Pritchard Avenue Bridge Replacement (PID 116200)

Geotechnical Engineering Report – Rev. 2

November 25, 2024 | Terracon Project No. N4235298

Prepared for:

EMH&T Engineers, Surveyors, Planners, Scientists
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November 25, 2024

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Re: Geotechnical Engineering Report – Rev. 2
Pritchard Avenue Bridge Replacement (PID 116200)
Pritchard Avenue
Lisbon, Columbiana County, Ohio
Terracon Project No. N4235298

Dear Ms. Cueva:

Terracon Consultants, Inc. (Terracon) has completed the scope of Geotechnical Engineering Services for the above reference in general accordance with Terracon Proposal No. PN4235298 dated July 7, 2023.

This revised report presents the findings of the subsurface exploration, laboratory testing results, and the results of our foundation analyses performed for the proposed replacement of the existing Pritchard Avenue bridge located in Lisbon, Columbiana County, Ohio and incorporates comments on the shallow foundation section provided by Mr. Tyler Adams with EMH&T via email dated February 9, 2024, comments on the scour data section of this report provided by Mr. Adams via email dated February 26, 2024, and ODOT comments on the Stage 1 submittal provided by Mr. Adams via email dated July 31, 2024.

This report was further revised based on ODOT comments on the Stage 2 submittal provided by Mr. Adams via email dated November 7, 2024 on the scour data section of this report to calculate $D_{50,\ equivalent}$ for bedrock, and providing documentation for the methodology used to determine the drilled shaft tip and side resistance.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon

Ahmad Al-Hosainat

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Note: This report was originally delivered in a web-based format. **Blue Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the Terracon logo will bring you back to this page. For more interactive features, please view your project online at **client.terracon.com**. Refer to each individual Attachment for a listing of contents.

Geotechnical Engineering Services Report

Pritchard Avenue Bridge Replacement (PID 116200)
Lisbon, Columbiana County, Ohio
Terracon Project No. N4235298
November 25, 2024

Executive Summary

This report presents the findings of our geotechnical exploration performed for the proposed replacement of the existing bridge located along Pritchard Avenue between East High Street and Mill Street in Lisbon, Columbiana County, Ohio. The bridge is located approximately 100 feet to the south of Mill Street and 100 feet north of East High Street. The existing structure is a single-span beam bridge with a maximum span of approximately 51.5 feet, center to center of bearings. The proposed replacement structure is anticipated to include new foundation elements, abutments, and 6- to 9-inch-thick concrete composite deck. The horizontal and vertical alignments will closely replicate the existing alignments.

Terracon performed two (2) borings, designated as Borings B-001-0-23 and B-002-0-23 at the forward and rear abutments of Pritchard Avenue bridge to approximate depths of 20 to 21 feet below the existing ground surface. In addition, two (2) Dynamic Cone Penetrometer (DCP) tests designated as D-001-0-24 and D-002-0-24 were performed to depths of about 9.3 to 10.7 feet below the existing ground surface to support the findings of borings B-001-0-23 and B-002-0-23. The borings B-001-0-23 and B-002-0-23 encountered an aggregate base of approximately 8 inches and topsoil of approximately 6 inches, respectively. The borings encountered fill materials to depths varying from about 3 to 4.5 feet below the existing ground surface. The fill materials consisted of cohesive soils described as silty clay (A-6b), or granular soils described as sandy silt (A-4a).

The native cohesive soils encountered in the borings included stiff, silt and clay (A-6a). The native granular soils encountered in the borings included medium dense to very dense, coarse and fine sand (A-3a), and gravel and/or stone fragments with sand (A-1-b) in boring B-001-0-23. Bedrock was encountered in borings B-001-0-23, and B-002-0-23 at a depth of approximately 7.5 feet, which corresponds to elevations varying from about EL 935.4 to EL 938.7 feet. The bedrock encountered in the borings consisted of slightly to moderately weathered, weak to moderately strong shale interbedded with slightly to moderately weathered, moderately strong siltstone.

In boring B-001-0-23 groundwater was encountered at a depth of 4.5 feet below ground surface during drilling and not encountered upon completion of drilling. In boring B-002-

0-23 groundwater was encountered at a depth of 7.5 feet below ground surface during drilling and at 12 feet upon completion of drilling.

Based on the subsurface conditions encountered at the site, and the requirements outlined in section 305.3.3 of ODOT Bridge Design Manual (BDM), it is recommended that a shallow foundation system consisting of spread footings be used for support of the proposed bridge replacement structure. The recommended bearing capacity values and estimated settlement under structure foundations are presented in this report. Alternatively, a deep foundation system consisting of steel H-piles driven to refusal into bedrock or drilled shafts can be employed for support of the proposed bridge foundation elements. The recommended pile tip elevation and the corresponding ultimate bearing values (P_r), for each abutment are presented in the report. In addition, the estimated top of rock socket elevations and the corresponding unfactored nominal tip and side resistance for rock socketed drilled shafts at each abutment are presented in this report.

The embankments at the bridge abutments slope down towards the tributary of Little Beaver Creek at slope inclinations of about 3 Horizontal (H) to 1 Vertical (V) to 4H to 1V. Additional evaluation including slope stability analyses would be required to determine stability of the embankments. Once the plan and profile drawings for the bridge are available, we would be able to perform this evaluation, however this service would involve an additional fee.

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled **General Comments** should be read for an understanding of the report limitations.

Introduction

A structure foundation exploration has been completed for the proposed replacement of the existing bridge located along Pritchard Avenue between East High Street and Mill Street in Lisbon Columbiana County, Ohio. The bridge is located approximately 100 feet to the south of Mill Street and 100 feet north of East High Street. The existing structure is a single-span beam bridge with a maximum span of approximately 51.5 feet, center to center of bearings.

At the time of writing this report, it is our understanding that the proposed replacement structure is anticipated to include new foundation elements, and abutments. In addition, the horizontal and vertical alignments will closely replicate the existing alignments.

Site Location and Description

The following description of site conditions is derived from our site visits in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Location	The project site is located along Pritchard Avenue between East High Street and Mill Street in Columbiana County, Ohio, over the Tributary of Little Beaver Creek. The bridge is located approximately 100 feet to the south of Mill Street and 100 feet north of East High Street. The approximate latitude/longitude coordinates of the site are 40.77480, -80.75939. See **Site Location**
Existing Improvements	The existing structure is a single-span steel beam on a reinforced concrete deck and reinforced concrete/sandstone abutments. The deck also has an asphalt concrete wearing surface. The site is located near McKinley Elementary School and Lisbon Dog Park and is surrounded by a few residential dwellings in all directions.
Existing Topography	Based on our site reconnaissance and the provided topo survey, surface elevations of the bridge at the north and south abutments are approximately 946.2 and 942.9 feet, respectively. The creek bed elevation is approximately 938.7 feet.

Project Description

Item	Description
Site Layout	A plan and profile drawing for the proposed bridge is not available at the time of preparation of the report.
Proposed Construction	It is our understanding that the proposed structure is a single-span prestressed concrete box beam with a composite reinforced concrete deck and reinforced concrete substructure and that the new structure will maintain the existing horizontal and vertical alignments. The proposed replacement structure is anticipated to include new foundation elements, abutments, and deck. The new abutments are planned to be supported on driven piles or drilled shafts.
Grading	A grading plan is currently not available at the time of this report.

We would like the opportunity to review our recommendations and make modifications if required, once plan and profile drawings of the proposed bridge are available. We have assumed for the purposes of this report that the scour analyses will be performed by EMH&T and that protective measures will be provided in the design to mitigate erosion and global slope stability issues at the bridge abutments and the new structure will maintain the existing horizontal and vertical alignments. However, once the plan and profile drawings are developed, slope stability analyses should be performed to verify the global factor of safety of the abutment slopes.

Reconnaissance

At the time of our site reconnaissance visit on November 1, 2023, the existing Pritchard Avenue was observed to be a two-lane, asphalt paved roadway aligned in a north to south orientation, close to the existing substation. Existing bridge railing line both sides of Pritchard Avenue at the bridge structure, and sections of non-standard steel railing off structure line the West side of Pritchard Avenue only. The tributary of Little Beaver Creek was observed to be a relatively small, low flow waterway with a general flow direction towards the east at the subject structure. At the existing structure, surface drainage is directed into the existing creek. Based on Google Earth™, the side slopes of the abutment embankment appear to range from 3H:1V to 4H:1V.

General Geology

Based on the Ohio Department of Natural Resources (ODNR) Quaternary Geology Map of Ohio, the site is mapped within Late Wisconsinan aged Ground Moraine geology. A ground moraine consists of an irregular blanket of till deposited under a glacier. Composed mainly of clay and sand, it is the most widespread deposit of continental glaciers. According to ODNR's bedrock geology mapping, the site is underlain by Ohio Shale. The bedrock geology consists of Pennsylvanian-aged Allegheny and Pottsville group, consisting of shale, siltstone, and sandstone.

Exploration

Field Exploration

A total of two (2) borings, designated as B-001-0-23 and B-002-0-23 were performed on November 20, 2023 to depths of approximately 20 and 21 feet below the existing ground surface, respectively. In addition, two (2) Dynamic Cone Penetrometer (DCP) tests designated as D-001-0-24 and D-002-0-24 were performed on January 17, 2024 to depths of approximately 9.3 and 10.7 feet below the existing ground surface, respectively to support the findings of borings B-001-0-23 and B-002-0-23.

The borings were performed in general accordance with Section 303.3 of the Ohio Department of Transportation (ODOT) Specifications for Geotechnical Explorations (SGE) Type E1 bridge borings. The DCP tests were performed in general accordance with Section 406.3 of the ODOT SGE.

The approximate locations of the borings and DCP tests are illustrated on the attached **Exploration Plan** and summarized in the following table.

Boring/DCP ID	Elevation[1] (feet)	Lat [1]	Long.[1]	Total Depth (feet) [3]	Top of Rock Elevation (feet)	Top of Rock Depth (feet) [3]
B-001-0-23	942.9	40.77466	-80.75909	20	935.4	7.5
B-002-0-23	946.2	40.77498	-80.75950	21	938.7	7.5
D-001-0-24	943.0	40.77472	-80.75947	9.3	--	--
D-002-0-24	947.0	40.77486	-80.75931	10.7	--	--

1. Surface elevations at the boring/DCP locations were obtained from survey data provided by EMH&T.
2. Boring and DCP coordinates were obtained using a handheld GPS unit.
3. Below ground surface.

The borings and DCP locations were located in the field prior to drilling/testing operations by Terracon personnel using a handheld GPS unit. Ground surface elevations were obtained from survey data provided by EMH&T. Boring and DCP coordinates and elevations presented in the preceding table, and on the logs presented in Appendix A are approximate. The location and elevation information should be considered accurate only to the degree implied by the means and methods used to define them.

The borings were drilled with a track-mounted drill rig utilizing a 3¼-inch I.D. continuous flight hollow stem auger to advance the boreholes between sampling attempts. We performed continuous sampling using a split-barrel sampler to depths of approximately 10.5 feet, and at 5-foot intervals thereafter to the bedrock depth. We observed and recorded groundwater levels during drilling and upon completion of drilling.

In the split-barrel sampling procedure, the number of blows required to advance a standard 2-inch O.D. split-barrel sampler the last 12 inches of the typical total 18-inch penetration by means of a 140-pound automatic hammer with a free fall of 30 inches, is the standard penetration resistance value (SPT-N). This value is corrected to an equivalent (60 percent) energy ratio (N_{60}) utilizing the hammer efficiency energy ratio which is approximately 83.5% for the equipment used during our exploration.

Rock coring was performed using a NQ-size double tube-swivel core barrel. Percentage of recovery and rock quality designation (RQD) were calculated for the core samples and are noted at their depths of occurrence on the boring logs.

In the field, the samples recovered at the boring locations were examined and field logs were prepared indicating the conditions encountered at each location. Representative portions of soil samples obtained during the field exploration were preserved in sealable glass jars and rock cores in wooden boxes and delivered to our laboratory for additional examination and testing. Following the completion of drilling, the boreholes were sealed with auger cuttings mixed with bentonite chips.

DCP tests were performed using the dual-mass dynamic cone penetrometer by utilizing a 35 lb. hammer, which free falls 15 inches, to drive a cone point with a 60-degree tip. The tests were performed to a depth ranging from about 9.3 to 10.7 feet. The DCP test was terminated at the refusal depth.

Laboratory Testing Program

As part of the testing program, all samples were examined in our laboratory by a geotechnical engineer. Soil samples were classified in general accordance with ODOT SGE Section 600 Laboratory Testing based on the texture and plasticity of the soils.

Visual soil classification was performed on all recovered soil and rock samples. Atterberg limits, moisture content, and grain size analysis testing were performed on selected soil samples to obtain accurate information. In addition, unconfined compression and slake durability index tests were performed on selected rock sample to obtain rock properties information. The results of lab testing are shown on the boring logs and/or presented in the **Exploration and Laboratory Testing Results** of this report.

Findings

Boring logs have been prepared based on the information obtained from the field logs prepared at the time of drilling, the visual examination performed in the laboratory, and the laboratory testing results. Also, DCP logs have been prepared based on the information obtained from the field log prepared at the time of testing. Soil and rock classification was performed in general accordance with the current ODOT SGE. The following sections summarize the subsurface conditions encountered at the boring locations.

Subsurface Profile

Borings B-001-0-23 and B-002-0-23 were performed at the forward and rear abutments of Pritchard Avenue bridge over the tributary of Little Beaver Creek, respectively. Boring B-001-0-23 encountered a surficial layer consisting of aggregate base with a thickness of approximately 8 inches. Boring B-001-0-23 encountered a surficial layer consisting of topsoil with a thickness of approximately 6 inches.

Borings B-001-0-23 and B-002-0-23 encountered fill materials to depths ranging from about 3 to 4.5 feet below the existing ground surface. The fill materials consisted of cohesive soils described as silty clay (A-6b) and sandy silt (A-4a).

The native cohesive soils encountered in the borings included stiff, silt and clay (A-6a). The native granular soils encountered in the borings included medium dense to very dense, coarse and fine sand (A-3a), and gravel and/or stone fragments with sand (A-1-b) in boring B-001-0-23.

Bedrock

Bedrock was encountered in borings B-001-0-23, and B-002-0-23 at a depth of approximately 7.5 feet, which corresponds to elevations varying from about EL 935.4 to EL 938.7 feet. The bedrock encountered in the borings consisted of slightly to moderately weathered, weak to moderately strong shale interbedded with slightly to moderately weathered, moderately strong siltstone. D-001-0-24 and D-002-0-24 tests were terminated at refusal depths between approximately 9.3 and 10.7 feet below existing grade, indicating bedrock depths of about 9.3 feet and 10.7 feet below existing grade, respectively (approximate elevations of 933.7 feet and 936.3, respectively).

Groundwater Conditions

In boring B-001-0-23 groundwater was encountered at a depth of 4.5 feet below ground surface during drilling and was not encountered upon completion. In boring B-002-0-23 groundwater was encountered at a depth of 7.5 feet below ground surface during drilling and at 12 feet upon completion of drilling.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff, creek water level and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the proposed structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

Analysis and Recommendations

We understand that drilled shaft or driven H-Piles deep foundation systems are being considered to support abutments of the proposed bridge. Additionally, support of the abutments on shallow foundations bearing within bedrock may be a feasible alternate. The new structure will maintain the existing horizontal and vertical alignments.

Drilled Shaft Recommendations

Based on the subsurface conditions encountered at the site, and the requirements outlined in Section 305.4 of ODOT Bridge Design Manual (BDM), a deep foundation system consisting of drilled shaft foundations can be considered for supporting the proposed bridge. Based on the test borings, we recommend that the drilled shafts be socketed at least 1.5 times the rock socket diameter into the bedrock below the estimated top of rock elevations presented in the table below. The actual socket length may be greater based axial loading/ lateral loading conditions and final shaft lengths should be determined by the designer.

Based on the encountered subsurface conditions, drilled shafts that derive resistance from end bearing and side resistance in bedrock can be used for the proposed bridge structures. The designer should refer to AASHTO LRFD Section 10.8.3.5.4d for guidance on proportioning the resistance between tip resistance and side resistance. Documentation for the methodology used to determine the drilled shaft tip and side resistance is provided in the **Supporting Information** section of this report.

The following sections provide recommendations regarding the design of drilled shaft foundations to resist axial compressive and uplift loads, as well as soil and bedrock parameters to design the drilled shafts to resist lateral loads. Our recommendations consider the soil and bedrock conditions encountered in the test borings.

Drilled Shaft Design

Boring ID	Estimated Top of Rock Socket Elevation (feet) [1]	Minimum Rock Socket Length (feet)	Embedment Material	Minimum Shaft Diameter (inches) [2]	Unfactored Nominal Unit Tip Resistance, q_p (ksf) [3]	Unfactored Nominal Unit Side Resistance, q_s (ksf) [4]	Resistance Factor, φ_{stat}
B-001-0-23	935.4	1.5 x Shaft Diameter	Interbedded siltstone/shale	36	120	19	0.50 (Tip) 0.55 (Side) 0.4 (uplift resistance)
B-002-0-23	938.7						

1. See **Findings** and the boring logs for soil and bedrock stratigraphy details. Top of rock socket elevations listed in this table are interpreted from test borings. The drilled shaft lengths will vary depending upon the depth to top of rock of the siltstone and shale bedrock. Since bedrock exists within the required minimum 10-ft pile length per BDM section 305.3.5.2, the drilled shafts should be placed in prebored holes that extend a minimum of 5 feet into bedrock to meet the minimum required lengths of the shafts. Due to anticipated variation in top of rock elevation, top of rock socket elevations should be field verified with pre-bored holes per ODOT C&MS Items 524.08 & 524.09 during construction.
2. Rock socket diameter should at least 6 inches less than the actual diameter of the shaft.
3. Rock socketed drilled shaft should be designed following BDM Section 305.4.2. For uplift, a resistance factor of 0.4 should be applied to the Nominal Unit Side Resistance. The weight of the shaft can also be used to resist any uplift forces. The buoyant weight of the shaft should be used below the anticipated groundwater level to resist uplift forces.
4. The geotechnical resistances provided herein are based on the laboratory Unconfined Compression Test results performed on rock core samples obtained below the top of rock socket elevation.

The drilled shaft length will need to be designed to satisfy axial compressive, uplift, and lateral load requirements. The penetration of the drilled shaft into shale/siltstone bedrock may need to be increased over the minimum rock socket for axial compressive capacity based on the lateral resistance or uplift resistance requirements of the drilled shaft foundations. In general, based on the geotechnical resistances provided above, drilled shafts should be designed per BDM section 305.4.

Recommended L-Pile Parameters for Lateral Pile Analysis

The following table provides input values for use in LPILE analyses. LPILE estimated values of k_h and E_{50} were based on strength; however, non-default values of k_h were used where provided. The soil parameters were estimated based on the test borings, laboratory test results, and our experience with these soil types. For the portion of the drilled shaft within 36 inches of finished grade, any lateral soil resistance should be ignored due to frost considerations and construction disturbance.

The tables below present the recommended L-Pile parameters for each boring to be used for lateral pile analysis.

BORING B-001-0-23 (South Abutment)							
Soil Layer/Type[1]	**Approx. Depth to Bottom of Layer (feet)[2]**	**LPILE Model**	**Total Unit Weight (pcf)**	**Undrained Shear Strength (psf)**	**Soil Friction Angle (deg)**	**K (pci)**	**ε_{50}**
Silt and Clay (A-6a)	3.0	Stiff Clay w/o Free Water (Reese)	126	1,500	--	500	0.007
Gravel and/or Stone Fragments with sand (A-1-b)	4.5	Sand (Reese)	118	--	31	60	--
Silt and Clay (A-6a)	6.0	Stiff Clay with Free Water (Reese)	126	1,500	--	500	0.007
Coarse and fine Sand (A-3a)	7.5	Sand (Reese)	125	--	34	80	--
Weathered Bedrock	10.0	Sand (Reese)	127	--	35	100	--
Bedrock[3]	20.0	Weak Rock	See Table Below for Rock Properties				

1. Below existing ground surface.
2. See test boring logs and Findings for more details on Stratigraphy.
3. Boring terminated within this layer.

Layer Number	Rock Type	Approx. Depth to Bottom of Layer (feet)	Total Unit Weight (pcf)	Rock Compressive Strength (psi)	Elastic Modulus (psi)	RQD (%)	k_{rm}
7	Shale	12.0	140	1,000	4,000	5	0.0005
8	Siltstone	17.0	145	4,000	60,000	75	0.00005
9	Shale	20.0	140	1,000	36,000	61	0.0005

BORING B-002-0-23 (North Abutment)							
Soil Layer/Type[1]	Approximate Bottom Elevation of Layer (feet) [2]	LPILE Model	Total Unit Weight (pcf)	Undrained Shear Strength (psf)	Soil Friction Angle (deg)	K (pci)	ε_{50}
Silt and Clay (A-6a)	3.0	Stiff Clay w/o Free Water (Reese)	125	1,000	--	300	0.010
Sandy Silt (A-4a)	4.5	Sand (Reese)	118	--	31	60	--
Silt and Clay (A-6a)	6.0	Stiff Clay w/o Free Water (Reese)	126	1,500	--	500	0.007
Coarse and fine Sand (A-3a)	7.5	Sand (Reese)	125	--	34	80	--
Weathered Bedrock	11.0	Sand (Reese)	127	--	35	100	--
Bedrock[2]	21.0	Weak Rock	See Table Below for Rock Properties				

1. Below existing ground surface.
2. See test boring logs and Findings for more details on Stratigraphy.
3. Boring terminated within this layer.

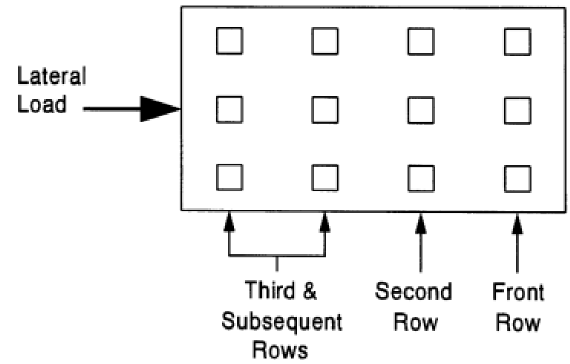
Layer Number	Rock Type	Approx. Depth to Bottom of Layer (feet)	Total Unit Weight (pcf)	Rock Compressive Strength (psi)	Elastic Modulus (psi)	RQD (%)	k_{rm}
7	Shale	12.0	140	1,000	4,000	5	0.0005

Layer Number	Rock Type	Approx. Depth to Bottom of Layer (feet)	Total Unit Weight (pcf)	Rock Compressive Strength (psi)	Elastic Modulus (psi)	RQD (%)	k_{rm}
8	Siltstone	13.0	145	6,000	60,000	100	0.00005
9	Shale	16.5	140	1,000	6,000	5	0.0005
10	Siltstone	18.5	145	6,000	60,000	79	0.00005
11	Shale	21.0	140	1,000	30,000	30	0.0005

The structural capacity of the drilled shafts should be checked to assure that they can safely accommodate the combined stresses induced by axial and lateral forces. Lateral deflections of drilled shaft foundations should be evaluated using an appropriate analysis method, and will depend upon the element's diameter, length, configuration, stiffness and "fixed head" or "free head" condition. We can provide additional analyses and estimates of lateral deflections for specific loading conditions upon request, at an additional fee. The load-carrying capacity of drilled shaft foundations may be increased by increasing the section. Proper reinforcing steel should be included in the drilled shaft designs for resistance of the combined axial loads and bending moments.

Group action for lateral resistance of drilled shaft foundations should be considered when the center-to-center spacing is less than 6 diameters. For a group of shafts oriented parallel to a lateral load, design parameters for allowable passive resistance within soil should be reduced in accordance with BDM section C305.4.4.1 as shown in the following table. Group reduction factor is not applicable for the portion of the shafts socketed in rock.

Laterally Loaded Shafts – Group Reduction Factors			
Shaft Spacing [1] (Diameters)	**Leading Row Reduction Factor**	**Second Row Reduction Factor**	**Third or Higher Row Reduction Factor**
6D	1.0	1.0	1.0
5D	1.0	0.85	0.7
3D	0.8	0.4	0.3



1. Center-to-center spacing in the direction of loading. If the loading direction for a single row of shafts is perpendicular to the row, a group reduction factor should be used if the shaft spacing is less than 5D.

Drilled Shaft Construction Considerations

In general, drilled shaft installation should be in accordance with C&MS Items 524 and BDM section 305.4. Key considerations include:

- The concrete shall have a minimum 28-day specified compressive strength of 4,500 psi.
- It is recommended that the top of rock and design rock socket be shown for each drilled shaft on the plans, with these elevations being determined using the test borings and minimum embedment requirements from axial load analyses.
- The final tip elevation should be determined by inspection of each shaft excavation in the field by a qualified geotechnical technician.
- The foundation drawings should identify those shafts where the minimum embedment lengths are based on axial and/or lateral load analyses.
- The drilled shaft specifications should be clear that the design bottom of the drilled shaft elevations shown on the plans is for estimation purposes only. The actual determination of the bottom elevation will be made during the installation per C&MS Items 524.08 & 09.
- Typical drilled shaft construction notes should be prepared by the designer per BDM section 606.8.

Driven Pile Recommendations

Alternatively, the proposed bridge structure can be supported on driven steel H-piles. As per the ODOT BDM, refusal is met during driving when the pile penetration is an inch or less after receiving at least 20 blows from the pile hammer. Since bedrock exists within the required minimum 10-ft pile length per BDM section 305.3.5.2, piles should be installed in prebored holes that extend a minimum of 5 feet into bedrock to meet the minimum required lengths of the piles per BDM section 305.3.5.2 and CMS section

507.04. The following table shows the recommended factored structural resistance values (P_r) at each abutment.

Boring Number	Ground Surface Elevation[1]	Pile Type	Est. Pile Top Elevation[2]	P_r [3, 4] (kips/pile)
B-001-0-23	942.9	HP 10x42	933.9	310
		HP 12x53	933.9	380
		HP 14x73	933.9	530
B-002-0-23	946.2	HP 10x42	937.2	310
		HP 12x53	937.2	380
		HP 14x73	937.2	530

1. Ground surface elevation is the existing ground elevation at the boring location, obtained by handheld GPS. The elevations should be adjusted to surveyed site elevations once survey data is available.
2. The top of pile elevation is at the proposed bottom of footing elevation at the abutment locations, a 10-foot minimum pile length is required per the BDM section 305.3.5.2. Pile tip elevation is taken at sample with split-spoon refusal at the nearest boring per ODOT BDM section 305.3.5.2. Plan set was not provided to us during Geotechnical report preparation. Estimated pile length should be verified by Structural Engineer.
3. Factored structural resistance per BDM C305.3.3-1 for piles driven to refusal.
4. We assumed that the slopes supporting the abutments will be protected against scour to prevent loss from scour.

Per the most recent version of the ODOT BDM, the factored resistance for piles driven to refusal on bedrock is typically governed by the structural resistance of the pile element. The maximum factored structural resistances listed in the table above assume an axially loaded pile with negligible moment, no appreciable loss of section due to deterioration throughout the life of the structure, a steel yield strength of 50 ksi, a structural resistance factor for H-piles subject to damage due to severe driving conditions (LRFD 6.5.4.2: φ_c = 0.50) and a pile fully braced along its length. In addition, pile points/shoes are required for this project.

These bearing values should not be used for piles that are subjected to bending moments or are not supported by soil for their entire length. Therefore, it is recommended that slope erosion protection (riprap) be specified to line the slopes supporting the abutments to prevent a loss of support from scour.

Please note that it is anticipated that the piles will be able to be driven a short distance into the bedrock before satisfying the driving conditions that meet the refusal criteria.

The piles should be driven to refusal as defined in Section 305.3.1.2 of the ODOT BDM. Settlement is estimated to be less than 1.0 inch for H-Piles driven to refusal on bedrock.

A pre-construction wave equation analysis such as GRLWEAP analysis should be performed by the contractor to assess the ability of the proposed driving system to develop a point resistance of at least 90% of the pile yield stress without damaging the pile. Typically, single-acting diesel pile driving hammers having a rated energy of up to 44,000-ft-lb are commonly used for pile driving. The results of the wave equation should be submitted to the Engineer of Record for review and approval of the proposed pile driving system prior to construction. Piles should be installed according to Item 507 of the most recent ODOT Construction and Material Specifications (CMS). In addition, field dynamic testing with a pile driving analyzer per ODOT CMS Item 523 is recommended to confirm that the contractor's equipment is imparting the required energy to the piles without causing damage to the piles.

The recommendations require the piles bear in the existing shale/siltstone bedrock. Note that weak, highly weathered rock such as shale may exhibit a decrease in capacity after pile driving has ended due to a phenomenon known as relaxation. Therefore, piles should be driven to refusal within the shale bedrock, then re-driven to refusal after the relaxation has occurred. Based on ODOT BDM (Section 600), a wait period of 7 days is recommended before re-strike.

Pile Drivability Analysis

According to the comments on section C305.3.1.2 in the ODOT Bridge Design Manual (BDM), a wave equation analysis is not necessary for point bearing piles on bedrock when all three of the following apply:

- Bedrock is 50 ft or less in depth.
- All granular soils are medium dense or less in density (SPT N160 ≤ 30 bpf).
- All cohesive soils are very stiff or less in consistency (SPT N60 ≤ 30 bpf).

Therefore, the preliminary drivability analysis for borings B-001-0-24 and B-002-0-24 was not performed as all the above-mentioned conditions were met for these borings. Prior to construction, the contractor shall perform a drivability analysis using the pile hammer-cushion combination that will be used.

Pile driving conditions, hammer efficiency, and stress on the pile during driving could be better evaluated during installation using a Pile Driving Analyzer (PDA) performed by a qualified pile testing contractor on selected piles. Driving criteria for the driven piles should be recommended by the pile testing contractor, based on the PDA results. During driving a maximum of 20 blows per inch is recommended to reduce the potential of damage to the piles. Each pile should be observed and checked for buckling, crimping, and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations. The pile driving process should be performed under the direction of the Geotechnical Engineer or their representative. They should document the pile installation process including hammer blow counts, consistency with expected conditions, and details of the installed pile.

Shallow Foundation Recommendation

A shallow foundation system could be utilized for support of the forward and rear abutments. Please note that the proposed shallow foundations should be properly protected against scour.

The proposed shallow foundations/spread footings for the bridge abutments are anticipated to be embedded at elevations of 932.9 and 935.2 feet on bedrock at borings B-001-0-23 and B-002-0-23, respectively (approximately 10 and 11 feet below existing ground surface, respectively). We recommend that the shallow foundations bearing on unweathered shale/siltstone bedrock be designed for a nominal bearing resistance of 20,000 psf with a resistance factor of ϕ_b = 0.45, corresponding to a factored bearing resistance of 9,000 psf. We estimate that total settlements will be on the order of 1/2 inch or less. Note that the nominal bearing resistance of the foundations should not be greater than the compressive resistance of the footing concrete.

Additionally, scour evaluation of the bedrock encountered at both boring locations was performed to determine the scour potential below the bearing elevation of spread footings, the results of the analysis are provided in the **Supporting Information** section of this report and indicated that the shale/siltstone bedrock was scourable to the depths explored, due to the scour potential of the rock encountered below the bearing elevation of the spread footings, scour analysis should be performed to determine the non scourable depth of bedrock before considering the spread footing foundation system.

The coefficient of sliding friction recommended for contact between the concrete and foundation rock is 0.35 with a resistance factor of ϕ_T = 0.9. If the bottom of the footings is keyed at least 3-in into the weak to moderately strong rock, a sliding resistance check will not be required.

The foundation excavations should be examined after excavations to verify that the entire bearing surface consists of suitable bedrock. All decomposed/weathered bedrock should be removed from the bottoms of the excavations prior to concrete placement over the shale/siltstone bedrock. Confine the excavation into bedrock for the minimum specified depth of keying within the area bounded by the outer edge of the footing. Fill excavation outside these limits and within and below the keyed depth with concrete per CMS 503.05. It is recommended that the geotechnical engineer be retained to observe and test the foundation bearing materials.

The excavations will encounter weak to moderately strong shale/siltstone. The weak bedrock encountered can be excavated using heavy earth moving equipment. Moderately strong bedrock will likely require special rock removal techniques such as rippers, hydraulic hammers, and/or blasting to remove and properly size the excavated bedrock for placement as fill. The bedrock is weak to moderately strong shale/siltstone. Therefore, excavations should be maintained per OSHA requirements for stable slopes.

Based on the depth of groundwater, a cofferdam may be required to keep the footing excavation dry at the abutment locations. Due to shallow bedrock, sheet pile will not be able to penetrate deep enough to provide adequate resistance. The contractor is responsible for determining the cost-effective solution for the cofferdam to meet the minimum requirements for the project including water flow and safety. After excavation into the bedrock is complete, if water infiltrates into the cofferdam, a concrete seal coat should be utilized to provide a watertight seal. The seal coat should be made of Class SC Concrete tremied underwater.

Lateral Earth Pressures

Retaining walls, and excavation support systems must be designed to withstand lateral earth pressures, as well as hydrostatic pressure, that may develop behind the structures. The magnitude of lateral earth pressure varies on the basis of soil type, permissible wall movement, and type of backfill.

In order to minimize lateral earth pressures, the zone behind the structures should be effectively drained. For effective drainage, a zone of porous backfill (ODOT CMS Item 518.03) should be used directly behind the structures for a minimum thickness of 2 feet in accordance with ODOT CMS Item 518.05. The granular zone should be designed to drain to either weepholes or a pipe, to alleviate the build-up of hydrostatic pressures against the walls.

The type of backfill beyond the free-draining granular zone will govern the pressure to be used for structural design. Pressures of a relatively low magnitude will be generated by granular backfill materials, whereas cohesive backfill materials will result in the development of higher lateral pressures. Therefore, it is recommended that granular backfill be utilized whenever possible. Granular backfill behind structures should be placed and compacted in accordance with ODOT CMS Item 203.

Retaining walls that are fixed and unable to rotate or deflect will be subjected to at-rest earth pressure conditions. Earth pressure distributions should be based on the mobilization of active earth pressure conditions for retaining walls that are free to deflect or rotate. Retaining walls exerting a force on the soil (such as soil in front of the footing on the face side of the wall) are subject to a passive resistance. However, due to the potential for erosion, this passive resistance is typically ignored.

The tables presented below include the recommended unfactored and factored equivalent fluid unit weights for walls subject to the mobilization of both at-rest and active earth pressure conditions as described above. A load factor of 1.5 has been used for the determination of the factored equivalent fluid unit weights. The values presented in the following table assume a flat backslope behind the walls, and that the backfill material will not be subject to any additional load (such as uniformly distributed soil surcharge near the top and immediately behind the face of the wall). Two cases have been considered for backfill behind the wall: a two-foot-wide zone of granular porous backfill with filter fabric, and backfilling with a wedge of granular material.

For a two-foot-wide zone of granular porous backfill, the earth pressure was calculated assuming an angle of internal friction of 24 degrees, a moist soil unit weight of 125 pcf, and a soil/concrete interface friction angle of 16 degrees.

Wall Type	Pressure Distribution	Unfactored Equivalent Fluid Weight (pcf)	Factored Equivalent Fluid Weight (pcf)	Earth Pressure Coefficient
Cantilever Retaining Wall – Free Head	Active	47.5	71	$K_a = 0.38$
Rigid Retaining Wall – Fixed Head	At-rest[1]	74	100	$K_o = 0.59$

1. Due to the fixity condition at the top of the wall, it is recommended that the triangular pressure distribution should be converted into a uniform or rectangular pressure distribution along the height of the wall.

For a wedge of granular material (assuming 2:1 backslope from bottom of backfill), the earth pressure was computed assuming an angle of internal friction of 30 degrees, a moist soil unit weight of 120 pcf, and a soil/concrete interface friction angle of 20 degrees.

Wall Type	Pressure Distribution	Unfactored Equivalent Fluid Weight (pcf)	Factored Equivalent Fluid Weight (pcf)	Earth Pressure Coefficient
Cantilever Retaining Wall Free Head	Active	36	54	$K_a = 0.30$
Rigid Retaining Wall Fixed Head	At-rest[1]	60	81	$K_o = 0.50$

1. Due to the fixity condition at the top of the wall, it is recommended that the triangular pressure distribution should be converted into a uniform or rectangular pressure distribution along the height of the wall.

The earth pressure values presented in the preceding tables assume that provisions for positive gravity drainage will be provided, and that the abutments and walls will be backfilled with free-draining coarse aggregate, such as ODOT No. 57 stone.

We do not recommend using passive earth pressures in design of permanent retaining walls and/or bridge abutments due to the potential for erosion, or possibility of removal of the soils in front of the wall in the future.

Scour Data

Continuous sampling was performed to a depth of 10.5 feet and thereafter at an interval of 5 feet in each boring. The sampling was performed to determine the median grain size (D_{50}) of the collected soil samples. Based on the conditions encountered at the boring locations, it is anticipated that the streambed soils will consist of granular soils consisting of coarse and fine sand (A-3a), sandy silt (A-4a), and gravel and/or stone fragments (A-1-b). Note that borings were not drilled within the creek as part of this exploration. Recovered soil samples evaluated for potential scour were from borings performed behind the existing abutments. As such, actual soil conditions and potential scour within the creek may vary from the conditions encountered in the borings performed behind the abutments. Based on the grain size analyses performed by Terracon, the following table summarizes the D_{50} values from testing of soil samples from the borings. Additionally, the critical shear stress (τ_c), the equivalent D_{50} ($D_{50,\ equiv}$), and Erosion Category (EC) were calculated based on the equations provided in GDM sections 1302.1, 1302.2 and 1403, and also summarized in the following table.

Boring Number	Sample Number	Elevation (feet)	D_{50} Value (mm)	τ_c (psf)	$D_{50,\ equiv}$ (mm)	Erosion Category (EC)
B-001-0-23	SS-3	939.9-938.4	0.663	0.01385	0.663	1.986
	SS-4	938.4-936.9	0.091	0.06040	1.139	3.412
	SS-5	936.9-935.4	0.259	0.00541	0.259	1.496
	Shale	932.9-930.9	--	2.04	97.70	2.08
	Siltstone	930.9-925.9	--	99.81	4,770	2.66
	Shale	925.9-922.9	--	89.08	4,260	2.44
B-002-0-23	SS-3	943.2-941.7	0.144	0.01909	0.144	1.19
	SS-4	941.7-940.2	0.200	0.01126	0.539	3.0747
	SS-5	940.2-938.7	0.139	0.04896	0.139	1.172
	Shale	935.2-934.2	--	2.04	97.70	1.722
	Siltstone	934.2-933.2	--	35.19	1,680	2.656
	Shale	933.2-929.5	--	2.04	97.7	2.084
	Siltstone	929.5-927.7	--	102.82	4,900	2.561
	Shale	927.7-925.2	--	7.29	348.8	2.295

Additionally, scour evaluation of the bedrock encountered at both boring locations was performed to determine the scour potential below the bearing elevation of spread footings, the results of the analysis are provided in the **Supporting Information** section of this report and indicated that the shale/siltstone bedrock was scourable to the depths explored, due to the scour potential of the rock encountered below the bearing elevation of the spread footings, scour analysis should be performed to determine the non scourable depth of bedrock before considering the spread footing foundation system.

According to section 305.4.1.1 in the ODOT Bridge Design Manual (BDM), friction drilled shafts should be extended to penetrate a minimum of 15 feet below the controlling scour elevation. If socketed into non-scour resistant bedrock, when the rock has $Q_u < 2.5$ ksi, drilled shafts should be extended to penetrate a minimum of 10 feet below the controlling scour elevation in the bedrock; when the rock has $Q_u \geq 2.5$ ksi, drilled shafts should be extended to penetrate a minimum of 5 feet below the controlling scour elevation in the bedrock. In boring B-001-0-23, the laboratory results of Q_u showed about 9 ksi in the rock at an elevation of 930.9. In boring B-002-0-23, testing was only performed on the siltstone layer (Elev 927.7), so the Q_u values were not reported above this elevation. However, an estimated value of more than 2.5 ksi can be given based on the similar rock conditions encountered at boring B-001-0-23 and the experience with similar rock conditions.

Seismic Site Classification

Code Used	Site Classification
AASHTO LRFD Bridge Design Specifications, Ninth Edition, 2020 [1]	C [2]

1. In general accordance with Section 3.10.3 of the AASHTO LRFD Bridge Design Specifications, Ninth Edition, 2020.
2. AASHTO LRFD Bridge Design Specifications, requires a site subsurface profile determination extending to a depth of 100 feet for seismic site classification. Borings for this study extended to a maximum depth of approximately 21 feet and this seismic site class definition considers that bedrock continues below the maximum depth of the subsurface exploration. Additional exploration to deeper depths could be performed to confirm the conditions below the current depth of exploration. Alternatively, a geophysical exploration could be utilized in order to attempt to justify a higher seismic site class. The current scope requested does not include the required 100-foot subsurface profile determination.

Construction Considerations

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

The geotechnical engineer should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during subgrade preparation, proof-rolling, placement and compaction of controlled compacted fills, and backfilling of any excavations into the completed subgrade.

Earthwork Considerations

Subgrade preparation for the new foundations, pavement, shoulder areas, and embankments should be performed in accordance with ODOT CMS Items 203 and 204. Prior to subgrade preparation, perform clearing and grubbing, including removal of stumps and roots, in accordance with ODOT CMS Item 201. Remove existing pavement and base materials as well as other structures or obstructions, as necessary, in accordance with ODOT CMS Item 202. The subgrade should be stripped of any topsoil, organics, or other deleterious or unsuitable materials.

All embankment materials should be spread and compacted in accordance with Items 203.06 and 203.07 and subgrade materials should be spread and compacted in accordance with Items 204.07 and 204.03. Frozen materials should not be incorporated into any new fill nor should new fill, pavement materials, or structures be placed on top of frozen materials. Material to be utilized as borrow should be restricted to conform to Item 203.02R and 203.3 for embankment construction and Item 204.2 for subgrade. Clay with high plasticity should not be used for the embankment.

Earthwork, including subgrade preparation should be performed in accordance with respective items in Section 200 of the current ODOT CMS. Consideration may be given to using the in-situ soils or from the local borrow sources. However, the material may require moisture adjustments to achieve proper compaction. Potentially, chemical treatment may be used for any borrow materials and existing embankment soil with high moisture contents. Chemical treatment should be performed in accordance with ODOT Item 205.

If applicable, it is recommended that any benching required for embankment construction for the project be performed in accordance with “A. General Case: Special Benched Embankment Construction” of ODOT Geotechnical Bulletin 2 (GB-2).

Grading and Drainage

During construction, site grading should be developed to direct surface water flow away from, or around, the site. Exposed subgrades should be sloped to provide positive drainage so that saturation of subgrades is avoided. Surface water should not be permitted to accumulate on the site.

Final surrounding grades should be sloped away from the proposed embankments on all sides to prevent ponding of water. Due to the nature of the soil profile, trapped water infiltration or groundwater seepage may be encountered, particularly after periods of precipitation. In such an event, sump and pumping methods may be used for temporary dewatering.

Excavation Considerations

As a minimum, all excavations should be sloped or braced as required by Occupational Health and Safety Administration (OSHA) regulations to provide stability and safe working conditions. Reference to OSHA 29 CFR, Part 1926, Subpart P should be included in the job specifications. current OSHA excavation and trench safety standards.

The grading contractor, by his contract, is usually responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required, to maintain stability of both the excavation sides and bottom. Slope heights, slope inclinations and/or excavation depths should in no case exceed those specified in local, state, or federal safety regulations, including the current OSHA Excavation and Trench Safety Standards.

Under no circumstances should the information provided in this report be interpreted to mean that Terracon is responsible for construction site safety or the contractor's activities. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods, and sequencing of the construction operations.

Groundwater Considerations

In boring B-001-0-23 groundwater was encountered at a depth of 4.5 feet below ground surface during drilling and not encountered upon completion. In boring B-002-0-23 groundwater was encountered at a depth of 7.5 feet below ground surface during drilling and at 12 feet upon completion of drilling.

Groundwater is anticipated during construction at the normal water elevation of the creek. Where encountered during construction, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 5 feet below the deepest excavation. Any seepage or groundwater encountered during foundation excavation should be able to be controlled by pumping from temporary sumps. However, additional measures may be required depending on seasonal fluctuations of the creek/groundwater level. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

Slope Stability Analyses

The embankments at the bridge abutments slope down towards the tributary of Little Beaver Creek at slope inclinations of about 3 Horizontal (H) to 1 Vertical (V) to 4H to 1V, additional evaluation including slope stability analyses would be required to determine stability of the embankments. Once the plan and profile drawings for the bridge are available, we would be able to perform this evaluation, however this would involve additional costs.

General Comments

Terracon should be retained to review the final design plans and specifications, so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. Terracon should also be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

This Geotechnical Engineering Report has been prepared to present the findings of our exploration and present our recommendations pertaining to proposed improvements. The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials, or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

Appendices

Appendix A – Field Exploration Information

Contents:

Site Location
Exploration Plan

Note: All attachments are one page unless noted above

Site Location

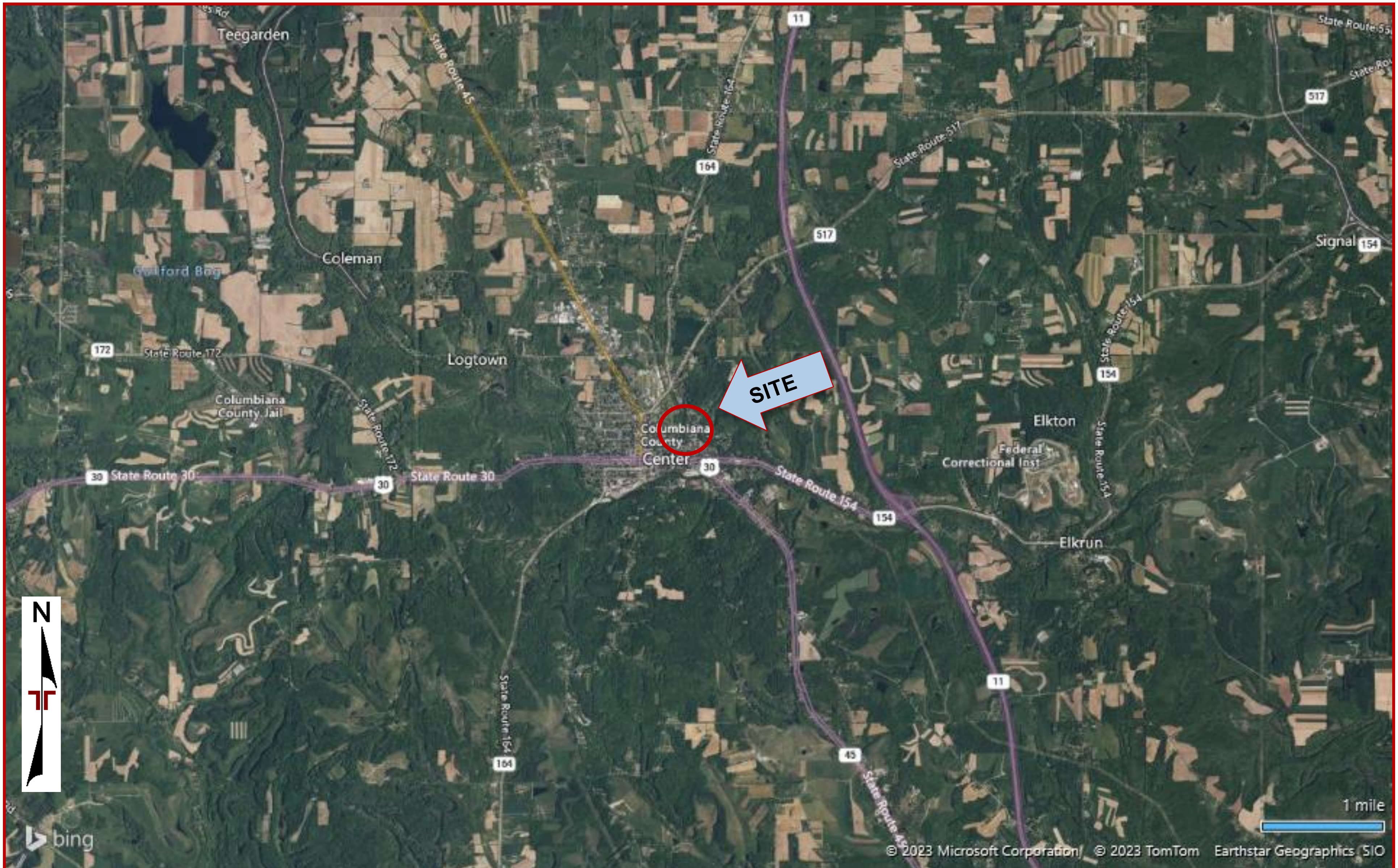


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan

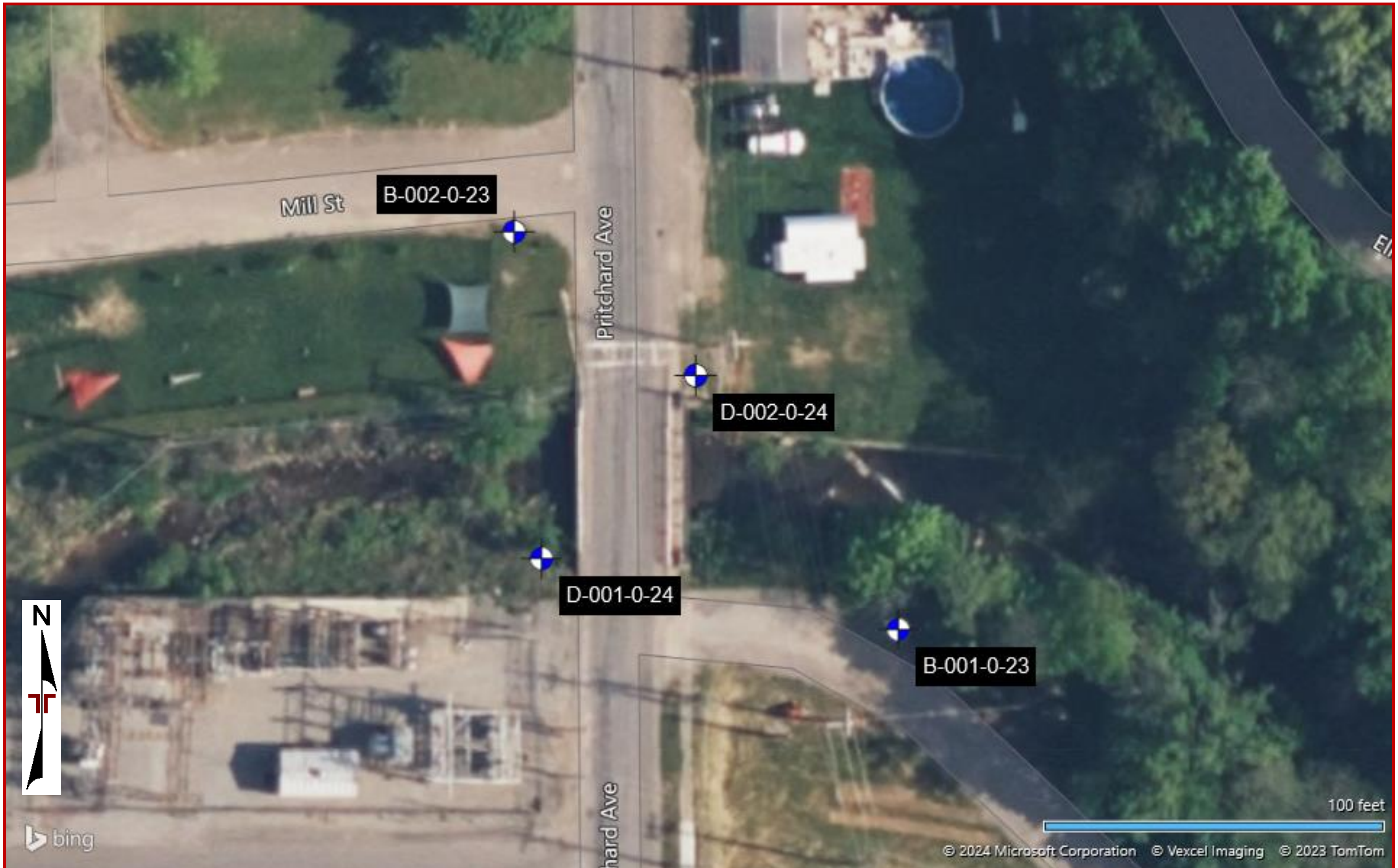


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Appendix B – Exploration and Laboratory Testing Results

Contents:

Boring Logs (B-001-0-23 and B-002-0-23)
Atterberg Limits
Grain Size Distribution (2 Pages)
Unconfined Compression Test (3 pages)
Slake Durability Index Test
Rock Core Photographs (2 pages)
DCP Logs (2 pages)

Note: All attachments are one page unless noted above.

PROJECT: PRITCHARD AVENUE	DRILLING FIRM / OPERATOR: TC / B. COGER	DRILL RIG: CME 55 TRUCK	STATION / OFFSET: 5+04, 65' RT.
TYPE: BRIDGE REPLCEMENT	SAMPLING FIRM / LOGGER: TC / C. HOLLEY	HAMMER: CME AUTOMATIC	ALIGNMENT: PRITCHARD AVENUE
PID: 116200 SFN: 1560875	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 2/20/23	ELEVATION: 942.9 (MSL) EOB: 20.0 ft.
START: 11/20/23 END: 11/20/23	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 83.5	LAT / LONG: 40.77466, -80.75909

EXPLORATION ID B-001-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 942.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
AGGREGATE BASE (8")	942.2																	
DARK BROWN AND GRAY, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, **FILL**, MOIST		1	5-5-6	15	67	SS-1	1.5	-	-	-	-	-	-	-	-	12	A-6a (V)	
	939.9	2	4-5-5	14	100	SS-2	-	-	-	-	-	-	-	-	-	14	A-6a (V)	
LOOSE, BROWN, **GRAVEL AND/OR STONE FRAGMENTS WITH SAND**, LITTLE SILT, TRACE CLAY, WET	938.4	3–4; W 938.4	3-4-4	11	56	SS-3	-	37	18	23	14	8	23	17	6	15	A-1-b (0)	
STIFF, GRAY, **SILT AND CLAY**, "AND" SAND, TRACE GRAVEL, WET	936.9	5	3-5-5	14	17	SS-4	1.5	10	18	25	21	26	34	19	15	28	A-6a (4)	
VERY DENSE, GRAY, **COARSE AND FINE SAND**, SOME GRAVEL, LITTLE SILT, TRACE CLAY, CONTAINS SHALE FRAGMENTS, WET	935.4	6–7; TR	7-50/2"	-	75	SS-5	-	25	15	36	15	9	NP	NP	NP	22	A-3a (0)	
SHALE, GRAY, SEVERELY WEATHERED.		8	50/2"	-	100	SS-6	-	-	-	-	-	-	-	-	-	9	A-3a (V)	
	932.9	9	50/2"	-	100	SS-7	-	-	-	-	-	-	-	-	-	12	Rock (V)	
SHALE, GRAY, MODERATELY WEATHERED, WEAK, FINE GRAINED, VERY THIN BEDDED, ARGILLACEOUS, FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 5%, REC 100%.	930.9	10–12	64		100	NQ2-1											CORE	
SILTSTONE, GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, FINE GRAINED, VERY THIN BEDDED, ARENACEOUS, MODERATELY FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 75%, REC 100%. @14.2' TO 15.0'; Qu = 9,210 psi	925.9	13–17																
SHALE, GRAY, MODERATELY WEATHERED, WEAK, FINE GRAINED, VERY THIN BEDDED, ARGILLACEOUS, FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 61%, REC 83%. @18.0' TO 18.4'; Qu = 9,030 psi	922.9	18–20; EOB	50		92	NQ2-2											CORE	

NOTES: WATER WAS ENCOUNTERED AT 4.5 FEET DURING DRILLING

ABANDONMENT METHODS, MATERIALS, QUANTITIES: AUGER CUTTINGS MIXED WITH BENTONITE CHIPS

PROJECT: PRITCHARD AVENUE	DRILLING FIRM / OPERATOR: TC / B. COGER	DRILL RIG: CME 55 TRUCK	STATION / OFFSET: 6+25, 20' LT.
TYPE: BRIDGE REPLCEMENT	SAMPLING FIRM / LOGGER: TC / C. HOLLEY	HAMMER: CME AUTOMATIC	ALIGNMENT: PRITCHARD AVENUE
PID: 116200 SFN: 1560875	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 2/20/23	ELEVATION: 946.2 (MSL) EOB: 21.0 ft.
START: 11/20/23 END: 11/20/23	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 83.5	LAT / LONG: 40.77498, -80.75950

EXPLORATION ID B-002-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 946.2
TOPSOIL (6")	945.7
BROWN, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, **FILL**, MOIST	943.2
BROWN AND GRAY, **SANDY SILT**, LITTLE GRAVEL, LITTLE CLAY, **FILL**, WET	941.7
STIFF, BROWN AND GRAY, **SILT AND CLAY**, SOME SAND, SOME GRAVEL, SLIGHTLY ORGANIC, WET	940.2
VERY DENSE, GRAY, **COARSE AND FINE SAND**, TRACE GRAVEL, LITTLE SILT, LITTLE CLAY, CONTAINS SHALE FRAGMENTS, WET	938.7
SHALE, GRAY, SEVERELY WEATHERED.	935.2
SHALE, GRAY, MODERATELY WEATHERED, WEAK, FINE GRAINED, VERY THIN BEDDED, ARGILLACEOUS, FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 5%, REC 100%.	934.2
SILTSTONE, GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, FINE GRAINED, VERY THIN BEDDED, ARENACEOUS, MODERATELY FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 100%, REC 100%.	933.2
INTERBEDDED SHALE (70%) AND SILTSTONE (30%), FRACTURED, NARROW, SLIGHTLY ROUGH, RQD 5%, REC. 100%; **SHALE**, GRAY, MODERATELY WEATHERED, WEAK, FINE GRAINED, VERY THIN BEDDED, ARGILLACEOUS; **SILTSTONE**, GRAY, MODERATELY WEATHERED, WEAK TO MODERATELY STRONG, FINE GRAINED, VERY THIN BEDDED, ARENACEOUS, FRACTURED, NARROW, SLIGHTLY ROUGH.	929.5
SILTSTONE, GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, FINE GRAINED, VERY THIN BEDDED, ARENACEOUS, MODERATELY FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 79%, REC 100%. @17.2' TO 18.0'; Qu = 9,280 psi	927.7
SHALE, GRAY, MODERATELY WEATHERED, WEAK, FINE GRAINED, VERY THIN BEDDED, ARGILLACEOUS, FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 30%, REC 100%.	925.2

DEPTHS: W 938.7 TR; water level at 934.2 (12 ft); EOB at 21

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
4 4 4	11	100	SS-1	1.0	-	-	-	-	-	-	-	-	19	A-6a (V)
3 3 2	7	22	SS-2	0.75	-	-	-	-	-	-	-	-	20	A-6a (V)
3 3 4	10	44	SS-3	-	19	18	20	26	17	33	23	10	33	A-4a (2)
3 4 4	11	56	SS-4	1.5	30	12	19	25	14	30	19	11	44	A-6a (1)
13 50/3"	-	89	SS-5	-	5	11	52	20	12	NP	NP	NP	45	A-3a (0)
50/5"	-	80	SS-6	-	-	-	-	-	-	-	-	-	19	A-3a (V)
50/3"	-	100	SS-7	-	-	-	-	-	-	-	-	-	24	A-3a (V)
50/2"	-	100	SS-8	-	-	-	-	-	-	-	-	-	6	Rock (V)
50		100	NQ2-1											CORE
62		100	NQ2-2											CORE

NOTES: WATER WAS ENCOUNTERED AT 12 FEET DURING DRILLING, WATER WAS OBSERVED AT 12 FEET UPON COMPLETION OF DRILLING

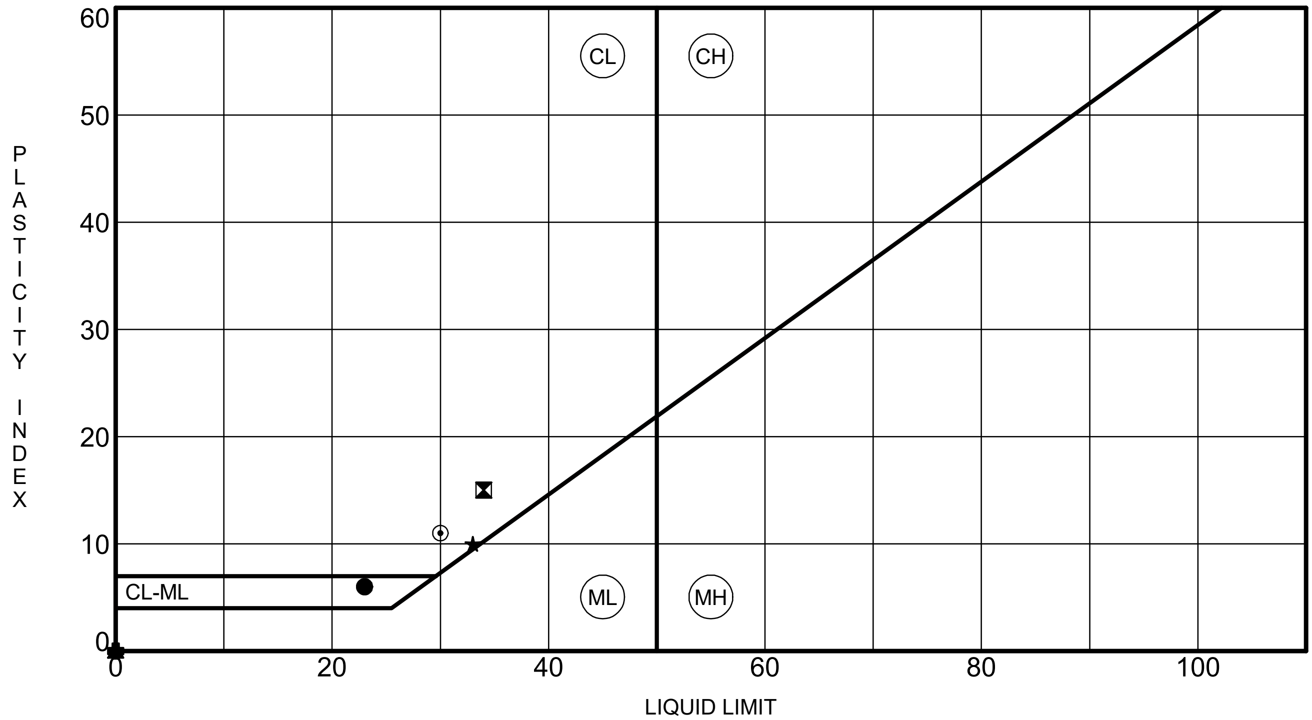
ABANDONMENT METHODS, MATERIALS, QUANTITIES: AUGER CUTTINGS MIXED WITH BENTONITE CHIPS

ATTERBERG LIMITS' RESULTS

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT PRITCHARD AVENUE **PID** 116200

OGE NUMBER N4235298 **PROJECT TYPE** STRUCTURE FOUNDATION



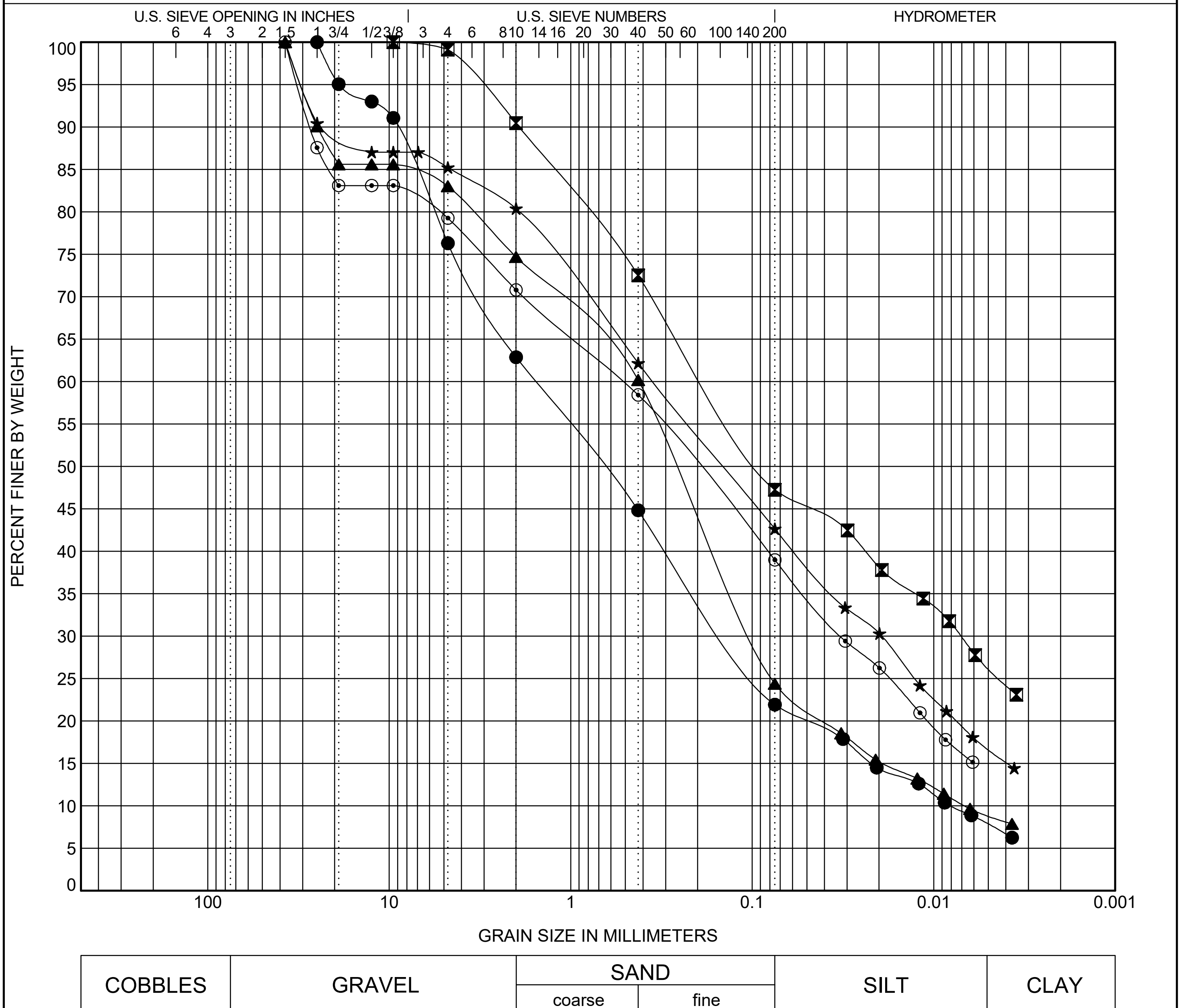
	Specimen Identification		LL	PL	PI	Fines	Classification
●	B-001-0-23	3.0	23	17	6	22	A-1-b ~ SILTY, CLAYEY SAND with GRAVEL(SC-SM)
⊠	B-001-0-23	4.5	34	19	15	47	A-6a ~ CLAYEY SAND(SC)
▲	B-001-0-23	6.0	NP	NP	NP	24	A-3a ~ SILTY SAND with GRAVEL(SM)
★	B-002-0-23	3.0	33	23	10	43	A-4a ~ CLAYEY SAND(SC)
⊙	B-002-0-23	4.5	30	19	11	39	A-6a ~ CLAYEY SAND with GRAVEL(SC)
✿	B-002-0-23	6.0	NP	NP	NP	32	A-3a ~ SILTY SAND(SM)

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT PRITCHARD AVENUE **PID** 116200

OGE NUMBER N4235298 **PROJECT TYPE** STRUCTURE FOUNDATION



Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	B-001-0-23	3.0	A-1-b ~ SILTY, CLAYEY SAND with GRAVEL(SC-SM)	23	17	6
⧗	B-001-0-23	4.5	A-6a ~ CLAYEY SAND(SC)	34	19	15
▲	B-001-0-23	6.0	A-3a ~ SILTY SAND with GRAVEL(SM)	NP	NP	NP
★	B-002-0-23	3.0	A-4a ~ CLAYEY SAND(SC)	33	23	10
⊙	B-002-0-23	4.5	A-6a ~ CLAYEY SAND with GRAVEL(SC)	30	19	11

Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-001-0-23	3.0	9.036	0.663	0.138	0.008	37	18	23	14	8	1.53	195.40
⧗	B-001-0-23	4.5	1.923	0.091	0.007		10	18	25	21	26		
▲	B-001-0-23	6.0	24.881	0.259	0.098	0.007	25	15	36	15	9	3.42	62.54
★	B-002-0-23	3.0	22.808	0.144	0.019		19	18	20	26	17		
⊙	B-002-0-23	4.5	27.066	0.2	0.032		30	12	19	25	14		

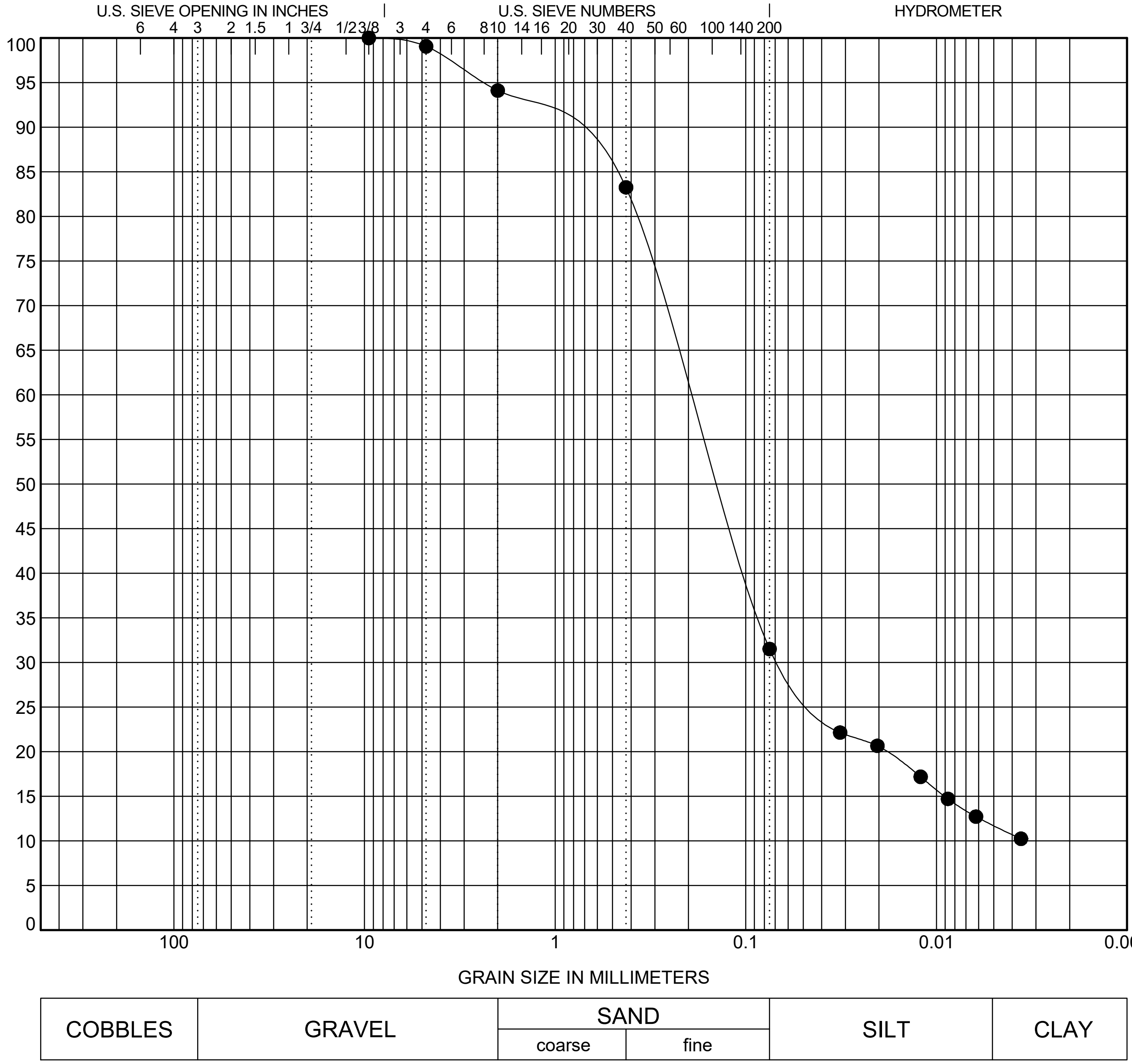
GRAIN SIZE - OH DOT.GDT - 12/12/23 15:34 - E:\PROJECTS\2023\N4235298\WORKING FILES\LABORATORY-FIELD DATA-BORING LOGS\N4235298 PRITCHARD AVENUE_ODOT.GPJ

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT PRITCHARD AVENUE **PID** 116200

OGE NUMBER N4235298 **PROJECT TYPE** STRUCTURE FOUNDATION



Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
● **B-002-0-23**	**6.0**	**A-3a ~ SILTY SAND(SM)**	**NP**	**NP**	**NP**

Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
● **B-002-0-23**	**6.0**	**1.113**	**0.139**	**0.065**		**5**	**11**	**52**	**20**	**12**		

Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying Stress and Temperatures
ASTM D 7012 Method C

Laboratory Services Group

Project No.:	N4235298		Tested By: HMR	Date:	12/13/2023
Project Name:	Pritchard Avenue Bridge Replacement		Calculated By: SAM	Date:	12/14/2023
			Checked By: SG	Date:	12/14/2023
Boring No.	B-001-0-23	Run No.: R-1			
Depth (ft):	14.2-15.0				
Description:	SILTSTONE				

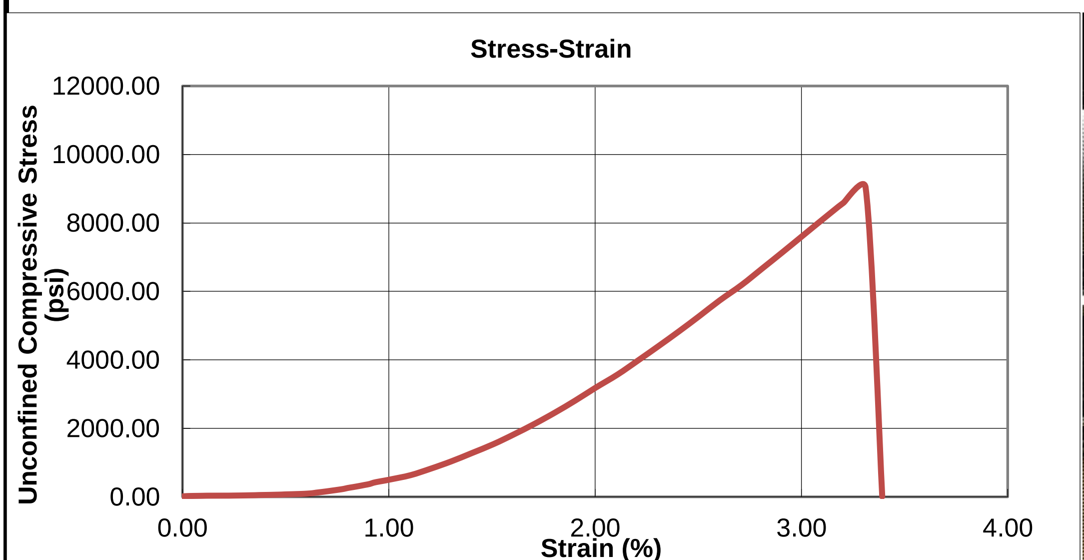
Rock Sample Moisture Condition at Test: [X] As Received [ ] See Remarks [ ] Saturated [ ] Oven Dry

TOLERANCE CHECK

Side Straightness		Maxumum Gap ≤ 0.020 in.						Tolerance Met	Yes
End Flatness: Max.	Diameter 1a	0.0003	in	Diameter 1b	0.0009	in	≤ 0.0020	Tolerance Met	Yes
End Flatness: Max.	Diameter 2a	0.0001	in	Diameter 2b	0.0004	in	≤ 0.0020	Tolerance Met	Yes
Perpendicularity Slope	Diameter 1a	0.00015		Diameter 1b	0.00005		≤ 0.0043	Tolerance Met	Yes
Perpendicularity Slope	Diameter 2a	0.00046		Diameter 2b	0.00020		≤ 0.0043	Tolerance Met	Yes

	1)	2)	3)	Avg.	
Length (in):	3.957	3.956	3.956	3.956	in
Diameter (in):	1.972	1.970	1.975	1.972	in

Uniaxial Compressive Strength:	**9,207**	psi	Mass:	467.83	g
Load:	**28,129**	lbs.	**Wet Unit Weight:**	**147.4**	pcf
L/D:	**2.0**		**Dry Unit Weight:**	**146.8**	pcf
Water Content:	**0.4**	%			



Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying Stress and Temperatures
ASTM D 7012 Method C

Laboratory Services Group

Project No.:	N4235298	Tested By:	SAM	Date:	12/15/2023
Project Name:	Pritchard Ave. Bridge Replacement	Calculated By:	SAM	Date:	12/15/2023
		Checked By:	SG	Date:	12/15/2023

Boring No. B-001-0-23 Run No.: R-2
Depth (ft): 18.0'-18.4'
Description: SHALE

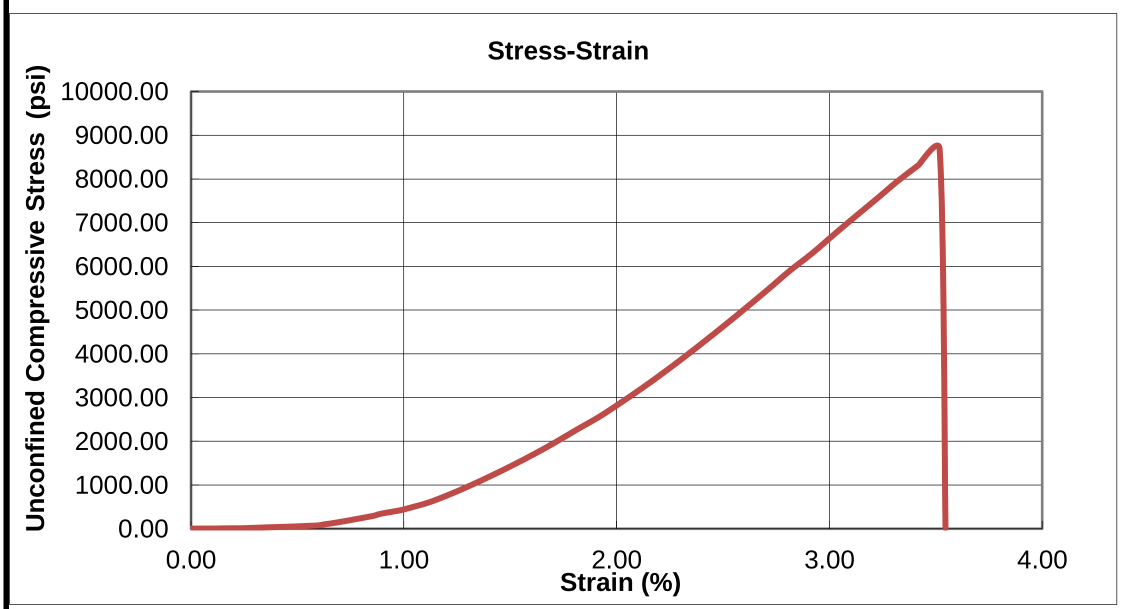
Rock Sample Moisture Condition at Test: [X] As Received [ ] See Remarks [ ] Saturated [ ] Oven Dry

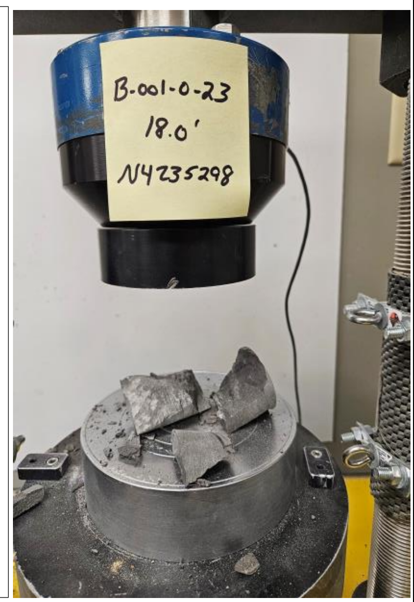
TOLERANCE CHECK

Side Straightness	Maxumum Gap ≤ 0.020 in.							Tolerance Met	Yes
End Flatness: Max.	Diameter 1a	0.0001	in	Diameter 1b	0.0005	in	≤ 0.0020	Tolerance Met	Yes
End Flatness: Max.	Diameter 2a	0.0006	in	Diameter 2b	0.0002	in	≤ 0.0020	Tolerance Met	Yes
Perpendicularity Slope	Diameter 1a	0.00005		Diameter 1b	0.00031		≤ 0.0043	Tolerance Met	Yes
Perpendicularity Slope	Diameter 2a	0.00025		Diameter 2b	0.00010		≤ 0.0043	Tolerance Met	Yes

	1)	2)	3)	Avg.	
Length (in):	3.982	3.964	3.961	3.972	in
Diameter (in):	1.970	1.960	1.970	1.967	in

Uniaxial Compressive Strength:	**9,033**	psi	Mass:	505.25	g
Load:	**27,440**	lbs.	**Wet Unit Weight:**	**161.1**	pcf
L/D:	**3.0**		**Dry Unit Weight:**	**159.5**	pcf
Water Content:	**0.1**	%			





Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying Stress and Temperatures
ASTM D 7012 Method C

Laboratory Services Group

Project No.: N4235298
Project Name: Pritchard Ave. Bridge Replacement

Boring No. B-002-0-23 Run No.: R-2
Depth (ft): 17.2-18.0
Description: SILTSTONE

Tested By: SAM Date: 12/13/2023
Calculated By: SAM Date: 12/14/2023
Checked By: SG Date: 12/14/2023

Rock Sample Moisture Condition at Test: [X] As Received [ ] See Remarks [ ] Saturated [ ] Oven Dry

TOLERANCE CHECK

Side Straightness		Maxumum Gap ≤ 0.020 in.						Tolerance Met	Yes
End Flatness: Max.	Diameter 1a	0.0006	in	Diameter 1b	0.0010	in	≤ 0.0020	Tolerance Met	Yes
End Flatness: Max.	Diameter 2a	0.0004	in	Diameter 2b	0.0010	in	≤ 0.0020	Tolerance Met	Yes
Perpendicularity Slope	Diameter 1a	0.00031		Diameter 1b	0.00020		≤ 0.0043	Tolerance Met	Yes
Perpendicularity Slope	Diameter 2a	0.00051		Diameter 2b	0.00051		≤ 0.0043	Tolerance Met	Yes

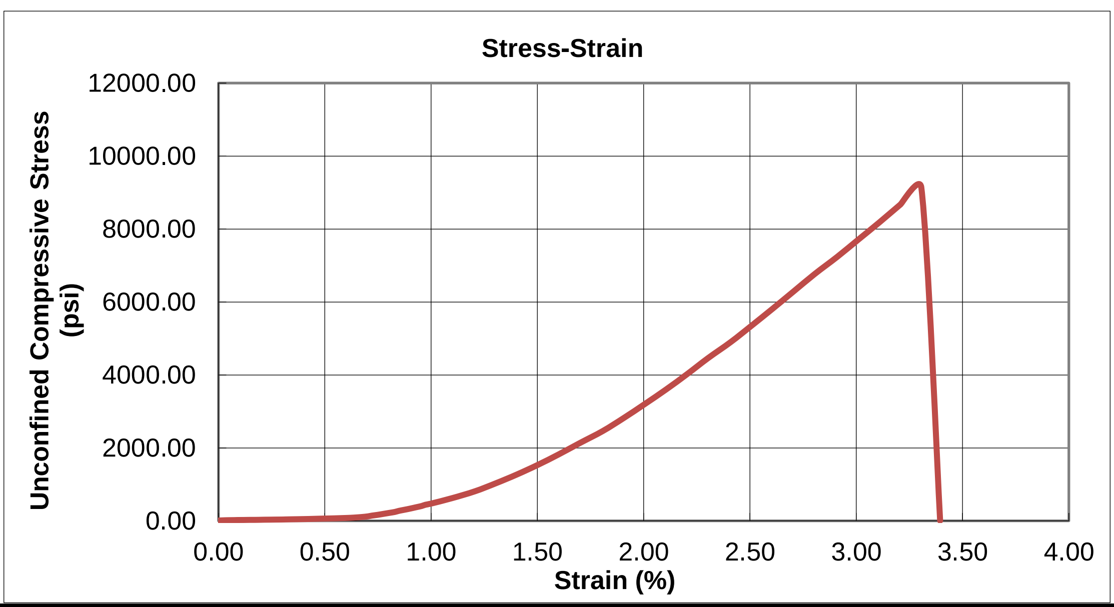
	1)	2)	3)	Avg.	
Length (in):	4.021	4.021	4.023	4.022	in
Diameter (in):	1.952	1.955	1.967	1.958	in

Uniaxial Compressive Strength: 9,279 psi — Mass: 466.12 g

Load: 27,940 lbs. — **Wet Unit Weight:** 146.6 pcf

L/D: 2.1 — **Dry Unit Weight:** 145.9 pcf

Water Content: 0.5 %



SLAKE DURABILITY INDEX (SDI) TEST SUMMARY (ASTM D4644)

Terracon

Client:	EMH&T - Columbus, OH	Date:	12/15/2023
Project:	Pritchard Avenue Bridge Replacement	Project Number:	N4235298
Location:	Lisbon, OH		

Boring No.			B-001-0-23
Depth (ft)			13.3'
Tare Weight:			840.0
Moist weight (Sample+Tare):			1489.98
Dry weight (Sample+Tare):			1478.91
Natural Moisture Content (%):			1.7
After Cycle No. 1			
Temperature (°F)			Dry Weight (Sample+Tare)
Start	End	Average	
92.8	90.7	91.8	1475.2
After Cycle No. 2			
Temperature (°F)			Dry Weight (Sample+Tare)
Start	End	Average	
70.9	74.3	72.6	1471.8
SLAKE DURABILITY INDEX:			**98.9**
Fragments Retained - Type:			**I**

Before Test

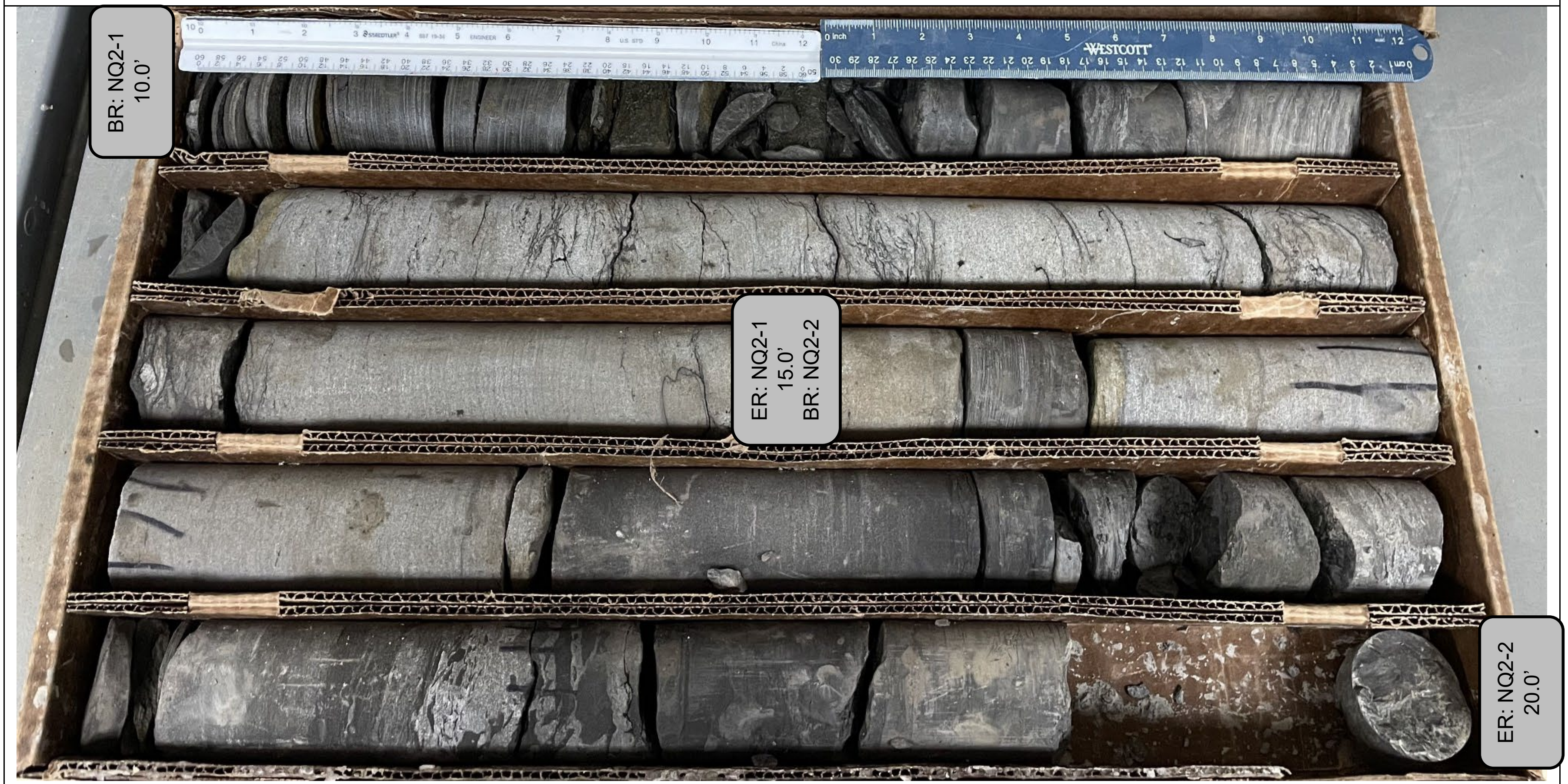
After Test

Material Description:	**SANDSTONE**
Notes/Comments:	

Ohio Department of Transportation
Division of Engineering

Office of Geotechnical Engineering

B-001-0-23



Run #:	Depth		Recovery		RQD	
NQ2-1	10.0'	15.0'	60/60	100%	38/60	64%
NQ2-2	15.0'	20.0'	55/60	92%	30/60	50%
Pritchard Avenue Bridge (PID 116200)						

ODOT Ohio Department of Transportation
Division of Engineering

Office of Geotechnical Engineering

B-002-0-23

Run #:	Depth		Recovery		RQD	
NQ2-1	11.0’	16.0’	60/60	100%	30/60	50%
NQ2-2	16.0’	21.0’	60/60	100%	37/60	62%
Pritchard Avenue Bridge (PID 116200)						

WILDCAT DYNAMIC CONE LOG

Terracon Consultants Inc.
800 Morrison Road
Columbus, OH 43230

PROJECT NUMBER: N4235298
DATE STARTED: 1.17.24
DATE COMPLETED: 1.17.24

HOLE #: D-001-0-24
CREW: A. Bolek, Z. Coman
PROJECT: Pritchard Avenue Bridge Replacement (PID 116200)
ADDRESS: Pritchard Avenue
LOCATION: Lisbon, OH

SURFACE ELEVATION: 943 feet
WATER ON COMPLETION: None observed
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY NON-COHESIVE	TESTED CONSISTENCY COHESIVE
-	39	173.2	••	25+	DENSE	HARD
-	34	151.0	••	25+	DENSE	HARD
- 1 ft	8	35.5	••••••••••	10	LOOSE	STIFF
-	8	35.5	••••••••••	10	LOOSE	STIFF
-	13	57.7	•••••••••••••••	16	MEDIUM DENSE	VERY STIFF
- 2 ft	7	31.1	•••••••••	8	LOOSE	MEDIUM STIFF
-	6	26.6	•••••••	7	LOOSE	MEDIUM STIFF
-	24	106.6	••••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
- 3 ft	12	53.3	••••••••••••••	15	MEDIUM DENSE	STIFF
- 1 m	24	106.6	••••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
-	25	96.5	•••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
- 4 ft	15	57.9	•••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-	12	46.3	••••••••••••	13	MEDIUM DENSE	STIFF
-	10	38.6	••••••••••	11	MEDIUM DENSE	STIFF
- 5 ft	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
-	23	88.8	•••••••••••••••••••••••	25	MEDIUM DENSE	VERY STIFF
-	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
- 6 ft	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
-	10	38.6	••••••••••	11	MEDIUM DENSE	STIFF
- 2 m	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
- 7 ft	8	27.4	•••••••	7	LOOSE	MEDIUM STIFF
-	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
-	7	23.9	••••••	6	LOOSE	MEDIUM STIFF
- 8 ft	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	10	34.2	•••••••••	9	LOOSE	STIFF
-	20	68.4	••••••••••••••••••	19	MEDIUM DENSE	VERY STIFF
- 9 ft	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
-	55	188.1	••	25+	VERY DENSE	HARD
-						
- 3 m 10 ft						
-						
-						
-						
- 11 ft						
-						
-						
- 12 ft						
-						
-						
- 4 m 13 ft						

WILDCAT DYNAMIC CONE LOG

Terracon Consultants Inc.
800 Morrison Road
Columbus, OH 43230

PROJECT NUMBER: N4235298
DATE STARTED: 1.17.24
DATE COMPLETED: 1.17.24

HOLE #: D-002-0-24
CREW: A. Bolek, Z. Coman
PROJECT: Pritchard Avenue Bridge Replacement (PID 116200)
ADDRESS: Pritchard Avenue
LOCATION: Lisbon, OH

SURFACE ELEVATION: 947 feet
WATER ON COMPLETION: None observed
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY NON-COHESIVE	COHESIVE
-	43	190.9	•••	25+	VERY DENSE	HARD
-	8	35.5	••••••••••	10	LOOSE	STIFF
- 1 ft	5	22.2	••••••	6	LOOSE	MEDIUM STIFF
-	7	31.1	•••••••••	8	LOOSE	MEDIUM STIFF
-	4	17.8	•••••	5	LOOSE	MEDIUM STIFF
- 2 ft	2	8.9	••	2	VERY LOOSE	SOFT
-	1	4.4	•	1	VERY LOOSE	VERY SOFT
-	2	8.9	••	2	VERY LOOSE	SOFT
- 3 ft	11	48.8	••••••••••••••	13	MEDIUM DENSE	STIFF
- 1 m	13	57.7	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
- 4 ft	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
-	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
-	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
- 5 ft	10	38.6	•••••••••••	11	MEDIUM DENSE	STIFF
-	9	34.7	••••••••••	9	LOOSE	STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
- 6 ft	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
- 2 m	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
- 7 ft	3	10.3	••	2	VERY LOOSE	SOFT
-	3	10.3	••	2	VERY LOOSE	SOFT
-	3	10.3	••	2	VERY LOOSE	SOFT
- 8 ft	3	10.3	••	2	VERY LOOSE	SOFT
-	3	10.3	••	2	VERY LOOSE	SOFT
-	4	13.7	•••	3	VERY LOOSE	SOFT
- 9 ft	4	13.7	•••	3	VERY LOOSE	SOFT
-	8	27.4	•••••••	7	LOOSE	MEDIUM STIFF
-	24	82.1	•••••••••••••••••••••••	23	MEDIUM DENSE	VERY STIFF
- 3 m 10 ft	27	92.3	••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
-	27	82.6	•••••••••••••••••••••••	23	MEDIUM DENSE	VERY STIFF
-	10	30.6	••••••••	8	LOOSE	MEDIUM STIFF
-	60	183.6	•••	25+	VERY DENSE	HARD
- 11 ft						
-						
-						
- 12 ft						
-						
-						
- 4 m 13 ft						

Appendix C – Supporting Information

Contents:

Drilled Shaft Tip and Side Resistance Calculations (8 pages)
Evaluation of Non-Scourable Bedrock Depth (2 pages)
Unified Soil Classification System
ODOT Quick Reference for Visual Description of Soils
ODOT Classification of Soils
ODOT Quick Reference Guide for Rock Description (2 pages)

Note: All attachments are one page unless noted above.

CLIENT:	EMH&T
PROJECT:	Pritchard Avenue Bridge Replacement (PID 116200)
W.O.:	N4235298
Date:	11/3/2023
Case:	B-001-0-23

UCS and RQD Data

Boring	RQD (per 10 ft.)
B-001-0-23	50
	62

Average	56

Boring	Depth	UCS (ksf)
B-001-0-23	38.3	1326
		1300

Average	1313

Rock Mass Rating (RMR) per LRFD Table 10.4.6.4-1 Per Section 10 LRFD 5th Edition

B-001-0-23	Relative Rating
Strength of Rock	
1313	7
RQD	
56	13
Maximum Spacing of Joints (in.)	
12	10
Condition of Joints	12
GW Condition	4
Total	46
Adj for Joint Orientation	7
RMR	39
Class	4
Rock Type	Poor Rock
GSI	40

from 10.4.6.4-2

Choose appropriate m and s paramters Per LRFD Table 10.4.6.4-4

m	0.183
s	0.00009

TIP RESISTANCE USING GSI

mi	9		from Table 10.4.6.4-1
mb	1.06		from eq 10-25
s	0.0013		from eq 10-26
a	0.5114		from eq 10-27
σ'bv	3.6	ksf	from D-2 boring
A	82		using eq 10-28
Nominal Base Resistance, Qbn	411	ksf	using eq 10-29

Side Resistance per LRFD 10.8.3.5.4b

$q_s = 0.65\ \alpha_E\ Pa\ (q_u/P_a)^{0.5} < 7.8\ P_a\ (f'_c / P_a)^{0.5}$ (q_s = shaft resistance in ksf)

α_E = reduction factor to account for the jointing in rock

E_m =	769.78	ksi	(Elastic modulus of the rock mass per Eq. 10.4.6.5-1)
E_i =	2500	ksi	(Elastic modulus of intact rock)
E_m / E_i =	0.31		

αE =	0.8		
P_a =	2.12	ksf	(Atmospheric Pressure)
f'_c =	3	ksi	(Concrete compressive strength)
q_u =	1313	ksf	(Uniaxial compressive strength of rock)

q_s = 27.43 < 19.67 NOT OK

Therefore,
q_s = 19.67 ksf

Resistance Factors for Drilled Shafts per LRFD Table 10.5.5.2.4-1

Side Resistance in Rock (Horwarth and Kenney (1979)	0.55
Uplift Resistance in Rock (Horwath and Kenney (1979)	0.4

Factored Drilled Caisson Capacities

Factored Side Resistance =	**10.8**	**ksf**
Factored Uplift Resistance =	**7.9**	**ksf**

Per WVDOH for RMR < 50, use Bieniawski (1989) method to evaluate c and φ_f of the weak rock

$c =$ 104 x RMR (psf)

= 4056 psf

= 4.056 ksf

$\varphi_f =$ 5 + RMR/2

= 25 °

Per LRFD Eq. 10.6.3.1.2a-1 nominal bearing resistance of soil/weak rock in ksf is taken as

$q_n = c\ Ncm + \gamma Df\ Nqm\ Cwq + 0.5\gamma\ BN\gamma m\ Cw\gamma$

in which

Ncm = Nc Sc ic

Nqm = Nq Sq dq iq

Nγm = Nγ Sγ iγ

c =	4.06			
Nc =	20.7	From Table 10.6.3.1.2a-1		
Nq =	10.7	From Table 10.6.3.1.2a-1		
Nγ =	10.9	From Table 10.6.3.1.2a-1		
γ =	120			
Df =	2			
B =	3	L =	3	
Cwq, Cwγ =	1	(correction factors for location of GW, assume 1 for rock)		
Sc =	1.52			
Sγ =	0.60			
Sq =	1.46			
dq =	1	(assume 1 for rock)		
ic, iγ, iq =	1	(since inclination of load is unknown)		

$q_n =$ 132.3 ksf

Resistance Factors for Tip Resistance per LRFD Table 10.5.5.2.2-1

Resistance Factor for Bearing in rock	0.5

Factored Bearing Resistance =	**66.1**	**ksf**

CLIENT:	EMH&T
PROJECT:	Pritchard Avenue Bridge Replacement (PID 116200)
W.O.:	N4235298
Date:	11/3/2023
Case:	B-002-0-23

UCS and RQD Data

Boring	RQD (per 10 ft.)
B-002-0-23	50
	62

Average	56

Boring	Depth	UCS (ksf)
B-002-0-23	19.5	1336

Average	1336

Rock Mass Rating (RMR) per LRFD Table 10.4.6.4-1 Per Section 10 LRFD 5th Edition

B-002-0-23	Relative Rating
Strength of Rock	
1336	7
RQD	
56	13
Maximum Spacing of Joints (in.)	
12	10
Condition of Joints	12
GW Condition	4
Total	46
Adj for Joint Orientation	7
RMR	39
Class	4
Rock Type	Poor Rock
GSI	40

from 10.4.6.4-2

Choose appropriate m and s paramters Per LRFD Table 10.4.6.4-4

m	0.183
s	0.00009

TIP RESISTANCE USING GSI

mi	13		from Table 10.4.6.4-1
mb	1.53		from eq 10-25
s	0.0013		from eq 10-26
a	0.5114		from eq 10-27
σ'bv	3.6	ksf	from D-2 boring
A	83		using eq 10-28
Nominal Base Resistance, Qbn	485	ksf	using eq 10-29

Side Resistance per LRFD 10.8.3.5.4b

$q_s = 0.65\ \alpha_E\ Pa\ (q_u/P_a)^{0.5} < 7.8\ P_a\ (f'_c\ /\ P_a)^{0.5}$ (q_s = shaft resistance in ksf)

α_E = reduction factor to account for the jointing in rock

E_m =	769.78	ksi	(Elastic modulus of the rock mass per Eq. 10.4.6.5-1)
E_i =	2500	ksi	(Elastic modulus of intact rock)
E_m / E_i =	0.31		

αE =	0.8		
P_a =	2.12	ksf	(Atmospheric Pressure)
f'_c =	3	ksi	(Concrete compressive strength)
q_u =	1336	ksf	(Uniaxial compressive strength of rock)

q_s = 27.67 < 19.67 NOT OK

Therefore,
q_s = 19.67 ksf

Resistance Factors for Drilled Shafts per LRFD Table 10.5.5.2.4-1

Side Resistance in Rock (Horwarth and Kenney (1979)	0.55
Uplift Resistance in Rock (Horwath and Kenney (1979)	0.4

Factored Drilled Caisson Capacities

Factored Side Resistance =	**10.8**	**ksf**
Factored Uplift Resistance =	**7.9**	**ksf**

Per WVDOH for RMR < 50, use Bieniawski (1989) method to evaluate c and φ_f of the weak rock

c = 104 x RMR (psf)

= 4056 psf

= 4.056 ksf

φ_f = 5 + RMR/2

= 25 °

Per LRFD Eq. 10.6.3.1.2a-1 nominal bearing resistance of soil/weak rock in ksf is taken as

q_n = c Ncm + γDf Nqm Cwq + 0.5γ BNγm Cwγ

in which

Ncm = Nc Sc ic

Nqm = Nq Sq dq iq

Nγm = Nγ Sγ iγ

c = 4.06

Nc = 20.7 From Table 10.6.3.1.2a-1

Nq = 10.7 From Table 10.6.3.1.2a-1

Nγ = 10.9 From Table 10.6.3.1.2a-1

γ = 120

Df = 2

B = 3 L = 3

Cwq, Cwγ = 1 (correction factors for location of GW, assume 1 for rock)

Sc = 1.52

Sγ = 0.60

Sq = 1.46

dq = 1 (assume 1 for rock)

ic, iγ, iq = 1 (since inclination of load is unknown)

q_n = 132.3 ksf

Resistance Factors for Tip Resistance per LRFD Table 10.5.5.2.2-1

Resistance Factor for Bearing in rock	0.5

Factored Bearing Resistance =	**66.1**	**ksf**

Evaluation of Non-Scourable Rock Depth Per ODOT BDM sec: 305.2.1.2.b(3)

Boring B-001-0-23 Data :

- Top of rock elevation = 935.4 (7.5') * Top of Intact bedrock Elev. = 932.9 (10')

Run 1: 10 to 15 feet REC = 100% RQD = 64% shale/siltstone

Run 2: 15 to 20 feet REC = 92% RQD = 50% shale/siltstone

1- Q_u = 9,210 psi, 9,030 psi > 2500 psi ✓

2- Slake Durability Index = 98.9% > 90% ✓

3- RQD = 50% < 65% ×

4- Total Unit weight = 106.2 pcf < 150 pcf ×

5- RMR = 56 < 75 × ← AASHTO LRFD (Sabatini et al. 2002)

6- Erodibility Index $K = (M_s)(K_b)(K_d)(J_s)$ ← FHWA-HIF-12.003 (HEC 18) sec 4.7.2

Table 4.22 $M_s = 70$ $K_b = \frac{RQD}{J_n}$ $J_n = 3.34$ table 4.23

$K_d = \frac{J_r}{J_a} = \frac{1.5}{6} = 0.25$ $\Rightarrow$ $\boxed{K_b = \frac{50}{3.34} = 14.97}$

$J_r = 1.5$ Table 4.24 $J_a = 6$ table 4.25 $J_s = 1$ Table 4.26

$\Rightarrow K = (70)(14.97)(0.25)(1) = 261.98 > 100$ ✓

7- Shale was considered in the Analysis. ✓

8. No Ordovician rock formations. ✓

Rock is Non Scour Resistant $\Rightarrow$ Scourable to 25'

Terracon

PROJECT: Pritchard Avenue Bridge Replacement (PID 116200) Page 2 of 2

JOB NO. N4235298 Date 2/9/2024 Comp. By AAH CHECKED BY: KME

Non scourable Rock Depth

Boring B-002-0-23 Data:

- Top of rock elevation = 938.7 (7.5') - Top of intact rock Elev. = 935.2 (11

Run 1: 11 to 16 feet REC = 100% RQD = 50 shale/siltst

Run 2: 16 to 21 feet REC = 100% RQD = 62 shale/siltst

1- Q_u = 9000 psi > 2500 psi ✓

2- Slake Durability Index = 98.9% > 90% ✓

3- RQD = 50% < 65% ×

4- Total Unit weight = 106.2 pcf < 150 pcf ×

5- RMR = 56 < 65% ×

6- Erodibility Index $K = (70)(14.97)(0.25)(1) = 261.98 > 100$ ✓

$M_s = 70$ $K_b = \frac{50}{3.34} = 14.97$ $J_n = 3.34$ $K_d = \frac{1.5}{6} = 0.25$

$J_s = 1$

7- Shale was considered in the Analysis. ✓

8- No Ordovician Rock formation. ✓

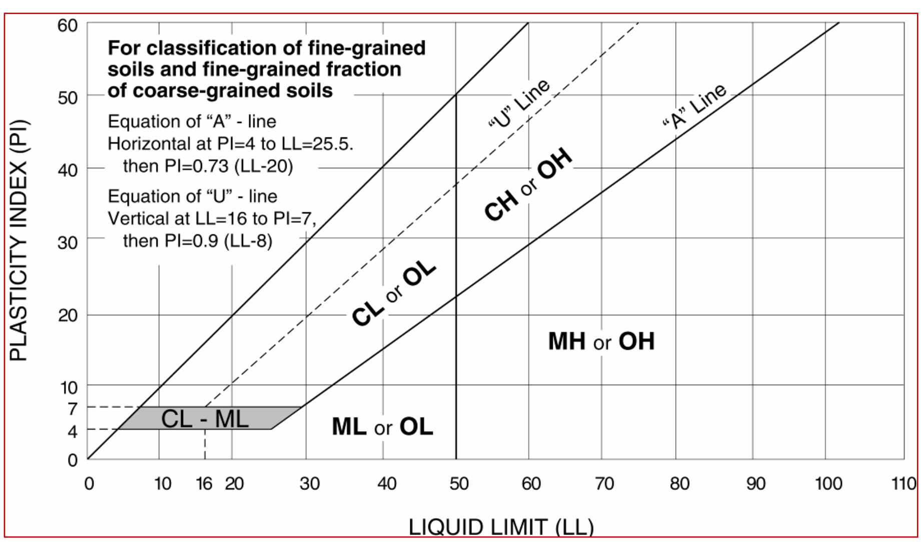
Rock is Non Scour Resistant ⇒ Scourable to 26'

Bottom of footing should be located one foot below the rock scour depth as calculated according to HEC 18 (Scour analysis is Required to Determine Scour depth).

Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	**Group Name** [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [E]	GW	Well-graded gravel [F]
			$Cu < 4$ and/or $[Cc < 1$ or $Cc > 3.0]$ [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]	SW	Well-graded sand [I]
			$Cu < 6$ and/or $[Cc < 1$ or $Cc > 3.0]$ [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots above "A" line [J]	CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



APPENDIX A.1 - ODOT Quick Reference for Visual Description of Soils

1) Strength of Soil:

Non-Cohesive (granular) Soils - Compactness

Description	Blows Per Ft.
Very Loose	≤ 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	> 50

Cohesive (fine grained) Soils - Consistency

Description	Qu (TSF)	Blows Per Ft.	Hand Manipulation
Very Soft	<0.25	<2	Easily penetrates 2” by fist
Soft	0.25-0.5	2 - 4	Easily penetrates 2” by thumb
Medium Stiff	0.5-1.0	5 - 8	Penetrates by thumb with moderate effort
Stiff	1.0-2.0	9 - 15	Readily indents by thumb, but not penetrate
Very Stiff	2.0-4.0	16 - 30	Readily indents by thumbnail
Hard	>4.0	>30	Indent with difficulty by thumbnail

2) Color :

If a color is a uniform color throughout, the term is single, modified by an adjective such as light or dark. If the predominate color is shaded by a secondary color, the secondary color precedes the primary color. If two major and distinct colors are swirled throughout the soil, the colors are modified by the term “mottled”

3) Primary Component

Use **DESCRIPTION** from ODOT Soil Classification Chart on Back

4) Component Modifiers:

Description	Percentage By Weight
Trace	0% - 10%
Little	>10% - 20%
Some	>20% - 35%
“And”	>35%

5) Soil Organic Content

Description	% by Weight
Slightly Organic	2% - 4%
Moderately Organic	4% - 10%
Highly Organic	> 10%

6) Relative Visual Moisture

Description	Criteria	
	Cohesive Soil	**Non-cohesive Soils**
Dry	Powdery; Cannot be rolled; Water content well below the plastic limit	No moisture present
Damp	Leaves very little moisture when pressed between fingers; Crumbles at or before rolled to $^1/_8$”; Water content below plastic limit	Internal moisture, but no to little surface moisture
Moist	Leaves small amounts of moisture when pressed between fingers; Rolled to $^1/_8$” or smaller before crumbling; Water content above plastic limit to -3% of the liquid limit	Free water on surface, moist (shiny) appearance
Wet	Very mushy; Rolled multiple times to $^1/_8$” or smaller before crumbles; Near or above the liquid limit	Voids filled with free water, can be poured from split spoon.

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	Classifcation AASHTO	Classifcation OHIO	$LL_O/LL \times 100$*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
	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	
F S	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.	$\leqq$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
- P Peat

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

APPENDIX A.2 – ODOT Quick Reference Guide for Rock Description

1: ROCK TYPE: Common rock types are: Claystone; Coal; Dolomite; Limestone; Sandstone; Siltstone; & Shale.

2: COLOR: To be determined when rock is wet. When using the GSA Color charts use only Name, not code.

3: WEATHERING

Description	Field Parameter
Unweathered	No evidence of any chemical or mechanical alternation of the rock mass. Mineral crystals have a bright appearance with no discoloration. Fractures show little or no 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 too 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.

4: TEXTURE

Component		Grain Diameter
Boulder		>12"
Cobble		3"-12"
Gravel		0.08"-3"
Sand	Coarse	0.02"-0.08"
Sand	Medium	0.01"-0.02"
Sand	Fine	0.005"-0.01"
Sand	Very Fine	0.003"-0.005"

5: RELATIVE STRENGTH

Description	Field Parameter
Very Weak	Core can be carved with a knife and scratched by fingernail. Can be excavated readily with a point of a pick. Pieces 1 inch or more in thickness can be broken by finger pressure.
Weak	Core can be grooved or gouged readily by a knife or pick. Can be excavated in small fragments by moderate blows of a pick point. Small, thin pieces can be broken by finger pressure.
Slightly Strong	Core can be grooved or gouged 0.05 inch deep by firm pressure of a knife or pick point. Can be excavated in small chips to pieces about 1-inch maximum size by hard blows of the point of a geologist's pick.
Moderately Strong	Core can be scratched with a knife or pick. Grooves or gouges to ¼" deep can be excavated by hand blows of a geologist's pick. Requires moderate hammer blows to detach hand specimen.
Strong	Core can be scratched with a knife or pick only with difficulty. Requires hard hammer blows to detach hand specimen. Sharp and resistant edges are present on hand specimen.
Very Strong	Core cannot be scratched by a knife or sharp pick. Breaking of hand specimens requires hard repeated blows of the geologist hammer.
Extremely strong	Core cannot be scratched by a knife or sharp pick. Chipping of hand specimens requires hard repeated blows of the geologist hammer.

6: BEDDING

Description	Thickness
Very Thick	>36"
Thick	18" – 36"
Medium	10" – 18"
Thin	2" – 10"
Very Thin	0.4" – 2"
Laminated	0.1" – 0.4"
Thinly Laminated	<0.1"

7: DESCRIPTORS

Arenaceous – sandy	Argillaceous - clayey	Brecciated – contains angular to subangular gravel
Calcareous - contains calcium carbonate	Carbonaceous - contains carbon	Cherty- contains chert fragments
Conglomeritic - contains rounded to subrounded gravel	Crystalline – contains crystalline structure	Dolomitic- contains calcium/magnesium carbonate
Ferriferous – contains iron	Fissile – thin planner partings	Fossiliferous – contains fossils
Friable – easily broken down	Micaceous – contains mica	Pyritic – contains pyrite
Siliceous – contains silica	Stylolitic – contain stylotites (suture like structure)	Vuggy – contains openings

APPENDIX A.2 – ODOT Quick Reference Guide for Rock Description

8: Discontinuities

a: Discontinuity Types

Type	Parameters
Fault	Fracture which expresses displacement parallel to the surface that does not result in a polished surface.
Joint	Planar fracture that does not express displacement. Generally occurs at regularly spaced intervals.
Shear	Fracture which expresses displacement parallel to the surface that results in polished surfaces or slickensides.
Bedding	A surface produced along a bedding plane.
Contact	A surface produced along a contact plane. (generally not seen in Ohio)

b: Degree of Fracturing

Description	Spacing
Unfractured	> 10 ft.
Intact	3 ft. – 10 ft.
Slightly fractured	1 ft. – 3 ft.
Moderately fractured	4 in. – 12 in.
Fractured	2 in. – 4 in.
Highly fractured	< 2 in.

c: Aperture Width

Description	Spacing
Open	> 0.2 in.
Narrow	0.05 in. - 0.2 in.
Tight	<0.05 in.

d: Surface Roughness

Description	Criteria
Very Rough	Near vertical steps and ridges occur on the discontinuity surface.
Slightly Rough	Asperities on the discontinuity surface are distinguishable and can be felt.
Slickensided	Surface has a smooth, glassy finish with visual evidence of striation.

11: Recovery

$Run\ Recovery = \left(\frac{R_R}{L_R}\right) * 100$	$Unit\ Recovery = \left(\frac{R_U}{L_U}\right) * 100$
L_R = Run Length R_R – Run Recovery	L_U = Rock Unit Length R_U – Rock Unit Recovery

9: GSI Description

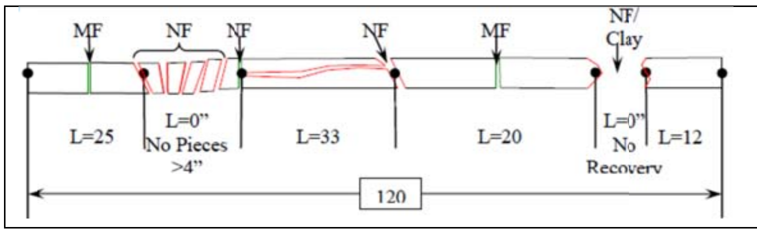
a: Structure

Description	Parameters
Intact or Massive	Intact rock with few widely spaced discontinuities
Blocky	Well interlocked undisturbed rock mass consisting of cubical blocks formed by three interesting discontinuity sets
Very Blocky	Interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets
Blocky/Disturbed/ Seamy	Angular blocks formed by many intersecting discontinuity sets, Persistence of bedding planes
Disintegrated	Poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces
Laminated/Sheared	Lack of blockiness due to close spacing of weak shear planes

b: Surface Condition

Description	Parameters
Very Good	Very rough, fresh unweathered surfaces
Good	Rough, slightly weathered, iron stained surface
Fair	Smooth, moderately weathered and altered surfaces
Poor	Slickensided, highly weathered surface with compact coatings or fillings or angular fragments
Very Poor	Slickensided, highly weathered surfaces with soft clay coating or fillings

10: RQD



$$RQD = \left(\frac{\sum Length\ \ of\ \ Pieces > 4inches}{Total\ Length\ of\ Core}\right) * 100$$

$$RQD = \left(\frac{25+33+20+12}{120}\right) * 100 = 75\%$$