SR 141 Bridge Replacement - PID No. 103975

Structure Foundation Report

September 25, 2023 | Terracon Project No. N4225280

Prepared for:

ADR & Associates Ltd.
88 West Church Street
Newark, OH 43055

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September 25, 2023

ADR & Associates Ltd.
88 West Church Street
Newark, OH 43055

Attn: Mr. Justin Hartfield, P.E.
P: (740) 345-1921
E: JHartfield@adrinnovation.com

Re: Structure Foundation Report
SR 141 Bridge Replacement - PID No. 103975
Willow Wood
Lawrence County, Ohio
Terracon Project No. N4225280

Dear Mr. Hartfield:

Terracon Consultants, Inc. (Terracon) has completed the structure foundation exploration for the above referenced project. This study was performed in general accordance with Terracon Proposal No. PN4225280 dated August 1, 2022, which was authorized by ADR & Associates Ltd. on January 23, 2023. This report presents the findings of the subsurface exploration, laboratory testing results, and the results of our foundation analysis performed for the proposed replacement of the existing State Route 141 bridge structure located in Willow Wood, 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

Kevin M. Ernst

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

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

KEVIN MICHAEL ERNST
E-56245
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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.

Executive Summary

This report presents the findings of the geotechnical exploration performed for the proposed replacement of the existing bridge on State Route (SR) 141 over Long Creek located in Willow Wood, Ohio. The existing structure is a single span concrete bridge on spread footings. The proposed replacement structure will include three-sided flat-topped culvert on capped pile abutments. The proposed abutments are anticipated to be supported on drilled shafts.

Terracon performed two (2) borings, designated as Borings B-001-0-22 and B-002-0-22 to depths ranging from about 30 to 35 feet below the existing ground surface. The borings encountered a surficial layer consisting of approximately 13 to 14 inches of asphalt pavement over 4 to 5 inches of aggregate base. Beneath the pavement and base materials, the borings encountered fill consisting of medium stiff silt and clay and loose gravel with sand, silt and clay (A-2-6) up to depths of 5 to 6.5 feet below existing grades.

Native soils were encountered beneath the fill and consisted of soft to stiff cohesive soils classified as silt and clay (A-6a), clay (A-7-6), sandy silt (A-4a) and loose cohesionless soils classified as gravel with sand and silt (A-2-4), gravel with sand, silt and clay (A-2-6). Underlying the native soils, bedrock consisting of shale and siltstone was encountered in the borings to the depths explored. Borings B-001-0-22 and B-002-0-22 were terminated in bedrock at a depth of 35 feet and 30 feet below existing grades, respectively.

Boring B-001-0-22 encountered bedrock at a depth of 23.5 feet below existing grade (about an elevation of 587 feet) consisting of very weak to weak, severely to moderately weathered shale underlain by moderately strong, slightly weathered siltstone. Boring B-002 encountered bedrock at a depth of 18.5 feet below existing grade (about an elevation of 592.3 feet) consisting of very weak, severely weathered shale underlain by slightly strong to strong, highly to slightly weathered siltstone.

Groundwater was encountered in borings B-001-0-22 and B-002-0-22 during drilling at a depth of approximately 18.5 feet and 20 feet below the existing ground surface, corresponding to elevation of about 592 feet and 590.8 feet respectively.

Based on the subsurface conditions encountered at the site, and the requirements outlined in section 305.4 of ODOT Bridge Design Manual (BDM), it is recommended that a deep foundation system consisting of drilled shafts be used for support of the proposed bridge replacement structure. The estimated top of rock socket elevations and the corresponding unfactored nominal tip and side resistance for rock socketed drilled shafts are presented in this report.

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

This structure foundation exploration report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed replacement of the existing bridge on State Route 141 over Long Creek located in Willow Wood, Ohio. The existing structure is a single span concrete bridge on spread footings.

It is our understanding that the existing structure is to be replaced with three-sided-flat-topped culvert on capped pile abutments.

Site Location and Description

Item	Description
Location	The project site is located about 700 feet south of the intersection of Town highway 536 & route 141 in Willow Wood, Lawrence County, Ohio. The approximate latitude/longitude coordinates of the site are: 38.63496°, -82.46408°. See **Site Location**
Existing Structure	The existing structure is a single span concrete bridge on spread footings. The total span of the existing culvert is approximately 40 feet The site is surrounded by vegetated areas and few residential dwellings in all directions.
Existing Topography	The existing ground is relatively flat at an elevation of 610 feet.

Project Description

Item	Description
Proposed Structure	Based on the preliminary site plan and profile drawings, the precast structure will be a Three-Sided Flat-Topped Box type structure supported on new foundation elements.
Grading/Slopes	Based on provided preliminary site plan and profile drawings, we have assumed that the new structure will maintain the existing horizontal and vertical alignments.

Terracon should be notified if any of the above information is inconsistent with the planned construction, especially the grading limits, as modifications to our recommendations may be necessary.

General Geology

Based on the Ohio Department of Natural Resources Quaternary Geology Map of Ohio, the project site is located within the Holocene and Pleistocene aged Colluvium. Typically, this area is characterized by colluvium derived from local bedrock in unglaciated areas, includes scattered areas of residuum, weathered material, and bedrock outcrop. The geology in this region is located within the Marietta Plateau region within the Allegheny Plateaus section of the Appalachian Plateaus physiographic province of Ohio. This region is characterized as a dissected high-relief plateau with mostly fine-grained rocks consisting of red shales, claystones, and siltstones. Landslides are also common, along with remnants of the ancient, lacustrine, clay-filled Teays drainage system.

Locally, the overburden soils generally consist of silt loam deposits. Soils encountered within the borings generally agree with the anticipated geologic conditions.

Reconnaissance

At the time of our site reconnaissance visit on January 24, 2023, the existing bridge was observed to be two-lane, asphalt paved aligned in a north south orientation, traversing primarily agricultural properties. Asphalt pavement cracking was observed on the bridge. Guardrails line both sides of route 141 at the bridge structure. Water level in Long Creek was shallow, with a general flow direction towards the east at the subject structure. At the existing structure, surface drainage was directed into the existing creek. Concrete rubble was observed right next to the southwest wingwall. A warning road sign was found sitting on the ground.

Exploration

Field Exploration

Two (2) borings, designated as B-001-0-22 and B-002-0-22, were performed on January 30, 2023. The borings were performed in general accordance with the most recent 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 (Appendix A) and summarized in the following table.

Boring Number	Elevation [1]	Latitude [2]	Longitude [2]	Depth (feet) [3]	Top of Rock Elevation	Top of Rock Depth [3]
B-001-0-22	610.5	38.635020	-82.464078	35	587.0	23.5
B-002-0-22	610.8	38.634903	-82.464043	30	592.3	18.5

1. Surface elevations were obtained from the provided bridge plan and profile provide by ADR & Associates Ltd.
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. Ground surface elevations were obtained from survey data provided by ADR & Associates Ltd. Borings coordinates and elevations presented in the preceding table, and on the boring 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 an ATV-mounted rotary drill rig utilizing a 3¼-inch I.D. continuous flight hollow stem auger to advance the boreholes between sampling attempts. The split-barrel samples were obtained at the boring locations at continuously up to a depth of 12.5 feet below existing grades and then at an interval of 5-foot to the top of the bedrock. Upon encountering auger refusal, 10 feet of rock was cored in borings B-001-0-22 and B-002-0-22 using diamond bit, double-tube methods. We observed and recorded groundwater levels during drilling and upon completion.

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.

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 recovered rock core was placed in partitioned boxes. The soil and rock samples were delivered to our laboratory for additional examination and testing.

Following the completion of drilling, the boreholes were backfield with auger cuttings and bentonite chips. Where borings penetrated the existing pavement surface, the roadway surface was repaired using cold mixed asphalt patch.

Laboratory Testing Program

As part of the testing program, all samples were examined in the 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 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, two uniaxial compressive tests were performed on rock samples to evaluate the strength parameters of the bedrock encountered. The results of lab testing are shown on the boring logs and presented in the appendix 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 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.

Soil Conditions

Borings B-001-0-22 and B-002-0-22 were performed within the existing drive lanes of State Route 141. The borings encountered a pavement section consisting of approximately 13 to 14 inches of asphalt pavement over 4 to 5 inches of aggregate base.

Beneath the pavement and base materials, the borings encountered fill material consisting of medium stiff silt and clay and loose gravel with sand, silt and clay (A-2-6) up to depths of 5 to 6.5 feet below existing grades. The native soils consisted of soft to stiff cohesive soils classified as silt and clay (A-6a), clay (A-7-6), sandy silt (A-4a) and loose cohesionless soils classified as gravel with sand and silt (A-2-4), gravel with sand, silt and clay (A-2-6). Underlying the native soils, bedrock consisting of shale and siltstone was encountered in the borings to the depths explored. Borings B-001-0-22 and B-002-0-22 were terminated in bedrock at a depth of 35 feet and 30 feet below existing grades, respectively.

Bedrock

Boring B-001-0-22 encountered bedrock at a depth of 23.5 feet below existing grade (about an elevation of 587 feet) consisting of very weak to weak, severely to moderately weathered shale underlain by moderately strong, slightly weathered siltstone. Boring B-002 encountered bedrock at a depth of 18.5 feet below existing grade (about an elevation of 592.3 feet) consisting of very weak, severely weathered shale underlain by slightly strong to strong, highly to slightly weathered siltstone.

Groundwater

Groundwater was encountered in borings B-001-0-22 and B-002-0-22 during drilling at a depth of approximately 18.5 feet and 20 feet below the existing ground surface, corresponding to elevation of about 592 feet and 590.8 feet respectively.

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

Based on the information obtained from the preliminary drawings, we understand that the proposed structure will be Three-Sided Flat-Topped Box type structure founded on drilled shafts. Based on our evaluation of the subsurface conditions encountered at the site, and the requirements outlined in Section 305.4 of ODOT Bridge Design Manual (BDM), it is recommended that deep foundation system consisting of drilled shaft be used for support of the proposed structure. The new structure will generally maintain the existing horizontal and vertical alignments.

Drilled Shaft Foundations

Based on the test borings, we recommend that the drilled shafts should be socketed least 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 piers. 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 bedrock conditions encountered in the test borings.

Drilled Shaft Design

Boring ID	Estimated Top of Rock Socket Elevation (ft) [1]	Minimum Rock Socket Length (feet)	Embedment Material	Minimum Drilled 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-001-0-22	587.0	1.5 x Shaft Diameter	Siltstone Bedrock	36	650	13	0.50 (Tip) 0.55 (Side) 0.4 (uplift resistance)
B-002-0-22	592.3						

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. 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 & 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. Side resistance of drilled shafts can be used to resist either compressive or uplift forces. 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 here-in 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 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 drilled shafts should be designed per BDM section 305.4.

Recommended L-Pile Parameters for Lateral Load Analyses

The following tables 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 3 feet 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.

B-001-0-22

Soil Layer/ Type[1]	Approximate Bottom Elevation of Layer (feet)	LPILE Model	Unit Weight (pcf)		Soil Friction Angle (deg)	Undrained Cohesion (psf)	ε_{50}	K (pci)	Uniaxial Compressive Strength (psi)
			Moist	Buoyant [2]					
Silt and Clay (A-6a)	605.5	Stiff Clay with free water	124	61.5	--	750	0.015	100	--
Clay (A-7-6)	599.5	Soft Clay (Matlock)	122	59.5	--	250	0.02	20	--
Sandy Silt (A-4a)	597.0	Stiff Clay with free water	124	61.5	--	750	0.015	100	--
Clay (A-7-6)	587.0	Stiff Clay with free water	126	63.5	--	1500	0.007	500	--
Shale Bedrock	582.4 [3]	Weak Rock	150	-	-	--	0.0005	-	1,000
Siltstone Bedrock	575.5 [4]	Weak Rock	150	-	-	--	0.00005	-	8,000

1. See test boring logs and **Findings** for more details on stratigraphy.
2. Buoyant unit weight values should be used below the water table. However, designer may assume a depth of water table based on design flood information, whichever conservative.
3. Use Initial Modulus of Rock Mass = 15 ksi, RQD = 40%

4. Boring terminated within this layer. Parameters for this layer should be used for layers below this depth. Use Initial Modulus of Rock Mass = 400 ksi, RQD = 85%

B-002-0-22

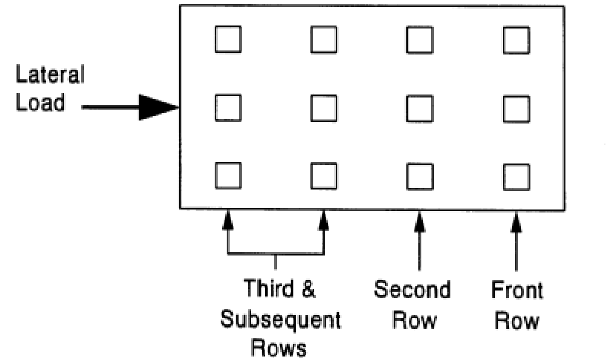
Soil Layer/ Type[1]	Approximate Bottom Elevation of Layer (feet)	LPILE Model	Unit Weight (pcf)		Soil Friction Angle (deg)	Undrained Cohesion (psf)	ε_{50}	K (pci)	Uniaxial Compressive Strength (psi)
			Moist	**Buoyant[2]**					
Gravel with sand, silt and clay (A-2-6)	604.3	Sand	118	55.5	31	--	-	60	--
Silt and Clay (A-6a)	599.8	Soft Clay (Matlock)	123	60.5	--	250	0.02	20	--
Gravel with sand, silt (A-2-4)	592.3	Sand	118	55.5	31	--	-	60	--
Shale Bedrock	590.8 [3]	Rock	150	-	-	--	0.0005	-	1,000
Siltstone Bedrock	580.8 [4]	Rock	150	-	-	--	0.00005	-	8,000

1. See test boring logs and **Findings** for more details on stratigraphy.
5. Buoyant unit weight values should be used below the water table. However, designer may assume a depth of water table based on design flood information, whichever conservative.
2. Use Initial Modulus of Rock Mass = 15 ksi, RQD = 40%
3. Boring terminated within this layer. Parameters for this layer should be used for layers below this depth. Use Initial Modulus of Rock Mass = 400 ksi, RQD = 85%

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.

Drilled Shaft Construction Considerations

In general, drilled shaft installation should be designed and constructed 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.

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 the backfill.

In order to minimize lateral earth pressures, the zone behind the structures should be drained effectively. For effective drainage, a zone of porous backfill (ODOT C&MS Item 518.03) should be used directly behind the structures for a minimum thickness of 2 feet in accordance with ODOT C&MS 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. Prefabricated Geocomposite Drainage (PGD) system in accordance with C&MS Item 518 is another option to provide drainage for the retained earth, if the wall will not experience applied earth pressure exceeding active pressure.

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 C&MS 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. We do not recommend using passive earth pressure resistance in design of permanent retaining walls and/or culvert headwalls due to the potential for erosion, or possibility of removal of the soils in front of the wall in the future.

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 1.35 and 1.5 have been used for the determination of the factored equivalent fluid unit weights under at-rest and active earth pressure conditions respectively. 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). A wedge of granular material have been considered for backfill behind the wall.

For a wedge of granular material, 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.

Level Backslope Behind the Wall				
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	Ka = 0.30
Rigid Retaining Wall – Fixed Head	At-rest[1]	60	81	Ko = 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.

3H: 1V Backslope (18 degrees) Behind the Wall				
Wall Type	**Pressure Distribution**	**Unfactored Equivalent Fluid Weight (pcf)**	**Factored Equivalent Fluid Weight (pcf)**	**Earth Pressure Coefficient**
Cantilever Retaining Wall – Free Head	Active	48	74	K_a = 0.41
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.

For a wedge of drained cohesive/native material, the earth pressure was computed 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.

Level Backslope Behind the Wall

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.

3H: 1V Backslope (18 degrees) Behind the Wall

Wall Type	Pressure Distribution	Unfactored Equivalent Fluid Weight (pcf)	Factored Equivalent Fluid Weight (pcf)	Earth Pressure Coefficient
Cantilever Retaining Wall – Free Head	Active	69	103	$K_a = 0.55$
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.

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. Note that if cohesive material is used as backfill, proper drainage should be provided according to ODOT Item 518, and a wall movement in excess of 0.01H, where H is the height of the wall, is allowed to occur to mobilize an active earth pressure condition.

Surcharge effect (such as traffic) in addition to the vertical load resulting from the weight of any fill and pavement to be placed over the structures should be considered to design the culvert structure. To estimate vertical loading, a total unit weight of 125 pcf may be used for soil.

We do not recommend using passive earth pressure resistance 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

Based on the conditions encountered at the boring locations, it is anticipated that the streambed soils will consist of fine-grained soils that can be classified as clay (A-7-6) and silt and clay (A-6a). Based on the grain size analyses performed for this project, the following table summarizes the D_{50} values encountered at each boring location.

Boring Number	Ground Surface Elevation	Sample Number	Depth (feet)	Elevation (feet)	D_{50} Value (mm)
B-001-0-22	610.5	SS-5	8.0-9.5	602.5 -601.0	0.008
		SS-8	13.5-15.0	597.0-595.5	0.038
B-002-0-22	610.8	SS-7	11.0-12.5	599.8-598.3	0.215

Scour Depth

The Hydraulic Design Flood for this project is a 10-Year (or Q10) flow event, which requires a Scour Design Flood for 25-Year (or Q25) flow event and Scour Check Flood for the 50-year (or Q50) flow event in accordance with the State of Ohio Department of Transportation (ODOT) Location & Design Manual Volume 2 Drainage Design – Section 1008.10.5. The scour analysis utilized the Federal Highway Administration (FHWA) Hydraulic Toolbox 5.2.0.0 (Build Date: October 18, 2022) modeling software in compliance with ODOT Location & Design Manual Volume 2 Drainage Design – Section 1008.10. The scour analysis relied on previously performed hydraulic modeling analysis, as provided by ADR and Associates Ltd. and the hydrologic parameters contained therein for scour analysis data inputs. The scour analysis considered long-term aggradation or degradation scour, contraction scour (live-bed, clear-water, pressure condition), and local scour at the structure abutments. Tabulated below is a summary of calculated scours for the Scour Design Flood and the Scour Check Flood. The scour calculations are presented on the Appendix C.

Flood Event	Abutment	Total Scour Depth (feet)
25- year	Left	3.1
	Right	
50- year	Left	5.2
	Right	5.3

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 soil profile determination extending to a depth of 100 feet for seismic site classification. Borings for this study extended to a maximum depth of approximately 28 feet and this seismic site class definition considers that competent soils 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 soil profile determination.

Construction Considerations

All site work should conform to local codes and to the latest ODOT Construction and Material Specifications (C&MS), including that all structure removal, excavation and embankment preparation and construction should follow ODOT C&MS 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 C&MS Items 203 and 204. Prior to subgrade preparation, perform clearing and grubbing, including removal of stumps and roots, in accordance with ODOT C&MS Item 201. Remove existing pavement and base materials as well as other structures or obstructions, as necessary, in accordance with ODOT C&MS 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.

Earthwork, including subgrade preparation should be performed in accordance with respective items in Section 200 of the current ODOT C&MS. 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 29CFR, Part 1926, Subpart P should be included in the job specifications.

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

Groundwater was encountered in borings B-001-0-22 and B-002-0-22 during drilling at a depth of approximately 18.5 feet and 20 feet below the existing ground surface, corresponding to elevation of about 592 feet and 590.8 feet respectively. 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. Any seepage or groundwater encountered during foundation excavation should be able to be controlled by pumping from temporary sumps. Water from the creek will need to be diverted away from the foundation excavation area during excavation and construction of the foundations. However, additional measures may be required depending on seasonal fluctuations of the stream/groundwater level. Please note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

General Comments

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Variations will occur between exploration point locations 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. Terracon should be retained as the Geotechnical

Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services 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.

Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

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 effect 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. Construction and site development have the potential to affect adjacent properties. Such impacts can include damages due to vibration, modification of groundwater/surface water flow during construction, foundation movement due to undermining or subsidence from excavation, as well as noise or air quality concerns. Evaluation of these items on nearby properties are commonly associated with contractor means and methods and are not addressed in this report. The owner and contractor should consider a preconstruction/precondition survey of surrounding development. 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.

Attachments

APPENDIX A – FIELD EXPLORATION INFORMATION

Contents:

Site Location (Landscape)

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

MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan (Landscape)

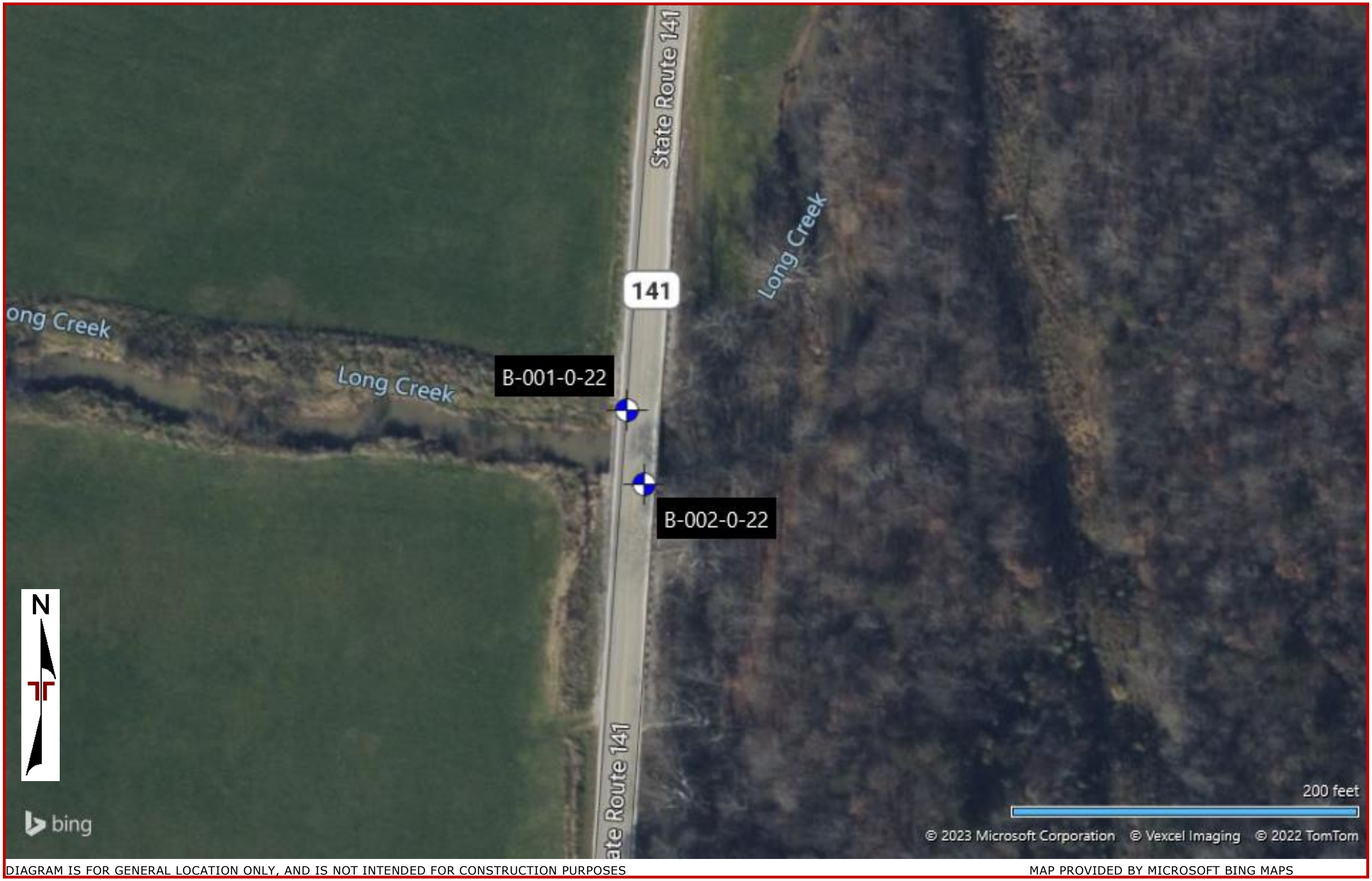
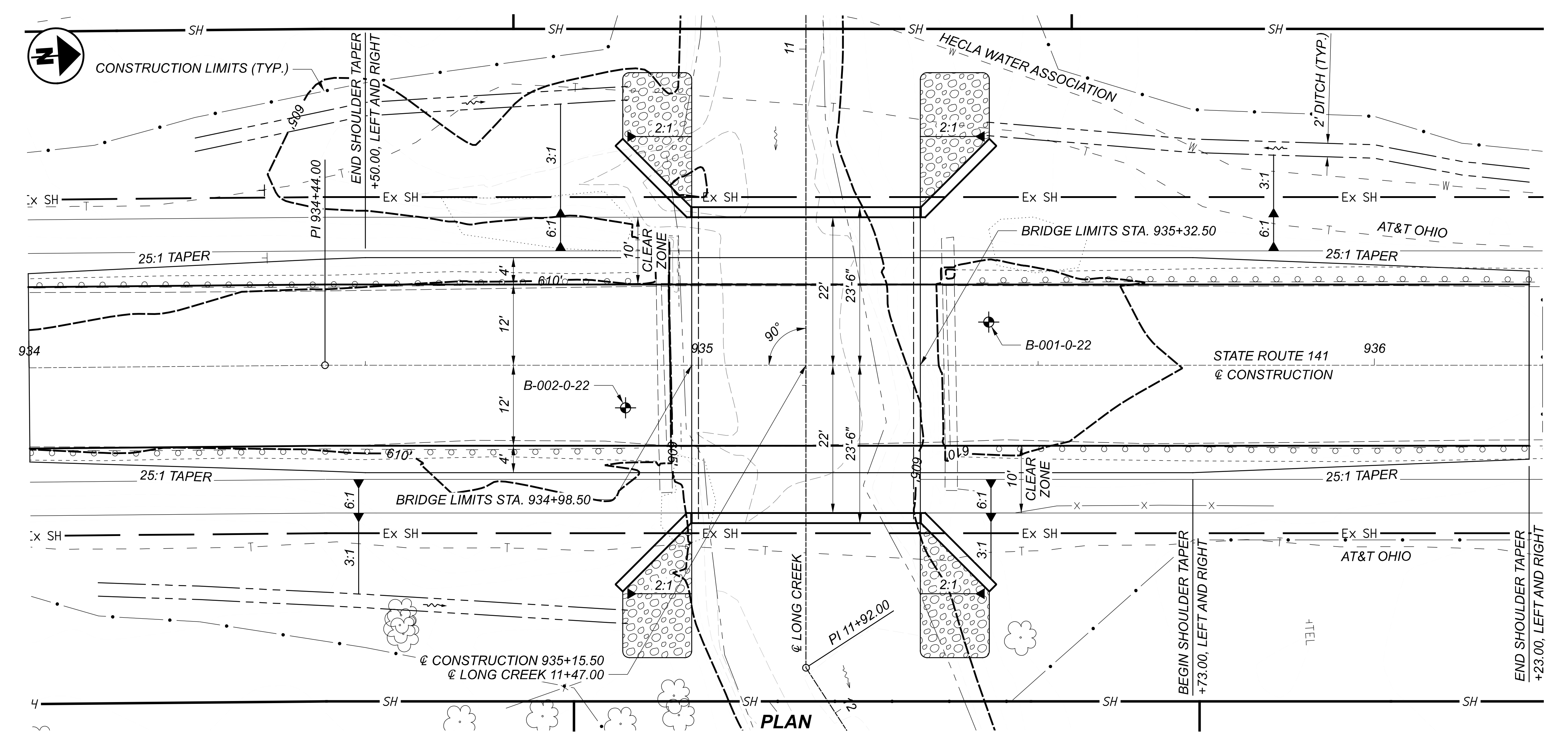
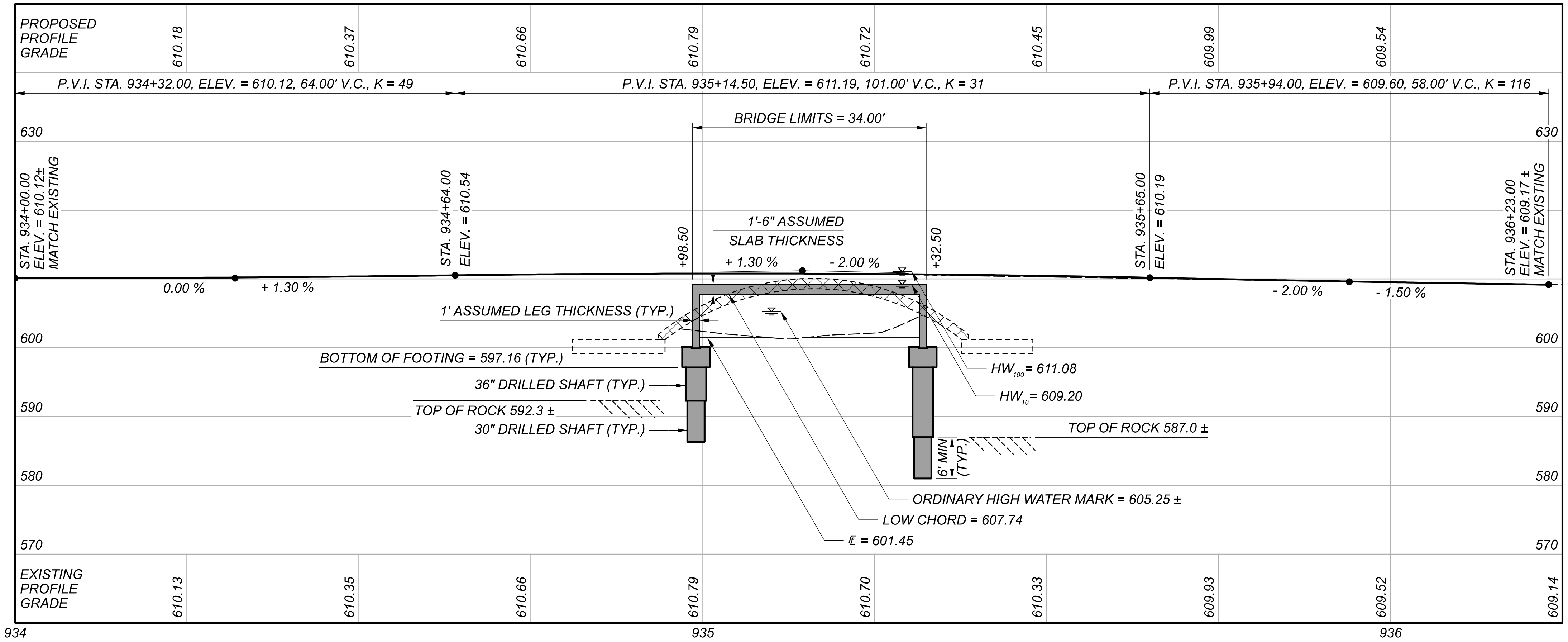


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS





PROFILE ALONG ℄ CONSTRUCTION

BENCHMARK DATA

CP1	STA.928+99.06,	ELEV. 624.90,	OFFSET 16.71', LEFT
CP2	STA. 933+46.50,	ELEV. 609.63,	OFFSET 14.58', LEFT
CP3	STA. 936+97.95,	ELEV. 607.45,	OFFSET 15.72', LEFT

FOR ADDITIONAL BENCHMARK INFORMATION. SEE ROADWAY PLAN SHEET 2.

NOTES

EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

DESIGN TRAFFIC:

2018 ADT = 550	2018 ADTT = 17
2043 ADT = 550	2043 ADTT = 17

DIRECTIONAL DISTRIBUTION = 56%

LEGEND

⊕	BORING LOCATION
▨	EXISTING STRUCTURE TO BE REMOVED
░	ROCK CHANNEL PROTECTION, TYPE B WITH AGGREGATE FILTER, 30" THICK

HYDRAULIC DATA

DRAINAGE AREA = 8.91 SQUARE MILES

Q (10) = 1,430 CFS	V (10) = 7.09 FPS
Q (100) = 2,890 CFS	V (100) = 8.04 FPS

EXISTING STRUCTURE

TYPE: SINGLE SPAN CONCRETE ARCH WITH NON-COMPOSITE BOX BEAM FASCIA BEAMS ON SPREAD FOOTINGS.

SPANS: 40'-0"±

ROADWAY: 24'-0"± F/F GUARDRAIL

LOADING: HS20-44

SKEW: NONE

WEARING SURFACE: ASPHALT CONCRETE

APPROACH SLABS: NONE

ALIGNMENT: TANGENT

CROWN: NORMAL

STRUCTURE FILE NUMBER: 4402618

DATE BUILT: 1928, MAJOR RECONSTRUCTION: 1975

DISPOSITION: TO BE REPLACED

PROPOSED STRUCTURE

TYPE: THREE SIDED FLAT TOPPED CULVERT (32'-0" SPAN X 7'-9" RISE X 47'-0" LENGTH) ON CAPPED PILE ABUTMENTS.

SPANS: 32.00' F/F WALL

ROADWAY: 44'-0" F/F FORESLOPE WALL

LOADING: HL93 AND 60 PSF FUTURE WEARING SURFACE

SKEW: NONE

WEARING SURFACE: ASPHALT CONCRETE

APPROACH SLABS: NONE

ALIGNMENT: TANGENT

CROWN: 0.016 FT / FT

DECK AREA: 1,598 SQUARE FEET

STRUCTURE FILE NUMBER: 4402619

COORDINATES: LATITUDE 38°38'05.73" NORTH
LONGITUDE 82°27'50.53" WEST

HORIZONTAL SCALE IN FEET
0 5 10 20

SITE PLAN
BRIDGE NO. LAW-141-1694
OVER LONG CREEK

SFN
4402619

DESIGN AGENCY
ADR
DESIGN • ENGINEERING • INNOVATION
An Employee Owned Company
88 West Church Street
Newark, OH 43055
(740) 345-1921 (ph)
www.adrinnovation.com

DESIGNER	CHECKER
JTH	RBK

REVIEWER
BCK 3-2-23

PROJECT ID
103975

SUBSET	TOTAL
1	8

SHEET	TOTAL
P.16	28

LAW-141-16.94

MODEL: Sheet PAPERSIZE: 17x11 (in.) DATE: 9/25/2023 TIME: 8:00:00 AM USER: bklingenberg
P:\ADR\2022\22-067 LAW-141-1694 (PID 103975) Bridge Replacement\103975\400-Engineering\Structures\SFN_4402619\Sheets\103975_SFN_4402619_SP001.dgn

APPENDIX B -EXPLORATION RESULTS

Contents:

PROJECT: SR141 BRIDGE REPLACEMENT
TYPE: CULVERT
PID: 103975 SFN: 4402619
START: 1/30/23 END: 1/30/23

DRILLING FIRM / OPERATOR: TERRACON / TOMMY
SAMPLING FIRM / LOGGER: TERRACON / WAYLON
DRILLING METHOD: 3.25" HSA / NQ2
SAMPLING METHOD: SPT

DRILL RIG: MOBILE B-57 ATV #594
HAMMER: AUTOMATIC HAMMER
CALIBRATION DATE: 8/11/22
ENERGY RATIO (%): 75.3

STATION / OFFSET: 935+43, 6' LT.
ALIGNMENT: SR 141
ELEVATION: 610.5 (MSL) EOB: 35.0 ft.
LAT / LONG: 38.635020, -82.464078

EXPLORATION ID B-001-0-22
PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 610.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	ABAN- DONED
								GR	CS	FS	SI	CL	LL	PL	PI			
ASPHALT (14")	609.3																	
AGGREGATE BASE (4")	609.0																	
MEDIUM STIFF, BROWN, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, AGGREGATE BASE FRAGMENTS, DAMP (FILL)			2 2 2	5	6	SS-1	1.00	-	-	-	-	-	-	-	-	10	A-6a (V)	
@3.5'; WEATHERED SANDSTONE FRAGMENTS	605.5		4 2 3	6	17	SS-2	1.00	-	-	-	-	-	-	-	-	12	A-6a (V)	
SOFT, BROWN, **CLAY**, "AND" SILT, LITTLE TO TRACE SAND, TRACE GRAVEL, WEATHERED SANDSTONE FRAGMENTS, MOIST			2 1 2	4	33	SS-3	0.25	-	-	-	-	-	-	-	-	28	A-7-6 (V)	
			0 1 0	1	6	SS-4	0.25	-	-	-	-	-	-	-	-	20	A-7-6 (V)	
			1 1 1	3	100	SS-5	0.25	4	0	5	51	40	43	21	22	36	A-7-6 (13)	
	599.5		4 2 2	5	56	SS-6	0.25	-	-	-	-	-	-	-	-	33	A-7-6 (V)	
STIFF, BROWN, **SANDY SILT**, TRACE GRAVEL, WEATHERED SANDSTONE FRAGMENTS, MOIST	597.0		3 2 2	5	22	SS-7	1.25	-	-	-	-	-	-	-	-	19	A-4a (V)	
STIFF, BROWN, **CLAY**, "AND" SAND, LITTLE SILT, TRACE GRAVEL, WEATHERED SHALE AND SANDSTONE FRAGMENTS, MOIST			1 3 4	9	100	SS-8	1.00	9	15	24	20	32	43	16	27	20	A-7-6 (10)	
@18.5'; BROWN AND GRAY	587.0	W 592.0	2 4 5	11	100	SS-9	-	-	-	-	-	-	-	-	-	25	A-7-6 (V)	
SHALE, GRAY, SEVERELY WEATHERED, VERY WEAK.	585.5	TR	44 50/2"	-	88	SS-10	-	-	-	-	-	-	-	-	-	9	Rock (V)	
SHALE, GRAY, HIGHLY TO MODERATELY WEATHERED, WEAK, SLIGHTLY, LAMINATED TO THIN BEDDED, ARGILLACEOUS, FRACTURED TO MODERATELY FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 58%, REC 98%. @27.4'-27.7'; Unit Weight = 157 pcf; Qu =1,076 psi	582.4		58		98	NQ2-R1											CORE	
SILTSTONE, GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, SLIGHTLY, ARENACEOUS, MODERATELY TO SLIGHTLY FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 78%, REC 97%.	575.5	EOB	78		97	NQ2-R2											CORE	

NOTES: AUGER REFUSAL @25'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; MIXED WITH BENTONITE CHIPS

PROJECT: SR141 BRIDGE REPLACEMENT	DRILLING FIRM / OPERATOR: TERRACON / TOMMY	DRILL RIG: MOBILE B-57 ATV #594	STATION / OFFSET: 934+89, 6' RT.	EXPLORATION ID B-002-0-22
TYPE: CULVERT	SAMPLING FIRM / LOGGER: TERRACON / WAYLON	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: SR 141	
PID: 103975 SFN: 4402619	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 8/11/22	ELEVATION: 610.8 (MSL) EOB: 30.0 ft.	PAGE 1 OF 1
START: 1/30/23 END: 1/30/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 75.3	LAT / LONG: 38.634903, -82.464043	

MATERIAL DESCRIPTION AND NOTES	ELEV. 610.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	ABAN- DONED
ASPHALT (13")	609.7																	
AGGREGATE BASE (5")	609.3																	
LOOSE, BROWN, **GRAVEL WITH SAND, SILT, AND CLAY**, AGGREAGTE BASE AND SANSTONE FRAGMENTS, MOIST (FILL)			2 3 3	8	83	SS-1	-	24	13	29	17	17	28	15	13	15	A-2-6 (1)	
			2 3 4	9	11	SS-2	-	-	-	-	-	-	-	-	-	18	A-2-6 (V)	
	604.3		8 4 2	8	100	SS-3	-	-	-	-	-	-	-	-	-	17	A-2-6 (V)	
SOFT, BROWN, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, WEATHERED SANSTONE AND SHALE FRAGMENTS, MOIST			2 2 2	5	100	SS-4	0.25	-	-	-	-	-	-	-	-	20	A-6a (V)	
			0 1 1	3	100	SS-5	0.25	-	-	-	-	-	-	-	-	26	A-6a (V)	
@ 9.5'-11'; SLIGHTLY ORGANIC	599.8		1 2 2	5	100	SS-6	0.25	-	-	-	-	-	-	-	-	20	A-6a (V)	
LOOSE, BROWN, **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, WEATHERED SANSTONE AND SHALE FRAGMENTS, WET			2 3 3	8	50	SS-7	-	6	27	44	12	11	21	13	8	18	A-2-4 (0)	
	592.3	TR	2 3 3	8	83	SS-8	-	-	-	-	-	-	-	-	-	-	A-2-4 (V)	
SHALE, GRAY, SEVERELY WEATHERED, VERY WEAK.	590.8	W 590.8	50/5"	-	60	SS-9	-	-	-	-	-	-	-	-	-	10	Rock (V)	
SILTSTONE, GRAY, HIGHLY TO MODERATELY WEATHERED, SLIGHTLY STRONG, VERY FINE TO FINE GRAINED, ARENACEOUS, HIGHLY FRACTURED TO FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 52%, REC 100%.	585.8		52		100	NQ2-R1											CORE	
SILTSTONE, GRAY, SLIGHTLY WEATHERED, STRONG, VERY FINE TO FINE GRAINED, MODERATELY FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 95%, REC 100%. @28.0' -28.3'; Unit Weight = 163 pcf; Qu = 8,037 psi	580.8	EOB	95		100	NQ2-R2											CORE	

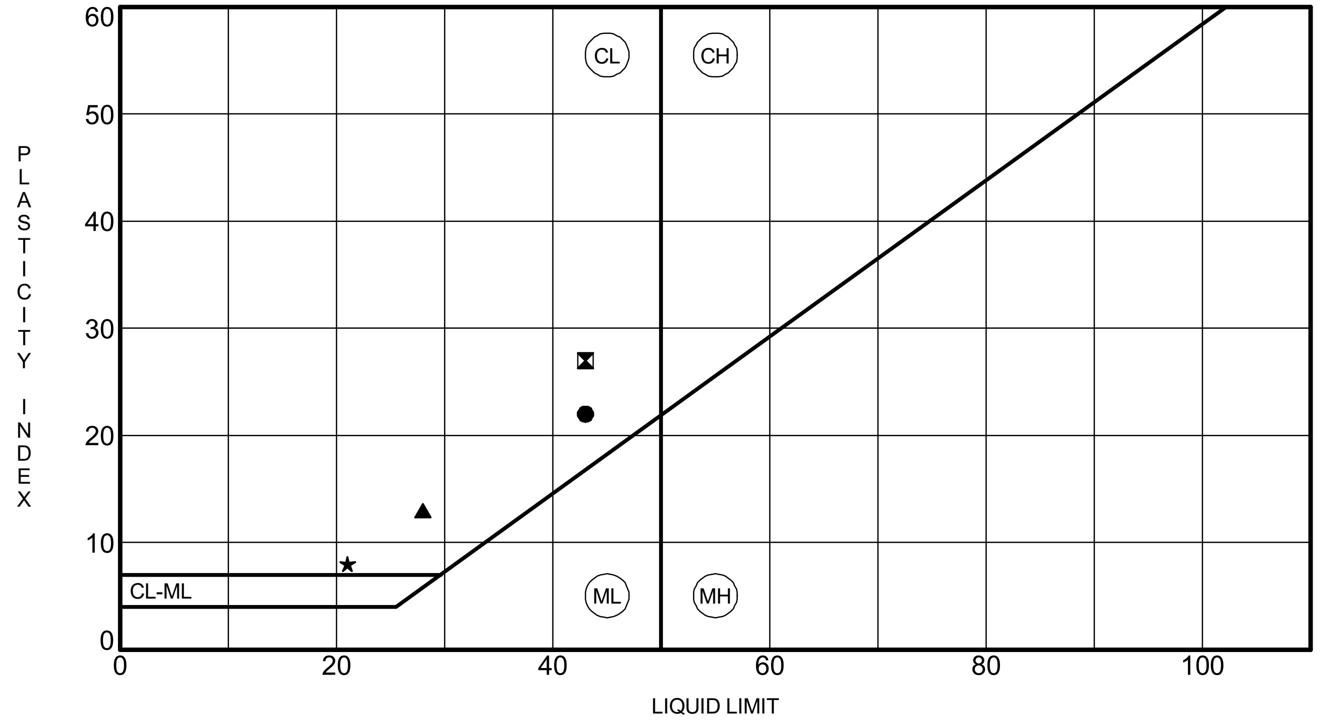
NOTES: AUGER REFUSAL @20'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; MIXED WITH BENTONITE CHIPS

ATTERBERG LIMITS' RESULTS

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT SR 141 BRIDGE REPLACEMENT
PID 103975
OGE NUMBER N4225280
PROJECT TYPE STRUCTURE FOUNDATION

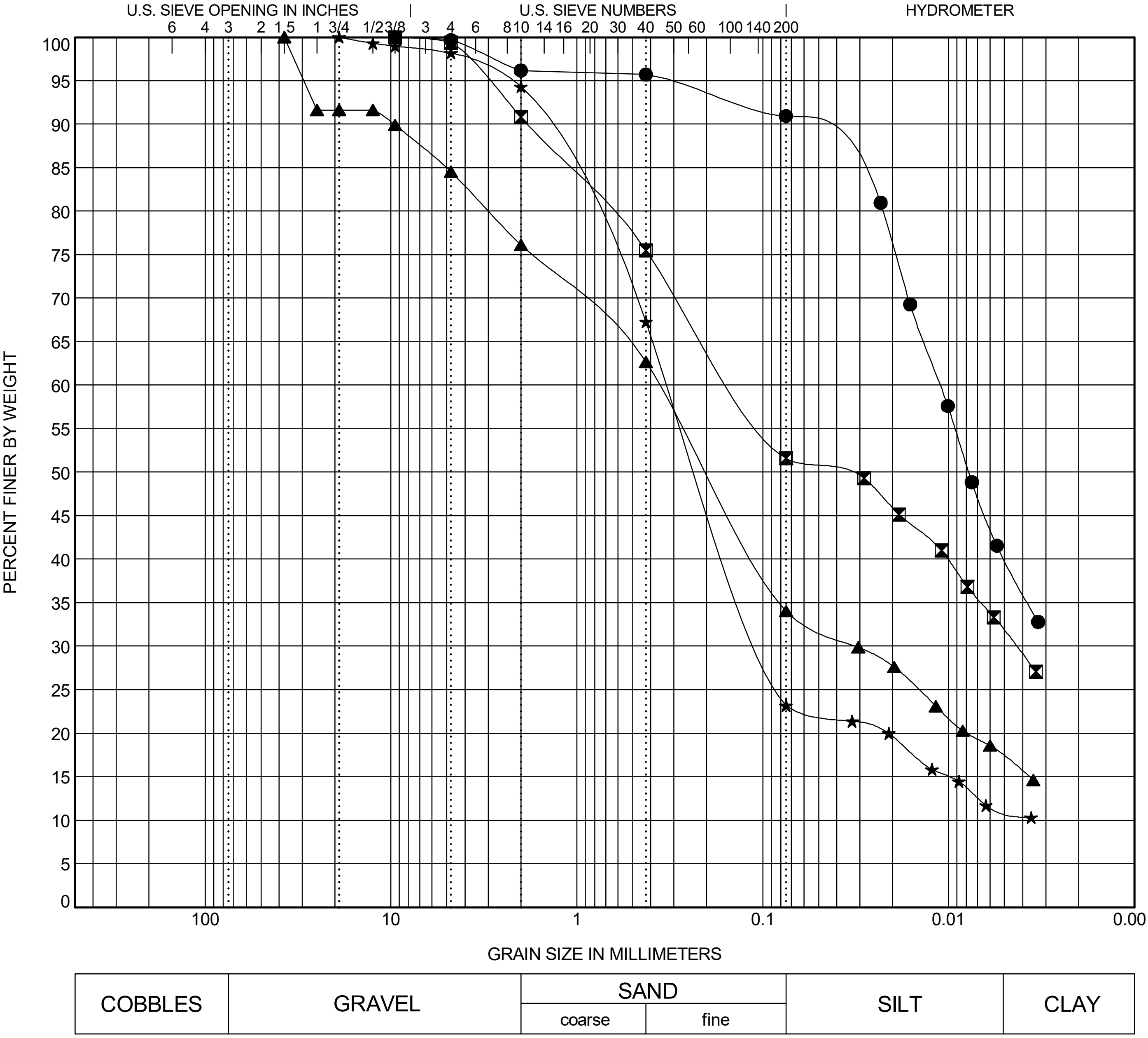


	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-001-0-22**	**8.0**	**43**	**21**	**22**	**91**	**LEAN CLAY(CL)**
☒	**B-001-0-22**	**13.5**	**43**	**16**	**27**	**52**	**SANDY LEAN CLAY(CL)**
▲	**B-002-0-22**	**2.0**	**28**	**15**	**13**	**34**	**CLAYEY SAND with GRAVEL(SC)**
★	**B-002-0-22**	**11.0**	**21**	**13**	**8**	**23**	**CLAYEY SAND(SC)**

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT SR 141 BRIDGE REPLACEMENT
PID 103975
OGE NUMBER N4225280
PROJECT TYPE STRUCTURE FOUNDATION



Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-001-0-22 8.0**	**A-7-6 ~ LEAN CLAY(CL)**	**43**	**21**	**22**
☒	**B-001-0-22 13.5**	**A-7-6 ~ SANDY LEAN CLAY(CL)**	**43**	**16**	**27**
▲	**B-002-0-22 2.0**	**A-2-6 ~ CLAYEY SAND with GRAVEL(SC)**	**28**	**15**	**13**
★	**B-002-0-22 11.0**	**A-2-4 ~ CLAYEY SAND(SC)**	**21**	**13**	**8**

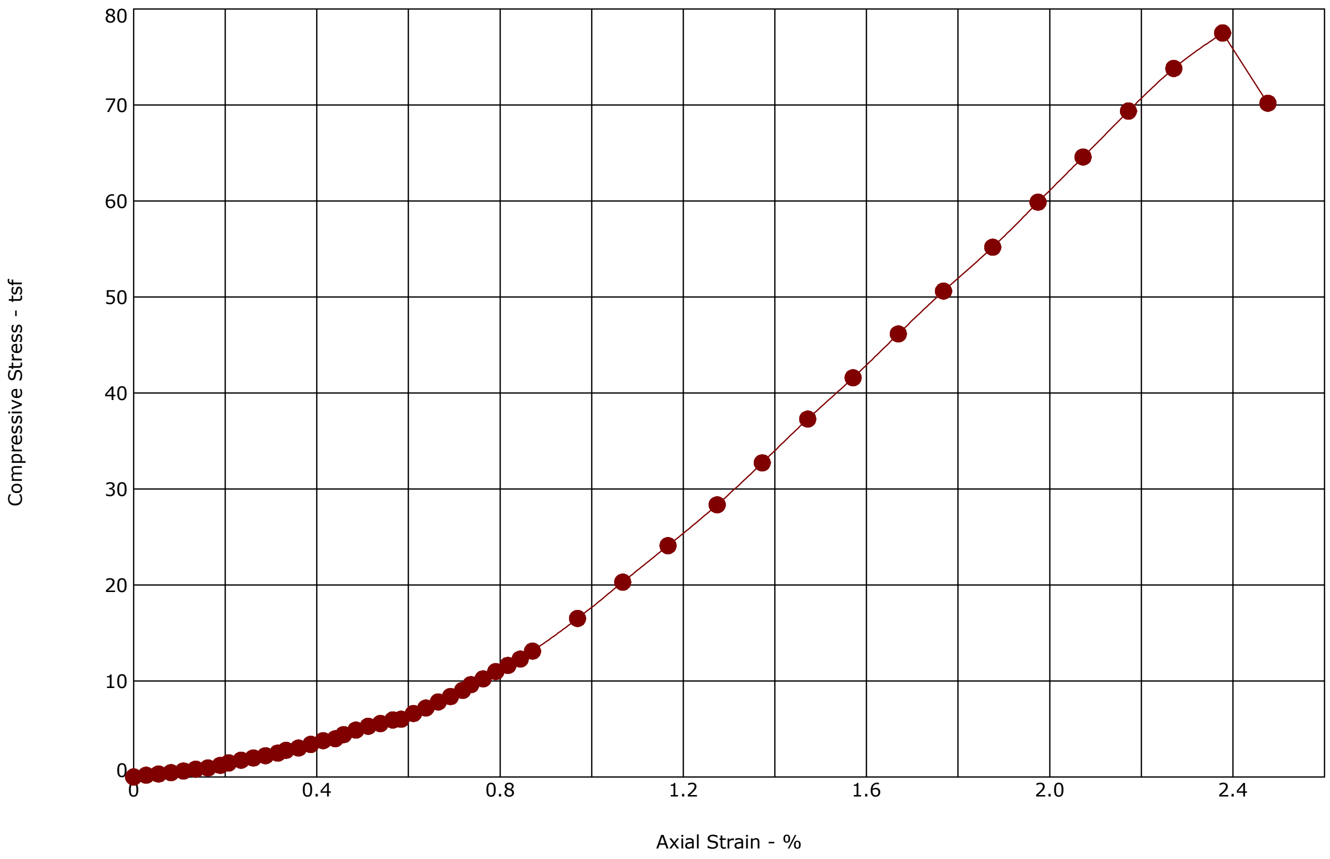
Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-001-0-22 8.0**	**0.067**	**0.008**			**4**	**0**	**5**	**51**	**40**		
☒	**B-001-0-22 13.5**	**1.841**	**0.038**	**0.004**		**9**	**15**	**24**	**20**	**32**		
▲	**B-002-0-22 2.0**	**9.634**	**0.197**	**0.032**		**24**	**13**	**29**	**17**	**17**		
★	**B-002-0-22 11.0**	**1.562**	**0.215**	**0.098**		**6**	**27**	**44**	**12**	**11**		

LAW-141-1694 (PID 103975)
SR141 & Highway 536 | Wilgus, OH
Terracon Project No. N4225280

800 Morrison Rd
Gahanna, OH

Unconfined Compression Test

ASTM D7012 "C"



Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B-001-0-22	27.4 - 27.7	NQ2-R1					SHALE

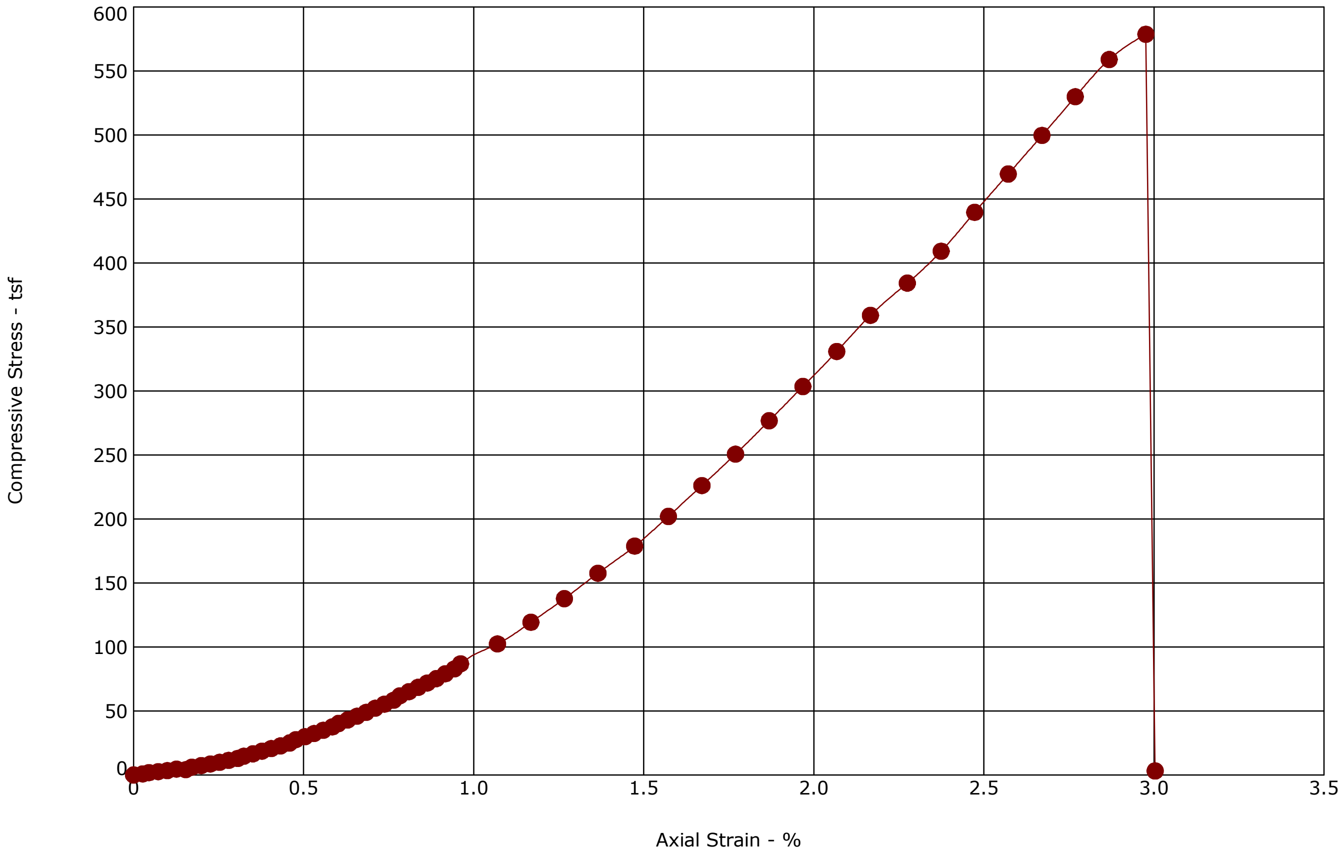
Specimen Failure Mode

Specimen Test Data

Moisture Content (%):	2.1
Dry Density (pcf):	157
Diameter (in.):	1.96
Height (in.):	4.01
Height / Diameter Ratio:	2.05
Calculated Saturation (%):	
Calculated Void Ratio:	
Assumed Specific Gravity:	
Failure Strain (%):	2.38
Unconfined Compressive Strength (tsf):	77.50
Undrained Shear Strength (tsf):	38.75
Strain Rate (in/min):	0.0404
Remarks:	

Unconfined Compression Test

ASTM D7012 "C"



Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B-002-0-22	28.0 - 28.3	NQ2- R2					SILTSTONE

Specimen Failure Mode

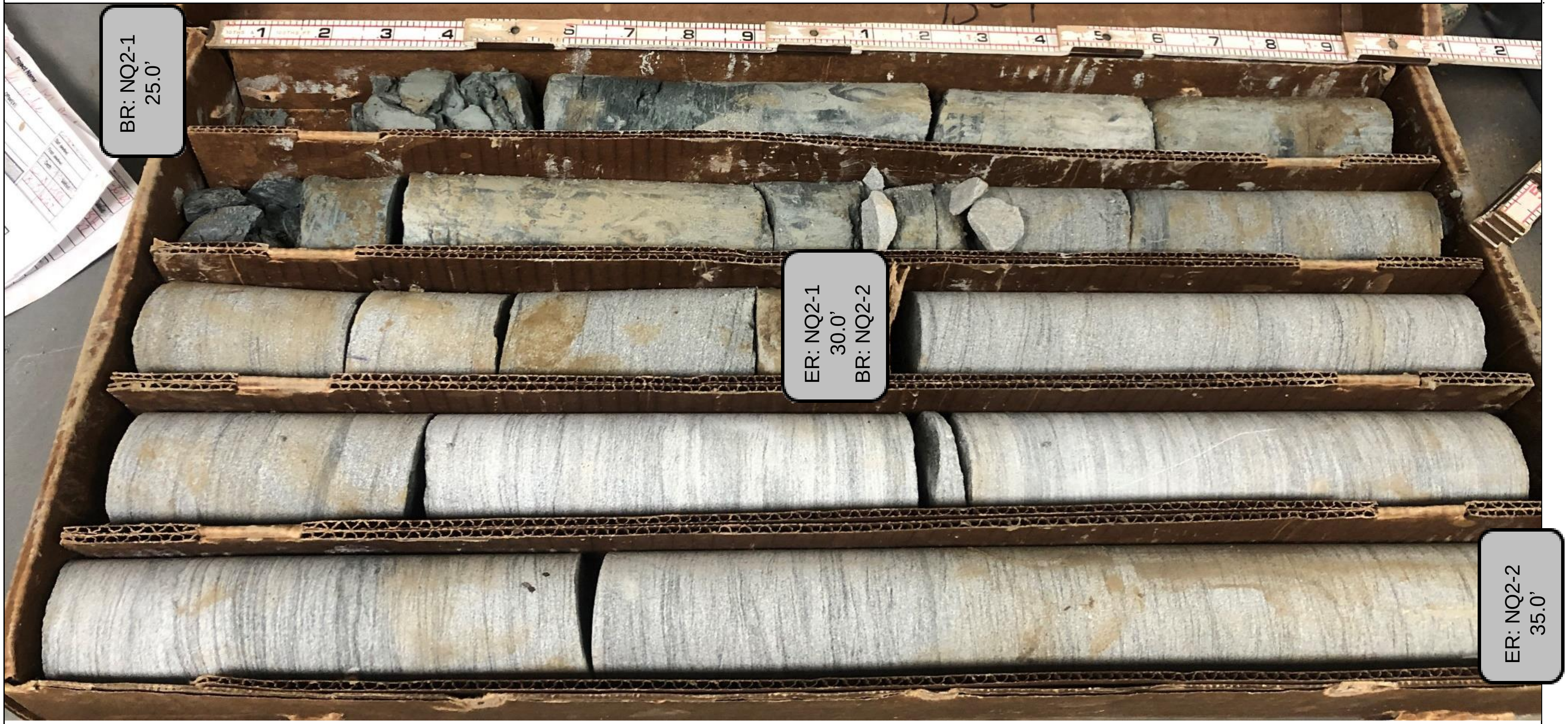
Specimen Test Data

Moisture Content (%):	0.5
Dry Density (pcf):	163
Diameter (in.):	1.98
Height (in.):	4.05
Height / Diameter Ratio:	2.04
Calculated Saturation (%):	
Calculated Void Ratio:	
Assumed Specific Gravity:	
Failure Strain (%):	2.98
Unconfined Compressive Strength (tsf):	578.71
Undrained Shear Strength (tsf):	289.36
Strain Rate (in/min):	0.0405
Remarks:	

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

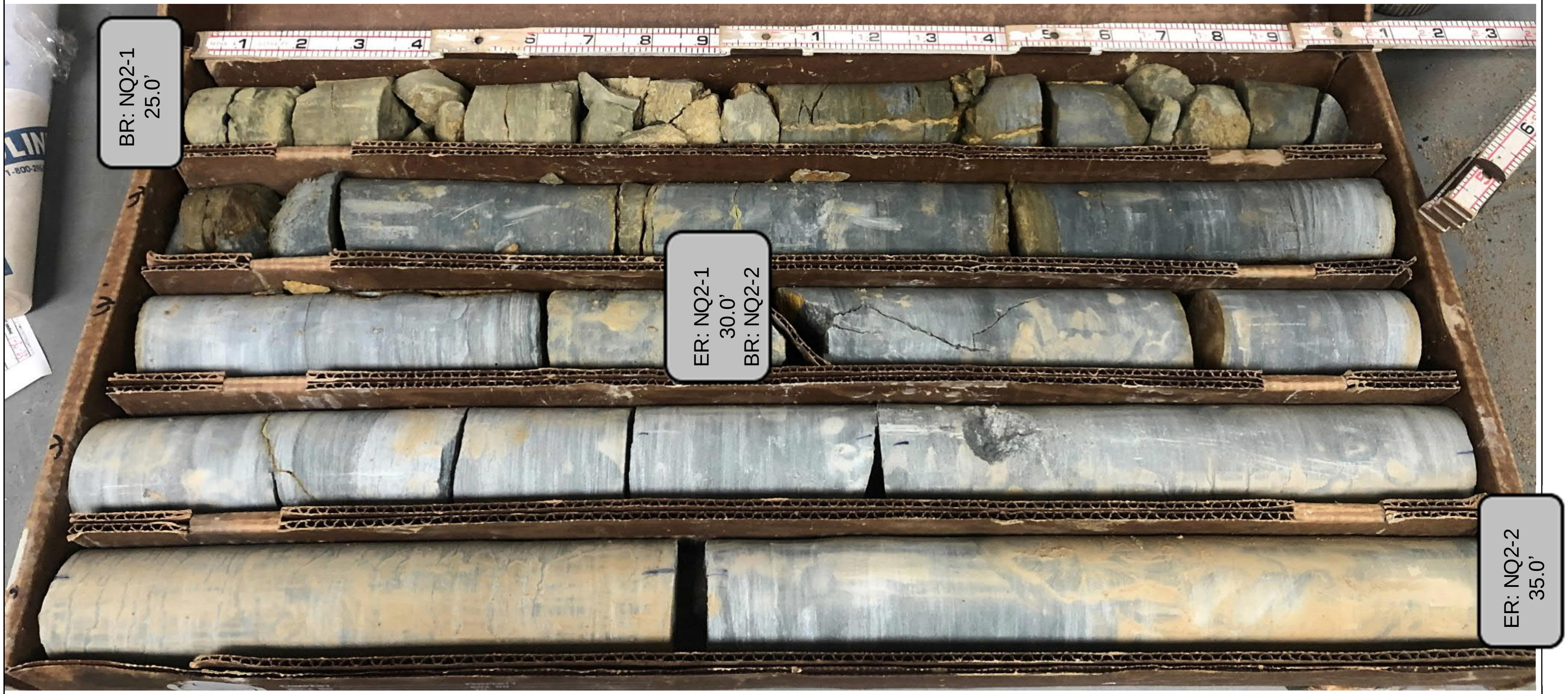
B-001-0-22



Run #:	Depth		Recovery		RQD	
NQ2-1	25.0’	30.0’	59/60	98%	35/60	58%
NQ2-2	30.0’	35.0’	58/60	97%	47/60	78%

LAW-141-1694 PID 103975

B-002-0-22



Run #:	Depth		Recovery		RQD	
NQ2-1	25.0’	30.0’	60/60	100%	31/60	52%
NQ2-2	30.0’	35.0’	60/60	100%	57/60	95%
LAW-141-1694 PID 103975						

APPENDIX C – SUPPORTING INFORMATION

Contents:

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]	$Cu\geq4$ and $1\leq Cc\leq3$ [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]	$Cu\geq6$ and $1\leq Cc\leq3$ [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.

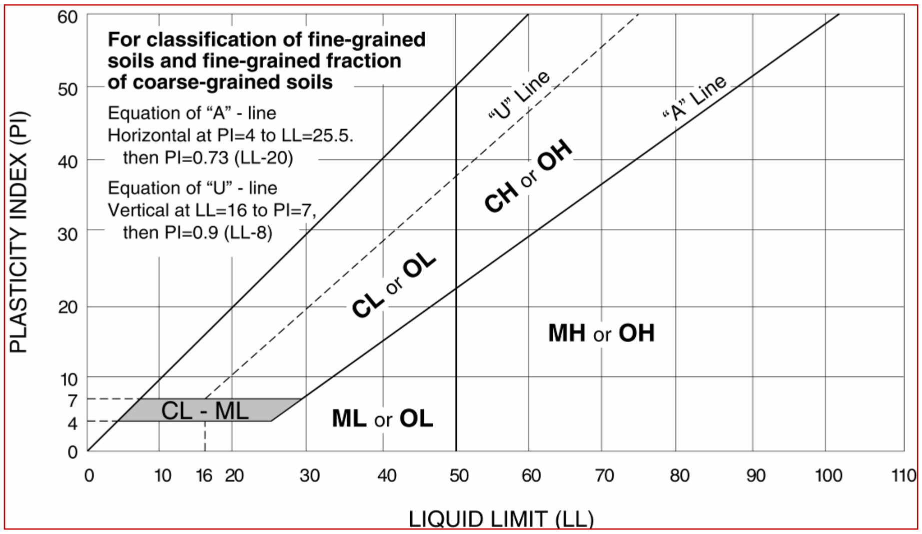
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[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.



Rock Classification Notes

WEATHERING

Term	Description
Fresh	Mineral crystals appear bright; show no discoloration. Features show little or now staining on surfaces. Discoloration does not extend into intact rock.
Slightly weathered	Rock generally fresh except along fractures. Some fractures stained and discoloration may extend <0.5 inches into rock.
Moderately weathered	Significant portions of rock are dull and discolored. Rock may be significantly weaker than in fresh state near fractures. Soil zones of limited extent may occur along some fractures.
Highly weathered	Rock dull and discolored throughout. Majority of rock mass is significantly weaker and has decomposed and/or disintegrated; isolated zones of stronger rock and/or soil may occur throughout.
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The rock mass or fabric is still evident and largely intact. Isolated zones of stronger rock may occur locally.

STRENGTH OR HARDNESS

Description	Field Identification	Uniaxial Compressive Strength, psi
Extremely strong	Can only be chipped with geological hammer. Rock rings on hammer blows. Cannot be scratched with a sharp pick. Hand specimens require several hard hammer blows to break.	>36,000
Very strong	Several blows of a geological hammer to fracture. Cannot be scratched with a 20d common steel nail. Can be scratched with a geologist's pick only with difficulty.	15,000-36,000
Strong	More than one blow of a geological hammer needed to fracture. Can be scratched with a 20d nail or geologist's pick. Gouges or grooves to ¼ inch deep can be excavated by a hard blow of a geologist's pick. Hand specimens can be detached by a moderate blow.	7,500-15,000
Medium strong	One blow of geological hammer needed to fracture. Can be distinctly scratched with 20d nail. Can be grooved or gouged 1/16 in. deep by firm pressure with a geologist's pick point. Can be fractured with single firm blow of geological hammer. Can be excavated in small chips (about 1-in. maximum size) by hard blows of the point of a geologist's pick;	3,500-7,500
Weak	Shallow indent by firm blow with geological hammer point. Can be gouged or grooved readily with geologist's pick point. Can be excavated in pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.	700-3,500
Very weak	Crumbles under firm blow with geological hammer point. Can be excavated readily with the point of a geologist's pick. Pieces 1-in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.	150-700

DISCONTINUITY DESCRIPTION

Fracture Spacing (Joints, Faults, Other Fractures)		Bedding Spacing (May Include Foliation or Banding)	
Description	**Spacing**	**Description**	**Spacing**
Intensely fractured	< 2.5 inches	**Laminated**	< ½-inch
Highly fractured	2.5 – 8 inches	**Very thin**	½ – 2 inches
Moderately fractured	8 inches to 2 feet	**Thin**	2 inches – 1 foot
Slightly fractured	2 to 6.5 feet	**Medium**	1 – 3 feet
Very slightly fractured	> 6.5 feet	**Thick**	3 – 10 feet
		Massive	> 10 feet

ROCK QUALITY DESIGNATION (RQD) [1]

Description	RQD Value (%)
Very Poor	0 - 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 - 100

[1] The combined length of all sound and intact core segments equal to or greater than 4 inches in length, expressed as a percentage of the total core run length.

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

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

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

4) COMPONENT MODIFIERS:

Description	Percentage By Weight
Trace	0% - 10%
Little	10% - 20%
Some	20% - 35%
“And”	35% -50%

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		$LL_O/LL \times 100$*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
		AASHTO	OHIO							
○	Gravel and/or Stone Fragments	A-1-a			30 Max.	15 Max.		6 Max.	0	Min. of 50% combined gravel, cobble and boulder sizes
○	Gravel and/or Stone Fragments with Sand	A-1-b			50 Max.	25 Max.		6 Max.	0	
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.	≤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
Calcareous - contains calcium carbonate
Conglomeritic - contains rounded to subrounded gravel
Ferriferous – contains iron
Friable – easily broken down
Siliceous – contains silica

Argillaceous - clayey
Carbonaceous - contains carbon
Crystalline – contains crystalline structure
Fissile – thin planner partings
Micaceous – contains mica
Stylolitic – contain stylotites (suture like structure)

Brecciated – contains angular to subangular gravel
Cherty- contains chert fragments
Dolomitic- contains calcium/magnesium carbonate
Fossiliferous – contains fossils
Pyritic – contains pyrite
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

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

9: GSI DESCRIPTION

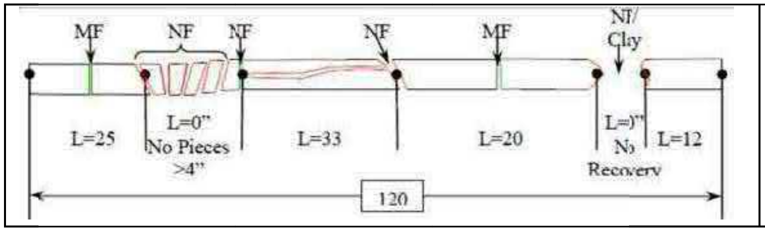
a: Structure

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:	ADR
PROJECT:	SR-141
W.O.:	N4225280
Date:	3/9/2023
Case:	Drilled Shaft (for borings B-001-0-22 & B-002-0-22)

UCS and RQD Data

Boring	RQD (per 10 ft.)
B-001-0-22	55
B-002-0-22	59

Average	57

Boring	Depth	UCS (ksf)
B-001-0-22	27	155
B-002-0-22	28	1157

Harmonic Average	273

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 =	686.07	ksi	(Elastic modulus of the rock mass per Eq. 10.4.6.5-1)
E_i =	1300	ksi	(Elastic modulus of intact rock)
E_m / E_i =	0.53		

αE =	0.84

P_a =	2.12	ksf	(Atmospheric Pressure)
f'_c =	4	ksi	(Concrete compressive strength)
q_u =	273.3765	ksf	(Uniaxial compressive strength of rock)

q_s = 13.14 < 22.71 OK

Therefore,
q_s = 13.14 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 =	**7.2**	**ksf**
Factored Uplift Resistance =	**5.3**	**ksf**

Bearing Capacity per LRFD 10.8.3.5.4c

q_p = 2.5 q_u (Per 10.8.3.5.4c-1 if rock below base of the drilled shaft to a depth of 2.0B is intact or tightly Jointed)

q_p = 683.441311 ksf

$q_p = (s^{0.5} + (mxs^{0.5} + s)^{0.5})$ x qu (Per 10.8.3.5.4c-2 if rock below base of shaft to 2.0B is jointed)

s = 0.00009
m = 0.183

q_p = 14.28 ksf Use 10 ksf

Is rock below base of the drilled shaft to a depth of 2.0B intact or tightly jointed? Yes

q_p = 683.44 ksf use 650 ksf

Resistance Factors for Tip Resistance per LRFD Table 10.5.5.2.4-1

Resistance Factor for Tip Resistance in rock	0.5

Factored Bearing Resistance =	**341.7**	**ksf**

Scour Analysis
Hydraulic Toolbox Report

Hydraulic Toolbox Report
(Pressure Flow)

Hydraulic Analysis Report

Project Data

Project Title: LAW 141-1694

Designer: CFB

Project Date: Wednesday, March 1, 2023

Project Units: U.S. Customary Units

Notes: Utilized HEC-RAS V 6.3.1 and project files as provided by ADR & Associates, LTD.

Bridge Scour Analysis:25 YEAR SCOUR

Notes: Pressure

Scenario: 25 YEAR

Long Term Degradation

Controlled by Equilibrium Slope

Long Term Degradation (LTD) 0.46 ft

Long Term Degradation does not apply, due to Pressure Contraction Scour

Contraction Scour Summary

Pressure Flow

Applied Contraction Scour Depth 3.09 ft

Contraction and Long Term Degradation do NOT apply

Contraction Scour Depth and Long Term Degradation (LTD) 1.02 ft

Clear Water Contraction Scour Depth 25.98 ft

Live Bed Contraction Scour Depth 1.02 ft

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: Froehlich Method

Abutment Scour Depth 0.79 ft

Total Scour at Abutment 0.00 ft

Right Abutment

Abutment Scour Method: Froehlich Method

Abutment Scour Depth 0.89 ft

Total Scour at Abutment 0.00 ft

Long Term Details

Long-Term Degradation

Computation Type: Controlled by Equilibrium Slope

Input Parameters

Slope Equation: No Sediment Supply, Meyer-Peter, Muller

D50: 0.196901 mm

D90: 9.634118 mm

Shield's Parameter: 0.0390

Depth or Hydraulic Radius: 7.29 ft

Manning's n Value: 0.0450

Discharge Per Unit Width: 61.60 cfs/ft

Current Slope: 0.0008 ft/ft

Distance Upstream of Base Level Control: 579.00 ft

Result Parameters

Equilibrium Slope: 0.000003 ft/ft

Ultimate Degradation Amount: 0.46 ft

Main Channel Contraction Scour

Computation Type:

Input Parameters

Average Depth Upstream of Contraction: 7.29 ft

D50: 0.196901 mm

Average Velocity Upstream: 3.53 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 1.34 ft/s

Contraction Scour Condition: Live-Bed

Live Bed and/or Clear Water Input Parameters

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section: 0.0130 ft/ft

Flow in Contracted Section: 1970.00 cfs

Flow Upstream that is Transporting Sediment: 1970.00 cfs

Width in Contracted Section: 32.00 ft

Width Upstream that is Transporting Sediment: 33.50 ft

Depth Prior to Scour in Contracted Section: 6.50 ft

Unit Weight of Water: 62.40 lb/ft^3

Unit Weight of Sediment: 165.00 lb/ft^3

Results of Live Bed Method

Shear Velocity: 1.75 ft/s

Fall Velocity: 0.08 ft/s

Average Depth in Contracted Section after Scour: 7.52 ft

Scour Depth for Live Bed: 1.02 ft

Scour may be limited by armoring. Compute all methods to check.

Upstream Channel Flow Depth: 7.29 ft

Average Velocity Upstream: 3.53 ft/s

D50: 0.196901 mm

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 1.34 ft/s

Contraction Scour Condition: Live Bed

Input Parameters for Bridge Scour
Width of the Contracted Section: 32.00 ft

Flow through bridge opening: 1970.00 cfs

Width of the Upstream Section: 33.50 ft

Flow in Upstream Section: 1970.00 cfs

Slope of Energy Grade Line at Approach Section: 0.0130 ft/ft

Vertical Size of Bridge Opening Prior to Scour: 6.50 ft

Deck Thickness: 3.00 ft

Result Parameters
K1: 0.69

Diameter of Smallest Non-moving Particle: 0.246126 mm

Average Depth In Contracted Section: 7.52 ft

Flow Separation Thickness: 2.07 ft

Scour Depth: 3.09 ft

Left Abutment Details

Abutment Scour
Computation Type: Froehlich's

Input Parameters

Froehlich's Method
Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.00 Degrees

Centerline Length of Embankment: 761.00 ft

Projected Length of Embankment: 761.00 ft

Length of Active Flow Obstructed by Embankment: 0.58 ft

Flow Obstructed by Abutment and Approach Embankment: 214.91 cfs

Flow Area Obstructed by Embankment: 367.73 ft^2

Result Parameters

Average Depth of Flow on the Floodplain: 0.48 ft

Average Velocity: 0.58 ft/s

Froude Number: 0.15

Length to Depth Ratio: 1574.85

Scour Hole Depth from Froehlich Method: 0.79 ft

Right Abutment Details

Abutment Scour

Computation Type: Froehlich's

Input Parameters

Froehlich's Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.00 Degrees

Centerline Length of Embankment: 249.49 ft

Projected Length of Embankment: 249.49 ft

Length of Active Flow Obstructed by Embankment: 0.66 ft

Flow Obstructed by Abutment and Approach Embankment: 87.72 cfs

Flow Area Obstructed by Embankment: 133.53 ft^2

Result Parameters

Average Depth of Flow on the Floodplain: 0.54 ft

Average Velocity: 0.66 ft/s

Froude Number: 0.16

Length to Depth Ratio: 466.15

Scour Hole Depth from Froehlich Method: 0.89 ft

Scour Summary Table

Long Term Degradation

Parameter	Value	Units	Notes
Pier Name			
Long Term Degradation (LTD)	0.46	ft	Controlled by Equilibrium Slope

Contraction Scour

Parameter	Value	Units	Notes
Applied Contraction Scour Depth	3.09	ft	Pressure Flow
Contraction Scour Depth and Long Term Degradation (LTD)	1.02	ft	Contraction and Long Term Degradation do NOT apply
Live Bed Contraction Scour Depth	1.02	ft	
Pressure Scour Depth	3.09	ft	

Local Scour at Abutments

Parameter	Value	Units	Notes
Abutment scour currently cannot be computed with pressure flow! Use an abutment scour countermeasure with pressure flow.			

Bridge Scour Analysis:50 YEAR SCOUR

Notes: Utilized HEC-RAS V 6.3.1 and project files as provided by ADR & Associates, LTD.

Scenario: 50 YEAR

Long Term Degradation

Controlled by Equilibrium Slope

Long Term Degradation (LTD) 0.46 ft

Long Term Degradation does not apply, due to Pressure Contraction Scour

Contraction Scour Summary

Pressure Flow

Applied Contraction Scour Depth 4.57 ft

Contraction and Long Term Degradation do NOT apply

Contraction Scour Depth and Long Term Degradation (LTD) 2.41 ft

Clear Water Contraction Scour Depth 32.10 ft

Live Bed Contraction Scour Depth 2.41 ft

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: Froehlich Method

Abutment Scour Depth 2.30 ft

Total Scour at Abutment 0.00 ft

Right Abutment

Abutment Scour Method: Froehlich Method

Abutment Scour Depth 2.43 ft

Total Scour at Abutment 0.00 ft

Long Term Details

Long-Term Degradation

Computation Type: Controlled by Equilibrium Slope

Input Parameters

Slope Equation: No Sediment Supply, Meyer-Peter, Muller

D50: 0.196901 mm

D90: 9.634118 mm

Shield's Parameter: 0.0390

Depth or Hydraulic Radius: 8.63 ft

Manning's n Value: 0.0450

Discharge Per Unit Width: 75.30 cfs/ft

Current Slope: 0.0008 ft/ft

Distance Upstream of Base Level Control: 579.00 ft

Result Parameters

Equilibrium Slope: 0.000003 ft/ft

Ultimate Degradation Amount: 0.46 ft

Main Channel Contraction Scour

Computation Type:

Input Parameters

Average Depth Upstream of Contraction: 8.63 ft

D50: 0.196901 mm

Average Velocity Upstream: 1.93 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 1.38 ft/s

Contraction Scour Condition: Live-Bed

Live Bed and/or Clear Water Input Parameters

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section: 0.0195 ft/ft

Flow in Contracted Section: 2410.00 cfs

Flow Upstream that is Transporting Sediment: 2410.00 cfs

Width in Contracted Section: 32.00 ft

Width Upstream that is Transporting Sediment: 33.50 ft

Depth Prior to Scour in Contracted Section: 6.50 ft

Unit Weight of Water: 62.40 lb/ft^3

Unit Weight of Sediment: 165.00 lb/ft^3

Results of Live Bed Method

Shear Velocity: 2.33 ft/s

Fall Velocity: 0.08 ft/s

Average Depth in Contracted Section after Scour: 8.91 ft

Scour Depth for Live Bed: 2.41 ft

Scour may be limited by armoring. Compute all methods to check.

Upstream Channel Flow Depth: 8.63 ft

Average Velocity Upstream: 1.93 ft/s

D50: 0.196901 mm

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 1.38 ft/s

Contraction Scour Condition: Live Bed

Input Parameters for Bridge Scour

Width of the Contracted Section: 32.00 ft

Flow through bridge opening: 2410.00 cfs

Width of the Upstream Section: 33.50 ft

Flow in Upstream Section: 2410.00 cfs

Slope of Energy Grade Line at Approach Section: 0.0195 ft/ft

Vertical Size of Bridge Opening Prior to Scour: 6.50 ft

Deck Thickness: 3.00 ft

Result Parameters

K1: 0.69

Diameter of Smallest Non-moving Particle: 0.246126 mm

Average Depth In Contracted Section: 8.91 ft

Flow Separation Thickness: 2.16 ft

Scour Depth: 4.57 ft

Left Abutment Details

Abutment Scour
Computation Type: Froehlich's

Input Parameters

Froehlich's Method
Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.00 Degrees

Centerline Length of Embankment: 788.10 ft

Projected Length of Embankment: 788.10 ft

Length of Active Flow Obstructed by Embankment: 0.68 ft

Flow Obstructed by Abutment and Approach Embankment: 970.64 cfs

Flow Area Obstructed by Embankment: 1414.35 ft^2

Result Parameters

Average Depth of Flow on the Floodplain: 1.79 ft

Average Velocity: 0.69 ft/s

Froude Number: 0.09

Length to Depth Ratio: 439.14

Scour Hole Depth from Froehlich Method: 2.30 ft

Right Abutment Details

Abutment Scour
Computation Type: Froehlich's

Input Parameters

Froehlich's Method
Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.00 Degrees

Centerline Length of Embankment: 255.26 ft

Projected Length of Embankment: 255.26 ft

Length of Active Flow Obstructed by Embankment: 0.76 ft

Flow Obstructed by Abutment and Approach Embankment: 360.94 cfs

Flow Area Obstructed by Embankment: 473.22 ft^2

Result Parameters

Average Depth of Flow on the Floodplain: 1.85 ft

Average Velocity: 0.76 ft/s

Froude Number: 0.10

Length to Depth Ratio: 137.69

Scour Hole Depth from Froehlich Method: 2.43 ft

Scour Summary Table

Long Term Degradation

Parameter	Value	Units	Notes
Pier Name			
Long Term Degradation (LTD)	0.46	ft	Controlled by Equilibrium Slope

Contraction Scour

Parameter	Value	Units	Notes
Applied Contraction Scour Depth	4.57	ft	Pressure Flow
Contraction Scour Depth and Long Term Degradation (LTD)	2.41	ft	Contraction and Long Term Degradation do NOT apply
Live Bed Contraction Scour Depth	2.41	ft	
Pressure Scour Depth	4.57	ft	

Local Scour at Abutments

Parameter	Value	Units	Notes
Abutment scour currently cannot be computed with pressure flow! Use an abutment scour countermeasure with pressure flow.			

Hydraulic Toolbox Report
(Pressure Flow Excluded)

Hydraulic Analysis Report

Project Data

Project Title: LAW 141-1694

Designer: CFB

Project Date: Wednesday, March 1, 2023

Project Units: U.S. Customary Units

Notes: Utilized HEC-RAS V 6.3.1 and project files as provided by ADR & Associates, LTD.

Bridge Scour Analysis:25 YEAR SCOUR

Notes: Excludes pressure scour condition for total scour to evaluate abutment scour.

Scenario: 25 YEAR

Long Term Degradation

Controlled by Equilibrium Slope

Long Term Degradation (LTD) 0.46 ft

Contraction Scour Summary

Contraction & Long Term Scour is applied method due to greater scour.

Applied Contraction Scour Depth 1.02 ft

Contraction & Long Term Scour is applied method due to greater scour.

Pressure Scour Depth 3.09 ft

Clear Water Contraction Scour Depth 25.98 ft

Live Bed Contraction Scour Depth 1.02 ft

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: Froehlich Method

Abutment Scour Depth 0.79 ft

Total Scour at Abutment 0.79 ft

Right Abutment

Abutment Scour Method: Froehlich Method

Abutment Scour Depth 0.89 ft

Total Scour at Abutment 0.89 ft

Long Term Details

Long-Term Degradation

Computation Type: Controlled by Equilibrium Slope

Input Parameters

Slope Equation: No Sediment Supply, Meyer-Peter, Muller

D50: 0.196901 mm

D90: 9.634118 mm

Shield's Parameter: 0.0390

Depth or Hydraulic Radius: 7.29 ft

Manning's n Value: 0.0450

Discharge Per Unit Width: 61.60 cfs/ft

Current Slope: 0.0008 ft/ft

Distance Upstream of Base Level Control: 579.00 ft

Result Parameters

Equilibrium Slope: 0.000003 ft/ft

Ultimate Degradation Amount: 0.46 ft

Main Channel Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 7.29 ft

D50: 0.196901 mm

Average Velocity Upstream: 3.53 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 1.34 ft/s

Contraction Scour Condition: Live-Bed

Live Bed and/or Clear Water Input Parameters

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section: 0.0130 ft/ft

Flow in Contracted Section: 1970.00 cfs

Flow Upstream that is Transporting Sediment: 1970.00 cfs

Width in Contracted Section: 32.00 ft

Width Upstream that is Transporting Sediment: 33.50 ft

Depth Prior to Scour in Contracted Section: 6.50 ft

Unit Weight of Water: 62.40 lb/ft^3

Unit Weight of Sediment: 165.00 lb/ft^3

Results of Clear Water Method

Diameter of the smallest nontransportable particle in the bed material: 0.246126 mm

Average Depth in Contracted Section after Scour: 32.48 ft

Scour Depth: 25.98 ft

Results of Live Bed Method

Shear Velocity: 1.75 ft/s

Fall Velocity: 0.08 ft/s

Average Depth in Contracted Section after Scour: 7.52 ft

Scour Depth for Live Bed: 1.02 ft

Shear Applied to Bed by Live-Bed Scour: 0.3704 lb/ft^2

Shear Required for Movement of D50 Particle: 0.0026 lb/ft^2

Recommendations

Recommended Scour Depth: 1.02 ft

Left Abutment Details

Abutment Scour

Computation Type: Froehlich's

Input Parameters

Froehlich's Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.00 Degrees

Centerline Length of Embankment: 761.00 ft

Projected Length of Embankment: 761.00 ft

Length of Active Flow Obstructed by Embankment: 0.58 ft

Flow Obstructed by Abutment and Approach Embankment: 214.91 cfs

Flow Area Obstructed by Embankment: 367.73 ft^2

Result Parameters

Average Depth of Flow on the Floodplain: 0.48 ft

Average Velocity: 0.58 ft/s

Froude Number: 0.15

Length to Depth Ratio: 1574.85

Scour Hole Depth from Froehlich Method: 0.79 ft

Right Abutment Details

Abutment Scour

Computation Type: Froehlich's

Input Parameters

Froehlich's Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.00 Degrees

Centerline Length of Embankment: 249.49 ft

Projected Length of Embankment: 249.49 ft

Length of Active Flow Obstructed by Embankment: 0.66 ft

Flow Obstructed by Abutment and Approach Embankment: 87.72 cfs

Flow Area Obstructed by Embankment: 133.53 ft^2

Result Parameters

Average Depth of Flow on the Floodplain: 0.54 ft

Average Velocity: 0.66 ft/s

Froude Number: 0.16

Length to Depth Ratio: 466.15

Scour Hole Depth from Froehlich Method: 0.89 ft

Scour Summary Table

Long Term Degradation

Parameter	Value	Units	Notes
Long Term Degradation (LTD)	0.46	ft	Controlled by Equilibrium Slope

Contraction Scour

Parameter	Value	Units	Notes
Applied Contraction Scour Depth	1.02	ft	Contraction & Long Term Scour is applied method due to greater scour.
Clear Water Contraction Scour Depth	25.98	ft	
Live Bed Contraction Scour Depth	1.02	ft	

Local Scour at Piers

Local Scour at Abutments

Parameter	Value	Units	Notes
Left Abutment			
Abutment Scour Depth	0.79	ft	Froehlich Method
Max Flow Depth including Abutment Scour	0.00	ft	
Total Scour at Abutment	2.27	ft	
Right Abutment			
Abutment Scour Depth	0.89	ft	Froehlich Method
Max Flow Depth including Abutment Scour	0.00	ft	
Total Scour at Abutment	2.37	ft	

Bridge Scour Analysis:50 YEAR SCOUR

Notes: Utilized HEC-RAS V 6.3.1 and project files as provided by ADR & Associates, LTD.

Scenario: 50 YEAR

Long Term Degradation

Controlled by Equilibrium Slope

Long Term Degradation (LTD) 0.46 ft

Contraction Scour Summary

Contraction & Long Term Scour is applied method due to greater scour.

Applied Contraction Scour Depth 2.41 ft

Contraction & Long Term Scour is applied method due to greater scour.

Pressure Scour Depth 4.57 ft

Clear Water Contraction Scour Depth 32.10 ft

Live Bed Contraction Scour Depth 2.41 ft

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: Froehlich Method

Abutment Scour Depth 2.30 ft

Total Scour at Abutment 2.30 ft

Right Abutment

Abutment Scour Method: Froehlich Method

Abutment Scour Depth 2.43 ft

Total Scour at Abutment 2.43 ft

Long Term Details

Long-Term Degradation

Computation Type: Controlled by Equilibrium Slope

Input Parameters

Slope Equation: No Sediment Supply, Meyer-Peter, Muller

D50: 0.196901 mm

D90: 9.634118 mm

Shield's Parameter: 0.0390

Depth or Hydraulic Radius: 8.63 ft

Manning's n Value: 0.0450

Discharge Per Unit Width: 75.30 cfs/ft

Current Slope: 0.0008 ft/ft

Distance Upstream of Base Level Control: 579.00 ft

Result Parameters

Equilibrium Slope: 0.000003 ft/ft

Ultimate Degradation Amount: 0.46 ft

Main Channel Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 8.63 ft

D50: 0.196901 mm

Average Velocity Upstream: 1.93 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 1.38 ft/s

Contraction Scour Condition: Live-Bed

Live Bed and/or Clear Water Input Parameters

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section: 0.0195 ft/ft

Flow in Contracted Section: 2410.00 cfs

Flow Upstream that is Transporting Sediment: 2410.00 cfs

Width in Contracted Section: 32.00 ft

Width Upstream that is Transporting Sediment: 33.50 ft

Depth Prior to Scour in Contracted Section: 6.50 ft

Unit Weight of Water: 62.40 lb/ft^3

Unit Weight of Sediment: 165.00 lb/ft^3

Results of Clear Water Method

Diameter of the smallest nontransportable particle in the bed material: 0.246126 mm

Average Depth in Contracted Section after Scour: 38.60 ft

Scour Depth: 32.10 ft

Results of Live Bed Method

Shear Velocity: 2.33 ft/s

Fall Velocity: 0.08 ft/s

Average Depth in Contracted Section after Scour: 8.91 ft

Scour Depth for Live Bed: 2.41 ft

Shear Applied to Bed by Live-Bed Scour: 0.4185 lb/ft^2

Shear Required for Movement of D50 Particle: 0.0026 lb/ft^2

Recommendations

Recommended Scour Depth: 2.41 ft

Left Abutment Details

Abutment Scour

Computation Type: Froehlich's

Input Parameters

Froehlich's Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.00 Degrees

Centerline Length of Embankment: 788.10 ft

Projected Length of Embankment: 788.10 ft

Length of Active Flow Obstructed by Embankment: 0.68 ft

Flow Obstructed by Abutment and Approach Embankment: 970.64 cfs

Flow Area Obstructed by Embankment: 1414.35 ft^2

Result Parameters

Average Depth of Flow on the Floodplain: 1.79 ft

Average Velocity: 0.69 ft/s

Froude Number: 0.09

Length to Depth Ratio: 439.14

Scour Hole Depth from Froehlich Method: 2.30 ft

Right Abutment Details

Abutment Scour

Computation Type: Froehlich's

Input Parameters

Froehlich's Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.00 Degrees

Centerline Length of Embankment: 255.26 ft

Projected Length of Embankment: 255.26 ft

Length of Active Flow Obstructed by Embankment: 0.76 ft

Flow Obstructed by Abutment and Approach Embankment: 360.94 cfs

Flow Area Obstructed by Embankment: 473.22 ft^2

Result Parameters

Average Depth of Flow on the Floodplain: 1.85 ft

Average Velocity: 0.76 ft/s

Froude Number: 0.10

Length to Depth Ratio: 137.69

Scour Hole Depth from Froehlich Method: 2.43 ft

Scour Summary Table

Long Term Degradation

Parameter	Value	Units	Notes
Long Term Degradation (LTD)	0.00	ft	Controlled by Equilibrium Slope

Contraction Scour

Parameter	Value	Units	Notes
Applied Contraction Scour Depth	2.41	ft	Contraction & Long Term Scour is applied method due to greater scour.
Clear Water Contraction Scour Depth	32.10	ft	
Live Bed Contraction Scour Depth	2.41	ft	

Local Scour at Piers

Local Scour at Abutments

Parameter	Value	Units	Notes
Left Abutment			
Abutment Scour Depth	2.30	ft	Froehlich Method
Max Flow Depth including Abutment Scour	0.00	ft	
Total Scour at Abutment	5.17	ft	
Right Abutment			
Abutment Scour Depth	2.43	ft	Froehlich Method
Max Flow Depth including Abutment Scour	0.00	ft	
Total Scour at Abutment	5.30	ft	