Structure Foundation Exploration – Revised Final
CUY-14-12.12E Bridge Replacement
Bedford, Cuyahoga County, Ohio
S&ME Project No. 1117-18-036

PREPARED FOR:

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October 31, 2022

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Attention: Ms. Joan M. Zbin, P.E.

Reference: **Structure Foundation Exploration – Revised Final**
CUY-14-12.12E (PID 13184)
Bedford, Cuyahoga County, Ohio
S&ME Project No. 1117-18-036

Ms. Zbin:

S&ME, Inc. (S&ME) has completed a supplemental structure foundation exploration for the above referenced project in accordance with our MOD 2 proposal dated September 22, 2021, which was authorized by Pennoni Associates, Inc. (Pennoni) on December 14, 2021, with an Amendment to our Consultant Agreement dated October 5, 2018. The supplemental exploration was performed due to a change in the proposed structure and foundation type for the planned replacement of the existing concrete arch bridge carrying Union Street over Tinkers Creek in Bedford, Cuyahoga County, Ohio. The approximate site location is depicted on the Vicinity Map presented as Plate 1 in Appendix I of this report. This revised report has been prepared to address comments received from ODOT which were provided to S&ME by Pennoni on August 17, 2022, and supersedes our original final report dated June 24, 2022.

In accordance with Section 701 of the current ODOT *Specifications for Geotechnical Explorations* (*SGE*), S&ME is herewith submitting a "final" version of this report, which is also to be provided to the ODOT District Geotechnical Engineer. Final Soil Profile – Structure sheets will be submitted under separate cover.

We appreciate being given the opportunity to be of service. Please do not hesitate to contact our office if you have any questions concerning our report.

Sincerely,
S&ME, Inc.

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1.0 Executive Summary

The project consists of replacing the existing Tinkers Creek bridge along Union Street, which was designed by the Ohio Department of Highways Bureau of Bridges in 1910 and was modified in 1934. S&ME understands that the existing arch bridge structure will be replaced with a single-span, prestressed concrete beam bridge with semi-integral abutments. The proposed abutments will be situated slightly behind the existing abutments and are anticipated to be supported on drilled shafts socketed into shale and/or sandstone bedrock. A span length of approximately 117 feet is anticipated. Minimal to no changes to the horizontal and vertical alignments are anticipated with minimal work beyond the bridge. Prior phases of work have been performed by S&ME including obtaining cores of the existing arch structure (S&ME Letter of Transmittal dated 1/10/19) and structure borings for a previously conceived bridge replacement supported on shallow spread foundations (S&ME Structure Foundation Exploration – Draft Report dated 2/25/20).

Multiple phases of drilling have been performed for this project and include four (4) structure borings and one exploratory probe boring. Two structure borings (B-001-0-19 and B-002-0-19) were performed in 2020 to a depth of at least 5 feet below the then proposed shallow spread footing elevations. Following a change in bridge type and foundation support system, three (3) supplemental borings/probes (B-001-1-21, B-001-2-21, and B-002-1-21) were performed to obtain additional rock core information, and to locate the outer longitudinal limits of the existing bridge foundations.

The primary pavement materials encountered consisted of about 3½ to 5½ inches of asphalt, over 8½ to 14 inches of concrete, over 4 to 13½ inches of granular base. In Boring B-002-0-19, 3¼ to 4-inch-thick layers of brick, slag concrete and/or flowable fill were encountered below the concrete layer. Beneath the pavement sections, soils visually described as fill were encountered in borings B-001-0, B-001-1, and B-001-2 to depths between roughly 20 and 26 feet at the rear abutment. The materials consisted of SILT AND CLAY (A-6a), GRAVEL (A-1-a), SANDY SILT (A-4a), and GRAVEL WITH SAND AND SILT (A-2-4). Many sandstone fragments, a few brick fragments and slag were noted in the fill.

Beneath the fill materials in Borings B-001-0, B-001-1, and B-001-2 and the pavement section in Boring B-002-0, natural soils were encountered and consisted of very-dense GRAVEL WITH SAND AND SILT (A-2-4), hard SILT AND CLAY (A-6a) and hard CLAY (A-7-6).

Borings at the rear abutment encountered SANDSTONE at depths of 26 and 27 feet which was underlain by interbedded SHALE with SANDSTONE at a depth of 33 feet and extended to the termination depth. At the forward abutment borings SANDSTONE was encountered at depths of roughly 8.5 to 9.5 feet below the existing road surface and extended to roughly 35 feet in Boring B-002-1 where the interbedded shale and sandstone was encountered to the termination depth.

Based on Section 305.4.1.1 of the ODOT *Bridge Design Manual* (*BDM*), the length of the bedrock socket should be designed to meet the requirements of axial resistance and lateral stability. Recommendations for unit end bearing and unit side resistance of the drilled shafts are presented in Section 6.3.1. Recommended lateral load analyses parameters are provided in Section 6.3.5. Recommended soil parameters for the design of the bridge abutments and wingwalls are presented in Section 6.4.

2.0 Introduction

This project consists of replacing the existing bridge over Tinkers Creek along Union Street. The existing bridge is a 72-foot long, single filled concrete arch bridge, designed by the Ohio Department of Highways Bureau of Bridges in 1910 and was modified in 1934. S&ME previously performed exploration work at the existing bridge site consisting of concrete bridge and pavement cores, the results of which were submitted in a transmittal dated January 10, 2019. An original structure foundation exploration was also performed in 2020, which included two (2) structure borings, laboratory testing, and recommendations for a shallow spread bridge foundation system. The results of this original exploration were submitted in a Draft Structure Foundation Exploration report dated February 25, 2020.

Following the submission of our original draft report, the proposed structure type was changed by others from a twin leaf arch bridge supported on shallow spread footings to the currently proposed 117-foot, single-span, prestressed concrete beam bridge with semi-integral abutments supported on drilled shafts. Due to the change in structure and foundation type, supplemental field exploration and recommendations were required. The combined information obtained from the original and supplemental explorations is included in this revised final report.

S&ME understands the proposed abutments are situated slightly behind the existing abutments. Minimal to no changes to the horizontal and vertical alignments are anticipated with minimal work beyond the bridge.

In addition to supplemental structure borings performed to obtain deeper rock core to design drilled shaft foundations for the proposed structure, one or more probe borings were requested to assist in ascertaining the longitudinal limits of the existing bridge foundations. One (1) probe boring was performed near the rear abutment. No probe boring was required at the forward abutment on account of performing the structure boring at the front side of the proposed drilled shaft foundations without encountering the existing bridge foundation.

For the sake of clarity, the borings performed in 2020 are referred to as the “original borings” and the borings for the current subsurface exploration are referred to as the “supplemental borings.” This geotechnical exploration has been performed in general accordance with the ODOT *Specifications for Geotechnical Explorations* (*SGE),* including July 2019 and July 2021 updates.

3.0 Geology and Observations of the Project

3.1 Site Reconnaissance

S&ME personnel visited the site on January 24, 2020, to observe the site and to mark the original structure foundation boring locations prior to drilling. At that time, significant longitudinal and transverse cracks were observed in the asphalt pavement on either side of the approach slabs and also within the approach slabs and bridge deck. Sandstone bedrock outcrops are visible at the north (forward) abutment and in portions of the creek channel. No significant evidence of scour within the banks of Tinkers Creek was observed and the approach embankments appeared to be in generally good condition, with a few areas of over-steepened slopes.

S&ME personnel visited the site again on January 28, 2022, to mark the supplemental structure foundation boring locations. Union Street had recently been resurfaced; therefore, pavement distress was not observed in the vicinity of the supplemental borings.

3.2 Geology

The project site is within a previously glaciated portion of the state within the Killbuck-Glaciated Pittsburgh Plateau physiographic region. This region is characterized by clay to loam till of the Wisconsinan-age underlain by Mississippian and Pennsylvanian-age shales, sandstones and conglomerates. According to the Cuyahoga County Soil Survey as performed by the United States Department of Agriculture (USDA), the soils at the bridge are primarily composed of Brecksville Silt Loam (BrF) which is derived from weathered shale.

Bedrock topography mapping suggests that rock may be present at relatively shallow depths. An approximate top of rock surface is shown on some of the available plans from the 1934 project and is anticipated approximately 10 to 30 feet below the existing roadway surface. Bedrock has also been observed during our previous site visits for the coring project. In the original borings previously completed for this project, bedrock was encountered at depths of 27 and 9.5 feet in the rear and forward abutments, respectively. Similar conditions were encountered in the supplemental boring with bedrock noted at depths of 26 and 8.5 feet in the rear and forward abutments, respectively.

A review of the ODNR "Ohio Karst Areas" map reveals that the site lies in an area not known to contain karst features. A review of the ODNR "Landslides in Ohio" map reveals that the project site lies in an area of low incidence and low susceptibility to landslides, and the ODNR "Abandoned Underground Mines of Ohio" map indicates the site lies in areas with no mapped abandoned mines near the area of the project site.

3.3 Available Information

The existing structure was designed by the Ohio Department of Highways Bureau of Bridges in 1910, modified in 1934, and is a 72-foot long, single filled arch. We understand that the original construction was in 1910 with a widening in 1934. The on-line ODOT Transportation Information Mapping System (TIMS) records was searched for historic boring information with no available historic boring records being located for the site.

As referenced above, S&ME previously performed work at the existing bridge site consisting of concrete cores, structure foundation borings for shallow spread foundations, and associated laboratory testing. The transmittal for the bridge and pavement cores was submitted under separate cover and dated January 10, 2019. The original structure borings, laboratory testing, and recommendations for the support of a replacement structure on shallow spread foundations were submitted in the draft Structure Foundation Exploration report dated February 25, 2020.

4.0 Field Exploration

The boring locations were selected by S&ME in accordance with the ODOT *SGE* and discussions with Pennoni. Two (2) original borings, B-001-0-19 and B-002-0-19, were field marked by S&ME personnel on January 24, 2020, and were positioned behind the south and north abutments, respectively. Supplemental Borings B-001-1-21, B-001-2-21, and B-002-1-21 were field marked by S&ME personnel on January 28, 2022. The approximate locations of the borings are shown on the Plan of Borings submitted as Plate 2 in Appendix I. The boring locations were obtained using a handheld GPS unit (with sub-meter accuracy) and ground surface elevations at the marked locations were estimated from available topographic and plan drawing information. All boring locations will be referred to hereafter without their two-digit year extension.

The original structure borings (B-001-0 and B-002-0) were performed with a truck-mounted drill rig on February 4, 2020. The supplemental structure borings (B-001-1 and B-002-2) were performed with a truck-mounted drill rig between February 7 to 9, 2022. All borings were advanced using 3-1/4" I.D. hollow-stem augers to the top of bedrock. At regular intervals in the original borings, disturbed (but representative) soil samples were obtained by lowering a 2-inch O.D. split-barrel sampler to the sampling depth where it was driven 18 inches into the soil strata by blows from a calibrated 140-pound hammer freely falling 30 inches (AASHTO T206, Standard Penetration Test – SPT). Split-barrel samples were examined immediately after recovery and representative samples were preserved in airtight containers. No SPT samples were attempted within the soil strata in the supplemental borings.

In accordance with ODOT specifications, the hammer system on the drilling rig was calibrated (ASTM D4633) to determine the drill rod energy ratio. The same truck rig was used for both phases of drilling, but was recalibrated in between the two phases of work. For the borings performed in February 2020, the hammer with a drill rod energy ratio of 93.8% was calibrated on November 5, 2018. The hammer system was recalibrated on November 25, 2020, prior to the second phase of drilling, with a drill rod energy ratio of 98.6%. The drill rod energy ratio for both calibrations has been limited to 90% in accordance with the ODOT *SGE*.

Rock coring was initiated at depths ranging from 26.4 to 28.5 feet at the rear (south) abutment and at depths of 9.9 to 10 feet at the forward (north) abutment. The original borings were extended to the termination depths of 38.5 feet in B-001-0 and 35 feet in B-002-0. The supplemental structure borings were extended to termination depths of 45 feet in B-001-1 and 44 feet in B-002-1.

A probe boring, B-001-2, to assist in locating the existing concrete foundation for the rear abutment, was completed on February 21, 2022. Auger refusal was encountered at a depth of 20.5 feet. A split-barrel sampler was lowered to the bottom of the boring and driven approximately 3 inches. Concrete fragments, presumed to be the existing bridge foundation, were observed in the sampler. The probe boring was then terminated.

Groundwater observations were made as the borings were being advanced and prior to coring. After drilling, the borings were sealed with a cement and bentonite slurry and/or with soil cuttings mixed with portland cement. At all boring locations in the roadway, the existing pavement surface was repaired with an equivalent thickness of cold patch asphalt. In Boring B-002-1, the rock core bit and associated rods had to be abandoned in place from about 0.8 to 32 feet below the pavement surface due to the rock core barrel becoming lodged into the sandstone formation while attempting to remove the tooling.

In the field, experienced personnel performed the following specific duties: preserved all recovered samples; prepared a log for the borings; made seepage and groundwater observations; obtained hand-penetrometer measurements in soil samples exhibiting cohesion; and coordinated with S&ME personnel so that the program of explorations could be modified, if necessary, because of unanticipated conditions. All samples were transported to the laboratory of S&ME for further identification and testing.

4.1 Laboratory Testing

In the laboratory, under the direction of a Professional Engineer, all soil samples from the original borings were visually identified and tested for moisture content. Select samples were tested for liquid and plastic (Atterberg) limits, and particle-size distribution. The rock cores were visually identified, photographed, and percent recoveries and RQDs were estimated. Five (5) rock core samples, including one (1) interbedded shale/sandstone sample and four (4) sandstone samples were tested for unconfined compressive strength in accordance with ASTM D 7012, Method C. Results of the laboratory tests are included on the boring logs presented in Appendix I.

Based on the results of the laboratory identification and testing program, soil and rock descriptions on the field logs were modified, where necessary, and copies of the laboratory-corrected logs of the borings have been submitted as Plates 5 through 13 in Appendix I. Shown on these logs are: descriptions of the soil and rock stratigraphy encountered; depths from which samples were attempted and preserved; sampling efforts (blow-counts) required to obtain the samples in the borings; seepage and groundwater observations; and percent recovery and RQD measurements.

Soils samples described in this report have been classified in general accordance with Section 603 of the July 2021 ODOT *SGE*. The soil and rock samples have been described in general accordance with Sections 602 and 605 of the January 2021 ODOT *SGE*, including the use of adjectives to designate the approximate percentages of minor soil components. An explanation of the symbols and terms used on the boring logs and definitions of the special adjectives used to denote the minor soil components are presented on Plate 3 of Appendix I. An explanation of the symbols and terms used on the boring logs to describe the bedrock are presented on Plate 4 of Appendix I.

5.0 Exploration Findings

5.1 Existing Pavement Thicknesses

The various pavement materials encountered in the borings ranged from 13 to 23 inches over 4 to 13½ inches of granular base below the pavement. The existing pavement section thicknesses measured from the borings and recovered pavement cores are summarized in Table 5-1. Photos of the pavement cores obtained at the original boring locations are presented on Plate 28 of Appendix I.

Table 5-1 Summary of Pavement Material Thicknesses

Boring/ Core ID	Approx. Location	Asphalt (in.)	Concrete (in.)	Flowable Fill (in.)	Slag Concrete (in.)	Slag Concrete/ Brick (in.)	Granular Base (in.)	Total (in.)
B-001-0-19	Rear Abutment	3 ¾	9 ¼	--	--	--	4	17
B-001-1-21	Rear Abutment	4 ½	8 ½	--	--	--	12	25
B-001-2-21	Rear Abutment	5 ½	14	--	--	--	12	31 ½
B-002-0-19	Forward Abutment	3 ½	8 ½	3 ¾	3 ¼	4	4	27
B-002-1-21	Forward Abutment	4 ½	12	--	--	--	13 ½	30

5.2 General Subsurface Conditions

Beneath the pavement sections listed above in Table 5-1, soils visually described as fill were encountered in all borings except B-002-0 to depths between roughly 20 and 26 feet at the rear abutment. The materials consisted of SILT AND CLAY (A-6a), GRAVEL (A-1-a), SANDY SILT (A-4a), and GRAVEL WITH SAND AND SILT (A-2-4). Many sandstone fragments, a few brick fragments and slag were noted in the fill.

Beneath the fill materials in Borings B-001-0, B-001-1, and B-001-2, and the pavement section in Boring B-002-0, natural soils were encountered and consisted of very-dense GRAVEL WITH SAND AND SILT (A-2-4), hard SILT AND CLAY (A-6a) and hard CLAY (A-7-6).

Borings at the rear abutment encountered SANDSTONE at depths of 26 and 27 feet which was underlain by interbedded SHALE with SANDSTONE at a depth of 33 feet and extended to the termination depth. At the forward abutment borings SANDSTONE was encountered at depths of roughly 8.5 to 9.5 feet below the existing road surface and extended to roughly 35 feet in Boring B-002-1 where the interbedded shale and sandstone was encountered to the termination depth of 44 feet. Photos of the rock cores are presented on Plates 23 and 27 of Appendix I.

5.3 Seepage and Groundwater Observations

Groundwater observations were made as the borings were being advanced, and again before commencing rock coring. Groundwater was observed at 20 feet below the pavement surface at B-001-1 after drilling was complete on February 7, 2022, and at about 26 feet from the pavement surface before rock coring on February 8, 2022. Groundwater was not encountered during drilling or before rock coring at any other boring location. Note that water was added to the boreholes during the rock coring process in each of the structure borings; therefore, after drilling water measurements are not considered to reflect long-term or static water levels.

All groundwater levels and seepage measurements should be considered as temporary, short-term observations and should not be assumed to be representative of the long-term static groundwater level. Groundwater levels can fluctuate due to seasonal variations in precipitation, construction activities, etc.

5.4 Bedrock Testing Results

Table 5-2 summarizes the results of the unconfined compressive strength (UCS) tests performed on recovered sections of the bedrock core.

Table 5-2 Unconfined Compression Test Results on Bedrock

Boring No.	Core Run No.	Top Depth of Core Run	Bottom Depth of Core Run	Test Sample Depth	Rock Type	Unconfined Compressive Strength (psi)
B-001-1-21	NQ-3	27.1	31.4	29.7 – 30.1	Sandstone	7,247
B-002-1-21	NQ-3	13.9	23.9	17.1 – 17.5	Sandstone	5,208
B-002-1-21	NQ-5	34.0	44.0	39.1 – 39.4	Shale in/b w/ Sandstone	2,133
B-002-0-19	NQ-8	25.0	35.0	29.7 – 30.5	Sandstone	5,533
B-002-0-19	NQ-8	25.0	35.0	32.5 – 33.3	Sandstone	7,891

6.0 Analyses and Recommendations

6.1 General Project Discussion

Based on information provided by Pennoni, S&ME understands that the project consists of replacing the existing 72-foot long, single filled concrete arch bridge carrying Union Street over Tinkers Creek with a replacement structure that is anticipated to be a 117-foot, single-span bridge with prestressed concrete I-Beams supported on semi-integral abutments with turnback wingwalls. Based on discussions with Pennoni and ODOT District 12, S&ME understands that the decision has been made to support the proposed abutments on drilled shafts socketed into the shale and/or sandstone bedrock. Pennoni is currently considering two options for the drilled shafts:

- Five (5) shafts per abutment, spaced at 9 feet, 7 inches at the rear abutment and 10 feet, 3¾ inches at the forward abutment.
- Four (4) shafts per abutment, spaced at 12 feet, 9 inches at the rear abutment and 13 feet, 9 inches at the forward abutment.

Pennoni's preliminary calculations indicate that the anticipated max factored service reactions are approximately 550 kips per shaft and 650 kips per shaft for the five and four shaft options, respectively. Minimal to no changes to the horizontal and vertical alignments are anticipated with minimal work beyond the bridge.

6.2 Scour

Grain-size analyses were performed on soil samples recovered from the continuously sampled scour zone between 18.5 and 25.6 feet below the existing ground surface in Boring B-001-0. A summary of the results is provided in Table 6-1. Scour design is to be performed by others. In addition to the particle size information provided in Table 6-1, the sandstone bedrock near the channel flow elevation is considered to be medium grained, which according to the ODOT *SGE*, corresponds to a particle size from 0.25 to 0.5 mm.

Table 6-1 Summary of Grain-Size Data for Scour Analysis

Boring ID	Sample ID	Depth Interval (ft)	D_{50} (mm)	D_{95} (mm)
B-001-0-19	SS-4	18.5′ – 20.0′	0.301	24.267
	SS-5	20.0′ – 21.5′	0.046	0.367
	SS-6	21.5′ – 23.0′	0.036	0.262
	SS-7	23.0′ – 24.5′	0.043	0.906
	SS-8	24.5′ – 25.6′	0.283	14.863

Stage 1 plans indicate the existing structure is planned to be cut off and concrete slope protection is to be drilled and grouted into the portion of existing abutment to remain. However, if riprap is elected to be used to protect against scour, it should be recognized that riprap is not a permanent countermeasure against, nor does it completely eliminate the potential for scour. For this reason, if riprap is used, we recommended that the project plans and specifications also contain provisions for routine maintenance of the rip-rap blanket to ensure that the design blanket thickness is preserved over the design life of the bridge. Additionally, in all cases where riprap is used for scour protection, the bridge must be monitored during and inspected after periods of high flow.

Section 305.2.1.2.b of the January 2020 ODOT *Bridge Design Manual* (*BDM*) defines several criteria that bedrock must meet or exceed to be considered as resistant to scour. These requirements are presented in Table 6-2.

Table 6-2 Bedrock Scour Criteria

ODOT *BDM* Requirement for Scour Resistant Bedrock
Unconfined compressive strength $\geq$ 2,500 psi
Slake Durability Index (SDI) $\geq$ 90%
Rock Quality Designation (RQD) $\geq$ 65%
Unit weight $\geq$ 150 pcf
Rock Mass Rating (RMR) $\geq$ 75 or Geologic Strength Index (GSI) $\geq$ 75
Erodibility Index (K) $\geq$ 100

Test results indicate that the sandstone and shale do not meet all the requirements in Table 6-2. However, we understand that Pennoni has performed a rock scour analysis based on HEC-18 which indicated that no scour is anticipated to occur in the sandstone bedrock at the creek bed level.

6.3 Bridge Foundation Recommendations

6.3.1 Drilled Shafts – Axial Load Resistance

Recommended unit end bearing and side resistance values have been calculated for both the sandstone and the underlying layer of shale interbedded with sandstone, as drilled shaft bearing elevations have not yet been determined. Drilled shafts should be designed in accordance with Section 305.4 of the 2020 ODOT *BDM*, with shaft and rock socket diameters determined in accordance with Section 305.4.4.2.

As the amount of movement necessary to develop shaft friction resistance is less than that needed to develop end bearing (tip) resistance, unless an on-site static load test is planned at this site, drilled shafts used to support the proposed abutments and piers should be designed for axial load carrying capacity using either shaft friction resistance only or end bearing (tip) resistance only.

LRFD resistance values for use during drilled shaft design of substructure elements at the bridge abutments are presented in Table 6-3 (End Bearing Tip Resistance) and Table 6-4 (Unit Side Resistance).

Table 6-3 Recommended Nominal and Factored Unit End Bearing Resistance Values for Drilled Shafts Socketed into Bedrock (Strength Limit State)

Substructure Element	Rock Type	Elevation Range	Nominal Unit Tip Resistance* (q_p)	Resistance Factor (φ_{qp}) for Tip Resistance**	Factored Unit Tip Resistance*
Rear Abutment Drilled Shafts	Scour Resistant Sandstone	870.0 – 862.7	2,325 ksf	0.5	1162.5 ksf
	Interbedded Shale and Sandstone	862.7 – 850.5	765 ksf	0.5	382.5 ksf
Forward Abutment Drilled Shafts	Scour Resistant Sandstone	886.1 – 876.5	1,875 ksf	0.5	937.5 ksf
	Scour Resistant Sandstone	876.5 – 869.3	2,100 ksf	0.5	1050.0 ksf
	Scour Resistant Sandstone	869.3 – 860.6	2,325 ksf	0.5	1162.5 ksf
	Interbedded Shale and Sandstone	860.6 – 852.0	760 ksf	0.5	380.0 ksf

* For vertical loading only, calculated using AASHTO LRFD Eqn. 10.8.3.5.4c-1.
** Table 10.5.5.2.4-1 of the AASHTO LRFD (tip resistance in rock).

Table 6-4 Recommended Nominal and Factored Unit Side Resistance Values for Drilled Shafts Socketed into Bedrock (Strength Limit State)

Substructure Element	Rock Type	Elevation Range	Nominal (Unfactored) Unit Shaft Resistance (q_s)*	Resistance Factor (φ_{qs}) for Shaft Resistance**	Factored Unit Shaft Resistance*
Rear Abutment Drilled Shafts	Scour Resistant Sandstone	870.0 – 862.7	34.9 ksf	0.55	19.1 ksf
	Interbedded Shale and Sandstone	862.7 – 850.5	25.5 ksf	0.55	14.0 ksf
Forward Abutment Drilled Shafts	Scour Resistant Sandstone	886.1 – 876.5	34.9 ksf	0.55	19.1 ksf
	Scour Resistant Sandstone	876.5 – 869.3	34.9 ksf	0.55	19.1 ksf
	Scour Resistant Sandstone	869.3 – 860.6	34.9 ksf	0.55	19.1 ksf
	Interbedded Shale and Sandstone	860.6 – 852.0	25.5 ksf	0.55	14.0 ksf

* For vertical loading only, calculated using AASHTO LRFD Eqn. 10.8.3.5.4b-1.
** Table 10.5.5.2.4-1 of the AASHTO LRFD (side resistance in rock)

A calculation package with geologic strength index (GSI) determinations and estimates of nominal bedrock bearing resistance is included in Appendix II.

During a conference call with Pennoni, ODOT District 12, ODOT OGE and S&ME on August 29, 2022, ODOT OGE indicated that since no bedrock scour is expected based on the scour analysis performed by Pennoni that the drilled shafts at the forward abutment are not required to terminate below the stream bed in Tinkers Creek. Accordingly, the length for drilled shafts should be determined following an assessment of the axial and lateral loading at the forward abutment with the expectation that the shafts will bear within the scour resistant sandstone bedrock above El. 860.6.

In accordance with the *BDM* Section 305.4.2, the plans should not specify the tip elevation of drilled shafts socketed into bedrock, but should provide the approximate top of the bedrock elevation and the length of the bedrock socket in the profile view on the Final Structure Site Plan. The minimum length of rock socket required will be the greater of 1.5D below the anticipated scour depth (i.e., creek bed elevation since no scour is anticipated) if the axial load is supported using end-bearing resistance only, the socket length required to support the axial loads using shaft friction only (negating friction in the top 2 feet of the rock socket), or the socket length determined based on results of a lateral load analyses.

6.3.2 Drilled Shaft Construction Recommendations

The drilled shafts should be constructed in accordance with Item 524 of the ODOT *Construction and Materials Specifications* (*CMS*) and should be at a minimum center-to-center spacing of 3 shaft diameters (3D). Provisions should be made for providing a temporary casing during drilled shaft excavation through the soil overburden above the bedrock. Each of the borings encountered uncontrolled fill above bedrock, which may cave during drilled shaft construction. The casing should extend to the underlying bedrock to attempt to seal the shafts from influx of water, soil, and rock fragments. The temporary casing may then be removed during concrete placement; however, precautions should be taken to ensure that the structural integrity of the shafts is not compromised by caving of material during removal of the casing. The concrete level (head) should also be maintained a minimum of 4 feet above the bottom of the casing during withdrawal to prevent the entry of soil/rock and water into the shafts. The need for continual pumping should be anticipated to remove water accumulation (seepage) from the drilled shafts as seepage may occur; otherwise, placement of concrete should use approved tremie or pumping methods. All drilled shaft construction should be observed by a qualified geotechnical engineer or an experienced technician working under direction of the engineer to ensure that the drilled shafts are installed plumb, that the shaft bottoms are sufficiently clean and dry prior to concrete placement, and that the shafts extend into the appropriate bearing stratum as recommended.

In addition, S&ME also suggests that the following items be considered:

- Determination/Verification of Bearing Surface: Verification of the bearing surface will be required. Ideally, the bedrock socket and bottom surface should be directly observed by a trained inspector. To facilitate this, the contract plans should indicate that the contractor should attempt to dewater the shafts following drilling. However, if it is impossible to fully dewater the shafts, determination of the bearing surface will have to be made based on the type of material extracted from the hole and the degree of drilling difficulty.
- Bottom Clean-Out: Whether the shafts are designed to resist axial loads in end-bearing, side-friction, or a combination of both, bottom clean out is important. In general, the specifications contained in Item 524 of the ODOT *CMS* and *Construction Administration Manual of Procedures* (*MOP*) are acceptable. Verification of the clean-out may be performed by visual inspection if the excavations are dry or by using a submersible electronic inspection device (MiniSID) if the excavations are wet.
- Steel Reinforcement: If it is intended to fully reinforce the shafts, provisions will need to be made to permit either lengthening or shortening the reinforcing cages on site as required to reach the shaft bottom.
- Concrete Integrity: If the shafts are constructed in the dry, the potential for the inclusion of voids or pockets of deleterious material within the shafts is minimized.

6.3.3 Foundation Settlement

Abutment foundation settlement is anticipated to be limited to the elastic compression of the drilled shafts, provided that the shafts bear in intact sandstone or interbedded shale/sandstone layers.

6.3.4 *Downdrag Considerations*

As no additional fill is planned in the abutments above the drilled shafts, no downdrag loads are anticipated on the drilled shaft foundations.

6.3.5 *LPILE Lateral Resistance Analysis Design Parameters*

Once the final structure configuration and loads have been determined, a lateral load analysis should be performed to determine the minimum required rock socket length at both abutments to provide lateral stability per ODOT *BDM* Section 305.4.1.1. Provided that the existing soil overburden is completely removed above the top of rock elevation during the design flood, Table 6-5 includes recommended p-y models, rock unit weights, strain, and rock strength parameters to be used in the lateral load analyses for drilled shaft foundations. Effective unit weights shown for bedrock reflect a water table being encountered above the point where bedrock becomes scour resistant. These parameters are based on the complete removal of the soil overburden due to scour along with bedrock scour, and recommended values given in the LPile 2019 user's manual. These parameters may be used for drilled shaft foundations at both abutments below the depth of potential bedrock scour.

Table 6-5 LPile 2019 Input Parameters for Drilled Shafts

Stratum	Rear Abutment Elevation Range	Forward Abutment Elevation Range	p-y Rock Model	Effective Unit Weight	Unconfined Compressive Strength
Sandstone Above OHWM	--	886.1 – 874.3	Strong Rock	130 pcf	5,000 psi
Sandstone Below OHWM	870.0 – 862.7	874.3 – 860.6	Strong Rock	64 pcf	5,000 psi
Interbedded Shale and Sandstone	862.7 – 850.5	860.6 – 852.0	Strong Rock	100 pcf	2,000 psi

The designer should verify that the shaft-head deflection amounts and internal forces (shear and moment) are accounted for and are acceptable for the design of the structures.

6.4 Lateral Earth Pressures

6.4.1 *Wingwalls and Abutment Walls*

The proposed bridge abutments and any wingwalls must be designed to withstand lateral earth pressures, as well as hydrostatic pressures, that may develop behind the structures. The magnitude of the lateral earth pressures varies on the basis of soil type, permissible wall movement, and the configuration of the backfill.

To minimize lateral earth pressures, the zone behind abutment walls should be backfilled with granular soil, and the backfill should be effectively drained. For effective drainage, a zone of free-draining gravel (ODOT Item

518.03) should be used directly behind the structures for a minimum thickness of 24 inches in accordance with ODOT Item 518.05. This granular zone should drain to either weepholes or a pipe, so that hydrostatic pressures do not develop against the walls.

The type of backfill beyond the free-draining granular zone, however, will govern the magnitude of the pressure to be used for structural design. Pressures of a relatively low magnitude will be developed by the use of granular backfill, whereas a cohesive (clay) backfill will result in the development of much higher pressures.

It is recommended that granular backfill be used behind the abutments and any wingwalls. The backfill should be placed in a wedge formed by the back of the structure and a line rising from the base of the wall abutment foundations at an angle no greater than 60 degrees from horizontal. Granular backfill behind the structures should be compacted in accordance with ODOT *CMS* Item 203, "Embankment Compaction". Over-compaction in areas directly behind the walls should be avoided, as this might cause damage to the structure.

If proper drainage is provided and compacted granular backfill is provided as described above, an equivalent fluid unit weight of 35 lb/ft^3 (pcf) may be used if movement equivalent to 0.25 percent of the height of the abutment or wingwall (H) is allowed to occur. Such movement is considered sufficient to mobilize an active earth pressure condition, and the resultant lateral force should be taken as acting at 0.33H. If this movement is not anticipated or cannot occur, it is recommended that an “at-rest” equivalent fluid unit weight of 55 pcf be used.

Compacted cohesive materials tend alternatively to shrink, expand and creep over periods of time and create significant lateral pressures on any adjacent structures. Cohesive materials also require a greater amount of movement to mobilize an active earth pressure condition. For these reasons, if proper drainage (ODOT Item 518) is provided and a wall movement in excess of 1.0 percent of the height of the abutment or wingwall (H) is allowed to occur, an equivalent fluid unit weight of 65 pcf may be used for design of the abutment walls to resist the lateral loads imparted by drained cohesive backfill. If this amount of movement is not anticipated or cannot occur, it is recommended that an “at-rest” equivalent fluid unit weight of 95 pcf be used.

The structures must also be designed to withstand the surcharge effect of traffic in addition to the vertical load resulting from the weight of any fill and pavement to be placed over the structures. To estimate vertical loading, a total unit weight of 125 pcf and 135 pcf may be used for compacted granular and cohesive soil, respectively.

6.4.2 Soil Parameters

The following parameters in Table 6-6 may be used for determination of earth pressures from soils above bedrock acting on the proposed abutments where existing materials are to remain in place. Soils encountered above the bedrock in Boring B-001-0 are described as existing fill. These parameters only apply where new granular backfill cannot be placed in a wedge 60 degrees from the horizontal behind the abutments.

Table 6-6 Lateral Earth Pressures due to Soil

Boring ID	Elevation Range	Angle of Internal Friction (deg.)	At-Rest Coefficient (K_o)	Active Coefficient (K_a)	Passive Coefficient (K_p)	Recommended Unit Weight (pcf)
B-001-0-19 Rear Abutment	894.6 - 887.1	28	0.53	0.36	2.77	120
	887.1 - 884.0	32	0.47	0.31	3.25	115
	884.0 - 878.0	28	0.53	0.36	2.77	120
	878.0 - 876.0	30	0.50	0.33	3.00	110
	876.0 - 871.5	28	0.53	0.36	2.77	110
	871.5 - 869.0	36	0.41	0.26	3.85	135
B-002-0-19 Forward Abutment	894.6 - 889.0	28	0.53	0.36	2.77	120
	889.0 - 887.5	28	0.53	0.36	2.77	120

6.5 Temporary Excavation Considerations

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P". This document was issued to better ensure the safety of workers entering trenches or excavations. It is mandated by this federal regulation that excavations be constructed in accordance with the OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely 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. The contractor's "responsible person", as defined in 29 CFR, Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. If an excavation, including a trench is extended to a depth of more than twenty (20) feet, it will be necessary to have the side slopes designed by a professional engineer registered in the state where the construction is occurring.

We are providing this information solely as a service to our client. S&ME does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations.

6.6 Groundwater Considerations

S&ME is of the opinion that the long-term groundwater level at this site will be approximately the same as, and vary with, the level of water in Tinkers Creek. Some water seepage may emanate from granular seams or zones encountered above the level of water in the creek; however, the quantity of water is expected to be limited and may potentially be controlled by bailing or using portable pumps. Provisions for continuous pumping from sumps should be made for the larger groundwater flows that may be encountered in excavations extending below the level of water in the creek.

It is recommended that groundwater and surface water runoff be controlled during construction, as soil in excavations may exhibit instability in the presence of water and construction vibrations. S&ME recommends that the sides and bottoms of all excavations be closely monitored by the Geotechnical Engineer of Record or their designated representative during construction. If the soils at the bottom of an excavation become disturbed by construction activity or channel flow, it is recommended that the disturbed material be undercut and replaced in accordance with the recommendations provided in this report, or be removed and the footing elevation lowered to more suitable soils.

Localized sheeting and continuous dewatering, in conjunction with stream diversion, may aid in minimizing disturbance of the soil/rock at the foundation bearing elevation, and it is recommended that all excavations for the proposed structure foundations be protected from stream, groundwater, and storm water flow. Even with stream flow diversion, provisions for continuous pumping from sumps should be made for the expected larger groundwater flows that may be encountered in excavations extending below the level of water in the stream and into the underlying granular soil.

7.0 Final Considerations and Report Limitations

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations contained in this report are based upon applicable standards of our practice in this geographic area at the time this report was prepared. No other representation or warranty either express or implied, is made.

We relied on project information given to us to develop our conclusions and recommendations. If project information described in this report is not accurate, or if it changes during project development, we should be notified of the changes so we can modify our recommendations based on this additional information if necessary.

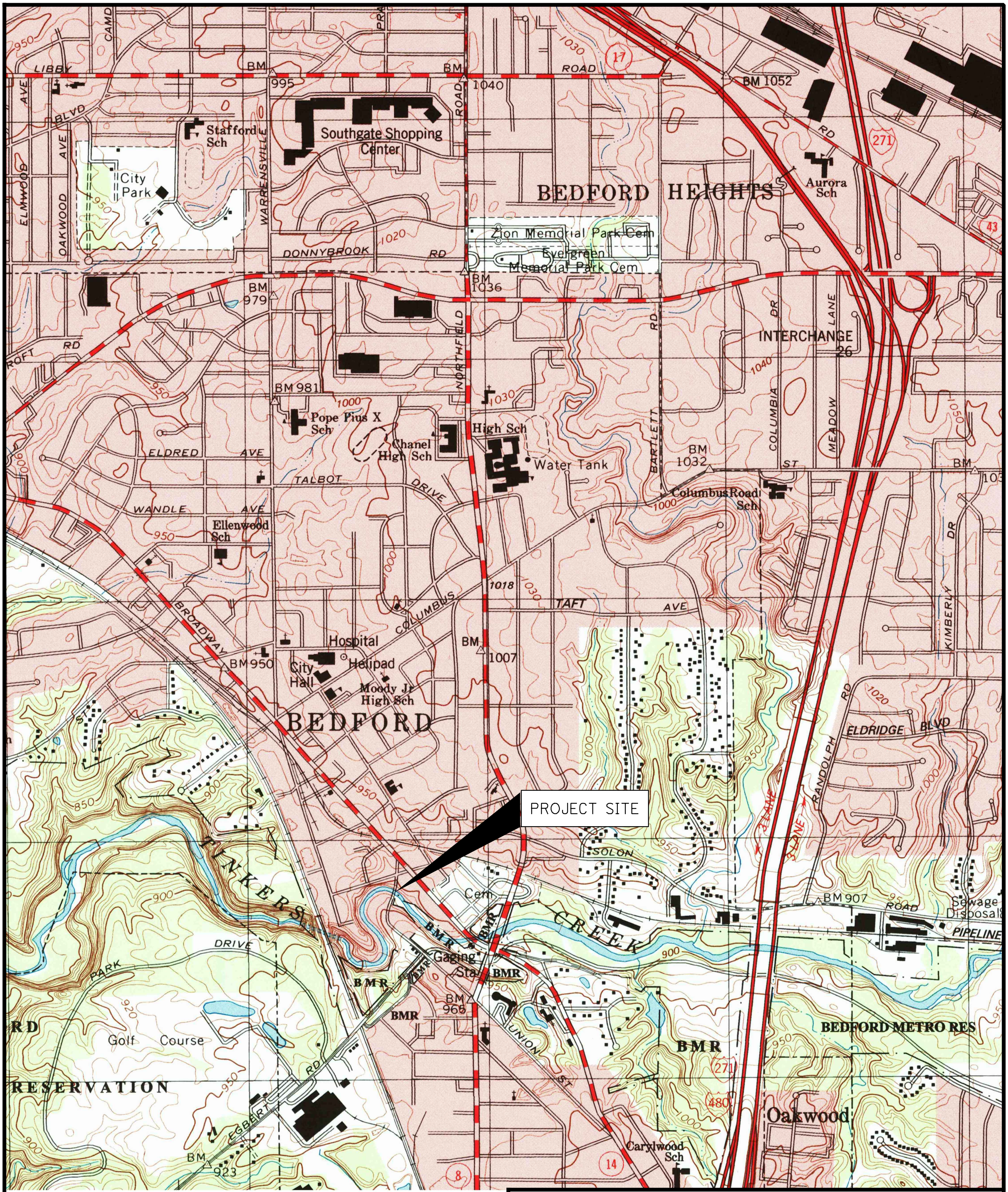
Our conclusions and recommendations are based on limited data from a field exploration program. Subsurface conditions can vary widely between explored areas. Some variations may not become evident until construction. If conditions are encountered which appear different than those described in our report, we should be notified. This report should not be construed to represent subsurface conditions for the entire site.

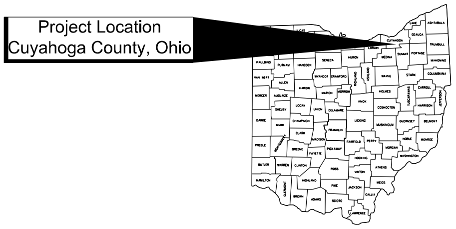
Unless specifically noted otherwise, our field exploration program did not include an assessment of regulatory compliance, environmental conditions or pollutants or presence of any biological materials (mold, fungi, bacteria). If there is a concern about these items, other studies should be performed. S&ME can provide a proposal and perform these services if requested.

S&ME should be retained to review the final plans and specifications to confirm that earthwork, foundation, and other recommendations are properly interpreted and implemented. The recommendations in this report are contingent on S&ME's review of final plans and specifications followed by our observation and monitoring of earthwork and foundation construction activities.

Appendices

Appendix I – Vicinity Map, Boring Logs, Laboratory Testing, and Rock Core Photos





0 2000 4000

SCALE IN FEET

Vicinity Map

CUY-14-12.12E Bridge Replacement
Bedford, Cuyahoga County, Ohio

Project: 1117-18-036
Drawing Date: 2/24/2020
Revision Date: 2/24/2020
Drawn By: KAH
Approved By: BKS
Scale: GRAPHIC

WWW.SMEINC.COM

ENGINEERING LICENSE NO: F-0176

PLATE 1

NOTE: AERIAL IMAGERY COURTESY
OF OGRIP, OBTAINED SPRING 2017.

LEGEND

● B-001-0-21 BORING NUMBER
AND LOCATION

0 40 80
GRAPHIC SCALE (IN FEET)

Plan of Borings

Structure Foundation Exploration
CUY-14-12.12E Bridge Replacement
Bedford, Cuyahoga County, Ohio

SCALE:	GRAPHIC
DATE:	03-23-2022
PROJECT NUMBER	1117-18-036
FIGURE NO.	2

Drawing Path: T:\GEO\Projects\2018\1117-18-036 CUY-14-12.12 Cores-GPR\CAD\Construction\Plan of Borings.dwg

EXPLANATION OF SYMBOLS AND TERMS USED ON BORING LOGS FOR SAMPLING AND DESCRIPTION OF SOIL

SAMPLING DATA

▮ - Indicates sample was attempted within this depth interval.

2 3 5 - The number of blows required for each 6-inch increment of penetration of a “Standard” 2-inch O.D. split-barrel sampler, driven a distance of 18 inches by a 140-pound hammer freely falling 30 inches (SPT). The raw “blowcount” or “N” is equal to the sum of the second and third 6-inch increments of penetration.

N_{60} - Corrected Blowcount = [(Drill Rod Energy Ratio) / (0.60 Standard)] X N

90* - Calibrated energy ratio exceeds 90% but is limited to 90% per ODOT SGE.

SS - Split-barrel sampler, any size.

ST - Shelby tube sampler, 3″ O.D., hydraulically pushed.

R - Refusal of sampler in very-hard or dense soil, or on a resistant surface.

50-4” - Number of blows (50) to drive a split-barrel sampler a certain distance (4 inches), other than the normal 6-inch increment.

DEPTH DATA

W - Depth of water or seepage encountered during drilling.

▽ - Depth to water in boring at the end of drilling (EOD).

▼ 5 days - Depth to water in monitoring well or piezometer in boring a certain number of days (5) after termination of drilling.

TR - Depth to top of rock.

SOIL DESCRIPTIONS

Soils have been classified in general accordance with Section 603 of the most recent ODOT SGE, and described in general accordance with Section 602, including the use of special adjectives to designate approximate percentages of minor components as follows:

Adjective	Percent by Weight
trace	1 to 10
little	10 to 20
some	20 to 35
“and”	35 to 50

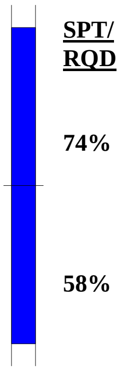
The following terms are used to describe density and consistency of soils:

Term (Granular Soils)	Blows per foot (N_{60})
Very-loose	Less than 5
Loose	5 to 10
Medium-dense	11 to 30
Dense	31 to 50
Very-dense	Over 50

Term (Cohesive Soils)	Qu (tsf)
Very-soft	Less than 0.25
Soft	0.25 to 0.5
Medium-stiff	0.5 to 1.0
Stiff	1.0 to 2.0
Very-stiff	2.0 to 4.0
Hard	Over 4.0

EXPLANATION OF SYMBOLS AND TERMS USED ON BORING LOGS FOR SAMPLING AND DESCRIPTION OF ROCK

SAMPLING DATA



When bedrock is encountered and rock core samples are attempted, the length of core recovered and lost during the core run is reported in the "REC" column. The type of rock core barrel utilized is recorded under the heading "Sampling Method" at the top of the boring log, and also in the "SAMPLE ID" column. Rock-core barrels can be of either single- or double-tube construction, and a special series of double-tube barrels, designated by the suffix M, may also be used to obtain maximum core recovery in very-soft or fractured rock. Four basic groups of barrels are used most often in subsurface investigations for engineering purposes, and these groups and the diameters of the cores obtained are as follows:

AX, AW, AXM, AWM	-	1-1/8 inches
BX, BW, BXM, BWM	-	1-5/8 inches
NX, NW, NXM, NWM	-	2-1/8 inches
NQ, NQ2	-	1-7/8 inches

Rock Quality Designation (RQD) is expressed as a percentage and is obtained by summing the total length of all core pieces which are at least 4 inches long and then dividing this sum by, either, the total length of core run or the length of the core run in a particular bedrock stratum. The RQD value is reported as a percentage in the "SPT/RQD" column. It has been found that there is a reasonably good relationship between the RQD value and the general quality of rock for engineering purposes. This relationship is shown as follows:

RQD - %	General Quality
0 - 25	Very-poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

ROCK HARDNESS

Recovered bedrock samples are described in general accordance with Section 605 of the latest ODOT SGE and subsequent revisions, where necessary. The following terms are used to describe rock hardness:

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

S&ME JOB: 1117-18-036

PROJECT: CUY-14-1212E
TYPE: BRIDGE REPLACEMENT
PID: 13184 BR ID: 1801929
START: 2/4/20 END: 2/4/20

DRILLING FIRM / OPERATOR: OTB / A. FAY
SAMPLING FIRM / LOGGER: S&ME / A. MAINS
DRILLING METHOD: 3.25" HSA / NQ
SAMPLING METHOD: SPT / NQ

DRILL RIG: OTB MOBILE B-57 (2018)
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 11/5/18
ENERGY RATIO (%): 90*

STATION / OFFSET: 9+97, 9' RT
ALIGNMENT: UNION STREET
ELEVATION: 896.0 (MSL) EOB: 38.5 ft.
LAT / LONG: 41.386110 N, 81.531006 W

EXPLORATION ID
B-001-0-19

PAGE
1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 896.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
ASPHALT - 3-3/4 INCHES	895.7																	
CONCRETE - 9-1/4 INCHES	894.9	1																
GRANULAR BASE - 4 INCHES	894.6	2																
FILL: Hard brown **SILT AND CLAY**, some fine to coarse gravel, little fine to coarse sand, few sandstone fragments, damp.		3 4 5 6 7 8	2 2 3	8	44	SS-1	4.5	28	6	11	28	27	30	16	14	15	A-6a (6)	
- Few organic pockets.	887.1	9	3 9 2	17	61	SS-2A	4.5	-	-	-	-	-	-	-	-	19	A-6a (V)	
FILL: Medium-dense brown **GRAVEL**, little fine to coarse sand, predominantly composed of sandstone fragments, dry.	884.0	10 11				SS-2B		-	-	-	-	-	-	-	-	8	A-1-a (V)	
FILL: Hard brown **SANDY SILT**, some clay, little fine to coarse gravel (sandstone fragments), damp.	878.0	12 13 14 15 16 17 18	4 2 4	9	33	SS-3	4.5	-	-	-	-	-	-	-	-	13	A-4a (V)	
FILL: Loose brown to dark-brown **GRAVEL WITH SAND AND SILT**, trace clay, many sandstone fragments, damp.	876.0	19	2 2 2	6	56	SS-4		41	6	21	22	10	31	23	8	20	A-2-4 (0)	
FILL: Very-loose to medium-dense dark-gray to black **SANDY SILT**, little clay, trace fine to coarse gravel, few brick and sandstone fragments, moist to wet.		20 21	3 2 2	6	56	SS-5		5	3	34	42	16	-	-	-	29	A-4a (V)	
		22	1 1 1	3	100	SS-6		1	3	33	45	18	20	17	3	22	A-4a (6)	
	871.5	23 24	1 2 5	11	78	SS-7		8	5	28	41	18	-	-	-	22	A-4a (V)	
Very-dense gray, **GRAVEL WITH SAND AND SILT**, trace clay, damp to moist.	869.0	25 26	6 14 50-1"	-	92	SS-8		31	9	34	18	8	-	-	-	15	A-2-4 (V)	
SANDSTONE, gray, highly to severely weathered.	867.5	TR 27 28	50-1"	-	100	SS-9		-	-	-	-	-	-	-	-	15	Rock (V)	
		29	62		100	NQ-10											CORE	

S&ME JOB: 1117-18-036

PID: 13184	BR ID: 1801929	PROJECT: CUY-14-1212E	STATION / OFFSET: 9+97, 9' RT	START: 2/4/20	END: 2/4/20	PG 2 OF 2	**B-001-0-19**

MATERIAL DESCRIPTION AND NOTES	ELEV. 866.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
SANDSTONE, gray, slightly weathered, strong, medium grained, medium bedded to thin bedded, slightly to moderately fractured, iron-stained; RQD=84%; REC=100%. *(continued)*	862.7	31 32 33																
SHALE, gray, highly weathered, very weak to weak, thinly laminated, moderately to highly fractured; RQD=10%; REC=92%.		34 35 36 37	47		92	NQ-11											CORE	
	857.5	EOB 38	0		100	NQ-12											CORE	

NOTES:
- No seepage or groundwater encountered during drilling.
- Borehole was observed to be dry at completion.
- Borehole was sealed upon completion.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: ASPHALT PATCH; 1 BAG BENTONITE POWDER; 1/2 BAG CEMENT; PLASTIC HOLE PLUG DEVICE; 40 GAL. WATER

S&ME ODOT LOG (8.5X11) - SGE 01/2019 - OH DOT.GDT - 3/23/22 14:33 - T:\CS\RESOURCES\COLUMBUS\GINTW\PROJECTS\111718036.GPJ

PROJECT: CUY-14-1212E
TYPE: BRIDGE REPLACEMENT
PID: 13184 BR ID: 1801929
START: 2/7/22 END: 2/8/22

DRILLING FIRM / OPERATOR: OTB / C. SVITAK
SAMPLING FIRM / LOGGER: S&ME / A. ROLF
DRILLING METHOD: 3.25" HSA / NQ
SAMPLING METHOD: SPT / NQ

DRILL RIG: OTB MOBILE B-57 (2020)
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 11/25/20
ENERGY RATIO (%): 90*

STATION / OFFSET: 10+06, 11' RT
ALIGNMENT: UNION STREET
ELEVATION: 896.0 (MSL) EOB: 45.5 ft.
LAT / LONG: 41.386120 N, 81.531000 W

EXPLORATION ID
B-001-1-21
PAGE
1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 896.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
ASPHALT - 4-1/2 INCHES	895.7																	
CONCRETE - 8-1/2 INCHES	894.9	1																
GRANULAR BASE - 12 INCHES	893.9	2																
NO SAMPLING PERFORMED - SEE BORING B-001-0-19		3 4 5 6																
- Few brick fragments from 6.5' to 9.0'.		7 8 9 10 11 12 13 14																
- Some slag. - Blue staining due to slag from 15.0' to 26.0'.		15 16 17 18 19																
		W 876.0 20 21 22 23 24 25																
SANDSTONE, gray, weathered to highly weathered, fine to medium grained	870.0 869.6	W TR 869.8 26	50-5"	-	100	SS-1		-	-	-	-	-	-	-	-	-		
			0		100	NQ-2											CORE	
SANDSTONE, gray, slightly weathered, fine to medium grained, moderately strong to strong, thick bedded, slightly fractured, narrow, slightly rough; RQD 81%, REC 99%. @29.7'-30.1', UC = 7,247 psi.		27 28 29	87		100	NQ-3											CORE	

S&ME JOB: 1117-18-036

PID: 13184	BR ID: 1801929	PROJECT: CUY-14-1212E	STATION / OFFSET: 10+06, 11' RT	START: 2/7/22	END: 2/8/22	PG 2 OF 2	**B-001-1-21**

MATERIAL DESCRIPTION AND NOTES	ELEV. 866.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
SANDSTONE, gray, slightly weathered, fine to medium grained, moderately strong to strong, thick bedded, slightly fractured, narrow, slightly rough; RQD 81%, REC 99%. *(continued)*	862.7	31 32 33																
INTERBEDDED SHALE (60%) AND SANDSTONE (40%), RQD 11%, REC 82%. **SHALE**, gray, highly weathered, very fine grained, weak, very thin bedded, fractured to highly fractured, narrow, slightly rough. **SANDSTONE**, gray, unweathered, fine grained, strong, thin bedded, fractured to highly fractured, narrow, slightly rough.		34 35 36	37		91	NQ-4											CORE	
		37 38 39	11		50	NQ-5											CORE	
		40	0		42	NQ-6											CORE	
	850.5	41 42 43 44 45 EOB	20		100	NQ-7											CORE	

NOTES:
- Rock fragment jammed the core barrel during coring between depths of 36.4' and 40.4', limiting recovery of rock core within that range.
- Groundwater was measured at 20' at the end of the day on 2/7/22. Groundwater was measured at 26.2' on 2/8/22 prior to coring.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: 1/3 BAG ASPHALT PATCH; CEMENT MIXED WITH SOIL CUTTINGS

S&ME ODOT LOG (8.5X11) - SGE 01/2019 - OH DOT.GDT - 3/23/22 14:33 - T:\CS\RESOURCES\COLUMBUS\GINT\WPROJECTS\111718036.GPJ

S&ME JOB: 1117-18-036

PROJECT: CUY-14-1212E	DRILLING FIRM / OPERATOR: OTB / C. SVITAK	DRILL RIG: OTB MOBILE B-57 (2020)	STATION / OFFSET: 10+12, 11' RT	EXPLORATION ID **B-001-2-21**
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: S&ME / A. ROLF	HAMMER: CME AUTOMATIC	ALIGNMENT: UNION STREET	
PID: 13184 BR ID: 1801929	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 11/25/20	ELEVATION: 896.0 (MSL) EOB: 20.7 ft.	PAGE 1 OF 1
START: 2/21/22 END: 2/21/22	SAMPLING METHOD: SPT	ENERGY RATIO (%): 90*	LAT / LONG: 41.386130 N, 81.531000 W	

MATERIAL DESCRIPTION AND NOTES	ELEV. 896.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
ASPHALT - 5-1/2 INCHES	895.5																	
CONCRETE - 14 INCHES	894.4	1																
GRANULAR BASE - 12 INCHES	893.4	2																
NO SAMPLING PERFORMED - SEE BORING B-001-0-19	875.5	3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20																
CONCRETE	875.3	EOB	50-3"	-	100	SS-1	-	-	-	-	-	-	-	-	-	-	Visual (V)	

NOTES:
- No seepage or groundwater encountered during drilling.
- Borehole was observed to be dry at completion.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: 1 BAG ASPHALT PATCH; SOIL CUTTINGS

S&ME ODOT LOG (8.5X11) - SGE 01/2019 - OH DOT.GDT - 3/23/22 14:33 - T:\CS\RESOURCES\COLUMBUS\GINTW\PROJECTS\111718036.GPJ

PLATE 9

S&ME JOB: 1117-18-036

PROJECT: CUY-14-1212E	DRILLING FIRM / OPERATOR: OTB / A. FAY	DRILL RIG: OTB MOBILE B-57 (2018)	STATION / OFFSET: 11+50, 13' RT
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: S&ME / A. MAINS	HAMMER: CME AUTOMATIC	ALIGNMENT: UNION STREET
PID: 13184 BR ID: 1801929	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 11/5/18	ELEVATION: 897.0 (MSL) EOB: 35.0 ft.
START: 2/4/20 END: 2/4/20	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 90*	LAT / LONG: 41.386504 N, 81.530836 W

EXPLORATION ID
B-002-0-19
PAGE
1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 897.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
ASPHALT - 3-1/2 INCHES	896.7																	
CONCRETE - 8-1/2 INCHES	895.9																	
FLOWABLE FILL - 3-3/4 INCHES	895.5																	
SLAG CONCRETE - 3-1/4 INCHES	895.2																	
SLAG CONCRETE/BRICK - 4 INCHES	894.9																	
GRANULAR BASE - 4 INCHES	894.6																	
Hard brown **SILT AND CLAY**, little fine to coarse sand, trace fine gravel, damp.			3 6 10	24	100	SS-1	4.5	8	7	13	39	33	28	16	12	16	A-6a (8)	
			6 9 13	33	100	SS-2	4.5	-	-	-	-	-	-	-	-	16	A-6a (V)	
	889.0		4 8 11	29	100	SS-3	4.5	-	-	-	-	-	-	-	-	14	A-6a (V)	
Hard gray **CLAY**, some silt, trace fine to coarse sand, trace fine gravel, damp to moist.	887.5		3 50-4"	-	100	SS-4	4.5	3	3	6	26	62	43	21	22	26	A-7-6 (13)	
SANDSTONE, brownish-gray, highly weathered.	887.0	TR	50-5"	-	100	SS-5	-	-	-	-	-	-	-	-	-	9	Rock (V)	
SANDSTONE, gray, slightly weathered, moderately strong, medium grained, thin to medium bedded, fractured to slightly fractured, narrow to open, slightly rough, zones of iron staining; RQD=18%, REC=83%.	882.0		18		83	NQ-6											CORE	
SANDSTONE, gray and reddish-brown, slightly weathered, moderately strong, medium grained, very thin to thin bedded, ferriferous, fractured to moderately fractured, narrow, slightly rough; RQD=32%, REC=100%.	876.2		36		100	NQ-7											CORE	
SANDSTONE, brown and gray, slightly weathered, moderately strong, medium grained, thin to medium bedded, fractured to moderately fractured, narrow to open, slightly rough, few iron stained zones; RQD=39%, REC=100%.	870.5																	
SANDSTONE, gray, slightly weathered to unweathered, moderately strong, medium grained, very thick bedded, intact, narrow, slightly rough; RQD=85%, REC=100%. @29.7'-30.5', UC = 5,533 psi.			77		100	NQ-8											CORE	

S&ME JOB: 1117-18-036

PID: 13184	BR ID: 1801929	PROJECT: CUY-14-1212E	STATION / OFFSET: 11+50, 13' RT	START: 2/4/20	END: 2/4/20	PG 2 OF 2 **B-002-0-19**

MATERIAL DESCRIPTION AND NOTES		ELEV. 866.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
									GR	CS	FS	SI	CL	LL	PL	PI			
SANDSTONE, gray, slightly weathered to unweathered, moderately strong, medium grained, very thick bedded, intact, narrow, slightly rough; RQD=85%, REC=100%. *(continued)* @32.5'-33.3', UC = 7,891 psi.		862.0	32 33 34 EOB 35																

NOTES:
- No seepage or groundwater encountered during drilling.
- Borehole was observed to be dry at completion.
- Borehole was sealed upon completion.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: ASPHALT PATCH; 1 BAG BENTONITE POWDER; 1/2 BAG CEMENT; PLASTIC HOLE PLUG DEVICE; 40 GAL. WATER

S&ME ODOT LOG (8.5X11) - SGE 01/2019 - OH DOT.GDT - 3/23/22 14:33 - T:\CS\RESOURCES\COLUMBUS\GINTW\PROJECTS\111718036.GPJ

S&ME JOB: 1117-18-036

PROJECT: CUY-14-1212E	DRILLING FIRM / OPERATOR: OTB / C. SVITAK	DRILL RIG:OTB MOBILE B-57 (2020)	STATION / OFFSET: 11+15, 8' RT	EXPLORATION ID **B-002-1-21**
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: S&ME / S. SMITH	HAMMER: CME AUTOMATIC	ALIGNMENT: UNION STREET	
PID: 13184 BR ID: 1801929	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 11/25/20	ELEVATION: 896.0 (MSL) EOB: 44.0 ft.	PAGE 1 OF 2
START: 2/9/22 END: 2/9/22	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 90*	LAT / LONG: 41.386410 N, 81.530900 W	

MATERIAL DESCRIPTION AND NOTES	ELEV. 896.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
ASPHALT - 4-1/2 INCHES	895.7																	
CONCRETE - 12 INCHES	894.7																	
GRANULAR BASE - 13-1/2 INCHES	893.5																	
NO SAMPLING PERFORMED - SEE BORING B-002-0-19	887.5																	
SANDSTONE, light brown, highly weathered, fine to medium grained.	886.1	TR	50	-	100	SS-1	-	-	-	-	-	-	-	-	-	-		
SANDSTONE, light brown to gray, moderately weathered, slightly to moderately strong, fine to medium grained, thin bedded, fractured, narrow, slightly rough; RQD 30%, REC 95%. - Iron staining observed at 9.9' and from 10.3' to 10.7'.	882.1		28		95	NQ-2											CORE	
SANDSTONE, light brown to reddish brown, slightly weathered, fine to medium grained, moderately strong, thin bedded, moderately fractured, narrow, slightly rough; RQD 32%, REC 100%. - Iron staining observed from 14.0' to 20.0'. @17.1'-17.5', UC = 5,208 psi.	876.5		55		100	NQ-3											CORE	
SANDSTONE, light brown to reddish brown, slightly to moderately weathered, fine to medium grained, moderately strong, thin bedded, moderately fractured, narrow, slightly rough, little iron staining; RQD 87%, REC 100%.	869.3																	
SANDSTONE, gray, unweathered to slightly weathered, fine to medium grained, moderately strong to strong, thick bedded, slightly fractured, narrow, slightly rought; RQD 97% REC 100%.			95		100	NQ-4											CORE	

S&ME JOB: 1117-18-036

PID: 13184 | BR ID: 1801929 | PROJECT: CUY-14-1212E | STATION / OFFSET: 11+15, 8' RT | START: 2/9/22 | END: 2/9/22 | PG 2 OF 2 | **B-002-1-21**

MATERIAL DESCRIPTION AND NOTES	ELEV. 866.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
SANDSTONE, gray, unweathered to slightly weathered, fine to medium grained, moderately strong to strong, thick bedded, slightly fractured, narrow, slightly rought; RQD 97% REC 100%. *(continued)*	860.6	31 32 33 34 35																
INTERBEDDED SHALE (90%) AND SANDSTONE (10%), RQD 14%, REC 98%. **SHALE**, gray, highly weathered, very weak to slightly strong, very thinly bedded, highly fractured, narrow, slightly rough. 1" thick clay infilling observed at 40'. **SANDSTONE**, gray, slightly weathered, fine grained, strong, thin bedded, highly fractured, narrow, slightly rough. @39.1'-39.4', UC = 2,133 psi.	852.0	36 37 38 39 40 41 42 43 EOB 44	23		98	NQ-5											CORE	

NOTES:
- No groundwater noted during drilling.
- Borehole was observed to be dry prior to coring.
- After completion of rock coring and during removal of tools, the rock core barrel became lodged. Driller attempted to remove tools, but was unable to do so and tooling was left in the ground from approximately 0.8' to 32' below the ground surface. The borehole/tooling was filled with cuttings mixed with cement and the pavement surface repaired.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: 1/3 BAG ASPHALT PATCH; CEMENT MIXED WITH SOIL CUTTINGS

S&ME ODOT LOG (8.5X11) - SGE 01/2019 - OH DOT.GDT - 3/23/22 14:33 - T:\CS\RESOURCES\COLUMBUS\GINTW\PROJECTS\111718036.GPJ

Form No. TR-43-D7012C-02
Revision No. : 0
Revision Date: 08/22/18

UNCONFINED COMPRESSION
(ASTM D7012 Method C)

S&ME, Inc. - Knoxville 1413 Topside Road, Louisville, TN 37777

Project Name: CUY-14-12.12E Bridge Replacement
Project Number: 1117-18-036

Report Date: February 17, 2020
Reviewed By: N. Randy Rainwater

Boring No.	Sample No.	Depth (ft)	Dimensions, in.		Shape (See Key)	Area (in^2)	Unit Weight (lbs/ft^3)	Loading Rate (psi/sec)	Maximum Load (lbs)	Strength (psi)	Moisture (%)
			Length	Diameter							
B-002-0-19	NQ-8	29.7 - 30.5	4.16	1.97	C	3.05	127.5	67	16,877	5,533	0.2
B-002-0-19	NQ-8	32.5 - 33.3	4.17	1.98	A	3.08	129.6	64	24,304	7,891	0.2

NOTES: Effective (as received) unit weight as determined by RTH 109-93.

Loading rates were selected to target reaching failure between 2 and 15 minutes.

Test results for specimens not meeting the requirements of ASTM D4543-19 may differ from a test specimen that meets the requirements of ASTM D4543.

SHAPE KEY

ASTM D4543-19 Standard Practice for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerance Section 1.2 - "Rock is a complex engineering material that can vary greatly as a function of lithology, stress history, weathering, moisture content and chemistry, and other natural geologic processes. As such, it is not always possible to obtain or prepare rock core specimens that satisfy the desirable tolerances given in this practice. Most commonly, this situation presents itself with weaker, more porous, and poorly cemented rock types and rock types containing significant or weak (or both) structural features. For rock types which are difficult to prepare, all reasonable efforts shall be made to prepare a specimen in accordance with this practice and for the intended test procedure. However, when it has been determined by trial and error that this is not possible, prepare the rock specimen to the closest tolerances practicable and consider this to be the best effort and report it as such and if allowable or necessary for the intended test, capping the ends of the specimen as discussed in this practice is permitted."

A	Test specimen measurements met the desired shape tolerances of ASTM D4543-19 (side straightness, end flatness & parallelism, and end perpendicularity to axis)
B	Test specimen measurements met the desired shape tolerances of ASTM D4543-19 for end flatness & parallelism, and end perpendicularity to axis. Specimen did not meet the desired tolerance for side straightness. Specimen prepared to closest tolerances practicable.
C	Test specimen measurements met the desired shape tolerances of ASTM D4543-19 for end flatness & parallelism. Specimen did not meet the desired tolerances for side straightness and end perpendicularity to axis. Specimen prepared to closest tolerances practicable.
D	Test specimen measurements met the desired shape tolerances of ASTM D4543-19 for end flatness. Specimen did not meet the desired tolerances for side straightness, parallelism and end perpendicularity to axis. Specimen prepared to closest tolerances practicable.
E	Test specimen measurements met the desired shape tolerances of ASTM D4543-19 for end flatness and end perpendicularity to axis. Specimen did not meet the desired tolerance for side straightness and parallelism. Specimen prepared to closest tolerances practicable.

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PREPARING ROCK CORE AS CYLINDRICAL TEST SPECIMENS AND VERIFYING CONFORMANCE TO DIMENSIONAL AND SHAPE TOLERANCES
(ASTM D4543)

1413 Topside Road, Louisville, TN 37777

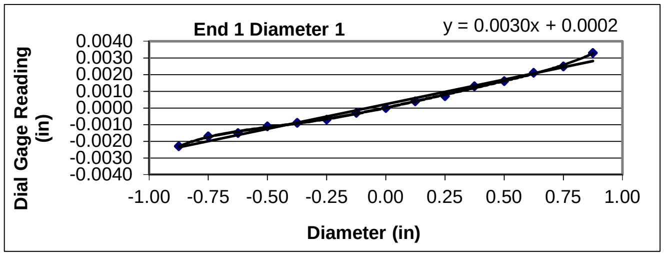
Project:	CUY-14-12.12E Bridge Replacement	**Diameter (in):**	1.97	**Date:**	2/14/2020
Project No.:	1117-18-036	**Length (in):**	4.16	**Tested by:**	Tori Igoe
Boring Id:	B-002-0-19	**Unit Weight (pcf):**	127.5	**Reviewed by:**	Ben Painter
Sample No.:	NQ-8	**Moisture Content (%):**	0.2		
Depth (ft):	29.7 - 30.5				

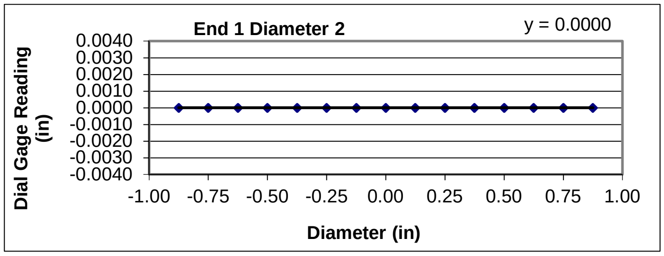
Deviation From Straightness (Procedure S1)

Is the maximum gap ≤ 0.02 in.? **NO** Straightness Tolerance Met? **NO**

End Flatness and Parallelism Readings (Procedure FP1)

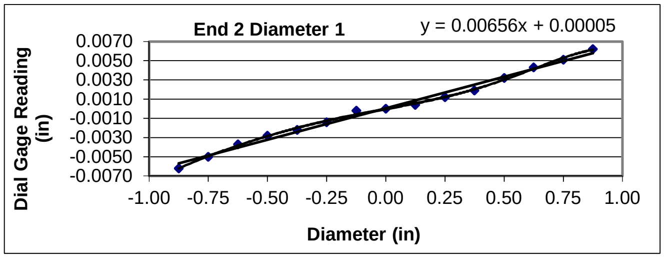
Position	End 1	End 1(90)	End 2	End 2(90)
- 7/8	-0.0023	0.0000	-0.0062	0.0030
- 6/8	-0.0017	0.0000	-0.0050	0.0024
- 5/8	-0.0015	0.0000	-0.0037	0.0020
- 4/8	-0.0011	0.0000	-0.0028	0.0016
- 3/8	-0.0009	0.0000	-0.0022	0.0011
- 2/8	-0.0007	0.0000	-0.0014	0.0009
- 1/8	-0.0003	0.0000	-0.0002	0.0001
0	0.0000	0.0000	0.0000	0.0000
1/8	0.0004	0.0000	0.0004	0.0000
2/8	0.0007	0.0000	0.0012	-0.0005
3/8	0.0013	0.0000	0.0019	-0.0006
4/8	0.0016	0.0000	0.0032	-0.0012
5/8	0.0021	0.0000	0.0043	-0.0015
6/8	0.0025	0.0000	0.0051	-0.0017
7/8	0.0033	0.0000	0.0062	-0.0023

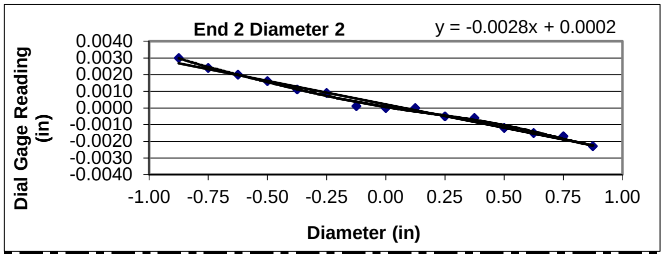




Flatness is met when the difference at any point between a smooth curve drawn through points and a visual best fit line is ≤ 0.001 in.

Flatness Tolerance Met? **YES**





Parallelism is met when the angular difference between best fit lines on opposing ends is ≤ 0.25°.

Parrallelism Diameter 1

End 1:	Slope of Best Fit Line:	0.00295
	Angle of Best Fit Line:	0.16910
End 2:	Slope of Best Fit Line:	0.00656
	Angle of Best Fit Line:	0.37570
	Max Angular Difference:	**-0.21**

Parrallelism Diameter 2

End 1:	Slope of Best Fit Line:	0.00000
	Angle of Best Fit Line:	0.00000
End 2:	Slope of Best Fit Line:	-0.00281
	Angle of Best Fit Line:	-0.16108
	Max Angular Difference:	**0.16**

Parallelism Tolerance Met? **YES**

Perpendicularity (Procedure P1) is met when the difference between max and min readings along each line divided by the diameter is ≤ 0.0043.

	Difference b/w max & min	Divide by Diameter	Meets Tolerance
End 1 Diam 1	0.0056	0.0028	**YES**
End 1 Diam 2	0.0000	0.0000	**YES**
End 2 Diam 1	0.0124	0.0063	**NO**
End 2 Diam 2	0.0053	0.0027	**YES**

Perpendicularity Tolerance Met? **NO**

PREPARING ROCK CORE AS CYLINDRICAL TEST SPECIMENS AND VERIFYING CONFORMANCE TO DIMENSIONAL AND SHAPE TOLERANCES (ASTM D4543)

1413 Topside Road, Louisville, TN 37777

Project:	CUY-14-12.12E Bridge Replacement	**Diameter (in):**	1.98	**Date:**	2/14/2020
Project No.:	1117-18-036	**Length (in):**	4.17	**Tested by:**	Tori Igoe
Boring Id:	B-002-0-19	**Unit Weight (pcf):**	129.6	**Reviewed by:**	Ben Painter
Sample No.:	NQ-8	**Moisture Content (%):**	0.2		
Depth (ft):	32.5 - 33.3				

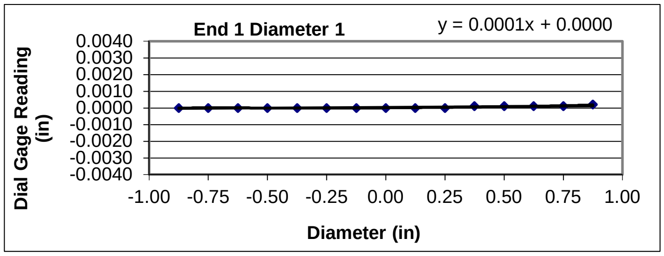
Deviation From Straightness (Procedure S1)

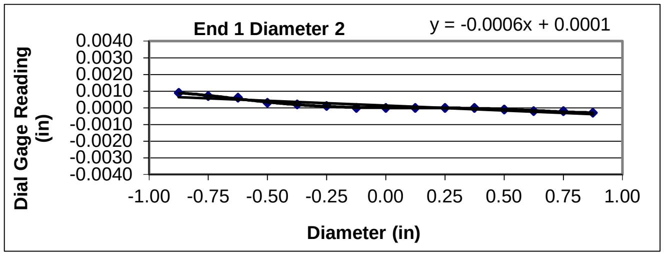
Is the maximum gap ≤ 0.02 in.? YES

Straightness Tolerance Met? YES

End Flatness and Parallelism Readings (Procedure FP1)

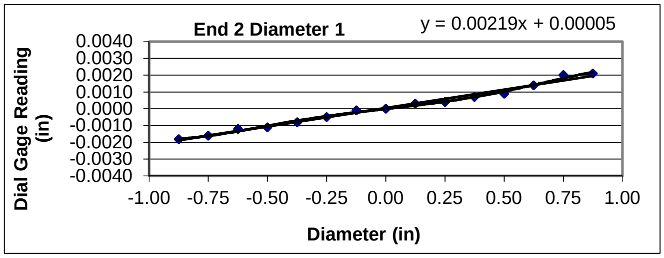
Position	End 1	End 1(90)	End 2	End 2(90)
- 7/8	0.0000	0.0009	-0.0018	-0.0014
- 6/8	0.0000	0.0007	-0.0016	-0.0012
- 5/8	0.0000	0.0006	-0.0012	-0.0011
- 4/8	0.0000	0.0003	-0.0011	-0.0009
- 3/8	0.0000	0.0002	-0.0008	-0.0007
- 2/8	0.0000	0.0001	-0.0005	-0.0004
- 1/8	0.0000	0.0000	-0.0001	-0.0003
0	0.0000	0.0000	0.0000	0.0000
1/8	0.0000	0.0000	0.0003	0.0005
2/8	0.0000	0.0000	0.0004	0.0008
3/8	0.0001	0.0000	0.0007	0.0011
4/8	0.0001	-0.0001	0.0009	0.0012
5/8	0.0001	-0.0002	0.0014	0.0014
6/8	0.0001	-0.0002	0.0020	0.0015
7/8	0.0002	-0.0003	0.0021	0.0017

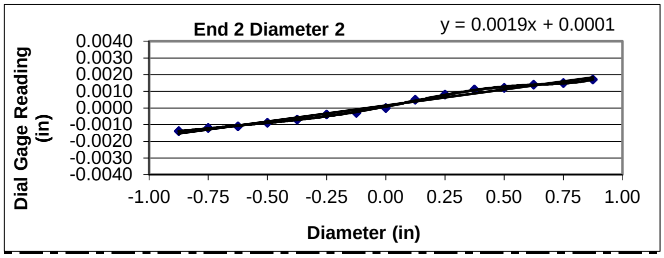




Flatness is met when the difference at any point between a smooth curve drawn through points and a visual best fit line is ≤ 0.001 in.

Flatness Tolerance Met? YES





Parallelism is met when the angular difference between best fit lines on opposing ends is ≤ 0.25°.

Parrallelism Diameter 1

End 1:	Slope of Best Fit Line:	0.00009
	Angle of Best Fit Line:	0.00524
End 2:	Slope of Best Fit Line:	0.00219
	Angle of Best Fit Line:	0.12540
	Max Angular Difference:	**-0.12**

Parrallelism Diameter 2

End 1:	Slope of Best Fit Line:	-0.00058
	Angle of Best Fit Line:	-0.03307
End 2:	Slope of Best Fit Line:	0.00193
	Angle of Best Fit Line:	0.11034
	Max Angular Difference:	**-0.14**

Parallelism Tolerance Met? YES

Perpendicularity (Procedure P1) is met when the difference between max and min readings along each line divided by the diameter is ≤ 0.0043.

	Difference b/w max & min	Divide by Diameter	Meets Tolerance
End 1 Diam 1	0.0002	0.0001	**YES**
End 1 Diam 2	0.0012	0.0006	**YES**
End 2 Diam 1	0.0039	0.0020	**YES**
End 2 Diam 2	0.0031	0.0016	**YES**

Perpendicularity Tolerance Met? **YES**

Date: 2/14/2020

Photographer: Tori Igoe

1	**Location / Orientation**	B-002-0-19 NQ-8, (29.7′ – 30.5′)
	Remarks	Unconfined Compressive Strength of Rock Core Specimen Before/After (ASTM D7012 Method C)

Date: 2/14/2020

Photographer: Tori Igoe

2	**Location / Orientation**	B-002-0-19 NQ-8 (32.5′ – 33.3′)
	Remarks	Unconfined Compressive Strength of Rock Core Specimen Before/After (ASTM D7012 Method C)

Form No. TR-D7012C-01
Revision No. 0
Devision Date: 06/25/15

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D 7012 Method C

Quality Assurance

S&ME, Inc. - Lexington: 2020 Liberty Road, Suite 105, Lexington, KY 40505

Project No.:	1117-18-036	Report Date:	03/15/22
Project Name:	CUY-14-12.12 Bridge Replacement	Test Date(s):	03/14/22
Client Name:	Pennoni Associates, Inc.		
Client Address:	323 West Camden Street, Suite 600, Baltimore, MD	Received Date:	03/03/22
Location:	B-001-1-21 NQ-3	Depth/Elev., ft:	29.7 - 30.1
Sample Description:	Gray Sandstone		

Angle of load relative to lithology: Approximately perpendicular

Test Results

Moisture Content *7.0* % ***Dry Unit Weight*** *129.6* pcf

Compressive Strength *7,247* psi

Strain rate: 0.015 in/min.

Notes / Deviations / References:

J. Folsom	Jacob Folsom	Lab Services Manager	3/15/2022
Technical Responsibility	*Signature*	*Position*	*Date*

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Form No. TR-D7012C-01
Revision No. 0
Devision Date: 06/25/15

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D 7012 Method C

S&ME, Inc. - Lexington: 2020 Liberty Road, Suite 105, Lexington, KY 40505

Project Name: CUY-14-12.12 Bridge Replacement Location: B-001-1-21 NQ-2 Depth, feet: 29.7 - 30.1

Summary of Specimen Tolerances

Length/diameter target:	**MET**	Perpendicularity target:	**MET**
Side straightness target:	**MET**	Planeness target:	**MET**
Parallelism target:	**MET**		

*ASTM D4543-08 Standard Practice for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerance, Section 1.2 - "Rock is a complex engineering material that can vary greatly as a function of lithology, stress history, weathering, moisture content, chemistry, and other natural geologic processes. As such, it is not always possible to obtain or prepare rock core specimens that satisfy the desirable tolerances given in this practice. Most commonly, this situation presents itself with weaker, more porous, and poorly cemented rock types and rock types containing significant or weak (or both) structural features. For these and other rock types which are difficult to prepare, all reasonable efforts shall be made to prepare a specimen in accordance with this practice and for the intended test procedure. However, when it has been determined by trial that this is not possible, the rock specimen will be prepared to the closest tolerance practicable and be considered the best effort and report it as such. If allowable or necessary for the intended test, capping the ends of the specimen as discussed in ASTM D7012 is permitted."

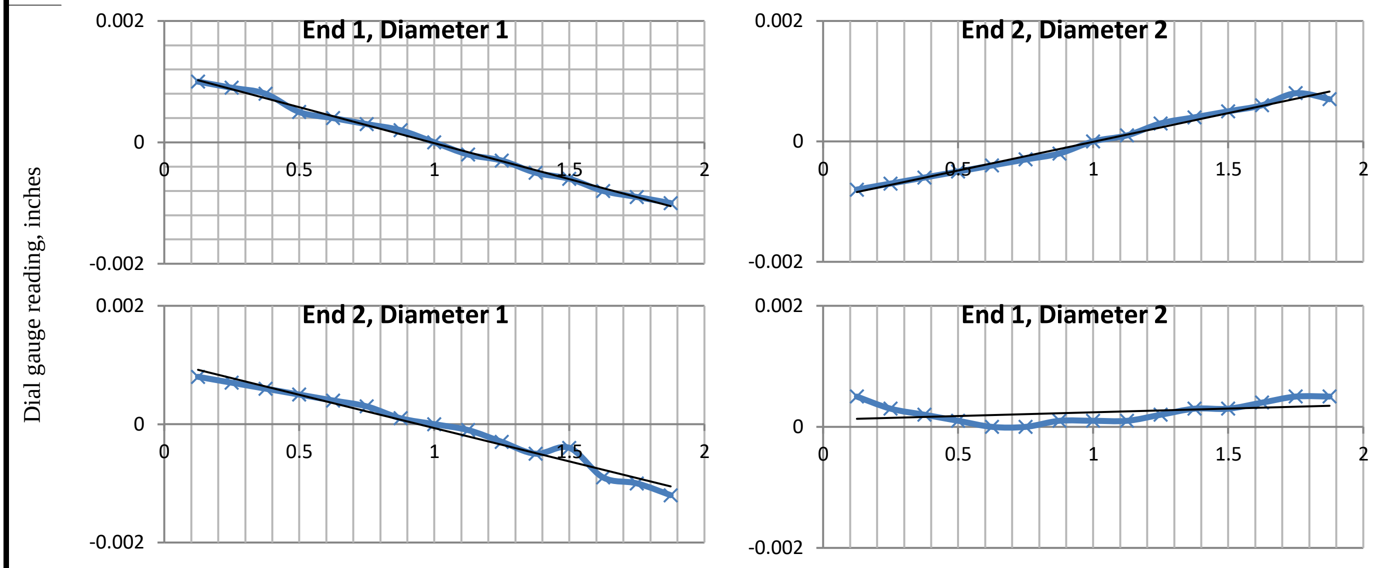
Length to Diameter Ratio

Length, inches: 4.33 Diameter, inches: 1.979
Ratio: 2.19 length to 1 diameter
Target tolerance: L:D ratio between 2 to 1 and 2.5 to 1

Side Straightness

Maximum gap between side of core and reference plate, inches: < .02
Target tolerance: Maximum gap less than .02 inches

Planeness



Distance along diameter, inches

Maximum point-line deviation, inches: < .001
Target Tolerance: No individually measured point should deviate from the best fit line by more than .001 inches.

Parallelism

Slope difference, Diameter 1, degrees: 0.00
Slope difference, Diameter 2, degrees: 0.05
Target Tolerance: Difference between slopes on each end less than 0.25º

Perpendicularity

Slope of End 1, Diameter 1, degrees: -0.07
Slope of End 2, Diameter 1, degrees: -0.06
Slope of End 1, Diameter 2, degrees: 0.01
Slope of End 2, Diameter 2, degrees: 0.05
Target Tolerance: Each diameter perpendicular to the long axis to within 0.25º

Test Information

Strain rate, in/min: 0.015
OR
Stress rate, lbs/sec:
Time to failure, min: 2.7
Temperature: room temperature

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Form No. TR-D7012C-01
Revision No. 0
Devision Date: 06/25/15

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D 7012 Method C

Quality Assurance

S&ME, Inc. - Lexington: 2020 Liberty Road, Suite 105, Lexington, KY 40505

Project No.:	1117-18-036	Report Date:	03/15/22
Project Name:	CUY-14-12.12 Bridge Replacement	Test Date(s):	03/14/22
Client Name:	Pennoni Associates, Inc.		
Client Address:	323 West Camden Street, Suite 600, Baltimore, MD	Received Date:	03/03/22
Location:	B-002-1-21 NQ-3	Depth/Elev., ft:	17.1 - 17.5
Sample Description:	Gray Sandstone		

Angle of load relative to lithology: Approximately perpendicular

Test Results

Moisture Content ***9.1*** **%** ***Dry Unit Weight*** ***126.6*** **pcf**

Compressive Strength ***5,208*** **psi**

Strain rate: 0.015 in/min.

Notes / Deviations / References:

J. Folsom	Jacob Folsom	Lab Services Manager	3/15/2022
Technical Responsibility	*Signature*	*Position*	*Date*

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Form No. TR-D7012C-01
Revision No. 0
Devision Date: 06/25/15

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D 7012 Method C

S&ME, Inc. - Lexington: 2020 Liberty Road, Suite 105, Lexington, KY 40505

Project Name:	CUY-14-12.12 Bridge Replacement	Location:	B-002-1-21 NQ-3	Depth, feet:	17.1 - 17.5

Summary of Specimen Tolerances

Length/diameter target:	**MET**	Perpendicularity target:	**MET**
Side straightness target:	**MET**	Planeness target:	**MET**
Parallelism target:	**MET**		

*ASTM D4543-08 Standard Practice for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerance, Section 1.2 - "Rock is a complex engineering material that can vary greatly as a function of lithology, stress history, weathering, moisture content, chemistry, and other natural geologic processes. As such, it is not always possible to obtain or prepare rock core specimens that satisfy the desirable tolerances given in this practice. Most commonly, this situation presents itself with weaker, more porous, and poorly cemented rock types and rock types containing significant or weak (or both) structural features. For these and other rock types which are difficult to prepare, all reasonable efforts shall be made to prepare a specimen in accordance with this practice and for the intended test procedure. However, when it has been determined by trial that this is not possible, the rock specimen will be prepared to the closest tolerance practicable and be considered the best effort and report it as such. If allowable or necessary for the intended test, capping the ends of the specimen as discussed in ASTM D7012 is permitted."

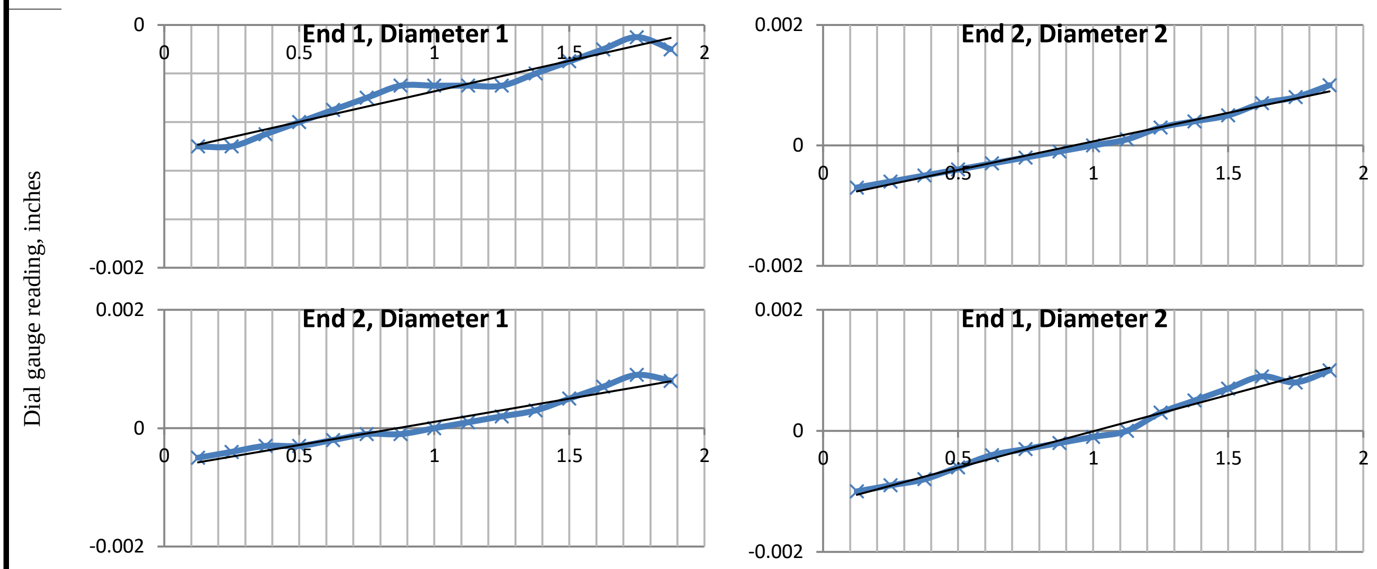
Length to Diameter Ratio

Length, inches: 4.29 Diameter, inches: 1.977
Ratio: 2.17 length to 1 diameter
Target tolerance: L:D ratio between 2 to 1 and 2.5 to 1

Side Straightness

Maximum gap between side of core and reference plate, inches: < .02
Target tolerance: Maximum gap less than .02 inches

Planeness



Distance along diameter, inches

Maximum point-line deviation, inches: < .001
Target Tolerance: No individually measured point should deviate from the best fit line by more than .001 inches.

Parallelism

Slope difference, Diameter 1, degrees:	0.02
Slope difference, Diameter 2, degrees:	0.01

Target Tolerance: Difference between slopes on each end less than 0.25º

Perpendicularity

Slope of End 1, Diameter 1, degrees:	0.03
Slope of End 2, Diameter 1, degrees:	0.04
Slope of End 1, Diameter 2, degrees:	0.07
Slope of End 2, Diameter 2, degrees:	0.05

Target Tolerance: Each diameter perpendicular to the long axis to within 0.25º

Test Information

Strain rate, in/min:	0.015
OR	
Stress rate, lbs/sec:	
Time to failure, min:	2.6
Temperature:	room temperature

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PLATE 21

UNCONFINED COMPRESSION TEST REPORT

ASTM D7012 METHOD C

PROJECT INFORMATION

CLIENT: Pennoni
PROJECT NUMBER: 1117-18-036
PROJECT NAME: CUY-14-12.12 Bridge
PROJECT LOCATION: Bedford, Cuyahoga, Ohio

SAMPLE INFORMATION

BORING ID: B-002-1-21
SAMPLE NUMBER: NQ-5
SAMPLE DEPTH: 38.8' - 39.3'
DATE OF TEST: 2/16/2022

SAMPLE DESCRIPTION: SHALE interbedded with SANDSTONE, gray, slighty strong.

SPECIMEN MEASUREMENTS

MOISTURE CONTENT:	2.87%	
AVERAGE DIAMETER:	1.9783	in.
AVERAGE HEIGHT:	4.2177	in.
HEIGHT/DIAMETER RATIO:	2.13	
WET DENSITY:	162.80	pcf
DRY DENSITY:	158.25	pcf
SPECIFIC GRAVITY:	2.75	(est.)
SATURATION:	93.06%	(est.)
VOID RATIO:	0.0849	(est.)

TEST RESULTS

MAXIMUM LOAD:	**6,556**	lbs
UNCONFINED STRENGTH:	**2,133**	psi
STRAIN RATE:	**~ 75**	psi/s
STRAIN AT FAILURE:	**N/A**	%

ADDITIONAL TESTING REMARKS:

End preparation (per ASTM D4543-08) could not be performed due to the condition of the sample. The sample was capped with gypsum compound. Results reported may differ from results obtained from a test specimen meeting the requirements of D4543.

TESTED BY: EDP

CHECKED BY: BKS

SPECIMEN BEFORE TESTING

SPECIMEN AFTER TESTING

1	**Photographer:**	BKS	**Remarks:**	B-001-0-19
	Date Taken:	2/5/2020		28.5' - 38.5'
	Location / Orientation:	Rear Abutment		

2	**Photographer:**	BKS	**Remarks:**	B-002-0-19
	Date Taken:	2/5/2020		10.0' - 20.0'
	Location / Orientation:	Forward Abutment		

3	**Photographer:**	BKS	**Remarks:**	B-002-0-19
	Date Taken:	2/5/2020		20.0' - 30.0'
	Location / Orientation:			Forward Abutment

4	**Photographer:**	BKS	**Remarks:**	B-002-0-19
	Date Taken:	2/5/2020		30.0' - 35.0'
	Location / Orientation:			Forward Abutment

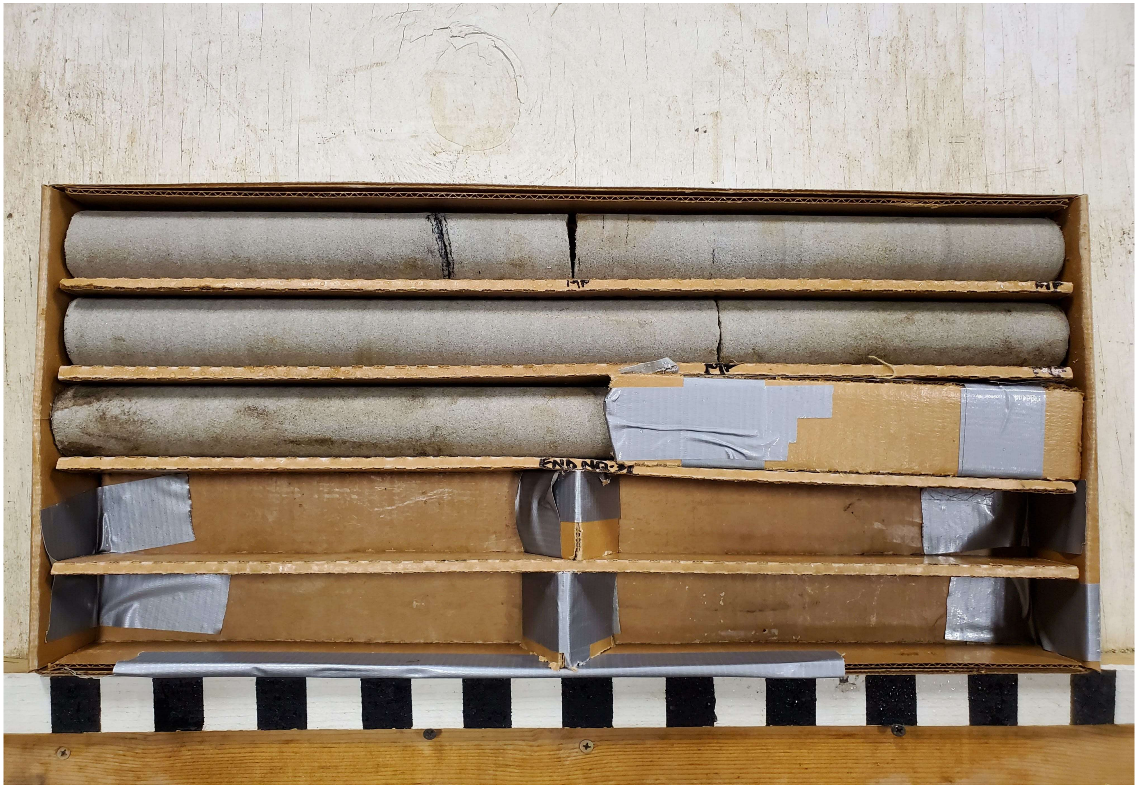
SHEET 1 OF 3

1	**Photographer:**	BKS	**Remarks:**	B-001-1-21
	Date Taken:	2/17/2022		26.4' - 36.4'
	Location / Orientation:		Rear Abutment	

2	**Photographer:**	BKS	**Remarks:**	B-001-1-21
	Date Taken:	2/17/2022		36.4' - 45.5'
	Location / Orientation:		Rear Abutment	

3	**Photographer:**	BKS	**Remarks:**	B-002-1-21 9.9' - 19.6'
	Date Taken:	2/17/2022		
	Location / Orientation:		Forward Abutment	

4	**Photographer:**	BKS	**Remarks:**	B-002-1-21 19.6' - 28.0'
	Date Taken:	2/17/2022		
	Location / Orientation:		Forward Abutment	



5	**Photographer:**	BKS	**Remarks:**	B-002-1-21
	Date Taken:	2/17/2022		28.0' - 34.0'
	Location / Orientation:	Forward Abutment		

6	**Photographer:**	BKS	**Remarks:**	B-002-1-21
	Date Taken:	2/17/2022		34.0' - 44.0'
	Location / Orientation:	Forward Abutment		

1	**Photographer:**	BKS	**Remarks:**	B-001-0-19
	Date Taken:	2/5/2020		
	Location / Orientation:		Rear Abutment	

2	**Photographer:**	BKS	**Remarks:**	B-002-0-19
	Date Taken:	2/5/2020		
	Location / Orientation:		Forward Abutment	

Important Information About Your
Geotechnical Engineering Report

Variations in subsurface conditions can be a principal cause of construction delays, cost overruns and claims. The following information is provided to assist you in understanding and managing the risk of these variations.

Geotechnical Findings Are Professional Opinions

Geotechnical engineers cannot specify material properties as other design engineers do. Geotechnical material properties have a far broader range on a given site than any manufactured construction material, and some geotechnical material properties may change over time because of exposure to air and water, or human activity.

Site exploration identifies subsurface conditions at the time of exploration and only at the points where subsurface tests are performed or samples obtained. Geotechnical engineers review field and laboratory data and then apply their judgment to render professional opinions about site subsurface conditions. Their recommendations rely upon these professional opinions. Variations in the vertical and lateral extent of subsurface materials may be encountered during construction that significantly impact construction schedules, methods and material volumes. While higher levels of subsurface exploration can mitigate the risk of encountering unanticipated subsurface conditions, no level of subsurface exploration can eliminate this risk.

Geotechnical Findings Are Professional Opinions

Professional geotechnical engineering judgment is required to develop a geotechnical exploration scope to obtain information necessary to support design and construction. A number of unique project factors are considered in developing the scope of geotechnical services, such as the exploration objective; the location, type, size and weight of the proposed structure; proposed site grades and improvements; the construction schedule and sequence; and the site geology.

Geotechnical engineers apply their experience with construction methods, subsurface conditions and exploration methods to develop the exploration scope. The scope of each exploration is unique based on available project and site information. Incomplete project information or constraints on the scope of exploration increases the risk of variations in subsurface conditions not being identified and addressed in the geotechnical report.

Services Are Performed for Specific Projects

Because the scope of each geotechnical exploration is unique, each geotechnical report is unique. Subsurface conditions are explored and recommendations are made for a specific project.

Subsurface information and recommendations may not be adequate for other uses. Changes in a proposed structure location, foundation loads, grades, schedule, etc. may require additional geotechnical exploration, analyses, and consultation. The geotechnical engineer should be consulted to determine if additional services are required in response to changes in proposed construction, location, loads, grades, schedule, etc.

Geo-Environmental Issues

The equipment, techniques, and personnel used to perform a geo-environmental study differ significantly from those used for a geotechnical exploration. Indications of environmental contamination may be encountered incidental to performance of a geotechnical exploration but go unrecognized. Determination of the presence, type or extent of environmental contamination is beyond the scope of a geotechnical exploration.

Geotechnical Recommendations Are Not Final

Recommendations are developed based on the geotechnical engineer's understanding of the proposed construction and professional opinion of site subsurface conditions. Observations and tests must be performed during construction to confirm subsurface conditions exposed by construction excavations are consistent with those assumed in development of recommendations. It is advisable to retain the geotechnical engineer that performed the exploration and developed the geotechnical recommendations to conduct tests and observations during construction. This may reduce the risk that variations in subsurface conditions will not be addressed as recommended in the geotechnical report.

Portion obtained with permission from "Important Information About Your Geotechnical Engineering Report", ASFE, 2004

Appendix II – Foundation Calculations

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Calculated By: BKS
Date: 10/25/2022
Checked By: BLM
Date: 10/26/2022

Version 3.0 (10/25/22)

DRILLED SHAFTS IN ROCK - RESISTANCE CALCULATION SUMMARY (AASHTO LRFD, 9th EDITION)

(Example calculations with reference equations and information are provided on additional sheets)

Bridge Structure Identification		Bridge over Tinkers Creek	
Boring ID	B-001-1-21	Foundation Element Description	Rear Abut
Surface Elev.	896	Top of Shaft / Base of Shaft Cap Elevation	883.55

Analysis Desc.	Term/Info Description	Unit	Layer 1	Layer 2	Layer 3	Layer 4
Boring/Layer Information	Bedrock Type/Description		Sandstone	SH in/b SS		
	Layer Top Depth (from G.S.)	ft	26	33.3		
	Layer Top Elevation	MSL	870	862.7		
	Layer Bottom Depth (from G.S.)	ft	33.3	45.5		
	Layer Bottom Elevation	MSL	862.7	850.5		
	Layer Thickness	ft	7.3	12.2		
GSI Index Calculation (AASHTO LRFD, 9th Edition; Hoek, et al., 2013; Bieniawski, Z.T. 1989)	RQD	%	71	30		
	Discontinuity Length Rating		A	A		
	Separation Rating		B	C		
	Roughness Rating		B	C		
	Infilling Rating		A	C		
	Weathering Rating		B	D		
	Estimated JCond89 Value		27	16		
	Estimated GSI Value (quan.)		76	39		
	Estimated GSI Value (qual.)		75	35		
	Design GSI Value		75	37		
Unit Side Resistance Calculations (AASHTO LRFD, 9th Edition)	Compressive Strength, q_u	psi	6470	2130		
	Concrete Strength, f'_c	psi	4000	4000		
	Fractured Rock? (Susceptible to Caving?)		No	No		
	Joint Condition		Closed	Closed		
	Regression Coefficient, C		1.0	1.0		
	q_s (Eqn. 10.8.3.5.4b-1)	ksf	34.94	25.5		
	Reduction Factor, α_E		0.86	0.5		
	q_s (Eqn. 10.8.3.5.4b-2)	ksf	19.53	8.29		
	Design Nominal Side Resistance, q_s	**ksf**	**34.9**	**25.5**		
	Resistance Factor, φ_{qs} (Table 10.5.5.2.4-1)		**0.55**	**0.55**		
	Design Factored Side Resistance, $\varphi_{qs}q_s$	**ksf**	**19.1**	**14**		

Definition of Bedrock Type Abbreviations:

SS = Sandstone	*SH = Shale*	*in/b = interbedded with*
SLTS = Siltstone	*CLST = Claystone*	

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Calculated By: BKS
Date: 10/25/2022
Checked By: BLM
Date: 10/26/2022

Version 3.0 (10/25/22)

DRILLED SHAFTS IN ROCK - RESISTANCE CALCULATION SUMMARY (AASHTO LRFD, 9TH EDITION) - CONTINUED

	Term/Info Description	Unit	Layer 1	Layer 2	Layer 3	Layer 4
Intermediate Parameters Required for GSI Empirical Approach, AASHTO LRFD Eqn. 10.8.3.5.4c-2	Compressive Strength, q_u	ksf	931.68	306.72		
	Disturbance Factor, D		0.2	0.5		
	Empirical Parameter, s		0.0509867	0.0002249		
	Empirical Parameter, a		0.5009	0.5139		
	Constant, m_i (Table 10.4.6.4-1)		17	6		
	Empirical Parameter, m_b		6.3038	0.2987		
	Depth of Soil Cover	ft	26	26		
	Average γ_m of Soil Cover	pcf	120	120		
	Average γ_m of Bedrock	pcf	138.7	140		
	Depth to Water Table	ft	20	20		
	Estimated Shaft Tip Depth (BGS)	ft	32	40		
	Vertical Effective Stress, σ'_{vb}	ksf	3.203	3.832		
	Intermediate Parameter, A		253.75	21.70		
Unit Tip Resistance Calculations (AASHTO LRFD, 9th Edition)	q_p (Eqn. 10.8.3.5.4c-1)	ksf	2329.2	766.8		
	q_p (Eqn. 10.8.3.5.4c-2)	ksf	1493.15	64.19		
Design Nominal Tip Resistance, q_p		**ksf**	**2325**	**765**		
Resistance Factor, φ_{qp} (Table 10.5.5.2.4-1)			**0.5**	**0.5**		
Design Factored Tip Resistance, $\varphi_{qp}q_p$		**ksf**	**1162.5**	**382.5**		

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-001-1-21
Layer Depth Range: 26' - 33.3'
Layer Elevation Range: 870' - 862.7'
Foundation Element: Rear Abut

Calculated By: BKS
Date: 10/25/2022
Checked By: BLM
Date: 10/26/2022

Version 3.0 (10/25/22)

ESTIMATION OF JOINT CONDITION FACTOR (JCond$_{89}$) FOR BEDROCK LAYERS (See Hoek, et al., 2013; Bieniawski, 1989)

Parameter	Specimen Result	Relative Rating	RANGE OF VALUES AND RELATIVE RATINGS				
Discontinuity Length (Persistence) Rating	A	6	A	B	C	D	E
			< 1 m	1 m to 3 m	3 m to 10 m	10 m to 20 m	> 20 m
			RELATIVE RATING				
			6	4	2	1	0
Separation (Aperature) Rating	B	5	A	B	C	D	E
			None	< 0.1 mm	0.1 mm to 1.0 mm	1.0 mm to 5.0 mm	> 5.0 mm
			RELATIVE RATING				
			6	5	4	1	0
Roughness Rating	B	5	A	B	C	D	E
			Very Rough	Rough	Slightly Rough	Smooth	Slickensided
			RELATIVE RATING				
			6	5	3	1	0
Infilling (Gouge) Rating	A	6	A	B	C	D	E
			None	Hard Infilling < 5 mm	Hard Infilling > 5 mm	Soft Infilling < 5 mm	Soft Infilling > 5 mm
			RELATIVE RATING				
			6	4	2	2	0
Weathering Rating	B	5	A	B	C	D	E
			Unweathered	Slightly Weathered	Moderate Weathering	Highly Weathered	Decomposed
			RELATIVE RATING				
			6	5	3	1	0

Layer JCond$_{89}$	References:
27	Hoek, E., Carter, T.G., Diederichs, M.S., *Quantification of the Geological Strength Index Chart* , 47th US Rock Mechanics / Geomechanics Symposium, San Francisco, CA, June 2013 Bieniawski, Z.T. 1989. *Engineering Rock Mass Classification* . New York: Wiley Interscience.

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-001-1-21
Layer Depth Range: 33.3' - 45.5'
Layer Elevation Range: 862.7' - 850.5'
Foundation Element: Rear Abut

Calculated By: BKS
Date: 10/25/2022
Checked By: BLM
Date: 10/26/2022

Version 3.0 (10/25/22)

ESTIMATION OF JOINT CONDITION FACTOR (JCond_{89}) FOR BEDROCK LAYERS (See Hoek, et al., 2013; Bieniawski, 1989)

Parameter	Specimen Result	Relative Rating	RANGE OF VALUES AND RELATIVE RATINGS				
Discontinuity Length (Persistence) Rating	A	6	A	B	C	D	E
			< 1 m	1 m to 3 m	3 m to 10 m	10 m to 20 m	> 20 m
			RELATIVE RATING				
			6	4	2	1	0
Separation (Aperature) Rating	C	4	A	B	C	D	E
			None	< 0.1 mm	0.1 mm to 1.0 mm	1.0 mm to 5.0 mm	> 5.0 mm
			RELATIVE RATING				
			6	5	4	1	0
Roughness Rating	C	3	A	B	C	D	E
			Very Rough	Rough	Slightly Rough	Smooth	Slickensided
			RELATIVE RATING				
			6	5	3	1	0
Infilling (Gouge) Rating	C	2	A	B	C	D	E
			None	Hard Infilling < 5 mm	Hard Infilling > 5 mm	Soft Infilling < 5 mm	Soft Infilling > 5 mm
			RELATIVE RATING				
			6	4	2	2	0
Weathering Rating	D	1	A	B	C	D	E
			Unweathered	Slightly Weathered	Moderate Weathering	Highly Weathered	Decomposed
			RELATIVE RATING				
			6	5	3	1	0

Layer JCond_{89}	References: Hoek, E., Carter, T.G., Diederichs, M.S., *Quantification of the Geological Strength Index Chart* , 47th US Rock Mechanics / Geomechanics Symposium, San Francisco, CA, June 2013 Bieniawski, Z.T. 1989. *Engineering Rock Mass Classification* . New York: Wiley Interscience.
16	

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-001-1-21
Layer Depth Range: 26' - 33.3'
Layer Elevation Range: 870' - 862.7'
Analysis Purpose: Rear Abut

Version 3.0 (10/25/22)

Calc / Check By: BKS BLM
Date: 10/25/22 10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine Unit Side Resistance, q_s (Utilizing 2 Methods)*

Method 1: AASHTO LRFD Equation 10.8.3.5.4b-1

$$\frac{q_s}{p_a} = C\sqrt{\frac{q_u}{p_a}}$$

where:

q_s = unit side resistance (ksf)
q_u = compressive strength of rock (ksf)
p_a = atmospheric pressure (2.12 ksf)
C = Regression Coefficient (see right)

Input Information

q_u = 6470 psi
f'_c = 4000 psi
C = 1.0

Note: *The lesser of q_u or f'_c (compressive strength of concrete) should be used for the value of q_u in Equation 10.8.3.5.4b-1.*

q_s = 34.94 ksf

q_s = 17.47 tsf

Discussion on Regression Coefficient C (from C10.8.3.5.4b)

"The recommended value of the regression coefficient C = 1.0 is applicable to normal rock sockets, defined as sockets constructed with conventional equipment and resulting in nominally clean sidewalls without resorting to special procedures or artificial roughening. Rock that is prone to smearing or rapid deterioration upon exposure to atmospheric conditions, water, or slurry are outside the normal range and may require additional measures to insure reliable side resistance. Rocks exhibiting this type of behavior include clay shales and other argillaceous rocks. Rock that cannot support construction of an unsupported socket without caving is also outside the normal and will likely exhibit lower side resistance than given by Eq. 10.8.3.5.4b-1 with C = 1.0. For additional guidance on assessing the magnitude of C, See Brown et al. (2010)."

Discussion on Regression Coefficient C (from Brown et al. 2010)

"The most recent regression analysis of available load test data is reported by Kulhawy et al. (2005) and demonstrates that the mean value of the coefficient C is approximately equal to 1.0. The authors recommend the use of Equation [10.8.3.5.4b-1] with C = 1.0 for design of "normal" rock sockets. A lower bound value of C = 0.63 was shown to encompass 90% of the load test results...Considering the most recent research on side resistance in rock, in particular the work cited above by Kulhawy et al. (2005) that incorporates the original data of Horvath and Kenney (1979) plus additional data compiled over the ensuing 25+ years, Equation [10.8.3.5.4b-1] with C = 1.0 is recommended for routine design of rock sockets. For rock that cannot be drilled without some type of artificial support, such as casing or by grouting ahead of the excavation, the reduction factors ... based on RQD are recommended for application to the resistance calculated by Equation [10.8.3.5.4b-2]. The resistance factor recommended with use of Equations [10.8.3.5.4b-1] and [10.8.3.5.4b-2] is ϕ = 0.55 based on fitting to ASD with a factor of safety FS = 2.5, as discussed in Chapter 10 and presented in Table 10-5. Artificial roughening of rock sockets through the use of grooving tools or other measures can increase side resistance compared to normal sockets. Regression analysis of the available load test data by Kulhawy and Prakoso (2007) suggests a mean value of C = 1.9 with use of Equation [10.8.3.5.4b-1] for roughened sockets. It is strongly recommended that load tests or local experience be used to verify values of C greater than 1.0. However, the advantages of achieving higher resistance by sidewall roughening often justify the cost of load tests." (emphasis added)

Version 3.0 (10/25/22)

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-001-1-21
Layer Depth Range: 26' - 33.3'
Layer Elevation Range: 870' - 862.7'
Analysis Purpose: Rear Abut

Calc / Check By: BKS BLM
Date: 10/25/22 10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine Unit Side Resistance, q_s (Utilizing 2 Methods) - Continued*

Method 2: AASHTO LRFD Equation 10.8.3.5.4b-2

$$\frac{q_s}{p_a} = 0.65\alpha_E\sqrt{\frac{q_u}{p_a}}$$

where:

q_s = unit side resistance (ksf)
q_u = compressive strength of rock (ksf)
p_a = atmospheric pressure (2.12 ksf)
α_E = joint modification factor (Table 10.8.3.5.4b-1)

Joint Modification Factor, α_E

Table 10.8.3.5.4b-1

RQD (%)	Closed Joints	Open or Gouge-Filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

Input Information

q_u = 6470 psi
f'_c = 4000 psi
RQD = 71 %
Fractured Rock = No *(i.e. susceptible to caving)*
Joint Type = Closed

α_E = 0.86 (Table 10.8.3.5.4b-1)

q_s = 19.53 ksf

q_s = 9.77 tsf

SUMMARY

q_s (routine design) = 34.94 ksf *(eqn. 10.8.3.5.4b-1)*
q_s (fractured rock) = 19.53 ksf *(eqn. 10.8.3.5.4b-2)*

Nominal Side Resistance, q_s = 34.9 ksf

Resistance Factor, φ_{qs} = 0.55 ksf

Factored Side Resistance, $\varphi_{qs}q_s$ = 19.1 ksf

Version 3.0 (10/25/22)

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-001-1-21
Layer Depth Range: 26' - 33.3'
Layer Elevation Range: 870' - 862.7'
Analysis Purpose: Rear Abut

Calc / Check By: BKS BLM
Date: 10/25/22 10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 2 Methods)*

Method 1: AASHTO LRFD Equation 10.8.3.5.4c-1

$$q_p = 2.5q_u$$

where:

q_p = unit end bearing resistance (ksf)
q_u = compressive strength of rock (ksf)

Note: *The lesser of q_u or f'_c (compressive strength of concrete) should be used for the value of q_u in Equation 10.8.3.5.4b-1.*

q_u = 6470 psi
f'_c = 4000 psi

q_p = 2329.2 ksf

q_p = 1164.6 tsf

Discussion on the use of Equation 10.8.3.5.4c-1

"If the rock below the base of the drilled shaft to a depth of 2.0B is either intact or tightly jointed, i.e., no compressible material or gouge-filled seams (including no solution cavities or voids below the base of the drilled shaft per C10.8.3.5.4c), and the depth of the socket is greater than 1.5B."

Method 2: AASHTO LRFD Equation 10.8.3.5.4c-2

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a$$

where:

q_u = compressive strength of rock (ksf)
A = defined by Equation 10.8.3.5.4c-3 (see right)
m_b, s, a = Hoek-Brown strength parameters for the fractured rock mass determined from *GSI* (see Article 10.4.6.4)

Note: *The lesser of q_u or f'_c (compressive strength of concrete) should be used for the value of q_u in Equation 10.8.3.5.4b-2.*

Discussion on the use of Equation 10.8.3.5.4c-2

"If the rock below the base of the shaft to a depth of 2.0B is jointed, the joints have random orientation and the condition of the joints can be evaluated per Equation 10.8.3.5.4c-2....Equation 10.8.3.5.4c-1 should be used as un upper-bound limit to base resistance calculated by Equation 10.8.2.5.4c-2, unless local experience or load tests can be used to validate higher values.

Equation 10.8.3.5.4c-3

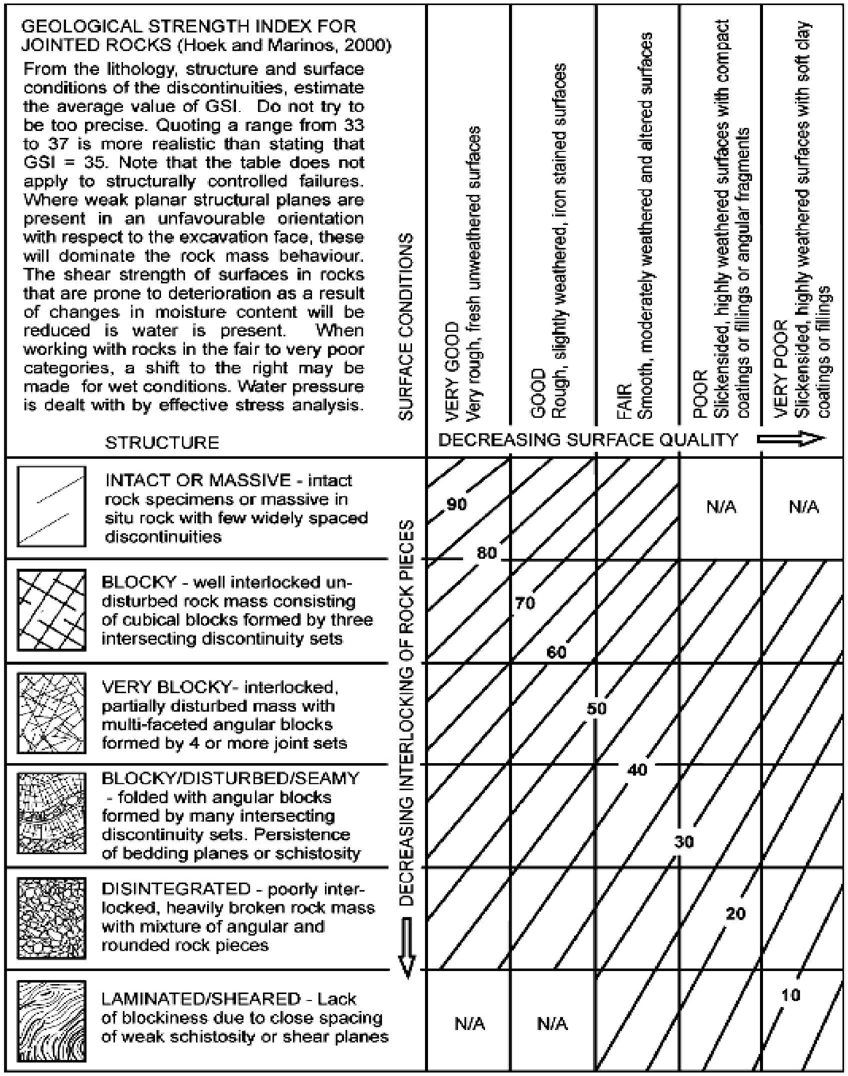
$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{v,b})}{q_u} + s\right]^a$$

where:

$\sigma'_{v,b}$ = vertical effective stress at the socket bearing elevation (tip elevation)

Project Number:	1117-18-036	Boring(s):	B-001-1-21		
Project Name:	CUY-14-12.12 Bridge Replacement	Layer Depth Range:	26' - 33.3'		Version 3.0 (10/25/22)
Project Location:	Bedford, Ohio	Layer Elevation Range:	870' - 862.7'	Calc / Check By:	BKS BLM
Client Name:	Pennoni	Analysis Purpose:	Rear Abut	Date:	10/25/22 10/26/22

Driiled Shafts in Rock - Example Calculations - Determine End Bearing Resistance, q_p (Utilizing 2 Methods) - Continued



From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced is water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.

Figure 10.4.6.4-1—Determination of *GSI* for Jointed Rock Mass (Hoek and Marinos, 2000)

From Article 10.4.6.4

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \qquad \text{Equation 10.4.6.4-2}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \qquad \text{Equation 10.4.6.4-3}$$

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)} \qquad \text{Equation 10.4.6.4-4}$$

where:

GSI = Geological Strength Index *(see Figures 10.4.6.4-1 and 10.4.6.4-2)*

D = Disturbance factor (dim)

m_i = Constant by Rock Group (see Table 10.4.6.4-1)

Note: Only the portion of Table 10.4.6.4-1 including rock types found in Ohio is shown below. Full table may be viewed in Article 10.4.6.4.

Table 10.4.6.4-1 Values of the Constant m_i by Rock Group (after Marinos and Hoek 2000, with updated values from Rocscience, Inc., 2007)

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
		Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2

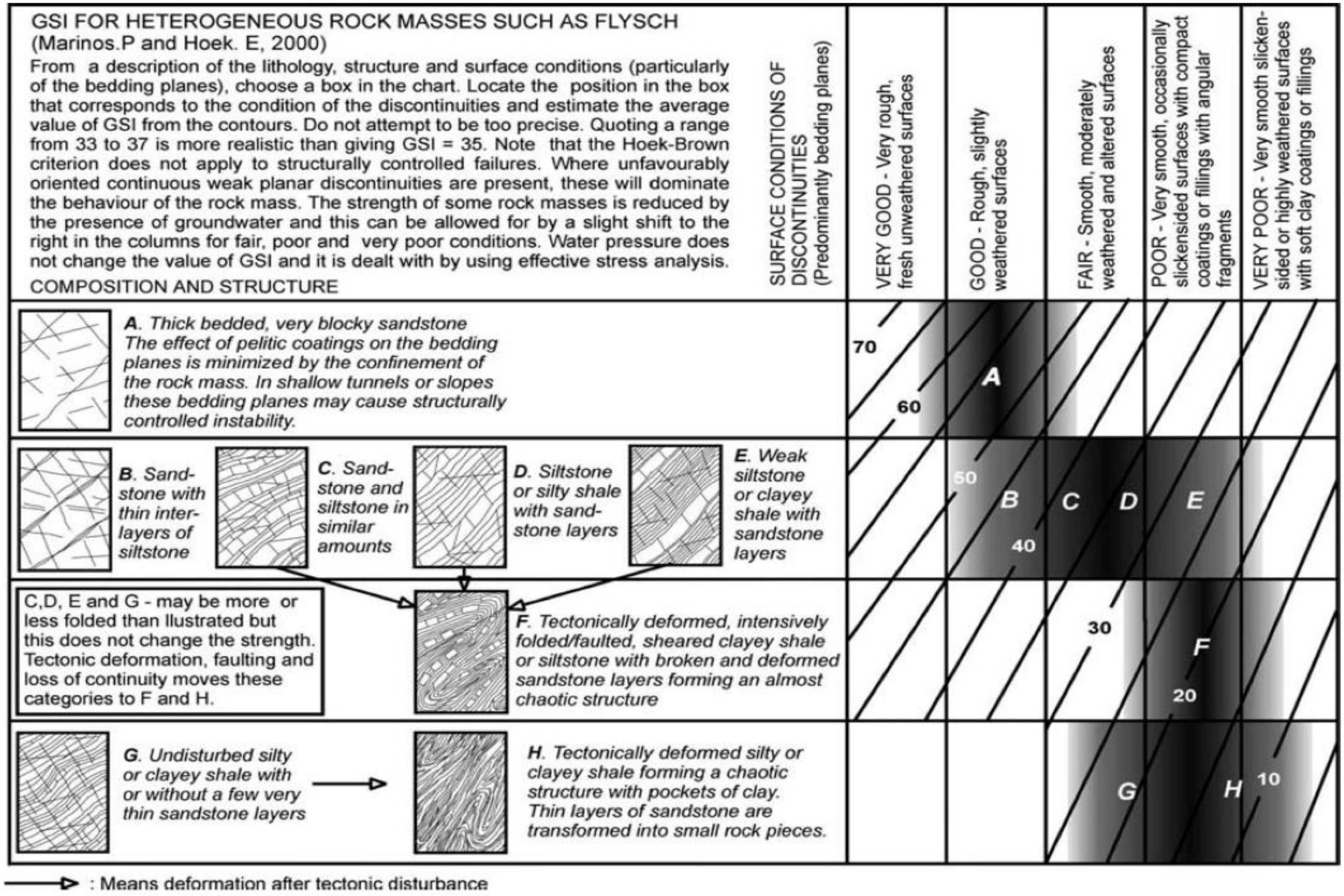
Version 3.0 (10/25/22)

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-001-1-21
Layer Depth Range: 26' - 33.3'
Layer Elevation Range: 870' - 862.7'
Analysis Purpose: Rear Abut

Calc / Check By: BKS BLM
Date: 10/25/22 10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 2 Methods) - Continued*



GSI FOR HETEROGENEOUS ROCK MASSES SUCH AS FLYSCH (Marinos.P and Hoek. E, 2000)

From a description of the lithology, structure and surface conditions (particularly of the bedding planes), choose a box in the chart. Locate the position in the box that corresponds to the condition of the discontinuities and estimate the average value of GSI from the contours. Do not attempt to be too precise. Quoting a range from 33 to 37 is more realistic than giving GSI = 35. Note that the Hoek-Brown criterion does not apply to structurally controlled failures. Where unfavourably oriented continuous weak planar discontinuities are present, these will dominate the behaviour of the rock mass. The strength of some rock masses is reduced by the presence of groundwater and this can be allowed for by a slight shift to the right in the columns for fair, poor and very poor conditions. Water pressure does not change the value of GSI and it is dealt with by using effective stress analysis.

COMPOSITION AND STRUCTURE

SURFACE CONDITIONS OF DISCONTINUITIES (Predominantly bedding planes)

VERY GOOD - Very rough, fresh unweathered surfaces

GOOD - Rough, slightly weathered surfaces

FAIR - Smooth, moderately weathered and altered surfaces

POOR - Very smooth, occasionally slickensided surfaces with compact coatings or fillings with angular fragments

VERY POOR - Very smooth slicken-sided or highly weathered surfaces with soft clay coatings or fillings

A. Thick bedded, very blocky sandstone
The effect of pelitic coatings on the bedding planes is minimized by the confinement of the rock mass. In shallow tunnels or slopes these bedding planes may cause structurally controlled instability.

B. Sandstone with thin inter-layers of siltstone

C. Sandstone and siltstone in similar amounts

D. Siltstone or silty shale with sandstone layers

E. Weak siltstone or clayey shale with sandstone layers

C,D, E and G - may be more or less folded than illustrated but this does not change the strength. Tectonic deformation, faulting and loss of continuity moves these categories to F and H.

F. Tectonically deformed, intensively folded/faulted, sheared clayey shale or siltstone with broken and deformed sandstone layers forming an almost chaotic structure

G. Undisturbed silty or clayey shale with or without a few very thin sandstone layers

H. Tectonically deformed silty or clayey shale forming a chaotic structure with pockets of clay. Thin layers of sandstone are transformed into small rock pieces.

: Means deformation after tectonic disturbance

Figure 10.4.6.4-2—Determination of *GSI* for Tectonically Deformed Heterogeneous Rock Masses (Marinos and Hoek 2000)

Note: Additional information on the GSI method may be found in "Hoek's Corner" on the Rocsciences website (https://www.rocscience.com/education/hoeks_corner), which contains additional articles on the background, assumption, purposes, estimation and calculation of GSI. Of special note are the articles titled "GSI: A Geologically Friendly Tool for Rock Mass Strength Estimation" (Marinos, P. and Hook, E. 2000) and "Quantification of the Geological Strength Index Chart" (Hoek, E., Carter, T.G., Diederichs, M.S., 2013).

Version 3.0 (10/25/22)

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-001-1-21
Layer Depth Range: 26' - 33.3'
Layer Elevation Range: 870' - 862.7'
Analysis Purpose: Rear Abut

Calc / Check By: BKS BLM
Date: 10/25/22 10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 2 Methods) - Continued*

Step 1: Estimate GSI and Hoek-Brown strength parameters using analytical method outlined in "Quantification of the Geological Strength Index Chart" (Hoek, E., Carter, T.G., Diederichs, M.S., 2013) and visually by using Figures 10.4.6.4-1 and 10.4.6.4-2

RQD = 71
$JCond_{89}$ = 27

GSI (Quan.) = 76 $GSI = 1.5JCond_{89} + RQD/2$
GSI (Qual.) = 75 from Figures 10.4.6.4-1 & 10.4.6.4-2

GSI (Design) = 75

D = 0.2
m_i = 17

s = 0.0509867
a = 0.5009
m_b = 6.3038

Step 2: Determine vertical effective stress at shaft tip and intermediate paremeter, *A*

Unconfined Compressive Strength of Bedrock (q_u) = 576.00 ksf
Depth to bottom of Soil Cover & Decomposed Rock (D_s) = 26 ft
Average Unit Weight of Soil Cover ($\gamma_{m,soil}$) = 120 pcf
Average Unit Weight of Bedrock ($\gamma_{m,rock}$) = 138.7 pcf
Depth to Water Table (D_w) = 20 ft
Estimated Shaft Tip Depth Below Ground Surface (D_t) = 32 ft

$$\sigma'_{v,b} = D_s\gamma'_{soil} + (D_t - D_s)\gamma'_{rock}$$

where:

$\gamma'_{soil} = \gamma_{m,soil\ -62.4}$	$\gamma'_{soil} = \gamma_{m,soil}$
$\gamma'_{rock} = \gamma_{m,rock\ -62.4}$	$\gamma'_{rock} = \gamma_{m,rock}$
when below water table	when above water table

σ'_{vb} = 3.203 ksf

A = 171.79

Step 3: Determine estimated tip resistance

q_p = 972.69 ksf

q_p = 486.35 tsf

ect Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-001-1-21
Layer Depth Range: 26' - 33.3'
Layer Elevation Range: 870' - 862.7'
Analysis Purpose: Rear Abut

Version 3.0 (10/25/22)

Calc / Check By:	BKS	BLM
Date:	10/25/22	10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 2 Methods) - Continued*

End Bearing Resistance, q_p Summary

Method	Reference	q_p Value	Unit
1	AASHTO LRFD Eqn. 10.8.3.5.4c-1	2329.2	ksf
2	AASHTO LRFD Eqn. 10.8.3.5.4c-2	972.69	ksf

Nominal Tip Resistance, q_p = 2325 ksf

Resistance Factor, φ_{qp} = 0.5

Factored Tip Resistance, $\varphi_{qp}q_p$ = 1162.5 ksf

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Calculated By: BKS
Date: 10/25/2022
Checked By: BLM
Date: 10/26/2022

Version 3.0 (10/25/22)

DRILLED SHAFTS IN ROCK - RESISTANCE CALCULATION SUMMARY (AASHTO LRFD, 9th EDITION)

(Example calculations with reference equations and information are provided on additional sheets)

Bridge Structure Identification		Bridge over Tinkers Creek	
Boring ID	B-002-1-21	Foundation Element Description	Fwd Abut
Surface Elev.	896	Top of Shaft / Base of Shaft Cap Elevation	884.1

Analysis Desc.	Term/Info Description	Unit	Layer 1	Layer 2	Layer 3	Layer 4
Boring/Layer Information	Bedrock Type/Description		Sandstone	Sandstone	Sandstone	SH in/b SS
	Layer Top Depth (from G.S.)	ft	9.9	19.5	26.7	35.4
	Layer Top Elevation	MSL	886.1	876.5	869.3	860.6
	Layer Bottom Depth (from G.S.)	ft	19.5	26.7	35.4	44
	Layer Bottom Elevation	MSL	876.5	869.3	860.6	852
	Layer Thickness	ft	9.6	7.2	8.7	8.6
GSI Index Calculation (AASHTO LRFD, 9th Edition; Hoek, et al., 2013; Bieniawski, Z.T. 1989)	RQD	%	31	87	97	14
	Discontinuity Length Rating		A	A	B	A
	Separation Rating		C	B	C	C
	Roughness Rating		B	B	C	C
	Infilling Rating		A	A	A	D
	Weathering Rating		C	C	B	D
	Estimated JCond89 Value		24	25	22	16
	Estimated GSI Value (quan.)		51.5	81	81.5	31
	Estimated GSI Value (qual.)		45	60	80	27
	Design GSI Value		50	70	80	30
Unit Side Resistance Calculations (AASHTO LRFD, 9th Edition)	Compressive Strength, q_u	psi	5210	5840	6470	2130
	Concrete Strength, f'_c	psi	4000	4000	4000	4000
	Fractured Rock? (Susceptible to Caving?)		No	No	No	No
	Joint Condition		Closed	Closed	Closed	Closed
	Regression Coefficient, C		1.0	1.0	1.0	1.0
	q_s (Eqn. 10.8.3.5.4b-1)	ksf	34.94	34.94	34.94	25.5
	Reduction Factor, α_E		0.51	0.94	0.99	0.45
	q_s (Eqn. 10.8.3.5.4b-2)	ksf	11.58	21.35	22.49	7.46
Design Nominal Side Resistance, q_s		**ksf**	**34.9**	**34.9**	**34.9**	**25.5**
Resistance Factor, φ_{qs} (Table 10.5.5.2.4-1)			**0.55**	**0.55**	**0.55**	**0.55**
Design Factored Side Resistance, $\varphi_{qs}q_s$		**ksf**	**19.1**	**19.1**	**19.1**	**14**

Definition of Bedrock Type Abbreviations:

SS = Sandstone
SH = Shale
in/b = interbedded with
SLTS = Siltstone
CLST = Claystone

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Calculated By: BKS
Date: 10/25/2022
Checked By: BLM
Date: 10/26/2022

Version 3.0 (10/25/22)

DRILLED SHAFTS IN ROCK - RESISTANCE CALCULATION SUMMARY (AASHTO LRFD, 9TH EDITION) - CONTINUED

	Term/Info Description	Unit	Layer 1	Layer 2	Layer 3	Layer 4
Intermediate Parameters Required for GSI Empirical Approach, AASHTO LRFD Eqn. 10.8.3.5.4c-2	Compressive Strength, q_u	ksf	750.24	840.96	931.68	306.72
	Disturbance Factor, D		0.3	0.1	0	0.6
	Empirical Parameter, s		0.0020853	0.0318004	0.1083680	0.0000599
	Empirical Parameter, a		0.5057	0.5014	0.5006	0.5223
	Constant, m_i (Table 10.4.6.4-1)		17	17	17	6
	Empirical Parameter, m_b		2.08	5.5036	8.3222	0.1687
	Depth of Soil Cover	ft	8	8	8	8
	Average γ_m of Soil Cover	pcf	120	120	120	120
	Average γ_m of Bedrock	pcf	128	128	128	162.8
	Depth to Water Table	ft	19	19	19	19
	Estimated Shaft Tip Depth (BGS)	ft	15	23.5	31	39.5
	Vertical Effective Stress, σ'_{vb}	ksf	1.856	2.663	3.155	4.809
	Intermediate Parameter, A		63.89	188.47	347.02	18.79
Unit Tip Resistance Calculations (AASHTO LRFD, 9th Edition)	q_p (Eqn. 10.8.3.5.4c-1)	ksf	1875.6	2102.4	2329.2	766.8
	q_p (Eqn. 10.8.3.5.4c-2)	ksf	378.38	1134.7	2016.95	47.03
Design Nominal Tip Resistance, q_p		**ksf**	**1875**	**2100**	**2325**	**760**
Resistance Factor, φ_{qp} (Table 10.5.5.2.4-1)			**0.5**	**0.5**	**0.5**	**0.5**
Design Factored Tip Resistance, $\varphi_{qp}q_p$		**ksf**	**937.5**	**1050**	**1162.5**	**380**

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-002-1-21
Layer Depth Range: 9.9' - 19.5'
Layer Elevation Range: 886.1' - 876.5'
Foundation Element: Fwd Abut

Calculated By: BKS
Date: 10/25/2022
Checked By: BLM
Date: 10/26/2022

Version 3.0 (10/25/22)

ESTIMATION OF JOINT CONDITION FACTOR (JCond$_{89}$) FOR BEDROCK LAYERS (See Hoek, et al., 2013; Bieniawski, 1989)

Parameter	Specimen Result	Relative Rating	RANGE OF VALUES AND RELATIVE RATINGS				
Discontinuity Length (Persistence) Rating	A	6	A	B	C	D	E
			< 1 m	1 m to 3 m	3 m to 10 m	10 m to 20 m	> 20 m
			RELATIVE RATING				
			6	4	2	1	0
Separation (Aperature) Rating	C	4	A	B	C	D	E
			None	< 0.1 mm	0.1 mm to 1.0 mm	1.0 mm to 5.0 mm	> 5.0 mm
			RELATIVE RATING				
			6	5	4	1	0
Roughness Rating	B	5	A	B	C	D	E
			Very Rough	Rough	Slightly Rough	Smooth	Slickensided
			RELATIVE RATING				
			6	5	3	1	0
Infilling (Gouge) Rating	A	6	A	B	C	D	E
			None	Hard Infilling < 5 mm	Hard Infilling > 5 mm	Soft Infilling < 5 mm	Soft Infilling > 5 mm
			RELATIVE RATING				
			6	4	2	2	0
Weathering Rating	C	3	A	B	C	D	E
			Unweathered	Slightly Weathered	Moderate Weathering	Highly Weathered	Decomposed
			RELATIVE RATING				
			6	5	3	1	0

Layer JCond$_{89}$
24

References:
Hoek, E., Carter, T.G., Diederichs, M.S., *Quantification of the Geological Strength Index Chart* , 47th US Rock Mechanics / Geomechanics Symposium, San Francisco, CA, June 2013
Bieniawski, Z.T. 1989. *Engineering Rock Mass Classification* . New York: Wiley Interscience.

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-002-1-21
Layer Depth Range: 19.5' - 26.7'
Layer Elevation Range: 876.5' - 869.3'
Foundation Element: Fwd Abut

Calculated By: BKS
Date: 10/25/2022
Checked By: BLM
Date: 10/26/2022

Version 3.0 (10/25/22)

ESTIMATION OF JOINT CONDITION FACTOR ($JCond_{89}$) FOR BEDROCK LAYERS (See Hoek, et al., 2013; Bieniawski, 1989)

Parameter	Specimen Result	Relative Rating	RANGE OF VALUES AND RELATIVE RATINGS				
Discontinuity Length (Persistence) Rating	A	6	A	B	C	D	E
			< 1 m	1 m to 3 m	3 m to 10 m	10 m to 20 m	> 20 m
			RELATIVE RATING				
			[✓ 6]	4	2	1	0
Separation (Aperature) Rating	B	5	A	B	C	D	E
			None	< 0.1 mm	0.1 mm to 1.0 mm	1.0 mm to 5.0 mm	> 5.0 mm
			RELATIVE RATING				
			6	[✓ 5]	4	1	0
Roughness Rating	B	5	A	B	C	D	E
			Very Rough	Rough	Slightly Rough	Smooth	Slickensided
			RELATIVE RATING				
			6	[✓ 5]	3	1	0
Infilling (Gouge) Rating	A	6	A	B	C	D	E
			None	Hard Infilling < 5 mm	Hard Infilling > 5 mm	Soft Infilling < 5 mm	Soft Infilling > 5 mm
			RELATIVE RATING				
			[✓ 6]	4	2	2	0
Weathering Rating	C	3	A	B	C	D	E
			Unweathered	Slightly Weathered	Moderate Weathering	Highly Weathered	Decomposed
			RELATIVE RATING				
			6	5	[✓ 3]	1	0

Layer $JCond_{89}$
25

References:
Hoek, E., Carter, T.G., Diederichs, M.S., *Quantification of the Geological Strength Index Chart* , 47th US Rock Mechanics / Geomechanics Symposium, San Francisco, CA, June 2013
Bieniawski, Z.T. 1989. *Engineering Rock Mass Classification* . New York: Wiley Interscience.

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-002-1-21
Layer Depth Range: 26.7' - 35.4'
Layer Elevation Range: 869.3' - 860.6'
Foundation Element: Fwd Abut

Calculated By: BKS
Date: 10/25/2022
Checked By: BLM
Date: 10/26/2022

Version 3.0 (10/25/22)

ESTIMATION OF JOINT CONDITION FACTOR ($JCond_{89}$) FOR BEDROCK LAYERS (See Hoek, et al., 2013; Bieniawski, 1989)

Parameter	Specimen Result	Relative Rating	RANGE OF VALUES AND RELATIVE RATINGS				
Discontinuity Length (Persistence) Rating	B	4	A	B	C	D	E
			< 1 m	1 m to 3 m	3 m to 10 m	10 m to 20 m	> 20 m
			RELATIVE RATING				
			6	4	2	1	0
Separation (Aperature) Rating	C	4	A	B	C	D	E
			None	< 0.1 mm	0.1 mm to 1.0 mm	1.0 mm to 5.0 mm	> 5.0 mm
			RELATIVE RATING				
			6	5	4	1	0
Roughness Rating	C	3	A	B	C	D	E
			Very Rough	Rough	Slightly Rough	Smooth	Slickensided
			RELATIVE RATING				
			6	5	3	1	0
Infilling (Gouge) Rating	A	6	A	B	C	D	E
			None	Hard Infilling < 5 mm	Hard Infilling > 5 mm	Soft Infilling < 5 mm	Soft Infilling > 5 mm
			RELATIVE RATING				
			6	4	2	2	0
Weathering Rating	B	5	A	B	C	D	E
			Unweathered	Slightly Weathered	Moderate Weathering	Highly Weathered	Decomposed
			RELATIVE RATING				
			6	5	3	1	0

Layer $JCond_{89}$	References:
22	Hoek, E., Carter, T.G., Diederichs, M.S., *Quantification of the Geological Strength Index Chart* , 47th US Rock Mechanics / Geomechanics Symposium, San Francisco, CA, June 2013 Bieniawski, Z.T. 1989. *Engineering Rock Mass Classification* . New York: Wiley Interscience.

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-002-1-21
Layer Depth Range: 35.4' - 44'
Layer Elevation Range: 860.6' - 852'
Foundation Element: Fwd Abut

Calculated By: BKS
Date: 10/25/2022
Checked By: BLM
Date: 10/26/2022

Version 3.0 (10/25/22)

ESTIMATION OF JOINT CONDITION FACTOR (JCond$_{89}$) FOR BEDROCK LAYERS (See Hoek, et al., 2013; Bieniawski, 1989)

Parameter	Specimen Result	Relative Rating	RANGE OF VALUES AND RELATIVE RATINGS				
Discontinuity Length (Persistence) Rating	A	6	A	B	C	D	E
			< 1 m	1 m to 3 m	3 m to 10 m	10 m to 20 m	> 20 m
			RELATIVE RATING				
			6	4	2	1	0
Separation (Aperature) Rating	C	4	A	B	C	D	E
			None	< 0.1 mm	0.1 mm to 1.0 mm	1.0 mm to 5.0 mm	> 5.0 mm
			RELATIVE RATING				
			6	5	4	1	0
Roughness Rating	C	3	A	B	C	D	E
			Very Rough	Rough	Slightly Rough	Smooth	Slickensided
			RELATIVE RATING				
			6	5	3	1	0
Infilling (Gouge) Rating	D	2	A	B	C	D	E
			None	Hard Infilling < 5 mm	Hard Infilling > 5 mm	Soft Infilling < 5 mm	Soft Infilling > 5 mm
			RELATIVE RATING				
			6	4	2	2	0
Weathering Rating	D	1	A	B	C	D	E
			Unweathered	Slightly Weathered	Moderate Weathering	Highly Weathered	Decomposed
			RELATIVE RATING				
			6	5	3	1	0

Layer JCond$_{89}$	References: Hoek, E., Carter, T.G., Diederichs, M.S., *Quantification of the Geological Strength Index Chart* , 47th US Rock Mechanics / Geomechanics Symposium, San Francisco, CA, June 2013 Bieniawski, Z.T. 1989. *Engineering Rock Mass Classification* . New York: Wiley Interscience.
16	

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-002-1-21
Layer Depth Range: 9.9' - 19.5'
Layer Elevation Range: 886.1' - 876.5'
Analysis Purpose: Fwd Abut

Version 3.0 (10/25/22)

Calc / Check By:	BKS	BLM
Date:	10/25/22	10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine Unit Side Resistance, q_s (Utilizing 2 Methods)*

Method 1: AASHTO LRFD Equation 10.8.3.5.4b-1

$$\frac{q_s}{p_a} = C\sqrt{\frac{q_u}{p_a}}$$

where:

q_s = unit side resistance (ksf)
q_u = compressive strength of rock (ksf)
p_a = atmospheric pressure (2.12 ksf)
C = Regression Coefficient (see right)

Input Information

q_u = 5210 psi
f'_c = 4000 psi
C = 1.0

Note: *The lesser of q_u or f'_c (compressive strength of concrete) should be used for the value of q_u in Equation 10.8.3.5.4b-1.*

q_s = 34.94 ksf

q_s = 17.47 tsf

Discussion on Regression Coefficient C (from C10.8.3.5.4b)

"The recommended value of the regression coefficient C = 1.0 is applicable to normal rock sockets, defined as sockets constructed with conventional equipment and resulting in nominally clean sidewalls without resorting to special procedures or artificial roughening. Rock that is prone to smearing or rapid deterioration upon exposure to atmospheric conditions, water, or slurry are outside the normal range and may require additional measures to insure reliable side resistance. Rocks exhibiting this type of behavior include clay shales and other argillaceous rocks. Rock that cannot support construction of an unsupported socket without caving is also outside the normal and will likely exhibit lower side resistance than given by Eq. 10.8.3.5.4b-1 with C = 1.0. For additional guidance on assessing the magnitude of C, See Brown et al. (2010)."

Discussion on Regression Coefficient C (from Brown et al. 2010)

"The most recent regression analysis of available load test data is reported by Kulhawy et al. (2005) and demonstrates that the mean value of the coefficient C is approximately equal to 1.0. The authors recommend the use of Equation [10.8.3.5.4b-1] with C = 1.0 for design of "normal" rock sockets. A lower bound value of C = 0.63 was shown to encompass 90% of the load test results...Considering the most recent research on side resistance in rock, in particular the work cited above by Kulhawy et al. (2005) that incorporates the original data of Horvath and Kenney (1979) plus additional data compiled over the ensuing 25+ years, Equation [10.8.3.5.4b-1] with C = 1.0 is recommended for routine design of rock sockets. For rock that cannot be drilled without some type of artificial support, such as casing or by grouting ahead of the excavation, the reduction factors ... based on RQD are recommended for application to the resistance calculated by Equation [10.8.3.5.4b-2]. The resistance factor recommended with use of Equations [10.8.3.5.4b-1] and [10.8.3.5.4b-2] is ϕ = 0.55 based on fitting to ASD with a factor of safety FS = 2.5, as discussed in Chapter 10 and presented in Table 10-5. Artificial roughening of rock sockets through the use of grooving tools or other measures can increase side resistance compared to normal sockets. Regression analysis of the available load test data by Kulhawy and Prakoso (2007) suggests a mean value of C = 1.9 with use of Equation [10.8.3.5.4b-1] for roughened sockets. It is strongly recommended that load tests or local experience be used to verify values of C greater than 1.0. However, the advantages of achieving higher resistance by sidewall roughening often justify the cost of load tests." (emphasis added)

Project Number:	1117-18-036	Boring(s):	B-002-1-21			
Project Name:	CUY-14-12.12 Bridge Replacement	Layer Depth Range:	9.9' - 19.5'			
Project Location:	Bedford, Ohio	Layer Elevation Range:	886.1' - 876.5'	Calc / Check By:	BKS	BLM
Client Name:	Pennoni	Analysis Purpose:	Fwd Abut	Date:	10/25/22	10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine Unit Side Resistance, q_s (Utilizing 2 Methods) - Continued*

Method 2: AASHTO LRFD Equation 10.8.3.5.4b-2

$$\frac{q_s}{p_a} = 0.65\alpha_E\sqrt{\frac{q_u}{p_a}}$$

where:

q_s = unit side resistance (ksf)

q_u = compressive strength of rock (ksf)

p_a = atmospheric pressure (2.12 ksf)

α_E = joint modification factor (Table 10.8.3.5.4b-1)

Joint Modification Factor, α_E

Table 10.8.3.5.4b-1

RQD (%)	Closed Joints	Open or Gouge-Filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

Input Information

q_u =	5210	psi
f'_c =	4000	psi
RQD =	31	%
Fractured Rock =	No	*(i.e. susceptible to caving)*
Joint Type =	Closed	

α_E = 0.51 (Table 10.8.3.5.4b-1)

q_s = 11.58 ksf

q_s = 5.79 tsf

SUMMARY

q_s (routine design) = 34.94 ksf *(eqn. 10.8.3.5.4b-1)*

q_s (fractured rock) = 11.58 ksf *(eqn. 10.8.3.5.4b-2)*

Nominal Side Resistance, q_s = 34.9 ksf

Resistance Factor, φ_{qs} = 0.55 ksf

Factored Side Resistance, $\varphi_{qs}q_s$ = 19.1 ksf

Project Number: 1117-18-036

Project Name: CUY-14-12.12 Bridge Replacement

Project Location: Bedford, Ohio

Client Name: Pennoni

Boring(s): B-002-1-21

Layer Depth Range: 9.9' - 19.5'

Layer Elevation Range: 886.1' - 876.5'

Analysis Purpose: Fwd Abut

Version 3.0 (10/25/22)

Calc / Check By:	BKS	BLM
Date:	10/25/22	10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 2 Methods)*

Method 1: AASHTO LRFD Equation 10.8.3.5.4c-1

$$q_p = 2.5q_u$$

where:

q_p = unit end bearing resistance (ksf)

q_u = compressive strength of rock (ksf)

Note: The lesser of q_u or f'_c (compressive strength of concrete) should be used for the value of q_u in Equation 10.8.3.5.4b-1.

q_u = 5210 psi

f'_c = 4000 psi

q_p = 1875.6 ksf

q_p = 937.8 tsf

Discussion on the use of Equation 10.8.3.5.4c-1

"If the rock below the base of the drilled shaft to a depth of 2.0B is either intact or tightly jointed, i.e., no compressible material or gouge-filled seams (including no solution cavities or voids below the base of the drilled shaft per C10.8.3.5.4c), and the depth of the socket is greater than 1.5B."

Method 2: AASHTO LRFD Equation 10.8.3.5.4c-2

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a$$

where:

q_u = compressive strength of rock (ksf)

A = defined by Equation 10.8.3.5.4c-3 (see right)

m_b, *s*, *a* = Hoek-Brown strength parameters for the fractured rock mass determined from *GSI* (see Article 10.4.6.4)

Note: The lesser of q_u or f'_c (compressive strength of concrete) should be used for the value of q_u in Equation 10.8.3.5.4b-2.

Discussion on the use of Equation 10.8.3.5.4c-2

"If the rock below the base of the shaft to a depth of 2.0B is jointed, the joints have random orientation and the condition of the joints can be evaluated per Equation 10.8.3.5.4c-2....Equation 10.8.3.5.4c-1 should be used as un upper-bound limit to base resistance calculated by Equation 10.8.2.5.4c-2, unless local experience or load tests can be used to validate higher values.

Equation 10.8.3.5.4c-3

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{v,b})}{q_u} + s\right]^a$$

where:

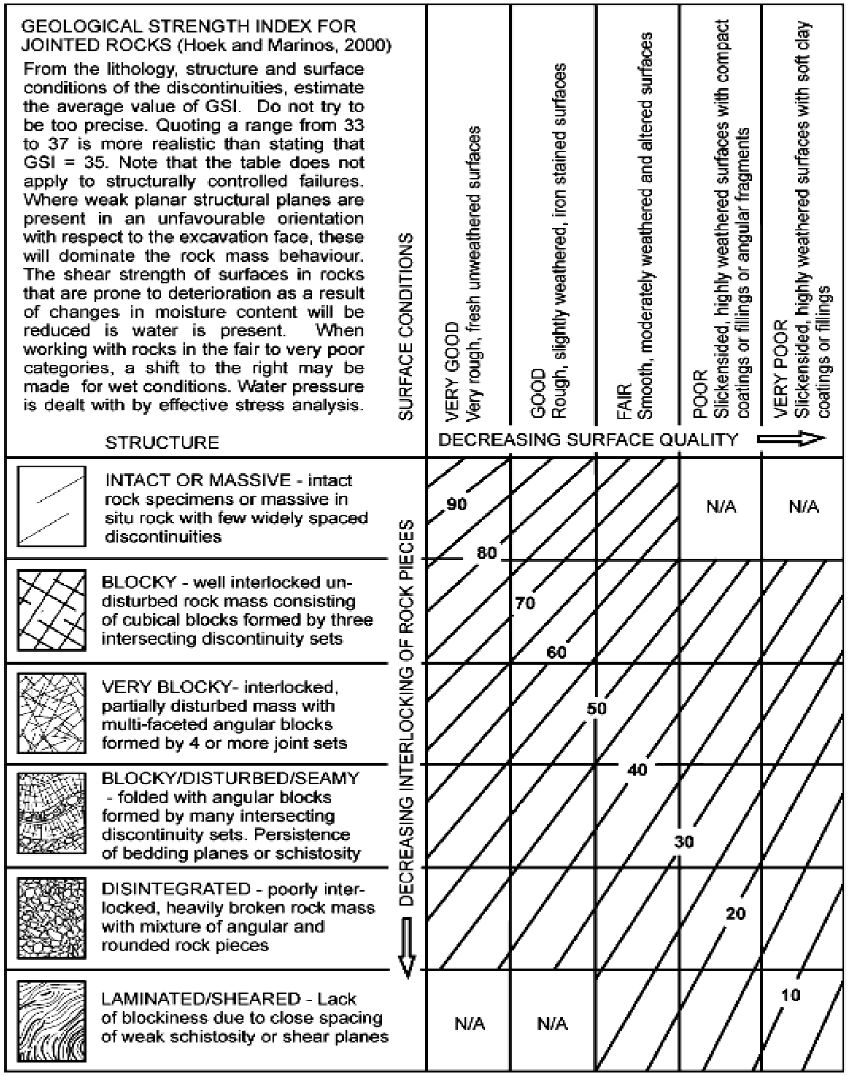
$\sigma'_{v,b}$ = vertical effective stress at the socket bearing elevation (tip elevation)

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-002-1-21
Layer Depth Range: 9.9' - 19.5'
Layer Elevation Range: 886.1' - 876.5'
Analysis Purpose: Fwd Abut

Version 3.0 (10/25/22)
Calc / Check By: BKS BLM
Date: 10/25/22 10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 2 Methods) - Continued*



GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek and Marinos, 2000)

From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced is water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.

INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities

BLOCKY - well interlocked un-disturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets

VERY BLOCKY- interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets

BLOCKY/DISTURBED/SEAMY - folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity

DISINTEGRATED - poorly inter-locked, heavily broken rock mass with mixture of angular and rounded rock pieces

LAMINATED/SHEARED - Lack of blockiness due to close spacing of weak schistosity or shear planes

Figure 10.4.6.4-1—Determination of *GSI* for Jointed Rock Mass (Hoek and Marinos, 2000)

From Article 10.4.6.4

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)}$$ Equation 10.4.6.4-2

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$ Equation 10.4.6.4-3

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)}$$ Equation 10.4.6.4-4

where:

GSI = Geological Strength Index *(see Figures 10.4.6.4-1 and 10.4.6.4-2)*

D = Disturbance factor (dim)

m_i = Constant by Rock Group (see Table 10.4.6.4-1)

Note: Only the portion of Table 10.4.6.4-1 including rock types found in Ohio is shown below. Full table may be viewed in Article 10.4.6.4.

Table 10.4.6.4-1 Values of the Constant m_i by Rock Group (after Marinos and Hoek 2000, with updated values from Rocscience, Inc., 2007)

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
		Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2

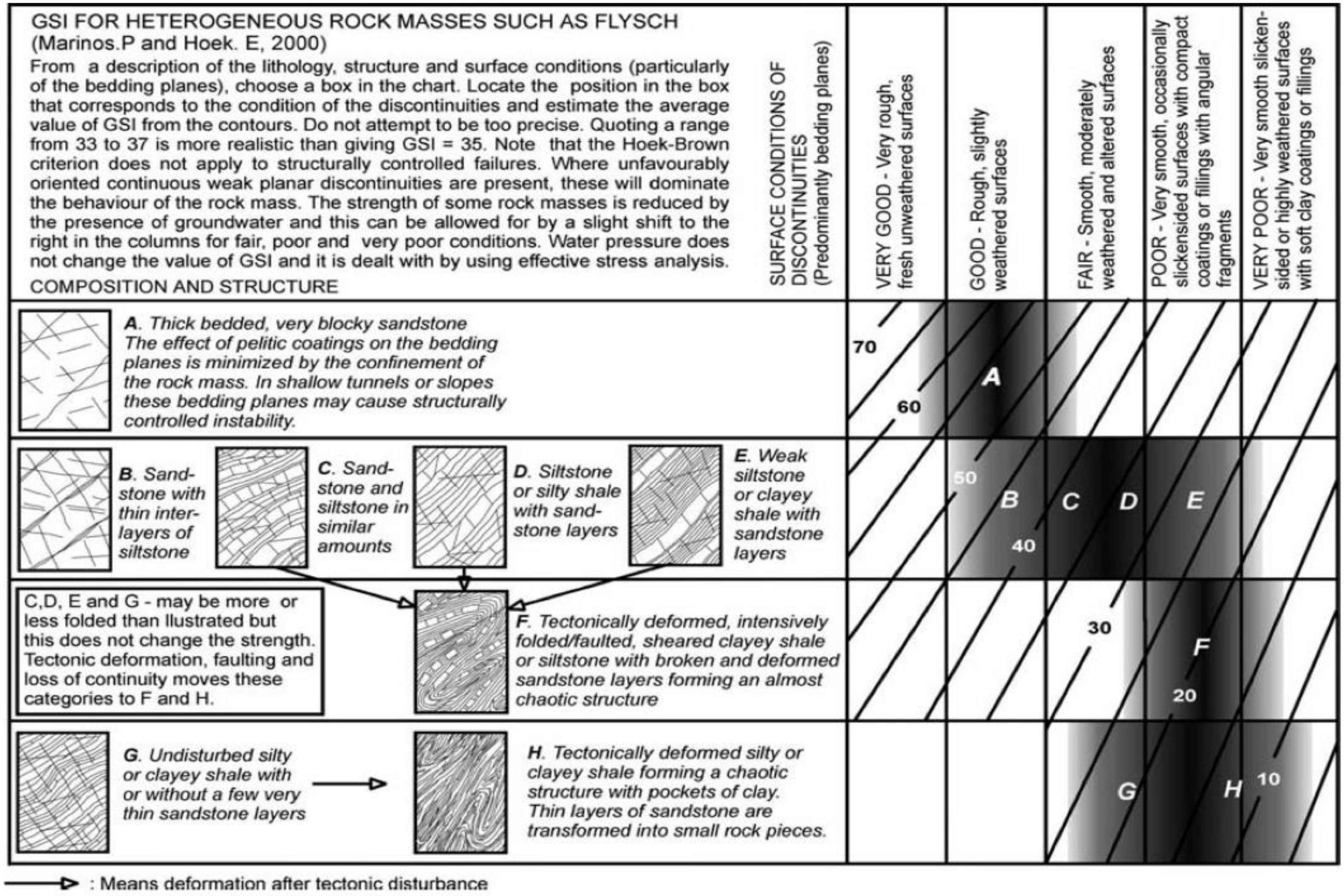
Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-002-1-21
Layer Depth Range: 9.9' - 19.5'
Layer Elevation Range: 886.1' - 876.5'
Analysis Purpose: Fwd Abut

Version 3.0 (10/25/22)

Calc / Check By:	BKS	BLM
Date:	10/25/22	10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 2 Methods) - Continued*


GSI FOR HETEROGENEOUS ROCK MASSES SUCH AS FLYSCH
(Marinos.P and Hoek. E, 2000)

From a description of the lithology, structure and surface conditions (particularly of the bedding planes), choose a box in the chart. Locate the position in the box that corresponds to the condition of the discontinuities and estimate the average value of GSI from the contours. Do not attempt to be too precise. Quoting a range from 33 to 37 is more realistic than giving GSI = 35. Note that the Hoek-Brown criterion does not apply to structurally controlled failures. Where unfavourably oriented continuous weak planar discontinuities are present, these will dominate the behaviour of the rock mass. The strength of some rock masses is reduced by the presence of groundwater and this can be allowed for by a slight shift to the right in the columns for fair, poor and very poor conditions. Water pressure does not change the value of GSI and it is dealt with by using effective stress analysis.

A. Thick bedded, very blocky sandstone
The effect of pelitic coatings on the bedding planes is minimized by the confinement of the rock mass. In shallow tunnels or slopes these bedding planes may cause structurally controlled instability.

B. Sandstone with thin inter-layers of siltstone

C. Sandstone and siltstone in similar amounts

D. Siltstone or silty shale with sandstone layers

E. Weak siltstone or clayey shale with sandstone layers

C,D, E and G - may be more or less folded than illustrated but this does not change the strength. Tectonic deformation, faulting and loss of continuity moves these categories to F and H.

F. Tectonically deformed, intensively folded/faulted, sheared clayey shale or siltstone with broken and deformed sandstone layers forming an almost chaotic structure

G. Undisturbed silty or clayey shale with or without a few very thin sandstone layers

H. Tectonically deformed silty or clayey shale forming a chaotic structure with pockets of clay. Thin layers of sandstone are transformed into small rock pieces.

→ : Means deformation after tectonic disturbance

Figure 10.4.6.4-2—Determination of *GSI* for Tectonically Deformed Heterogeneous Rock Masses (Marinos and Hoek 2000)

Note: Additional information on the GSI method may be found in "Hoek's Corner" on the Rocsciences website (https://www.rocscience.com/education/hoeks_corner), which contains additional articles on the background, assumption, purposes, estimation and calculation of GSI. Of special note are the articles titled "GSI: A Geologically Friendly Tool for Rock Mass Strength Estimation" (Marinos, P. and Hook, E. 2000) and "Quantification of the Geological Strength Index Chart" (Hoek, E., Carter, T.G., Diederichs, M.S., 2013).

Project Number: 1117-18-036
Project Name: CUY-14-12.12 Bridge Replacement
Project Location: Bedford, Ohio
Client Name: Pennoni

Boring(s): B-002-1-21
Layer Depth Range: 9.9' - 19.5'
Layer Elevation Range: 886.1' - 876.5'
Analysis Purpose: Fwd Abut

Version 3.0 (10/25/22)

Calc / Check By: BKS BLM
Date: 10/25/22 10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 2 Methods) - Continued*

Step 1: Estimate GSI and Hoek-Brown strength parameters using analytical method outlined in "Quantification of the Geological Strength Index Chart" (Hoek, E., Carter, T.G., Diederichs, M.S., 2013) and visually by using Figures 10.4.6.4-1 and 10.4.6.4-2

RQD = 31
$JCond_{89}$ = 24

GSI (Quan.) = 51.5 $GSI = 1.5JCond_{89} + RQD/2$
GSI (Qual.) = 45 from Figures 10.4.6.4-1 & 10.4.6.4-2

GSI (Design) = 50

D = 0.3
m_i = 17

s = 0.0020853
a = 0.5057
m_b = 2.08

Step 2: Determine vertical effective stress at shaft tip and intermediate paremeter, *A*

Unconfined Compressive Strength of Bedrock (q_u) = 576.00 ksf
Depth to bottom of Soil Cover & Decomposed Rock (D_s) = 8 ft
Average Unit Weight of Soil Cover ($\gamma_{m,soil}$) = 120 pcf
Average Unit Weight of Bedrock ($\gamma_{m,rock}$) = 128 pcf
Depth to Water Table (D_w) = 19 ft
Estimated Shaft Tip Depth Below Ground Surface (D_t) = 15 ft

$$\sigma'_{v,b} = D_s\gamma'_{soil} + (D_t - D_s)\gamma'_{rock}$$

where:

$\gamma'_{soil} = \gamma_{m,soil\,-62.4}$	$\gamma'_{soil} = \gamma_{m,soil}$
$\gamma'_{rock} = \gamma_{m,rock\,-62.4}$	$\gamma'_{rock} = \gamma_{m,rock}$
when below water table	when above water table

σ'_{vb} = 1.856 ksf

A = 54.41

Step 3: Determine estimated tip resistance

q_p = 308.73 ksf

q_p = 154.37 tsf

PLATE 23

ect Number: 1117-18-036

Project Name: CUY-14-12.12 Bridge Replacement

Project Location: Bedford, Ohio

Client Name: Pennoni

Boring(s): B-002-1-21

Layer Depth Range: 9.9' - 19.5'

Layer Elevation Range: 886.1' - 876.5'

Analysis Purpose: Fwd Abut

Version 3.0 (10/25/22)

Calc / Check By: BKS BLM

Date: 10/25/22 10/26/22

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 2 Methods) - Continued*

End Bearing Resistance, q_p Summary

Method	Reference	q_p Value	Unit
1	AASHTO LRFD Eqn. 10.8.3.5.4c-1	1875.6	ksf
2	AASHTO LRFD Eqn. 10.8.3.5.4c-2	308.73	ksf

Nominal Tip Resistance, q_p = $\boxed{1875}$ ksf

Resistance Factor, φ_{qp} = $\boxed{0.5}$

Factored Tip Resistance, $\varphi_{qp}q_p$ = $\boxed{937.5}$ ksf

Appendix III – ODOT Geotechnical Checklists

I. Geotechnical Design Checklists

Project: CUY-14-12.12E **PDP Path:**

PID: 13184 **Review Stage:** Stage 3

Y

Checklist	**Included in This Submission**
II. Reconnaissance and Planning	✓
III. A. Centerline Cuts	
III. B. Embankments	
III. C. Subgrade	
IV. A. Foundations of Structures	✓
IV. B. Retaining Wall	
V. A. Landslide Remediation	
V. B. Rockfall Remediation	
V. C. Wetland or Peat Remediation	
V. D. Underground Mine Remediation	
V. E. Surface Mine Remediation	
V. F. Karst Remediation	
VI. A. Soil Profile	
VI. D. Geotechnical Reports	✓

II. Reconnaissance and Planning Checklist

C-R-S:	CUY-14-12.12E	**PID:**	13184	**Stage 2**	NDA	**Date:**	6/24/2022

Reconnaissance		(Y/N/X)	Notes:
1	Based on Section 302.1 in the SGE, have the necessary plans been developed in the following areas prior to the commencement of the subsurface exploration reconnaissance:	Y	
	Roadway plans		
	Structures plans	Y	
	Geohazards plans		
2	Have the resources listed in Section 302.2.1 of the SGE been reviewed as part of the office reconnaissance?	Y	
3	Have all the features listed in Section 302.3 of the SGE been observed and evaluated during the field reconnaissance?	Y	
4	If notable features were discovered in the field reconnaissance, were the GPS coordinates of these features recorded?	X	

Planning - General		(Y/N/X)	Notes:
5	In planning the geotechnical exploration program for the project, have the specific geologic conditions, the proposed work, and historic subsurface exploration work been considered?	Y	
6	Has the ODOT Transportation Information Mapping System (TIMS) been accessed to find all available historic boring information and inventoried geohazards?	Y	
7	Have the borings been located to develop the maximum subsurface information while using a minimum number of borings, utilizing historic geotechnical explorations to the fullest extent possible?	Y	
8	Have the topography, geologic origin of materials, surface manifestation of soil conditions, and any other special design considerations been utilized in determining the spacing and depth of borings?	Y	
9	Have the borings been located so as to provide adequate overhead clearance for the equipment, clearance of underground utilities, minimize damage to private property, and minimize disruption of traffic, without compromising the quality of the exploration?	Y	

II. Reconnaissance and Planning Checklist

Planning - General		(Y/N/X)	Notes:
10	Have the scaled boring plans, showing all project and historic borings, and a schedule of borings in tabular format, been submitted to the District Geotechnical Engineer?	Y	
	The schedule of borings should present the following information for each boring:		
a.	exploration identification number	Y	
b.	location by station and offset	Y	
c.	estimated amount of rock and soil, including the total for each for the entire program.	Y	

Planning – Exploration Number		(Y/N/X)	Notes:
11	Have the coordinates, stations and offsets of all explorations (borings, probes, test pits, etc.) been identified?	Y	
12	Has each exploration been assigned a unique identification number, in the following format X-ZZZ-W-YY, as per Section 303.2 of the SGE?	Y	
13	When referring to historic explorations that did not use the identification scheme in 12 above, have the historic explorations been assigned identification numbers according to Section 303.2 of the SGE?	X	

II. Reconnaissance and Planning Checklist

Planning – Boring Types		(Y/N/X)	Notes:
14	Based on Sections 303.3 to 303.7.6 of the SGE, have the location, depth, and sampling requirements for the following boring types been determined for the project?	Y	
	Check all boring types utilized for this project:		
	Existing Subgrades (Type A)		
	Roadway Borings (Type B)		
	Embankment Foundations (Type B1)		
	Cut Sections (Type B2)		
	Sidehill Cut Sections (Type B3)		
	Sidehill Cut-Fill Sections (Type B4)		
	Sidehill Fill Sections on Unstable Slopes (Type B5)		
	Geohazard Borings (Type C)		
	Lakes, Ponds, and Low-Lying Areas (Type C1)		
	Peat Deposits, Compressible Soils, and Low Strength Soils (Type C2)		
	Uncontrolled Fills, Waste Pits, and Reclaimed Surface Mines (Type C3)		
	Underground Mines (C4)		
	Landslides (Type C5)		
	Rockfall (Type C6)		
	Karst (Type C7)		
	Proposed Underground Utilities (Type D)		
	Structure Borings (Type E)	✓	
	Bridges (Type E1)	✓	
	Culverts (Type E2 a,b,c)		
	Retaining Walls (Type E3 a,b,c)		
	Noise Barrier (Type E4)		
	CCTV & High Mast Lighting Towers (Type E5)		
	Buildings and Salt Domes (Type E6)		

IV.A Foundations of Structures Checklist

C-R-S: CUY-14-12.12E	**PID:** 13184	**Stage 2** NDA	**Date:** 6/24/2022

If you do not have such a foundation or structure on the project, you do not have to fill out this checklist.

Soil and Bedrock Strength Data		Y	Notes:
1	Has the shear strength of the foundation soils been determined?	X	
	Check method used:	Y	
	laboratory shear tests		
	estimation from SPT or field tests		
2	Have sufficient soil shear strength, consolidation, and other parameters been determined so that the required allowable loads for the foundation/structure can be designed?	X	
3	Has the shear strength of the foundation bedrock been determined?	Y	Unconfined Compression Tests were performed on select bedrock samples.
	Check method used:		
	laboratory shear tests		
	other (describe other methods)	✓	
Spread Footings		(Y/N/X)	Notes:
4	Are there spread footings on the project? If no, go to Question 11	N	
5	Have the recommended bottom of footing elevation and reason for this recommendation been provided?		
a.	Has the recommended bottom of footing elevation taken scour from streams or other water flow into account?		
6	Were representative sections analyzed for the entire length of the structure for the following:		
a.	factored bearing resistance?		
b.	factored sliding resistance?		
c.	eccentric load limitations (overturning)?		
d.	predicted settlement?		
e.	overall (global) stability?		
7	Has the need for a shear key been evaluated?		
a.	If needed, have the details been included in the plans?		
8	If special conditions exist (e.g. geometry, sloping rock, varying soil conditions), was the bottom of footing “stepped” to accommodate them?		
9	Have the Service I and Maximum Strength Limit States for bearing pressure on soil or rock been provided?		

IV.A Foundations of Structures Checklist

Spread Footings		(Y/N/X)	Notes:
10	If weak soil is present at the proposed foundation level, has the removal / treatment of this soil been developed and included in the plans?		
a.	Have the procedure and quantities related to this removal / treatment been included in the plans?		
Pile Structures		(Y/N/X)	Notes:
11	Are there piles on the project? If no, go to Question 17	N	
12	Has an appropriate pile type been selected?		
	Check the type selected:		
	H-pile (driven)		
	H-pile (prebored)		
	Cast In-place Reinforced Concrete Pipe		
	Micropile		
	Continuous Flight Auger (CFA)		
	other (describe other types)		
13	Have the estimated pile length or tip elevation and section (diameter) based on either the Ultimate Bearing Value (UBV) or the depth to top of bedrock been specified? Indicate method used.		
14	If scour is predicted, has pile resistance in the scour zone been neglected?		
15	Has a wave equation drivability analysis been performed as per BDM 305.4.1.2 to determine whether the pile can be driven to either the UBV, the pile tip elevation, or refusal on bedrock without overstressing the pile?		
16	If required for design, have sufficient soil parameters been provided and calculations performed to evaluate the:		
a.	Nominal unit tip resistance and maximum settlement of the piles?		
b.	Nominal unit side resistance for each contributing soil layer and maximum deflection of the piles?		
c.	Downdrag load on piles driven through new embankment or compressible soil layers, as per BDM 305.4.2.2?		
d.	Potential for and impact of lateral squeeze from soft foundation soils?		

IV.A Foundations of Structures Checklist

Pile Structures		(Y/N/X)	Notes:
17	If piles are to be driven to strong bedrock (Q_u >7.5 ksi) or through very dense granular soils or overburden containing boulders, have “pile points” been recommended in order to protect the tips of the steel piling, as per BDM 305.4.5.6?		
18	If subsurface obstacles exist, has preboring been recommended to avoid these obstructions?		
19	If piles will be driven through 15 feet or more of new embankment, has preboring been specified as per BDM 305.4.5.7?		

IV.A Foundations of Structures Checklist

Drilled Shafts		(Y/N/X)	Notes:
20	Are there drilled shafts on the project? If no, go to the next checklist.	Y	
21	Have the drilled shaft diameter and embedment length been specified?	N	To be determined by others.
22	Have the recommended drilled shaft diameter and embedment been developed based on the nominal unit side resistance and nominal unit tip resistance for vertical loading situations?	N	To be determined by others.
23	For shafts undergoing lateral loading, have the following been determined:	N	To be determined by others.
a.	total factored lateral shear?		
b.	total factored bending moment?		
c.	maximum deflection?		
d.	reinforcement design?		
24	If a bedrock socket is required, has a minimum rock socket length equal to 1.5 times the rock socket diameter been used, as per BDM 305.5.2?	X	To be determined by others.
25	Generally, bedrock sockets are 6" smaller in diameter than the soil embedment section of the drilled shaft. Has this factor been accounted for in the drilled shaft design?	X	To be determined by others.
26	If scour is predicted, has shaft resistance in the scour zone been neglected?	✓	No scour anticipated based on analyses performed by others.
27	Has the site been assessed for groundwater influence?	Y	
a.	If yes, and if artesian flow is a potential concern, does the design address control of groundwater flow during construction?	X	
28	Have all the proper items been included in the plans for integrity testing?	X	Plans being developed by others.
29	If special construction features (e.g., slurry, casing, load tests) are required, have all the proper items been included in the plans?	X	Plans being developed by others.
30	If necessary, have wet construction methods been specified?	X	
General		(Y/N/X)	Notes:
31	Has the need for load testing of the foundations been evaluated?	X	
a.	If needed, have details and plan notes for load testing been included in the plans?	X	

VI.B. Geotechnical Reports

C-R-S: CUY-14-12.12E	**PID:** 13184	**Stage 2** NDA	**Date:** 6/24/2022

General		(Y/N/X)	Notes:
1	Has an electronic copy of all geotechnical submissions been provided to the District Geotechnical Engineer (DGE)?	Y	
2	Has the first complete version of a geotechnical report being submitted been labeled as ‘Draft’?	Y	
3	Subsequent to ODOT’s review and approval, has the complete version of the revised geotechnical report being submitted been labeled ‘Final’?	Y	
4	Has the boring data been submitted in a native format that is DIGGS (Data Interchange for Geotechnical and Geoenvironmental) compatable? gINT files may be used for this.	X	
5	Does the report cover format follow ODOT's Brand and Identity Guidelines Report Standards found at http://www.dot.state.oh.us/brand/Pages/default.aspx ?	Y	
6	Have all geotechnical reports being submitted been titled correctly as prescribed in Section 705.1 of the SGE?	Y	
Report Body		(Y/N/X)	Notes:
7	Do all geotechnical reports being submitted contain the following:	Y	
a.	an Executive Summary as described in Section 705.2 of the SGE?	Y	
b.	an Introduction as described in Section 705.3 of the SGE?	Y	
c.	a section titled "Geology and Observations of the Project," as described in Section 705.4 of the SGE?	Y	
d.	a section titled "Exploration," as described in Section 705.5 of the SGE?	Y	
e.	a section titled "Findings," as described in Section 705.6 of the SGE?	Y	
f.	a section titled "Analyses and Recommendations," as described in Section 705.7 of the SGE?	Y	
Appendices		(Y/N/X)	Notes:
8	Do all geotechnical reports being submitted contain all applicable Appendices as described in Section 705.8 of the SGE?	Y	
9	Do the Appendices present a site Boring Plan showing all boring locations as described in Section 705.8.1 of the SGE?	Y	

VI.B. Geotechnical Reports

Appendices		(Y/N/X)	Notes:
10	Do the Appendices include boring logs and color pictures of rock, if applicable, as described in Section 705.8.2 of the SGE?	Y	
11	Do the Appendices include reports of undisturbed test data as described in Section 705.8.3 of the SGE?	Y	
12	Do the Appendices include calculations in a logical format to support recommendations as described in Section 705.8.4 of the SGE?	Y	