JEF-Reese Street Bridge Replacement (PID 119880)

Geotechnical Engineering Report – Rev.1

Irondale, Jefferson County, Ohio

September 19, 2024 | Terracon Project No. N4235428

Prepared for:

EMH&T Engineers, Surveyors, Planners, Scientists
5500 New Albany Road
Columbus, Ohio 43054

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September 19, 2024

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Attn: Mr. Tyler Adams, Senior Bridge Engineer
P: (614) 775-4602
E: Tadams@emht.com

Re: Geotechnical Engineering Report - Rev.1
JEF-Reese Street Bridge Replacement (PID 119880)
Reese Street
Irondale, Jefferson County, Ohio
Terracon Project No. N4235428

Dear Mr. Adams:

Terracon Consultants, Inc. (Terracon) has completed the scope of Geotechnical Engineering Services for the above reference in general accordance with Terracon Proposal No. PN4235428 dated October 9, 2023.

This revised report was prepared based on the email received from Mr. Tyler Adams on March 15, 2024 with the request to add the drilled shaft deep foundations recommendations. Another request was received in the email received from Mr. Tyler Adams on March 23, 2024 with the request to revise the scour data and include more scour parameters for soils and bedrock. 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 Reese Street bridge located at Jefferson County, Ohio.

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

Ahmad Al-Hosainat, Ph.D.
Senior Staff Engineer

Kevin M. Ernst, P.E.
Principal, Regional Manager

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Appendices

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

JEF-Reese Street Bridge Replacement (PID 119880)
Irondale, Jefferson County, Ohio
Terracon Project No. N4235428
September 19, 2024

Executive Summary

This report presents the findings of our geotechnical exploration performed for the proposed replacement of the existing bridge located along Reese Street, south of the intersection of Reese Street and Saline Street in Irondale, Jefferson County, Ohio. The bridge is located approximately 35 feet to the south of Saline Street. The existing structure is a single-span weathering steel beam bridge with steel decking and asphalt concrete wearing surface on concrete gravity wall abutments. The existing bridge has a maximum span of approximately 23 feet center to center of bearings. The proposed replacement structure is anticipated to include new concrete footers, headwall stems, and three-sided precast concrete box culvert. 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 Reese Street bridge to approximate depths of 23.9 to 43.9 feet below the existing ground surface. Borings B-001-0-23 and B-002-0-23 encountered topsoil approximately 9 to 10 inches thick. The borings encountered fill materials to depths varying from about 3 to 6 feet below the existing ground surface. The fill materials consisted of cohesive soils described as silt and clay (A-6a).

Native cohesive soils encountered in the borings included stiff to hard, silt and clay (A-6a) and silty clay (A-6b). Native granular soils encountered in the boring B-001-0-23 included medium dense, gravel and/or stone fragments with sand and silt (A-2-4). Bedrock was encountered in borings B-001-0-23, and B-002-0-23 at a depth varying from about 9 to 33.5 feet, which corresponds to elevations varying from about EL 694.5 to EL 717.0 feet. The bedrock encountered in the borings consisted of severely weathered shale and claystone.

In boring B-001-0-23 groundwater was encountered at a depth of 35 feet below ground surface during drilling and was encountered at a depth of 45 feet below ground surface upon completion of drilling. In boring B-002-0-23 groundwater was not encountered during drilling and upon completion of drilling.

It is our understanding that a three-sided culvert system is selected as the proposed structure at this project. Based on the subsurface conditions encountered at the site, and the requirements outlined in section 305.2 of ODOT Bridge Design Manual (BDM), it is recommended that a shallow foundation system could be utilized for support of the box culvert. The recommended spread footing depth and the corresponding bearing resistance and estimated settlement are presented in the report.

The embankments at the bridge abutments slope down towards Salt Run 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.

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 Reese Street south of the intersection of Reese Street and Saline Street and in Irondale, Jefferson County, Ohio. The bridge is located approximately 35 feet south of Saline Street. The existing structure is a single-span weathering steel beam bridge with a maximum span of approximately 23 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 concrete footers, headwall stems, and three-sided precast concrete box culvert. 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 Reese Street south of the intersection of Reese Street and Saline Street in Irondale, Jefferson County, Ohio. The bridge is located approximately 35 feet south of Saline Street and crosses over Salt Run creek. The approximate latitude/longitude coordinates of the site are 40.57243, -80.73020. See **Site Location**
Existing Improvements	The existing structure is a single-span weathering steel beam with steel decking and asphalt concrete wearing surface on concrete gravity wall abutments.
Existing Topography	Based on our site reconnaissance and the provided topographic survey, surface elevations of the bridge at the north and south abutments are approximately 730.0 and 726.0 feet, respectively.

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.

Item	Description
Proposed Construction	It is our understanding that the proposed structure is a single-span three-sided box culvert, and that the new structure will maintain the existing horizontal and vertical alignments. The proposed replacement structure is anticipated to include new concrete footers, headwall stems, and three-sided precast concrete box culvert. The new abutments are planned to be supported on 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 the abutment slopes are stable from a global stability perspective. 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 December 19, 2023, the existing Reese Street was observed to be a two-lane, asphalt paved roadway aligned in a north to south orientation. Weathering steel beams line both sides of Reese Street at the bridge structure. The Salt Run 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 with approximate slope heights to the water surface ranging from 7 to 10 feet.

General Geology

Based on the Ohio Department of Natural Resources (ODNR) Quaternary Geology Map of Ohio, the project site is mapped within the Holocene aged Cenozoic Colluvium Region. The surficial geology at the project site consists of clay and silt. The Colluvium is characterized as a heterogeneous mixture of high terraces or as eroded remnants of Lacustrine clays and silts. This unit is often covered with loess and/or colluvium; sometimes underlain by sand and gravel. The bedrock geology consists of Pennsylvanian-aged Allegheny group, consisting of shale, limestone, and sandstone.

Based on our review of the ODNR Mine maps, there are no mapped abandoned underground coal mines at the project site. The closest abandoned underground mine identified as JFN-043 is about 800 feet north of the project site and was abandoned in 1913 with an unknown coal elevation. Another abandoned underground mine identified as JFN-028 about 1600 feet east of the project site and was abandoned in 1922 with a coal elevation of 728. A third abandoned underground mine identified as JFN-278/279 located about 1900 feet southeast of the project site and was abandoned in 1921 with unknown coal elevation. The accuracy and quality of the mine maps are highly variable, and there are limitations regarding the accuracy of the georeferencing effort. Due to these limitations, a 500-foot buffer should be applied around the limits of the mapped mines, mine spoils were not encountered in any of the two borings.

Exploration

Field Exploration

A total of two (2) borings, designated as B-001-0-23 and B-002-0-23 were performed on January 8, 2024, to depths of approximately 43.9 and 23.9 feet below the existing ground surface, respectively.

The borings were performed in general accordance with Section 303.3 of the Ohio Department of Transportation (ODOT) Specifications for Geotechnical Explorations (SGE) Type E2 culvert borings.

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

Boring ID	Surface Elevation[1] (feet)	Latitude[2]	Longitude[2]	Total Depth (feet) [3]	Top of Rock Elevation (feet)	Top of Rock Depth (feet) [3]
B-001-0-23	728.0	40.57235	-80.73022	43.9	694.5	33.5
B-002-0-23	726.0	40.57255	-80.73028	23.9	717.0	9.0

1. Surface elevations at the boring locations were obtained from Google Earth.
2. Boring coordinates were obtained using a handheld GPS unit.
3. Below ground surface.

The borings were located in the field prior to drilling operations by Terracon personnel using a handheld GPS unit. Survey information was not available as of this report's preparation. Ground surface elevations were obtained from Google Earth software. Borings coordinates and elevations presented in the preceding table, and on the boring logs presented in Appendix A, were obtained from handheld GPS readings recorded during our site visit. 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 15 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 78.8% for the equipment used during our exploration.

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 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.

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 samples. Atterberg limits, moisture content, and grain size analysis testing were performed on selected soil samples to obtain accurate 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. Soil 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 Reese Street bridge over the Salt Run Creek, respectively. The borings B-001-0-23 and B-002-0-23 encountered a surficial layer consisting of topsoil with a thickness of approximately 9 to 10 inches.

The borings encountered fill materials to a depth ranging from about 3 to 6 feet below the existing ground surface. The fill materials consisted of cohesive soils described as silt and clay (A-6a).

The native cohesive soils encountered in the borings included stiff to hard, silt and clay (A-6a) and silty clay (A-6b). The native granular soils encountered in the boring B-001-0-23 included medium dense, gravel and/or stone fragments with sand and silt (A-2-4).

Bedrock

Bedrock was encountered in borings B-001-0-23, and B-002-0-23 at depths of approximately 33.5 and 9 feet, respectively, which corresponds to elevations varying from about EL 694.5 and EL 717 feet. The bedrock encountered in the borings consisted of severely weathered shale and claystone.

Groundwater Conditions

In boring B-001-0-23 groundwater was encountered at a depth of 35 feet below ground surface during drilling and encountered at a depth of 43 feet upon completion. In boring B-002-0-23 groundwater was not encountered during drilling and upon completion of drilling.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, water level of the existing creek, runoff, 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

It is our understanding that a three-sided culvert system has been selected as the proposed structure for this project. Based on our evaluation of the subsurface conditions encountered at the site, and the requirements outlined in Section 305.2 of ODOT Bridge Design Manual (BDM), it is recommended that a shallow foundation system be used to support the proposed structure. Alternatively, the proposed bridge can be supported on abutments founded either on Drilled Shaft deep foundation system for this project. The new structure will maintain the existing horizontal and vertical alignments.

The design of shallow foundations and headwalls for three-sided culvert should be in accordance with ODOT GDM section 1400 and BDM section 305.2. We recommend that free-draining granular materials be used for backfill against the sides of the culvert and associated wing walls per the manufacturer's requirements. At a minimum, the granular backfill zone should be at least 24 inches thick measured normal to the back face of the retaining structure. Granular backfill should be compacted in accordance with 2023 ODOT CMS Item 203.06, "Spreading and Compacting". Provided that free-draining granular material is used for backfill, the lateral earth pressures on the sides of the box culvert and the culvert's wingwalls may be calculated using the following parameters:

- At-Rest Earth Pressure Coefficient (Ko) = 0.5
- Active Earth Pressure Coefficient (Ka) = 0.33
- Internal Angle of Friction (φ) = 30°
- Moist Unit Weight of Backfill (γ) = 120 psf.

At-rest earth pressures should be used where little wall yield is expected (such as the culvert). Active earth pressures may be used for the design of wingwalls, if wall movement equivalent to 0.1 to 0.2 percent of the height of the wingwall is allowed to occur. The culvert must also be designed to withstand the surface effect of traffic and the vertical load resulting from weight of any fill and pavement to be placed over the culvert. The design of wingwalls should take into consideration the additional loading of sloping backfill conditions. Additional details on shallow foundations design and lateral earth pressures are provided in the following sections of this report.

Shallow Foundation Recommendations

A shallow foundation system could be utilized for support of the proposed three-sided culvert. Please note that the proposed shallow foundations should be properly protected against scour.

The proposed shallow foundations/spread footings for the culvert at boring B-001-0-23 are anticipated to be embedded at elevation of 714 feet on very stiff silt and clay (A-6a) (approximately 16 feet below existing ground surface). We recommend that the shallow foundations bearing on stiff to very stiff silt and clay (A-6a) be designed for a nominal bearing resistance of 9,000 psf with a resistance factor of ϕ_b = 0.45, corresponding to a factored bearing resistance of 4,000 psf. Considering structural loading equivalent to this factored bearing resistance, we estimate that total settlement of the footings bearing on stiff to very stiff silt and clay at elevation 714 feet will be on the order of 1 inch or less. The proposed shallow foundations/spread footings for the culvert at boring B-002-0-23 are anticipated to be embedded at elevation of 714 feet on hard silty clay (A-6b) (approximately 12 feet below existing ground surface). We recommend that the shallow foundations bearing on hard silty clay (a-6b) at elevation 714 feet be designed for a nominal bearing resistance of 10,000 psf with a resistance factor of ϕ_b = 0.45, corresponding to a factored bearing resistance of 4,500 psf. Considering structural loading equivalent to this factored bearing resistance, we estimate that total settlements of the footings bearing on stiff to hard silt and clay (A-6a) or silty clay (A-6b) at elevation 714 feet will be on the order of 1 inch or less. The differential settlement between the forward and rear culvert foundations is anticipated to be about ½ to 1 inch. The anticipated differential settlement should be evaluated by the structural engineer as a consideration in the design of a shallow foundation system. 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 bedrock scour potential below the bearing elevation of spread footings, the results of the analysis indicated that the shale/claystone 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 foundation system.

The ultimate coefficient of sliding friction recommended for contact between the concrete and foundation soil at boring B-001-0-23 is 0.35 with a resistance factor of ϕ_T =

0.9. The ultimate coefficient of sliding friction recommended for contact between the concrete and foundation soil at boring B-002-0-23 is 0.35 with a resistance factor of ϕ_T = 0.9.

The foundation excavations should be examined after excavations to verify that the entire bearing surface consists of suitable soil/bedrock. Subgrade preparation for the new foundations 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 (if any). For foundations bearing in 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 at boring B-002-0-23 will encounter very weak to weak shale. We anticipate that the weak bedrock encountered can be excavated using heavy earth moving equipment. Confined excavations in bedrock may require equipment such as chisels or rock hammers to facilitate excavation. All excavations should be maintained at OSHA requirements for stable slopes.

Due to shallow bedrock encountered at boring B-002-0-23, 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.

Drilled Shaft Recommendations

As an alternative, a deep foundation system consisting of drilled shaft foundations 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. 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 rock conditions encountered in the test borings.

Drilled Shaft Design

Location	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)	Resistance Factor, φ_{stat}
North Abutment	707.0	1.5 x Shaft Diameter	Shale	36	20	3.5	0.50 (Tip) 0.55 (Side) 0.4 (uplift resistance)
South Abutment	702.0		Claystone		30	2.0	

1. Below existing ground surface. 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 claystone and shale bedrock. 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. Top of rock socket elevations should be adjusted to an elevation at least 2 feet below the top of rock elevations.
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.

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/claystone 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 drilled shafts should be designed per BDM section 305.4.

Drilled Shaft Construction Consideration

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.

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} 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. The portion of the drilled shaft within 36 inches of finished grade should ignore any lateral soil resistance due to frost considerations.

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

BORING B-001-0-23							
Soil Layer/Type[1]	**Approximate Bottom Elevation of Layer (feet)**	**LPILE Model**	**Total Unit Weight (pcf)**	**Undrained Shear Strength (psf)**	**Soil Friction Angle (deg)**	**K (pci)**	**ε_{50}**
Silt and Clay (A-6a)	7.5	Stiff Clay w/o Free Water (Reese)	130	3,500	--	1,200	0.005
Gravel and/or Stone Fragments with sand and silt (A-2-4)	12	Sand (Reese)	115	--	30	80	--
Silt and Clay (A-6a)	33.5	Stiff Clay w/o Free	131	4,000	--	1,500	0.004

BORING B-001-0-23

Soil Layer/Type[1]	Approximate Bottom Elevation of Layer (feet)	LPILE Model	Total Unit Weight (pcf)	Undrained Shear Strength (psf)	Soil Friction Angle (deg)	K (pci)	ε_{50}
		Water (Reese)					
Silty Clay (A-6b)	38.5	Stiff Clay with Free Water (Reese)	131	4,500	--	1,600	0.004
Weathered Bedrock [2]	43.9	Stiff Clay with Free Water (Reese)	135	7,000	--	2,000	0.004

1. See test boring logs and Findings for more details on Stratigraphy.
2. Boring terminated within this layer

BORING B-002-0-23

Soil Layer/Type[1]	Approximate Bottom Elevation of Layer (feet)	LPILE Model	Total Unit Weight (pcf)	Undrained Shear Strength (psf)	Soil Friction Angle (deg)	K (pci)	ε_{50}
Silt and Clay (A-6a)	6.0	Stiff Clay w/o Free Water (Reese)	128	2,000	--	650	0.008
Silty Clay (A-6b)	7.5	Stiff Clay w/o Free Water (Reese)	131	4,500	--	1,600	0.004
Silt and Clay (A-6a)	9	Stiff Clay w/o Free Water (Reese)	131	4,500	--	1,600	0.004
Weathered Bedrock [2]	23.9	Stiff Clay with Free Water (Reese)	140	8,000	--	2,000	0.004

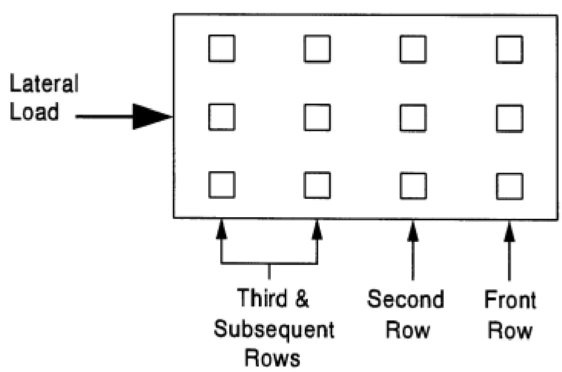
1. See test boring logs and Findings for more details on Stratigraphy.
2. Boring terminated within this layer

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.

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

Lateral Earth Pressures for Permanent Retaining Walls Associated with Culvert Structures

Retaining walls 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 the 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	39	54	K_a = 0.33
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 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 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 15 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 gravel and/or stone fragments with sand and silt (A-2-4) and cohesive soils consisting of silt and clay (A-6a). Note that specific 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 samples from the borings. Also, the critical shear stress (τ_c), the equivalent D_{50} ($D_{50,\ equiv}$), Erosion Category (EC), and Erodibility Index (K), were calculated based on the equations provided in GDM sections 1302.1, 1302.2 and 1403, and summarized in the following table.

Boring Number	Sample Number	Elevation (feet)	D_{50} (mm)	Erodibility Index, K	τ_c (psf)	$D_{50,\ equiv}$ (mm)	Erosion Category, EC
B-001-0-23	SS-1&2[1]	728.0-725.0	0.015	--	1.7430	83.436	3.413
	SS-3-5[1]	725.0-720.5	0.015	--	1.1531	55.197	3.413
	SS-6	720.5-719.0	0.182	--	0.0021	3.872	2.754
	SS-7	719.0-717.5	0.229	--	0.0048	0.229	1.432
	SS-8	717.5-716.0	0.270	--	0.0056	0.270	1.518
	SS-9&10[1]	716.0-709.5	0.020	--	0.3666	17.547	3.168
	SS-11-13[1]	709.5-694.5	0.020	--	0.5059	24.219	3.168
	Claystone	694.5-684.1	--	0.1093	1.748	83.691	1.245
B-002-0-23	SS-4	721.5-720.0	0.143	--	0.0282	1.349	3.075
	SS-5	720.0-718.5	0.008	--	0.7922	37.920	3.670
	SS-6	718.5-717.0	0.026	--	0.8694	41.620	3.255
	Shale	717.0-702.1	--	0.1690	2.1738	104.061	1.456

1. Soil data required to calculate scour parameters were estimated for these samples based on the similar conditions obtained from boring B-002-0-23 and experience with similar conditions.

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 indicated that the shale/claystone 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 foundation system.

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 43.9 feet and this seismic site class definition considers that bedrock continue 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 pile installation 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 conforming 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 section 807 "Special Benching For Embankment Stability Over Soft Foundation Soil" of ODOT Geotechnical Design Manual (GDM).

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 35 feet below ground surface during drilling and encountered at a depth of 43 feet below ground surface upon completion. In boring B-002-0-23 groundwater was not encountered during drilling and 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:

Note: All attachments are one page unless noted above

Site Location

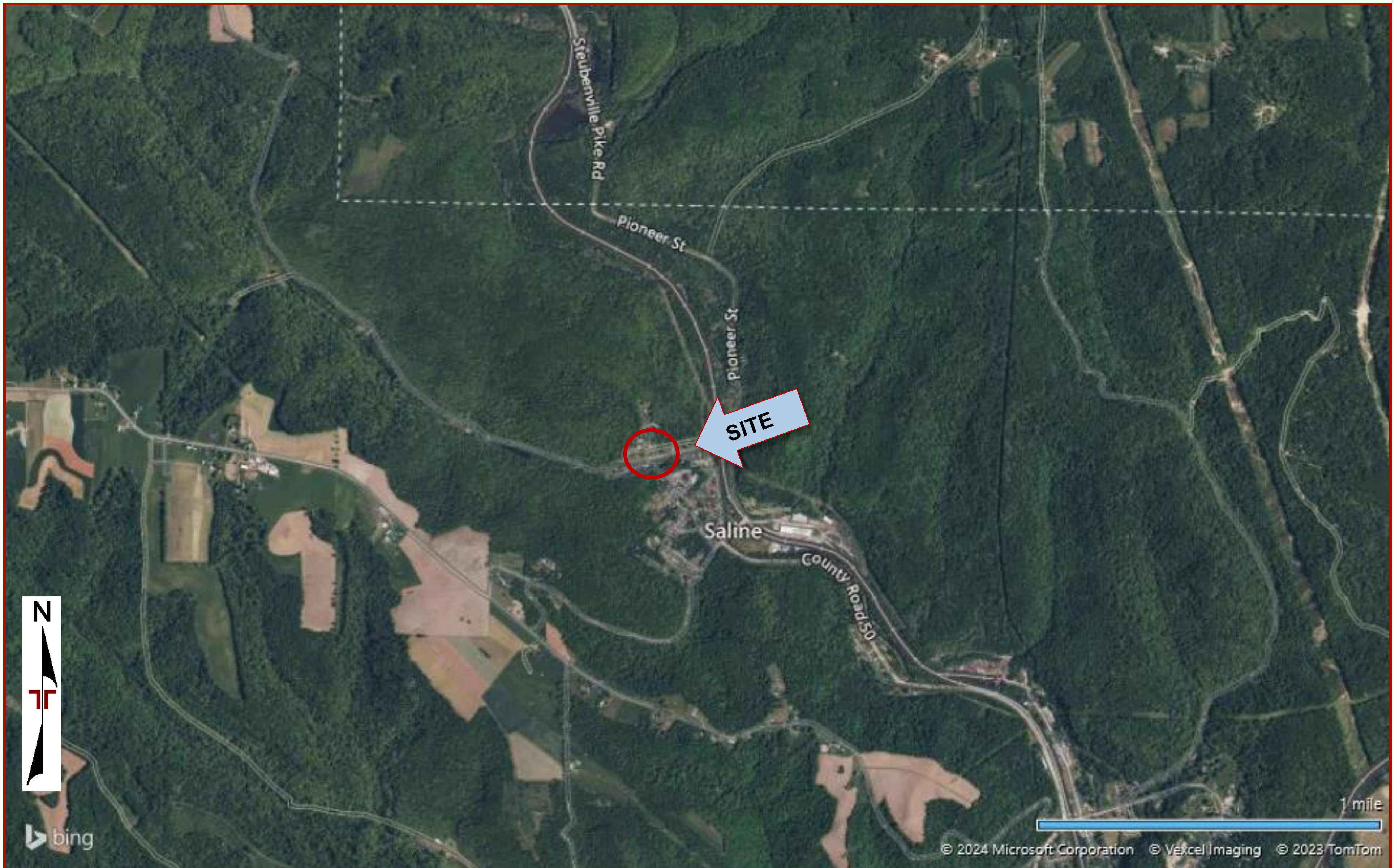


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan

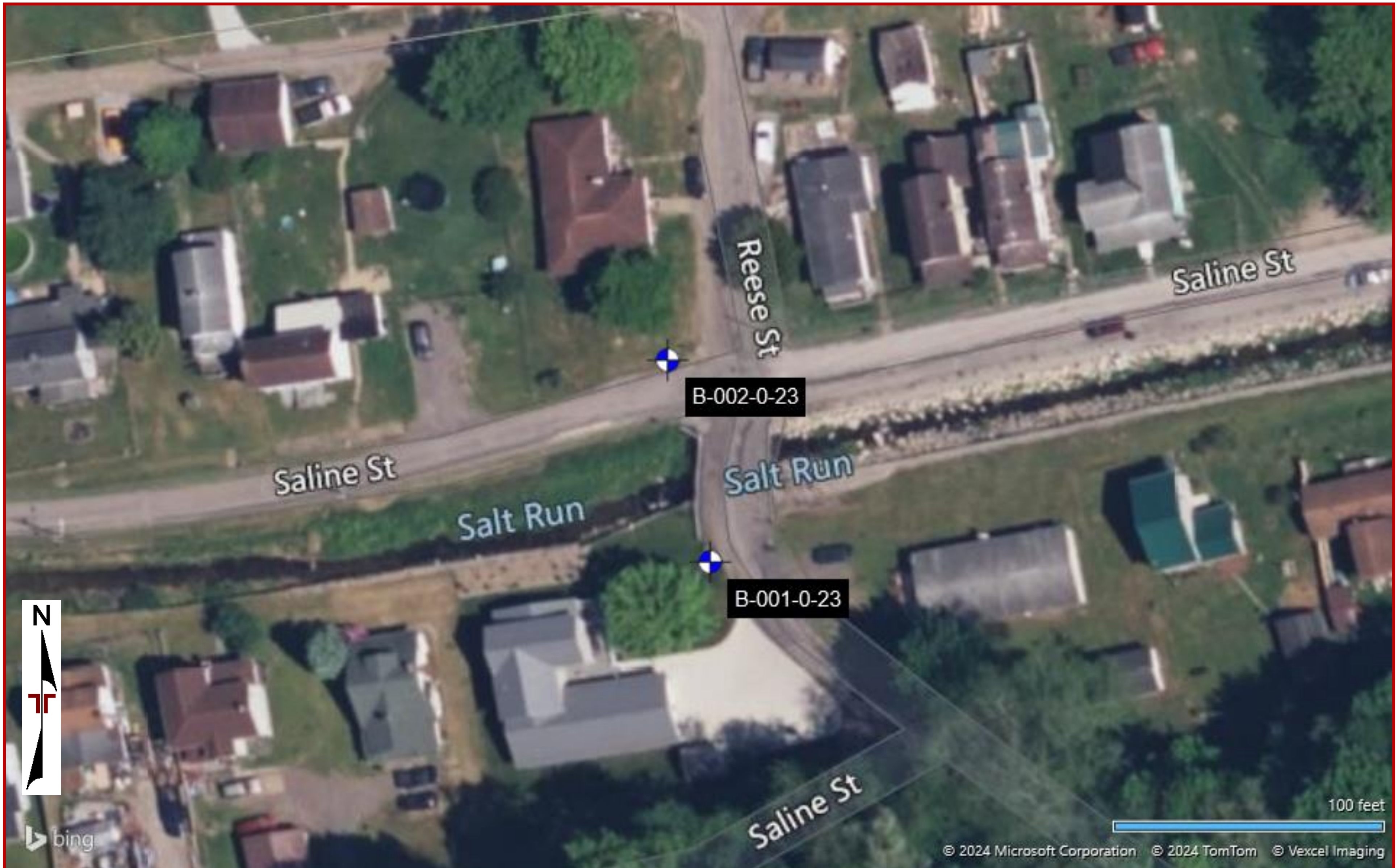


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Appendix B – Exploration and Laboratory Testing Results

Contents:

Note: All attachments are one page unless noted above.

PROJECT: JEFF - REESE STREET BRIDGE	DRILLING FIRM / OPERATOR: TC / K. REINHERT	DRILL RIG: MOBILE B-57	STATION / OFFSET: 10' RT.
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: TC / T. CAROTHERS	HAMMER: SAFETY HAMMER	ALIGNMENT: REESE STREET
PID: 119880 SFN: 4130774	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 8/11/22	ELEVATION: 728.0 (MSL) EOB: 43.92 ft.
START: 1/8/24 END: 1/8/24	SAMPLING METHOD: SPT	ENERGY RATIO (%): 78.8	LAT / LONG: 40.57235, -80.73022

EXPLORATION ID B-001-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 728.0
TOPSOIL (9")	727.3
DARK BROWN AND GRAY, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, **FILL**, DAMP	725.0
VERY STIFF, BROWN AND GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP @4.5' to 6.0'; ENCOUNTERED COBBLES/BOULDERS	720.5
MEDIUM DENSE, BROWN, **GRAVEL AND/OR STONE FRAGMENTS WITH SAND AND SILT**, LITTLE CLAY, DAMP	716.0
STIFF TO VERY STIFF, BROWN, **SILT AND CLAY**, LITTLE TO SOME SAND, LITTLE GRAVEL, MOIST	

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
2-3-5	11	39	SS-1	-	-	-	-	-	-	-	-	-	13	A-6a (V)
9-10-9	25	39	SS-2	-	-	-	-	-	-	-	-	-	6	A-6a (V)
6-7-6	17	44	SS-3	1.50	-	-	-	-	-	-	-	-	8	A-6a (V)
3-9-8	22	56	SS-4	1.50	-	-	-	-	-	-	-	-	-	A-6a (V)
6-9-10	25	61	SS-5	1.00	-	-	-	-	-	-	-	-	10	A-6a (V)
10-9-9	24	67	SS-6	1.25	13	19	37	16	15	23	15	8	10	A-2-4 (0)
7-5-5	13	50	SS-7	1.00	22	17	32	16	13	22	14	8	11	A-2-4 (0)
6-5-6	14	56	SS-8	1.50	29	13	28	14	16	25	15	10	12	A-2-4 (0)
4-7-9	21	61	SS-9	2.00	-	-	-	-	-	-	-	-	-	A-6a (V)
6-7-7	18	44	SS-10	2.00	-	-	-	-	-	-	-	-	13	A-6a (V)
8-7-9	21	67	SS-11	1.50	-	-	-	-	-	-	-	-	-	A-6a (V)
6-7-7	18	72	SS-12	-	-	-	-	-	-	-	-	-	13	A-6a (V)
10-9-11	26	44	SS-13	3.00	-	-	-	-	-	-	-	-	12	A-6a (V)

PID: 119880	SFN: 4130774	PROJECT: JEFF - REESE STREET BRIDGE	STATION / OFFSET: 10' RT.	START: 1/8/24	END: 1/8/24	PG 2 OF 2	B-001-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 698.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
STIFF TO VERY STIFF, BROWN, **SILT AND CLAY**, LITTLE TO SOME SAND, LITTLE GRAVEL, MOIST *(continued)*	694.5	31, 32, 33 — TR																
CLAYSTONE, GRAY, SEVERELY WEATHERED.		34	50 50/6"	-	92	SS-14	4.00	-	-	-	-	-	-	-	-	-	A-6b (V)	
		W 693.0; 35, 36, 37, 38																
		39	23 50/5"	-	27	SS-15	-	-	-	-	-	-	-	-	-	-	Rock (V)	
		40, 41, 42; ▽ 685.0; 43																
	684.1	EOB	50/5"	-	80	SS-16	-	-	-	-	-	-	-	-	-	-	Rock (V)	

NOTES: NONE

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

PROJECT: JEFF - REESE STREET BRIDGE	DRILLING FIRM / OPERATOR: TC / K. REINHERT	DRILL RIG: MOBILE B-57	STATION / OFFSET: 5' LT.	EXPLORATION ID B-002-0-23
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: TC / T. CAROTHERS	HAMMER: SAFETY HAMMER	ALIGNMENT: REESE STREET	
PID: 119880 SFN: 4130774	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 8/11/22	ELEVATION: 726.0 (MSL) EOB: 23.92 ft.	PAGE 1 OF 1
START: 1/8/24 END: 1/8/24	SAMPLING METHOD: SPT	ENERGY RATIO (%): 78.8	LAT / LONG: 40.57255, -80.73028	

MATERIAL DESCRIPTION AND NOTES	ELEV. 726.0	DEPTHS
TOPSOIL (10")	725.2	
DARK BROWN AND GRAY, **SILT AND CLAY**, 'AND' SAND, LITTLE GRAVEL, **FILL**, DAMP TO MOIST @3.0' to 6.0'; ENCOUNTERED ORGANIC ODOR AND STAINING	720.0	
HARD, BROWN AND GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, MOIST	718.5	
HARD, BROWN AND GRAY, **SILT AND CLAY**, SOME SAND, LITTLE GRAVEL, SHALE FRAGMENTS, DAMP	717.0	TR
SHALE, GRAY, SEVERELY WEATHERED.	702.1	EOB

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
3 4 6	13	56	SS-1	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
5 6 8	18	67	SS-2	-	-	-	-	-	-	-	-	-	13	A-6a (V)	
2 1 1	3	56	SS-3	-	-	-	-	-	-	-	-	-	15	A-6a (V)	
7 13 12	33	39	SS-4	-	18	18	22	15	27	31	20	11	27	A-6a (2)	
4 19 50/5"	-	94	SS-5	3.50	9	16	12	22	41	39	20	19	14	A-6b (9)	
8 50/5"	-	91	SS-6	3.00	18	9	17	23	33	29	16	13	9	A-6a (5)	
15 16 40	74	39	SS-7	-	-	-	-	-	-	-	-	-	5	Rock (V)	
50/5"	-	100	SS-8	-	-	-	-	-	-	-	-	-	-	Rock (V)	
29 50/5"	-	55	SS-9	-	-	-	-	-	-	-	-	-	3	Rock (V)	
50/6"	-	83	SS-10	-	-	-	-	-	-	-	-	-	-	Rock (V)	
50/5"	-	100	SS-11	-	-	-	-	-	-	-	-	-	-	Rock (V)	
50/5"	-	100	SS-12	-	-	-	-	-	-	-	-	-	-	Rock (V)	

NOTES: NONE

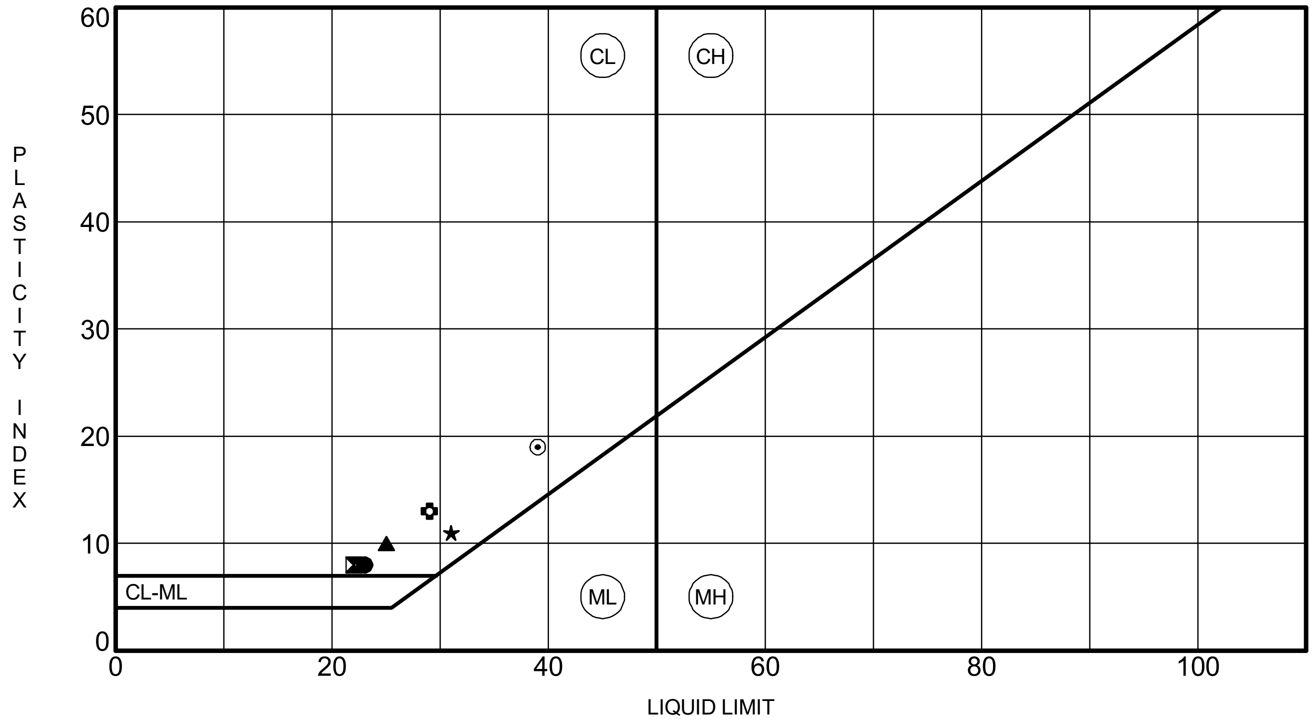
ABANDONMENT METHODS, MATERIALS, QUANTITIES: AUGER CUTTINGS MIXED WITH AUGER CUTTINGS

ATTERBERG LIMITS' RESULTS

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT REESE STREET BRIDGE REPLACEMENT **PID** 119880

OGE NUMBER N4235428 **PROJECT TYPE** STRUCTURE FOUNDATION



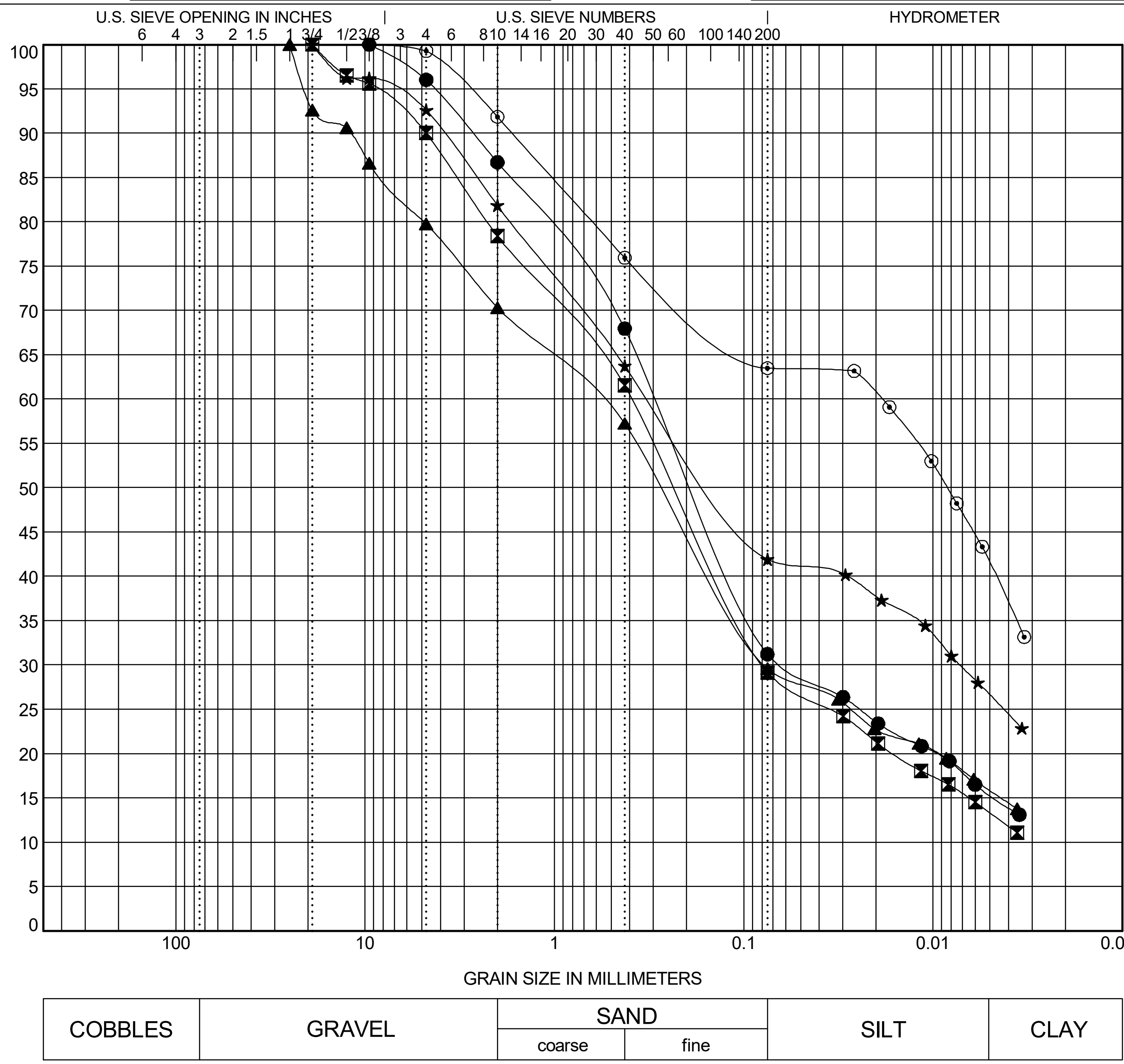
	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-001-0-23**	**7.5**	**23**	**15**	**8**	**31**	**A-2-4 ~ CLAYEY SAND(SC)**
⧗	**B-001-0-23**	**9.0**	**22**	**14**	**8**	**29**	**A-2-4 ~ CLAYEY SAND(SC)**
▲	**B-001-0-23**	**10.5**	**25**	**15**	**10**	**30**	**A-2-4 ~ CLAYEY SAND with GRAVEL(SC)**
★	**B-002-0-23**	**4.5**	**31**	**20**	**11**	**42**	**A-6a ~ CLAYEY SAND(SC)**
⊙	**B-002-0-23**	**6.0**	**39**	**20**	**19**	**63**	**A-6b ~ SANDY LEAN CLAY(CL)**
✿	**B-002-0-23**	**7.5**	**29**	**16**	**13**	**56**	**A-6a ~ SANDY LEAN CLAY(CL)**

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT REESE STREET BRIDGE REPLACEMENT **PID** 119880

OGE NUMBER N4235428 **PROJECT TYPE** STRUCTURE FOUNDATION



Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
● B-001-0-23	7.5	**A-2-4 ~ CLAYEY SAND(SC)**	23	15	8
⧗ B-001-0-23	9.0	**A-2-4 ~ CLAYEY SAND(SC)**	22	14	8
▲ B-001-0-23	10.5	**A-2-4 ~ CLAYEY SAND with GRAVEL(SC)**	25	15	10
★ B-002-0-23	4.5	**A-6a ~ CLAYEY SAND(SC)**	31	20	11
⊙ B-002-0-23	6.0	**A-6b ~ SANDY LEAN CLAY(CL)**	39	20	19

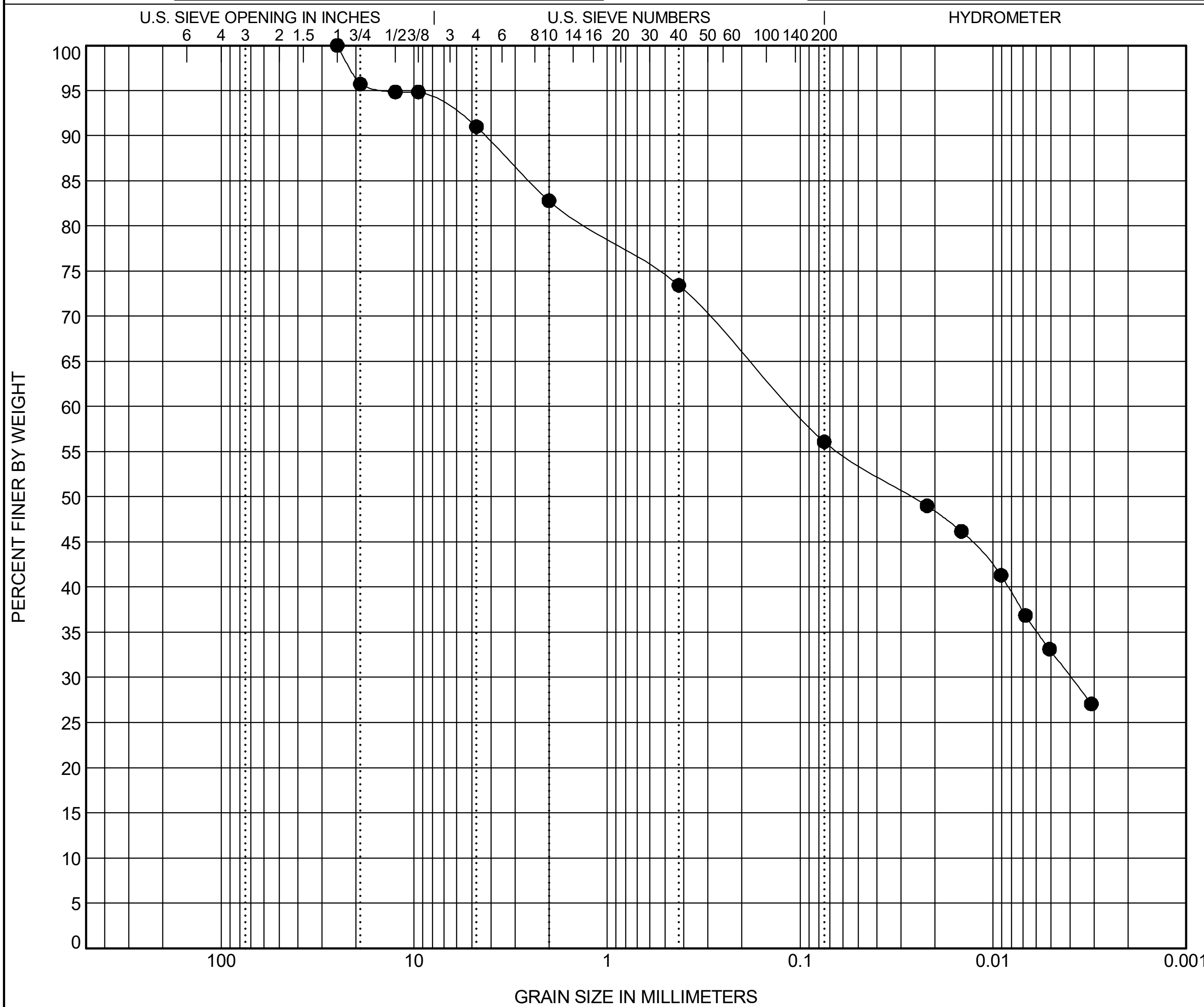
Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
● B-001-0-23	7.5	2.715	0.182	0.06		13	19	37	16	15		
⧗ B-001-0-23	9.0	4.748	0.229	0.079		22	17	32	16	13		
▲ B-001-0-23	10.5	12.008	0.27	0.077		29	13	28	14	16		
★ B-002-0-23	4.5	3.85	0.143	0.007		18	18	22	15	27		
⊙ B-002-0-23	6.0	1.671	0.008			9	16	12	22	41		

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT REESE STREET BRIDGE REPLACEMENT **PID** 119880

OGE NUMBER N4235428 **PROJECT TYPE** STRUCTURE FOUNDATION



COBBLES	GRAVEL	SAND		SILT	CLAY
		coarse	fine		

	Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-002-0-23**	**7.5**	**A-6a ~ SANDY LEAN CLAY(CL)**	**29**	**16**	**13**

	Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-002-0-23**	**7.5**	**4.275**	**0.026**	**0.004**		**18**	**9**	**17**	**23**	**33**		

Appendix C – Supporting Information

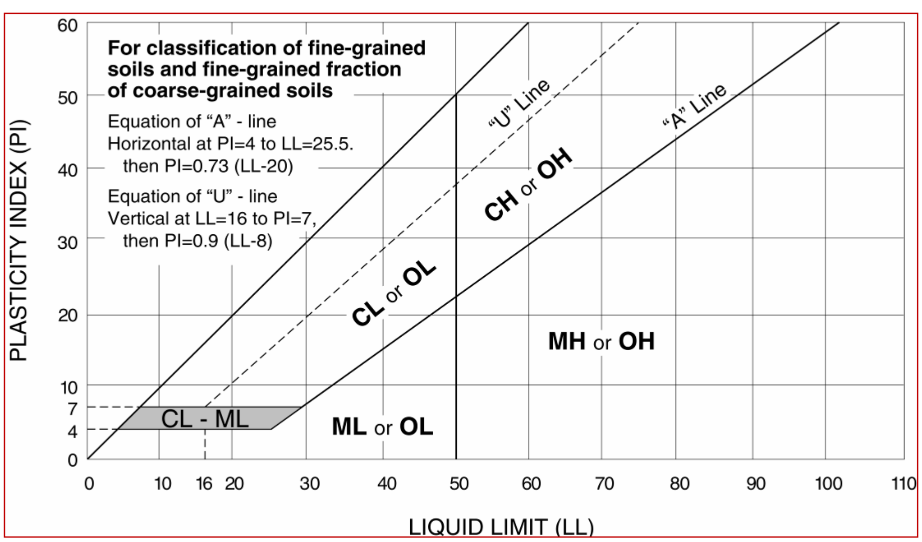
Contents:

Note: All attachments are one page unless noted above.

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≥4 and 1≤Cc≤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≥6 and 1≤Cc≤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] $Cu = D_{60}/D_{10}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[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 × 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"
	Medium	0.01"-0.02"
	Fine	0.005"-0.01"
	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

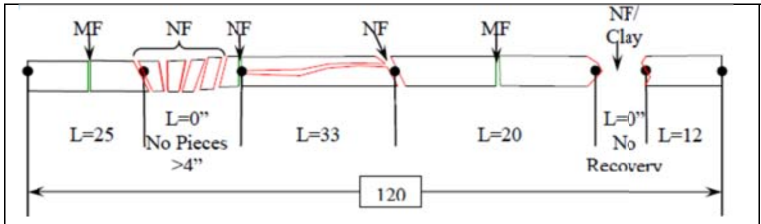
a: Structure

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



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

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

Client:	EMH&T
Project Name:	JEF-Reese Street Bridge Replacement
Project No.#	N4235428
Date:	4/1/2024
Bedrock Type:	Claystone
Boring ID:	B-001-0-23
Calculated By:	AA-H
Checked By:	BBR/YSR

Erodibility Index (K) for Bedrock

GDM Section 1302.1.3, BDM Section 305.2.1.2.b(B.6.a) and HEC 18 Equation 4.17

Ms (Qu<10 Mpa)	Kd	Kb
1.093300875	1	0.1

Qu (psi)	Qu (Mpa)
200	1.37931

BDM Section 305.2.1.2.b(B) and HEC 18 Section 4.7.2

RQD (%)	0	
Jn	2.24	HEC 18 Section 4.7.2 Table 4.23
Jr	3	HEC 18 Section 4.7.2 Table 4.24
Ja	3	HEC 18 Section 4.7.2 Table 4.25
Js	1	HEC 18 Section 4.7.2 Table 4.26

K
0.10933

Ms = Qu for Qu ≥ 10-MPa, or Ms = (0.78) $Qu^{1.05}$ for Qu < 10-Mpa

$K = (M_s)(K_b)(K_d)(J_s)$, $K_b = RQD/J_n \geq 0.10$, and $K_d = J_r/J_a$

Where RQD = 0, Block Size Parameter $K_b = RQD/J_n$ = 0, and subsequently Erodibility Index K = 0. In the scour calculations that depend on K, it is in the denominator, and K = 0 will result in a divide by zero error.

If RQD = 0, do not set Block Size Parameter K_b = 0 and subsequently Erodibility Index K = 0. In this case, set the minimum value of K_b = 0.010. In the scour calculations that depend on K, it is in the denominator, and K = 0 will results in divide by zero error.

Client:	EMH&T
Project Name:	JEF-Reese Street Bridge Replacement
Project No.#	N4235428
Date:	4/1/2024
Bedrock Type:	Shale
Boring ID:	B-002-0-23
Calculated By:	AA-H
Checked By:	BBR/YSR

Erodibility Index (K) for Bedrock

GDM Section 1302.1.3, BDM Section 305.2.1.2.b(B.6.a) and HEC 18 Equation 4.17

Ms (Qu<10 Mpa)	Kd	Kb
1.673537768	1	0.1

BDM Section 305.2.1.2.b(B) and HEC 18 Section 4.7.2

Qu (psi)	Qu (Mpa)
300	2.068966

RQD, assumed (%)	0	
Jn	2.24	HEC 18 Section 4.7.2 Table 4.23
Jr	3	HEC 18 Section 4.7.2 Table 4.24
Ja	3	HEC 18 Section 4.7.2 Table 4.25
Js	1.01	HEC 18 Section 4.7.2 Table 4.26

K
0.169027315

Ms = Qu for Qu ≥ 10-MPa, or Ms = (0.78) $Qu^{1.05}$ for Qu < 10-Mpa

$K = (M_s)(K_b)(K_d)(J_s)$, $K_b = RQD/J_n \geq 0.10$, and $K_d = J_r/J_a$

Where RQD = 0, Block Size Parameter $K_b = RQD/J_n$ = 0, and subsequently Erodibility Index K = 0. In the scour calculations that depend on K, it is in the denominator, and K = 0 will result in a divide by zero error.

If RQD = 0, do not set Block Size Parameter K_b = 0 and subsequently Erodibility Index K = 0. In this case, set the minimum value of K_b = 0.010. In the scour calculations that depend on K, it is in the denominator, and K = 0 will results in divide by zero error.