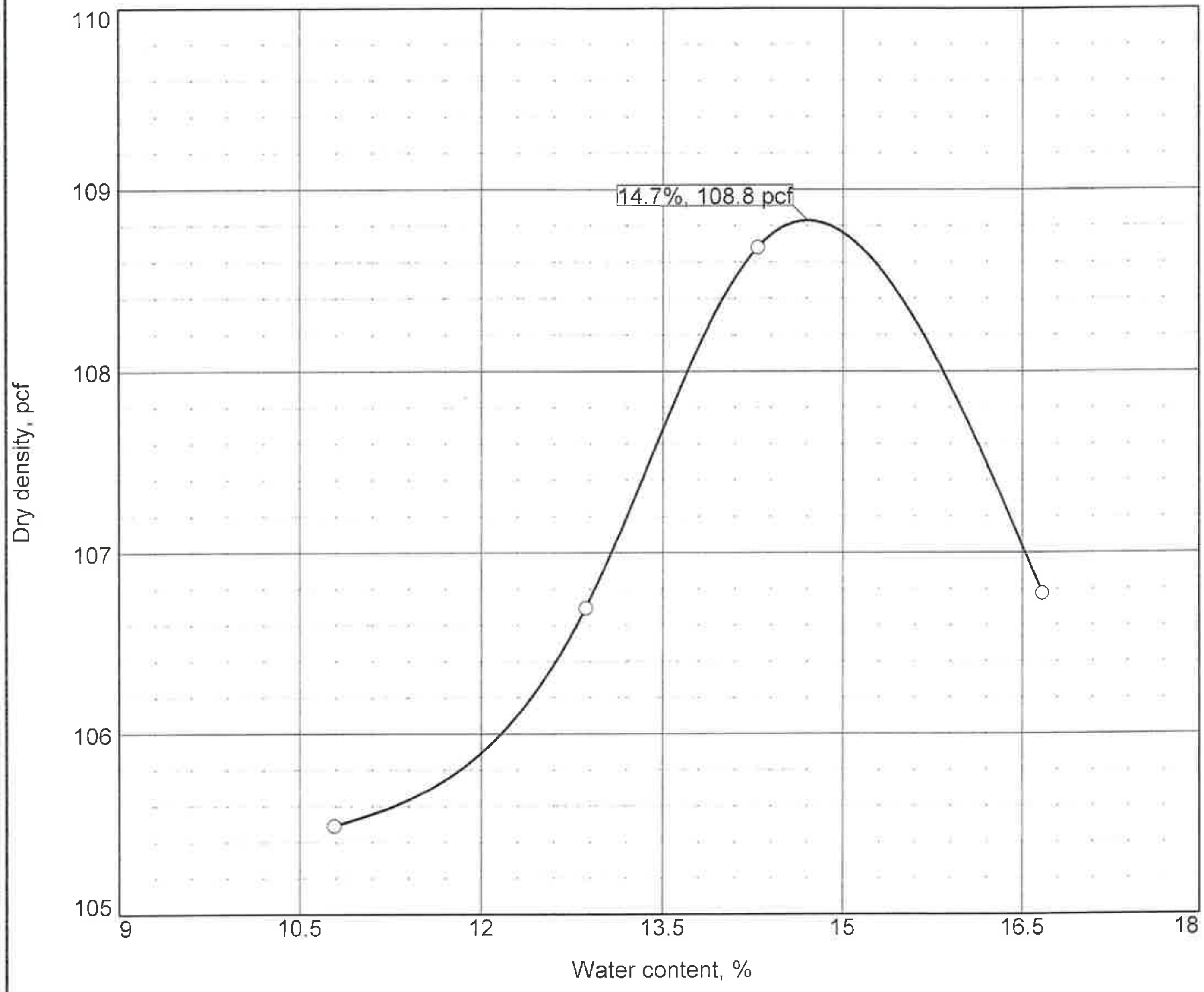
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Remarks:
Site 5
Station 523+82.25
Location 34' Left

Figure 6

COMPACTION TEST REPORT



Test specification: AASHTO T 99-15 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL	A-6(9)			34	21		60.7

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 108.8 pcf Optimum moisture = 14.7 %	Tan sandy clay (CL)

Project No. 200518 **Client:** Neel-Schaffer

Project: ARDOT Highway 230 Alicia to Bono, Arkansas

Lawrence and Craighead Counties

○ **Location:** S6-B **Sample Number:** 3

BURNS COOLEY DENNIS, INC.

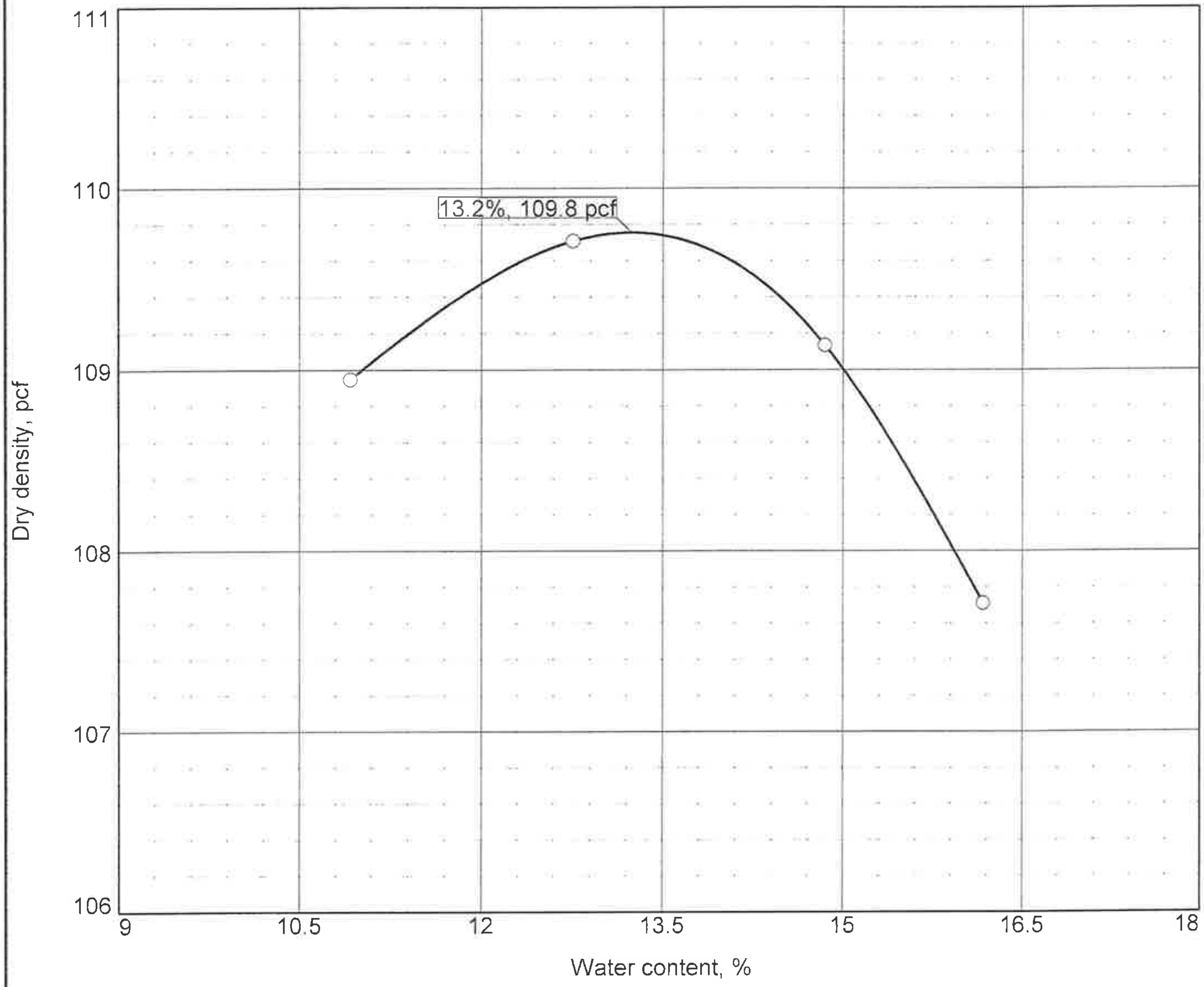
Ridgeland, Mississippi

Remarks:

Site 6
Station 613+14
Location 68' Left

Figure 7

COMPACTION TEST REPORT



Test specification: AASHTO T 99-15 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL	A-6(8)			29	17		64.7

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 109.8 pcf Optimum moisture = 13.2 %	Tan sandy clay (CL)

Project No. 200518 **Client:** Neel-Schaffer
Project: ARDOT Highway 230 Alicia to Bono, Arkansas
Lawrence and Craighead Counties
○ **Location:** S7-A **Sample Number:** 11

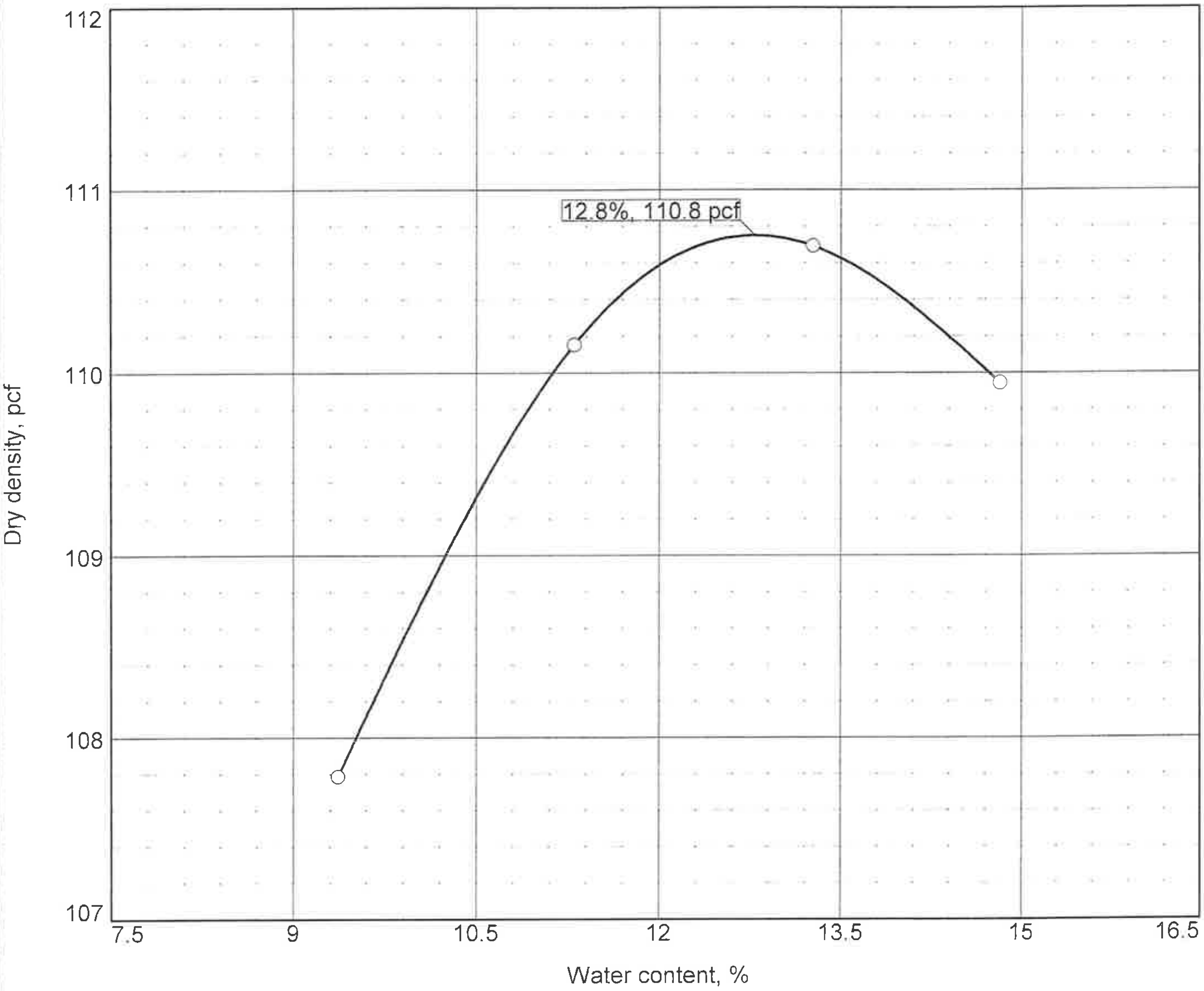
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Remarks:
Site 7
Station 700+84.75
Location 9.5' Left

Figure 8

COMPACTION TEST REPORT



Test specification: AASHTO T 99-15 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	SC	A-6(6)			32	20		49.8

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 110.8 pcf Optimum moisture = 12.8 %	Tan clayey sand (SC)

Project No. 200518 **Client:** Neel-Schaffer
Project: ARDOT Highway 230 Alicia to Bono, Arkansas
Lawrence and Craighead Counties
○ **Location:** S7-B **Sample Number:** 4

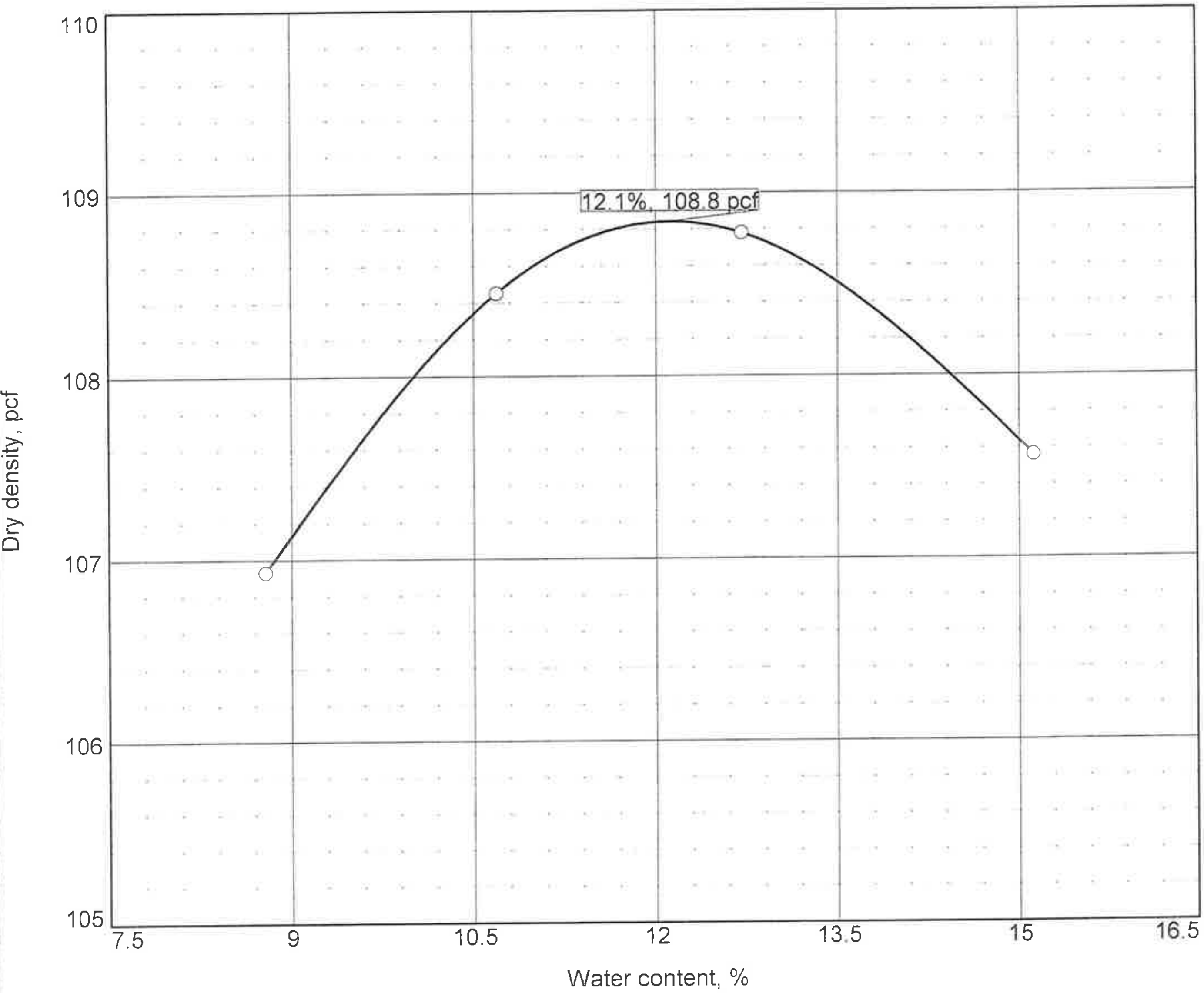
BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Remarks:
Site 7
Station 704+37
Location 66' Left

Figure 9

COMPACTION TEST REPORT



Test specification: AASHTO T 99-15 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL	A-6(8)			31	17		62.7

TEST RESULTS

Maximum dry density = 108.8 pcf

Optimum moisture = 12.1 %

MATERIAL DESCRIPTION

Tan sandy clay (CL)

Project No. 200518 **Client:** Neel-Schaffer

Project: ARDOT Highway 230 Alicia to Bono, Arkansas

Lawrence and Craighead Counties

○ **Location:** S8-A **Sample Number:** 5

Remarks:

Site 8

Station 819+74.33

Location 29' Left

BURNS COOLEY DENNIS, INC.

Ridgeland, Mississippi

Figure 10

BURNS COOLEY DENNIS, INC.

GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

Corporate Office
551 Sunnybrook Road
Ridgeland, MS 39157
Phone: (601) 856-9911
Fax: (601) 853-2077

Mailing Address
Post Office Box 12828
Jackson, MS 39236

www.bcdgeo.com

Materials Laboratory
278 Commerce Park Drive
Ridgeland, MS 39157
Phone: (601) 856-2332
Fax: (601) 856-3552

APPENDIX B

RESILIENT MODULUS OF SUBGRADE SOILS – AASHTO T307 RECOMPACTED SAMPLES

ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

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

Job No.	200518	**Material Code**	CL
Date Sampled:	March 1. 2021	**Station No.:**	105+43.5. 9' LT
Date Tested:	March 27, 2021	**Location:**	S1-B
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 38 **Name:** Lawrence		
Sampled By:	BCD	**Depth:**	0 - 5
Lab No.:	16297	**AASHTO Class:**	A-6(5)
Sample ID:	8	**Material Type (1 or 2):**	2
LATITUDE:	35° 53' 39.47"	**LONGITUDE:**	91° 4' 27.66"

---

1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No)	N
Testing - Permanent Strain > 5% (Y=Yes or N=No)	N
Number of Load Sequences Completed (0-15)	15

2. Specimen Information:

Specimen Diameter (in):	
Top	3.96
Middle	3.96
Bottom	3.96
Average	3.96
Membrane Thickness (in):	0.025
Height of Specimen, Cap and Base (in):	13.43
Height of Cap and Base (in):	5.38
Initial Length, Lo (in):	8.05
Initial Area, Ao (sq. in):	12.32
Initial Volume, AoLo (cu. in):	99.18

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	5000

4. Soil Properties:

Optimum Moisture Content (%):	14.3
Maximum Dry Density (pcf):	110.4
95% of MDD (pcf):	104.9
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	3094.7
Compaction Moisture Content (%):	14.2
Compaction Wet Density (pcf):	118.4
Compaction Dry Density (pcf):	103.7
Moisture Content After Mr Test (%):	14.2

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

7. Resilient Modulus, Mr: Mr = 12831 (Sc)^-0.12226 (S3)^0.16745

8. Comments ____________________

9. Tested By: Scott Bivings **Date:** March 29, 2021

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

Job No. 200518
Date Sampled 3/1/2021
Date Tested: 3/27/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: BCD
Lab No.: 16297
Sample ID: 8
LATITUDE: 35° 53' 39.47"

Material Code: CL
Station No.: 105+43.5, 9' LT
Location: S1-B
Depth: 0 - 5
AASHTO Class: A-6(5)
Material Type (1 or 2) 2
LONGITUDE: 91° 4' 27.66"

PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Actual Applied Max. Axial Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Axial Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Average Recov Def. LVDT 1 and 2	Resilient Strain	Resilient Modulus
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	5.8	1.8	59.8	53.8	6.0	5.0	4.5	0.5	0.00222	0.00028	16,063
Sequence 2	5.8	3.6	84.0	75.4	8.8	7.0	6.3	0.7	0.00330	0.00041	15,382
Sequence 3	5.8	5.4	107.6	96.6	11.0	9.0	8.0	0.9	0.00452	0.00056	14,237
Sequence 4	5.8	7.2	131.2	117.6	13.2	10.9	9.8	1.1	0.00590	0.00073	13,416
Sequence 5	5.6	9.0	155.6	139.6	16.0	13.0	11.6	1.3	0.00740	0.00092	12,679
Sequence 6	3.8	1.8	60.2	54.2	6.0	5.0	4.5	0.5	0.00250	0.00031	14,583
Sequence 7	3.8	3.6	84.6	76.0	8.6	7.0	6.3	0.7	0.00370	0.00046	13,784
Sequence 8	3.8	5.4	108.0	96.6	11.0	9.0	8.0	0.9	0.00498	0.00062	13,080
Sequence 9	3.8	7.2	131.2	117.6	13.2	10.9	9.8	1.1	0.00628	0.00078	12,554
Sequence 10	3.8	9.0	155.2	139.6	16.0	12.9	11.6	1.3	0.00772	0.00096	12,083
Sequence 11	1.9	1.8	59.8	53.8	6.0	5.0	4.5	0.5	0.00274	0.00034	13,077
Sequence 12	1.9	3.6	83.8	75.6	8.4	7.0	6.3	0.7	0.00410	0.00051	12,308
Sequence 13	1.9	5.4	107.8	96.8	11.0	9.0	8.1	0.9	0.00556	0.00069	11,675
Sequence 14	1.9	7.2	131.8	118.6	13.0	11.0	9.9	1.1	0.00700	0.00087	11,315
Sequence 15	1.9	9.0	155.4	140.0	16.0	12.9	11.6	1.3	0.00850	0.00105	11,046

TESTED BY: S. Bivings DATE: 3/29/2021

REVIEWED BY: A. Cooley DATE: 4/5/2021

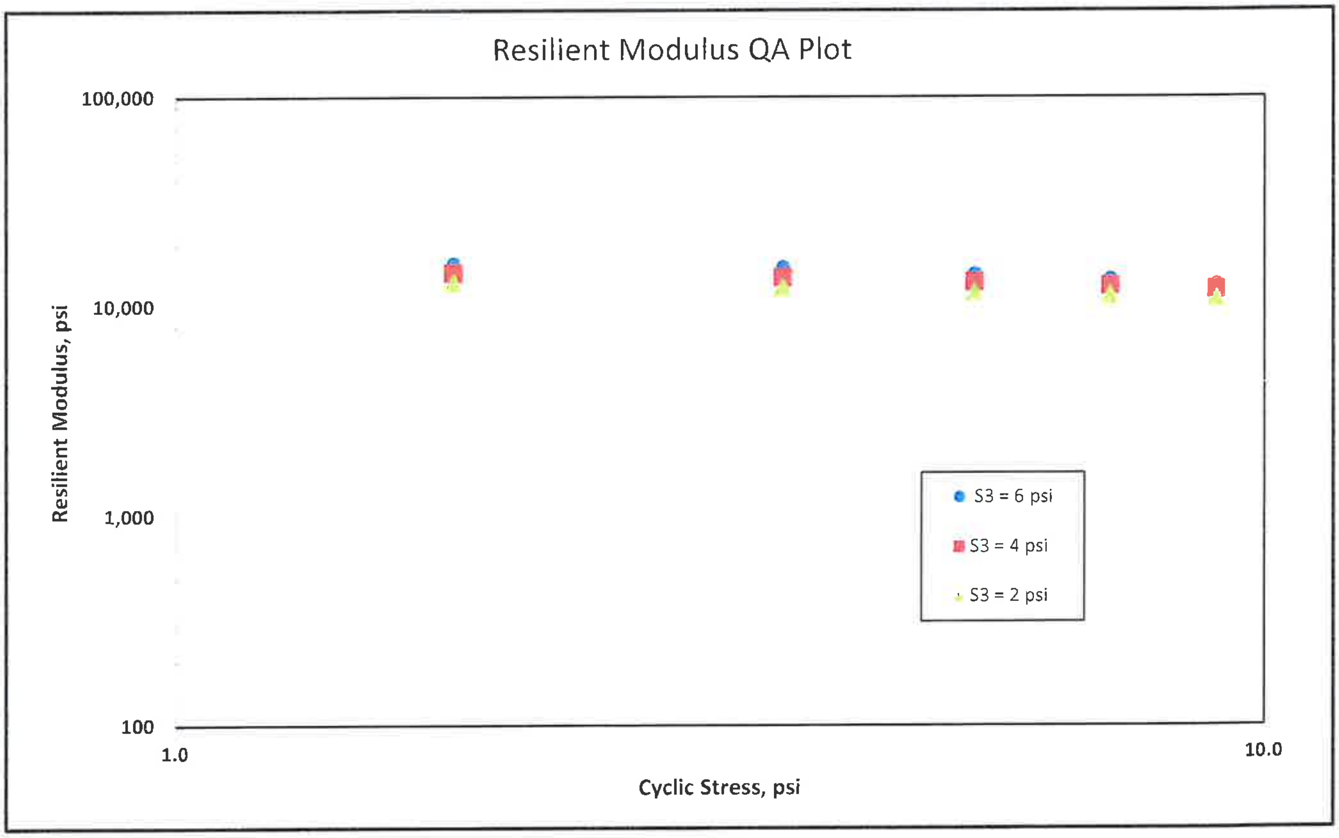
AASHTO T 307-99 - RESILIENT MODULUS OF SUBGRADE SOILS
RECOMPACTED SAMPLES

Job No. 200518
Date Sampled 3/1/2021
Date Tested: 3/27/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: BCD
Lab No.: 16297
Sample ID: 8
LATITUDE: 35° 53' 39.47"

Material Code: CL
Station No.: 105+43.5, 9' LT
Location: S1-B
Depth: 0 - 5
AASHTO Class: A-6(5)
Material Type (1 or 2) 2
LONGITUDE: 91° 4' 27.66"

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

K1 = 12,831
K2 = -0.12226
K5 = 0.16745
R^2 = 0.97



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

Job No.	200518	**Material Code**	CL
Date Sampled:	March 1, 2020	**Station No.:**	117+40, 26.5' LT
Date Tested:	March 27, 2021	**Location:**	S2-A
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 38 **Name:** Lawrence		
Sampled By:	BCD	**Depth:**	0 - 5
Lab No.:	16298	**AASHTO Class:**	A-6(13)
Sample ID:	9	**Material Type (1 or 2):**	2
LATITUDE:	35° 53' 39.03"	**LONGITUDE:**	91° 4' 13.13"

1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No)	N
Testing - Permanent Strain > 5% (Y=Yes or N=No)	N
Number of Load Sequences Completed (0-15)	15

2. Specimen Information:

Specimen Diameter (in):	
Top	3.96
Middle	3.96
Bottom	3.96
Average	3.96
Membrane Thickness (in):	0.025
Height of Specimen, Cap and Base (in):	13.43
Height of Cap and Base (in):	5.38
Initial Length, Lo (in):	8.05
Initial Area, Ao (sq. in):	12.32
Initial Volume, AoLo (cu. in):	99.18

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	5000

4. Soil Properties:

Optimum Moisture Content (%):	15.3
Maximum Dry Density (pcf):	106.9
95% of MDD (pcf):	101.6
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	3045.1
Compaction Moisture Content (%):	15.3
Compaction Wet Density (pcf):	117.0
Compaction Dry Density (pcf):	101.8
Moisture Content After Mr Test (%):	14.9

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

7. Resilient Modulus, Mr: Mr = 10176 (Sc)^-0.08937 (S3)^0.20672

8. Comments

9. Tested By: Scott Bivings **Date:** March 29, 2021

BURNS COOLEY DENNIS, INC.

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

Job No. 200518
Date Sampled 3/1/2021
Date Tested: 3/27/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: BCD
Lab No.: 16298
Sample ID: 9
LATITUDE: 35° 53' 39.03"

Material Code: CL
Station No.: 117+40, 26.5' LT
Location: S2-A

Depth: 0 - 5
AASHTO Class: A-6(13)
Material Type (1 or 2) 2
LONGITUDE: 91° 4' 13.13"

PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Actual Applied Max. Axial Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Axial Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Average Recov Def. LVDT 1 and 2	Resilient Strain	Resilient Modulus
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	5.7	1.8	60.0	53.8	6.0	5.0	4.5	0.5	0.00260	0.00032	13,995
Sequence 2	5.8	3.6	82.6	74.4	8.6	6.9	6.2	0.7	0.00366	0.00045	13,629
Sequence 3	5.8	5.4	108.8	97.8	11.0	9.1	8.1	0.9	0.00510	0.00064	12,828
Sequence 4	5.8	7.2	132.4	119.0	13.4	11.0	9.9	1.1	0.00660	0.00082	12,030
Sequence 5	5.6	9.0	155.8	140.0	16.0	13.0	11.6	1.3	0.00814	0.00101	11,515
Sequence 6	3.8	1.8	59.4	53.2	6.0	4.9	4.4	0.5	0.00286	0.00035	12,509
Sequence 7	3.8	3.6	83.6	75.0	8.6	7.0	6.3	0.7	0.00418	0.00052	12,043
Sequence 8	3.8	5.4	108.2	97.2	11.0	9.0	8.1	0.9	0.00560	0.00070	11,557
Sequence 9	3.8	7.2	131.6	118.4	13.0	11.0	9.8	1.1	0.00710	0.00088	11,149
Sequence 10	3.8	9.0	155.0	139.0	16.0	12.9	11.6	1.3	0.00858	0.00107	10,872
Sequence 11	1.9	1.8	60.0	53.6	6.0	5.0	4.5	0.5	0.00336	0.00042	10,642
Sequence 12	1.8	3.6	84.4	75.8	8.6	7.0	6.3	0.7	0.00492	0.00061	10,340
Sequence 13	1.9	5.4	108.2	97.2	11.0	9.0	8.1	0.9	0.00650	0.00081	10,069
Sequence 14	1.9	7.2	132.4	119.0	13.2	11.0	9.9	1.1	0.00808	0.00100	9,886
Sequence 15	1.9	9.0	155.2	139.4	16.0	12.9	11.6	1.3	0.00962	0.00120	9,701

TESTED BY: S. Bivings DATE: 3/29/2021
REVIEWED BY: A. Cooley DATE: 4/5/2021

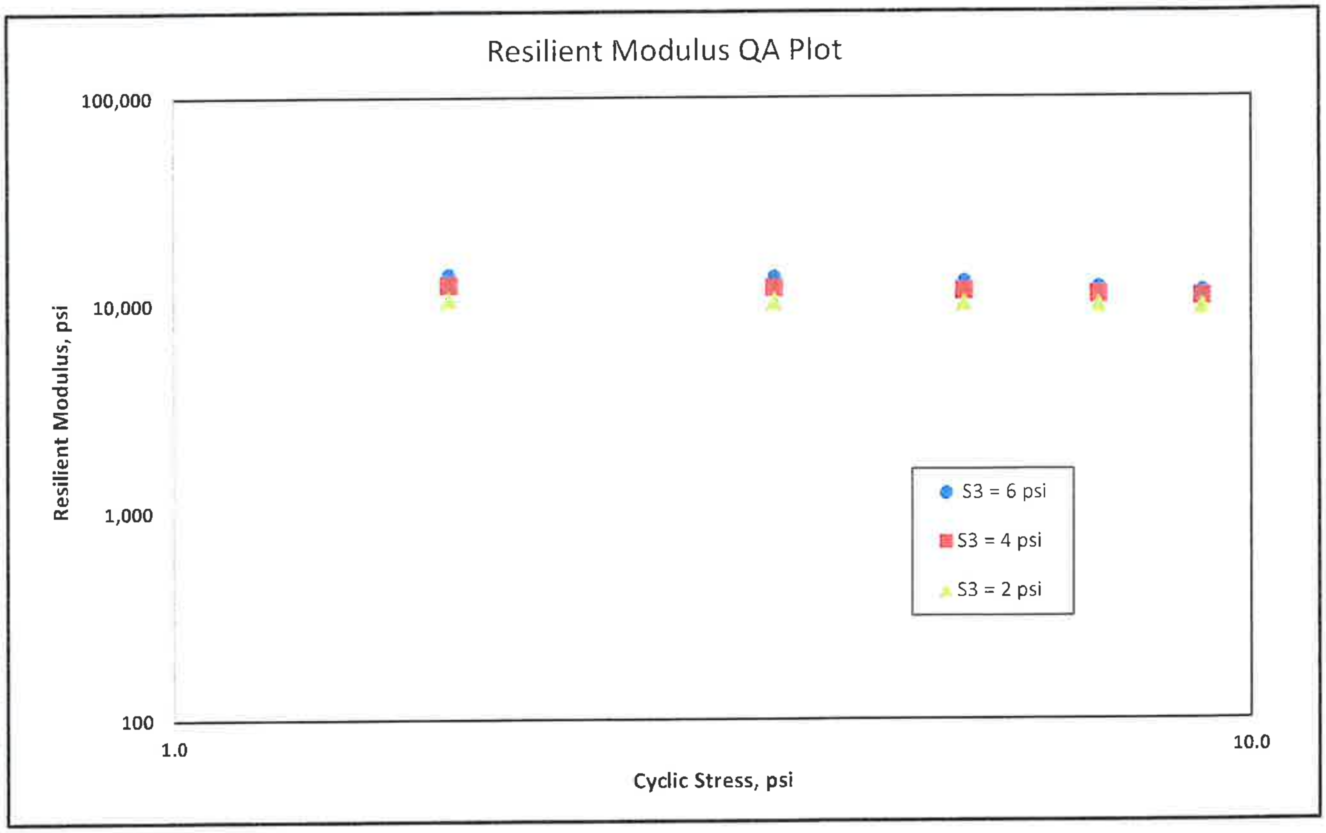
AASHTO T 307-99 - RESILIENT MODULUS OF SUBGRADE SOILS
RECOMPACTED SAMPLES

Job No. 200518
Date Sampled 3/1/2021
Date Tested: 3/27/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: BCD
Lab No.: 16298
Sample ID: 9
LATITUDE: 35° 53' 39.03"

Material Code: CL
Station No.: 117+40, 26.5' LT
Location: S2-A
Depth: 0 - 5
AASHTO Class: A-6(13)
Material Type (1 or 2) 2
LONGITUDE: 91° 4' 13.13"

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

K1 = 10,176
K2 = -0.08937
K5 = 0.20672
R^2 = 0.96



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

Job No.	200518	**Material Code**	CL
Date Sampled:	March 1, 2021	**Station No.:**	305+10, 18' RT
Date Tested:	March 27, 2021	**Location:**	S3-A
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 38 **Name:** Lawrence		
Sampled By:	BCD	**Depth:**	0 - 5
Lab No.:	16311	**AASHTO Class:**	A-7-6(31)
Sample ID:	12	**Material Type (1 or 2):**	2
LATITUDE:	35° 53' 52.10"	**LONGITUDE:**	90° 57' 48.30"

---

1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No)	N
Testing - Permanent Strain > 5% (Y=Yes or N=No)	N
Number of Load Sequences Completed (0-15)	15

2. Specimen Information:

Specimen Diameter (in):	
Top	3.96
Middle	3.96
Bottom	3.96
Average	3.96
Membrane Thickness (in):	0.025
Height of Specimen, Cap and Base (in):	13.43
Height of Cap and Base (in):	5.38
Initial Length, Lo (in):	8.05
Initial Area, Ao (sq. in):	12.32
Initial Volume, AoLo (cu. in):	99.18

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	5000

4. Soil Properties:

Optimum Moisture Content (%):	16.4
Maximum Dry Density (pcf):	103.4
95% of MDD (pcf):	98.2
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	2963.7
Compaction Moisture Content (%):	16.5
Compaction Wet Density (pcf):	113.7
Compaction Dry Density (pcf):	97.6
Moisture Content After Mr Test (%):	16.6

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

7. Resilient Modulus, Mr: Mr = 14993 (Sc)^-0.06540 (S3)^0.17868

8. Comments

9. Tested By: Scott Bivings **Date:** March 29, 2021

BURNS COOLEY DENNIS, INC.

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

Job No. 200518
Date Sampled 3/1/2021
Date Tested: 3/27/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: BCD
Lab No.: 16311
Sample ID: 12
LATITUDE: 35° 53' 52.10"

Material Code: CL
Station No.: 305+10, 18' RT
Location: S3-A

Depth: 0 - 5
AASHTO Class: A-7-6(31)
Material Type (1 or 2) 2
LONGITUDE: 90° 57' 48.30"

PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Actual Applied Max. Axial Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Axial Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Average Recov Def. LVDT 1 and 2	Resilient Strain	Resilient Modulus
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	5.9	1.8	59.8	53.8	6.0	5.0	4.5	0.5	0.00180	0.00022	20,226
Sequence 2	5.9	3.6	84.2	75.8	8.6	7.0	6.3	0.7	0.00260	0.00032	19,580
Sequence 3	5.9	5.4	108.0	97.0	11.0	9.0	8.1	0.9	0.00350	0.00043	18,694
Sequence 4	5.9	7.2	133.8	120.0	13.2	11.1	10.0	1.1	0.00450	0.00056	17,823
Sequence 5	5.9	9.0	157.2	141.2	16.0	13.1	11.8	1.3	0.00552	0.00069	17,105
Sequence 6	3.9	1.8	60.0	53.8	6.0	5.0	4.5	0.5	0.00200	0.00025	18,086
Sequence 7	3.9	3.6	84.4	75.8	9.0	7.0	6.3	0.7	0.00290	0.00036	17,759
Sequence 8	3.9	5.4	108.8	97.8	11.0	9.1	8.2	0.9	0.00380	0.00047	17,211
Sequence 9	3.9	7.2	132.4	119.2	13.4	11.0	9.9	1.1	0.00474	0.00059	16,816
Sequence 10	3.9	9.0	157.8	142.0	16.0	13.1	11.8	1.3	0.00586	0.00073	16,298
Sequence 11	1.9	1.8	60.8	54.4	6.0	5.0	4.5	0.5	0.00232	0.00029	15,592
Sequence 12	1.9	3.6	83.8	74.8	9.0	7.0	6.3	0.7	0.00328	0.00041	15,376
Sequence 13	1.9	5.4	109.0	98.0	11.0	9.1	8.2	0.9	0.00430	0.00053	15,336
Sequence 14	1.9	7.2	131.4	118.0	13.4	10.9	9.8	1.1	0.00522	0.00065	15,039
Sequence 15	1.9	9.0	156.0	140.4	16.0	13.0	11.6	1.3	0.00640	0.00079	14,764

TESTED BY: S. Bivings DATE: 3/29/2021
REVIEWED BY: A. Cooley DATE: 4/5/2021

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

Job No.	200518		**Material Code:**	CL
Date Sampled	3/1/2021		**Station No.:**	305+10, 18' RT
Date Tested:	3/27/2021		**Location:**	S3-A
Name of Project:	SR 230 Bridges and Approaches			
County:	**Code:** 38	**Name:**	Lawrence	
Sampled By:	BCD		**Depth:**	0 - 5
Lab No.:	16311		**AASHTO Class:**	A-7-6(31)
Sample ID:	12		**Material Type (1 or 2)**	2
LATITUDE:	35° 53' 52.10"		**LONGITUDE:**	90° 57' 48.30"

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

K1 = 14,993

K2 = -0.06540

K5 = 0.17868

R^2 = 0.94

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

Job No. 200518
Date Sampled: March 2, 2021
Date Tested: March 27, 2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: BCD
Lab No.: 16299
Sample ID: 10
LATITUDE: 35° 54' 40.92"

Material Code CL
Station No.: 413+99. 8.5' RT
Location: S4-B

Depth: 0 - 5
AASHTO Class: A-6(9)
Material Type (1 or 2): 2
LONGITUDE: 90° 54' 58.15"

---

1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No)	N
Testing - Permanent Strain > 5% (Y=Yes or N=No)	N
Number of Load Sequences Completed (0-15)	15

2. Specimen Information:

Specimen Diameter (in):	
Top	3.96
Middle	3.96
Bottom	3.96
Average	3.96
Membrane Thickness (in):	0.025
Height of Specimen, Cap and Base (in):	13.43
Height of Cap and Base (in):	5.38
Initial Length, Lo (in):	8.05
Initial Area, Ao (sq. in):	12.32
Initial Volume, AoLo (cu. in):	99.18

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	5000

4. Soil Properties:

Optimum Moisture Content (%):	13.8
Maximum Dry Density (pcf):	109.3
95% of MDD (pcf):	103.8
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	3126.1
Compaction Moisture Content (%):	14.0
Compaction Wet Density (pcf):	120.1
Compaction Dry Density (pcf):	105.3
Moisture Content After Mr Test (%):	13.9

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

7. Resilient Modulus, Mr: Mr = 11,386 (Sc)^-0.07994 (S3)^0.22939

8. Comments

9. Tested By: Scott Bivings

Date: March 29, 2021

BURNS COOLEY DENNIS, INC.

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

Job No. 200518
Date Sampled 3/2/2021
Date Tested: 3/27/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: BCD
Lab No.: 16299
Sample ID: 10
LATITUDE: 35° 54' 40.92"

Material Code: CL
Station No.: 413+99, 8.5' RT
Location: S4-B
Depth: 0 - 5
AASHTO Class: A-6(9)
Material Type (1 or 2) 2
LONGITUDE: 90° 54' 58.15"

PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Actual Applied Max. Axial Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Axial Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Average Recov Def. LVDT 1 and 2	Resilient Strain	Resilient Modulus
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	5.8	1.8	60.0	53.8	6.0	5.0	4.5	0.5	0.00220	0.00027	16,567
Sequence 2	5.8	3.6	83.6	74.8	8.8	7.0	6.2	0.7	0.00310	0.00038	16,251
Sequence 3	5.8	5.4	108.6	97.6	11.0	9.0	8.1	0.9	0.00430	0.00053	15,195
Sequence 4	5.7	7.2	132.8	119.2	13.2	11.1	9.9	1.1	0.00560	0.00070	14,239
Sequence 5	5.7	9.0	156.4	140.8	16.0	13.0	11.7	1.3	0.00692	0.00086	13,591
Sequence 6	3.8	1.8	61.2	55.0	6.0	5.1	4.6	0.5	0.00256	0.00032	14,380
Sequence 7	3.8	3.6	83.8	75.4	8.6	7.0	6.3	0.7	0.00360	0.00045	14,006
Sequence 8	3.8	5.4	109.2	98.0	11.0	9.1	8.1	0.9	0.00484	0.00060	13,528
Sequence 9	3.8	7.2	132.4	119.0	13.0	11.0	9.9	1.1	0.00610	0.00076	13,113
Sequence 10	3.8	9.0	156.0	140.6	16.0	13.0	11.7	1.3	0.00740	0.00092	12,709
Sequence 11	1.9	1.8	58.6	52.6	6.0	4.9	4.4	0.5	0.00290	0.00036	12,099
Sequence 12	1.9	3.6	83.8	75.2	8.8	7.0	6.3	0.7	0.00420	0.00052	11,972
Sequence 13	1.9	5.4	107.4	96.4	11.0	8.9	8.0	0.9	0.00552	0.00069	11,671
Sequence 14	1.9	7.2	131.6	118.2	13.4	11.0	9.9	1.1	0.00688	0.00085	11,514
Sequence 15	1.9	9.0	156.0	140.0	16.0	13.0	11.7	1.3	0.00830	0.00103	11,286

TESTED BY: S. Bivings DATE: 3/29/2021
REVIEWED BY: A. Cooley DATE: 4/5/2021

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

Job No. 200518
Date Sampled 3/2/2021
Date Tested: 3/27/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: BCD
Lab No.: 16299
Sample ID: 10
LATITUDE: 35° 54' 40.92"

Material Code: CL
Station No.: 413+99, 8.5' RT
Location: S4-B
Depth: 0 - 5
AASHTO Class: A-6(9)
Material Type (1 or 2) 2
LONGITUDE: 90° 54' 58.15"

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

$K1 =$ 11,386
$K2 =$ -0.07994
$K5 =$ 0.22939
$R^2 =$ 0.94

BURNS COOLEY DENNIS, INC.

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

Job No.	200518	**Material Code**	CL
Date Sampled:	November 5, 2020	**Station No.:**	511+14, 27.5' LT
Date Tested:	January 4, 2021	**Location:**	S5-B
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 38 **Name:** Lawrence		
Sampled By:	SoilTech	**Depth:**	0 - 5
Lab No.:	16152	**AASHTO Class:**	A-4 (2)
Sample ID:	1	**Material Type (1 or 2):**	2
LATITUDE:	35° 54' 38.56"	**LONGITUDE:**	90° 53' 36.01"

---

1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No)	N
Testing - Permanent Strain > 5% (Y=Yes or N=No)	N
Number of Load Sequences Completed (0-15)	15

2. Specimen Information:

Specimen Diameter (in):	
Top	3.96
Middle	3.96
Bottom	3.96
Average	3.96
Membrane Thickness (in):	0.025
Height of Specimen, Cap and Base (in):	13.43
Height of Cap and Base (in):	5.38
Initial Length, Lo (in):	8.05
Initial Area, Ao (sq. in):	12.32
Initial Volume, AoLo (cu. in):	99.18

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	5000

4. Soil Properties:

Optimum Moisture Content (%):	12.2
Maximum Dry Density (pcf):	112.9
95% of MDD (pcf):	107.3
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	3094.7
Compaction Moisture Content (%):	12.2
Compaction Wet Density (pcf):	118.8
Compaction Dry Density (pcf):	105.9
Moisture Content After Mr Test (%):	12.2

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

7. Resilient Modulus, Mr: Mr = 7249 (Sc)^-0.03959 (S3)^0.29816

8. Comments ______________________

9. Tested By: Scott Bivings **Date:** January 4, 2021

BURNS COOLEY DENNIS, INC.

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

Job No. 200518
Date Sampled 11/5/2020
Date Tested: 1/4/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: SoilTech
Lab No.: 16152
Sample ID: 1
LATITUDE: 35° 54' 38.56"

Material Code: CL
Station No.: 511+14, 27.5' LT
Location: S5-B

Depth: 0 - 5
AASHTO Class: A-4 (2)
Material Type (1 or 2) 2
LONGITUDE: 90° 53' 36.01"

PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Actual Applied Max. Axial Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Axial Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Average Recov Def. LVDT 1 and 2	Resilient Strain	Resilient Modulus
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	5.8	1.8	60.0	54.0	6.0	5.0	4.5	0.5	0.00290	0.00037	12,296
Sequence 2	5.8	3.6	83.8	75.2	8.8	7.0	6.3	0.7	0.00414	0.00052	12,176
Sequence 3	5.8	5.4	107.4	96.2	11.0	8.9	8.0	0.9	0.00550	0.00068	11,748
Sequence 4	5.8	7.2	131.6	118.2	13.0	11.0	9.8	1.1	0.00700	0.00087	11,327
Sequence 5	5.8	9.0	155.0	139.0	16.0	12.9	11.6	1.3	0.00850	0.00105	10,998
Sequence 6	3.9	1.8	60.0	53.8	6.0	5.0	4.5	0.5	0.00350	0.00043	10,365
Sequence 7	3.9	3.6	84.0	75.0	9.0	7.0	6.3	0.7	0.00500	0.00062	10,026
Sequence 8	3.9	5.4	107.8	96.8	11.0	9.0	8.1	0.9	0.00656	0.00081	9,892
Sequence 9	3.9	7.2	132.2	118.4	13.0	11.0	9.9	1.1	0.00810	0.00100	9,856
Sequence 10	3.9	9.0	155.4	139.6	16.0	12.9	11.6	1.3	0.00958	0.00119	9,795
Sequence 11	1.9	1.8	60.8	54.2	6.0	5.0	4.5	0.5	0.00430	0.00053	8,520
Sequence 12	1.9	3.6	82.8	74.6	9.0	6.9	6.2	0.7	0.00600	0.00075	8,275
Sequence 13	1.9	5.4	108.0	97.0	11.0	9.0	8.1	0.9	0.00790	0.00098	8,219
Sequence 14	1.9	7.2	131.6	118.2	13.0	11.0	9.8	1.1	0.00950	0.00118	8,329
Sequence 15	1.9	9.0	155.8	140.2	16.0	13.0	11.7	1.3	0.01130	0.00141	8,302

TESTED BY: S. Bivings DATE: 1/4/2021
REVIEWED BY: A. Cooley DATE: 1/26/2021

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

Job No.	200518	**Material Code:**	CL
Date Sampled	11/5/2020	**Station No.:**	511+14, 27.5' LT
Date Tested:	1/4/2021	**Location:**	S5-B
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 38	**Name:**	Lawrence
Sampled By:	SoilTech	**Depth:**	0 - 5
Lab No.:	16152	**AASHTO Class:**	A-4 (2)
Sample ID:	1	**Material Type (1 or 2)**	2
LATITUDE:	35° 54' 38.56"	**LONGITUDE:**	90° 53' 36.01"

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

K1 = 7,249

K2 = -0.03959

K5 = 0.29816

R^2 = 0.97

BURNS COOLEY DENNIS, INC.

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

Job No. 200518
Date Sampled: November 5, 2020
Date Tested: January 4, 2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: SoilTech
Lab No.: 16153
Sample ID: 2
LATITUDE: 35° 54' 38.11"

Material Code CL
Station No.: 523+82.25. 34' LT
Location: S5-C

Depth: 0 - 5
AASHTO Class: A-6 (11)
Material Type (1 or 2): 2
LONGITUDE: 90° 53' 21.72"

---

1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No) N
Testing - Permanent Strain > 5% (Y=Yes or N=No) N
Number of Load Sequences Completed (0-15) 15

2. Specimen Information:

Specimen Diameter (in):
- Top 3.96
- Middle 3.96
- Bottom 3.96
- Average 3.96

Membrane Thickness (in): 0.025
Height of Specimen, Cap and Base (in): 13.43
Height of Cap and Base (in): 5.38
Initial Length, Lo (in): 8.05
Initial Area, Ao (sq. in): 12.32
Initial Volume, AoLo (cu. in): 99.18

3. Soil Specimen Weight:

Weight of Wet Soil Used (g): 5000

4. Soil Properties:

Optimum Moisture Content (%): 13.5
Maximum Dry Density (pcf): 107.4
95% of MDD (pcf): 102.0
In-Situ Moisture Content (%): N/A

5. Specimen Properties:

Wet Weight (g): 2976.4
Compaction Moisture Content (%): 13.5
Compaction Wet Density (pcf): 114.4
Compaction Dry Density (pcf): 101.2
Moisture Content After Mr Test (%): 13.0

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

7. Resilient Modulus, Mr: Mr = 12460 (Sc)^-0.12022 (S3)^0.18833

8. Comments

9. Tested By: Scott Bivings Date: January 4, 2021

BURNS COOLEY DENNIS, INC.

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

Job No. 200518
Date Sampled 11/5/2020
Date Tested: 1/4/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: SoilTech
Lab No.: 16153
Sample ID: 2
LATITUDE: 35° 54' 38.11"

Material Code: CL
Station No.: 523+82.25, 34' LT
Location: S5-C
Depth: 0 - 5
AASHTO Class: A-6 (11)
Material Type (1 or 2) 2
LONGITUDE: 90° 53' 21.72"

PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Actual Applied Max. Axial Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Axial Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Average Recov Def. LVDT 1 and 2	Resilient Strain	Resilient Modulus
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	5.7	1.8	60.0	53.6	6.0	5.0	4.5	0.5	0.00222	0.00028	16,140
Sequence 2	5.7	3.6	84.2	76.0	8.2	7.0	6.3	0.7	0.00330	0.00041	15,347
Sequence 3	5.7	5.4	107.8	96.8	11.0	9.0	8.0	0.9	0.00450	0.00056	14,378
Sequence 4	5.8	7.2	131.8	118.6	13.0	11.0	9.9	1.1	0.00586	0.00073	13,554
Sequence 5	5.7	9.0	156.8	141.0	16.0	13.1	11.7	1.3	0.00738	0.00091	12,844
Sequence 6	3.8	1.8	60.2	54.2	6.0	5.0	4.5	0.5	0.00250	0.00031	14,603
Sequence 7	3.8	3.6	83.8	75.2	8.2	7.0	6.3	0.7	0.00360	0.00045	13,932
Sequence 8	3.8	5.4	108.0	97.0	11.0	9.0	8.1	0.9	0.00490	0.00061	13,228
Sequence 9	3.8	7.2	132.2	118.8	13.0	11.0	9.9	1.1	0.00630	0.00079	12,565
Sequence 10	3.8	9.0	155.8	140.0	16.0	13.0	11.7	1.3	0.00776	0.00096	12,115
Sequence 11	1.9	1.8	59.4	53.4	6.0	4.9	4.4	0.5	0.00280	0.00035	12,786
Sequence 12	1.9	3.6	83.4	75.0	8.4	6.9	6.2	0.7	0.00412	0.00051	12,150
Sequence 13	1.9	5.4	108.0	97.0	11.0	9.0	8.1	0.9	0.00560	0.00069	11,634
Sequence 14	1.9	7.2	132.0	118.8	13.0	11.0	9.9	1.1	0.00710	0.00088	11,192
Sequence 15	1.9	9.0	155.0	139.2	16.0	12.9	11.6	1.3	0.00862	0.00107	10,804

TESTED BY: S. Bivings DATE: 1/4/2021
REVIEWED BY: A. Cooley DATE: 1/26/2021

BURNS COOLEY DENNIS, INC.

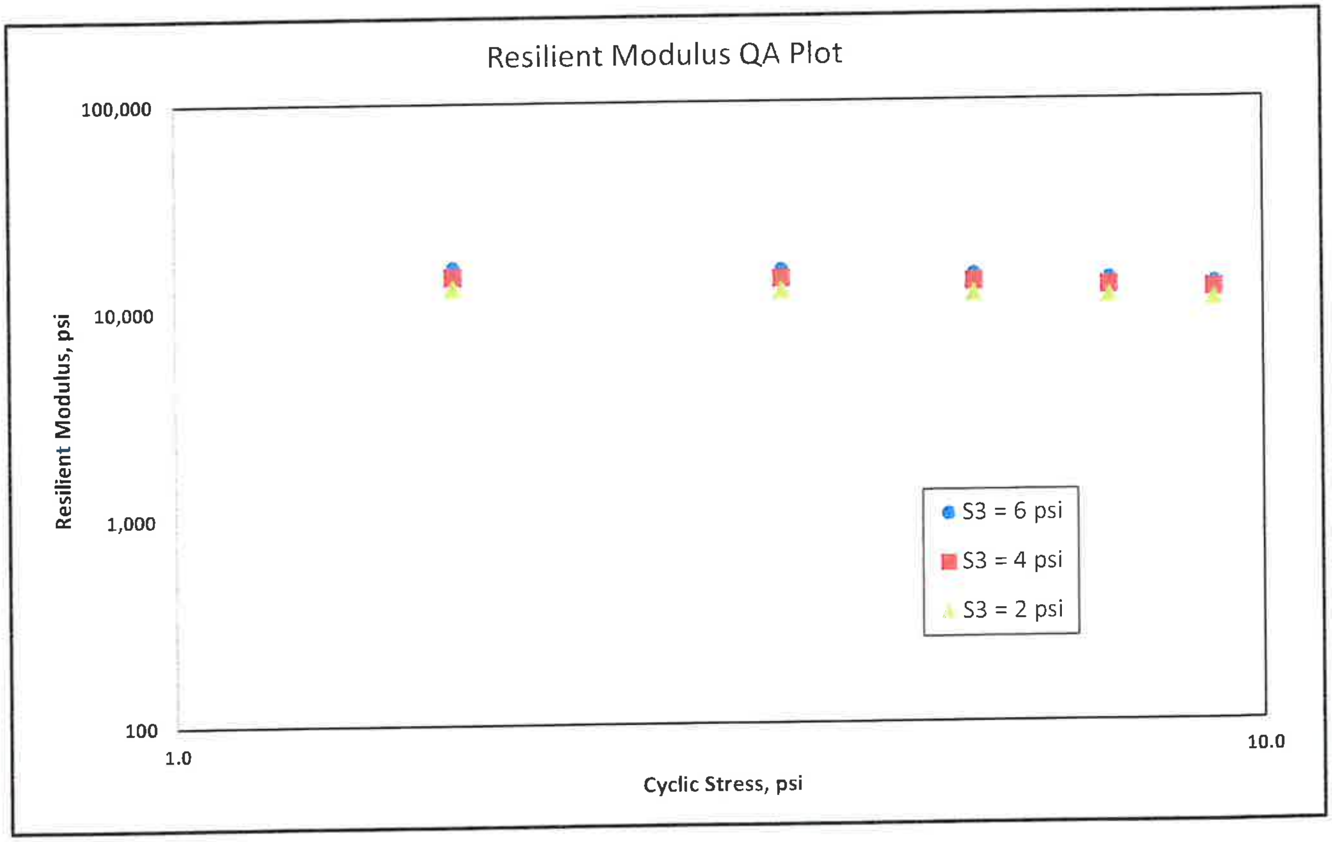
AASHTO T 307-99 - RESILIENT MODULUS OF SUBGRADE SOILS
RECOMPACTED SAMPLES

Job No. 200518
Date Sampled: 11/5/2020
Date Tested: 1/4/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: SoilTech
Lab No.: 16153
Sample ID: 2
LATITUDE: 35° 54' 38.11"

Material Code: CL
Station No.: 523+82.25, 34' LT
Location: S5-C
Depth: 0 - 5
AASHTO Class: A-6 (11)
Material Type (1 or 2) 2
LONGITUDE: 90° 53' 21.72"

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

K1 = 12,460
K2 = -0.12022
K5 = 0.18833
R^2 = 0.97



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

Job No.	200518	**Material Code**	CL
Date Sampled:	November 5, 2020	**Station No.:**	613 + 14. 68' LT
Date Tested:	January 4, 2021	**Location:**	S6-B
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 38 **Name:** Lawrence		
Sampled By:	SoilTech	**Depth:**	0 - 5
Lab No.:	16155	**AASHTO Class:**	A-6 (9)
Sample ID:	3	**Material Type (1 or 2):**	2
LATITUDE:	35° 54' 37.25"	**LONGITUDE:**	90° 52' 38.06"

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1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No)	N
Testing - Permanent Strain > 5% (Y=Yes or N=No)	N
Number of Load Sequences Completed (0-15)	15

2. Specimen Information:

Specimen Diameter (in):	
Top	3.96
Middle	3.96
Bottom	3.96
Average	3.96
Membrane Thickness (in):	0.025
Height of Specimen, Cap and Base (in):	13.43
Height of Cap and Base (in):	5.38
Initial Length, Lo (in):	8.05
Initial Area, Ao (sq. in):	12.32
Initial Volume, AoLo (cu. in):	99.18

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	5000

4. Soil Properties:

Optimum Moisture Content (%):	14.7
Maximum Dry Density (pcf):	108.8
95% of MDD (pcf):	103.4
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	3089.0
Compaction Moisture Content (%):	14.7
Compaction Wet Density (pcf):	118.7
Compaction Dry Density (pcf):	103.9
Moisture Content After Mr Test (%):	14.2

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

7. Resilient Modulus, Mr: Mr = 12941 (Sc)^-0.15716 (S3)^0.22235

8. Comments

9. Tested By: Scott Bivings

Date: January 4, 2021

BURNS COOLEY DENNIS, INC.

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

Job No. 200518
Date Sampled 11/5/2020
Date Tested: 1/4/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: SoilTech
Lab No.: 16155
Sample ID: 3
LATITUDE: 35° 54' 37.25"

Material Code: CL
Station No.: 613+14, 68' LT
Location: S6-B
Depth: 0 - 5
AASHTO Class: A-6 (9)
Material Type (1 or 2) 2
LONGITUDE: 90° 52' 38.06"

PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Actual Applied Max. Axial Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Axial Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Average Recov Def. LVDT 1 and 2	Resilient Strain	Resilient Modulus
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	5.8	1.8	59.8	53.8	6.0	5.0	4.5	0.5	0.00200	0.00025	17,660
Sequence 2	5.7	3.6	84.8	76.4	8.2	7.1	6.4	0.7	0.00310	0.00039	16,443
Sequence 3	5.8	5.4	107.2	96.2	11.0	8.9	8.0	0.9	0.00430	0.00054	14,995
Sequence 4	5.8	7.2	132.8	119.2	13.0	11.0	9.9	1.1	0.00580	0.00072	13,762
Sequence 5	5.8	9.0	154.8	139.6	16.0	12.9	11.6	1.3	0.00720	0.00089	12,982
Sequence 6	3.7	1.8	60.0	54.0	6.0	5.0	4.5	0.5	0.00230	0.00029	15,444
Sequence 7	3.7	3.6	83.8	75.4	8.6	7.0	6.3	0.7	0.00350	0.00044	14,332
Sequence 8	3.7	5.4	107.2	96.2	11.0	8.9	8.0	0.9	0.00482	0.00060	13,329
Sequence 9	3.7	7.2	132.2	118.8	13.0	11.0	9.9	1.1	0.00632	0.00079	12,562
Sequence 10	3.7	9.0	155.6	139.6	16.0	13.0	11.6	1.3	0.00780	0.00097	11,989
Sequence 11	1.9	1.8	59.6	53.4	6.0	5.0	4.4	0.5	0.00280	0.00034	13,006
Sequence 12	1.8	3.6	83.8	75.0	8.6	7.0	6.3	0.7	0.00412	0.00051	12,189
Sequence 13	1.8	5.4	107.2	96.2	11.0	8.9	8.0	0.9	0.00560	0.00070	11,490
Sequence 14	1.8	7.2	131.4	118.2	13.0	10.9	9.8	1.1	0.00722	0.00090	10,972
Sequence 15	1.8	9.0	154.8	139.2	16.0	12.9	11.6	1.3	0.00880	0.00109	10,613

TESTED BY: S. Bivings DATE: 1/4/2021
REVIEWED BY: A. Cooley DATE: 1/26/2021

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

Job No.	200518			**Material Code:**	CL
Date Sampled:	11/5/2020			**Station No.:**	613+14, 68' LT
Date Tested:	1/4/2021			**Location:**	S6-B
Name of Project:	SR 230 Bridges and Approaches				
County:		**Code:** 38	**Name:**	Lawrence	
Sampled By:	SoilTech		SoilTech	**Depth:**	0 - 5
Lab No.:	16155			**AASHTO Class:**	A-6 (9)
Sample ID:	3			**Material Type (1 or 2)**	2
LATITUDE:	35° 54' 37.25"			**LONGITUDE:**	90° 52' 38.06"

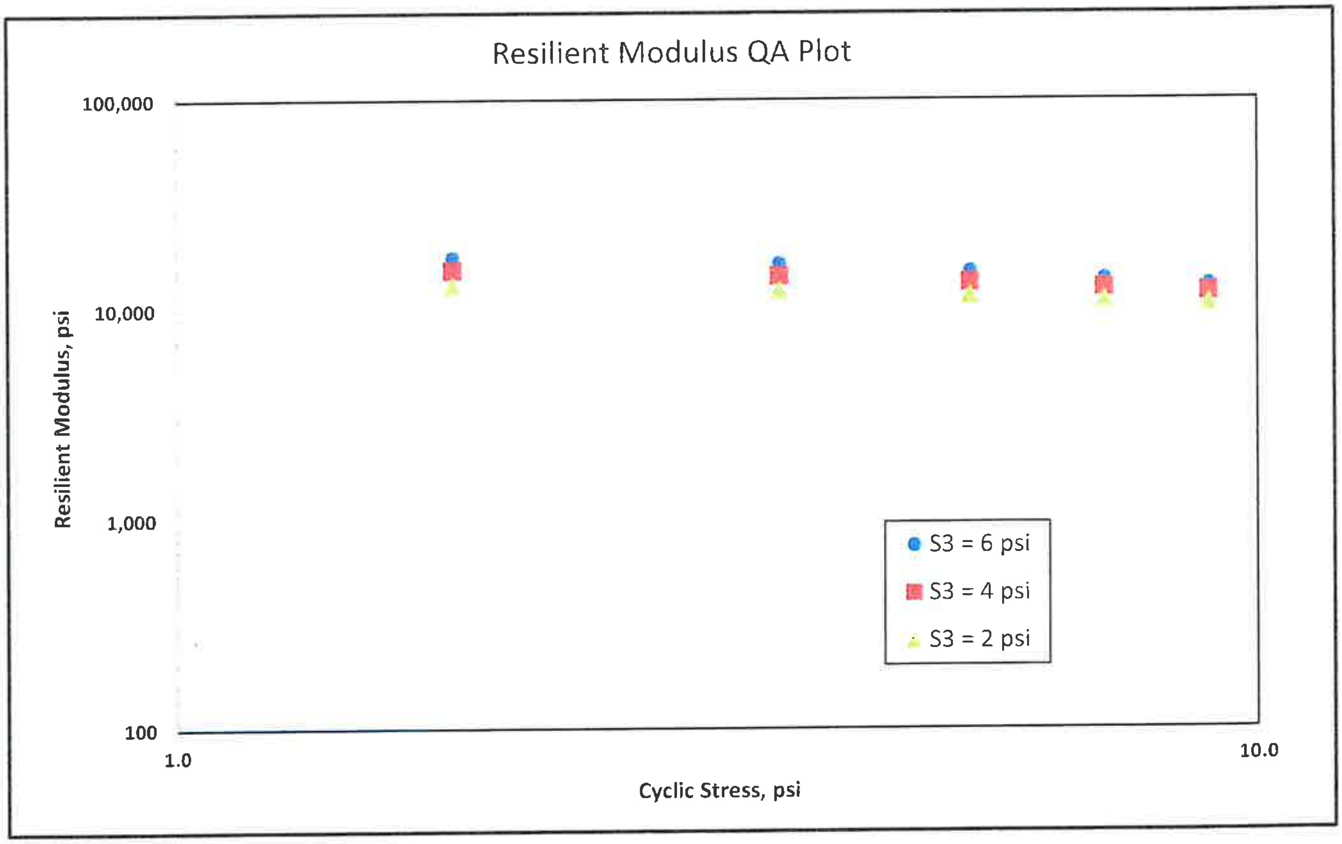
$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 12,941

K2 = -0.15716

K5 = 0.22235

R^2 = 0.96



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

Job No.	200518	**Material Code**	CL
Date Sampled:	March 2, 2021	**Station No.:**	700+84.75, 9.5' LT
Date Tested:	March 27, 2021	**Location:**	S7-A
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 38 **Name:** Lawrence		
Sampled By:	BCD	**Depth:**	0 - 5
Lab No.:	16300	**AASHTO Class:**	A-6(8)
Sample ID:	11	**Material Type (1 or 2):**	2
LATITUDE:	35° 54' 36.43"	**LONGITUDE:**	90° 52' 29.91"

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1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No)	N
Testing - Permanent Strain > 5% (Y=Yes or N=No)	N
Number of Load Sequences Completed (0-15)	15

2. Specimen Information:

Specimen Diameter (in):	
Top	3.96
Middle	3.96
Bottom	3.96
Average	3.96
Membrane Thickness (in):	0.025
Height of Specimen, Cap and Base (in):	13.43
Height of Cap and Base (in):	5.38
Initial Length, Lo (in):	8.05
Initial Area, Ao (sq. in):	12.32
Initial Volume, AoLo (cu. in):	99.18

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	5000

4. Soil Properties:

Optimum Moisture Content (%):	13.2
Maximum Dry Density (pcf):	109.8
95% of MDD (pcf):	104.3
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	3118.2
Compaction Moisture Content (%):	13.2
Compaction Wet Density (pcf):	119.8
Compaction Dry Density (pcf):	106.2
Moisture Content After Mr Test (%):	12.8

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

7. Resilient Modulus, Mr: Mr = 16879 (Sc)^-0.17103 (S3)^0.11074

8. Comments

9. Tested By: Scott Bivings **Date:** March 29, 2021

BURNS COOLEY DENNIS, INC.

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

Job No. 200518
Date Sampled 3/2/2021
Date Tested: 3/27/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: BCD
Lab No.: 16300
Sample ID: 11
LATITUDE: 35° 54' 36.43"

Material Code: CL
Station No.: 700+84.75, 9.5' LT
Location: S7-A
Depth: 0 - 5
AASHTO Class: A-6(8)
Material Type (1 or 2) 2
LONGITUDE: 90° 52' 29.91"

PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Actual Applied Max. Axial Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Axial Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Average Recov Def. LVDT 1 and 2	Resilient Strain	Resilient Modulus
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	5.9	1.8	59.2	53.0	6.0	4.9	4.4	0.5	0.00190	0.00024	18,458
Sequence 2	5.9	3.6	84.0	75.4	8.6	7.0	6.3	0.7	0.00290	0.00036	17,380
Sequence 3	5.9	5.4	108.4	97.4	11.0	9.0	8.1	0.9	0.00412	0.00052	15,765
Sequence 4	5.9	7.2	132.8	119.6	13.0	11.1	10.0	1.1	0.00558	0.00069	14,405
Sequence 5	5.9	9.0	156.0	140.2	16.0	13.0	11.7	1.3	0.00700	0.00087	13,472
Sequence 6	3.9	1.8	60.6	54.0	6.0	5.0	4.5	0.5	0.00210	0.00026	17,266
Sequence 7	3.9	3.6	83.8	75.2	8.8	7.0	6.3	0.7	0.00310	0.00039	16,151
Sequence 8	3.9	5.4	108.6	97.6	11.0	9.0	8.1	0.9	0.00436	0.00054	14,986
Sequence 9	3.9	7.2	132.6	119.0	13.2	11.0	9.9	1.1	0.00570	0.00071	14,042
Sequence 10	3.9	9.0	156.8	141.0	16.0	13.1	11.7	1.3	0.00720	0.00090	13,121
Sequence 11	1.9	1.8	61.0	55.0	6.0	5.1	4.6	0.5	0.00232	0.00029	15,723
Sequence 12	1.9	3.6	84.4	76.0	8.8	7.0	6.3	0.7	0.00340	0.00043	14,865
Sequence 13	1.9	5.4	107.6	96.2	11.0	9.0	8.0	0.9	0.00466	0.00058	13,891
Sequence 14	1.9	7.2	131.4	118.0	13.2	10.9	9.8	1.1	0.00602	0.00075	13,087
Sequence 15	1.9	9.0	155.2	139.6	16.0	12.9	11.6	1.3	0.00760	0.00094	12,308

TESTED BY: S. Bivings DATE: 3/29/2021
REVIEWED BY: A. Cooley DATE: 4/5/2021

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

Job No.	200518	**Material Code:**	CL
Date Sampled	3/2/2021	**Station No.:**	700+84.75, 9.5' LT
Date Tested:	3/27/2021	**Location:**	S7-A
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 38	**Name:**	Lawrence
Sampled By:	BCD	**Depth:**	0 - 5
Lab No.:	16300	**AASHTO Class:**	A-6(8)
Sample ID:	11	**Material Type (1 or 2)**	2
LATITUDE:	35° 54' 36.43"	**LONGITUDE:**	90° 52' 29.91"

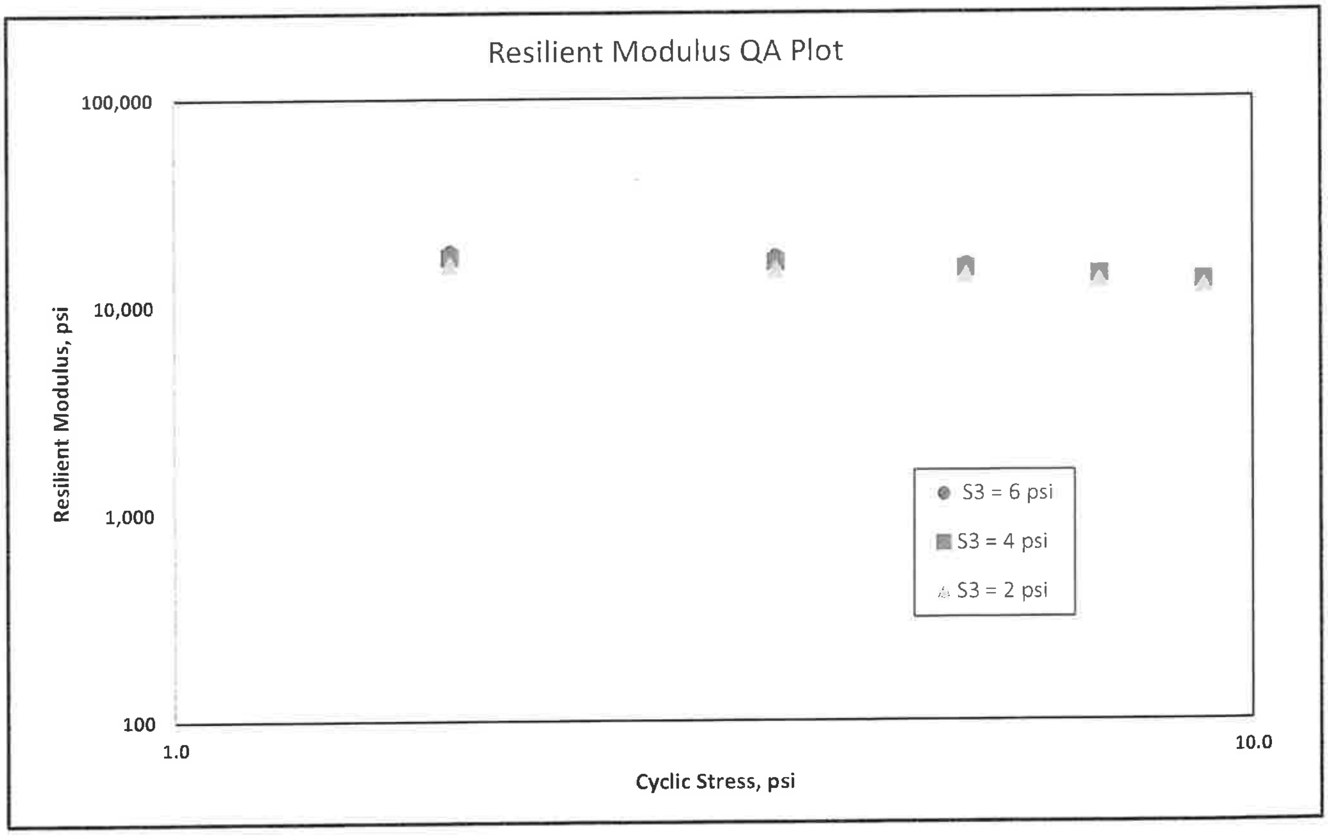
$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 16,879

K2 = -0.17103

K5 = 0.11074

R^2 = 0.93



BURNS COOLEY DENNIS, INC.

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

Job No.	200518	**Material Code**	SC
Date Sampled:	November 5, 2020	**Station No.:**	704+37, 66' LT
Date Tested:	January 4, 2021	**Location:**	S7-B
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 38 **Name:** Lawrence		
Sampled By:	SoilTech	**Depth:**	0 - 5
Lab No.:	16157	**AASHTO Class:**	A-6 (6)
Sample ID:	4	**Material Type (1 or 2):**	2
LATITUDE:	35° 54' 36.86"	**LONGITUDE:**	90° 52' 25.66"

1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No)	N
Testing - Permanent Strain > 5% (Y=Yes or N=No)	N
Number of Load Sequences Completed (0-15)	15

2. Specimen Information:

Specimen Diameter (in):	
Top	3.96
Middle	3.96
Bottom	3.96
Average	3.96
Membrane Thickness (in):	0.025
Height of Specimen, Cap and Base (in):	13.43
Height of Cap and Base (in):	5.38
Initial Length, Lo (in):	8.05
Initial Area, Ao (sq. in):	12.32
Initial Volume, AoLo (cu. in):	99.18

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	5000

4. Soil Properties:

Optimum Moisture Content (%):	12.8
Maximum Dry Density (pcf):	110.8
95% of MDD (pcf):	105.3
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	3094.7
Compaction Moisture Content (%):	12.8
Compaction Wet Density (pcf):	118.9
Compaction Dry Density (pcf):	105.6
Moisture Content After Mr Test (%):	12.6

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

7. Resilient Modulus, Mr: Mr = 9002 (Sc)^-0.09953 (S3)^0.22090

8. Comments ____________________

9. Tested By: Scott Bivings

Date: January 4, 2021

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

Job No. 200518
Date Sampled 11/5/2020
Date Tested: 1/4/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 38 **Name:** Lawrence
Sampled By: SoilTech
Lab No.: 16157
Sample ID: 4
LATITUDE: 35° 54' 36.86"

Material Code: CL
Station No.: 704+37, 66' LT
Location: S7-B
Depth: 0 - 5
AASHTO Class: A-6 (6)
Material Type (1 or 2) 2
LONGITUDE: 90° 52' 25.66"

PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Actual Applied Max. Axial Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Axial Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Average Recov Def. LVDT 1 and 2	Resilient Strain	Resilient Modulus
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	5.6	1.8	59.6	53.6	6.0	5.0	4.5	0.5	0.00284	0.00035	12,680
Sequence 2	5.6	3.6	83.2	74.6	8.0	6.9	6.2	0.7	0.00414	0.00052	12,061
Sequence 3	5.6	5.4	107.4	96.6	11.0	8.9	8.0	0.9	0.00578	0.00072	11,237
Sequence 4	5.6	7.2	132.0	119.0	13.0	11.0	9.9	1.1	0.00742	0.00092	10,690
Sequence 5	5.6	9.0	155.6	140.0	16.0	13.0	11.7	1.3	0.00910	0.00113	10,326
Sequence 6	3.7	1.8	59.8	53.8	6.0	5.0	4.5	0.5	0.00322	0.00040	11,144
Sequence 7	3.7	3.6	83.2	74.8	8.0	6.9	6.2	0.7	0.00476	0.00059	10,548
Sequence 8	3.7	5.4	107.8	97.0	11.0	9.0	8.1	0.9	0.00640	0.00080	10,102
Sequence 9	3.7	7.2	131.8	118.6	13.0	11.0	9.9	1.1	0.00810	0.00101	9,798
Sequence 10	3.7	9.0	154.8	139.2	16.0	12.9	11.6	1.3	0.00980	0.00122	9,514
Sequence 11	1.9	1.8	59.8	53.8	6.0	5.0	4.5	0.5	0.00378	0.00047	9,540
Sequence 12	1.9	3.6	84.0	75.4	8.0	7.0	6.3	0.7	0.00560	0.00069	9,055
Sequence 13	1.8	5.4	108.8	97.8	11.0	9.1	8.2	0.9	0.00748	0.00093	8,785
Sequence 14	1.8	7.2	131.8	118.6	13.0	11.0	9.9	1.1	0.00922	0.00114	8,628
Sequence 15	1.9	9.0	155.6	140.2	15.6	13.0	11.7	1.3	0.01110	0.00138	8,443

TESTED BY: S. Bivings DATE: 1/4/2021
REVIEWED BY: A. Cooley DATE: 1/26/2021

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

Job No.	200518	**Material Code:**	CL
Date Sampled:	11/5/2020	**Station No.:**	704+37, 66' LT
Date Tested:	1/4/2021	**Location:**	S7-B
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 38	**Name:**	Lawrence
Sampled By:	SoilTech	**Depth:**	0 - 5
Lab No.:	16157	**AASHTO Class:**	A-6 (6)
Sample ID:	4	**Material Type (1 or 2)**	2
LATITUDE:	35° 54' 36.86"	**LONGITUDE:**	90° 52' 25.66"

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

K1 = 9,002

K2 = -0.09953

K5 = 0.22090

R^2 = 0.97

BURNS COOLEY DENNIS, INC.

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

Job No.	200518	**Material Code**	CL
Date Sampled:	November 5, 2020	**Station No.:**	819+74.33, 29' LT
Date Tested:	January 4, 2021	**Location:**	S8-A
Name of Project:	SR 230 Bridges and Approaches		
County:	**Code:** 16 **Name:** Craighead		
Sampled By:	SoilTech	**Depth:**	0 - 5
Lab No.:	16158	**AASHTO Class:**	A-6 (8)
Sample ID:	5	**Material Type (1 or 2):**	2
LATITUDE:	35° 54' 35.08"	**LONGITUDE:**	90° 51' 42.72"

1. Testing Information:

Preconditioning - Permanent Strain > 5% (Y=Yes or N= No)	N
Testing - Permanent Strain > 5% (Y=Yes or N=No)	N
Number of Load Sequences Completed (0-15)	15

2. Specimen Information:

Specimen Diameter (in):	
Top	3.96
Middle	3.96
Bottom	3.96
Average	3.96
Membrane Thickness (in):	0.025
Height of Specimen, Cap and Base (in):	13.43
Height of Cap and Base (in):	5.38
Initial Length, Lo (in):	8.05
Initial Area, Ao (sq. in):	12.32
Initial Volume, AoLo (cu. in):	99.18

3. Soil Specimen Weight:

Weight of Wet Soil Used (g):	5000

4. Soil Properties:

Optimum Moisture Content (%):	12.1
Maximum Dry Density (pcf):	108.8
95% of MDD (pcf):	103.4
In-Situ Moisture Content (%):	N/A

5. Specimen Properties:

Wet Weight (g):	2996.0
Compaction Moisture Content (%):	12.1
Compaction Wet Density (pcf):	115.1
Compaction Dry Density (pcf):	103.1
Moisture Content After Mr Test (%):	11.7

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

7. Resilient Modulus, Mr: Mr = 13416 (SC)^-0.06879 (S3)^0.24946

8. Comments

9. Tested By: Scott Bivings **Date:** January 4, 2021

BURNS COOLEY DENNIS, INC.

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

Job No. 200518
Date Sampled 11/5/2020
Date Tested: 1/4/2021
Name of Project: SR 230 Bridges and Approaches
County: **Code:** 16 **Name:** Craighead
Sampled By: SoilTech
Lab No.: 16158
Sample ID: 5
LATITUDE: 35° 54' 35.08"

Material Code: CL
Station No.: 819+74.33, 29' LT
Location: S8-A

Depth: 0 - 5
AASHTO Class: A-6 (8)
Material Type (1 or 2) 2
LONGITUDE: 90° 51' 42.72"

PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Actual Applied Max. Axial Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Axial Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Average Recov Def. LVDT 1 and 2	Resilient Strain	Resilient Modulus
DESIGNATION	S_3	S_{cyclic}	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_{avg}	ε_r	M_r
UNIT	psi	psi	lbs	lbs	lbs	psi	psi	psi	in	in/in	psi
Sequence 1	5.8	1.8	60.0	54.0	6.0	5.0	4.5	0.5	0.00180	0.00022	20,474
Sequence 2	5.7	3.6	84.0	75.8	8.4	7.0	6.3	0.7	0.00260	0.00032	19,850
Sequence 3	5.7	5.4	108.4	97.4	11.0	9.0	8.1	0.9	0.00350	0.00043	18,758
Sequence 4	5.8	7.2	133.2	119.8	13.0	11.1	10.0	1.1	0.00450	0.00056	17,946
Sequence 5	5.8	9.0	157.6	142.0	16.0	13.1	11.8	1.3	0.00552	0.00069	17,241
Sequence 6	3.7	1.8	60.0	53.8	6.0	5.0	4.5	0.5	0.00210	0.00026	17,407
Sequence 7	3.7	3.6	84.4	75.8	8.4	7.0	6.3	0.7	0.00300	0.00037	17,003
Sequence 8	3.7	5.4	108.8	97.8	11.0	9.1	8.1	0.9	0.00398	0.00049	16,507
Sequence 9	3.6	7.2	133.8	120.2	13.0	11.1	10.0	1.1	0.00504	0.00063	15,981
Sequence 10	3.7	9.0	157.2	141.4	16.0	13.1	11.8	1.3	0.00610	0.00075	15,614
Sequence 11	1.8	1.8	59.8	53.8	6.0	5.0	4.5	0.5	0.00250	0.00031	14,458
Sequence 12	1.8	3.6	83.6	75.0	8.2	7.0	6.3	0.7	0.00354	0.00044	14,189
Sequence 13	1.8	5.4	107.4	96.8	11.0	8.9	8.0	0.9	0.00460	0.00057	14,017
Sequence 14	1.8	7.2	132.8	119.6	13.0	11.1	10.0	1.1	0.00580	0.00072	13,824
Sequence 15	1.8	9.0	157.4	142.0	16.0	13.1	11.8	1.3	0.00696	0.00086	13,690

TESTED BY: S. Bivings DATE: 1/4/2021
REVIEWED BY: A. Cooley DATE: 1/26/2021

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

Job No. 200518

Date Sampled: 11/5/2020

Date Tested: 1/4/2021

Name of Project: SR 230 Bridges and Approaches

County: **Code:** 16 **Name:** Craighead

Sampled By:

Lab No.: 16158

Sample ID: 5

LATITUDE: 35° 54' 35.08"

Material Code: CL

Station No.: 819+74, 33' LT

Location: S8-A

Depth: 0 - 5

AASHTO Class: A-6 (8)

Material Type (1 or 2) 2

LONGITUDE: 90° 51' 42.72"

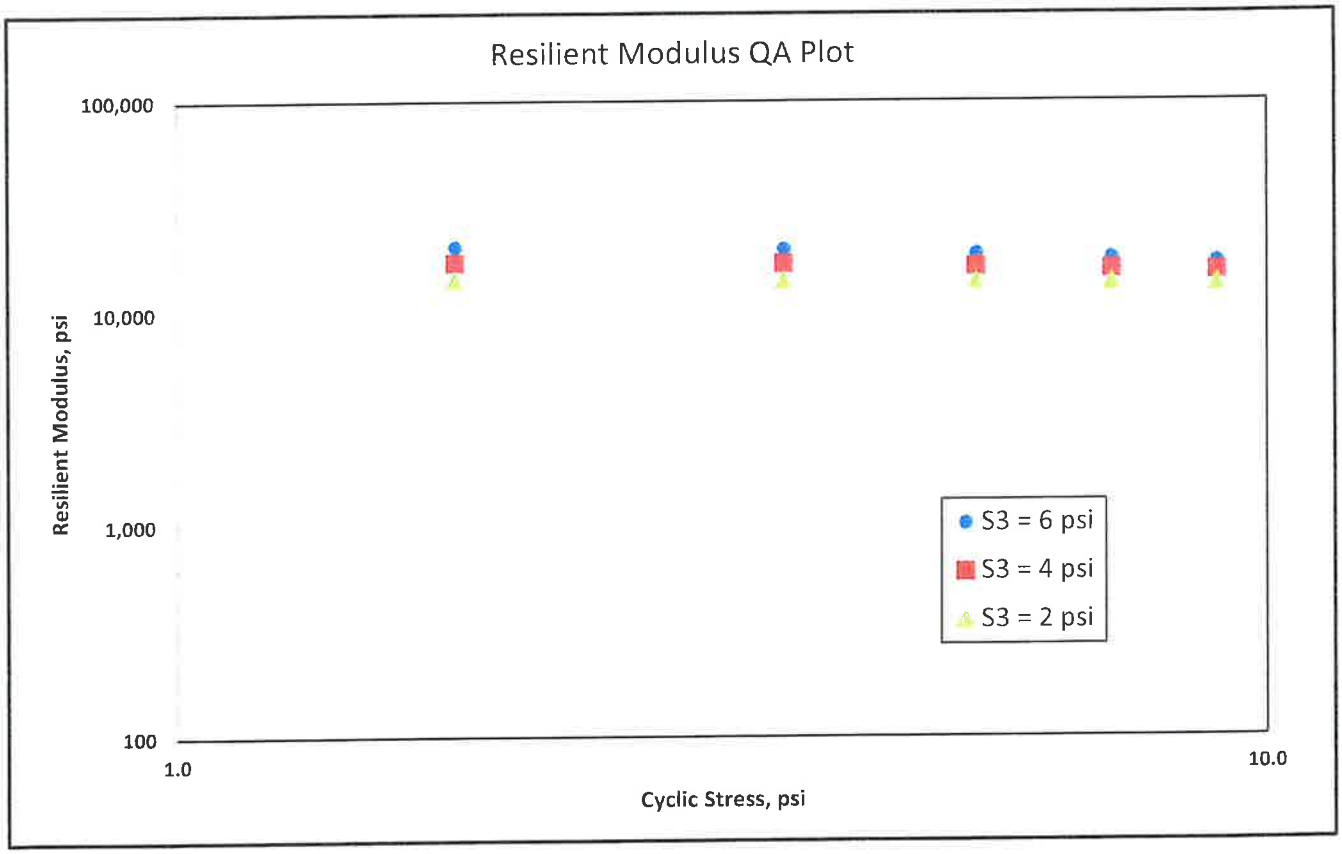
$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 13,416

K2 = -0.06879

K5 = 0.24946

R^2 = 0.96



BURNS COOLEY DENNIS, INC.

BURNS COOLEY DENNIS, INC.

GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

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Fax: (601) 853-2077

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Materials Laboratory
278 Commerce Park Drive
Ridgeland, MS 39157
Phone: (601) 856-2332
Fax: (601) 856-3552

June 1, 2021

Neel-Schaffer, Inc.
125 South Congress Street, Suite 1100
Jackson, Mississippi 39201

Attention: Keith Purvis, P.E.
Vice President, Transportation Department

BCD Project No. 200518

Re: Roadway Subgrade Boring Summary
ARDOT SR 230 – Alicia to Bono
Lawrence and Craighead Counties, Arkansas

Dear Mr. Purvis,

Submitted here is the BCD report presenting the summary of roadway subgrade borings for representative near surface subgrade soils encountered along the alignment of the referenced project. This field and laboratory testing was performed for Task Order 108 and was authorized by Subconsultant Agreement between Neel-Schaffer, Inc. and Burns Cooley Dennis, Inc.

We appreciate the opportunity to be of service. If you should have any questions, please do not hesitate to call us.

Very Truly Yours,
Burns Cooley Dennis, Inc.

Grant Jones E.I.

RC Ahlrich

R. C. Ahlrich, Ph.D., P.E.

A. E. (Eddie) Templeton, P.E.

No.7833
ALEXIS TEMPLETON

BURNS COOLEY DENNIS, INC.

GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

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Memorandum

ALEXIS TEMPLETON
No.7833

BCD Project No. 200518

To: Keith Purvis, P.E.
Neel-Schaffer, Inc.

From: Grant Jones E.I.
Randy Ahlrich, Ph.D., P.E.
Eddie Templeton, P.E.

Date: June 1, 2021

Subject: Roadway Subgrade Boring Summary
ARDOT SR 230 – Alicia to Bono
Lawrence and Craighead Counties, Arkansas

Plans are being made for the construction of replacement bridges and box culverts along with adjacent roadway alignments at 10 sites along Highway 230 between Alicia and Bono in Lawrence and Craighead Counties in Arkansas. Existing roadway subgrade soil conditions within Sites 1 through 9 were explored at twenty-three (23) locations utilizing shallow 5 ft auger borings. These roadway subgrade borings were obtained at selected locations along the roadway alignment by representatives of McCray Drilling, SoilTech Consultants, Inc. and Burns Cooley Dennis, Inc. The roadway subgrade boring locations (Station numbers with offset and GPS coordinates) are presented in the attached summary table (Table 1). The roadway boring locations are also presented on aerial images of the replacement bridges and box culverts sites along Highway 230 between Alicia and Bono, Arkansas (Figures 1A through 1I).

The classification of the representative roadway subgrade material was evaluated by visual observations and laboratory tests (Atterberg liquid and plastic limit tests and sieve analyses). These laboratory tests were performed by SoilTech Consultants, Inc. and Burns Cooley Dennis. Inc. Based on these visual observations and laboratory test results, the AASHTO classifications were determined for each representative subgrade material. The results of these tests are summarized in the attached Table 1. The subgrade soils along Highway 230 were typically found to be fine grained soils with AASHTO classifications of A-2, A-4, A-6 and A-7. The primary subgrade soil type along this roadway is A-6.

TABLE 1
ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

ROADWAY SUBGRADE BORING TABLE

BORING NO.	STATION	LATITUDE			LONGITUDE			LOCATION	DEPTH	LIQUID LIMIT	PLASTICITY INDEX	AASHTO CLASSIFICATION	COLOR
		DEG	MIN	SEC	DEG	MIN	SEC		FEET				
S1-A	101+38.5	35	53	39.69	91	4	32.56	7.25' LT	0-5	39	25	A-6(13)	GRAY/BROWN
S1-B	105+43.5	35	53	39.47	91	4	27.66	9' LT	0-5	28	13	A-6(5)	TAN
S1-C	115+90	35	53	39.13	91	4	14.99	25' LT	0-5	50	32	A-7-6(25)	BROWN
S2-A	117+40	35	53	39.03	91	4	13.13	26.5' LT	0-5	34	20	A-6(13)	TAN
S2-B	125+76	35	53	38.16	91	4	3.74	12.5' LT	0-5	30	14	A-6(2)	BROWN
S2-C	129+87.75	35	53	37.69	91	3	58.78	8' LT	0-5	35	19	A-6(7)	TAN/GRAY
S3-A	305+10	35	53	52.10	90	57	48.30	18' RT	0-5	47	32	A-7-6(31)	TAN
S3-B	315+00	35	53	51.80	90	57	36.80	18.5' RT	0-5	34	19	A-6(10)	TAN
S4-A	405+85.25	35	54	41.35	90	55	8.15	9' LT	0-5	23	7	A-4(1)	TAN
S4-B	413+99	35	54	40.92	90	54	58.15	8.5' RT	0-5	30	15	A-6(9)	TAN
S5-A	504+74.67	35	54	38.68	90	53	41.85	19' LT	0-5	29	14	A-6(2)	TAN
S5-B	511+14	35	54	38.56	90	53	36.01	27.5' LT	0-5	25	9	A-4(2)	TAN
S5-C	523+82.25	35	54	38.11	90	53	21.72	34' LT	0-5	33	19	A-6(11)	TAN
S5-D	528+74	35	54	37.73	90	53	15.73	13' LT	0-5	29	15	A-6(2)	BROWN
S6-A	608+94.75	35	54	36.88	90	52	43.12	15.5' LT	0-5	25	10	A-4(0)	TAN
S6-B	613+14	35	54	37.25	90	52	38.06	68' LT	0-5	34	21	A-6(9)	TAN
S6-C	616+85	35	54	36.62	90	52	33.62	16.5' LT	0-5	26	13	A-2-6(0)	TAN
S7-A	700+84.75	35	54	36.43	90	52	29.91	9.5' LT	0-5	30	18	A-6(6)	BROWN
S7-B	704+37	35	54	36.86	90	52	25.66	66' LT	0-5	32	20	A-6(6)	TAN
S7-C	707+74.5	35	54	36.24	90	52	21.61	16' LT	0-5	29	17	A-6(3)	BROWN
S8-A	819+74.33	35	54	35.08	90	51	42.72	29' LT	0-5	31	17	A-6(8)	TAN
S8-B	828+84.66	35	54	34.83	90	51	40.25	11.5' LT	0-5	34	19	A-6(10)	BROWN/GRAY
S9-A	905+66.75	35	54	33.89	90	50	52.36	68.5' LT	0-5	30	18	A-6(5)	TAN

Soil characteristics tabulated above are representative at the location of the sample. These data are shown for information only. The state will not be responsible for variations in the soil characteristics and/or extent of soil differing from the above tabulations.

BURNS COOLEY DENNIS, INC.

Approximate Boring Locations

SITE 1
ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

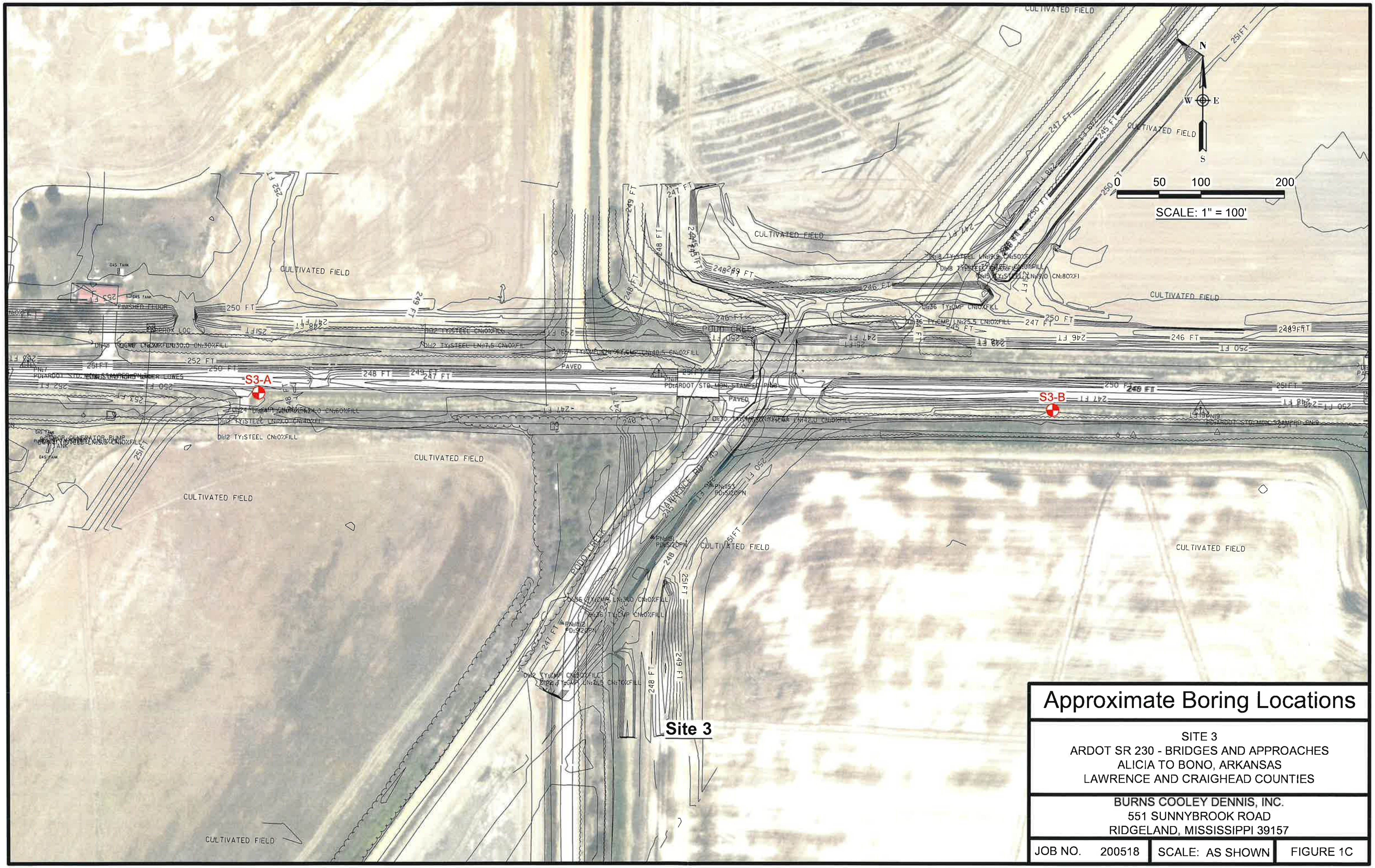
JOB NO. 200518
SCALE: AS SHOWN
FIGURE 1A

Approximate Boring Locations

SITE 2
ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO. 200518	SCALE: AS SHOWN	FIGURE 1B



Approximate Boring Locations

SITE 3
ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO.	200518	SCALE: AS SHOWN	FIGURE 1C

Approximate Boring Locations

SITE 4
ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

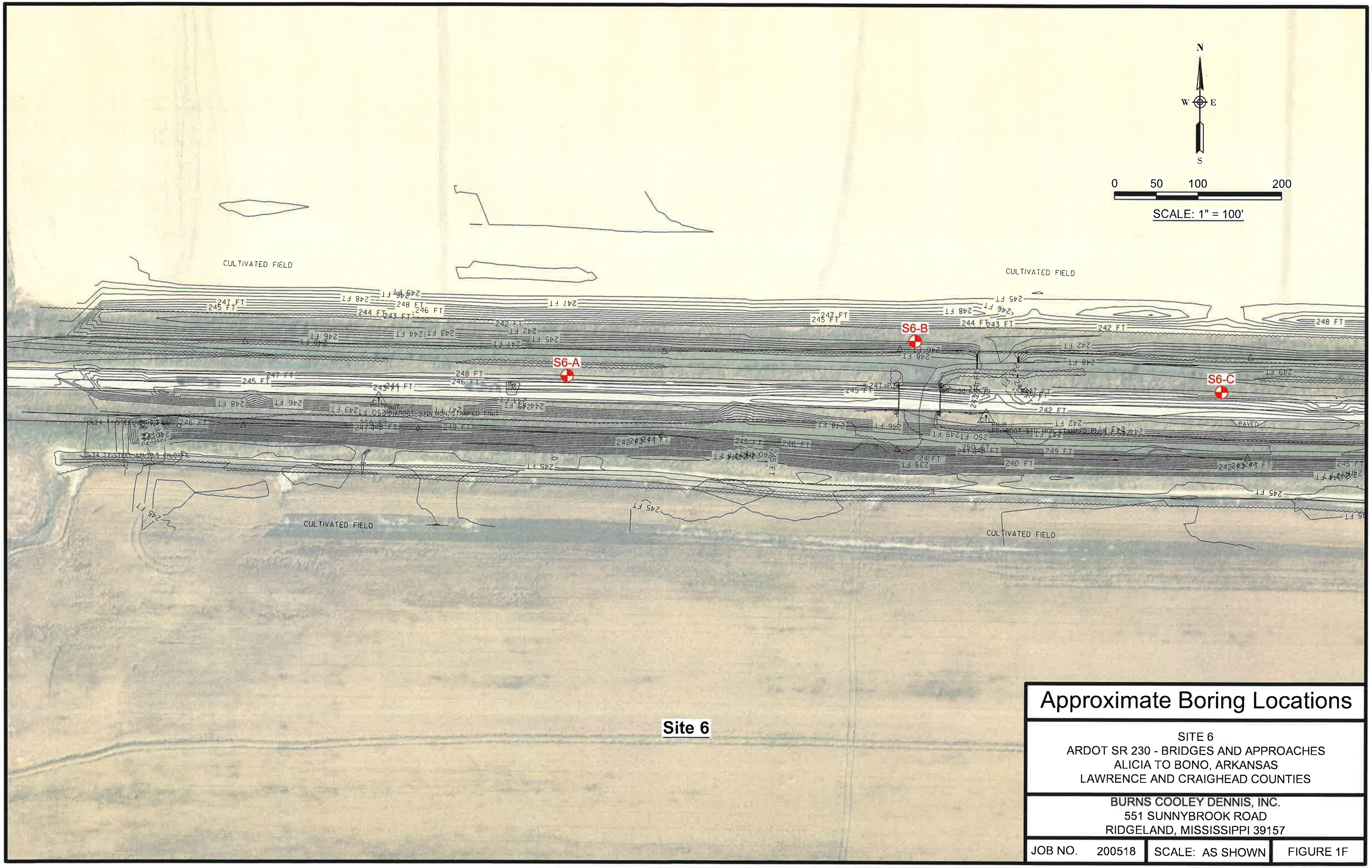
JOB NO. 200518	SCALE: AS SHOWN	FIGURE 1D

Approximate Boring Locations

SITE 5
ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO. 200518	SCALE: AS SHOWN	FIGURE 1E



Approximate Boring Locations

SITE 6
ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

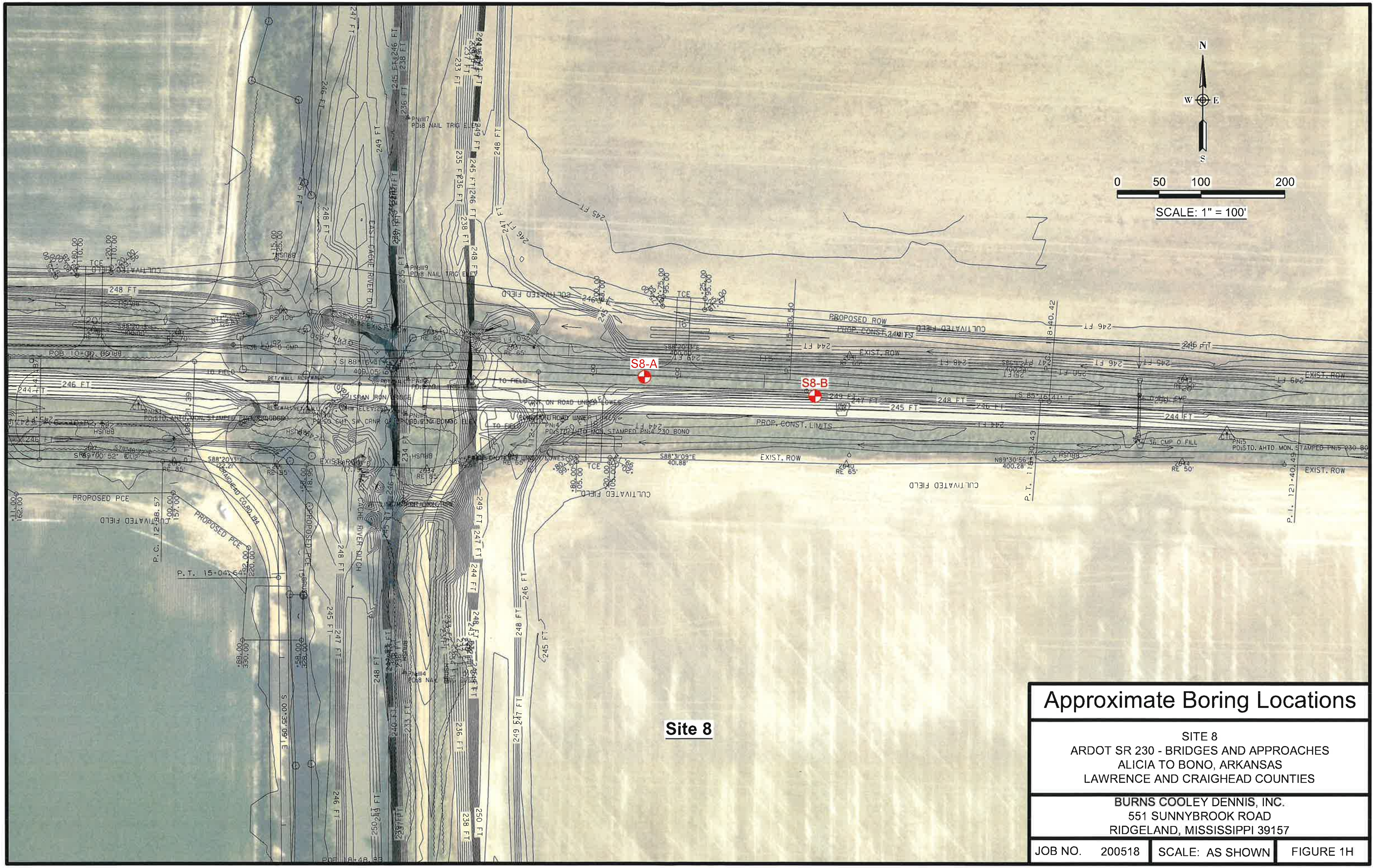
JOB NO.	200518	SCALE: AS SHOWN	FIGURE 1F

Approximate Boring Locations

SITE 7
ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO. 200518	SCALE: AS SHOWN	FIGURE 1G

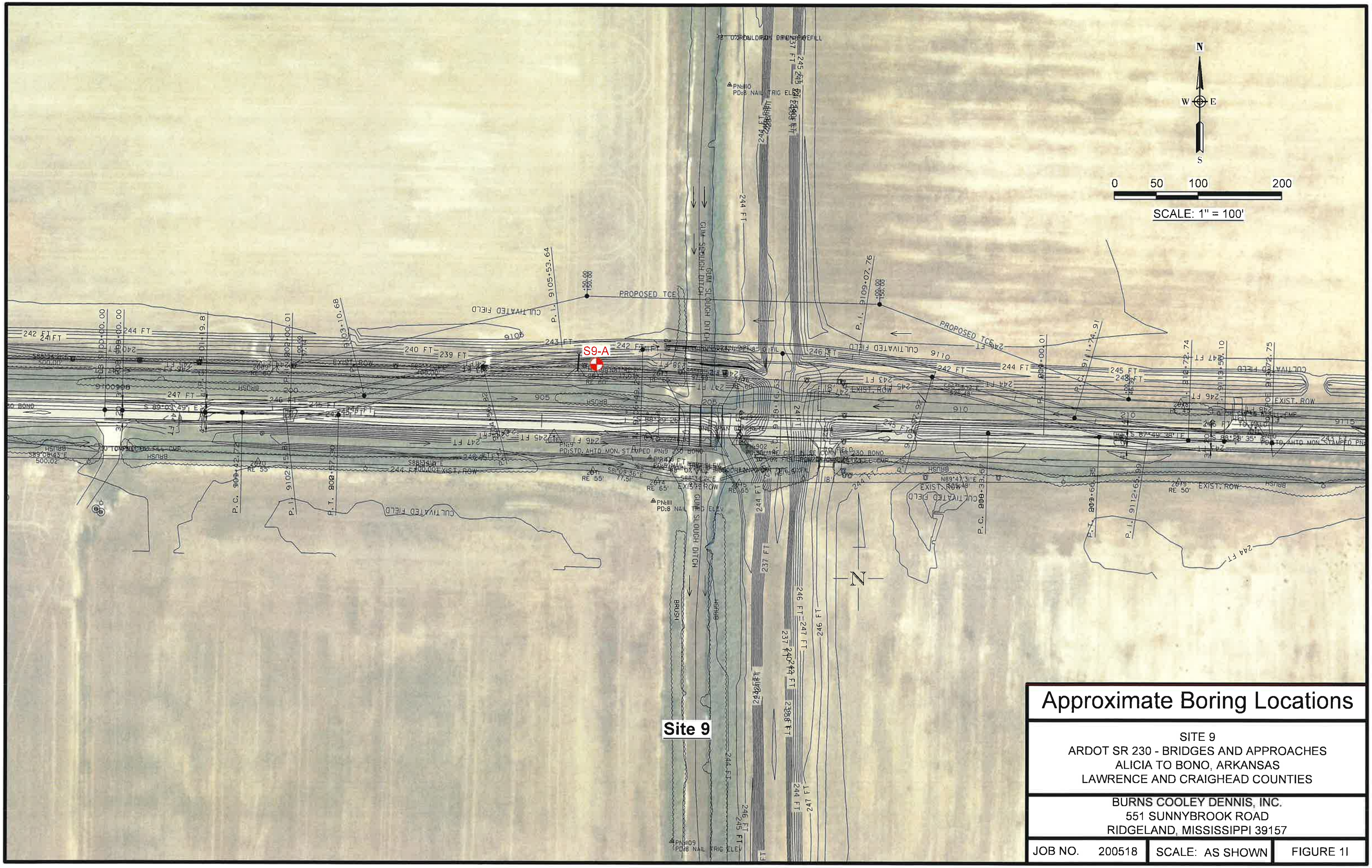


Approximate Boring Locations

SITE 8
ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO.	200518	SCALE: AS SHOWN	FIGURE 1H



Approximate Boring Locations

SITE 9
ARDOT SR 230 - BRIDGES AND APPROACHES
ALICIA TO BONO, ARKANSAS
LAWRENCE AND CRAIGHEAD COUNTIES

BURNS COOLEY DENNIS, INC.
551 SUNNYBROOK ROAD
RIDGELAND, MISSISSIPPI 39157

JOB NO. 200518 | SCALE: AS SHOWN | FIGURE 1I