ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 040847

FEDERAL AID PROJECT NO. NHFP-49-28(1)

HWY. 16/112 SPUR INTCHNG. IMPVTS. (F)

STATE HIGHWAY I-49 **SECTION** 28

IN WASHINGTON **COUNTY**

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

McCLELLAND CONSULTING ENGINEERS, INC.
DESIGNED TO SERVE

1810 N. College Avenue
P.O. Box 1229
Fayetteville, AR 73703/72702-1229
479-443-2377/Fax 479-443-9241
www.mce.us.com

November 7, 2017

Mr. Ryan Castor, PE
Burns & McDonnell
6815 Isaacs Orchard Road, Suite B3
Springdale, Arkansas 72762

Re: Final Geotechnical Investigation
Hwy. 16/112 Spur Intchng. Impvts (S)
Fayetteville, Arkansas
MCE Job # FY163809

Dear Mr. Castor:

We are submitting herewith the report for the Final Geotechnical Investigation on the above referenced project.

We appreciate the opportunity to provide this service to you. If there are any questions regarding the Geotechnical Investigation, please contact us.

Sincerely yours,

McCLELLAND CONSULTING ENGINEERS, INC.

Steven J. Head, PE
Geotechnical Engineer

Enclosure: Geotechnical Investigation

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FINAL GEOTECHNICAL INVESTIGATION

HWY. 16/112 SPUR INTCHNG. IMPVTS (S)
WASHINGTON COUNTY
Route I-49 Section 29
JOB NO. BB0411
FAP NO. IM-540-1(80)64

CERTIFICATE OF AUTH.
McCLELLAND CONSULTING ENGINEERS, Inc.
No. 24
ARKANSAS - ENGINEER

for

Burns & McDonnell
Springdale, AR

November, 2017

Project No. FY163809

STATE OF ARKANSAS
LICENSED PROFESSIONAL ENGINEER
No. 16434
STEVEN J. HEAD
11/7/2017

Prepared By:

DESIGNED TO SERVE

McClelland Consulting Engineers, Inc.
1810 North College, P.O. Box 1229
Fayetteville, Arkansas 72702-1229
(479) 443-2377, Fax (479)-443-9241

FINAL GEOTECHNICAL INVESTIGATION

HWY. 16/112 SPUR INTCHNG. IMPVTS (S)

WASHINGTON COUNTY
ROUTE I-49 SECTION 29
Job No. BB0411
FAP NO. IM-540-1(80)64

for

BURNS & MCDONNELL

SPRINGDALE, ARKANSAS

EXECUTIVE SUMMARY

This is a final report of findings from the subsurface exploration conducted at the planned I-49 Hwy. 16-112 Spur Interchange Improvements project in Washington County in Fayetteville, Arkansas. This report includes information on subsurface conditions and provides recommendations for design and construction of the bridge foundations and embankments, roadway embankments, and site retaining structures. The following is a summary of significant findings:

- **Exit 64 Bridge (Hwy16/W. Wedington Drive):** A total of six (6) borings were conducted at this bridge location in planned bridge widening and extension areas (B-B1 through B-B6). The borings at this location were conducted to auger refusal depth and then advanced between 10 and 20 feet into the basal rock formation. The locations of the referenced bridge borings are provided in Appendix A on Plate 1B.

- **North Shiloh Drive:** Two (2) borings (B-P1 and B-P2) were conducted near planned North Shiloh Drive improvement areas north and south of W. Wedington Drive, respectively. The borings at these locations were conducted to planned terminal depths of ten (10) feet below existing ground elevations. Boring B-P2 encountered auger-refusal material approximately eight (8) feet below existing ground elevation. The locations of the referenced borings along North Shiloh Drive are provided in Appendix A on Plates 1C and 1D.

- **Futrall Drive:** Two (2) borings (B-P3 and B-P4) were conducted near planned Futrall Drive improvement areas south of W. Wedington Drive and east of I-49. Borings B-P3 and B-P4 were unable to achieve planned terminal depths of ten (10) feet below existing ground elevations due to auger refusal material between eight (8) and six (6) feet below existing ground elevations, respectively. The locations of the referenced borings near

Futrall Drive are provided in Appendix A on Plates 1D and 1E. Boring B-5 was unable to be conducted at the time of the investigation due to site accessibility issues.

- In non-paved areas, a surface stratum of dark brown silty topsoil was encountered. The topsoil stratum contained grass, rootlets, and other organic matter and extended to approximately six (6) inches below existing ground elevations at boring locations, but is known to extend upwards of twelve (12) inches in the project area.

- Shallow subgrade soils in the project area were generally found to be fine-grained, sandy lean clays (CL), fat clays (CH), and sandy silts (ML). In-situ strengths varied in the fine-grained materials between firm and stiff. Deeper elevations revealed medium-dense, coarse-grained clayey gravel with sand (GC) material and medium-dense to dense silty sand (SM) and clayey sand (SC) materials. The coarse-grained soils were typically encountered directly above auger refusal material.

- The basal stratum at the project location consists of gray fine-to-coarse-grained limestone indicative of the Mississippian Age Pitkin Formation. Auger refusal resulted where the limestone formation was encountered.

- Groundwater in the form of a perched water table was encountered by Boring B-B3 approximately five (5) feet below existing pavement elevation. Groundwater was not encountered by the terminal depths of any other boring locations at the time of the subsurface investigation.

- Onsite soils conforming to the recommendations found in the **Select Fill Material** section of this report are considered adequate for use as fill material for paved areas and embankments on the project. Onsite Stratum I or Stratum II soils should not be used as Select Fill Material.

- The bridge bents for the Exit 64 Bridge may be supported by driven steel H-piles founded on the competent limestone formation or at pile refusal elevation using a nominal bearing capacity of 700 kips per square foot (ksf).

- Alternatively, intermediate bent foundations for the Exit 64 Bridge may be supported by drilled caisson systems founded into the competent cherty-limestone formation with a nominal bearing capacity of 700 ksf.

- Roadway embankments at the W. Wedington Bridge may be designed for 2H:1V end slopes and 3H:1V side slopes. Existing embankment slopes were observed to be in stable condition and did not exhibit signs of failure. At the time of this report, a slope stability analysis has not been conducted on the existing bridge embankment slopes as survey data has not been provided for utilization in the analysis. An analysis can be performed and provided in future report submittals or addendums, if requested.

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FINAL GEOTECHNICAL INVESTIGATION

HWY. 16/112 SPUR INTCHNG. IMPVTS (S)
WASHINGTON COUNTY
ROUTE I-49 SECTION 29
Job No. BB0411
FAP NO. IM-540-1(80)64

for

BURNS & MCDONNELL
SPRINGDALE, ARKANSAS

INTRODUCTION

An investigation of subsurface soil conditions was conducted by McClelland Consulting Engineers, Inc., for the planned I-49 Hwy. 16-112 Spur Interchange Improvements project in Washington County in Fayetteville, Arkansas. The investigation was requested and authorized by the Arkansas Highway and Transportation Department in conjunction with Mr. Benjamin Biller of Burns and McDonnell to investigate subsurface soil conditions at the project site and to prepare recommendations for site grading and embankments and bridge foundations for the planned widening and improvement areas.

The data was determined from the following three (3) phase program:

A. An investigation of the subsurface conditions and visual soil classification by use of sample borings.

B. An engineering analysis of the laboratory and field data for construction recommendations.

C. An engineering analysis of the laboratory and field data for foundation type and bearing capacity recommendations.

PROJECT DESCRIPTION

The referenced project is located along Interstate 49 at Exit 64 in Fayetteville, Arkansas. The planned project scope includes widening and extending the existing Highway 16 (W. Wedington Drive) Bridge, the widening and rerouting of the existing W. Wedington Drive, Futrall Drive, and North Shiloh Drive; and the new construction of interstate on-ramps and off-ramps. Mechanically Stabilized Earth (MSE) retention walls are understood to be planned near each bridge end bent (Bent 1 and Bent 4).

FIELD INVESTIGATION

Bridge Foundation Borings

Subsurface conditions at the W. Wedington Drive Bridge location were investigated by six (6) borings conducted to depths that varied from 16.5 to 47.0 feet below existing ground and/or pavement elevations. The Exit 64 Bridge boring locations are shown on Plate 1B and are referenced in this report as B-B1 through B-B6. Boring logs for the bridge borings can be found in Appendix B on Plates 2 through 7. Bridge loading recommendations and factors that are referenced in this report were obtained by the American Association of State Highway and Transportation Officials (AASHTO) *Load and Resistance Factor Design* (LRFD) *Seventh Edition, 2014* with 2015 and 2016 Interim Revisions. Table 1 on the following page shows the depths and elevations that refusal material and competent bearing material were encountered at the bridge foundation boring locations for the W. Wedington Drive Bridge expansion. Additional information regarding boring locations can be referenced in the Boring Location Table in Appendix F.

TABLE 1: **Exit 64 Bridge (W. Wedington Drive)**

Bridge Structure	Boring Description	Boring Station	Offset	Ground Elevation	End of Boring Elevation	Refusal Material Elevation	Competent Limestone Elevation
Bent 1L	B-B1	132+92.64	49.02' L	1252.4	1235.9	1244.9	1244.9
Bent 1R	B-B2	132+46.51	70.09' R	1251.9	1225.9	1245.9	1245.9
Bent 2L	B-B3	133+58.61	52.70' L	1255.0	1236.0	1246.0	1246.0
Bent 2R	B-B4	133+36.91	54.03' R	1255.0	1230.0	1245.0	1245.0
Bent 3L	B-B5	135+31.69	19.67' L	1270.5	1223.5	1243.5	1243.5
Bent 3R	B-B6	135+24.40	21.95' R	1270.9	1232.9	1242.9	1242.9

Note: Elevations shown in Table 1 are rounded to the nearest 0.1 feet.

Roadway Borings

The soil conditions along the existing lengths of Futrall Drive and North Shiloh Drive were investigated by four (4) borings. The roadway borings were conducted outside of existing pavement areas in planned widening/rerouting locations. The roadway boring locations are shown in Appendix A on Plates 1C through 1E and are referenced in this report as B-P1 through B-P4. Roadway borings were drilled to planned-terminal depths of ten (10) feet below existing ground elevations. Borings B-P2, B-P3, and B-P4 were unable to achieve their planned-terminal depth due to the presence of auger refusal material between six (6) and eight (8) feet below existing ground elevations. The original spacing for the planned roadway borings for pavement design requirements was every 250 feet along the relocated ramps and the widening of Highway 16/112 Spur. At the direction of Mr. Ryan Castor, P.E. with Burns and McDonnell, roadway borings were not required along planned new interstate on-ramp and off-ramp lengths. Additionally, existing utilities prevented roadway borings from being conducted in the planned widening area along the south side of W. Wedington Drive, west of I-49.

As such, roadway borings were drilled in locations to provide pavement design recommendations for Futrall Drive and North Shiloh Drive. The boring logs of the roadway borings and widening borings can be referenced in Appendix B on Plates 8 through 11. Table 2 below shows the locations, depths, and elevations that were achieved by roadway borings. Additional information regarding boring locations can be referenced in the Boring Location Table in Appendix E.

<u>Table 2:</u> **Roadway Borings**

Boring Description	Alignment	Boring Station	Offset	Ground Elevation	End of Boring Elevation	Refusal Material Elevation
B-P1	Shiloh Dr.	106+34.48	19.16' R	1260.3	1250.3	N/A
B-P2	Shiloh Dr.	162+74.96	23.01' L	1253.4	1245.4	1245.4
B-P3	Futrall Dr.	166+65.22	03.72' L	1247.4	1239.4	1239.4
B-P4	Futrall Dr.	169+99.75	18.39' L	1244.8	1238.8	1238.8

Note: Elevations shown in Table 2 are rounded to the nearest 0.1 feet.
N/A: Refusal material was not encountered by the terminal depth of B-P1.

Borings were originally planned along the south side of W. Wedington Drive, west of the existing bridge, from N. Tahoe Place to N. Shiloh Drive. The borings along W. Wedington Drive were unable to be conducted due to a high density of utilities in the area, namely fiber-optic, gas, and water. Borings in this area would require the utility lines being exposed, prior to drilling. As such, bulk sampling operations were conducted as a subsequent investigation for the purpose of obtaining California Bearing Ration (CBR) values relevant to the new roadway subgrade. Samples were conducted in the northwest, southwest, and eastern portions of the project area at locations referenced as B-P1, B-P2, and B-P3, respectively. Table 3 on the following page shows CBR values at relevant roadway boring locations.

Table 3: CBR Values at Relevant Roadway Boring Locations

Boring Description	Alignment	Boring Station	Offset	Sample Depth	CBR Value
B-P1	Shiloh Dr.	106+34.48	19.16' R	1' to 2'	2.4
B-P2	Shiloh Dr.	162+74.96	23.01' L	1' to 2'	1.5
B-P3	Futrall Dr.	166+65.22	03.72' L	1' to 2'	2.0

Further description and analysis regarding the bulk samples, CBR values, and corresponding Resilient Modulus (M_R) values are provided later in this document. Data for B-P3 should also be utilized for design in the location of B-P4. Laboratory testing results relevant to CBR values can be referenced in Appendix G.

Drilling and Sampling Methods and Procedures

The borings were drilled to terminal depths and/or “refusal depth” using a truck-mounted rotary drilling rig with a six and one-half (6 ½) inch hollow-stem auger. Soil samples were obtained at the depths indicated on the borings by the use of a two (2) inch split-spoon sampler for obtaining samples from non-cohesive or slightly cohesive soils. The split-spoon sampler was driven by blows from a 140-pound hammer dropped 30 inches. The number of blows required to drive the split-spoon sampler the final twelve (12) inches of an eighteen (18) inch drive, or portion thereof, is referred to as the Standard Penetration value, N, and is recorded on the boring logs in the blows-per-foot column.

The W. Wedington Drive Bridge borings were then advanced into the basal rock formation using a three (3) inch NX diamond core bit and core sampler. Continuous core samples of the basal limestone rock formation were obtained using the three (3) inch diamond-tipped NX double-tube core barrel sampler. The core interval, the percent recovery and percent Rock Quality Determination (RQD) are given on the boring logs in Appendix B on Plates 2 through 7.

The field tests performed included visual soil classifications and groundwater observations. The visual soil classifications are given on the boring logs in Appendix B on Plates 2 through 7. Groundwater in the form of a perched water table was encountered by Boring B-B3 approximately five (5) feet below existing ground elevation. Groundwater was not encountered by the terminal depths of any other boring locations at the time of the subsurface investigation. The location where groundwater was encountered can be referenced in Table 4 below.

TABLE 4: **Encountered Groundwater Depths and Elevations**

Boring	Depth (ft)	Elevation
B-B3	**5**	**1250.0**

Note: Elevations shown in Table 4 are rounded to the nearest 0.1 feet.

LABORATORY TESTING

Laboratory tests were performed on soil samples recovered from the borings. The laboratory tests were directed at determining the engineering properties of the project soil strata. The laboratory tests were conducted in accordance with the American Association of State Highway and Transportation Officials (AASHTO) designations. The tests performed on samples from the borings included moisture content, dry unit weight, Atterberg Limits, gradation, unconfined compressive strength, standard proctor and CBR testing.

The natural soil moisture content was determined for the selected soil samples to provide a moisture profile for each boring. Unit weight determinations were performed on suitable soil samples and the dry unit weight is given on the boring logs. Atterberg Limits tests (liquid and plastic limits) were performed on selected samples to aid in the soil classification and to help evaluate the volume-change characteristics of each soil stratum. Gradation analyses were performed on representative soil samples to aid in the soil classification of the selected soil strata. Unconfined compression tests were performed on selected samples for evaluation of the shear strength of the soil strata. The cohesive shear strength reported on the boring logs is the maximum observed compressive stress. The crushing strength of the encountered rock formation is reported as the maximum compressive stress in tons per square foot (tsf).

Bulk samples from the test pits were utilized for further laboratory testing. Standard Proctor (AASHTO T-99) testing was performed on the bulk samples to determine the relationship between moisture content and compacted dry unit weight of the material. CBR (AASHTO T-193) testing was then performed on the bulk sample material using the proctor values to evaluate the potential strength of the relevant subgrade material. The results of the Standard Proctor testing and the CBR value at ninety-five (95) percent Standard Proctor density for the subgrade materials are presented in Appendix G at the end of this report.

Results of laboratory testing are provided on the boring logs, on the Laboratory Test Results Summary in Appendix C on Plates 13 through 15, and in Appendix G on Figures 1, 2, and 3.

SITE GEOLOGY

The project site is underlain by the Mississippian Age Pitkin Formation. The Pitkin Limestone material is typically a fine- to coarse-grained well-cemented and often fossiliferous limestone. Sequences of dark gray shale and limey sandstone interbedding often occur with the limestone. Minor layers of chert are sometimes present near either the top or bottom of the formation, particularly when in the vicinity of the Boone Formation, which is also common in the project area, particularly to the north and west. The thickness of the Pitkin Limestone ranges from a thin edge to over 400 feet. The average thickness is about 50 feet in the project area.

GENERAL SOIL CONDITIONS

The subsurface soil conditions at the site are described below.

Stratum I
The borings conducted in non-paved areas encountered a surface stratum of dark brown silty topsoil. The topsoil thickness was determined to be six (6) inches at non-paved sampled locations, but is anticipated extending to a depth of twelve (12) inches across the planned project area.

Stratum II
Soils underlying the surface stratum and pavement sections consisted of very soft to very stiff fine-grained soils that classified as sandy lean clay (CL), fat clay (CH), and sandy silt (ML). Varying amounts and gradations of sand and gravel were encountered within the Stratum II soils. Standard Penetration Resistance values (N-values) varied within this stratum varied from 4 blows per foot to 50 blows with 5 inches of advancement of the split-spoon sampler. The wide range of shear strengths in this soil grouping was often related to the degree of saturation and the amount of sand and gravel at sampled intervals.

Stratum III
Borings B-B4, B-B5, B-P1, B-P3, and B-P4, encountered strata of coarse-grained soils underlying the Stratum II material. The Stratum III soils were directly above auger refusal material in the bridge borings, where encountered. Soils within this stratum classified as silty sand (SM) and clayey gravel (GC). Shear strengths varied from medium-dense to dense with N-values in this stratum ranging from 12 blows per foot to 50 blows with 5 inches of advancement of the split-spoon sampler. The wide range of shear strengths in this soil grouping was often related the amount of gravel in the soil samples.

Stratum IV
Borings B-B5 and B-B6 encountered coarse-grained embankment fill material beneath the existing pavement section of W. Wedington Drive. The embankment fill extended to approximate depths of 20 feet below existing pavement elevations. The fill soils were encountered as reddish-brown in color and classified as clayey chert gravel (GC) material. The fill material was determined to be stable with shear strengths varying from medium-dense to dense and N-values in this stratum ranging from 10 to 26 blows per foot. Borings B-B5 and B-B6 were conducted on the east side of I-49 within the eastern W. Wedington Drive embankment. Similar conditions are anticipated within the western embankment.

Stratum V
The basal stratum at the project location consisted of hard, gray to dark gray limestone. The limestone is consistent with the Pitkin Formation, which is known to exist in the project area. The rock samples varied in grain-size from fine to coarse and exhibited limey sandstone and shale interbedding at deeper elevations. Dark gray fossiliferous sections were encountered by B-B1, B-B2, B-B3, B-B4, and B-B5. The dark gray material exhibited a strong gasoline odor, where encountered.

Existing Pavement

Existing asphalt pavement was encountered by Borings B-B3 through B-B6. Borings B-B3 and B-B4 were conducted in the easternmost south-bound lane and westernmost northbound lane of Interstate 49, respectively. Borings B-B5 and B-B6 were conducted within the existing dimensions of W. Wedington Drive, east of the bridge. Table 5 below references the borings where existing pavement was encountered, the asphalt pavement thickness, and any encountered granular base course material.

Existing pavement sections were encountered by borings B-B3, B-B4, B-B5, and B-B6.

TABLE 5: Existing Pavement Thickness

Boring Reference	Alignment	Station	Offset	Asphalt Thickness (in.)	Base Course Thickness (in.)
B-B3	Hwy. 16/112 S	133+58.61	52.70' Lt.	18.0	12.0
B-B4	Hwy. 16/112 S	133+36.91	54.03’ Rt.	16.0	12.0
B-B5	Hwy. 16/112 S	135+31.69	19.67’ Lt.	14.0	N/A
B-B6	Hwy. 16/112 S	135+24.40	21.95’ Rt.	15.0	N/A

Note: Elevations shown in Table 5 are rounded to the nearest 0.1 feet.
Asphalt and base course thicknesses are rounded to the nearest 0.5 inch.

Potential Recognized Environmental Condition (REC)

The preliminary report referenced an REC in the areas of Borings B-B1, B-B2, B-B3, B-B4, and B-B5 due to a strong gasoline odor being detected during drilling operations within the dark gray fossiliferous limestone material. The potential REC was raised by the preliminary report with the intent of drawing a conclusion by the time the final report was submitted. At the time of this report, it is our opinion and recommendation that the odor was actually due to a very high content of sulfur within the limestone material and not due to the presence of gasoline or other petroleum-based substances.

Fine-grained Soil Analysis

The fine fraction of the onsite lean clay (CL) material has a low plasticity and a low-to-moderate potential for volumetric changes due to variation in the soil moisture content. The Liquid Limit (LL) of the onsite CL soils varied between 29 and 48 and the Plasticity Index (PI) of those soils ranged from 8 to 24. The fine fraction of the CL material makes up between 58 and 87 percent of the CL soil mass as indicated by the results of gradation analysis from the borings.

The fine fraction of the onsite fat clay (CH) material has a high plasticity and a moderate-to-high potential for volumetric changes due to variation in the soil moisture content. The LL of the onsite CH soils varied between 51 and 78 and the PI of those soils ranged from 27 to 42. The fine fraction of the CH material makes up between 77 and 87 percent of the CH soil mass as indicated by the results of gradation analysis from the borings.

The fine fraction of the onsite clayey gravel (GC) material has a moderate plasticity and a moderate potential for volumetric changes due to variation in the soil moisture content. The LL of the onsite GC soils varied between 41 and 59 and the PI of those soils ranged from 20 to 30. The fine fraction of the GC material makes up between 28 and 40 percent of the GC soil mass as indicated by the results of gradation analysis from the borings.

The fine fraction of the onsite sandy silt (ML) material was determined to be non-plastic and has a negligible potential for volumetric changes due to variation in the soil moisture content. The fine fraction of the ML material makes up approximately 60 percent of the ML soil mass as indicated by the results of gradation analysis from the borings.

The fine fraction of the onsite silty sand (SM) material was determined to be non-plastic and has a negligible potential for volumetric changes due to variation in the soil moisture content. The fine fraction of the SM material makes up between 37 and 48 percent of the SM soil mass as indicated by the results of gradation analysis from the borings.

The fine fraction of the onsite silty gravel (GM) material was determined to be non-plastic and has a negligible potential for volumetric changes due to variation in the soil moisture content. The fine fraction of the GM material makes up approximately 37 percent of the GM soil mass as indicated by the results of gradation analysis from the borings.

IBC Site Classification

The soil profile at this project site is a Site Class B according to Section 1613.5.2 of the *2012 International Building Code* and Section 3.10.3.1 of the *AASHTO LRFD Bridge Design Specifications Seventh Edition*. The values for the spectral response accelerations S_{DS} and S_{D1} are 0.112g and 0.062g, respectively, with reference to Section 1613.5.3 of the *2012 International Building Code* and current USGS information for the project area.

ANALYSIS AND RECOMMENDATIONS

The following bridge foundation recommendations were referenced with the *AASHTO LRFD Bridge Design Specifications Seventh Edition, 2014* with 2015 and 2016 Interim Revisions and with current AHTD Bridge Division criteria

Highway Bridge Foundation Recommendations – Deep Foundations

The end and intermediate bents for Exit 64 (W. Wedington Drive) Bridge may be supported by steel H-piles driven to practical refusal upon the limestone bedrock. The H-piles should be installed to the competent limestone elevations referenced in Table 1 on Page 3 at the corresponding bridge bents in accordance with the requirements of Section 805 in the *AHTD Standard Specifications for Highway Construction, 2014 Edition*. Piles should be driven with a suitable hammer, exhibiting a minimum of 15,000 foot-pounds of energy, to a depth where no more than one-quarter (¼) inch of penetration is observed for the last five (5) hammer blows.

A nominal bearing capacity of 700 kips per square foot (ksf) may be used for the driven piles on the competent limestone formation. Limestone elevations can be anticipated where referenced on the Boring Logs and in Table 1. The recommended nominal bearing capacity was determined by a combination of average laboratory unconfined compressive strengths of core samples and RQD with Terzaghi and Peck. Resistance factors for the driven piles include 0.5 for Tip Resistance (end-bearing in rock) and 0.25 for Uplift Resistance for single piles using the α-method, per Table 10.5.5.2.3-1 in the referenced AASHTO LRFD volume.

The provided performance factors are based on single pile values. The referenced pile referenced pile resistance factors and side resistance values can be referenced in Table 6 below.

TABLE 6: Bridge Bent Driven Pile Performance Factors

Bent (Boring)	Nominal Tip Resistance Factor (end-bearing in rock)	Nominal Side Resistance (ksf)	Side Resistance Factor (Compression)	Side Resistance Factor (Uplift)	Top Elevation	Bottom Elevation	Layer Thickness (ft.)
1 (B-B1)	0.5	2.0	0.5	0.25	1252.4	1244.9	7.5
1 (B-B2)	0.5	2.0	0.5	0.25	1251.9	1245.9	6
2 (B-B3)	0.5	2.0	0.5	0.25	1255.0	1246.0	9
2 (B-B4)	0.5	2.0	0.5	0.25	1255.0	1245.0	10
3 (B-B5)	0.5	4.0	0.5	0.25	1270.5	1243.5	27
3 (B-B6)	0.5	4.0	0.5	0.25	1270.9	1250.9	20
3 (B-B6)	0.5	2.0	0.5	0.25	1250.9	1242.9	8

Note: Elevations shown in Table 6 are rounded to the nearest 0.1 feet. Table 10.5.5.2.3-1 in the referenced AASHTO LRFD volume was used to reference the provided resistance factor values.

The piles may be designed for maximum load with the use of pile points to increase bearing area of the pile in contact with the limestone formation. Because of the gravelly nature of the overburden material found on the project site and the possibility of the slightly-sloping rock formation, it is recommended that the steel piles be fitted or fabricated with reinforced driving points, prior to installation. Due to the proximity of MSE wall excavations near the end bents, it is our recommendation that the piles are pre-bored. The piling should be pre-bored a minimum of five (5) feet into the basal competent limestone material or to a depth of five (5) feet below the bottom of the leveling pad, whichever is greater. Uplift loads will be resisted by the weight of the pile and frictional shaft resistance (skin friction).

The design allowable steel pile compressive stress used for construction should not exceed one-fourth (¼) of the yield strength of the steel. Exceeding the recommended maximum yield strength would allow for unpredictable factors, such as damage during driving, excessive corrosion, ineffective tip contact, or slight eccentricity. Post-construction settlement of bearing piles on rock should be negligible.

Down-drag at the end bent locations due to long-term embankment settlement will also be minor. Down-drag occurs when skin friction forces are in the same direction as axial loading; however, skin friction forces are not anticipated to develop due to the piles being end-bearing on competent rock. Adequate consolidation and testing of fill material in new embankment areas will alleviate potential for down-drag in those areas. The driven piles will not encounter forces from skin friction without approximately one-eighth (⅛) inch of settlement, which should not be anticipated. Therefore, skin friction should not be factored into the driven pile foundation bearing capacities, but may be used in calculating uplift resistance.

The installation of test piles and load tests according to ASTM D 1143, ASTM D 3966, and Section 805 of the *AHTD Standard Specifications for Highway Construction, 2014 Edition*, is recommended to confirm the computed compressive nominal pile capacity, confirm the planned tip elevations, and determine the lateral pile capacity. The installation of the test pile and load test is recommended to be observed and monitored by the Owner, Engineer, and/or Geotechnical Engineer, or their representative. A record of the driving resistance should be made for each test and foundation pile.

Alternate Intermediate Bent Recommendations - Deep Foundations

Alternatively, the interior bent foundations for the bridges may be supported by straight-shaft drilled caissons founded into the competent cherty limestone formation. A nominal bearing capacity of 700 ksf may be used for the drilled caissons in the limestone formation. The recommended nominal bearing capacity was determined by a combination of average laboratory unconfined compressive strengths of core samples and RQD with Terzaghi and Peck. The recommended minimum depth of embedment (rock socket) into the limestone is three (3) feet or one (1) caisson diameter, whichever is greater. A minimum length to diameter ratio of 2L:1D should be utilized for the caissons.

The axially-loaded drilled caisson resistance factors include 0.5 for single-drilled shaft Tip Resistance in rock, per Table 10.5.5.2.4-1 in the referenced AASHTO LRFD volume. The application of side resistance lengths in rock will be determined by the actual rock socket lengths, which will be field-verified. The provided resistance factors are based on single caisson values. The referenced axial and side resistance values can be referenced in Table 7 on the following page.

TABLE 7: **Intermediate Bent Drilled Caisson Performance Factors**

Bent (Boring)	Nominal Tip Resistance Factor (end-bearing in rock)	Nominal Side Resistance (ksf)	Side Resistance Factor (Compression)	Side Resistance Factor (Uplift)	Top Elevation	Bottom Elevation	Layer Thickness (ft.)
1 (B-B1)	0.5	2.0	0.45	0.35	1252.4	1244.9	7.5
1 (B-B2)	0.5	2.0	0.45	0.35	1251.9	1245.9	6
2 (B-B3)	0.5	2.0	0.45	0.35	1255.0	1246.0	9
2 (B-B4)	0.5	2.0	0.45	0.35	1255.0	1245.0	10
3 (B-B5)	0.5	4.0	0.45	0.35	1270.5	1243.5	27
3 (B-B6)	0.5	4.0	0.45	0.35	1270.9	1250.9	20
3 (B-B6)	0.5	2.0	0.45	0.35	1250.9	1242.9	8

Note: Elevations shown in Table 7 are rounded to the nearest 0.1 feet. Table 10.5.5.2.4-1 in the referenced AASHTO LRFD volume was used to reference the provided resistance factor values.

Foundation settlement under the bridge structures for the drilled caisson system should be less than one-eighth (⅛) inch. Differential settlement between foundations should be negligible to one-eighth (⅛) inch.

Uplift loads for the drilled caissons will be resisted by the weight of the concrete and skin friction between the caisson and the limestone rock socket. The drilled caissons will not encounter forces from skin friction without approximately one-eighth (⅛) inch of settlement, which should not be anticipated. Therefore, skin friction should not be factored into the drilled caisson foundation bearing capacities, but may be used in calculating uplift resistance.

Temporary and/or permanent casings may be required for installation of the drilled caissons due to the fine-grained overburden material; however, it is not anticipated that casing will be required for the majority of the drilled caisson installation on the project. A minimum of one (1) probe hole should be conducted at each intermediate bent location to proof the rock competency and to determine the presence of any weathered or fractured zones. In bent locations with drilled caissons, the probe holes should be drilled beyond the bottom of the foundations to an additional depth of twice the individual drilled caisson diameter into competent rock. The ultimate foundation depth should be a minimum of two (2) feet below any encountered fractured or weathered zones. If weathered or fractured zones are encountered during proofing operations, the frequency of probe holes may be increased at the discretion of the Department.

All foundation systems should be thoroughly cleaned of loose material after excavation and before concrete placement. The foundation construction should be observed by the Owner, Engineer, and/or Geotechnical Engineer, or their representatives to verify the adequacy of bearing material. Concrete should be placed directly down the center of the drilled caissons, uninterrupted by reinforcing bars or tie-wires. Multiple methods of concrete placement can be performed to accomplish this; however, the preferred method is to use a tremie pipe to place the concrete to the bottom of the caisson excavation, particularly when groundwater issues may present themselves. Heavy-duty drilling equipment will be warranted for drilled caisson installation. Coring equipment will be required for drilled caisson advancement and rock sockets into the cherty limestone formation.

Alternate Intermediate Bent Recommendations - Shallow Foundations

Competent limestone was encountered at relatively shallow depths at the B-B3 and B-B4 locations. As an alternative to deep foundations, interior/intermediate bridge bent foundations may be placed on shallow spread footings founded a minimum of one (1) foot into the competent limestone formation. The spread footings should be sized so as not to exceed a nominal bearing capacity of 165 ksf. The recommended nominal bearing capacity was determined by a combination of average laboratory unconfined compressive strengths of core samples, specifically for Borings B-B3 and B-B4 in the dark gray limestone material that was encountered near the top of the encountered rock formation. If dark gray limestone is excavated in its entirety to expose gray limestone material, a nominal bearing capacity of 465 ksf may be used for the interior/intermediate bent spread footings. The utilized nominal bearing capacity should reference a resistance factor of 0.45 for footings on rock as provided in Table 10.5.5.2.2-1 in the referenced AASHTO LRFD volume. Foundation settlement under bridge spread footing structures founded on rock should be less than one-eighth (⅛) inch. Differential settlement should be negligible to ⅛-inch between shallow foundations or along 40 feet of footing length.

Site Grading and Embankment Recommendations

Stratum I (topsoil) soils were encountered by the roadway borings and by bridge borings conducted in non-paved areas. The existing Stratum I material should be removed full-depth from beneath all pavement/site improvement area dimensions. The Stratum I soils are considered moisture-sensitive and may lose significant strength upon disturbance and/or saturation. The thickness of the existing Stratum I soils should be anticipated to be one (1) foot below existing ground elevations.

Generally, subgrade areas with fine-grained soils (CL and CH) near planned finish elevations will required undercut amounts in the order of one (1) foot in planned roadway areas, however; undercut depths will vary across the site to potential maximum undercut depths of three (3) feet. Thickened bridging lifts may be utilized to prevent extensive undercutting amounts beyond three (3) feet. Sub-pavement areas with coarse-grained soils (SM, GM, and GC) near planned finish elevations were generally stable across the site and will provide suitable subgrade material. Undercutting should be at the direction of the Engineer or Department and should be based on results of proof-rolling at planned subgrade elevations when exposed soils are at a moisture content that is near optimum.

All project excavation and embankment procedures should follow AHTD Standard Specifications for Highway Construction, 2014 Edition, Section 210. All embankment slopes, both of finished construction and at the completion of the various phases of construction, should be stabilized to prevent erosion by placement of topsoil and seeding in accordance with the project specifications. Alternatively, erosion control mats may be used to cover erodible materials in areas where construction is not complete but has been stopped for periods of time in excess of twenty-one (21) days.

Select Fill Material

Any select fill material required for the project is recommended to be an off-site borrow material of locally available reddish-brown silty or sandy clay with broken chert gravel meeting Unified Soils Classification as a GC or GM material and having a Plasticity Index of 30 or less, a Liquid Limit of 55 or less, a minimum of 30% retained on the ¾-inch sieve and a maximum of 35% passing the No. 200 sieve. Onsite materials meeting the requirements detailed in this report for “Select Fill” may be used. Any material to be used as select fill on the project should be reviewed and approved by the Owner, Engineer, and/or Geotechnical Engineer. Select fill material should be placed in maximum 8-inch compacted lifts at a minimum density of 95 percent of the maximum dry density as determined by the Modified Proctor Test, AASHTO T-180. All fill and backfill should be placed in horizontal lifts. When placing fill next to existing slopes, the slope face should be stripped of all vegetation and the face “benched” to allow placement of horizontal lifts and bonding to the slope face. This technique is imperative during earthwork operations for the new widened bridge embankments.

Adequate subgrade preparation of the roadway embankments is essential to satisfactory performance of the roadway pavement sections and earth fills. The subgrade should be stripped to sound materials before placing select fill material to reduce embankment settlement and to ensure overall stability. Determination of stable subgrade material and/or undercut amounts should be verified by proof-rolling. Generally, soils in the pavement and widening boring locations will require minimal stripping to reach stable subgrade support.

Slope Stability Analysis

Roadway embankments at the W. Wedington Bridge may be designed for 2H:1V end slopes and 3H:1V side slopes. Existing embankment slopes were observed to be in stable conditions and did not exhibit signs of failure. At the time of this report, a slope stability analysis has not been conducted on the existing bridge embankment slopes as survey data has not been provided for utilization in the analysis. An analysis can be performed and provided in future report submittals or addendums, if requested.

Site Retaining Structures – Lateral Earth Pressures

We anticipate that the project includes site retaining structures such as end-bent back walls and wing walls. The referenced retaining structures should be designed to resist the minimum equivalent fluid weights provided in Table 8 on the following page. The recommended minimum factor of safety against sliding and overturning is 1.5 and 2.0 respectively. The provided lateral earth pressures assume a drained condition for the backfill material.

The provided lateral earth pressures assume a drained condition for the backfill material. To achieve this, the retaining structures should be backfilled using a free-draining granular material and be provided with thru-wall drains or a gravity trench drain system graded to daylight for the release of any hydrostatic pressure which may develop. The values provided by Table 8 for No. 57 or No. 67 crushed stone gravel assume a 1H:1V maximum backfill slope from the heel of the retaining wall foundation. If a vertical "chimney drain" is provided by the No. 57 or No. 67 stone, then the values for select fill material or onsite soils should be used depending on relevancy towards the material behind the gravel.

<u>TABLE 8:</u> **Lateral Earth Pressure Design Values**

Soil/Backfill Type	Moist Unit Weight (pcf)	Friction Angle, φ (°)	Equivalent Fluid Pressure (pcf)		
			Active	Passive	At-Rest
Select Fill Material (GC or GM)	120	28	43	332	64
Onsite Soils (Stratum II)	115	10	81	163	95
No. 57 or No. 67 Stone	85	35	23	314	37

A coefficient of friction of 0.40 may be used, provided the retaining structure is supported on a minimum of 4 inches of placed and compacted Class 7 Base Course material. A friction value of 0.35 may be used, provided the retaining structures is supported directly on select fill material or native onsite soils.

MSE Retaining Walls

MSE retaining walls are understood to be planned near each bridge end bent (Bent 1 and Bent 4). It is also our understanding that an unreinforced concrete leveling pad will be placed beneath each wall foundation level for the length of the MSE walls. Prior to placement of the concrete leveling pad, the subgrade material beneath the MSE wall dimensions should be verified to meet or exceed a net allowable bearing capacity of 3,000 psf. The recommended bearing capacity applies to the material beneath the wall length and the material beneath the reinforcement zone. It is anticipated that limestone rock material will be encountered at or near the planned leveling pad elevation for the MSE walls, particularly at the lower bottom-of-wall elevations. It is also anticipated that the backfill material beneath the MSE wall foundations will consist of Class 7 base course material compacted to 95% of AASHTO T-99. As such, all MSE wall foundations should bear on a minimum of one (1) foot of properly compacted Class 7 base course material beneath the concrete leveling pad. Any encountered rock should be excavated as required to allow for this condition. Soft and saturated soils were encountered immediately above the basal limestone material. Where encountered, soft and/or unsuitable bearing soils should be undercut at the direction of the Engineer to expose suitable soils and then should be backfilled with Class 7 base course material. Undercut depths in the order of three (3) feet should be anticipated, particularly along the MSE wall lengths where the foundations step up in elevation. A minimum strap length in the reinforcement zone should extend behind the wall for a minimum distance of 0.8 times the wall height. The granular backfill material within the reinforcement zone should be wrapped with a Type 2 geotextile filter fabric. The MSE walls should be properly drained to alleviate the effects of hydrostatic pressure behind the wall lengths.

Pavement Design Data

As previously described, bulk samples were conducted at representative locations across the site as to provide comprehensive data for the roadway subgrade material, namely for results as they relate to CBR values for pavement design. Pavement design along the planned expansion of W. Wedington Drive, west of the existing bridge, may reference the conservative CBR value provided by B-P2, or may use an average CBR value between B-P1 and B-P2. Table 9 below provides the corresponding Resilient Modulus (M_R) that may be utilized for pavement design at respective locations across the project improvements area.

Table 9: **M_R Values at Relevant Roadway Boring Locations**

Boring Description	CBR Value	M_R (psi)
B-P1	2.4	4,474
B-P2	1.5	3,312
B-P3	2.0	3,982

Note: M_R values were obtained through the equation $M_R \text{ (psi)} = 2555 \text{ (CBR)}^{0.64}$

The provided M_R values were calculated through the above equation provided by the National Cooperative Highway Research Program (NCHRP) Project 1-37A, which was referenced through documentations published by the U.S. Department of Transportation Federal Highway Administration.

Overhead Sign Foundations

Loading requirements have not been provided at this time for planned overhead sign structures. Assuming that the foundations for these structures are placed in compacted embankment material meeting the requirements of the **Select Fill Material** section of this report, the structures may be supported by shallow spread foundations sized not to exceed a nominal bearing capacity of 5,000 pounds per square foot (psf). Overhead sign foundations should be placed a minimum of three (3) feet below ground elevations to protect against sliding and overturning. This elevation is well below the minimum depth to protect against frost heave in the project area. Uplift forces on the signs during wind loading events should also be considered when designing the foundation depth. Resistance factors of 0.50 for bearing capacity and 0.85 for sliding may be applied to arrive at allowable foundation design values. The performance factors were provided from 10.5.5.2.2-1 in the referenced AASHTO LRFD volume.

Rock Excavation

Rock removal techniques are not anticipated being required on the project within the Stratum I, Stratum II, Stratum III, or Stratum IV onsite soils. Excavations below elevations labeled "Auger Refusal" on the boring logs should anticipate encountering Stratum V limestone materials which will require rock removal techniques, such as the use of hammer-hoe attachments.

Quality Control Statement

Quality Control testing of the earthwork operation, concrete, paving and other phases is recommended to be utilized during construction to assure the Owner and Engineer that the construction complies with the specifications. All trenching and excavation should be conducted in accordance with Arkansas State Law and OSHA guidelines and requirements.

LIMITATIONS AND RESERVED RIGHTS

The recommendations and conclusions made in this report are based on the assumption that the subsoil conditions do not deviate appreciably from those disclosed in the subsurface exploration. Should significant subsoil variations or undesirable conditions be encountered during construction that are not described herein, the Geotechnical Engineer reserves the right to inspect these conditions for the purpose of reevaluating this report. A review of the final construction plans and specifications by this office is encouraged to ensure compliance with the intent of these recommendations.

Sincerely yours,
McCLELLAND CONSULTING ENGINEERS, INC.

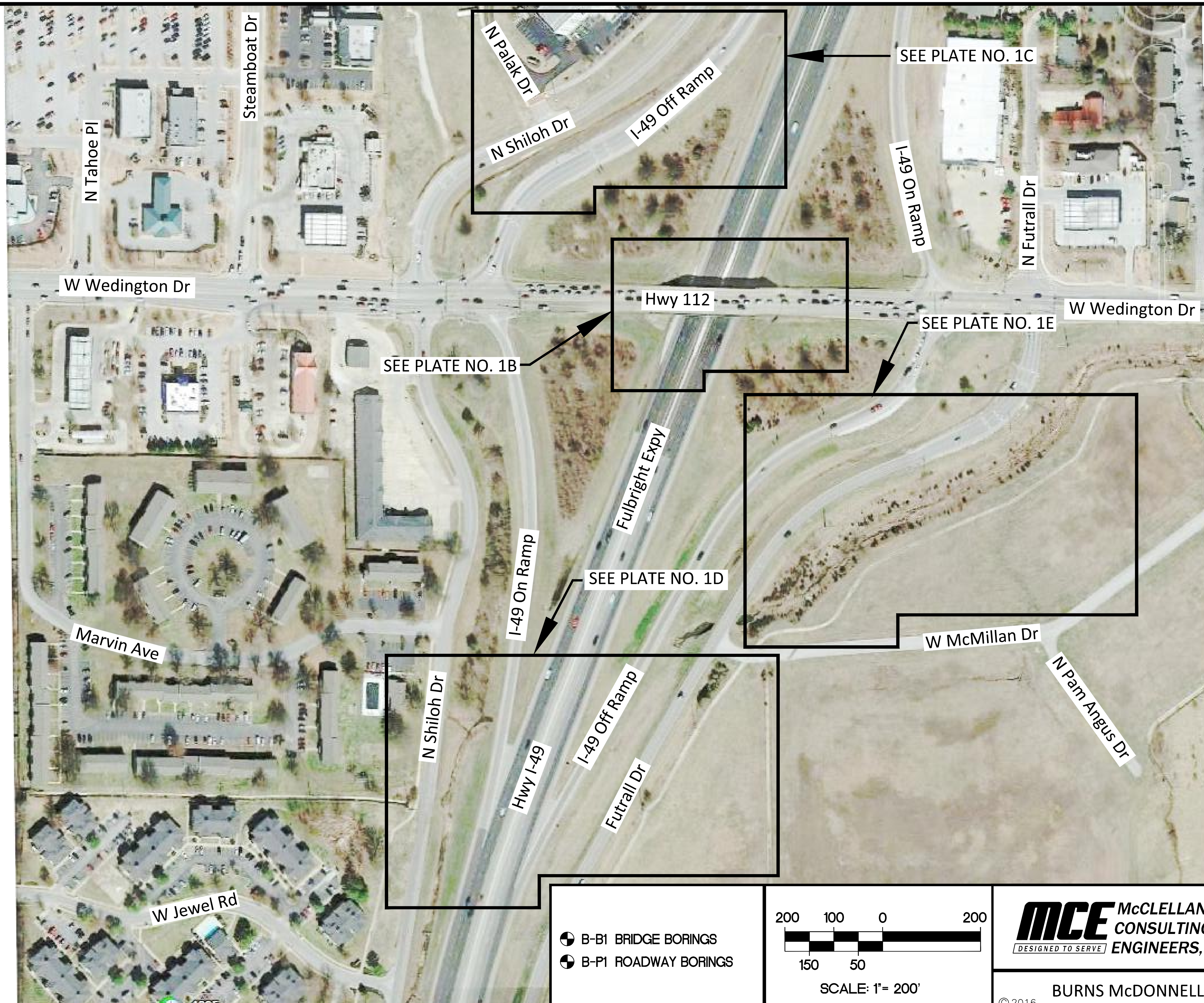
STATE OF ARKANSAS
LICENSED PROFESSIONAL ENGINEER
No. 16434
STEVEN J. HEAD
11/7/2017

Steven J. Head, P.E.
Geotechnical Engineer

Enclosures: Appendix A: Boring Layout
Appendix B: Boring Logs
Appendix C: Laboratory Testing Results
Appendix D: Bridge Profile
Appendix E: Roadway Boring Soil Log
Appendix F: Boring Location Table
Appendix G: CBR Testing Information

APPENDIX A

BORING LAYOUT



SHEET LOCATION INDEX

B-B1 BRIDGE BORINGS

B-P1 ROADWAY BORINGS

200 100 0 200

150 50

SCALE: 1"= 200'

MCE McCLELLAND CONSULTING ENGINEERS, INC.

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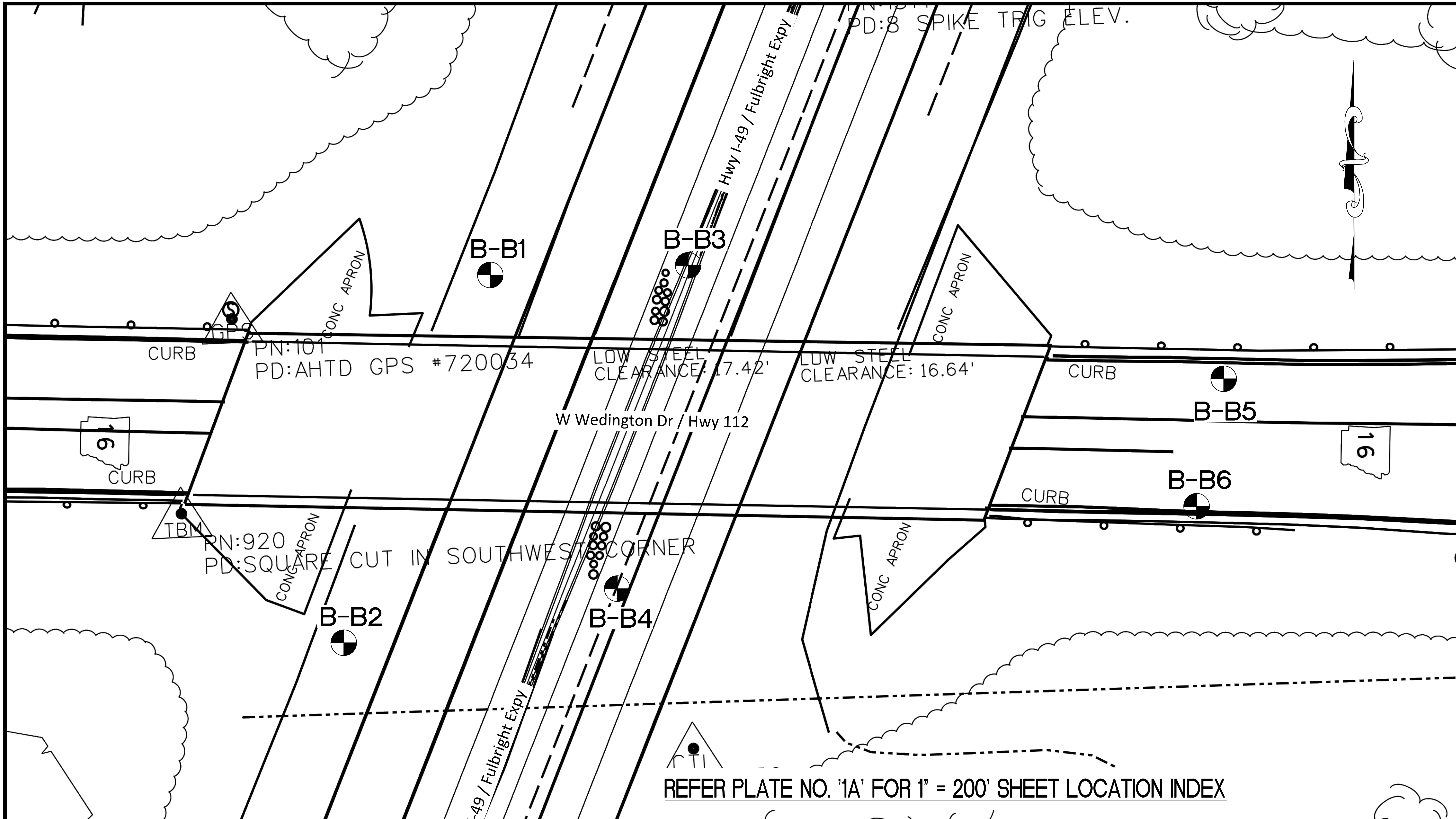
BORING PLAN
BB0411 HWY. 16-112 SPUR
INTCHNG. IMPVTS
FAYETTEVILLE, ARKANSAS

APPROVED	DRAWN BY	DATE	PLATE NO.
S.H.	L.E.W.	MAY, 2016	1A
SCALE	JOB. NO.	FIELD BOOK	
AS NOTED	FY163809	--	



REFER PLATE NO. '1A' FOR 1" = 200' SHEET LOCATION INDEX

B-B1 BRIDGE BORINGS

B-P1 ROADWAY BORINGS

30 15 0 30

SCALE: 1"= 30'

MCE McCLELLAND CONSULTING ENGINEERS, INC.

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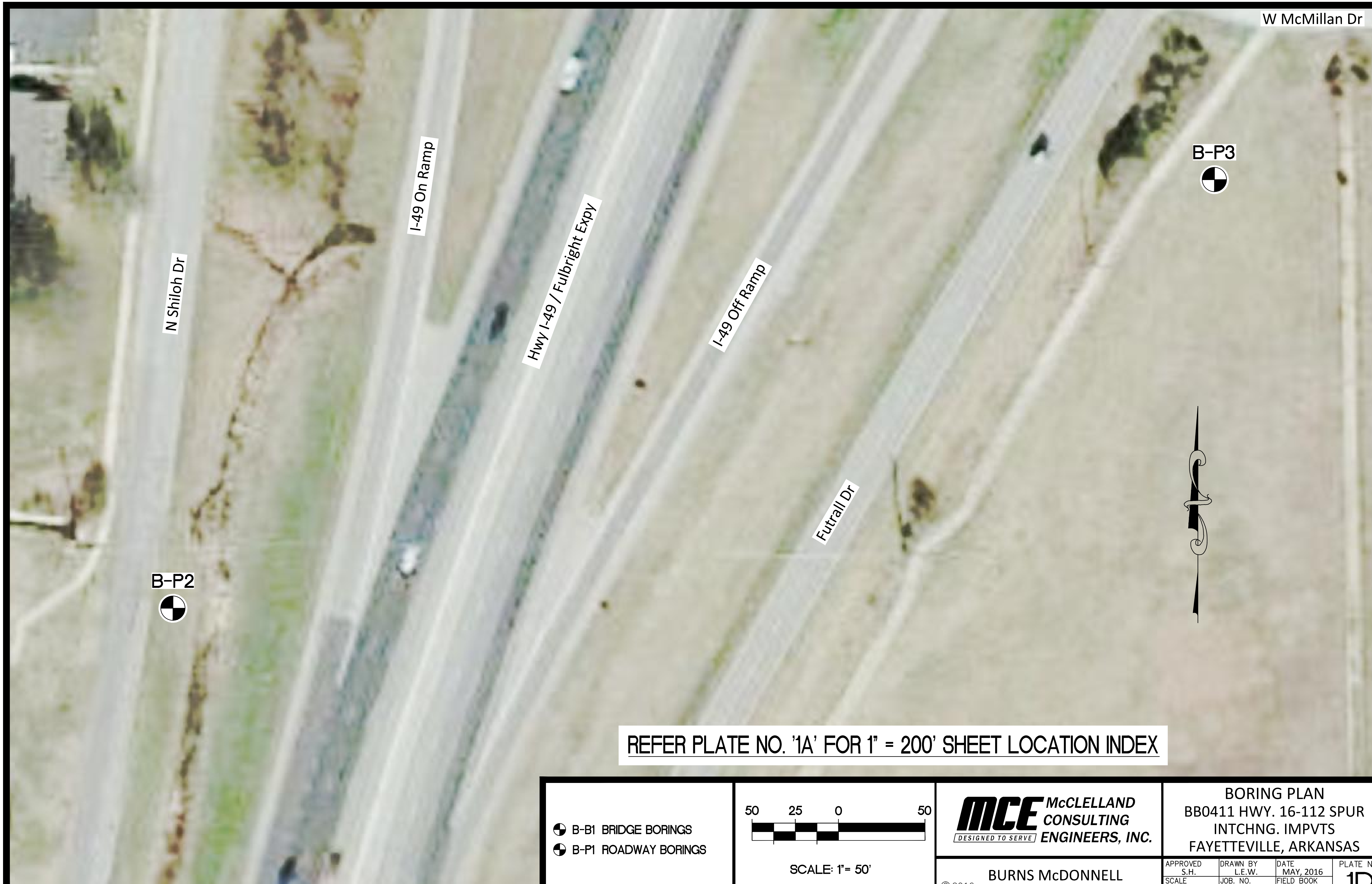
BORING PLAN
BB0411 HWY. 16-112 SPUR
INTCHNG. IMPVTS
FAYETTEVILLE, ARKANSAS

APPROVED	DRAWN BY	DATE	PLATE NO.
S.H.	L.E.W.	MAY, 2016	1B
SCALE	JOB. NO.	FIELD BOOK	
AS NOTED	FY163809	--	

REFER PLATE NO. '1A' FOR 1" = 200' SHEET LOCATION INDEX

B-B1 BRIDGE BORINGS B-P1 ROADWAY BORINGS	40 20 0 40 30 10 SCALE: 1"= 40'	MCE McCLELLAND CONSULTING ENGINEERS, INC. DESIGNED TO SERVE BURNS McDONNELL ©2016	BORING PLAN BB0411 HWY. 16-112 SPUR INTCHNG. IMPVTS FAYETTEVILLE, ARKANSAS

APPROVED	DRAWN BY	DATE	PLATE NO.
S.H.	L.E.W.	MAY, 2016	1C
SCALE: AS NOTED	JOB. NO.: FY163809	FIELD BOOK: --	



B-B1 BRIDGE BORINGS
B-P1 ROADWAY BORINGS

50 25 0 50

SCALE: 1"= 50'

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BORING PLAN
BB0411 HWY. 16-112 SPUR
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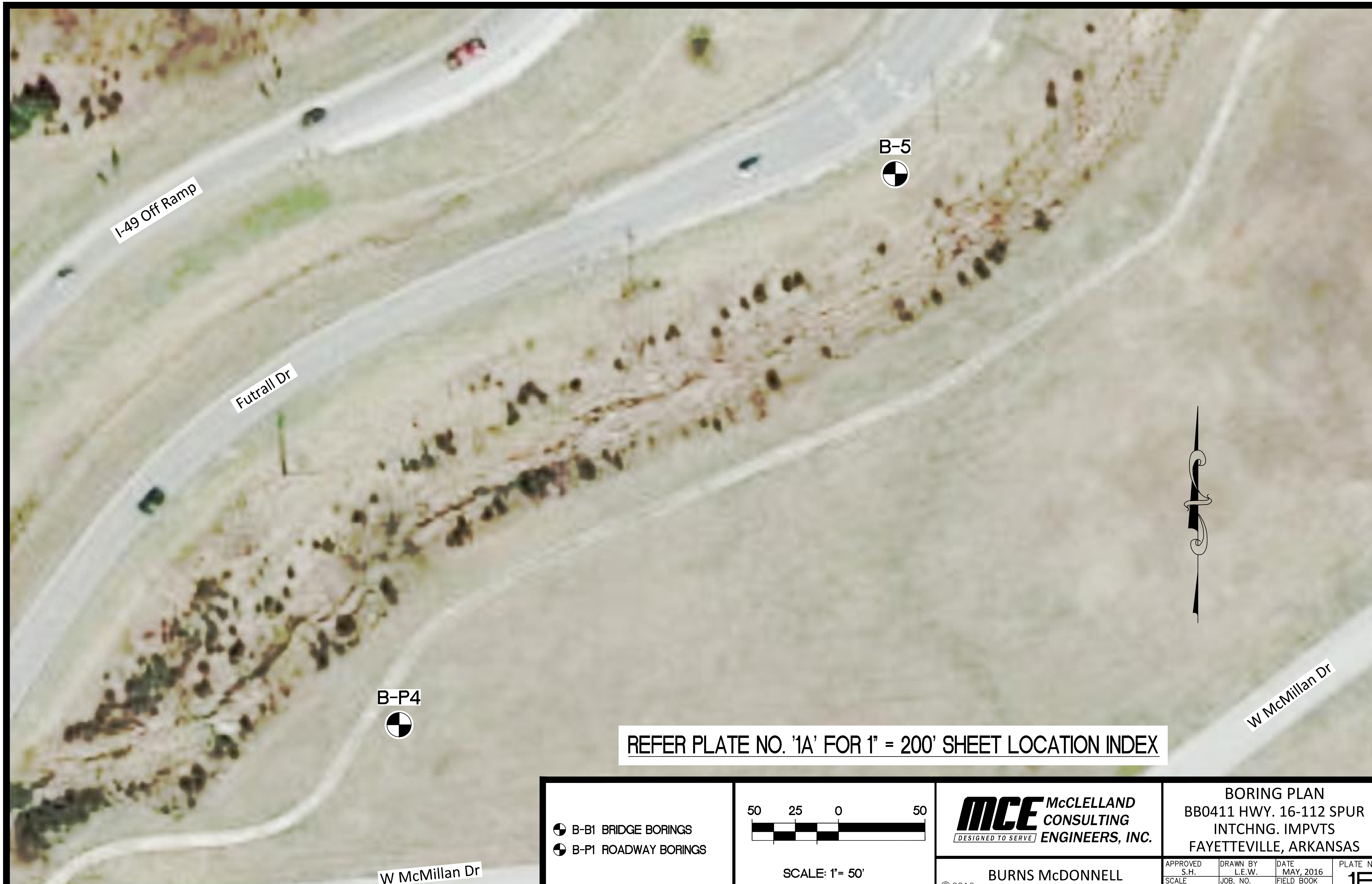
FAYETTEVILLE, ARKANSAS

APPROVED	DRAWN BY	DATE	PLATE NO.
S.H.	L.E.W.	MAY, 2016	1D
SCALE	JOB. NO.	FIELD BOOK	
AS NOTED	FY163809	--	



B-B1 BRIDGE BORINGS

B-P1 ROADWAY BORINGS

50 25 0 50

SCALE: 1"= 50'

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BORING PLAN

BB0411 HWY. 16-112 SPUR

INTCHNG. IMPVTS

FAYETTEVILLE, ARKANSAS

APPROVED	DRAWN BY	DATE	PLATE NO.
S.H.	L.E.W.	MAY, 2016	1E
SCALE	JOB. NO.	FIELD BOOK	
AS NOTED	FY163809	--	

APPENDIX B

BORING LOGS

WEDINGTON BRIDGE
BORING LOGS

LOG OF BORING NO. B - B1

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 3/7/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 132+92.64, 49.02' Lt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger & 3" NX Rock Bit
ELEVATION: 1252.39

USCS Type	Description of Material (Color, Type, Moisture, and Consistency)
	DARK BROWN SILTY TOPSOIL; Organics
GM	DARK BROWN TO REDDISH-BROWN SILTY GRAVEL; Loose; Ferrous Staining; Fine Gravel
CH	REDDISH-BROWN TO TAN FAT CLAY; Soft; Ferrous Staining; Gray Seams; Fine Gravel (FIRM FROM 4 TO 6 FEET)
CL	TAN TO GRAY SANDY CLAY; Firm; Highly Weathered Sandstone Fine Gravel **AUGER REFUSAL AT 7.5 FEET**
	GRAY LIMESTONE; Fine-Grained; Well-Cemented 80% Recovery 47% RQD
	DARK GRAY LIMESTONE; Fossiliferous; Strong Gasoline Odor from 9.5 to 11.5 feet
	GRAY LIMESTONE; Shale and Limey Sandstone Interbedding; Calcite Deposits 92% Recovery 82% RQD
	END OF BORING

Sample No.	Blows/ Foot	USCS Type	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
1			30.3							
2	8	GM	20.4			NP	36.9			
3	6	CH	34.1					0.54		88.9
4	10		32.5	52	25	27	86.8	0.36		89.3
5	16	CL	24.5							
6								498.4		164.8
7								132.6		157.9
8								371.5		159.2

Depth (Feet): 0, 5, 10, 15, 20, 25, 30
Elevation (Feet): 1250, 1245, 1240, 1235, 1230, 1225, 1220

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Completion Depth: 16.5 feet **Depth to Water:** Dry **Logged By:** J. Thomas

LOG OF BORING NO. B - B2

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 3/12/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 132+46.51, 70.09' Rt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger & 3" NX Rock Bit
ELEVATION: 1251.93

Depth (Feet)	Elevation (Feet)	Sample No.	Blows/ Foot	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
0		1			DARK BROWN SILTY TOPSOIL; Organics								
		2	4	CH	TAN TO GRAY FAT CLAY; Very Soft; Fine Gravel	25.1							
	1250	3	5			30.6	51	24	27	85.3			
5		4	27	CL	TAN TO GRAY LEAN CLAY WITH SAND; Stiff; Fine Gravel	47.6	29	21	8	79.9			
					AUGER REFUSAL AT 6 FEET								
	1245				TAN WEATHERED SANDSTONE 100% Recovery 82% RQD								
10		6			GRAY LIMESTONE; Shale and Limey Sandstone Interbedding; Calcite Deposits 100% Recovery 100% RQD						255.4		158.3
15	1240				DARK GRAY LIMESTONE; Fossiliferous; Strong Gasoline Odor from 14 to 21 feet 100% Recovery 87% RQD								
20	1235	7									218.8		164.6
25	1230	8			GRAY LIMESTONE; Shale and Limey Sandstone Interbedding; Calcite Deposits 95% Recovery 95% RQD						299.2		146.6
	1225				END OF BORING								
30	1220												

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Completion Depth: 26.0 feet **Depth to Water:** Dry **Logged By:** J. Thomas

LOG OF BORING NO. B - B3

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 3/13/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 133+58.61, 52.70' Lt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger & 3" NX Rock Bit
ELEVATION: 1255.04

Depth (Feet)	Elevation (Feet)	Sample No.	Blows/ Foot	Soil Legend	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
0	1255					18" OF ASPHALT								
						12" OF COARSE AGGREGATE								
						Loose, Poorly-Graded Gravel (Unable to Sample)								
5	1250				▽									
		1	8		CL	REDDISH-BROWN LEAN CLAY; Soft; Ferrous Staining; Fine Gravel	26.7	46	27	19	86.8			
		2	50/5			TAN TO GRAY LEAN CLAY WITH SAND; Very Stiff; Fine Gravel **AUGER REFUSAL AT 9 FEET**	27.5							
10	1245	3				GRAY LIMESTONE; Fine-Grained; Well-Cemented 100% Recovery 77% RQD						560.0		161.9
		4										146.0		155.6
						DARK GRAY LIMESTONE; Strong Gasoline Odor from 11 to 14 feet								
15	1240					GRAY LIMESTONE; Shale and Limey Sandstone Interbedding; Calcite Deposits 100% Recovery 100% RQD								
		5										491.4		157.0
						END OF BORING								
20	1235													
25	1230													
30	1225													

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Completion Depth: 19.0 feet **Depth to Water:** 5 feet **Logged By:** J. Thomas

LOG OF BORING NO. B - B4

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 3/24/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 133+36.91, 54.03' Rt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger & 3" NX Rock Bit
ELEVATION: 1255.02

Depth (Feet)	Elevation (Feet)	Sample No.	Blows/ Foot	Soil Legend	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
0	1255					16" of Asphalt								
						12" of Coarse Aggregate								
						Loose, Poorly-Graded Gravel (Unable to Sample)								
		1	17		CH	REDDISH-BROWN FAT CLAY WITH SAND; Firm; Ferrous Nodules and Staining	10.4							
5	1250													
		2	10				33.5	78	36	42	77.7			
		3	25		SM	TAN TO GRAY SILTY SAND WITH GRAVEL; Dense; Fine Gravel **AUGER REFUSAL AT 10 FEET**	14.4			NP	48.4			
10	1245					DARK GRAY LIMESTONE; Fossiliferous; Strong Gasoline Odor from 10 to 11.5 feet 98% Recovery 72% RQD								
		4				GRAY LIMESTONE; Shale and Limey Sandstone Interbedding; Calcite Deposits 100% Recovery 98% RQD						185.1		165.3
15	1240													
		5										439.0		158.1
20	1235					-Pyrite deposits from 20 to 25 feet 98% Recovery 78% RQD								
		6										335.2		171.4
25	1230					END OF BORING								
30	1225													

Completion Depth: 25.0 feet **Depth to Water:** Dry **Logged By:** J. Thomas

This information pertains only to this boring and should not be interpreted as being indicative of the site.

LOG OF BORING NO. B - B5

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 3/15/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 135+31.69, 19.67' Lt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger & 3" NX Rock Bit
ELEVATION: 1270.48

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Depth (Feet)	Elevation (Feet)	Sample No.	Blows/ Foot	Soil Legend	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
0	1270					14" OF ASPHALT (No Base Course Material)								
		1	11		GC	REDDISH-BROWN CLAYEY CHERT GRAVEL WITH SAND (Existing Embankment Fill Material); Medium-Dense	20.3							
5	1265	2	20				25.7							
		3	20				22.1	51	25	26	28.3			
		4	15			(MEDIUM-DENSE FROM 8 TO 10 FEET)	26.9							
10	1260	5	26			(DENSE FROM 10 TO 20 FEET)	31.2							
15	1255	6	19				27.7							
20	1250	7	40		SM	TAN TO GRAY SILTY SAND WITH GRAVEL; Dense	10.5			NP	37.1	1.122		121.8
25	1245	8	12			(MEDIUM-DENSE AND SATURATED AT 25 FEET) **AUGER REFUSAL AT 27 FEET**	36.7					1.229		79.7
		9				GRAY LIMESTONE; Fine-Grained; Well-Cemented 85% Recovery 63% RQD						351.8		165.0
30	1240													
						DARK GRAY LIMESTONE; Fossiliferous;								

Completion Depth: 47.0 feet **Depth to Water:** Dry **Logged By:** J. Thomas

LOG OF BORING NO. B - B5

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 3/15/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 135+31.69, 19.67' Lt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger & 3" NX Rock Bit
ELEVATION: 1270.48

Depth (Feet)	Elevation (Feet)	Sample No.	Blows/ Foot	Soil Legend	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
		10				Strong Gasoline Odor from 32 to 34.5 feet 92 Recovery 92 RQD						240.3		164.3
35	1235					GRAY LIMESTONE; Shale and Limey Sandstone Interbedding; Calcite Deposits								
						100% Recovery 100% RQD								
		11										510.0		157.7
40	1230													
						100% Recovery 80% RQD								
45	1225	12										320.8		165.4
						END OF BORING								
50	1220													
55	1215													
60	1210													
65	1205													

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

Completion Depth: 47.0 feet **Depth to Water:** Dry **Logged By:** J. Thomas

LOG OF BORING NO. B - B6

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 3/16/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 135+24.40, 21.95' Rt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger & 3" NX Rock Bit
ELEVATION: 1270.87

Depth (Feet): 0, 5, 10, 15, 20, 25, 30
Elevation (Feet): 1270, 1265, 1260, 1255, 1250, 1245, 1240

Sample No.	Blows/ Foot	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
			15" OF ASPHALT (No Base Course Material)								
1	10	GC	REDDISH-BROWN CLAYEY CHERT GRAVEL WITH SAND (Existing Embankment Fill Material); Medium-Dense	21.1	59	29	30	34.6			
2	14			16.2							
3	22		(DENSE FROM 6 TO 8 FEET)	26.3							
4	10		(MEDIUM-DENSE FROM 8 TO 10 FEET)	30.9							
5	20		(DENSE FROM 10 TO 15 FEET)	20.8							
6	12		(MEDIUM-DENSE FROM 15 TO 20 FEET)	15.5							
7	29	CL	TAN TO GRAY LEAN CLAY WITH SAND; Stiff	9.4							
8	8		(SOFT FROM 25 TO 28 FEET)	17.9	48	24	24	73.4			
			AUGER REFUSAL AT 28 FEET								
9			GRAY LIMESTONE; Shale and Limey Sandstone Interbedding; Calcite Deposits 92% Recovery 58% RQD						368.6		165.4

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Completion Depth: 38.0 feet **Depth to Water:** Dry **Logged By:** J. Thomas

LOG OF BORING NO. B - B6

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 3/16/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 135+24.40, 21.95' Rt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger & 3" NX Rock Bit
ELEVATION: 1270.87

Depth (Feet)	Elevation (Feet)	Sample No.	Blows/ Foot	Soil Legend	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
						100% Recovery 100% RQD								
35	1235													
		10										578.0		159.6
						END OF BORING								
40	1230													
45	1225													
50	1220													
55	1215													
60	1210													
65														

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

Completion Depth: 38.0 feet **Depth to Water:** Dry **Logged By:** J. Thomas

ROADWAY
BORING LOGS

LOG OF BORING NO. B - P1

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 5/10/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 106+34.48, 19.16' Rt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger
ELEVATION: 1260.34

Depth (Feet)	Elevation (Feet)	Sample No.	Blows/ Foot	Soil Legend	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
0	1260	1				DARK BROWN SILTY TOPSOIL; Organics	14.5							
		2	12		CL	DARK BROWN TO REDDISH-BROWN SANDY LEAN CLAY WITH GRAVEL; Firm; Trace Coarse Sand; Trace Medium Sand; Trace Fine Sand; Little Fine Gravel (STIFF FROM 4 TO 6 FEET)	15.9	32	20	12	61.8			
		3	14				19.1							
		4	26				17.2							
5	1255	5	36		ML	TAN TO REDDISH-BROWN SANDY SILT WITH GRAVEL; Stiff; Weathered Sandstone; Trace Coarse Sand; Trace Medium Sand; Little Fine Sand; Little Fine Gravel (FIRM FROM 8 TO 10 FEET)	20.6			NP	60.5			
		6	12				29.2					0.70		92.4
10	1250	7	50/7		GC	TAN TO REDDISH-BROWN CLAYEY GRAVEL WITH SAND; Very Dense; Weathered Sandstone	25.3							
						END OF BORING								
15	1245													
20	1240													
25	1235													
30	1230													

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Completion Depth: 11.5 feet **Depth to Water:** DRY **Logged By:** J. Thomas

LOG OF BORING NO. B - P2

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 5/10/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 162+74.96, 23.01' Lt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger
ELEVATION: 1253.39

Depth (Feet)	Elevation (Feet)	Sample No.	Blows/ Foot	Soil Legend	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
0		1				DARK BROWN SILTY TOPSOIL; Organics	28.4							
		2	10		CL	DARK BROWN TO TAN SANDY LEAN CLAY; Firm; Trace Coarse Sand; Little Medium Sand; Little Fine Sand; Trace Fine Gravel	20.0							
		3	10				21.0	48	25	23	58.5			
	1250													
		4	8				21.1					0.70		101.4
5														
		5	10				26.5					0.50		98.6
						AUGER REFUSAL AT 8 FEET END OF BORING								
	1245													
10														
	1240													
15														
	1235													
20														
	1230													
25														
	1225													
30														

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Completion Depth: 8.0 feet **Depth to Water:** DRY **Logged By:** J. Thomas

LOG OF BORING NO. B - P3

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 5/10/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 166+65.22. 03.72' Lt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger
ELEVATION: 1247.40

Depth (Feet)	Elevation (Feet)	Sample No.	Blows/ Foot	Soil Legend	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
0		1				DARK BROWN SILTY TOPSOIL; Organics	39.7							
		2	12		CL	DARK BROWN TO TAN LEAN CLAY WITH SAND; Firm; Trace Coarse Sand; Trace Medium Sand; Trace Fine Sand; Trace Fine GRavel	15.0							
	1245	3	12				20.9	44	23	21	76.2			
5		4	12				24.3					0.41		104.1
	1240	5	50/5		GC	TAN TO REDDISH-BROWN CLAYEY GRAVEL WITH SAND; Very Dense; Weeathered Sandstone	23.6							
						AUGER REFUSAL AT 8 FEET								
10						END OF BORING								
	1235													
15														
	1230													
20														
	1225													
25														
	1220													
30														
	1215													

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Completion Depth: 8.0 feet **Depth to Water:** DRY **Logged By:** J. Thomas

LOG OF BORING NO. B - P4

DESCRIPTION: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
PROJECT NO.: FY163809
DATE DRILLED: 5/10/2016
LOCATION: Fayetteville, AR
PROJECT ENGINEER: Steven J. Head, PE
PROJECT OWNER: Arkansas Hwy. & Trans. Department
BORING LOCATION: STA: 169+99.75, 18.39' Lt.
DRILLING METHOD: 6 1/2" Hollow Stem Auger
ELEVATION: 1244.76

Depth (Feet)	Elevation (Feet)	Sample No.	Blows/ Foot	Soil Legend	USCS Type	Description of Material (Color, Type, Moisture, and Consistency)	Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	P200	Lab Q_u (TSF)	Field Q_u (TSF)	Dry Density (pcf)
0		1				DARK BROWN SILTY TOPSOIL; Organics	19.3							
		2	8		CL	DARK BROWN TO TAN LEAN CLAY; Soft	18.8							
		3	10			(FIRM FROM 2 TO 4 FEET)	19.8					0.48		103.5
5	1240	4	27		GC	TAN TO REDDISH-BROWN CLAYEY GRAVEL WITH SAND; Dense; Weathered Sandstone; Trace Coarse Sand; Trace Medium Sand; Trace Fine Sand; Some Fine Gravel	13.7	41	21	20	39.9			
						AUGER REFUSAL AT 6 FEET								
						END OF BORING								
10	1235													
15	1230													
20	1225													
25	1220													
30	1215													

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Completion Depth: 6.0 feet **Depth to Water:** DRY **Logged By:** J. Thomas

SYMBOLS AND TERMS USED ON BORING LOGS

Symbol Description

Strata symbols

- High plasticity clay
- Low plasticity clay
- Gravel
- Silt
- Elastic silt
- Poorly graded sand
- Fill
- Granite
- Limestone
- Organics
- Sandstone
- Shale
- Topsoil

Misc. Symbols

- Water table during drilling
- Water table at second check

Soil Samplers

- Bulk sample taken from 6 in. auger
- Standard penetration test
- Undisturbed thin wall Shelby tube
- Rock core
- Denison

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE-GRAINED SOILS (major portion retained on #200 sieve): Includes (1) clean gravels and sands, and (2) silty or clayey gravels and sands. Condition is rated according to relative density, as determined by laboratory tests.

DESCRIPTIVE TERM	RELATIVE DENSITY
Loose	0 to 40%
Medium Dense	40 to 70%
Dense	70 to 100%

FINE-GRAINED SOILS (major portion passing #200 sieve): Includes (1) inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as indicated.

DESCRIPTIVE TERM	UNCONFINED COMPRESSION STRENGTH (TSF)
Very Soft	Less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	4.00 and higher

Note: Slickensided and fissured clays may have lower unconfined compressive strengths than shown above because of planes of weakness or cracks in the soil. The consistency rating of such soils are based on penetration readings.

TERMS CHARACTERIZING SOIL STRUCTURE

Slickensided	having inclined planes of weakness that are slick and glossy in appearance
Fissured	containing shrinkage cracks, frequently filled with fine sand or silt, usually vertical
Laminated	composed of thin layers of varying color and texture
Interbedded	composed of alternate layers of different soil types
Calcareous	containing appreciable quantities of calcium carbonate
Well Graded	having wide range in grain sizes and substantial amounts of all intermediate particle sizes
Poorly Graded	predominantly of one grain size, or having a range in sizes with some intermediate sizes missing

Terms used in this report for describing soils according to their texture or grain size distribution are in accordance with the UNITED SOIL CLASSIFICATION SYSTEM as described in ASTM D 2488

MCE DESIGNED TO SERVE McCLELLAND CONSULTING ENGINEERS, INC.

APPENDIX C

LABORATORY TESTING RESULTS

LABORATORY TEST RESULTS

PROJECT NUMBER: FY163809

PROJECT: BB0411 Hwy. 16/112 Spur Intchng. Impvts.

DATE: Wednesday, May 25, 2016

B #	S #	Description	Depth Feet	Moisture (%)	LL	PL	PI	USCS	AASHTO	SIEVE ANALYSIS % FINER 3/4 IN	No. 4	NO. 10	NO. 40	NO. 200	UDW pcf	U_c tsf
B1																
	1	Dark Brown Silty topsoil	0'-6"	30.3												
	2	Dark Brown to Reddish-Brown	6"-2'	20.4			NP	GM	A-4(0)	81.1	50.4	45.5	41.8	36.9		
	3	Reddish-Brown to Tan Fat Clay	2'-3'6"	34.1											88.9	0.54
	4	Reddish-Brown to Tan Fat Clay	4'-5'6"	32.5	52	25	27	CH	A-7-6(26)	100.0	96.7	94.6	91.8	86.8	89.3	0.36
	5	Tan to Gray Sandy Clay	6'-7'	24.5												
	6	Gray Limestone	8'6"-9'												164.8	498.40
	7	Dark Gray Limestone	10'6"-11'												157.9	132.60
	8	Gray Limestone	14'6"-15'												159.2	371.50
B2																
	1	Dark Brown Silty Topsoil	0'-6"													
	2	Tan to Gray Fat Clay	6"-2'	25.1												
	3	Tan to Gray Fat Clay	2'-3'6"	30.6	51	24	27	CH	A-7-6(25)	100.0	97.1	95.8	92.3	85.3		
	4	Tan to Gray Lean Clay with Sand	4'-5'6"	47.6	29	21	8	CL	A-4(5)	100.0	95.9	90.4	86.1	79.9		
	6	Gray Limestone	8'6"-9'												158.3	255.40
	7	Dark Gray Limestone	19'6"-20'												164.6	218.80
	8	Gray Limestone	25'-25'6"												146.6	299.20
B3																
	1	Reddish-Brown Lean Clay	6'-7'6"	26.7	46	27	19	CL	A-7-6(18)	100.0	98.5	96.6	93.8	86.8		
	2	Tan to Gray Lean Clay with Sand	8'-8'6"	27.5												
	3	Gray Limestone	10'-10'6"												161.9	560.00
	4	Dark Gray Limestone	12'-12'6"												155.6	146.00
	5	Gray Limestone	17'-17'6"												157.0	491.40
B4																
	1	Reddish-Brown Fat Clay with Sand	4'-5'6"	10.4												
	2	Reddish-Brown Fat Clay with Sand	6'-7'6"	33.5	78	36	42	CH	A-7-5(37)	100.0	94.8	92.2	87.4	77.7		
	3	Tan to Gray Silty Sand with Gravel	8'-9'	14.4			NP	SM	A-4(0)	100.0	75.1	68.3	60.7	48.4		
	4	Gray Limestone	12'6"-13'												165.3	185.10
	5	Gray Limestone	16'6"-17'												158.1	439.00
	6	Gray Limestone	23'-23'6"												171.4	335.20

LABORATORY TEST RESULTS

PROJECT NUMBER: FY163809

PROJECT: BB0411 Hwy. 16/112 Spur Intchng. Impvts.

DATE: Wednesday, May 25, 2016

B #	S #	Description	Depth Feet	Moisture (%)	LL	PL	PI	USCS	AASHTO	SIEVE ANALYSIS % FINER 3/4 IN	No. 4	NO. 10	NO. 40	NO. 200	UDW pcf	U_c tsf
B5																
	1	Reddish-Brown Clayey Chert Gravel with Sand	2'-3'6"	20.3												
	2	Reddish-Brown Clayey Chert Gravel with Sand	4'-5'6"	25.7												
	3	Reddish-Brown Clayey Chert Gravel with Sand	6'-7'6"	22.1	51	25	26	GC	A-2-7(2)	77.9	53.7	44.4	35.1	28.3		
	4	Reddish-Brown Clayey Chert Gravel with Sand	8'-9'6"	26.9												
	5	Reddish-Brown Clayey Chert Gravel with Sand	10'-11'6"	31.2												
	6	Reddish-Brown Clayey Chert Gravel with Sand	15'-16'6"	27.7												
	7	Tan to Gray Silty Sand with Gravel	20'-21'6"	10.5			NP	SM	A-4(0)	100.0	68.8	57.6	45.7	37.1	121.8	1.12
	8	Reddish-Brown Clayey Chert Gravel with Sand	25'-26'6"	36.7											79.7	1.23
	9	Gray Limestone	28'-28'6"												165.0	351.80
	10	Dark Gray Limestone	33'-33'6"												164.3	240.30
	11	Gray Limestone	38'-38'6"												157.7	510.00
	12	Gray Limestone	45'-45'6"												165.4	320.80
B6																
	1	Reddish-Brown Clayey Chert Gravel with Sand	1'3"-2'9"	21.1	59	29	30	GC	A-2-7(4)	85.1	62.9	53.8	42.6	34.6		
	2	Reddish-Brown Clayey Chert Gravel with Sand	4'-5'6"	16.2												
	3	Reddish-Brown Clayey Chert Gravel with Sand	6'-7'6"	26.3												
	4	Reddish-Brown Clayey Chert Gravel with Sand	8'-9'6"	30.9												
	5	Reddish-Brown Clayey Chert Gravel with Sand	10'-11'6"	20.8												
	6	Reddish-Brown Clayey Chert Gravel with Sand	15'-16'6"	15.5												
	7	Tan to Gray Lean Clay with Sand	20'-21'6"	9.4												
	8	Tan to Gray Lean Clay with Sand	25'-26'6"	17.9	48	24	24	CL	A-7-6(17)	100.0	92.8	86.3	80.0	73.4		
	9	Gray Limestone	30'6"-31'												165.4	368.60
	10	Gray Limestone	37'-37'6"												159.6	578.00

LABORATORY TEST RESULTS

PROJECT NUMBER: FY163809

PROJECT: BB0411 Hwy. 16/112 Spur Intchng. Impvts.

DATE: Wednesday, May 25, 2016

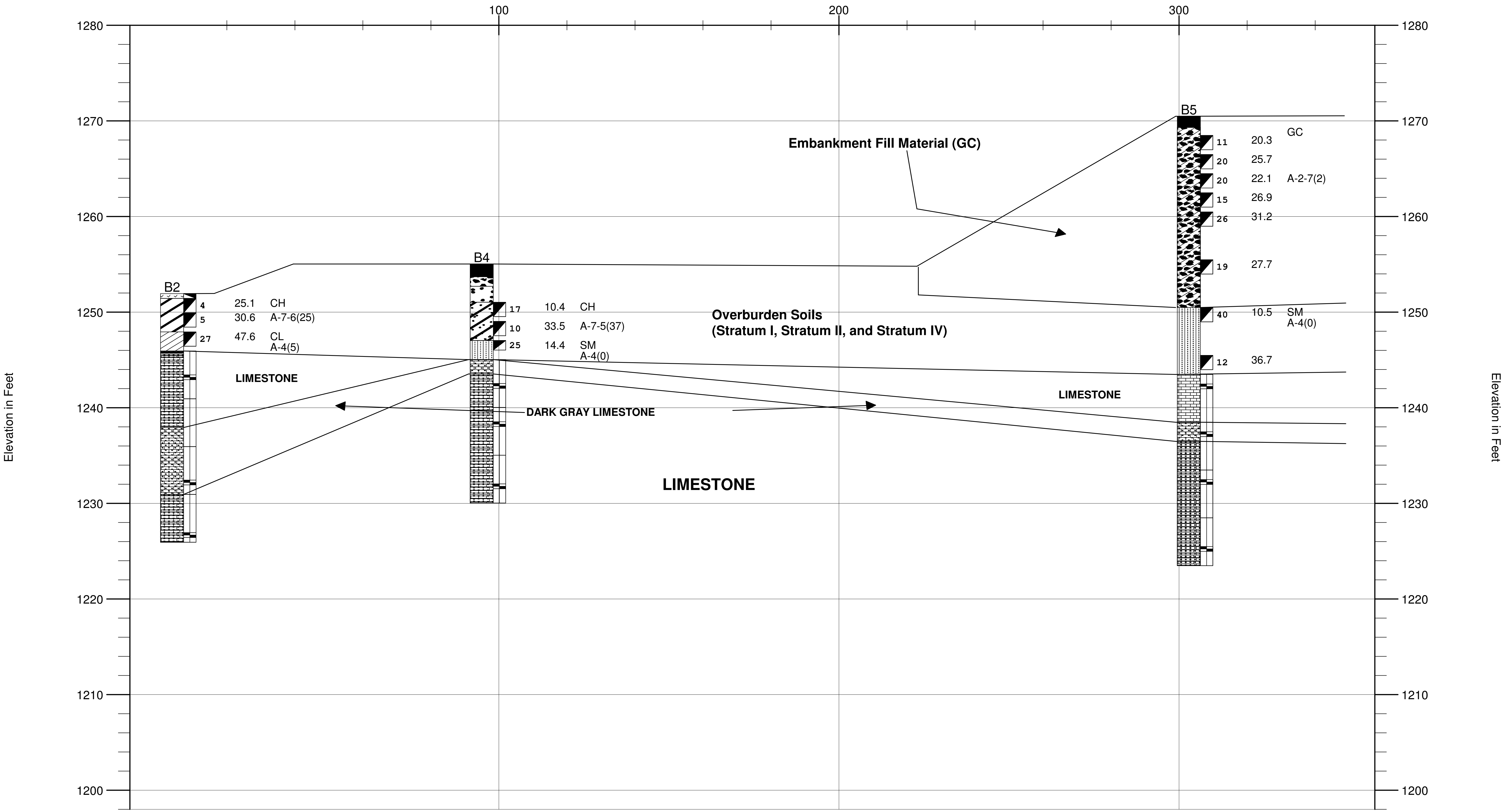
B #	S #	Description	Depth Feet	Moisture (%)	LL	PL	PI	USCS	AASHTO	SIEVE ANALYSIS % FINER 3/4 IN	No. 4	NO. 10	NO. 40	NO. 200	UDW pcf	U_c tsf
P1																
	1	Dark Brown Silty Topsoil	0'-6"	14.5												
	2	Dark Brown to Reddish-Brown Sandy Lean Clay with Gravel	6"-2'	15.9	32	20	12	CL	A-6(5)	100.0	81.6	75.8	70.5	61.8		
	3	Dark Brown to Reddish-Brown Sandy Lean Clay with Gravel	2'-3'6"	19.1												
	4	Dark Brown to Reddish-Brown Sandy Lean Clay with Gravel	4'-5'6"	17.2												
	5	Tan to Reddish-Brown Sandy Silt with Gravel	6'-7'6"	20.6			NP	ML	A-4(0)	100.0	82.6	77.0	71.3	60.5		
	6	Tan to Reddish-Brown Sandy Silt with Gravel	8'-9'6"	29.2											92.4	0.70
	7	Tan to Reddish-Brown Clayey Gravel with Sand	10'-11'6"	25.3												
P2																
	1	Dark Brown Silty Topsoil	0'-6"	28.4												
	2	Dark Brown to Tan Sandy Lean Clay	6"-2'	20.0												
	3	Dark Brown to Tan Sandy Lean Clay	2'-3'6"	21.0	48	25	23	CL	A-7-6(11)	100.0	91.4	84.0	71.5	58.5		
	4	Dark Brown to Tan Sandy Lean Clay	4'-5'6"	21.1											101.4	0.70
	5	Dark Brown to Tan Sandy Lean Clay	6'-7'6"	26.5											98.6	0.50
P3																
	1	Dark Brown Silty Topsoil	0'-6"	39.7												
	2	Dark Brown to Tan Lean Clay with Sand	6"-2'	15.0												
	3	Dark Brown to Tan Lean Clay with Sand	2'-3'6"	20.9	44	23	21	CL	A-7-6(16)	100.0	94.2	91.4	85.6	76.2		
	4	Dark Brown to Tan Lean Clay with Sand	4'-5'6"	24.3											104.1	0.41
	5	Tan to Reddish-Brown Clayey Gravel with Sand	6'-7'6"	23.6												
P4																
	1	Dark Brown Silty Topsoil	0'-6"	19.3												
	2	Dark Brown to Tan Lean Clay	6"-2'	18.8												
	3	Dark Brown to Tan Lean Clay	2'-3'6"	19.8											103.5	0.48
	4	Tan to Reddish-Brown Clayey Gravel with Sand	4'-5'6"	13.7	41	21	20	GC	A-7-6(4)	92.1	59.6	52.5	45.7	39.9		

APPENDIX D

BRIDGE ROCK PROFILE

PROFILE OF BORINGS

BB0411 Hwy. 16/112 Spur Intchng. Impvts.



Fayetteville, Arkansas

MCE McCLELLAND CONSULTING ENGINEERS, INC. DESIGNED TO SERVE

Little Rock, Arkansas

PLATE 16

McCLELLAND CONSULTING ENGINEERS, INC.
DESIGNED TO SERVE

MATERIALS LABORATORY
CTTP LAB NO. 914810

P. O. Box 1229
Fayetteville, Arkansas 72702-1229
479-587-1303
Fax 479-443-9241
www.mcclelland-engrs.com

APPENDIX E

ROADWAY BORINGS SOIL LOG

HWY. 16/112 SPUR INTCHNG. IMPVTS (S)
WASHINGTON COUNTY
ROUTE I-49 SECTION 29
Job No. BB0411
FAP NO. IM-540-1(80)64

Boring No.	Station	Offset	Elevation	Northing	Easting	Latitude	Longitude	Depth from Exist. (ft.)	LL	PI	AASHTO	Color
B-P1	106+34.48	19.16' Rt.	1260.34	642926.32162	661962.69169	36.07616648	-94.20041260	0 to 6	32	12	A-6(5)	Dark Brown to Reddish-Brown
B-P1	106+34.48	19.16' Rt.	1260.34	642926.32162	661962.69169	36.07616648	-94.20041260	6 to 10	NP	NP	A-4(0)	Tan to Reddish-Brown
B-P2	162+74.96	23.01' Lt.	1253.39	641415.09995	661508.38791	36.07673425	-94.19952114	0 to 8	48	23	A-7-6(11)	Dark Brown to Tan
B-P3	166+65.22	03.72' Lt.	1247.40	641636.75939	662123.16603	36.07551998	-94.20247564	0 to 6	44	21	A-7-6(16)	Dark Brown to Tan
B-P4	169+99.75	18.39' Lt.	1244.76	641837.50908	662391.16232	36.07969818	-94.20105303	4 to 6	41	20	A-7-6(4)	Tan to Reddish-Brown

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APPENDIX F - BORING LOCATIONS

HWY. 16/112 SPUR INTCHNG. IMPVTS (S)
WASHINGTON COUNTY
ROUTE I-49 SECTION 29
Job No. BB0411
FAP NO. IM-540-1(80)64

Boring No.	Station	Offset	Elevation	Northing	Easting	Latitude	Longitude
B-B1	132+92.64	49.02' Lt.	1252.393	642506.38070	662077.93427	36.07855196	-94.20063139
B-B2	132+46.51	70.09' Rt.	1251.928	642390.27105	662024.70784	36.07822982	-94.20080268
B-B3	133+58.61	52.70' Lt.	1255.042	642506.07968	662144.00676	36.07855519	-94.20040783
B-B4	133+36.91	54.03' Rt.	1255.022	642400.85693	662115.91743	36.07826449	-94.20049490
B-B5	135+31.69	19.67' Lt.	1270.484	642462.67807	662314.78197	36.07844648	-94.19982679
B-B6	135+24.40	21.95' Rt.	1270.871	642421.58065	662304.99715	36.07833301	-94.19985679
B-P1	106+34.48	19.16' Rt.	1260.344	642926.32162	661962.69169	36.07969818	-94.20105303
B-P2	162+74.96	23.01' Lt.	1253.391	641415.09995	661508.38791	36.07551998	-94.20247564
B-P3	166+65.22	03.72' Lt.	1247.399	641636.75939	662123.16603	36.07616648	-94.20041260
B-P4	169+99.75	18.39' Lt.	1244.764	641837.50908	662391.16232	36.07673425	-94.19952114

APPENDIX G

CBR TESTING RESULTS

MCE DESIGNED TO SERVE McCLELLAND CONSULTING ENGINEERS, INC. Materials Laboratory

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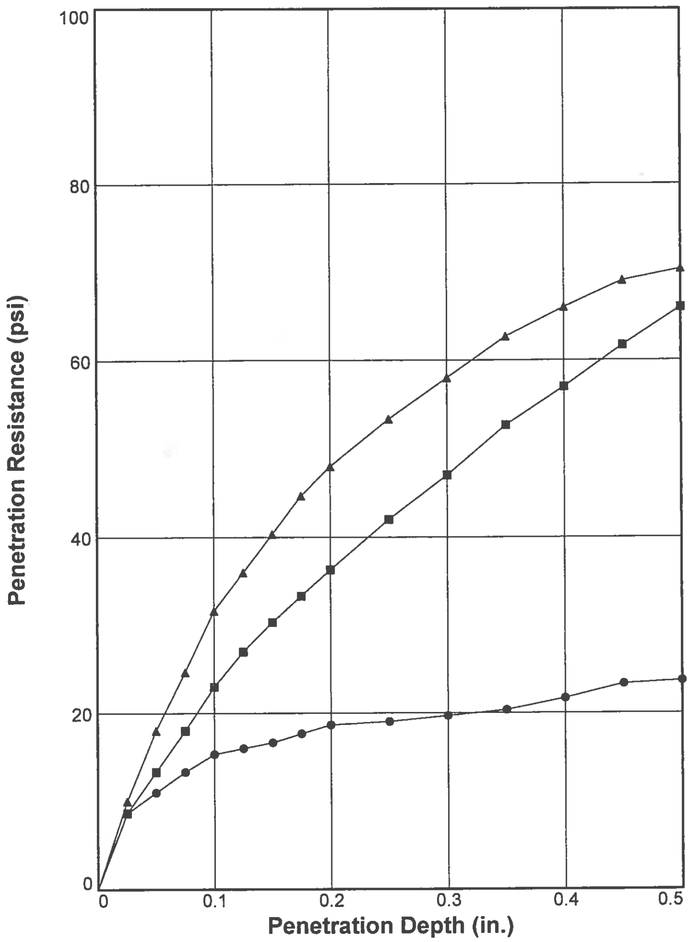
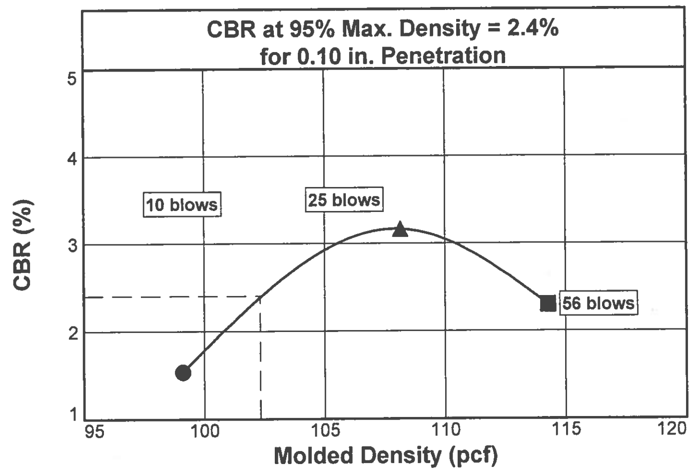
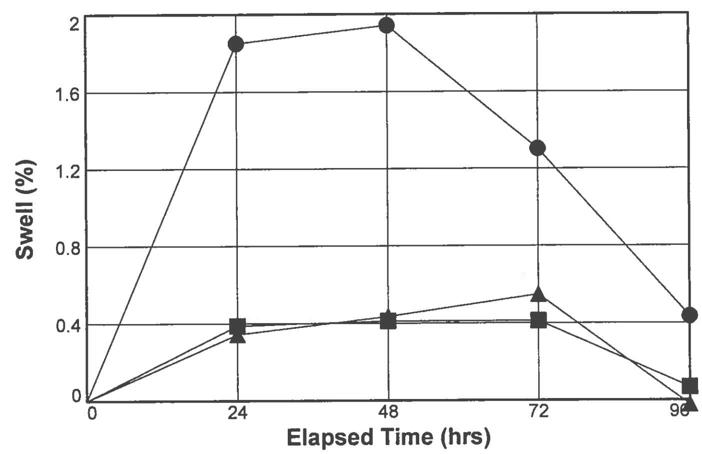
BEARING RATIO TEST REPORT

AASHTO T 193-99







	Molded			Soaked			CBR (%)		Linearity Correction (in.)	Surcharge (lbs.)	Max. Swell (%)
	Density (pcf)	Percent of Max. Dens.	Moisture (%)	Density (pcf)	Percent of Max. Dens.	Moisture (%)	0.10 in.	0.20 in.			
1 ○	99.1	92	19.6	97.2	90.3	16.5	1.5	1.2	0.000	25	1.9
2 △	108.2	100.5	17.3	107.6	99.9	15.4	3.2	3.2	0.000	25	0.5
3 □	114.3	106.1	17.4	113.8	105.7	14.9	2.3	2.4	0.000	25	0.4

Material Description	USCS	Max. Dens. (pcf)	Optimum Moisture (%)	LL	PI
DARK BROWN TO REDDISH-BROWN SANDY LEAN CLAY WITH GRAVEL; Firm; Trace Coarse Sand; Trace Medium Sand; Trace Fine Sand; Little Fine Gravel	CL	107.7	16.1	32	12

Project No: FY163809

Project: BB0411 Hwy. 16/112 Spur Intchng. Impvts.

Source of Sample: P1 **Depth:** .5

Sample Number: 2

Date: 6/7/2016

Test Description/Remarks:

Material sampled from B-P1, approximately 12 inches below existing ground elevations.

BEARING RATIO TEST REPORT

McCLELLAND CONSULTING ENGINEERS, INC.

Figure 1

McCLELLAND CONSULTING ENGINEERS, INC.
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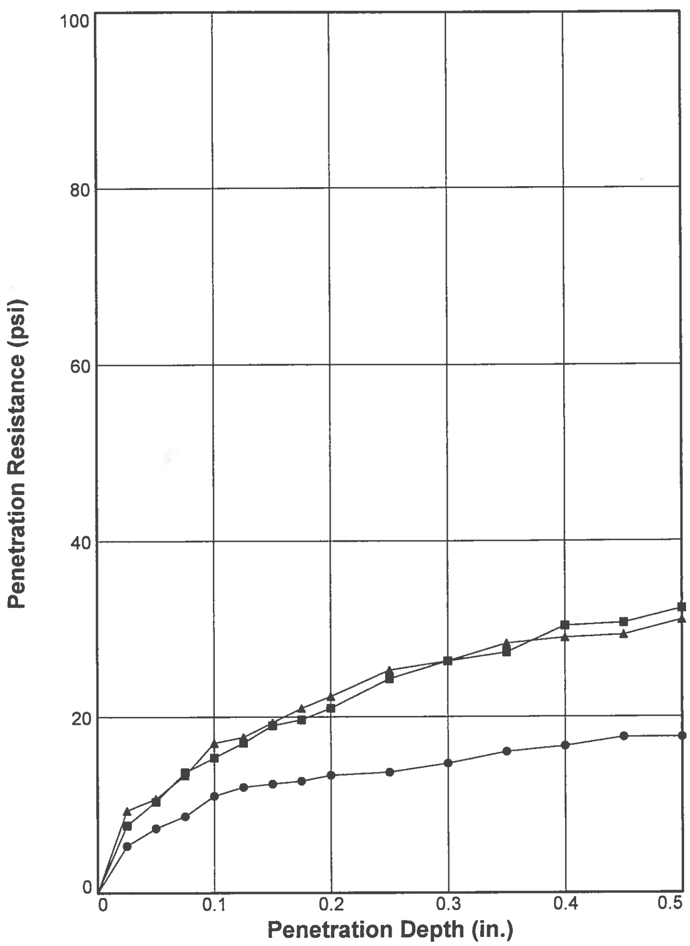
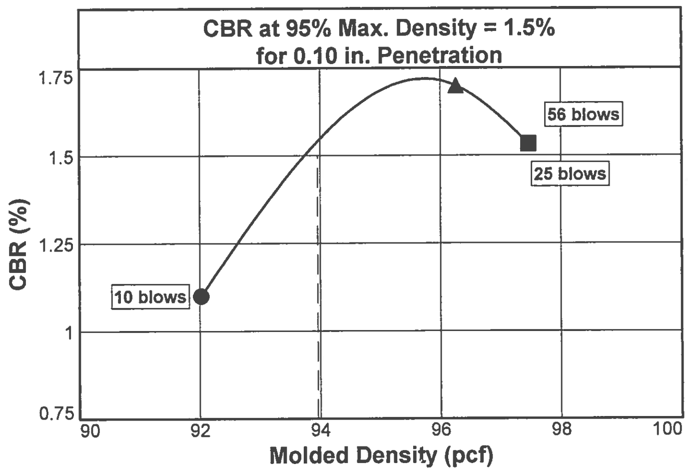
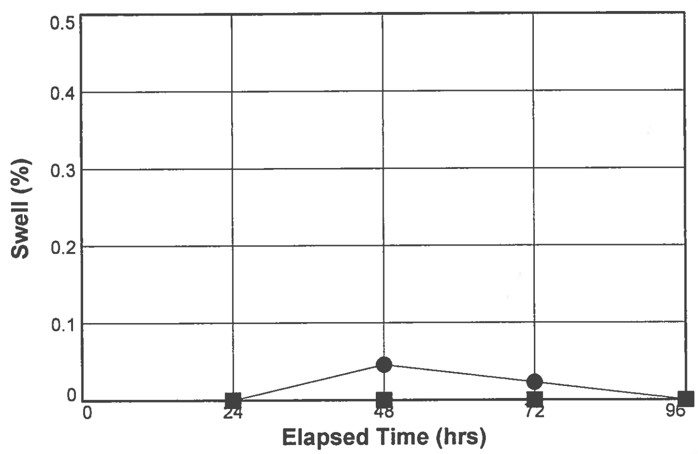
BEARING RATIO TEST REPORT
AASHTO T 193-99







	Molded			Soaked			CBR (%)		Linearity Correction (in.)	Surcharge (lbs.)	Max. Swell (%)
	Density (pcf)	Percent of Max. Dens.	Moisture (%)	Density (pcf)	Percent of Max. Dens.	Moisture (%)	0.10 in.	0.20 in.			
1 ○	92.0	93	21.2	92.0	93	18.1	1.1	0.9	0.000	25	0
2 △	96.3	97.4	28.2	96.3	97.3	12.1	1.7	1.5	0.000	25	0
3 □	97.5	98.6	27.0	97.5	98.5	12.7	1.5	1.4	0.000	25	0

Material Description	USCS	Max. Dens. (pcf)	Optimum Moisture (%)	LL	PI
DARK BROWN TO TAN SANDY LEAN CLAY; Firm; Trace Coarse Sand; Little Medium Sand; Little Fine Sand; Trace Fine Gravel	CL	98.9	21.2	48	25

Project No: FY163809

Project: BB0411 Hwy. 16/112 Spur Intchng. Impvts.

Source of Sample: P2 **Depth:** .5

Sample Number: 2

Date: 6/10/2016

Test Description/Remarks:

Material sampled from B-P2, approximately 12 inches below existing ground elevations.

BEARING RATIO TEST REPORT

McCLELLAND CONSULTING ENGINEERS, INC.

Figure 2

MCE DESIGNED TO SERVE McCLELLAND CONSULTING ENGINEERS, INC. Materials Laboratory

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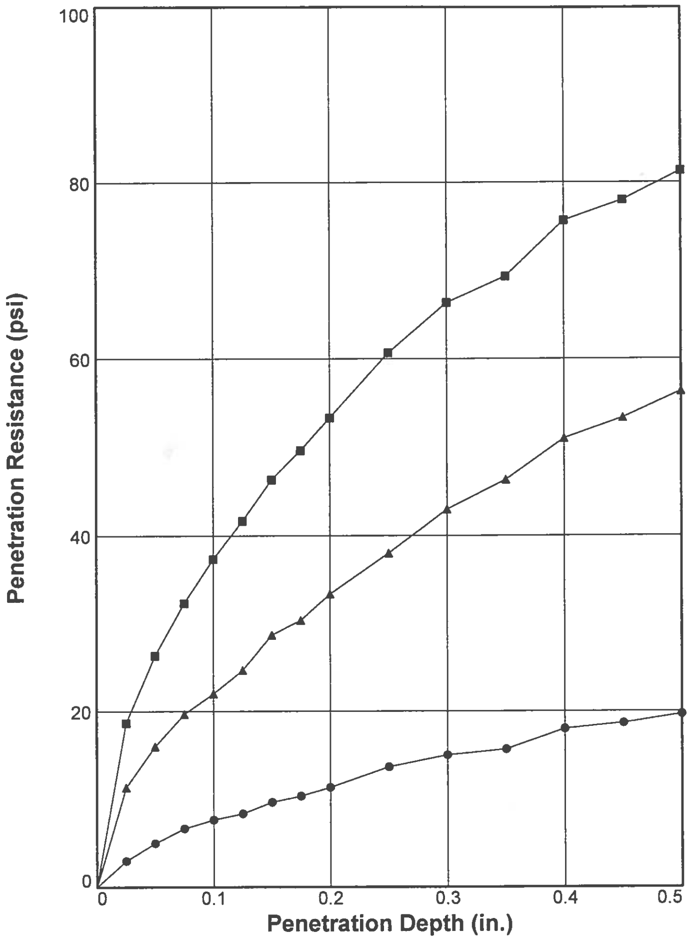
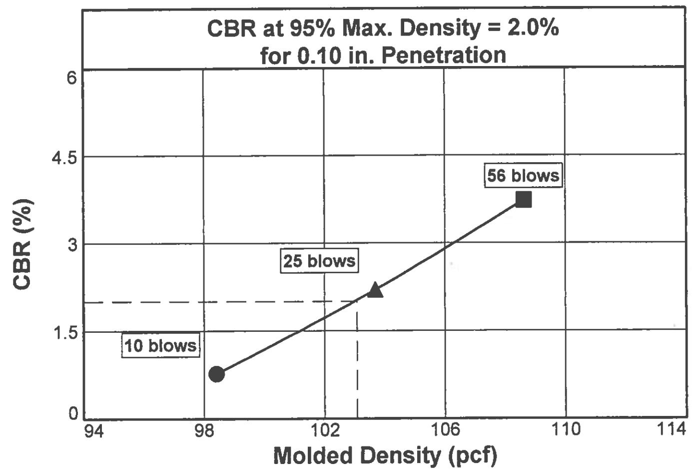
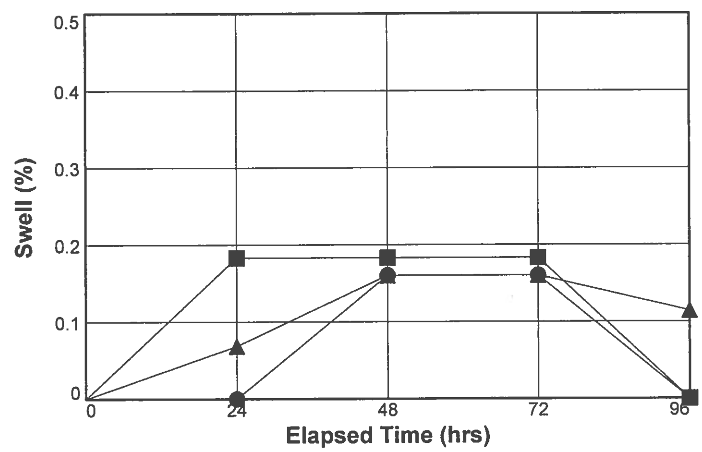
BEARING RATIO TEST REPORT
AASHTO T 193-99







	Molded			Soaked			CBR (%)		Linearity Correction (in.)	Surcharge (lbs.)	Max. Swell (%)
	Density (pcf)	Percent of Max. Dens.	Moisture (%)	Density (pcf)	Percent of Max. Dens.	Moisture (%)	0.10 in.	0.20 in.			
1 ○	98.4	90.7	20.2	98.2	90.6	12.1	0.8	0.8	0.000	25	0.2
2 △	103.7	95.6	22.2	103.5	95.4	14.7	2.2	2.2	0.000	25	0.2
3 □	108.6	100.1	18.1	108.4	99.9	14.2	3.7	3.6	0.000	25	0.2

Material Description	USCS	Max. Dens. (pcf)	Optimum Moisture (%)	LL	PI
DARK BROWN TO TAN LEAN CLAY WITH SAND; Firm; Trace Coarse Sand; Trace Medium Sand; Trace Fine Sand; Trace Fine Gravel	CL	108.5	17.0	44	21

Project No: FY163809
Project: BB0411 Hwy. 16/112 Spur Intchng. Impvts.
Source of Sample: P3 **Depth:** .5
Sample Number: 2
Date: 6/7/16

BEARING RATIO TEST REPORT
McCLELLAND CONSULTING ENGINEERS, INC.

Test Description/Remarks:

Material sampled from B-P3, approximately 12 inches below existing ground elevation.

Figure 3