

**JEF-213-15.18 BRIDGE REPLACEMENT
ODOT PID 86872
JEFFERSON COUNTY, OHIO**

STRUCTURE FOUNDATION EXPLORATION REPORT

Prepared For:

**Korda/Nemeth Engineering, Inc.
1650 Watermark Drive, Suite 200
Columbus, Ohio 43215**

Prepared By:

**Resource International, Inc.
6350 Presidential Gateway
Columbus, Ohio 43231**

Rii Project No. W-24-117

September 2026

**Planning, Engineering, Construction Management, Technology
6350 Presidential Gateway, Columbus, Ohio 43231
P 614.823.4949 F 614.823.4990**





RESOURCE INTERNATIONAL, INC.

6350 Presidential Gateway
Columbus, Ohio 43231
T: 614.823.4949

September 23, 2026

Ms. Christine Placek, P.E.
Senior Associate/Transportation Engineer
Korda/Nemeth Engineering, Inc.
1650 Watermark Drive, Suite 200
Columbus, Ohio 43215

**Re: DRAFT Structure Foundation Exploration Report
JEF-213-15.18 Bridge Replacement
ODOT PID 86872
Jefferson County, Ohio
Rii Project No. W-24-117**

Ms. Placek:

Resource International, Inc. (Rii) is pleased to submit this structure foundation exploration report for the above referenced project. Engineering logs have been prepared and are attached to this report along with the results of laboratory testing. This report includes geotechnical recommendations for the design and construction of the proposed replacement of the existing bridge carrying State Route 213 over North Fork Yellow Creek in Jefferson County, Ohio.

We sincerely appreciate the opportunity to submit this proposal, and look forward to working with you on this project. If you have any questions regarding this proposal, please contact us.

Sincerely,

RESOURCE INTERNATIONAL, INC.

Daniel E Karch, P.E.
Director – Geotechnical Services

Jonathan P. Sterenberg, P.E.
Vice President – Geotechnical Services

Enclosure: Structure Foundation Exploration Report

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration performed for the design and construction of the proposed replacement of the existing bridge (JEF-213-15.18) carrying State Route 213 over the North Fork Yellow Creek in the community of Hammondsville, in Jefferson County, Ohio.

Based on information from Korda, it is understood that the proposed abutment foundation configurations will consist of a total of six (6) drilled shaft under each abutment spaced at 11 feet center to center including one (1) under each wingwall. The abutment drilled shafts will be 3.0 feet in diameter with a 2.5-foot diameter rock socket. It is further understood that the proposed pier foundation configurations will consist of drilled shafts, four (4) shafts spaced at 11 feet center to center. The pier drilled shafts will be 3.5 feet in diameter with a 3.0-foot diameter rock socket. The maximum factored foundation loads on each abutment drilled shaft will be 774 kips, and maximum factored foundation loads on each pier drilled shaft will be 1,452 kips. Based on the available plans, the proposed grade at the bridge will approximately match the existing grades.

Exploration and Findings

Between July 9 and July 11, 2025, a total of four (4) borings, designated as B-001-0-25 through B-004-0-25, were performed for this project. The borings ranged in depth from 20.0 to 39.7 feet below the existing grade. In addition to the soil borings, pavement cores were obtained using a 4.0-inch diameter high-speed core barrel.

Borings B-001-0-25 and B-004-0-25 were performed near the abutments and encountered a composite pavement section with asphalt thicknesses of 5.75 inches overlying concrete with thicknesses of 13.25 and 13.5 inches, followed by aggregate base with thicknesses of 8.0 and 5.25 inches, respectively. Borings B-002-0-25 and B-003-0-25 required coring through the bridge deck. The deck cores were not retained; however, measurements of the deck materials were made, resulting in 6.0 inches of asphalt over 8.0 inches of concrete in each deck core location. Pavement cores X-001-0-25 through X-004-0-25 were performed in the driving lanes encountered asphalt with thicknesses ranging from 10.0 to 12.0 inches overlying aggregate base with thicknesses ranging from 4.0 to 5.0 inches. Pavement cores performed near the abutments in the shoulders encountered asphalt with thicknesses of 11.5 inches overlying aggregate based with thicknesses ranging from 2.0 to 2.5 inches.

Below the surficial materials, the soil borings encountered predominantly granular soils with intermittent layers of cohesive soils to the top of bedrock.

The natural granular soils were described as gravel, gravel with sand, gravel with sand and silt, gravel with sand, silt, and clay, and sandy silt (ODOT A-1-a, A-1-b, A-2-4, A-2-6,

A-4a). The natural cohesive soils were described as sandy silt, silt and clay, and silty clay (ODOT A-4a, A-6a, A-6b).

Natural moisture contents of the soil samples tested ranged from 8 to 28 percent. The natural moisture contents of the cohesive soil samples tested for plasticity index ranged from 4 percent to 7 percent below their corresponding plastic limits. In general, the soils exhibited natural moisture contents estimated to be slightly to significantly below their optimum moisture levels

Bedrock was encountered in the borings at depth ranging from 6.0 to 22.8 feet below the existing ground surface

During drilling groundwater was encountered in borings B-001-0-25, B-003-0-25, and B-004-0-25 at depths of 16.0, 6.7, and 18.5 feet below the existing ground surface, respectively. Groundwater upon completion was encountered in boring B-004-0-25 was measured at a depth of 19.0 feet below the existing ground surface.

Analyses and Recommendations

It is understood that drilled shaft foundations will be utilized to support the pier and abutment substructure units. It is recommended that the drilled shafts extend through the surficial soils, and be socketed a minimum of 1.5 shaft diameters into the competent siltstone, mudstone, shale or sandstone bedrock, as specified in Section 10.8.3.5.4c of the 2014 AASHTO LRDF BDS, and end bear on or within the competent bedrock encountered below the upper incompetent, weak units. Drilled shafts may be proportioned for an allowable bearing capacity as shown in the following table.

Drilled Shaft Recommendations

Substructure Unit (Boring)	Minimum Socket Tip Elevation (feet)	Uniaxial Compressive Strength of Rock Sample (psi)	Nominal End Resistance (ksf)	Resistance Factor, ϕ
Rear Abutment (B-001-0-25)	652.3	4,008	1,440	0.50
Pier 1 (B-002-0-25)	660.2	1,170	420	0.50
Pier 2 (B-003-0-25)	662.8	2,363	650	0.50
Forward Abutment (B-004-0-25)	659.5	1,695	610	0.50

Soil borings performed for the proposed bridge replacement encountered soils consisting of gravel with sand, gravel with sand and silt, gravel with sand, silt and clay, sandy silt, and silt and clay (ODOT A-1-b, A-2-4, A-2-6, A-4a, A-6a) at or near the existing creek bed. Classification tests were performed on the soil samples recovered from where the proposed ground surface elevation meets the face of the proposed abutments and near the creek bed elevation for the scour considerations. A summary of soil parameters for scour analysis are tabulated in the following table.

Parameters for Scour Analysis

Boring Number	Sample	Elevation, (feet)	Soil Type	D₅₀ (mm)
B-001-0-25 (Rear Abutment)	SS-2	686.6-685.1	A-2-4	0.7110
	SS-3	685.1-683.6	A-2-4	2.3670
	SS-4	683.6-682.1	A-1-b	2.5230
	SS-5	682.1-680.6	A-4a	0.2000
B-003-0-25 (Piers)	SS-1	673.5-672.0	A-4a	0.2000
	SS-2	672.0-670.5	A-4a	0.2000
	SS-3	670.5-669.0	A-6a	0.2000
	SS-4	669.0-667.5	A-2-4	0.3670
B-004-2-25 (Forward Abutment)	SS-3A	686.0-685.5	A-2-4	1.0480
	SS-3B	685.5-684.5	A-2-6	1.1350
	SS-4	684.5-683.0	A-2-4	0.7170
	SS-5	683.0-681.5	A-2-4	1.8660
	SS-6 & SS-7	681.5-678.5	A-1-b	1.2960

Please note that this executive summary does not contain all the information presented in the report. The unabridged subsurface exploration report should be read in its entirety to obtain a more complete understanding of the information presented.

1.0 INTRODUCTION

This report is a presentation of the structure foundation exploration performed for the design and construction of the replacement of the existing bridge (JEF-213-15.18) carrying State Route 213 over the North Fork Yellow Creek in the community of Hammondsville, in Jefferson County, Ohio.

Based on the information provided by Korda, the proposed bridge will consist of a three-span reinforced concrete deck on steel beam structure with a 10-degree skew and a span arrangement of 45.0 feet between the rear abutment and Pier 1, 56.25 feet between Pier 1 and Pier 2, and 45.0 feet between Pier 2 and the forward abutment, measured from center to center of the bearings. The length of the proposed bridge is 148.28 feet with a total width of 38.0 feet. Rii understands that the foundation for the proposed structure will consist of abutments and piers supported on deep foundations consisting of drilled shafts socketed into bedrock.

It is understood that the proposed abutment foundation configurations will consist of a total of six (6) drilled shafts under each abutment spaced at 11 feet center to center including one (1) under each wingwall. The abutment drilled shafts will be 3.0 feet in diameter with a 2.5-foot diameter rock socket. It is further understood that the proposed pier foundation configurations will consist of drilled shafts, four (4) shafts spaced at 11 feet center to center. The pier drilled shafts will be 3.5 feet in diameter with a 3.0-foot diameter rock socket. The maximum factored foundation loads on each abutment drilled shaft will be 774 kips, and maximum factored foundation loads on each pier drilled shaft will be 1,452 kips. Based on the available plans, the proposed grade at the bridge will approximately match the existing grades.

This exploration was performed within general accordance of the Ohio Department of Transportation (ODOT) Specifications for Geotechnical Explorations (SGE), the Geotechnical Design Manual (GDM), and the Bridge Design Manual (BDM).

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

Physiographically, the site lies within the Muskingum-Pittsburgh Plateau of the Allegheny Plateaus south of the glacial boundaries. The Muskingum-Pittsburgh Plateau is a dissected plateau having broad major valleys that contain outwash terraces, and tributaries with lacustrine terraces, with medium-grained bedrock sequences coarser than those in the Marietta Plateau but finer than those in the Ironston Plateau. The Muskingum-Pittsburgh Plateau also contains uncommon remnants of the ancient Teays-age drainage system. The plateau has an elevation ranging from 650 to 1,400 feet, with moderately high to high relief.

Based on the Bedrock Geology and Topography Maps obtained from the Ohio Department of Natural Resources (ODNR), the bedrock underlying these deposits consists of the Pennsylvanian-aged Conemaugh Group. The Conemaugh Group consists of shades of gray, green, red, brown, and black shale, siltstone, sandstone, mudstone, and lesser amounts of limestone and coal. Diagnostic features include multicolored mudstone, rare coal beds, thin to thick marine shale and limestone in lower two-thirds of the unit, and rapid vertical and horizontal changes of rock types. This formation ranges from 350 to 490 feet in thickness. Bedrock depth in this area generally ranges from 0 to 30 feet below the ground surface. Within the borings performed for this investigation, siltstone and mudstone bedrock was encountered at depths of

2.2 Existing Conditions

The site of the proposed structure replacement of the existing bridge (JEF-213-15.18) carrying State Route 213 over the North Fork Yellow Creek in the community of Hammondsville, in Jefferson County, Ohio. The existing structure is a non-composite continuous steel beam with reinforced concrete deck, stub abutment and capped pile piers. The span of the existing bridge consists of two other spans of 40 feet and one center span of 50 feet with a total bridge width of approximately 43.2 feet from curb to curb with a 7-inch wearing surface that is aligned north to south. Within the project limits, the roadway traverses primarily along a residential area. Overhead electric lines aligned east to west were observed along the north side of SR 213 within the project limits.

3.0 EXPLORATION

Between July 9 and July 11, 2025, a total of four (4) borings, designated as B-001-0-25 through B-004-0-25, were performed for this project. The borings ranged in depth from 20.0 to 39.7 feet below the existing grade. In addition to the soil borings, pavement cores were obtained using a 4.0-inch diameter high-speed core barrel.

The soil borings and pavement cores were performed at the locations shown on the boring plan provided in Appendix I of this report. A summary of soil borings and pavement core information is provided in Table 1.

Table 1. Test Boring and Pavement Core Summary

Boring Number	Station ¹	Offset ¹	Latitude	Longitude	Ground Elevation ¹ (feet)	Boring Depth (feet)
B-001-0-25	802+95	7' RT	40.557469	-80.705535	690.6	39.7
B-002-0-25	803+34	15' RT	40.557554	-80.705419	678.7 ²	27.0
B-003-0-25	803+63	15' RT	40.557494	-80.705288	673.5 ²	20.0
B-004-0-25	804+43	7' RT	40.557609	-80.705033	691.0	35.0
X-001-0-25	802+70	6' LT	40.557485	-80.705633	690.5	--
X-002-0-25	802+71	12' RT	40.557439	-80.705613	690.5	--
X-003-0-25	804+71	12' LT	40.557645	-80.704941	691.1	--
X-004-0-25	804+70	5' RT	40.557598	-80.704925	691.1	--

1. Station, offset and ground surface elevations were provided from survey performed by Korda/Nemeth Engineering.

2. Borings drilled from bridge deck. Elevation corresponds to top of streambed.

The boring and core locations were determined and located in the field by Rii personnel. Station, offset and ground surface elevations were provided from the survey performed by Korda/Nemeth Engineering. Borings B-001-0-25 and B-004-0-25 were performed at the abutment locations within the existing pavement of SR 213. Borings B-002-0-25 and B-003-0-25 were performed through the deck of the existing bridge and were extended to the stream bed within the North Fork Yellow Creek.

The borings were drilled using a Diedrich D-50 track mounted rotary drilling machine, utilizing 3.25-inch inside diameter hollow-stem augers to advance the holes. Standard penetration testing (SPT) and split spoon sampling were performed at 1.5-foot intervals to depths ranging from 6.0 to 13.0 feet below the existing ground surface, followed by 2.5-foot intervals to the top of rock.

The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a 140-pound hammer falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. A calibrated automatic drop hammer was utilized to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blows per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60} , by the following equation. Both values are represented on boring logs in Appendix III.

$$N_{60} = N^*(ER/60)$$

Where:

N = measured N value

ER = drill rod energy ratio, expressed as a percent, for the system used

The hammer for the Diedrich D-50 drill rig was calibrated on March 25, 2024, and had a drill rod energy ratio of 91.1 percent. Per the ODOT SGE, the energy ratio was limited to 90.0 percent for design purposes.

The determination of the bedrock surface was based on refusal in SPT sampling and/or auger refusal, increased resistance to the advancement of the drilling equipment, and recovered samples. Auger refusal is defined as no or insignificant movement of augers with the weight of the drill rig driving the augers. Where the borings required rock core samples, an NQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. Coring produced a 1.85-inch diameter core from which the type of rock and its geological characteristics were determined.

The rock cores obtained from the borings were logged in the field and visually classified in the Rii laboratory. The retrieved core was analyzed to identify the type of rock, color, mineral content, bedding planes and other geological and mechanical features of interest in this project. The Rock Quality Designation (RQD) for each rock core run was calculated according to the following equation:

$$RQD = \frac{\sum \text{segments equal to or longer than 4.0 inches}}{\text{core run length}} \times 100$$

The RQD value aids in estimating the general quality of the rock and is used in conjunction with other parameters to designate the quality of the rock mass.

At the completion of drilling in borings B-001-0-25 and B-004-0-25, the boreholes were sealed cement-bentonite grout. At the completion of drilling in borings B-002-0-24 and B-003-0-25, the augers were removed and the boreholes were allowed to collapse naturally. The pavement cores were patched with an equivalent thickness of quick set concrete. The bridge deck cores were patched in accordance with Section 408 of the ODOT SGE by over coring approximately one-third of the deck depth, using a steel plate to close of the through core, and then filling the over core hole with quick set concrete, flush with the deck surface.

In general, for instances of no recovery from standard split spoon sampling, a 2.5-inch outside diameter split spoon sampler was driven the full length of the standard split spoon interval plus an additional 6.0 inches to obtain a representative sample. These samples are designated with a "2S" preceding the sample number on the boring logs. Only the final 6.0 inches of sample were retained for classification.

During drilling, Rii personnel prepared field logs detailing the encountered subsurface conditions. Soil samples obtained from the drilling operation were preserved and sealed in glass jars and delivered to the soil laboratory. In the laboratory, the soil samples were visually classified and select samples were tested, as noted in Table 2.

Table 2. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	35
Plastic and Liquid Limits	AASHTO T89, T90	14
Gradation – Sieve/Hydrometer	AASHTO T88	19
Gradation – Sieve Only	AASHTO T88	0
Loss on Ignition	AASHTO T267	2
Uniaxial Compressive Strength of Intact Rock Cores	ASTM D7012, Method C	6
Point Load Strength Index of Rock	ASTM D5731	2

The tests performed are necessary to classify the existing soil according to the ODOT classification system and to estimate engineering properties of importance for foundation design and construction recommendations. Results of the laboratory testing are presented in Appendix IV, as well as partially on the boring logs in Appendix III.

4.0 FINDINGS

Interpreted engineering logs have been prepared based on the field logs, visual examination of samples and laboratory test results. Classification follows the version of the ODOT SGE at the time the exploration borings were performed. The following is a summary of what was found in the test borings and what is represented on the boring logs. A description of the soil and rock terms used throughout this report is presented in Appendix II.

4.1 Current Investigation

4.1.1 Surface Materials

Borings B-001-0-25 and B-004-0-25 were performed near the abutments and encountered a composite pavement section with asphalt thicknesses of 5.75 inches overlying concrete with thicknesses of 13.25 and 13.5 inches, followed by aggregate base with thicknesses of 8.0 and 5.25 inches, respectively. Borings B-002-0-25 and B-003-0-25 required coring through the bridge deck. The deck cores were not retained; however, measurements of the deck materials were made, resulting in 6.0 inches of asphalt over 8.0 inches of concrete in each deck core location. Pavement cores X-001-0-25 through X-004-0-25 were performed in the driving lanes encountered asphalt with thicknesses

ranging from 10.0 to 12.0 inches overlying aggregate base with thicknesses ranging from 4.0 to 5.0 inches. Pavement cores performed near the abutments in the shoulders encountered asphalt with thicknesses of 11.5 inches overlying aggregate based with thicknesses ranging from 2.0 to 2.5 inches.

Table 3. Surface Material Summary

Boring	Asphalt Thickness (in)	Concrete (in)	Aggregate Base (in)
B-001-0-25	5.75	13.25	8.00
B-004-0-25	5.75	13.5	5.25
X-001-0-25	10.0	--	5.0
X-002-0-25	11.5	--	2.0
X-003-0-25	11.5	--	2.5
X-004-0-25	12.0	--	4.0

4.1.2 Subsurface Soils

Below the surficial materials, the soil borings encountered predominantly granular soils with intermittent layers of cohesive soils to the top of bedrock.

The natural granular soils were described as gravel, gravel with sand, gravel with sand and silt, gravel with sand, silt, and clay, and sandy silt (ODOT A-1-a, A-1-b, A-2-4, A-2-6, A-4a). The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the relative density of the granular soils encountered ranged from loose ($5 < N_{60} < 10$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Blow counts recorded from the SPT ranged from 5 to 81 bpf.

The natural cohesive soils were described as sandy silt, silt and clay, and silty clay (ODOT A-4a, A-6a, A-6b). The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The consistency of the cohesive soils encountered were generally stiff ($1.0 < HP \leq 2.0$ tsf). The unconfined compressive strength of the natural cohesive soil samples tested, obtained from the hand penetrometer, ranged from 1.5 to over 2.0 tsf (limit of the instrument).

Natural moisture contents of the soil samples tested ranged from 8 to 28 percent. The natural moisture contents of the cohesive soil samples tested for plasticity index ranged from 4 percent to 7 percent below their corresponding plastic limits. In general, the soils exhibited natural moisture contents estimated to be slightly to significantly below their optimum moisture levels.

4.1.3 Bedrock

Bedrock was encountered in the borings at depth ranging from 6.0 to 22.8 feet below the existing ground surface. The determination of the bedrock surface was based on refusal in SPT sampling and/or auger refusal, increased resistance to the advancement of the drilling equipment, and recovered samples. Additional descriptions of the rock core samples are presented on the boring logs in Appendix III.

Table 4. Top of Bedrock Elevations

Boring Number	Ground Surface Elevation ¹ (feet)	Top of Bedrock		Top of cored rock	
		Depth (feet)	Elevation (feet)	Depth (feet)	Elevation (feet)
B-001-0-25	690.6	23.0	667.6	28.7	661.9
B-002-0-25	678.7	12.0	666.7	12.0	666.7
B-003-0-25	673.5	6.0	667.5	6.0	667.5
B-004-0-25	691.0	22.8	668.2	25.0	666.0

The percent recovery and RQD values of the bedrock core runs and unconfined compressive strength test results in borings are summarized in Table 5.

Table 5. Rock Core Summary

Boring	Core No.	Depth (feet)	Recovery (%)	RQD (%)	Uniaxial Compressive Strength (psi)	Point Load Strength Index (psi)
B-001-0-25	RC-1	28.8-29.7	93	46	--	--
	RC-2	29.7-34.7	93	70	525	--
	RC-3	34.7-39.7	100	72	4,008	--
B-002-0-25	RC-1	12.0-17.0	97	65	--	--
	RC-2	17.0-22.0	98	65	4,338	--
	RC-3	22.0-27.0	93	75	1,170	--
B-003-0-25	RC-1	6.0-10.0	81	31	--	188
	RC-2	10.0-15.0	98	75	--	2,363
	RC-3	15.0-20.0	100	93	1,812	--
B-004-0-25	RC-1	25.0-30.0	100	55	--	
	RC-2	30.0-35.0	100	78	1,695	--

It should be noted that bedrock experienced mechanical breaks during the drilling and coring processes. Rii attempted to account for fresh, manmade breaks during tabulation of the RQD analysis. The percent recovery of the rock core runs ranged from 81 to 100 percent, and the RQD values ranged from 31 to 93 percent. The rock mass quality, according to the RQD values, ranged from poor ($25\% \leq \text{RQD} \leq 50\%$) to excellent ($90\% < \text{RQD} \leq 100\%$), but were generally fair ($50\% < \text{RQD} \leq 75\%$).

4.1.4 Groundwater

Groundwater was encountered in the borings as shown in Table 6.

Table 6. Groundwater Levels

Boring Number	Ground Elevation (feet)	During Drilling		Upon Completion ¹	
		Depth (feet)	Elevation (feet)	Depth (feet)	Elevation (feet)
B-001-0-25	690.6	16.0	674.6	N/A	N/A
B-003-0-25	673.5	6.7	666.8	N/A	N/A
B-004-0-25	691.0	18.5	672.5	19.0 ²	672.0

1. Groundwater depth upon completion was not encountered prior to rock coring, and could not be measured due to the addition of wash water during the rock coring process.

2. Groundwater measured prior to beginning rock coring.

During drilling groundwater was encountered in borings B-001-0-25, B-003-0-25, and B-004-0-25 at depths of 16.0, 6.7, and 18.5 feet below the existing ground surface, respectively. Groundwater upon completion was encountered in boring B-004-0-25 was measured at a depth of 19.0 feet below the existing ground surface.

Please note that short-term water level readings are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels or the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation.

A more comprehensive description of what was encountered during the drilling process may be found on the boring logs in Appendix III.

4.2 Historic Borings

Two (2) historic geotechnical records were found for this project. The historic borings were designated as B-1 and B-8 and were performed north of the existing rear abutment and south of the existing forward abutment, respectively.

The borings encountered natural soil consisting of sandy silt, silty sand with stone fragments, silt sandy gravel, sandy gravel, sandy clay, and clayey silt. Cobbles and

boulders were encountered in each historic boring at a depth of 8 feet below the ground surface, corresponding to elevations of 673.8 and 674.7 feet, respectively. The cohesive soils were generally stiff to hard in consistency and the granular soils were generally loose to medium dense in relative density. Organics consisting of decayed wood were encountered in boring B-1 to a depth of 12.0 feet below the ground surface, corresponding to an elevation of 669.8 feet.

Bedrock was encountered in the historic borings at depths of 15.6 and 16.5 feet below the ground surface, corresponding to an elevation 666.2 feet each. The bedrock consisted of indurated clay, shale, and sandstone.

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the subsurface exploration has been used to determine the foundation support capabilities. These parameters have been used to provide guidelines for the design of foundation systems for the subject bridge, as well as the construction specifications related to the placement of foundation systems and general earthwork recommendations, which are discussed in the following paragraphs.

Design details of the proposed structure were provided by Korda. Based on the information provided, it is understood that the proposed bridge will consist of a three-span reinforced concrete deck on steel beam structure with a 10-degree skew and a span arrangement of 45.0 feet between the rear abutment and Pier 1, 56.25 feet between Pier 1 and Pier 2, and 45.0 feet between Pier 2 and the forward abutment, measured from center to center of the bearings. The length of the proposed bridge is 148.28 feet with a total width of 38.0 feet.

It is understood that the proposed abutment foundation configurations will consist of a total of six (6) drilled shafts under each abutment spaced at 11 feet center to center including one (1) under each wingwall. The abutment drilled shafts will be 3.0 feet in diameter with a 2.5-foot diameter rock socket. It is further understood that the proposed pier foundation configurations will consist of drilled shafts, four (4) shafts spaced at 11 feet center to center. The pier drilled shafts will be 3.5 feet in diameter with a 3.0-foot diameter rock socket. The maximum factored foundation loads on each abutment drilled shaft will be 774 kips, and maximum factored foundation loads on each pier drilled shaft will be 1,452 kips. Based on the available plans, the proposed grade at the bridge will approximately match the existing grades.

If the design details of the proposed structure and/or proposed site grading differs from what is described in this report, Rii should be provided the information for our report revised or amended, as necessary.

Table 7. Bridge Design Elevations

Substructure Unit (Boring)	Structure Component	Elevation (feet)	Design Maximum Factored Load
Rear Abutment (B-001-0-25)	Bottom of Footing / Top of Drilled Shaft	681.00	774
Pier 1 (B-002-0-25)	Bottom of Footing / Top of Drilled Shaft	676.58	1,452
Pier 2 (B-003-0-25)	Bottom of Footing / Top of Drilled Shaft	677.75	1,452
Forward Abutment (B-004-0-25)	Bottom of Footing / Top of Drilled Shaft	681.42	774

It is our understanding that drilled shaft foundation elements will support the bridge. The following design recommendations for drilled shafts are in accordance with Section 1306 of the GDM and 305.4 of the BDM.

5.1 Drilled Shaft Recommendations

5.1.1 Drilled Shafts Extended to Competent Bedrock

It is understood that drilled shaft foundations will be utilized to support the pier and abutment substructure units. It is recommended that the drilled shafts extend through the surficial soils, and be socketed a minimum of 1.5 shaft diameters into the competent siltstone, mudstone, shale or sandstone bedrock, as specified in Section 10.8.3.5.4c of the 2014 AASHTO LRFD BDS, and end bear on or within the competent bedrock encountered below the upper incompetent weak and incompetent units. Drilled shafts may be proportioned for an allowable bearing capacity as shown in Table 8.

Table 8. Drilled Shaft Recommendations

Substructure Unit (Boring)	Minimum Socket Tip Elevation (feet)	Uniaxial Compressive Strength of Rock Sample (psi)	Nominal End Resistance (ksf)	Resistance Factor, ϕ
Rear Abutment (B-001-0-25)	652.3	4,008	1,440	0.50
Pier 1 (B-002-0-25)	660.2	1,170	420	0.50
Pier 2 (B-003-0-25)	662.8	2,363	650	0.50
Forward Abutment (B-004-0-25)	659.5	1,695	610	0.50

Using equation 10.8.3.5.4c-1 of the AASHTO LRFD BDS, the nominal end bearing resistance for intact rock is 2.5 times the unconfined compressive strength of the rock. Based on unconfined compression tests performed on mudstone and siltstone rock cores obtained from borings B-001-0-25 to B-004-0-25, the unconfined compressive strength of the bedrock ranges from 1,170 to 4,338 psi. Using the above noted equation and the limiting unconfined compressive strength from the given range for the bedrock, it is recommended that drilled shaft foundations socketed into competent bedrock be proportioned for a maximum nominal end bearing resistances noted in Table 8 at the strength limit state. A resistance factor of $\phi = 0.50$ at the strength limit state should be utilized for design.

Group efficiency of the shafts, socketed into bedrock, is considered negligible for a minimum spacing of 2.0 shaft diameters center-to-center. Drilled shafts designed in accordance with the requirements presented above should experience a maximum settlement estimated to be less than 0.5 inches.

Due to the difference in the required displacement to mobilize side friction in the overburden soil, the calculated resistance should only consider end bearing and side resistance within the bedrock socket. Side resistance within the overburden soils should be neglected.

5.1.2 Drilled Shaft Considerations

The minimum requirements for proper inspection of drilled shaft construction are as follows:

- A qualified inspector should record the material types being removed from the hole as excavation proceeds.
- When the bearing material has been encountered and identified and/or the design tip elevation has been reached, the shaft walls and base should be observed for anomalies, unexpected soft soil conditions, obstructions or caving.
- Concrete placed freefall should not be allowed to hit the sidewalls of the excavation or the rebar cage and should not pass through any water.
- Structural stability of the rebar cage should be maintained during the concrete pour to prevent buckling.
- The volume of concrete should be checked to ensure voids did not result during extraction of the casing (if utilized).
- The placement of all concrete for the drilled shafts shall follow the American Concrete Institute's Design and Construction of Drilled Piers (ACI 336.3R-93).

- If concrete is placed by tremie method, it must be done so with an adequate head to displace water or slurry if groundwater has entered the caisson (all tremie procedures shall follow applicable ACI specifications).
- Pulling casing with insufficient concrete inside should be restricted.
- The bottom of drilled shaft excavation should be clean and free of loose material. Any loose material observed should be removed using a clean-out bucket (muck bucket).

The use of temporary casing for the drilled shafts is recommended for this project due to the expected caving of materials, given the granular soil conditions and the need for socketing into bedrock.

It is anticipated that conventional drilled shaft equipment (with a standard soil bit) will be able to penetrate the surficial soils to the bedrock depths noted in Table 4. However, it is the responsibility of the contractor to determine the most effective excavation procedures. However, depending on the conditions encountered, additional effort may be needed at or above this depth. Below the depths noted, it will likely be necessary to employ more specialized drilling techniques, such as the use of rock teeth or a rock bit. The ability to penetrate the bedrock will be entirely dependent on the drilled shaft contractor and the equipment employed. It is the responsibility of the contractor to determine the most effective excavation procedures. The elevation and hardness of bedrock is subject to change within the project area.

5.1.3 Lateral Design

If lateral loads or moments are expected to be applied on the foundation elements, they should be analyzed to verify the pile has enough lateral and bending resistance against these loads. A boring-by-boring tabulation of parameters that should be used for lateral loading design is provided in Appendix V. In order to evaluate the lateral capacity, it is recommended that a derivation of COM624, such as LPILE, be utilized to determine the proper embedment depth required to resist the lateral load for a given end condition and deflection. Table 9 lists the eleven different soil types internal to the LPILE program. These strata were utilized to define the soil strata in the soil profile for each boring provided in Appendix III.

Table 9. Subsurface Strata Description

Strata	Description
1	Soft Clay
2	Stiff Clay with Water
3	Stiff Clay without Free Water
4	Sand (Reese)
5	User Defined
6	Vuggy Limestone (Strong Rock)
7	Silt (with cohesion and internal friction angle)
8	API Sand
9	Weak Rock
10	Liquefiable Sand (Rollins)
11	Stiff Clay without free water with a specified initial K (Brown)

5.2 Scour Data

Soil borings performed for the proposed bridge replacement encountered soils consisting of gravel with sand, gravel with sand and silt, gravel with sand, silt and clay, sandy silt, and silt and clay (ODOT A-1-b, A-2-4, A-2-6, A-4a, A-6a) at or near the existing creek bed. Classification tests were performed on the soil samples recovered from where the proposed ground surface elevation meets the face of the proposed abutments and near the creek bed elevation for the scour considerations. A summary of soil parameters for scour analysis are tabulated in Table 10.

Table 10. Parameters for Scour Analysis

Boring Number	Sample	Elevation, (feet)	Soil Type	D ₅₀ (mm)
B-001-0-25 (Rear Abutment)	SS-2	686.6-685.1	A-2-4	0.7110
	SS-3	685.1-683.6	A-2-4	2.3670
	SS-4	683.6-682.1	A-1-b	2.5230
	SS-5	682.1-680.6	A-4a	0.2000
B-003-0-25 (Piers)	SS-1	673.5-672.0	A-4a	0.2000
	SS-2	672.0-670.5	A-4a	0.2000
	SS-3	670.5-669.0	A-6a	0.2000
	SS-4	669.0-667.5	A-2-4	0.3670
B-004-2-25 (Forward Abutment)	SS-3A	686.0-685.5	A-2-4	1.0480
	SS-3B	685.5-684.5	A-2-6	1.1350
	SS-4	684.5-683.0	A-2-4	0.7170
	SS-5	683.0-681.5	A-2-4	1.8660
	SS-6 & SS-7	681.5-678.5	A-1-b	1.2960

5.3 Lateral Earth Pressure

For the soil types encountered in the borings, the “in-situ” unit weight (γ), cohesion (c), effective angle of friction (ϕ'), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 11 and Table 12.

Table 11. Estimated Undrained (Short-term) Soil Parameters for Design

Soil Type	γ (pcf) ¹	c (psf)	ϕ	k_a	k_o	k_p
Stiff Cohesive Soil	115	1,000	0°	N/A	N/A	N/A
Very Stiff Cohesive Soil	125	2,500	0°	N/A	N/A	N/A
Hard Cohesive Soil	130	4,000	0°	N/A	N/A	N/A
Loose Granular Soil	120	0	26°	0.39	0.56	2.56
Medium Dense Granular Soil	125	0	30°	0.33	0.50	3.00
Very Dense Granular Soil	135	0	36°	0.26	0.41	3.85
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	130	0	32°	0.31	0.47	3.25

1. When below groundwater table, use effective unit weight, $\gamma' = \gamma - 62.4$ pcf and add hydrostatic water pressure.

Table 12. Estimated Drained (Long-term) Soil Parameters for Design

Soil Type	γ (pcf) ¹	c' (psf)	ϕ'	k_a	k_o	k_p
Stiff Cohesive Soil	115	0	25°	0.41	0.58	2.46
Very Stiff Cohesive Soil	125	0	27°	0.38	0.55	2.66
Hard Cohesive Soil	130	0	28°	0.36	0.53	2.77
Loose Granular Soil	120	0	26°	0.39	0.56	2.56
Medium Dense Granular Soil	125	0	30°	0.33	0.50	3.00
Very Dense Granular Soil	135	0	36°	0.26	0.41	3.85
Compacted Cohesive Engineered Fill	120	0	28°	0.36	0.53	2.77
Compacted Granular Engineered Fill	130	0	32°	0.31	0.47	3.25

1. When below groundwater table, use effective unit weight, $\gamma' = \gamma - 62.4$ pcf and add hydrostatic water pressure.

These parameters are considered appropriate for the design of all subsurface structures and any excavation support systems. Subsurface structures (where the top of the structure is restrained from movement) should be designed based on at-rest conditions (k_o). For proposed temporary retaining structures (where the top of the structure is allowed to move), earth pressure distributions should be based on active (k_a) and passive (k_p) conditions.

These recommendations do not take into consideration the effect of any surcharge loading or a sloped ground surface (a flat surface is assumed). Earth pressures on excavation support systems will be dependent on the type of sheeting and method of bracing or anchorage. Surcharge loads, such as that imposed by traffic loading, will create additional lateral loading on the subsurface structures and excavation support systems. The resulting lateral earth pressure should be evaluated based on active (k_a) and at-rest (k_o) conditions and the anticipated magnitude of the loading.

Where necessary, temporary retaining structures, such as sheet pile system, should be designed using the undrained soil parameters provided in Table 11, and the design should follow all applicable guidelines for the type of retaining structure utilized. Permanent retaining and subsurface structures should be designed using the drained soil parameters provided in Table 12. Regardless of whether the retaining structure is temporary or permanent, the effective unit weight ($\gamma' = \gamma - 62.4$ pcf) plus the hydrostatic water pressure ($\gamma_w * h_w$, where h_w is the height of water behind the wall above the base of the wall) should be utilized below the design groundwater level. The lateral earth pressure coefficients should only be applied to the horizontal pressure resulting from the effective overburden pressure, and should not be applied to the hydrostatic water pressure.

5.4 Construction Considerations

All site work shall conform to local codes and to the latest ODOT Construction and Materials Specifications (CMS), including that all excavation and embankment preparation and construction should follow ODOT Item 200 (Earthwork).

5.4.1 Excavation Considerations

All excavations should be shored / braced or laid back at a safe angle in accordance with Occupational Safety and Health Administration (OSHA) guidelines. During excavation, if slopes cannot be laid back to OSHA Standards due to adjacent structures or other obstructions, temporary shoring may be required. Actual excavation back slopes must be field verified by qualified personnel at the time of excavation in strict accordance with OSHA guidelines.

5.4.2 Groundwater Considerations

Groundwater should be anticipated at or near the elevation as indicated in Table 6. Groundwater should be anticipated within the upper granular soils at the creek elevation when the creek level is above the normal pool elevation. Additionally, groundwater conditions affecting construction may be encountered within the trapped/perched zones. These trapped/perched zones are generally the layer(s) of granular soils that are isolated within the fine-grained soil layers and may not be identified in boring logs. If excavation encounter such layers, temporary dewatering may be accomplished by placing localized sumps and pumps within and beyond the excavation. Seepage rates from these layers are difficult to predict and flow rate could be significant.

Groundwater, wherever encountered, proper groundwater control measures should be implemented to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or “boiling” condition if soft/loose silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36.0 inches below the deepest excavation. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

6.0 LIMITATIONS OF STUDY

The above recommendations are predicated upon construction inspection by a qualified soil technician under the direct supervision of a professional geotechnical engineer. Adequate testing and inspection during construction are considered necessary to assure an adequate foundation system and are part of these recommendations.

The recommendations for this project were developed utilizing subsurface information obtained from the test borings that were made at the proposed site for the current

investigation. Resource International is not responsible for the data, conclusions, opinions or recommendations made by others during previous investigations at this site. At this time, we would like to point out that soil borings only depict the subsurface conditions at the specific locations and time at which they were made. The conditions at other locations on the site may differ from those occurring at the boring locations.

The conclusions and recommendations herein have been based upon the available subsurface information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be brought to the attention of the geotechnical engineer to determine whether any changes in the foundation or earthwork recommendations are necessary. If deviations from the noted subsurface conditions are encountered during construction, they should also be brought to the attention of the geotechnical engineer.

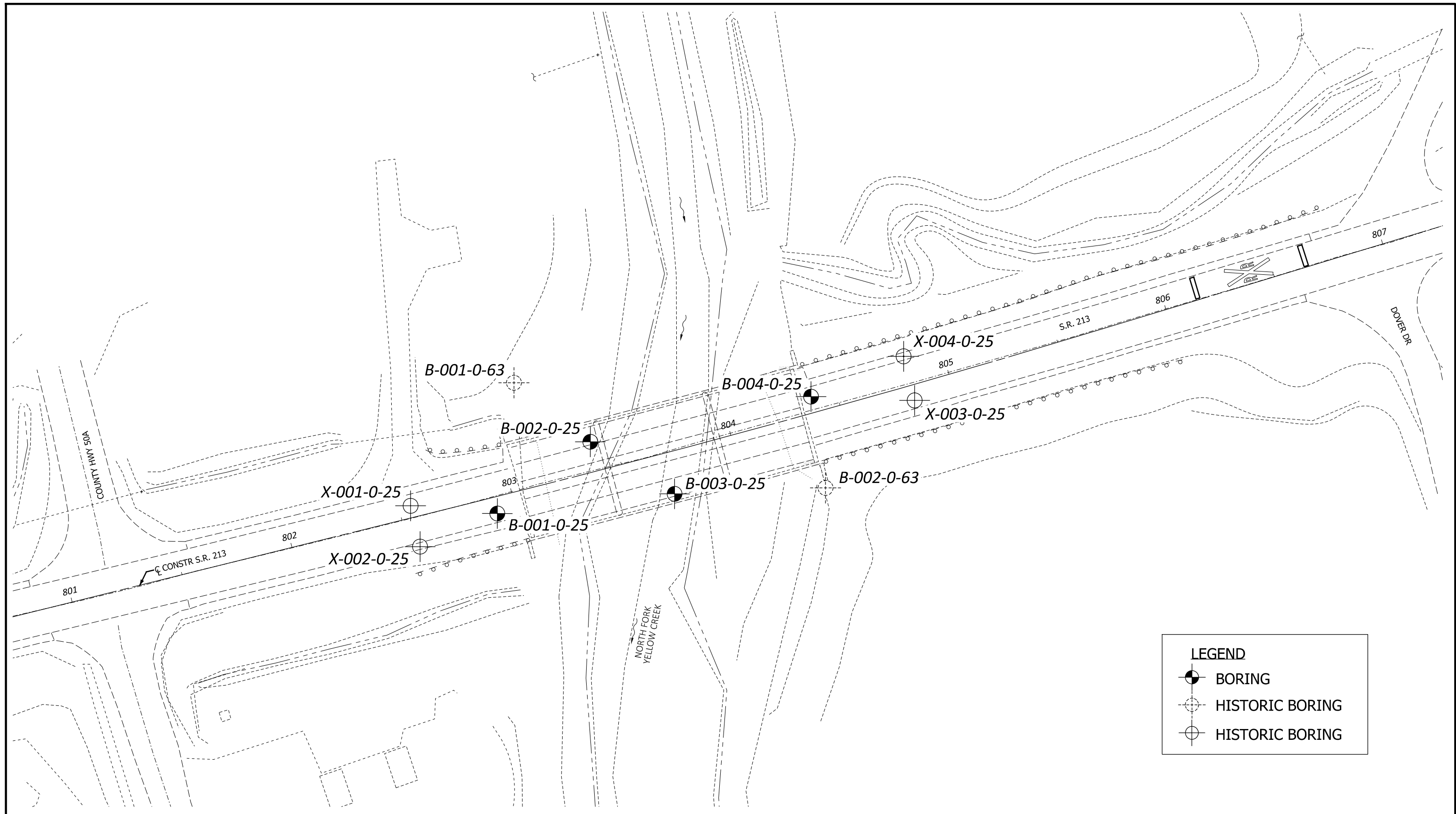
The scope of our services does not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, groundwater or surface water within or beyond the site studied. Any statements in this report or on the test boring logs regarding odors, staining of soils or other unusual conditions observed are strictly for the information of our client.

Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. Resource International is not responsible for the conclusions, opinions or recommendations made by others based upon the data included.



APPENDIX I

VICINITY MAP AND BORING PLAN



LEGEND

- BORING
- HISTORIC BORING
- HISTORIC BORING

BORING PLAN JEF-213-15.18 JEFFERSON COUNTY, OHIO	RII PROJECT NO. W-24-117		DRAWN ALF		
	SCALE: 1"=40' 0 20 40	REVIEWED DEK	DATE 7/22/2025		

APPENDIX II

DESCRIPTION OF SOIL AND ROCK TERMS



CLASSIFICATION OF SOILS

Ohio Department of Transportation

(The classification of a soil is found by proceeding from top to bottom of the chart.
The first classification that the test data fits is the correct classification.)

SYMBOL	DESCRIPTION	Classification		LL _O /LL × 100*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
		AASHTO	OHIO							
	Gravel and/or Stone Fragments	A-1-a			30 Max.	15 Max.		6 Max.	0	Min. of 50% combined gravel, cobble and boulder sizes
	Gravel and/or Stone Fragments with Sand	A-1-b			50 Max.	25 Max.		6 Max.	0	
	Fine Sand	A-3			51 Min.	10 Max.	NON-PLASTIC		0	
	Coarse and Fine Sand	--	A-3a			35 Max.		6 Max.	0	Min. of 50% combined coarse and fine sand sizes
	Gravel and/or Stone Fragments with Sand and Silt	A-2-4				35 Max.	40 Max.	10 Max.	0	
		A-2-5					41 Min.			
	Gravel and/or Stone Fragments with Sand, Silt and Clay	A-2-6				35 Max.	40 Max.	11 Min.	4	
		A-2-7					41 Min.			
	Sandy Silt	A-4	A-4a	76 Min.		36 Min.	40 Max.	10 Max.	8	Less than 50% silt sizes
	Silt	A-4	A-4b	76 Min.		50 Min.	40 Max.	10 Max.	8	50% or more silt sizes
	Elastic Silt and Clay	A-5		76 Min.		36 Min.	41 Min.	10 Max.	12	
	Silt and Clay	A-6	A-6a	76 Min.		36 Min.	40 Max.	11 - 15	10	
	Silty Clay	A-6	A-6b	76 Min.		36 Min.	40 Max.	16 Min.	16	
	Elastic Clay	A-7-5		76 Min.		36 Min.	41 Min.	≤ LL-30	20	
	Clay	A-7-6		76 Min.		36 Min.	41 Min.	> LL-30	20	
	Organic Silt	A-8	A-8a	75 Max.		36 Min.				W/o organics would classify as A-4a or A-4b
	Organic Clay	A-8	A-8b	75 Max.		36 Min.				W/o organics would classify as A-5, A-6a, A-6b, A-7-5 or A-7-6
MATERIAL CLASSIFIED BY VISUAL INSPECTION										
	Sod and Topsoil									
	Pavement or Base	Uncontrolled Fill (Describe)		Bouldery Zone		Peat				

* Only perform the oven-dried liquid limit test and this calculation if organic material is present in the sample.

DESCRIPTION OF SOIL TERMS

The following terminology was used to describe soils throughout this report and is generally adapted from ASTM 2487/2488 and ODOT Specifications for Geotechnical Explorations.

Granular Soils – ODOT A-1, A-2, A-3, A-4 (non-plastic)

The relative compactness of granular soils is described as:

<u>Description</u>	<u>Blows per foot – SPT (N₆₀)</u>		
Very Loose	Below		5
Loose	5	-	10
Medium Dense	11	-	30
Dense	31	-	50
Very Dense	Over		50

Cohesive Soils – ODOT A-4, A-5, A-6, A-7, A-8

The relative consistency of cohesive soils is described as:

<u>Description</u>	<u>Unconfined Compression (tsf)</u>		
Very Soft	Less than		0.25
Soft	0.25	-	0.5
Medium Stiff	0.5	-	1.0
Stiff	1.0	-	2.0
Very Stiff	2.0	-	4.0
Hard	Over		4.0

Gradation - The following size-related denominations are used to describe soils:

<u>Soil Fraction</u>	<u>Size</u>
Boulders	Larger than 12"
Cobbles	12" to 3"
Gravel coarse	3" to ¾"
fine	¾" to 2.0 mm (¾" to #10 Sieve)
Sand coarse	2.0 mm to 0.42 mm (#10 to #40 Sieve)
fine	0.42 mm to 0.074 mm (#40 to #200 Sieve)
Silt	0.074 mm to 0.005 mm (#200 to 0.005 mm)
Clay	Smaller than 0.005 mm

Modifiers of Components - The following modifiers indicate the range of percentages of the minor soil components:

<u>Term</u>	<u>Range</u>		
Trace	0%	-	10%
Little	10%	-	20%
Some	20%	-	35%
And	35%	-	50%

Moisture Table - The following moisture-related denominations are used to describe cohesive soils:

<u>Term</u>	<u>Range - ODOT</u>
Dry	Well below Plastic Limit
Damp	Below Plastic Limit
Moist	Above PL to 3% below LL
Wet	3% below LL to above LL

Organic Content – The following terms are used to describe organic soils:

<u>Term</u>	<u>Organic Content (%)</u>
Slightly organic	2-4
Moderately organic	4-10
Highly organic	>10

Bedrock – The following terms are used to describe the relative strength of bedrock:

<u>Description</u>	<u>Field Parameter</u>
Very Weak	Can be carved with knife and scratched by fingernail. Pieces 1 in. thick can be broken by finger pressure.
Weak	Can be grooved or gouged with knife readily. Small, thin pieces can be broken by finger pressure.
Slightly Strong	Can be grooved or gouged 0.05 in deep with knife. 1 in. size pieces from hard blows of geologist hammer.
Moderately Strong	Can be scratched with knife or pick. 1/4 in. size grooves or gouges from blows of geologist hammer.
Strong	Can be scratched with knife or pick with difficulty. Hard hammer blows to detach hand specimen.
Very Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to detach hand specimen.
Extremely Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to chip hand specimen.

DESCRIPTION OF ROCK TERMS

The following terminology was used to describe the rock throughout this report and is generally adapted from ASTM D5878 and the ODOT Specifications for Geotechnical Explorations.

Weathering – Describes the degree of weathering of the rock mass:

<u>Description</u>	<u>Field Parameter</u>
Unweathered	No evidence of any chemical or mechanical alteration of the rock mass. Mineral crystals have a right appearance with no discoloration. Fractures show little or not staining on surfaces.
Slightly Weathered	Slight discoloration of the rock surface with minor alterations along discontinuities. Less than 10% of the rock volume presents alteration.
Moderately Weathered	Portions of the rock mass are discolored as evident by a dull appearance. Surfaces may have a pitted appearance with weathering "halos" evident. Isolated zones of varying rock strengths due to alteration may be present. 10 to 15% of the rock volume presents alterations.
Highly Weathered	Entire rock mass appears discolored and dull. Some pockets of slightly to moderately weathered rock may be present and some areas of severely weathered materials may be present.
Severely Weathered	Majority of the rock mass reduced to a soil-like state with relic rock structure discernable. Zones of more resistant rock may be present but the material can generally be molded and crumbled by hand pressures.

Strength of Bedrock – The following terms are used to describe the relative strength of bedrock:

<u>Description</u>	<u>Field Parameter</u>
Very Weak	Can be carved with knife and scratched by fingernail. Pieces 1 in. thick can be broken by finger pressure.
Weak	Can be grooved or gouged with knife readily. Small, thin pieces can be broken by finger pressure.
Slightly Strong	Can be grooved or gouged 0.05 in deep with knife. 1 in. size pieces from hard blows of geologist hammer.
Moderately Strong	Can be scratched with knife or pick. 1/4 in. size grooves or gouges from blows of geologist hammer.
Strong	Can be scratched with knife or pick with difficulty. Hard hammer blows to detach hand specimen.
Very Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to detach hand specimen.
Extremely Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to chip hand specimen.

Bedding Thickness – Description of bedding thickness as the average perpendicular distances between bedding surfaces:

<u>Description</u>	<u>Thickness</u>
Very Thick	Greater than 36 inches
Thick	18 to 36 inches
Medium	10 to 18 inches
Thin	2 to 10 inches
Very Thin	0.4 to 2 inches
Laminated	0.1 to 0.4 inches
Thinly Laminated	Less than 0.1 inches

Fracturing – Describes the degree and condition of fracturing (fault, joint, or shear):

Degree of Fracturing

<u>Description</u>	<u>Spacing</u>
Unfractured	Greater than 10 feet
Intact	3 to 10 feet
Slightly Fractured	1 to 3 feet
Moderately Fractured	

Aperture Width

<u>Description</u>	<u>Width</u>
Open	Greater than 0.2 inches
Narrow	0.05 to 0.2 inches
Tight	Less than 0.05 inches

Surface Roughness

<u>Description</u>	<u>Criteria</u>
Very Rough	Near vertical steps and ridges occur on surface
Slightly Rough	Asperities on the surfaces distinguishable
Slickensided	Surface has smooth, glassy finish, evidence of Striations

RQD – Rock Quality Designation (calculation shown in report) and Rock Quality (ODOT, GB 3, January 13, 2006):

<u>RQD %</u>	<u>Rock Index Property Classification (based on RQD, not slake durability index)</u>
0 – 25%	Very Poor
26 – 50%	Poor
51 – 70%	Fair
71 – 85%	Good
86 – 100%	Very Good

BORING LOGS

Definitions of Abbreviations

AS	=	Auger sample
GI	=	Group index as determined from the Ohio Department of Transportation classification system
HP	=	Unconfined compressive strength as determined by a hand penetrometer (tons per square foot)
LL _o	=	Oven-dried liquid limit as determined by ASTM D4318. Per ASTM D2487, if LL _o /LL is less than 75 percent, soil is classified as "organic".
LOI	=	Percent organic content (by weight) as determined by ASTM D2974 (loss on ignition test)
PID	=	Photo-ionization detector reading (parts per million)
QR	=	Unconfined compressive strength of intact rock core sample as determined by ASTM D2938 (pounds per square inch)
QU	=	Unconfined compressive strength of soil sample as determined by ASTM D2166 (pounds per square foot)
RC	=	Rock core sample
REC	=	Ratio of total length of recovered soil or rock to the total sample length, expressed as a percentage
RQD	=	Rock quality designation – estimate of the degree of jointing or fracture in a rock mass, expressed as a percentage:

$$\frac{\sum \text{segments equal to or longer than 4.0 inches}}{\text{core run length}} \times 100$$

SO ₄	=	Sulfate content (parts per million)
SPT	=	Standard penetration test blow counts, per ASTM D1586. Driving resistance recorded in terms of blows per 6-inch interval while letting a 140-pound hammer free fall 30 inches to drive a 2-inch outer diameter (O.D.) split spoon sampler a total of 18 inches. The second and third intervals are added to obtain the number of blows per foot (N _m).
N ₆₀	=	Measured blow counts corrected to an equivalent (60 percent) energy ratio (ER) by the following equation: N ₆₀ = N _m *(ER/60)
SS	=	Split spoon sample
2S	=	For instances of no recovery from standard SS interval, a 2.5 inch O.D. split spoon is driven the full length of the standard SS interval plus an additional 6.0 inches to obtain a representative sample. Only the final 6.0 inches of sample is retained. Blow counts from 2S sampling are not correlated with N ₆₀ values.
3S	=	Same as 2S, but using a 3.0 inch O.D. split spoon sampler.
TR	=	Top of rock
W	=	Initial water level measured during drilling
▽	=	Water level measured at completion of drilling

Classification Test Data

Gradation (as defined on Description of Soil Terms):

GR	=	% Gravel
SA	=	% Sand
SI	=	% Silt
CL	=	% Clay

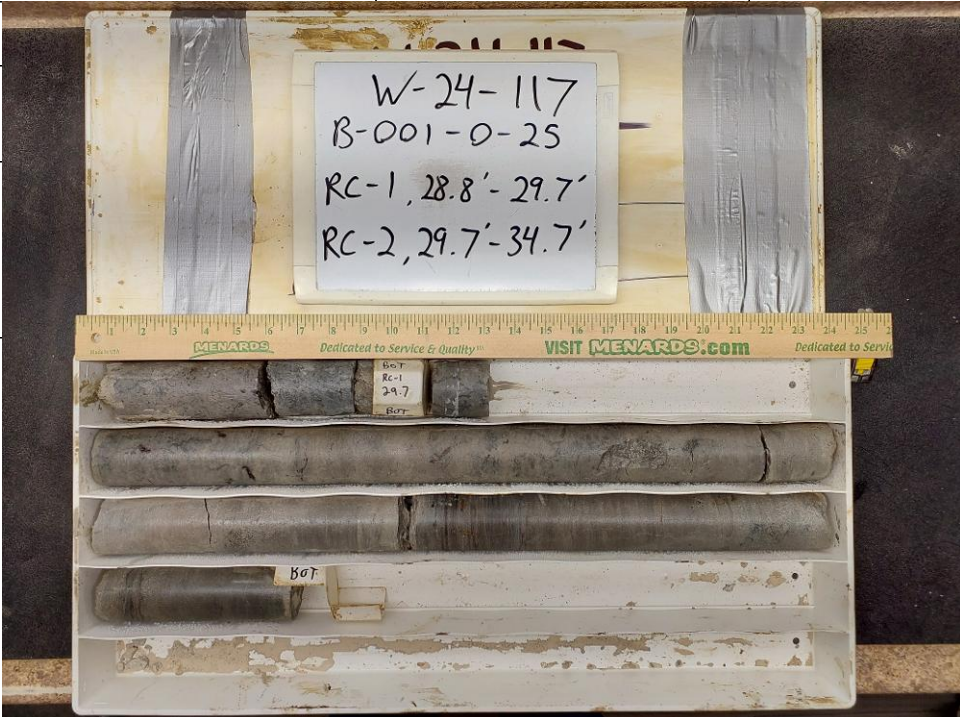
Atterberg Limits:

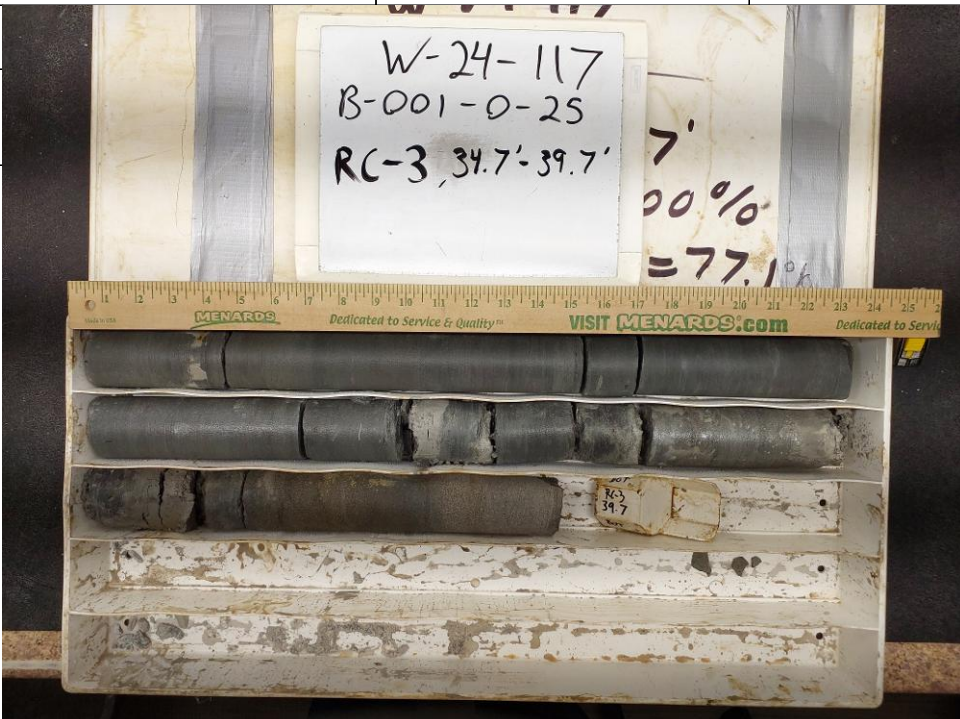
LL	=	Liquid limit
PL	=	Plastic limit
PI	=	Plasticity Index
WC	=	Water content (%)

PROJECT: JEF-213-15.18		DRILLING FIRM / OPERATOR: RII / IS		DRILL RIG: DIEDRICH D-50 (# 313)		STATION / OFFSET: 802+95 / 7' RT.		EXPLORATION ID B-001-0-25												
TYPE: BRIDGE REPLACEMENT		SAMPLING FIRM / LOGGER: RII / BH		HAMMER: AUTOMATIC		ALIGNMENT: SR-213														
PID: 86872 SFN: 4103181		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 3/25/24		ELEVATION: 690.6 ft. EOB: 39.7 ft.		PAGE 1 OF 2												
START: 7/8/25 END: 7/8/25		SAMPLING METHOD: SPT		ENERGY RATIO (%): 90		LAT / LONG: 40.557469, -80.705535														
MATERIAL DESCRIPTION AND NOTES		ELEV. 690.6	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
									GR	CS	FS	SI	CL	LL	PL	PI				
0.5' - ASPHALT (5.75")		690.1																		
1.1' - CONCRETE (13.25")		689.0	1																	
0.7' - AGGREGATE BASE (8.0")		688.3	2																	
MEDIUM DENSE, BROWN TO GRAY GRAVEL WITH SAND AND SILT , LITTLE TO TRACE CLAY, DAMP TO WET. -SS-3: LOI= 3.5%		683.6	3	3	18	89	SS-1A	-	28	16	23	21	12	25	17	8	18	A-2-4 (0)	-	
			4	9			SS-1B	-	-	-	-	-	-	-	-	-	15	A-2-4 (V)	-	
			5	10	27	75	SS-2	-	40	15	17	16	12	28	18	10	11	A-2-4 (0)	-	
			6	5																
			7	3	11	69	SS-3	-	54	12	13	12	9	24	17	7	10	A-2-4 (0)	-	
MEDIUM DENSE, BROWN GRAVEL WITH SAND , TRACE SILT, TRACE CLAY, MOIST.		682.1	8	4	11	67	SS-4	-	54	14	18	9	5	NP	NP	NP	8	A-1-b (0)	-	
LOOSE TO MEDIUM DENSE, BROWN TO DARK BROWN SANDY SILT , LITTLE CLAY, TRACE FINE GRAVEL, MOIST TO WET.		676.3	9	3	11	61	SS-5	-	9	9	42	27	13	NP	NP	NP	25	A-4a (1)	-	
			10	1	3															
			11	1	6	78	SS-6	-	-	-	-	-	-	-	-	-	28	A-4a (V)	-	
			12	1	3															
			13	2	8	86	SS-7	-	1	1	42	38	18	27	19	8	23	A-4a (4)	-	
MEDIUM DENSE, BROWN TO DARK BROWN GRAVEL WITH SAND, SILT, AND CLAY , MOIST.		675.1	14	4	12	94	SS-8A	-	-	-	-	-	-	-	-	-	20	A-4a (V)	-	
			15	5	3		SS-8B	-	-	-	-	-	-	-	-	-	16	A-2-6 (V)	-	
MEDIUM DENSE, ORANGE TO GRAYISH BROWN GRAVEL , SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, WET.		672.6	16	2																
LOOSE, BROWN SANDY SILT , LITTLE CLAY, TRACE FINE GRAVEL, MOIST.		671.6	17	3	18	78	SS-9	-	60	13	15	7	5	NP	NP	NP	15	A-1-a (0)	-	
			18	9																
STIFF, GRAYISH BROWN SILTY CLAY , LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.		667.6	19	2	8	75	SS-10A	-	-	-	-	-	-	-	-	-	17	A-4a (V)	-	
			20	3	2		SS-10B	1.50	0	3	15	46	36	36	20	16	25	A-6b (10)	-	
			21	2																
			22	5	27	72	SS-11	-	-	-	-	-	-	-	-	-	15	A-6b (V)	-	
			23	7	11															
SILTSTONE : LIGHT GRAY TO GRAY, HIGHLY WEATHERED, STRONG, VERY FINE GRAINED.		TR	24	3			SS-12A	-	-	-	-	-	-	-	-	-	13	Rock (V)	-	
			25	6	33	53	SS-12B	-	-	-	-	-	-	-	-	9	Rock (V)	-		
			26	16																
			27	11																
			28	18	75	92	SS-13	-	-	-	-	-	-	-	-	-	10	Rock (V)	-	
			29	32																
			30	60/3"	-	100	SS-14	-	-	-	-	-	-	-	-	-	13	Rock (V)	-	
			46		93	RC-1	-	-	-	-	-	-	-	-	-		CORE	-		

MATERIAL DESCRIPTION AND NOTES		ELEV. 660.6	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	SO4 ppm	BACK FILL
										GR	CS	FS	SI	CL	LL	PL	PI	WC			
SILTSTONE : LIGHT GRAY TO GRAY, HIGHLY WEATHERED, STRONG, VERY FINE GRAINED. <i>(continued)</i> -Qu @ 30.5' = 525 psi				31		70	93	RC-2										CORE			
				32																	
				33																	
				34																	
				35																	
		654.7				72	100	RC-3										CORE			
MUDSTONE : GRAY, UNWEATHERED, SLIGHTLY STRONG, VERY FINE GRAINED.		654.1		36																	
		653.5																			
SILTSTONE : GRAY, UNWEATHERED, SLIGHTLY STRONG, VERY FINE GRAINED.		652.8		37																	
		651.8		38																	
MUDSTONE : GRAY, UNWEATHERED, SLIGHTLY STRONG, VERY FINE GRAINED.		650.9		39																	

NOTES: SEEPAGE @ 14.5'. GROUNDWATER INITIALLY ENCOUNTERED @ 16.0'. GROUNDWATER WAS NOT ENCOUNTERED UPON COMPLETION. CAVE-IN DEPTH @ 5.0'.

Project Name: JEF-213-15.18		Location: Hammondsville, OH	Project No.: W-24-117
Photo No. 1			
Boring: B-001-0-25			
RC-1: 28.8'-29.7' REC (%): 93 RQD (%): 46			
RC-2: 29.7'-34.7' REC (%): 93 RQD (%): 70			

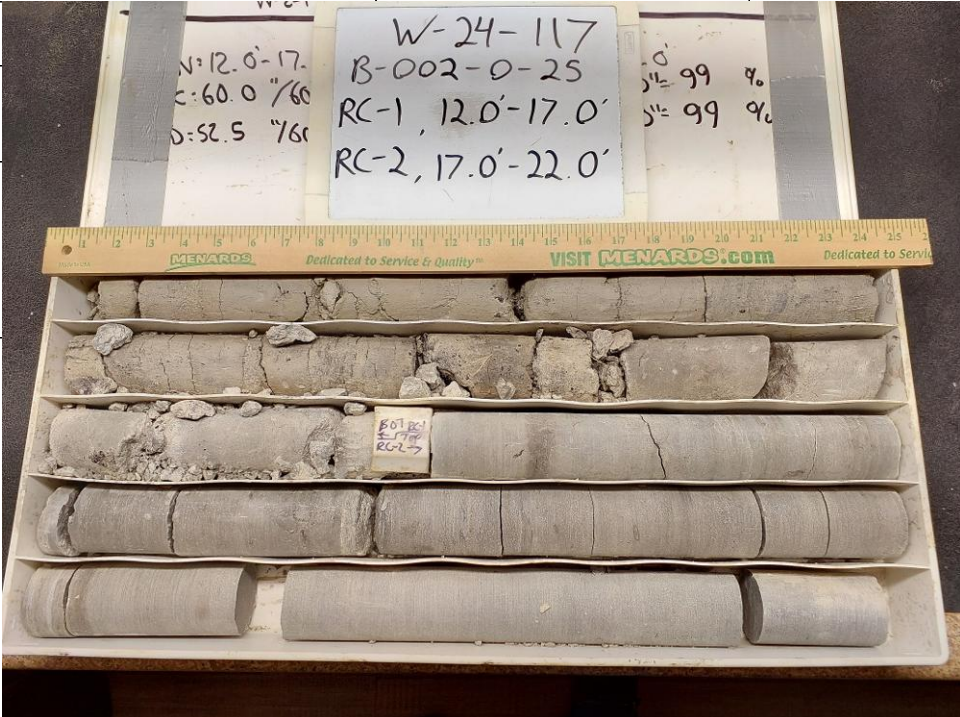
Project Name: JEF-213-15.18		Location: Hammondsville, OH	Project No.: W-24-117
Photo No. 2			
Boring: B-001-0-25			
RC-3: 34.7'-39.7' REC (%): 100 RQD (%): 72			

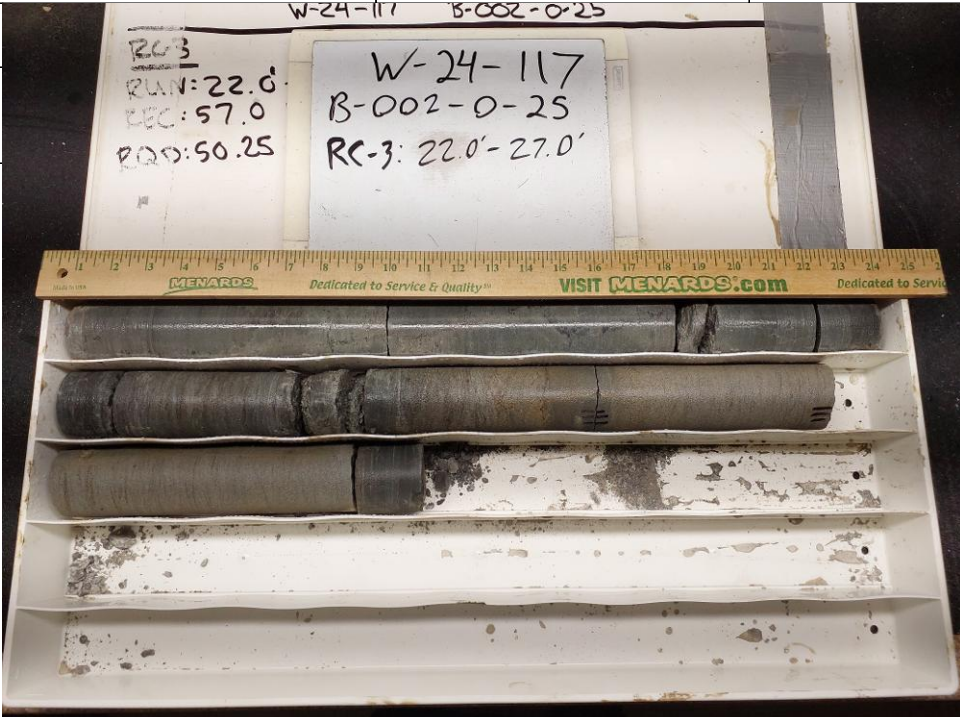
	PROJECT: JEF-213-15.18	DRILLING FIRM / OPERATOR: RII / IS	DRILL RIG: DIEDRICH D-50 (# 313)	STATION / OFFSET: 803+34 / 15' LT.	EXPLORATION ID B-002-0-25
	TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: RII / MJ	HAMMER: AUTOMATIC	ALIGNMENT: SR-213	
	PID: 86872 SFN: 4103181	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/25/24	ELEVATION: 678.7 ft. EOB: 27.0 ft.	PAGE 1 OF 1
	START: 7/10/25 END: 7/10/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 90	LAT / LONG: 40.557554, -80.705419	

MATERIAL DESCRIPTION AND NOTES	ELEV. 678.7	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	ABAN- DONED
								GR	CS	FS	SI	CL	LL	PL	PI				
NO SAMPLING FROM 0.0' TO 12.0'																			
		1																	
		2																	
		3																	
		4																	
		5																	
		6																	
		7																	
		8																	
		9																	
		10																	
		11																	
AUGER REFUSAL @ 12.0'	666.7	TR																	
MUDSTONE : LIGHT GRAY, SEVERELY WEATHERED, VERY WEAK, VERY FINE GRAINED.		12																	
		13																	
		14	65		97	RC-1											CORE		
	663.2	15																	
SILTSTONE : LIGHT GRAY, SLIGHTLY WEATHERED, SLIGHTLY STRONG TO STRONG, VERY FINE GRAINED.	661.7	16																	
MUDSTONE : LIGHT GRAY, UNWEATHERED, STRONG, VERY FINE GRAINED.		17																	
-Qu @ 17.0' = 4,338 psi		18																	
		19	65		98	RC-2											CORE		
		20																	
	656.7	21																	
SILTSTONE : DARK GRAY, SLIGHTLY WEATHERED, STRONG, VERY FINE GRAINED.		22																	
-Qu @ 23.3' = 1,170 psi	654.4	23																	
INTERBEDDED SANDSTONE AND SILTSTONE : LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, FINE GRAINED.	653.6	24	75		93	RC-3											CORE		
		25																	
SANDSTONE : LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, FINE GRAINED.	651.9	26																	
INTERBEDDED SANDSTONE AND SILTSTONE : LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, FINE GRAINED.	651.7	27																	
		EOB																	

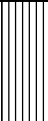
NOTES: GROUNDWATER WAS NOT ENCOUNTERED DURING DRILLING. BORING DRILLED FROM BRIDGE DECK. BRIDGE DECK 12.0' ABOVE STREAMBED ELEVATION.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: NATURAL COLLAPSE.

Project Name: JEF-213-15.18		Location: Hammondsville, OH	Project No.: W-24-117
Photo No. 3			
Boring: B-002-0-25			
RC-1: 12.0'-17.0' REC (%): 97 RQD (%): 65			
RC-2: 17.0'-22.0' REC (%): 98 RQD (%): 65			

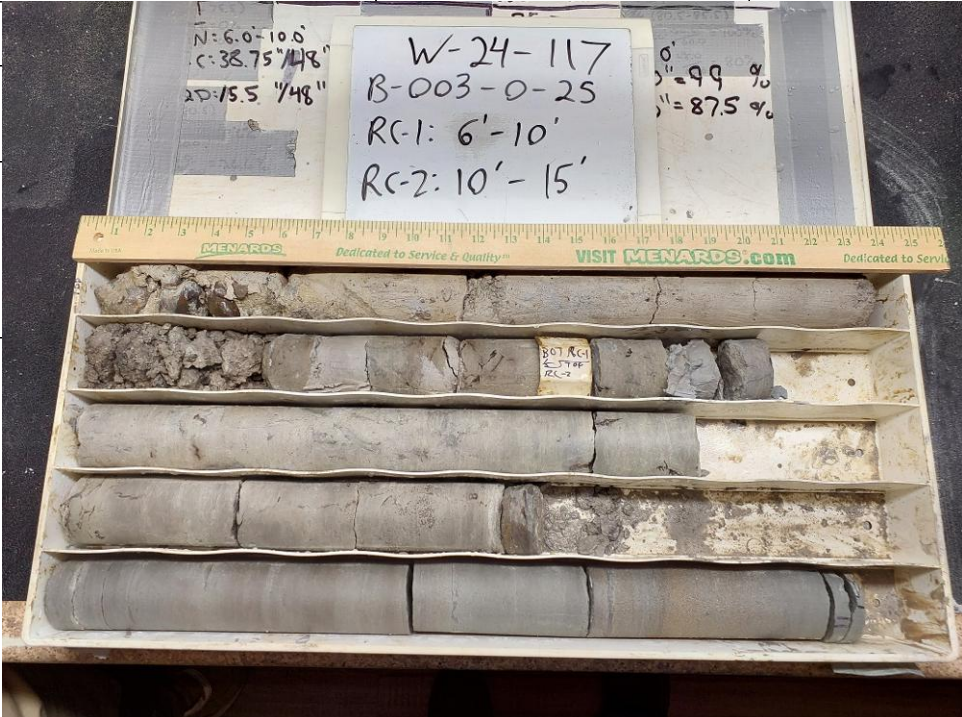
Project Name: JEF-213-15.18		Location: Hammondsville, OH	Project No.: W-24-117
Photo No. 4			
Boring: B-002-0-25			
 RC-3: 22.0'-27.0' REC (%): 93 RQD (%): 75			

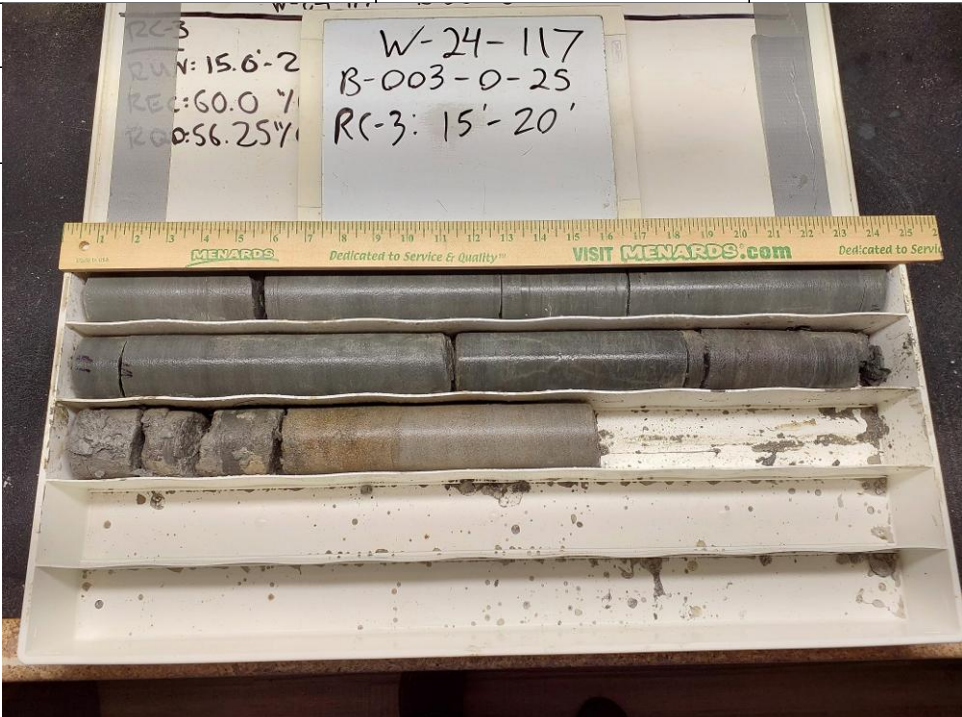
	PROJECT: JEF-213-15.18	DRILLING FIRM / OPERATOR: RII / IS	DRILL RIG: DIEDRICH D-50 (# 313)	STATION / OFFSET: 803+63 / 15' RT.	EXPLORATION ID B-003-0-25
	TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: RII / MJ	HAMMER: AUTOMATIC	ALIGNMENT: SR-213	
	PID: 86872 SFN: 4103181	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/25/24	ELEVATION: 673.5 ft. EOB: 20.0 ft.	PAGE 1 OF 1
	START: 7/10/25 END: 7/11/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 90	LAT / LONG: 40.557494, -80.705288	

MATERIAL DESCRIPTION AND NOTES	ELEV. 673.5	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	ABAN- DONED	
								GR	CS	FS	SI	CL	LL	PL	PI					
STIFF, GRAYISH BROWN SANDY SILT , SOME CLAY, TRACE FINE GRAVEL, MOIST TO WET.	670.5		2	12	72	SS-1	2.00	0	3	28	42	27	23	15	8	18	A-4a (7)	-		
			3																	5
			1																	
	669.0		2	15	67	SS-2	1.50	1	7	31	39	22	23	15	8	22	A-4a (5)	-		
			1																	8
			2																	
HARD, LIGHT GRAY TO LIGHT BROWN SILT AND CLAY , SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.	667.5		10	54	64	SS-3	-	1	2	16	57	24	25	14	11	10	A-6a (8)	-		
			14																	22
			4																	
VERY DENSE, LIGHT GRAY TO LIGHT BROWN GRAVEL WITH SAND AND SILT , LITTLE CLAY, MOIST.	664.3		12	81	36	SS-4	-	37	11	18	19	15	22	14	8	17	A-2-4 (0)	-		
			14																	40
			5																	
MUDSTONE : GRAY, SEVERELY WEATHERED, VERY WEAK, VERY FINE GRAINED.	664.3		6														CORE			
			7																	
			8																	
-Qu @ 6.0-10.0' = 188 psi (Point Load)	655.2		31		81	RC-1														
			9																	
			10																	
SILTSTONE : LIGHT GRAY TO DARK GRAY, SLIGHTLY WEATHERED, SLIGHTLY STRONG, VERY FINE GRAINED.	653.5		75		98	RC-2														
			11																	
			12																	
-Qu @ 10.0-15.0' = 2,363 psi (Point Load)	655.2		13																	
			14																	
			15																	
-Qu @ 17.3' = 1,812 psi	653.5		16																	
			17																	
			18																	
SANDSTONE : GRAY, SLIGHTLY WEATHERED, STRONG, FINE GRAINED.	653.5		93		100	RC-3														
			19																	
	653.5		20																	

NOTES: GROUNDWATER INITIALLY ENCOUNTERED @ 6.7'. GROUNDWATER WAS NOT ENCOUNTERED UPON COMPLETION. BROING DRILLED FROM BRIDGE DECK. BRIDGE DECK 17.3' ABOVE STREAMBED ELEVATION
 ABANDONMENT METHODS, MATERIALS, QUANTITIES: NATURAL COLLAPSE.

000-2025 RII ODOT L SULF (8.5X11) NO MSL - OH DOT.GDT - 1/27/26 08:34 - U:\GIB\PROJECTS\2024\W-24-117.GPJ

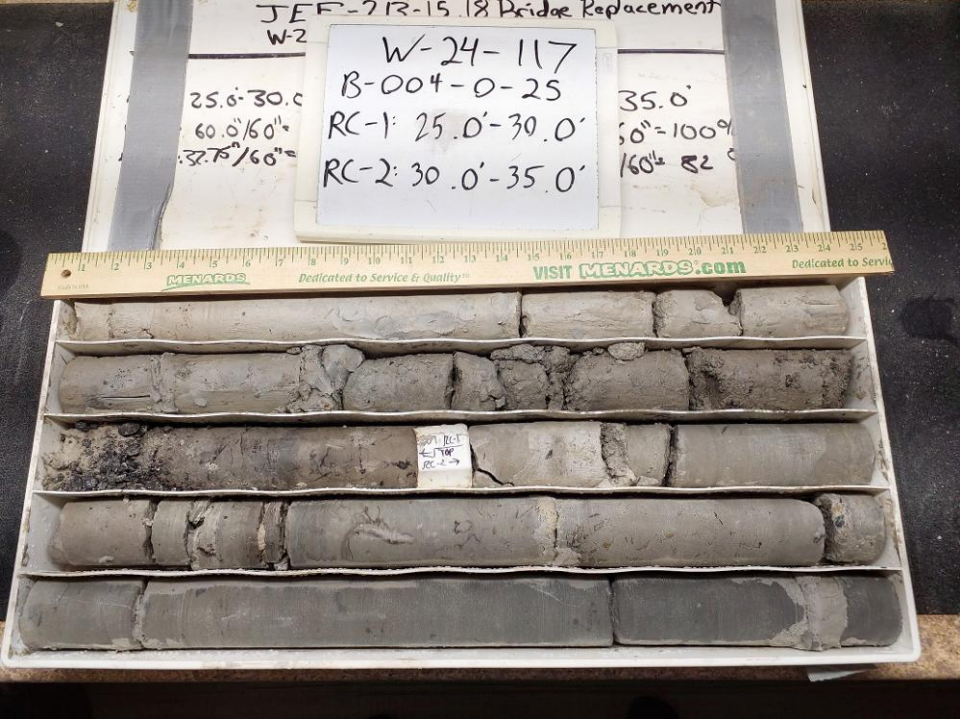
Project Name: JEF-213-15.18		Location: Hammondsville, OH	Project No.: W-24-117
Photo No. 5			
Boring: B-003-0-25			
RC-1: 6.0'-10.0' REC (%): 81 RQD (%): 31			
RC-2: 10.0'-15.0' REC (%): 98 RQD (%): 75			

Project Name: JEF-213-15.18		Location: Hammondsville, OH	Project No.: W-24-117
Photo No. 6			
Boring: B-003-0-25			
RC-3: 15.0'-20.0' REC (%): 100 RQD (%): 93			

	PROJECT: JEF-213-15.18		DRILLING FIRM / OPERATOR: RII / IS		DRILL RIG: DIEDRICH D-50 (# 313)		STATION / OFFSET: 804+43 / 7' LT.		EXPLORATION ID B-004-0-25												
	TYPE: BRIDGE REPLACEMENT		SAMPLING FIRM / LOGGER: RII / MJ		HAMMER: AUTOMATIC		ALIGNMENT: SR-213														
	PID: 86872 SFN: 4103181		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 3/25/24		ELEVATION: 691.0 ft. EOB: 35.0 ft.		PAGE 1 OF 2												
	START: 7/9/25 END: 7/9/25		SAMPLING METHOD: SPT		ENERGY RATIO (%): 90		LAT / LONG: 40.557609, -80.705033														
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	SO4 ppm	HOLE SEALED
			691.0							GR	CS	FS	SI	CL	LL	PL	PI	WC			
0.5' - ASPHALT (5.75")			690.5																		
1.1' - CONCRETE (13.5")			689.4	1																	
0.4' - AGGREGATE BASE (5.25")			689.0	2																	
MEDIUM DENSE, BROWNISH GRAY TO LIGHT BROWN TO DARK BROWN GRAVEL WITH SAND AND SILT, LITTLE CLAY, MOIST.				3	2	5	14	78	SS-1	-	29	14	23	24	10	25	16	9	17	A-2-4 (0)	-
				4	7	4															
				5	5	2	11	67	SS-2	-	-	-	-	-	-	-	-	15	A-2-4 (V)	-	
			685.5	6	4	2	18	83	SS-3A	-	42	17	16	17	8	25	16	9	15	A-2-4 (0)	-
MEDIUM DENSE, BROWNISH GRAY TO DARK GRAY GRAVEL WITH SAND, SILT, AND CLAY, DAMP.			684.5	7	7	5			SS-3B	-	45	15	11	20	9	28	17	11	13	A-2-6 (0)	-
				8	3	2	8	64	SS-4	-	38	19	19	16	8	24	14	10	14	A-2-4 (0)	-
LOOSE, BROWNISH GRAY GRAVEL WITH SAND AND SILT, TRACE CLAY, DAMP.			681.5	9	2	2	9	47	SS-5	-	50	13	17	13	7	23	15	8	11	A-2-4 (0)	-
				10	3	4	11	92	SS-6	-	-	-	-	-	-	-	-	-	15	A-1-b (V)	-
MEDIUM DENSE, LIGHT BROWN TO DARK BROWN GRAVEL WITH SAND, LITTLE SILT, TRACE CLAY, DAMP.				11	5	3															
			678.0	12	5	5	15	75	SS-7	-	46	15	21	14	4	NP	NP	NP	10	A-1-b (0)	-
				13																	
LOOSE TO MEDIUM DENSE, LIGHT BROWN TO DARK BROWN GRAVEL, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, WET.				14	5	7	23	89	SS-8	-	-	-	-	-	-	-	-	-	12	A-1-a (V)	-
				15		8															
				16	4																
-SS-9: LOI = 3.8%				17	2	1	5	78	SS-9	-	-	-	-	-	-	-	-	-	24	A-1-a (V)	-
				18																	
				19	4																
				20		5	14	56	SS-10	-	58	12	18	9	3	NP	NP	NP	17	A-1-a (0)	-
				21																	
				22	3	4	18	0	SS-11	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	-
			668.2	23	9		-	100	2S-11A	-	-	-	-	-	-	-	-	-	20	A-1-a (V)	-
MUDSTONE : LIGHT GRAY TO DARK GRAY, HIGHLY WEATHERED, VERY WEAK, VERY FINE GRAINED.				24	14	22	35	86	58	SS-12	-	-	-	-	-	-	-	-	11	Rock (V)	-
				25																	
				26																	
				27																	
				28	55			100	RC-1											CORE	
			661.0	29																	

000-2025 RII ODOT L SULF (8.5X11) NO MSL - OH DOT.GDT - 1/27/26 08:34 - U:\GIS\PROJECTS\2024\W-24-117.GPJ

PID: 86872	SFN: 4103181	PROJECT: JEF-213-15.18	STATION / OFFSET: 80443, 7' LT.					START: 7/9/25	END: 7/9/25	PG 2 OF 2	B-004-0-25										
MATERIAL DESCRIPTION AND NOTES		ELEV. 661.0	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	SO4 ppm	HOLE SEALED
SILTSTONE : GRAY TO LIGHT GRAY, SLIGHTLY WEATHERED, SLIGHTLY STRONG, VERY FINE GRAINED. -Qu @ 33.3' = 1,695 psi					78		100	RC-2										CORE			
			31																		
			32																		
			33																		
			34																		
	656.0	EOB	35																		
NOTES: SEEPAGE @ 16.5'. GROUNDWATER INITIALLY ENCOUNTERED @ 18.5'. GROUNDWATER UPON COMPLETION @19.0'. CAVE-IN DEPTH @ 5.0'. ABANDONMENT METHODS, MATERIALS, QUANTITIES: GROUTED WITH A MIXTURE OF CEMENT AND BENTONITE POWDER. PAVEMENT PATCHED WITH CONCRETE.																					

Project Name: JEF-213-15.18	Location: Hammondsville, OH	Project No.: W-24-117
Photo No. 7		
Boring: B-004-0-25		
RC-1: 25.0'-30.0' REC (%): 100 RQD (%): 55		
RC-2: 30.0'-35.0' REC (%): 100 RQD (%): 78		

BORING LOGS

Definitions of Abbreviations

AS	=	Auger sample
GI	=	Group index as determined from the Ohio Department of Transportation classification system
HP	=	Unconfined compressive strength as determined by a hand penetrometer (tons per square foot)
LL _o	=	Oven-dried liquid limit as determined by ASTM D4318. Per ASTM D2487, if LL _o /LL is less than 75 percent, soil is classified as "organic".
LOI	=	Percent organic content (by weight) as determined by ASTM D2974 (loss on ignition test)
PID	=	Photo-ionization detector reading (parts per million)
QR	=	Unconfined compressive strength of intact rock core sample as determined by ASTM D2938 (pounds per square inch)
QU	=	Unconfined compressive strength of soil sample as determined by ASTM D2166 (pounds per square foot)
RC	=	Rock core sample
REC	=	Ratio of total length of recovered soil or rock to the total sample length, expressed as a percentage
RQD	=	Rock quality designation – estimate of the degree of jointing or fracture in a rock mass, expressed as a percentage:

$$\frac{\sum \text{segments equal to or longer than 4.0 inches}}{\text{core run length}} \times 100$$

SO ₄	=	Sulfate content (parts per million)
SPT	=	Standard penetration test blow counts, per ASTM D1586. Driving resistance recorded in terms of blows per 6-inch interval while letting a 140-pound hammer free fall 30 inches to drive a 2-inch outer diameter (O.D.) split spoon sampler a total of 18 inches. The second and third intervals are added to obtain the number of blows per foot (N _m).
N ₆₀	=	Measured blow counts corrected to an equivalent (60 percent) energy ratio (ER) by the following equation: N ₆₀ = N _m *(ER/60)
SS	=	Split spoon sample
2S	=	For instances of no recovery from standard SS interval, a 2.5 inch O.D. split spoon is driven the full length of the standard SS interval plus an additional 6.0 inches to obtain a representative sample. Only the final 6.0 inches of sample is retained. Blow counts from 2S sampling are not correlated with N ₆₀ values.
3S	=	Same as 2S, but using a 3.0 inch O.D. split spoon sampler.
TR	=	Top of rock
W	=	Initial water level measured during drilling
▽	=	Water level measured at completion of drilling

Classification Test Data

Gradation (as defined on Description of Soil Terms):

GR	=	% Gravel
SA	=	% Sand
SI	=	% Silt
CL	=	% Clay

Atterberg Limits:

LL	=	Liquid limit
PL	=	Plastic limit
PI	=	Plasticity Index
WC	=	Water content (%)

B-001-0-63

State of Ohio
Department of Highways
Testing Laboratory

LOG OF BORING

Date Started 1-16-63 Sampler Type SS Dia. 1 3/8" Water Elev. 676.8
 Date Completed 1-17-63 Casing Length 15' Dia. 3 1/2"

Project Identification JEFFERSON
JEF-213-1537
JEF-213-1537
OVER NORTH FORK OF YELLOW CREEK

Boring No. B-1 Station & Offset 811+50, 42' It. (REAR ABUTMENT) Surface Elev. 681.8

Elev.	Depth	Std. Pen.	Rec.	Loss	Description	Field No.	Lab. Nos.	Physical Characteristics										SHTL Class
								% Agg.	% C.S.	% F.S.	% Silt	% Clay	LL	PI	W.C.			
681.8	0																	
	2																	
	4																	
676.8	6	5/7			Brown Sandy Silt with Decayed Wood	1	2340	0	10	29	21	40	NP	NP		39		
673.8	8				Sandstone cobbles													
671.8	10	4/8			Brown and Gray Silty Sand with Stone Fragments, Trace Decayed Wood.	2	2341	32	4	35	7	22	NP	NP		25		
669.3	12	5/9			Brown and Gray Silty Sand with Stone Frag.	3	2342	24	11	40	3	22	NP	NP		26		
666.8	14				Brown and Gray Sandy Clay	4	2343	14	8	8	22	48	31	11		9		
662.1	16	50.0 (2.7)			TOP OF ROCK													
	18		2.3	2.2	Indurated clay, gray, siliceous from 19.5' to 20.2', medium-firm, crumbly. Core loss 23%.													
	20																	
659.8	22																	
	24		4.7	0.3	Shale, gray, extremely siliceous, fissile, firm. No core loss.													
	26																	
653.8	28		5.0	0.0														
651.8	30				Sandstone, gray, very fine-grained, argillaceous, firm; grading intermittently into a **													
	32				BOTTOM OF BORING													
	34				*REFUSAL													
	36				** gray siliceous, firm shale (comprising about 17% of the interval). No core loss.													

Particle Size: Agg. >2.00mm, Coarse Sand=2.00-0.42mm, Fine Sand=0.42-0.074mm, Silt=0.074-0.005mm, Clay= <0.005mm

Particle Size: Agg* >2.00mm, Coarse Sand=2.00-0.42mm, Fine Sand=0.42-0.074mm, Silt=0.074-0.005mm, Clay* <0.005mm

APPENDIX IV

LABORATORY TEST RESULTS



RESOURCE INTERNATIONAL, INC.
Engineering Consultants

6350 Presidential Gateway.	9885 Rockside Road	4480 Lake Forest Drive
Columbus, OH 43231	Cleveland, OH 44125	Cincinnati, Ohio 45242
Phone (614) 823-4949	Phone (216) 573-0955	Phone (513) 769-6998

**Uniaxial Compressive Strength
of Intact Rock Core Specimens**
(ASTM D 7012-14)

Project: JEF-213-15.18
Project No.: W-24-117
Date of Testing: 7/28/2025
Test Performed by: KL/EM

Rock Description: _____
Rock Formation: _____

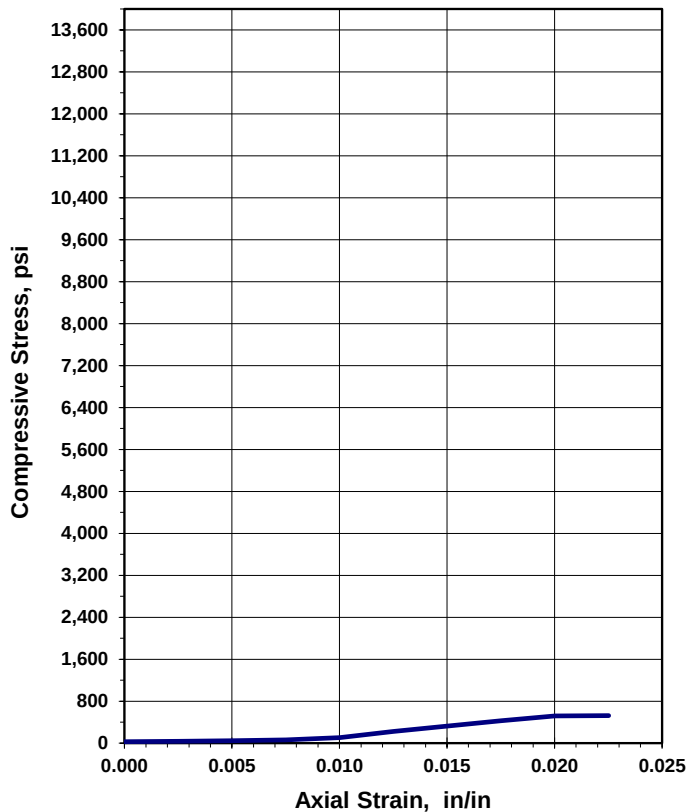
Boring No.: B-001-0-25
Sample No.: RC-2
Depth (ft): 30.5' feet
Moisture condition: As received
Sample Mass: 516.97 grams
Testing Temperature: 23 °C
Rate of Loading: 8.0 lbs/sec
Testing Time: 199 sec
(Rate 2-15 min)

Average Length: 3.997 in
Average Diameter: 1.968 in
Length to diameter ratio: 2.031
Cross Sectional Area: 3.042 in²
Volume: 0.0070 ft³
Unit Weight (sample specimen)*: 161.98 lbs/ft³
Failure Load: 1,597 lbs
Axial Strain at Failure: 0.0225 in/in
Compressive Strength: 525 psi

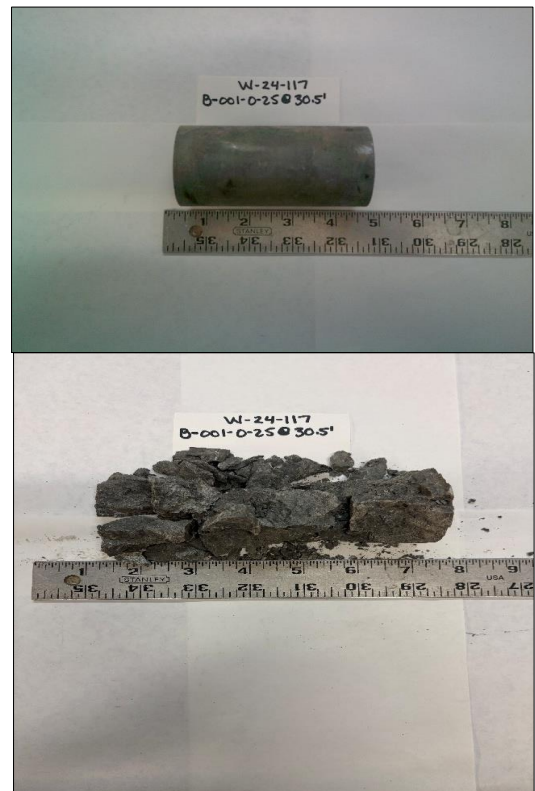
Sample Preparation: Per ASTM D4543

*Actual test sample used for unit weight prior to testing.

Unconfined Compression Test



Before Testing



REMARKS: _____



RESOURCE INTERNATIONAL, INC.

Engineering Consultants

6350 Presidential Gateway. 9885 Rockside Road 4480 Lake Forest Drive
Columbus, OH 43231 Cleveland, OH 44125 Cincinnati, Ohio 45242
Phone (614) 823-4949 Phone (216) 573-0955 Phone (513) 769-6998

Uniaxial Compressive Strength of Intact Rock Core Specimens

(ASTM D 7012-14)

Project: JEF-213-15.18

Project No.: W-24-117

Date of Testing: 7/28/2025

Test Performed by: KL/EM

Rock Description: _____
Rock Formation: _____

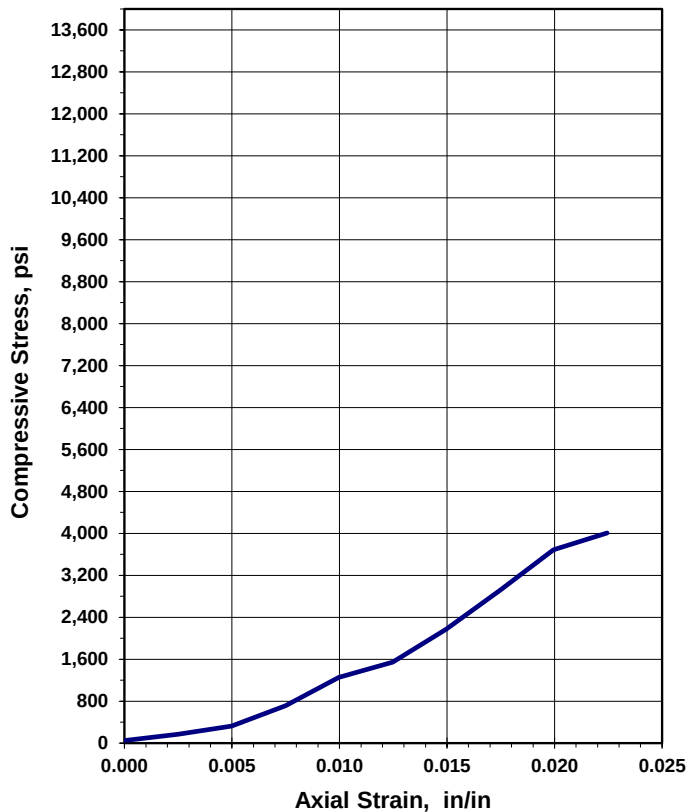
Boring No.: B-001-0-25
Sample No: RC-3
Depth (ft): 38.7' feet
Moisture condition: As received
Sample Mass: 588.75 grams
Testing Temperature: 23 °C
Rate of Loading: 112.1 lbs/sec
Testing Time: 112 sec
(Rate 2-15 min)

Average Length: 4.010 in
Average Diameter: 1.997 in
Length to diameter ratio: 2.008
Cross Sectional Area: 3.132 in²
Volume: 0.0073 ft³
Unit Weight (sample specimen)*: 178.57 lbs/ft³
Failure Load: 12,553 lbs
Axial Strain at Failure: 0.0224 in/in
Compressive Strength: 4,008 psi

Sample Preparation: Per ASTM D4543

*Actual test sample used for unit weight prior to testing.

Unconfined Compression Test



Before Testing



After Failure



REMARKS: _____



RESOURCE INTERNATIONAL, INC.
Engineering Consultants

6350 Presidential Gateway.	9885 Rockside Road	4480 Lake Forest Drive
Columbus, OH 43231	Cleveland, OH 44125	Cincinnati, Ohio 45242
Phone (614) 823-4949	Phone (216) 573-0955	Phone (513) 769-6998

**Uniaxial Compressive Strength
of Intact Rock Core Specimens**
(ASTM D 7012-14)

Project: JEF-213-15.18
Project No.: W-24-117
Date of Testing: 7/28/2025
Test Performed by: KL/EM

Rock Description: _____
Rock Formation: _____

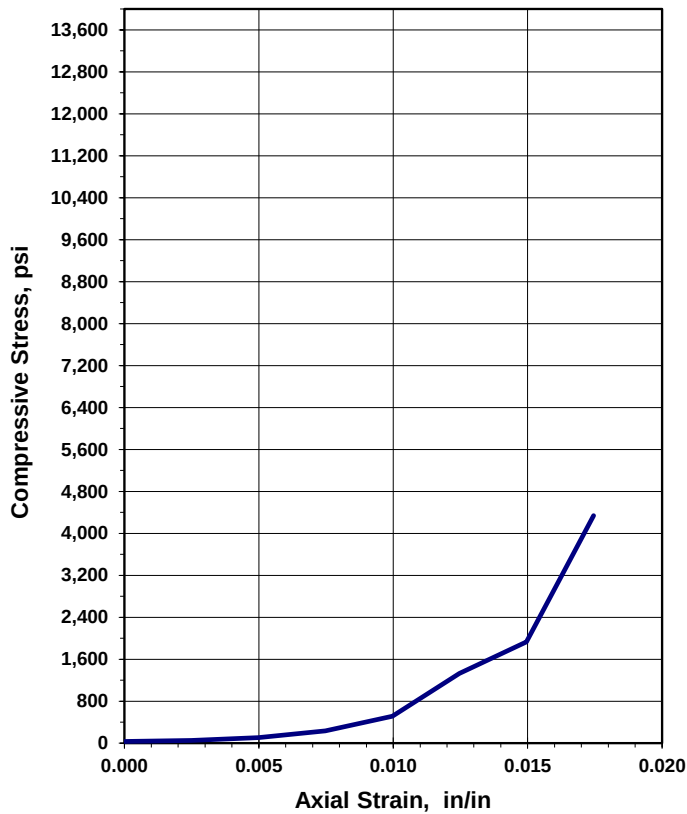
Boring No.: B-002-0-25
Sample No.: RC-2
Depth (ft): 17.0' feet
Moisture condition: As received
Sample Mass: 526.86 grams
Testing Temperature: 23 °C
Rate of Loading: 125.6 lbs/sec
Testing Time: 106 sec
(Rate 2-15 min)

Average Length: 4.011 in
Average Diameter: 1.977 in
Length to diameter ratio: 2.029
Cross Sectional Area: 3.070 in²
Volume: 0.0071 ft³
Unit Weight (sample specimen)*: 163.01 lbs/ft³
Failure Load: 13,317 lbs
Axial Strain at Failure: 0.0175 in/in
Compressive Strength: 4,338 psi

Sample Preparation: Per ASTM D4543

*Actual test sample used for unit weight prior to testing.

Unconfined Compression Test



Before Testing



After Failure



REMARKS: _____



RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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Phone (614) 823-4949	Phone (216) 573-0955	Phone (513) 769-6998

**Uniaxial Compressive Strength
of Intact Rock Core Specimens**
(ASTM D 7012-14)

Project: JEF-213-15.18
Project No.: W-24-117
Date of Testing: 7/28/2025
Test Performed by: KL/EM

Rock Description: _____
Rock Formation: _____

Boring No.: B-002-0-25
Sample No.: RC-3
Depth (ft): 23.3' feet
Moisture condition: As received
Sample Mass: 563.87 grams
Testing Temperature: 23 °C
Rate of Loading: 0.0 lbs/sec
Testing Time: 4.01 sec
(Rate 2-15 min)

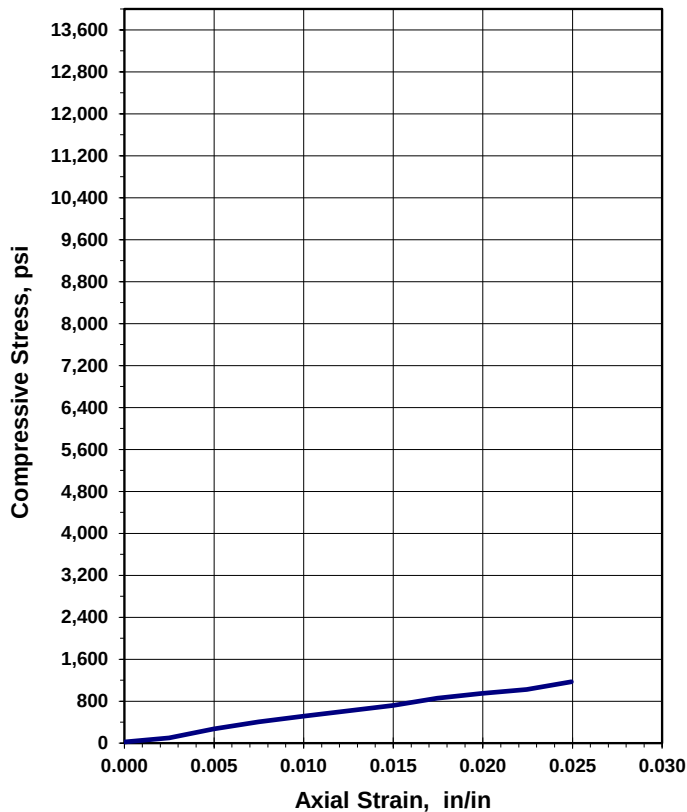
Average Length: 4.010 in
Average Diameter: 1.987 in
Length to diameter ratio: 2.018
Cross Sectional Area: 3.101 in²
Volume: 0.0072 ft³
Unit Weight (sample specimen)*: 172.75 lbs/ft³
Failure Load: 3,629 lbs
Axial Strain at Failure: 0.0249 in/in
Compressive Strength: 1,170 psi

Sample Preparation: Per ASTM D4543

*Actual test sample used for unit weight prior to testing.

Unconfined Compression Test

Before Testing



After Failure



REMARKS: _____



RESOURCE INTERNATIONAL, INC.
Engineering Consultants

**Point Load Strength Index
of Rock Specimens
(ASTM D 5731-08)**

6350 Presidential Gateway.
Columbus, OH 43231
Phone (614) 823-4949

9885 Rockside Road
Cleveland, OH 44125
Phone (216) 573-0955

4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

Project: JEF-231-15.18
Project No.: W-24-117
Date of Testing: 7/28/2025
Test Performed by: EM/KL

Rock Description: Mudstone

Boring No.: B-003-0-25
Station / Offset: 803+63 / 15' RT.
Sample No. / Depth: RC-1 / 6.0'-10.0'

Test Apparatus: Forney-LA 0080
Serial Number: A125/AZ/0014
Date of Calibration: 4/9/2025

Sample No.	Test Type	Depth (ft)	W (mm)	D (mm)	Load (N)	D_e^2 (mm ²)	D_e (mm)	F	Is (MPa)	Is ₍₅₀₎ (MPa)	σ_c (MPa)
1	a $\perp$	4.5-7.0	51.4	28.4	129	1,857	43.1	0.94	0.07	0.06	1.67
2	a $\perp$	4.5-7.0	51.1	29.9	94	1,944	44.1	0.94	0.05	0.05	1.16
3	a $\perp$	4.5-7.0	51.0	26.5	62	1,724	41.5	0.92	0.04	0.03	0.86
4	a $\perp$	4.5-7.0	51.1	22.4	85	1,454	38.1	0.89	0.06	0.05	1.40
5	a $\perp$	4.5-7.0	51.1	26.3	98	1,710	41.4	0.92	0.06	0.05	1.38

STATISTICS

Mean Is ₍₅₀₎ $\perp$	0.05 MPa (7 psi)
Mean Is ₍₅₀₎ $\parallel$	
la ₍₅₀₎	

Specific Specimen Shape:

d = diametrical
a = axial
b = block
i = irregular lump
 $\perp$ = perpendicular to bedding plane
 $\parallel$ = parallel to bedding plane

Estimated Uniaxial Compression, $\sigma_c = K \cdot Is$

K = 24
*Per Section 206.1.3 of 2011 ODOT
Rock Slope Design Guide

Mean σ_c = **1.29 MPa (0,188 psi)**

Remarks: _____ 1



RESOURCE INTERNATIONAL, INC.
Engineering Consultants

**Point Load Strength Index
of Rock Specimens
(ASTM D 5731-08)**

6350 Presidential Gateway.
Columbus, OH 43231
Phone (614) 823-4949

9885 Rockside Road
Cleveland, OH 44125
Phone (216) 573-0955

4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

Project: JEF-213-15.18
Project No.: W-24-117
Date of Testing: 7/28/2025
Test Performed by: EM/KL

Rock Description: Siltstone

Boring No.: B-003-0-25
Station / Offset: 803+63 / 15' RT
Sample No. / Depth: RC-2 / 10.0'-15.0'

Test Apparatus: Forney-LA 0080
Serial Number: A125/AZ/0014
Date of Calibration: 4/9/2025

Sample No.	Test Type	Depth (ft)	W (mm)	D (mm)	Load (N)	D_e^2 (mm ²)	D_e (mm)	F	Is (MPa)	Is ₍₅₀₎ (MPa)	σ_c (MPa)
1	a $\perp$	4.5-7.0	50.6	30.1	1,099	1,940	44.1	0.94	0.57	0.53	13.59
2	a $\perp$	4.5-7.0	50.6	26.2	841	1,686	41.1	0.92	0.50	0.46	11.97
3	a $\perp$	4.5-7.0	50.6	26.1	1,166	1,682	41.0	0.91	0.69	0.63	16.64
4	a $\perp$	4.5-7.0	50.6	21.6	966	1,392	37.3	0.88	0.69	0.61	16.65
5	a $\perp$	4.5-7.0	50.5	21.6	1,308	1,390	37.3	0.88	0.94	0.82	22.59

STATISTICS

Mean Is ₍₅₀₎ $\perp$	0.61 MPa (89 psi)
Mean Is ₍₅₀₎ $\parallel$	
la ₍₅₀₎	

Specific Specimen Shape:

d = diametrical
a = axial
b = block
i = irregular lump
 $\perp$ = perpendicular to bedding plane
 $\parallel$ = parallel to bedding plane

Estimated Uniaxial Compression, $\sigma_c = K \cdot Is$

K = 24
*Per Section 206.1.3 of 2011 ODOT
Rock Slope Design Guide

Mean σ_c = 16.29 MPa (2,363 psi)

Remarks: _____ 1



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**Uniaxial Compressive Strength
of Intact Rock Core Specimens**
(ASTM D 7012-14)

Project: JEF-213-15.18
Project No.: W-24-117
Date of Testing: 7/28/2025
Test Performed by: KL/EM

Rock Description: _____
Rock Formation: _____

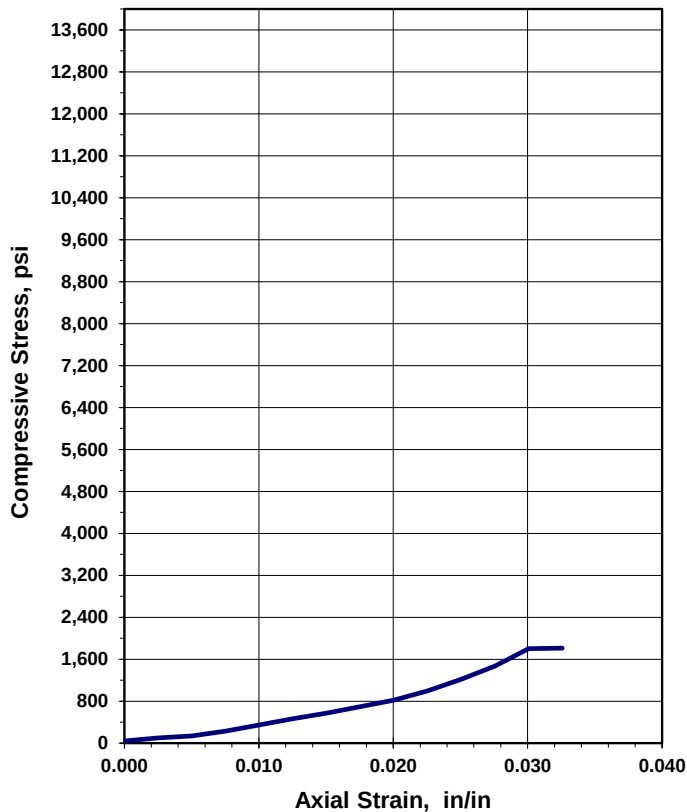
Boring No.: B-003-0-25
Sample No.: RC-3
Depth (ft): 17.3' feet
Moisture condition: As received
Sample Mass: 534.14 grams
Testing Temperature: 23 °C
Rate of Loading: 28.2 lbs/sec
Testing Time: 199 sec
(Rate 2-15 min)

Average Length: 3.991 in
Average Diameter: 1.986 in
Length to diameter ratio: 2.010
Cross Sectional Area: 3.098 in²
Volume: 0.0072 ft³
Unit Weight (sample specimen)*: 164.59 lbs/ft³
Failure Load: 5,612 lbs
Axial Strain at Failure: 0.0326 in/in
Compressive Strength: 1,812 psi

Sample Preparation: Per ASTM D4543

*Actual test sample used for unit weight prior to testing.

Unconfined Compression Test



Before Testing



After Failure



REMARKS: _____



RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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Columbus, OH 43231	Cleveland, OH 44125	Cincinnati, Ohio 45242
Phone (614) 823-4949	Phone (216) 573-0955	Phone (513) 769-6998

**Uniaxial Compressive Strength
of Intact Rock Core Specimens**
(ASTM D 7012-14)

Project: JEF-213-15.18
Project No.: W-24-117
Date of Testing: 7/28/2025
Test Performed by: KL/EM

Rock Description: _____
Rock Formation: _____

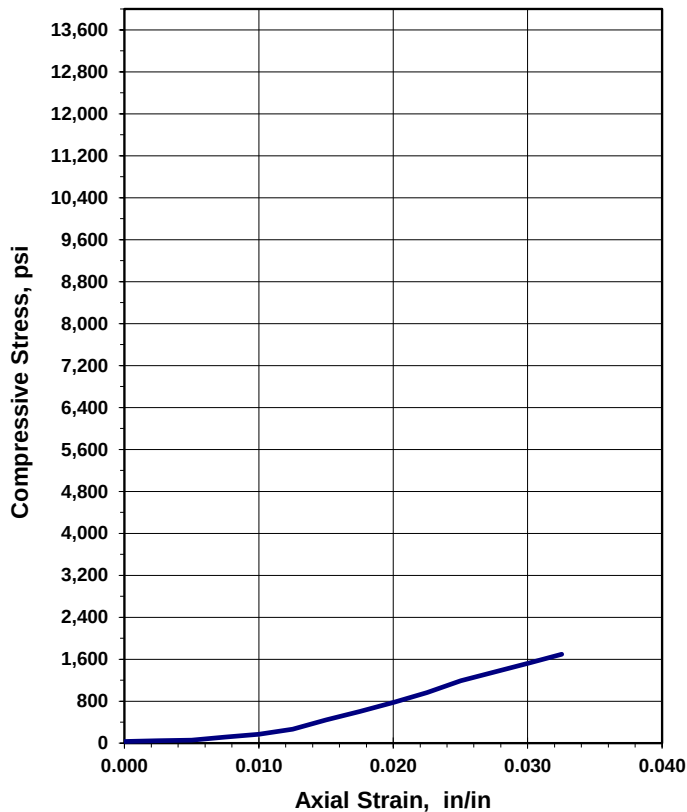
Boring No.: B-004-0-25
Sample No.: RC-2
Depth (ft): 33.3' feet
Moisture condition: As received
Sample Mass: 534.40 grams
Testing Temperature: 23 °C
Rate of Loading: 27.8 lbs/sec
Testing Time: 188 sec
(Rate 2-15 min)

Average Length: 3.996 in
Average Diameter: 1.982 in
Length to diameter ratio: 2.016
Cross Sectional Area: 3.085 in²
Volume: 0.0071 ft³
Unit Weight (sample specimen)*: 165.13 lbs/ft³
Failure Load: 5,230 lbs
Axial Strain at Failure: 0.0325 in/in
Compressive Strength: 1,695 psi

Sample Preparation: Per ASTM D4543

*Actual test sample used for unit weight prior to testing.

Unconfined Compression Test



Before Testing



After Failure



REMARKS: _____

APPENDIX V

LATERAL DESIGN PARAMETERS

Lateral Design Parameters
JEF-213-15.18
Rii Project No. W-24-117

Boring	Elevation (feet, NAVD 88)	Soil Class.	Soil Type	Strata	N ₆₀	γ (pcf)	γ' (pcf)	Strength Parameter	k (soil) k _{rm} (rock)	ε ₅₀ (soil) E _r (rock)	RQD (rock)
B-001-0-25	690.6 to 686.6	A-2-4	G	4	18	125	125	φ = 30°	35 pci	-	-
	686.6 to 685.1	A-2-4	G	4	27	130	130	φ = 32°	65 pci	-	-
	685.1 to 683.6	A-2-4	G	4	11	120	120	φ = 29°	25 pci	-	-
	683.6 to 682.1	A-1-b	G	4	11	120	120	φ = 29°	25 pci	-	-
	682.1 to 676.3	A-4a	G	4	9	120	57.6	φ = 27°	20 pci	-	-
	676.3 to 675.1	A-2-6	G	4	12	125	62.6	φ = 28°	20 pci	-	-
	675.1 to 672.6	A-1-a	G	4	18	125	62.6	φ = 31°	30 pci	-	-
	672.6 to 671.6	A-4a	G	4	8	120	57.6	φ = 26°	15 pci	-	-
	671.6 to 670.1	A-6b	C	2	8	115	52.6	Su = 1,000 psf	235 pci	0.0090	-
	670.1 to 667.6	A-6b	C	2	27	125	62.6	Su = 3,375 psf	1,125 pci	0.0049	-
	667.6 to 655.8	Rock	R	9	-	160	97.6	Qu = 525 psi	0.0005	47,000 psi	70
	655.8 to 650.9	Rock	R	9	-	150	87.6	Qu = 4,000 psi	0.00005	357,000 psi	72
B-002-0-25	678.7 to 661.7	Rock	R	9	-	150	87.6	Qu = 188 psi	0.0005	18,000 psi	65
	661.7 to 656.7	Rock	R	9	-	150	87.6	Qu = 4,338 psi	0.00005	388,000 psi	65
	656.7 to 651.7	Rock	R	9	-	160	97.6	Qu = 1,170 psi	0.0005	105,000 psi	75
B-003-0-25	673.5 to 670.5	A-4a	G	4	13	125	62.6	φ = 27°	25 pci	-	-
	670.5 to 669.0	A-6a	C	2	54	130	67.6	Su = 6,750 psf	2,250 pci	0.0038	-
	669.0 to 667.5	A-2-4	G	4	81	135	72.6	φ = 37°	225 pci	-	-
	667.5 to 663.5	Rock	R	9	-	150	87.6	Qu = 188 psi	0.0005	18,000 psi	31
	663.5 to 658.5	Rock	R	9	-	160	97.6	Qu = 2,363 psi	0.00005	210,000 psi	75
	658.5 to 653.5	Rock	R	9	-	160	97.6	Qu = 1,812 psi	0.0005	105,000 psi	93
B-004-0-25	691.0 to 685.5	A-2-4	G	4	12	125	125	φ = 29°	25 pci	-	-
	685.5 to 684.5	A-2-6	G	4	18	125	125	φ = 29°	35 pci	-	-
	684.5 to 681.5	A-2-4	G	4	8	120	57.6	φ = 28°	20 pci	-	-
	681.5 to 678.0	A-1-b	G	4	13	125	62.6	φ = 29°	20 pci	-	-
	678.0 to 675.5	A-1-a	G	4	23	125	62.6	φ = 33°	50 pci	-	-
	675.5 to 673.0	A-1-a	G	4	5	120	57.6	φ = 29°	15 pci	-	-
	673.0 to 668.2	A-1-a	G	4	16	125	62.6	φ = 31°	30 pci	-	-
	668.2 to 661.0	Rock	R	9	-	150	87.6	Qu = 188 psi	0.0005	18,000 psi	55
	661.0 to 656.0	Rock	R	9	-	160	97.6	Qu = 1,695 psi	0.0005	155,000 psi	78

APPENDIX VI

PAVEMENT CORE DATA SHEETS



6350 Presidential Gateway
Columbus, Ohio 43231
Telephone: (614) 823-4949
Fax Number: (614) 823-4990

Pavement Core Data Summary

PROJECT JEF-213-15.18 Bridge Replacement
LOCATION Hammondsville, Jefferson County, Ohio
JOB No. W-24-117

BORING/CORE No. B-001-0-25
DATE CORE OBTAINED 7/8/2025
CORE OBTAINED BY IS, JR, BH

Core Composition

Comments/Remarks

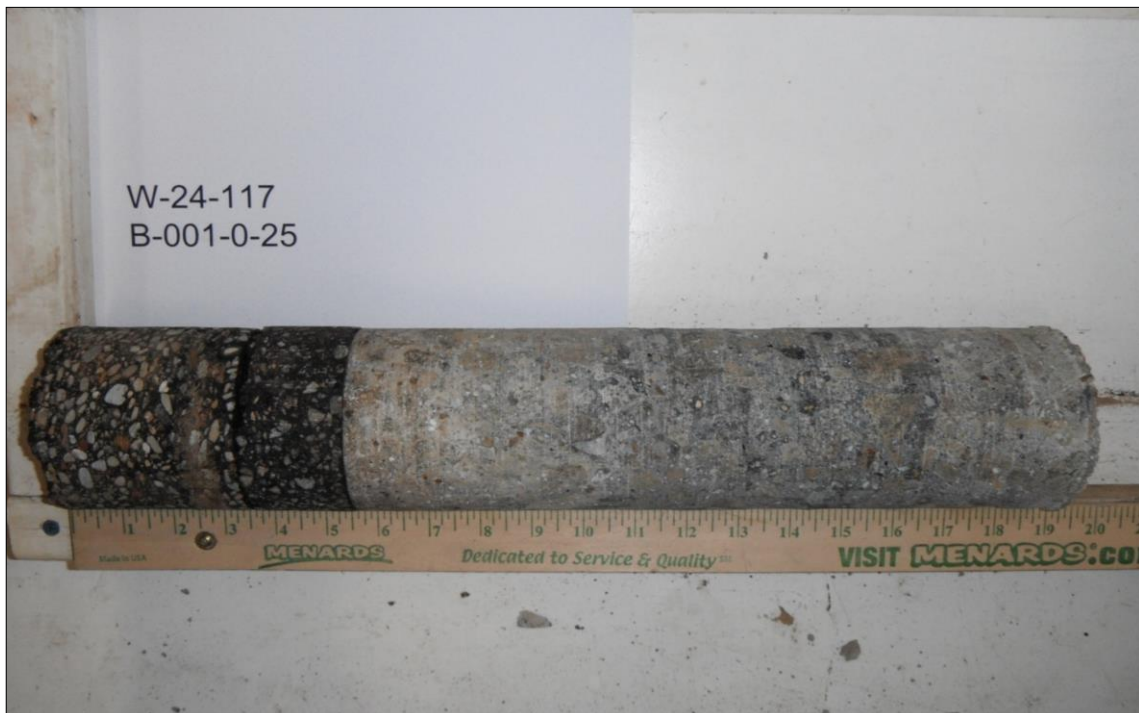
Core Number	Layer Thickness (in.)	Pavement Layer Number	Asphalt			Concrete	Aggregate/Granular Base	Other		
			Surface Binder	Intermediate Binder	Base Binder					
B-001-0-25	0.50	5	✓							
	1.50	4		✓						
	1.75	3		✓						
	2.00	2		✓						
	13.25	1				✓				
	8.00						✓			

- Layer 1 has slag and voids.
- Layer 2 has slag.
- Layers 3&5 have slight deterioration.
- Layer 4 has trace voids.
- Layers 1&2 and 2&3 are delaminated.



- Aggregate Base: ODOT A-1-a (visual)

Total Pavement Thickness = 19.00 in. Total Asphalt Thickness = 5.75 in. Total Concrete Thickness = 13.25 in. Total Base Thickness = 8.00 in.





6350 Presidential Gateway
Columbus, Ohio 43231
Telephone: (614) 823-4949
Fax Number: (614) 823-4990

Pavement Core Data Summary

PROJECT JEF-213-15.18 Bridge Replacement
LOCATION Hammondsville, Jefferson County, Ohio
JOB No. W-24-117

BORING/CORE No. B-004-0-25
DATE CORE OBTAINED 7/9/2025
CORE OBTAINED BY IS, JR, BH

Core Composition

Comments/Remarks

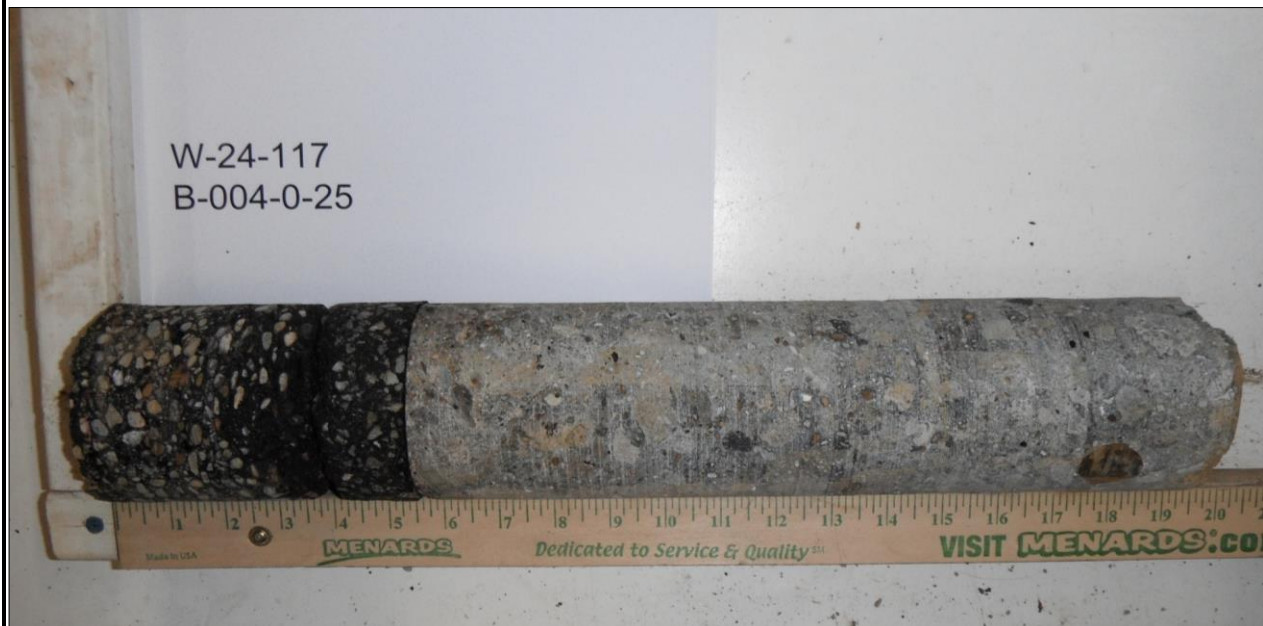
Core Number	Layer Thickness (in.)	Pavement Layer Number	Asphalt			Concrete	Aggregate/Granular Base	Other		
			Surface Binder	Intermediate Binder	Base Binder					
B-004-0-25	0.50	5	✓							
	2.00	4		✓						
	1.50	3		✓						
	1.75	2		✓						
	13.50	1				✓				
	5.25						✓			

- Layer 1 has slag, voids, and a 1" metal rod @ 11".
- Layer 2 has slag and slight deterioration.
- Layer 3 has slight deterioration.
- Layer 4 has slag and voids.
- Layer 5 has some voids.
- Layers 1&2 and 2&3 are delaminated.

- Aggregate Base:
ODOT A-1-a (visual)



Total Pavement Thickness = 19.25 in. Total Asphalt Thickness = 5.75 in. Total Concrete Thickness = 13.50 in. Total Base Thickness = 5.25 in.





6350 Presidential Gateway
Columbus, Ohio 43231
Telephone: (614) 823-4949
Fax Number: (614) 823-4990

Pavement Core Data Summary

PROJECT JEF-213-15.18 Bridge Replacement
LOCATION Hammondsville, Jefferson County, Ohio
JOB No. W-24-117

BORING/CORE No. X-001-0-25
DATE CORE OBTAINED 7/8/2025
CORE OBTAINED BY IS, JR, BH

Core Composition

Comments/Remarks

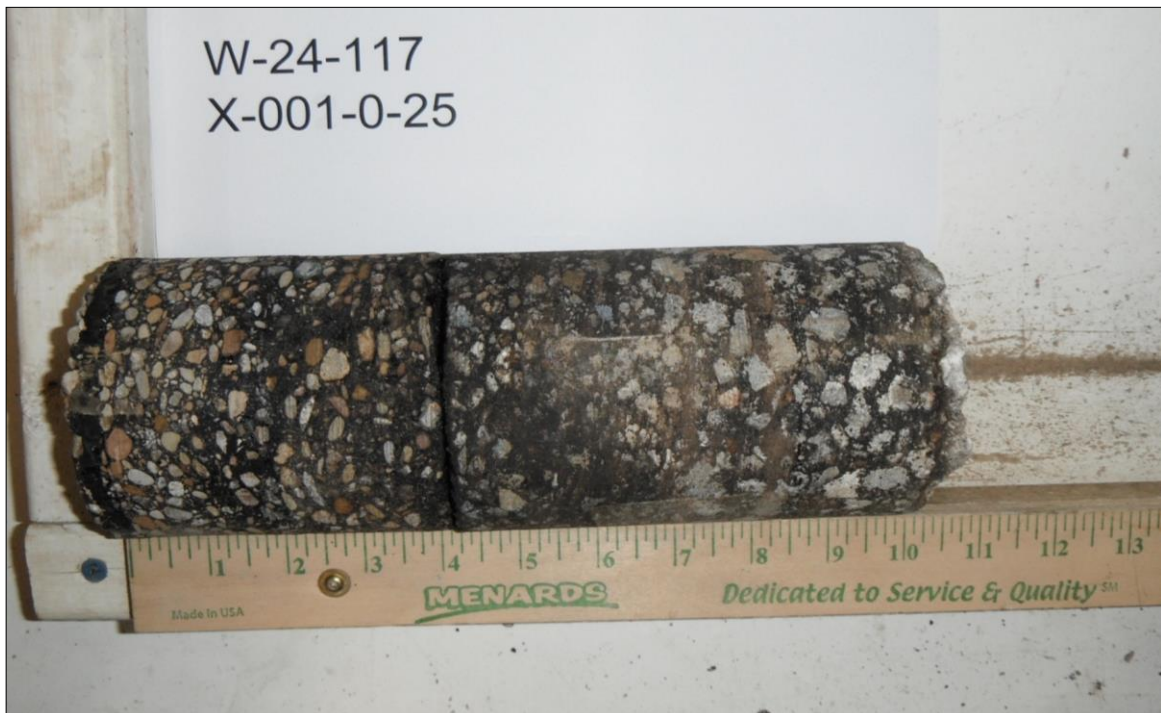
Core Number	Layer Thickness (in.)	Pavement Layer Number	Asphalt			Concrete	Aggregate/Granular Base	Other	
			Surface Binder	Intermediate Binder	Base Binder				
X-001-0-25	0.50	5	✓						
	1.75	4	✓						
	2.00	3	✓						
	2.25	2	✓						
	3.50	1	✓						
	5.00					✓			

- Layers 1 & 2 have slag.
- Layer 3 has slight deterioration.
- Layer 4 has voids.
- Layer 5 has bleeding.
- Layers 2&3 are delaminated.



- Aggregate Base: ODOT A-1-b (visual) phenol reaction

Total Pavement Thickness = 10.00 in. Total Asphalt Thickness = 10.00 in. Total Concrete Thickness = 0.00 in. Total Base Thickness = 5.00 in.





6350 Presidential Gateway
Columbus, Ohio 43231
Telephone: (614) 823-4949
Fax Number: (614) 823-4990

Pavement Core Data Summary

PROJECT JEF-213-15.18 Bridge Replacement
LOCATION Hammondsville, Jefferson County, Ohio
JOB No. W-24-117

BORING/CORE No. X-002-0-25
DATE CORE OBTAINED 7/8/2025
CORE OBTAINED BY IS, JR, BH

Core Composition

Comments/Remarks

Core Number	Layer Thickness (in.)	Pavement Layer Number	Asphalt			Concrete	Aggregate/Granular Base	Other	
			Surface Binder	Intermediate Binder	Base Binder				
X-002-0-25	11.50			✓					
	2.00						✓		

- Entire core received in bag, broken in several places. Cannot accurately discern layers and their placement. Core is deteriorating and has slag.

- Aggregate Base:
ODOT A-1-a (visual)
phenol reaction
slag present



Total Pavement Thickness = 11.50 in. Total Asphalt Thickness = 11.50 in. Total Concrete Thickness = 0.00 in. Total Base Thickness = 2.00 in.





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Pavement Core Data Summary

PROJECT JEF-213-15.18 Bridge Replacement
LOCATION Hammondsville, Jefferson County, Ohio
JOB No. W-24-117

BORING/CORE No. X-003-0-25
DATE CORE OBTAINED 7/8/2025
CORE OBTAINED BY IS, JR, BH

Core Composition

Comments/Remarks

Core Number	Layer Thickness (in.)	Pavement Layer Number	Asphalt			Concrete	Aggregate/Granular Base	Other	
			Surface Binder	Intermediate Binder	Base Binder				
X-003-0-25	2.00	2	✓						
	9.50	1	✓						
	2.50					✓			

- Layer 1 has voids and slight deterioration.
- Layer 2 is disintegrated.

- Aggregate Base: ODOT A-1-a (visual) phenol reaction slag present



Total Pavement Thickness = 11.50 in. Total Asphalt Thickness = 11.50 in. Total Concrete Thickness = 0.00 in. Total Base Thickness = 2.50 in.





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Pavement Core Data Summary

PROJECT JEF-213-15.18 Bridge Replacement
LOCATION Hammondsville, Jefferson County, Ohio
JOB No. W-24-117

BORING/CORE No. X-004-0-25
DATE CORE OBTAINED 7/8/2025
CORE OBTAINED BY IS, JR, BH

Core Composition

Comments/Remarks

Core Number	Layer Thickness (in.)	Pavement Layer Number	Asphalt			Concrete	Aggregate/Granular Base	Other	
			Surface Binder	Intermediate Binder	Base Binder				
X-004-0-25	0.50	4	✓						
	1.25	3		✓					
	1.50	2		✓					
	8.75	1	✓						
	4.00					✓			

- Layer 1 has slag, disintegration, broken, and bleeding. Cannot discern orientation of the different layers within (most of core came in bag).
- Layers 2&4 have slight deterioration.
- Layer 3 has trace voids.



- Aggregate Base: ODOT A-1-b (visual) phenol reaction slag present

Total Pavement Thickness = 12.00 in. Total Asphalt Thickness = 12.00 in. Total Concrete Thickness = 0.00 in. Total Base Thickness = 4.00 in.

