Structure Foundation Exploration Report - Final
COL- Market Street Bridge Replacement (PID 114501)
Lisbon, Columbiana County, Ohio
S&ME Project No. 216853A

PREPARED FOR:

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November 30, 2023

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Arcadis
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Cincinnati, Ohio 45246

Attention: Mr. Steven Butler, P.E.

Reference: **Structure Foundation Exploration Report - Final**
COL-Market Street Bridge Replacement (PID 114501)
Lisbon, Columbiana County, Ohio
S&ME Project No. 216853A

Dear Mr. Butler:

In accordance with our revised proposal dated October 15, 2021, which was authorized on February 24, 2022, by IBI Group, Inc., S&ME, Inc. (S&ME) has completed a Structure Foundation Exploration for the proposed replacement of the structure carrying South Market Street over Little Beaver Creek in Lisbon, Columbiana County, Ohio. The approximate location of this project is shown on the Vicinity Map submitted as Plate 1 in Appendix I of this report. We note that IBI Group, Inc. was acquired by Arcadis in September 2022. Accordingly, Arcadis will be used for all client references in this report.

In accordance with Section 701 of the ODOT *Specifications for Geotechnical Explorations* (*SGE*), S&ME submitted a "draft" version of this report dated September 12, 2023, which was provided to the ODOT District 11 Geotechnical Engineer. On November 30, 2023, Arcadis sent S&ME review comments prepared by ODOT District 11 and dated November 17, 2023. S&ME has addressed these comments and prepared this final version of the report. In addition to this report, a final set of Geotechnical Profile – Structure sheets is being 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 this report.

Respectfully,

S&ME, Inc.

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1.0 Executive Summary

The existing two-span bridge carrying South Market Street over Little Beaver Creek is to be replaced with a single-span structure supported on an extended foundations system composed of drilled shafts. The new bridge will be approximately 15 feet shorter than the existing structure with a span length of 135 feet (center to center of bearings), with the proposed abutments shifted approximately 20 feet east and 7 feet in front of the existing abutments. The Market Street roadway will be realigned to the east and tie into the existing roadway alignment approximately 265 feet to the south and 210 feet north of the respective bridge abutments.

Two (2) structure borings, Borings B-001-0-21 and B-002-0-21 were drilled between April 4 through April 5, 2022, to depths of 37 feet and 36.5 feet respectively. Two (2) auger probes X-001-1-21, and X-002-1-21 were drilled during the same period to a depth of 2 feet each to determine the pavement section thickness approximately 50 feet behind the borings. In each of these borings or probes, 6 to 7 inches of asphalt over 6 to 11 inches of granular base were encountered.

Both structure borings encountered existing fill materials to a depth of 23 feet which consisted of stiff to hard SILT AND CLAY (A-6a), SILTY CLAY (A-6b) and ELASTIC CLAY (A-7-5) with a layer of loose GRAVEL WITH SAND (A-1-b) at the base of the fill in Boring B-001-0-21. The elastic clay in Boring B-002-0-21 was moderately organic with an organic content of 6.6% and an oven-dried to air dried liquid limit ratio of 75.5%. Non-soil materials such as brick, tile, coal, and wood fragments were encountered throughout the layers of existing fill.

Below the fill, Boring B-001-0-21 encountered a layer of very-stiff to hard SILT (A-4b) over highly weathered shale bedrock at a depth of 25 feet (El. 919.2). Below the fill in Boring B-002-0-21, Relatively thin layers of soft to medium-stiff SILT AND CLAY (A-6a) and medium-dense GRAVEL WITH SAND (A-1-b) were encountered over highly weathered sandstone bedrock, also at a depth of 25 feet (El. 919.1). The bedrock consists predominantly of slightly to moderately weathered, strong brown and gray SANDSTONE. A few inches of SHALE were encountered at the bottom of the core in Boring B-001, beginning at a depth of 36.7 feet (El. 907.5). Groundwater was initially noted at depths of 22 and 23.5 feet in Borings B-001 and B-002, respectively.

It is understood that extended foundations consisting of drilled shafts will be used to support the new structure. The sandstone bedrock at this site is <u>not</u> scour resistant as defined by the ODOT *Bridge Design Manual* (*BDM*). See Sections 5.5 and 6.2 for additional information and discussion regarding bedrock scour susceptibility.

The seismic site classification for the site is estimated to be Seismic Site Class D.

Based on the results of borings, S&ME recommends that drilled shafts used to support the proposed rear and forward abutments and be designed using the either, but not both, the factored end bearing or side resistance values presented in Tables 6-1 and 6-2 in Section 6.4.1 of this report.

Settlement of foundations designed and constructed in accordance with the ODOT *Bridge Design Manual*, the ODOT *Construction and Material Specifications*, and the recommendations in this report should be limited to the elastic compression of the drilled shafts.

Drilled shaft construction recommendations, field testing requirements, and plan notes are presented in Sections 6.4.3, 6.4.4, and 6.4.5, respectively, of this report.

2.0 Introduction

It is proposed to replace the existing approximate 150-foot-long, 2-span bridge which carries South Market Street over Little Beaver Creek in Lisbon, Columbiana County, Ohio. The approximate location of the project site is shown on the Vicinity Map included as Plate 1 of Appendix I. The replacement bridge will be a single-span prestressed concrete I-beam structure supported on an extended foundations system composed of drilled shafts. The new bridge will be approximately 15 feet shorter than the existing structure with a span length of 135 feet (center to center of bearings), with the proposed abutments situated slightly east and in front of the existing abutments.

Market Street will be realigned and shifted approximately 20 feet to the east with the realigned roadway tying into the existing roadway at approximately 265 feet to the south and 210 feet north of the respective bridge abutments. Based on the Stage 2 status set plan information provided by Arcadis, only minimal adjustments to the vertical profile of realigned roadway are anticipated.

Arcadis requested that this Geotechnical Exploration be performed in general accordance with the July 2021 update of the ODOT *Specifications for Geotechnical Investigations* (*SGE*). Where possible, S&ME has incorporated more recent updates of the *SGE* and other publications, such as the *Bridge Design Manual* (*BDM*) and *Geotechnical Design Manual* (*GDM*).

3.0 Geology and Site Reconnaissance

3.1 Geology of the Site

Geologic resources indicate that this site is within the Killbuck-Glaciated Pittsburgh Plateau Physiographic Region of Ohio, which commonly contains Wisconsinan-age clay to loam till over Mississippian and Pennsylvanian-age shales, sandstones, conglomerates, and coals. Based on nearby historic borings performed by S&ME, a review of the Ohio Department of Natural Resources (ODNR) geologic survey, and available water well records, the natural soil overburden at the site is anticipated to consist of predominantly sand and gravel deposits with discontinuous layers of silty/clayey soils above bedrock. Bedrock units consist of Pennsylvanian-aged Allegheny and Pottsville Groups, the uppermost bedrock may be encountered between El. 908 to El. 926, or between approximately 15 to 35 feet below the existing grade of the bridge abutments (approximately 5 to 20 feet below the creek bed). Bedrock was encountered at approximately 25 feet below the ground surface (~El. 919) in both borings.

A review of the ODNR "Ohio Karst Areas" map reveals that this site is in an area not known to contain karst features. A review of the ODNR "Landslides in Ohio" map reveals that this portion of Columbiana County lies in an area susceptible to landslides. A review of the ODNR "Abandoned Underground Mines of Ohio" map reveals that this site lies close to an abandoned underground clay mine operated by Saratoga Fire Clay Company till 1942 but does not overlap with the project site.

3.2 Site Reconnaissance

S&ME visited this site on October 8, 2021, to observe the existing site, and visited again on March 4, 2022, to field mark the borings, and to note the locations of the existing underground utilities in the immediate vicinity of this project. Evidence of deterioration of the bridge structure (spalling, broken concrete, beam section loss, etc.) was noted and the presence of rock outcrops in the creek bed, beneath the creek flow, was observed. Additionally, a significant collection of trees and other debris were observed in the creek wedged up against the center pier of the existing bridge.

3.3 Historic Information

S&ME searched the online ODOT Transportation Information Mapping System (TIMS) records for historic boring information for the existing bridge; however, no available historic boring records were located for this site.

4.0 Exploration

4.1 Field Investigation

During the period of April 4 and 5, 2022, two (2) borings were drilled for this project. Structure Borings B-001-0-21 and B-002-0-21, hereinafter referred to as B-001 and B-002, were drilled to depths of 37.0 feet and 36.5 feet respectively. Both borings were completed in the southbound lane, with B-001 performed near the rear abutment and B-002 performed near the forward abutment. Auger probes X-001-1-21 and X-002-1-21, hereinafter referred to as X-001-1 and X-002-1, were drilled to depths of 2 feet approximately 50 feet behind the borings to determine the pavement section thickness at near the southern and northern ends of paving for this project. The boring locations were surveyed by Arcadis following completion of the drilling program. The approximate locations of the borings and probes are shown on the Plan of Borings included as Plate 2 in Appendix I.

Borings B-001, B-002, X-001-1, and X-002-1 were performed using an ATV-mounted drill rig using a 3¼-inch I.D. hollow-stem auger to advance the borings through the pavement and soil overburden. In the structure borings, disturbed but representative soil samples were obtained by lowering a 2-inch O.D. split-barrel sampler through the auger stem to the bottom of the boring and then driving the sampler into the soil at 2.5-foot intervals to a depth of 23.5 feet and continuously below 23.5 feet with blows from a 140-pound hammer freely falling 30 inches (ASTM D1586 - Standard Penetration Test, SPT). All soil samples were examined in the field and representative portions were preserved in airtight glass jars. In accordance with the current ODOT *SGE*, the hammer system on the drill rig had been calibrated in accordance with ASTM D4633 to determine the drill rod energy ratio (98.6%). The energy ratio, as used to determine corrected blow counts, has been limited to 90% as required by the SGE. Upon encountering bedrock, a changeover to rock coring techniques was made and 10.0 feet of bedrock was cored using an NQ-sized diamond tipped rock core barrel. Recovered rock cores were placed in wax impregnated and partitioned cardboard boxes. Following the completion of drilling, the existing pavement thickness was measured, backfilled with cuttings mixed with bentonite, and then the existing pavement was patched with an equivalent thickness of cold patch asphalt.

In the field, experienced personnel performed the following duties: 1) examined and preserved recovered soil samples; 2) prepared a log of each boring; 3) recorded seepage and groundwater observations and measurements; 4) obtained hand penetrometer measurements in soil samples exhibiting cohesion; and, 5) provided liaison between the fieldwork and the project manager so that any modifications to the exploration program could be expeditiously implemented in the event that unusual or unanticipated conditions were encountered. All recovered samples were transported to a soil laboratory of S&ME for further examination and testing.

4.2 Laboratory Testing

In the laboratory, the recovered samples were visually identified, and natural moisture content tests were performed on all recovered samples in accordance with ODOT specifications. Liquid and plastic limit determinations and grain-size analyses were also performed on selected representative specimens. The results of these classification tests permit an evaluation of the strength and compressibility characteristics of the soils encountered at this site by comparison with similar soils for which these characteristics have been previously determined. The results of all laboratory tests are recorded numerically on individual boring logs. Grain size testing was performed on all samples recovered from the continuously sampled scour zone in the structure borings. Strength and durability testing, which is required by the ODOT *Bridge Design Manual* (*BDM*), of the bedrock from each boring was performed to determine if the bedrock is scour resistant.

Based upon the results of the laboratory testing program, the field logs were modified, if necessary, and copies of the laboratory corrected boring logs are submitted as Plates 5 through 8 of Appendix I. Shown on these logs are: descriptions of the soil stratigraphy encountered; depths from which samples were preserved; sampling efforts (blow-counts) required to obtain the specimens in the borings; calculated N_{60} values; laboratory testing results; seepage and groundwater observations made at the time of drilling; and, values of hand-penetrometer measurements made in soil samples exhibiting cohesion. For your reference, hand-penetrometer values are roughly equivalent to the unconfined compressive strength of the cohesive fraction of the soil sample.

Soils have been classified in general accordance with Section 603 of the ODOT *SGE* and described in general in accordance with Section 602. Bedrock has been classified and described in general accordance with Section 605 of the ODOT *SGE*. An explanation of the symbols and terms used on the boring logs, definitions of the special adjectives used to denote the minor soil components, description of rock, and information pertaining to sampling and identification are presented on Plate 3 and 4 of Appendix I. Group Indices determined from the results of the laboratory testing program are also provided on the boring logs.

5.0 Findings

5.1 General Subsurface Conditions

The thicknesses of existing pavement encountered in the borings are summarized in Table 5-1. Beneath the asphalt pavement, the granular base course material was noted in every boring and auger probe.

Table 5-1 – Summary of Existing Pavement Section Materials

Exploration ID	Station	Offset	Asphalt (in.)	Granular base (in.)
B-001-0-21	9+04	14’ LT	7	8
X-001-1-21	8+50	3’ LT	7	10
B-002-0-21	11+03	14’ LT	6.5	7.5
X-002-1-21	11+50	CL	6	6

5.2 Fill Materials

Below the pavement section, both structure borings encountered 23.0 feet of fill materials consisting of stiff to hard brown, gray or black SILT AND CLAY (A-6a), SILTY CLAY (A-6b) and ELASTIC CLAY (A-7-5) with a layer of loose GRAVEL WITH SAND (A-1-b) at the base of the fill in Boring B-001. The 2.5-foot-thick layer of Elastic Clay in Boring B-002 was moderately organic with an organic content of 6.6% and an oven-dried to air dried liquid limit ratio of 75.5%. Non-soil materials such as brick, tile, coal, and wood fragments were encountered throughout the layers of fill.

5.3 Natural Materials

Below the fill, Boring B-001 encountered a 2-foot-thick layer of very-stiff to hard dark gray SILT (A-b) at a depth of 23 feet and immediately above the over highly weathered shale bedrock. Also at a depth of 23 feet, Boring B-002 encountered roughly 1-foot layers of soft to medium-stiff gray SILT AND CLAY (A-6a) and medium-dense gray GRAVEL WITH SAND (A-1-b) between the fill and underlying bedrock.

5.4 Bedrock

Bedrock was initially encountered at a depth of 25 feet in both borings. Boring B-001 encountered roughly 2 feet of highly weathered and weak gray SHALE, whereas Boring B-002 encountered approximately 1.5 feet of SANDSTONE described as being highly weathered and weak to slightly strong.

Bedrock coring commenced at a depth of 27.0 feet in Boring B-001 and at 26.5 feet in Boring B-002. The recovered bedrock was composed predominantly of slightly to moderately weathered, moderately strong to strong, gray and dark-gray SANDSTONE. RQD measurements ranged from 13% to 79% and core recovery ranged from 94% to 100%. Unconfined compressive strength test results ranged from 8,921 to 9,048 psi. Slake durability test results ranged from an index value of 97.9% to 98.3%. A few inches of SHALE were encountered at the bottom of the core in Boring B-001, beginning at a depth of 36.7 feet to the termination of the coring at 37.0 feet.

5.5 Scour Data

Grain-size data was originally provided to Arcadis on June 9, 2022, to assist others in determining the scour of soils below the creek bed and above the top of rock. In accordance with more recent guidance in Section 1302 of the ODOT *Geotechnical Design Manual* (*GDM*), Table 5-2 provides the critical shear stress, D_{50} grain size, and the Erosion Category for the soils encountered in the borings.

Table 5-2 – Sample Scour Data

Boring Number	Sample ID	Sample Elevation	Lab D50 (mm)	τ_c (psf)	$D_{50,\ equivalent}$ (mm)	Erosion Category (EC)
B-001-0-21	SS-6	930.7' - 929.2'	0.0831	0.2643	12.6524	3.41
	SS-8	925.7' - 924.2'	0.007	0.1759	8.4238	3.34
	SS-9	923.2' - 921.7'	0.5066	0.0106	0.5066	1.85
	SS-10	920.7' - 919.2'	0.0172	0.3868	18.5206	2.97
B-002-0-21	SS-6	930.6' - 929.1'	0.2683	0.1075	5.1492	3.17
	SS-8	925.6' - 924.1'	0.0433	0.0601	2.8793	3.48
	SS-9	923.1' - 921.6'	0.0455	0.0279	1.3342	3.67
	SS-10A	920.6' - 920.1'	0.1728	0.0253	1.2130	3.26
	SS-10B	920.1' - 919.1'	3.0568	0.0638	3.0568	2.78

In addition to the soil information provided above, the following parameters may be used to determine scour within the upper sandstone bedrock beneath the creek bed.

- Average Unconfined Compressive Strength (Q_u) = 8,985 psi
- Average Slake Durability Index (SDI) = 98.1%
- Average RQD = 21%
- Average Total Unit Weight = 140.8 pcf
- Range of Rock Mass Rating (RMR) = 42 to 68
- Range of Geologic Strength Index (GSI) = 20 to 65
- Erodibility Index (K) = 214.3 (based on the following coefficients)
  - J_n = 3.34; J_r = 1.5; J_a = 3.0; J_s = 1.1
- Critical Shear Stress = 3,705 Pa (calculated using Equations 7.38 and 7.39 in HEC-18).

Based on the properties of the bedrock summarized above, the bedrock at the site is not considered to be scour-resistant bedrock as defined in Section 305.2.1.2.b.B of the 2020 ODOT *Bridge Design Manual* (*BDM*). As such, if the anticipated scour depth using the soil criteria extends to the top of bedrock, then a scour analysis of the bedrock should be performed to determine the magnitude of expected bedrock scour.

5.6 Groundwater Observations

During drilling, groundwater was encountered at the depths of 22 and 23.5 feet in Borings B-001 and B-002, respectively. Water levels at the completion of the drilling were affected by the introduction of water during bedrock coring, and thus are not indicative of actual groundwater levels. Groundwater measurements should be considered 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.

6.0 Analyses and Recommendations

6.1 General Project Discussion

The existing two-span bridge carrying Market Street over Little Beaver Creek is to be replaced with a single-span structure with a span length of approximately 135 feet. The bridge will be supported on deep foundations consisting of drilled shafts socketed into bedrock at each abutment. In addition to the structure replacement, Market Street will be realigned and shifted approximately 20 feet to the east with the realigned roadway tying into the existing roadway at approximately 265 feet to the south and 210 feet north of the respective bridge abutments, requiring widening of the existing roadway embankment. Based on Stage 2 status set plan information provided by Arcadis, only minimal adjustments to the vertical profile of realigned roadway are anticipated.

6.2 Results of Scour Analysis

Using the scour zone information previously provided, Arcadis performed scour analyses for the proposed structure. On August 24, 2023, Arcadis informed S&ME that no bedrock scour was estimated to occur at the structure location.

We note that according to the Stage 2 status plan set provided on August 24, 2023, that scour protection consisting of a 2-foot-thick layer of Rock Channel Protection is proposed to be placed on the spill-through slopes in front of the abutments. It must be recognized that riprap is not a permanent countermeasure against, nor does it eliminate the potential for scour. For this reason, if riprap is used, we recommended that provisions for routine maintenance of the rip-rap blanket to ensure that the design blanket thickness is preserved over the design life of the structure. Additionally, in all cases where riprap is used for scour protection, the bridge and rip rap blanket must be monitored during and inspected after periods of high flow.

6.3 Earthen Approach Embankments

Based on the available information provided by Arcadis, we understand that only minor changes to the vertical profile (less than 2 feet) of the roadway at the proposed centerline of the roadway are anticipated. However, due to the shift in the alignment of Market Street to the east, new embankment fill will be required to construct the widened (eastern) portion of the new approach embankments. The realigned embankments will require placement of new fill ranging from small sidehill sliver fills to new fill embankment approximately 8 feet high and 40 feet wide at the base. The new fill will predominantly be placed to the right of centerline between approximately Sta. 8+50 to Sta. 9+26 (end of approach slab at rear abutment). A wedge of fill will also be required to the left of centerline from approximately Sta. 10+65 (beginning of approach slab at the forward abutment) to Sta. 10+75.

As such, the following sections present recommendations regarding embankment foundation preparation and construction for the realigned portion of Market Street and the modified approach embankments.

6.3.1 Embankment Preparation

Prior to commencing embankment widening, it is recommended that all unsuitable materials should be completely removed from the sides of the existing embankment and from the entire footprint of proposed embankment widening area. Unsuitable materials include sod, topsoil, trees, vegetation, salt, soft and/or organic soils, undocumented existing fill soils, miscellaneous debris, existing foundations and floor slabs, existing pavements and associated granular base materials, and any other materials judged unsuitable by the Geotechnical Engineer.

Once these materials have been removed, it is recommended that the entire exposed embankment foundation surface be examined by the Geotechnical Engineer of Record or their designated representative to identify any weak, wet, organic, or otherwise unsuitable soils that were not encountered during the geotechnical exploration. Any such materials identified should be removed and replaced with suitable compacted fill (ODOT *Construction and Materials Specifications* (*CMS*), Item 203, or Item 204 when within 12 inches of the proposed subgrade). If weak, wet, or soft zones are present, it is recommended that the materials contained in these zones should be either scarified, moisture conditioned, and thoroughly recompacted in place or be removed and the over-excavation filled in a controlled manner with compacted, suitable embankment material prior to attempting to place and compact any new fill.

6.3.2 *Benching*

After all unsuitable materials have been removed and prior to commencing fill placement, it is recommended that horizontal benches be cut into all existing sloping surfaces steeper than 8(H):1(V) to permit placement and compaction of new fill in horizontal lifts. At locations where the existing ground surface is steeper than 4(H):1(V), S&ME recommends "Special Benching" procedures as outlined in the Section 800 of the ODOT *Geotechnical Design Manual* (*GDM*) and the ODOT *Construction Inspection Manual of Procedures* (*CIMP*) should be performed. Additionally, in accordance with Section 809 of the *GDM*, wherever "Special Benching" is used, Plan Note G109 from the ODOT *L&D Manual, Vol. 3*, should be included in the General Notes.

During any required Special Benching procedures, S&ME also recommends the following: 1) only one bench be exposed at any given time and that excavation of the next bench should not be permitted until embankment fill placement and compaction has been completed to within 1 to 2 feet of the top of the backslope of the previous bench; and, 2) the length of any given bench that is exposed should not exceed the quantity of embankment fill which may be properly placed and compacted in one day. Additionally, S&ME recommends that the final, completed side slopes of embankments be constructed no steeper than 2(H):1(V).

6.3.3 *Borrow Requirements and Compaction Criteria*

New fill should consist of inorganic soil free of all miscellaneous materials, cobbles, and boulders, placed in uniform, thin layers and then compacted in accordance with either Item 203, or when within 12 inches of the proposed subgrade level, Item 204 of the ODOT *CMS*. Borrow materials should not be placed in a frozen condition or upon a frozen surface, and any sloping surfaces on which new fill is to be placed should first be benched in accordance with either Item 203.05 or Section 800 and 803 of the *GDM*, depending on the slope of the existing ground surface at each location. Also, borrow materials to be used as new fill or backfill within 3 feet of the proposed subgrade level be tested in the laboratory to determine that the borrow materials exhibit subgrade support characteristics that are no less than the CBR value used during design of the new approach embankment pavement.

Compaction requirements for the construction of earthen embankments are based on ODOT *CMS* Item 203.07.B (or ODOT *CMS* Item 204.03 when within 12 inches of subgrade level), which specifies a minimum percent compaction based on the dry unit weight of the type of soil fill being placed as borrow. At the time of this submittal, it is unknown if a borrow source will be required for this project. S&ME recommends that, if a borrow site is required, that sampling and testing of this borrow material be performed prior to construction to verify that the borrow soils are suitable for the planned construction.

6.3.4 *Compaction/Moisture Conditioning Concerns*

Exposed soil surfaces should be protected from exposure to water prior to regrading, benching, and new fill placement. Exposure of cohesive soils to water will result in a decrease in soil strength and an increase in compressibility and should be prevented. Seepage or surface runoff should not be permitted to collect and stand on exposed soil surfaces. Soils loosened/softened by standing water and/or by construction activities should be moisture conditioned (if feasible) or removed from the embankment or subgrade prior to the placement of additional embankment material or roadway base. The areas around the proposed construction should be graded such that all water runoff is directed away from the new site improvements during and upon completion of construction.

6.3.5 *Subgrade Preparation*

Once the design subgrade elevation has been attained for the realigned bridge approach embankments, the subgrade should be compacted and proof rolled in accordance with ODOT *CMS* Item 204, with all weak or unsuitable areas being repaired in accordance with ODOT *CMS* Item 204.07. Proof rolling should not be performed where the operations may potentially damage structures, utilities, etc.

6.3.6 *Groundwater Considerations for Roadway Construction*

Based upon observations made at the time of this investigation, significant groundwater problems are not anticipated for any proposed approach embankment widening. See Section 6.3.4 for recommendations regarding moisture conditions and concerns during construction.

The presence of water bearing granular layers or seams in the walls of any utility excavations may result in caving or sloughing of the excavation walls. S&ME recommends that all excavations be braced, or sloped back at a safe angle, in accordance with current Occupational Safety and Health Administration (OSHA) Excavation Regulations.

6.4 Bridge Foundation Recommendations – Drilled Shafts

Arcadis has indicated that drilled shafts are planned to support both abutments of the new bridge structure. These drilled shafts should be designed in accordance with Section 305.4 of the 2022 ODOT *BDM*, with shaft and rock socket diameters determined in accordance with Section 305.4.4.2. Boring B-001 encountered weathered shale bedrock at El. 919.2 and weathered sandstone bedrock in Boring B-002 at El. 919.1. Bedrock coring began at El. 917.2 in B-001 and at El. 917.6 in B-002.

The maximum axial load per shaft anticipated at the abutments by Arcadis is 494 kips. The Stage 2 status plan set provided to S&ME also indicates that six (6) 36-inch diameter drilled shafts with 30-inch diameter rock sockets, at approximate 11- to 12-foot center-to-center spacing are planned to support each abutment and associated wingwall.

6.4.1 *Axial Shaft Resistance*

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.

The length of the drilled shafts should be of sufficient length to resist the applied axial and lateral loading and also satisfy the shaft length requirements for non-scour resistant bedrock addressed in Section 305.4.1.1 of the ODOT *BDM*. LRFD resistance values recommended for use during drilled shaft design at the bridge abutments are presented in Tables 6-1 (End Bearing only) and Table 6-2 (Side Resistance only). See Appendix II for computations of these recommended values.

Table 6-1 – Recommended Nominal and Factored Unit End Bearing Resistance Values for Drilled Shafts Socketed into Bedrock (Strength Limit State)

Substructure Element	Rock Type	Nominal Unit Tip Resistance* (q_p)	Resistance Factor (φ_{qp}) for Tip Resistance**	Factored Unit Tip Resistance*
Forward and Rear Abutment Drilled Shafts	Lower Sandstone (El. 912.6/915.4 to El. 907)^	1,440 ksf	0.5	720 ksf

* For vertical loading only.
** Table 10.5.5.2.4-1 of the AASHTO LRFD.
^ Upper sandstone extends from the top of the bedrock down to approximately El. 912.6 at the rear abutment and El. 915.4 at the forward abutment.

Table 6-2 – Recommended Nominal and Factored Unit Side Resistance Values for Drilled Shafts Socketed into Bedrock (Strength Limit State)

Substructure Element	Rock Type	Nominal (Unfactored) Unit Shaft Resistance (q_s)*	Resistance Factor (φ_{qs}) for Shaft Resistance**	Factored Unit Shaft Resistance*
Forward and Rear Abutment Drilled Shafts	Upper Sandstone (El. 917.2/919.1 to 912.6/915.4)^	20 ksf	0.55	11 ksf
	Lower Sandstone (El. 912.6/915.4 to El. 907)^	30 ksf	0.55	16.5 ksf

* For vertical loading only.
** Table 10.5.5.2.4-1 of the AASHTO LRFD (side resistance in rock)
^ Upper sandstone extends from the top of the bedrock down to approximately El. 912.6 at the rear abutment and El. 915.4 at the forward abutment.

6.4.2 *Lateral Shaft Resistance – LPILE Parameters*

At the request of Arcadis, S&ME has provided recommended parameters to be used by others to perform a lateral load (LPILE) analysis on the shafts at each abutment. Tables 6-3 and 6-4 includes recommended p-y models, rock unit weights, and the unconfined compressive strength to be used in lateral load analyses for the retaining wall structures. These parameters are based on the bedrock and lab data shown on the boring logs, and recommended values given in the LPile 2019 user's manual and guidance provided by ODOT Office of Geotechnical Engineering (OGE).

Table 6-3 – LPile 2019 Input Parameters at Rear Abutment (Boring B-001)

Stratum	Depth Interval (ft.)	Elevation Range	p-y Soil Model	Effective Unit Weight	Subgrade Modulus, k	ϕ' (deg) / C (psf)	Strain ε_{50}
Silt and Clay (A-6a)	0 – 20.5	944.0 – 923.5	Stiff Clay w/o Free Water	120 pcf	1000 pci	3,000	0.005
Gravel w/ Sand (A-1b)	20.5 – 23.0	923.5 – 921.0	Reese Sand	53 pcf	20 pci	30°	---
Silt (A-4b)	23.0 – 25.0	921.0 – 919.0	Stiff Clay w/o Free Water	58 pcf	1000 pci	4,000	0.005
Weathered Shale	25.0 – 27.0	919.0 – 917.0	Reese Sand	90 pcf	125 pci	36°	---

Rock Type	Depth Interval (ft.)	Elevation Range	p-y Rock Model	Effective Unit Weight	Unconfined Compressive Strength	Hoek-Brown Material Index	Poisson's Ratio	GSI	Rock Mass Modulus (psi)
Sandstone	27.0 – 31.6	917.0 – 912.4	Massive Rock	80 pcf	9,048 psi	17	0.20	45	814,300
Sandstone	31.6 – 36.7	912.4 – 907.3	Massive Rock	80 pcf	7,500 psi	17	0.20	65	675,000

Table 6-4 – LPile 2019 Input Parameters at Forward Abutment (Boring B-002)

Stratum	Depth Interval (ft.)	Elevation Range	p-y Soil Model	Effective Unit Weight	Subgrade Modulus, k	ϕ' (deg) / C (psf)	Strain ε_{50}
Silt and Clay (A-6a)	0 – 15.5	945.0 – 929.5	Stiff Clay w/o Free Water	120 pcf	1000 pci	3,000	0.005
Silty Clay (A-6b)	15.5 – 20.5	929.5 – 924.5	Stiff Clay w/o Free Water	120 pcf	500 pci	1,500	0.007
Elastic Clay (A-7-5)/ Silt and Clay (A-6a)	20.5 – 24.0	924.5 – 921.0	Stiff Clay w/o Free Water	120 pcf	500 pci	1,000	0.007
Gravel with Sand (A-1-b)	24.0 – 25.0	921.0 – 920.0	Reese Sand	58 pcf	60 pci	33°	---
Weathered Sandstone	25.0 – 26.5	920.0 – 918.5	Reese Sand	70 pcf	125 pci	40°	---

Rock Type	Depth Interval (ft.)	Elevation Range	p-y Rock Model	Effective Unit Weight	Unconfined Compressive Strength	Hoek-Brown Material Index	Poisson's Ratio	GSI	Rock Mass Modulus (psi)
Sandstone	26.5 – 28.7	918.5 – 916.3	Massive Rock	80 pcf	6,000 psi	17	0.20	30	540,000
Sandstone	28.7 – 36.5	916.3 – 908.5	Massive Rock	80 pcf	8,921 psi	17	0.20	50	802,900

6.4.3 *Construction Recommendations*

The drilled shafts should be constructed in accordance with Item 524 of the ODOT *CMS* and should be at a minimum center-to-center spacing of 3 shaft diameters (3D). As both borings encountered a layer of granular soil near the top of bedrock, consideration should be given to providing a temporary casing during drilled shaft excavation, as these materials may cave during drilled shaft construction, resulting in bulging of the drilled shaft above the top of bedrock. To reduce this potential, the casing should extend to the underlying bedrock to attempt to seal the shafts from influx of water, soil, and rock fragments and reduce the potential for "wet" installation techniques being required. 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 particularly as groundwater inflow may be extensive when shafts extend below the level of water in the river; 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 the *Construction Inspection Manual of Procedures* (*CIMP*) 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.

6.4.4 *Field Testing*

Construction and verification of the drilled shafts should be performed in accordance with Item 524 of the ODOT *CMS* and Section 305.4 (including applicable subsections) of the ODOT *BDM*. Based on information from ODOT District 11, field testing of drilled shafts as identified in the *BDM* is not anticipated to be required for the drilled shafts supporting the proposed structure.

6.4.5 *Plan Notes*

The following general notes from Section 600 of the ODOT *BDM* should be included in the project plans.

NOTE 606.8-1: ROCK-SOCKETED DRILLED SHAFTS: The maximum factored load to be supported by each drilled shaft is _*_ kips at the abutments. This load is resisted by tip resistance. At the abutments, the factored tip resistance is _*_ kips.

Note to Designer:

_*_ Complete the loads in this note based on the foundation dimensions utilized and the Factored Unit Tip Resistance value provided in Tables 3 through 6 of this report.

NOTE 606.8-3: LATERALLY LOADED DRILLED SHAFTS: The maximum factored lateral load and bending moment to be supported by each drilled shaft are _*_ kips, and _*_ kip-feet, respectively. These loads produce a maximum factored bending moment of _*_ kip-feet, and a maximum factored shear of _*_ kips, within the drilled shaft.

Note to Designer:

_*_ Complete the loads in this note. If the maximum factored lateral loading of drilled shafts varies between substructure units, specify the drilled shaft groups and locations separately in the note.

6.4.6 *Settlement*

As the drilled shaft foundations supporting the rear and forward abutments will be supported on sandstone bedrock, settlement of the foundations is anticipated to be limited to the elastic compression of the drilled shafts.

6.4.7 *Downdrag Considerations*

Based on the available grading plans, new fill embankment will be placed near both abutments to construct the realigned approach embankments. The placement of this new fill is anticipated to induce settlement on the foundation soils. However, based on the Stage 2 status plans, structural loads are designed to be resisted by end bearing (tip) resistance with a factored resistance of 3,534 kips per shaft whereas the factored load per shaft is 494 kips per shaft. Accordingly, S&ME is of the opinion that any potential downdrag on the proposed drilled shafts will be resisted by the significant excess resistance (over 3,000 kips) available from each shaft.

6.5 Lateral Earth Pressures - Abutments

The proposed abutments must be designed to withstand lateral earth pressures, as well as hydrostatic pressures, which may develop behind the structures. The magnitude of the lateral earth pressures varies based on 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 *CMS* Item 518.03) should be used directly behind the structures for a minimum thickness of 24 inches in accordance with ODOT *CMS* Item 518.05. This granular zone should drain to either weepholes or a pipe drain, 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 using granular backfill, whereas a cohesive (clay) backfill will result in the development of much higher pressures.

To minimize lateral 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 may result in 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 *CMS* Item 518) is provided <u>and</u> a wall movement exceeding 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 cohesive and granular soil, respectively.

6.6 Seismic Site Classification

Based on the subsurface stratigraphy encountered within the borings, it is the opinion of S&ME that this site is best characterized by AASHTO *LRFD* Table 3.10.3.1-1 as seismic site class D. See Plate 30 in Appendix II for these computations.

6.7 Groundwater Considerations

During this exploration, groundwater was encountered in both borings at depths ranging from 22 to 23.5 feet below existing grade (El. 920.6 to El. 922.2). Accordingly, no significant sources of groundwater are anticipated to be encountered during construction above these elevations.

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 Little Beaver 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 river.

It is recommended that groundwater and surface water runoff be controlled during construction, as soil in excavation walls or at the bottom of the excavation 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 in the sides or 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 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 creek and into the underlying bedrock. The use of localized sheeting, however, may not be possible where the toe of the sheeting extends below the top of the underlying bedrock.

Additionally, all excavations should be either sloped back or braced in accordance with the most recent OSHA excavation guidelines (see Section 6.8).

6.8 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 provide 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.

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 that we can modify our recommendations based on this additional information if necessary.

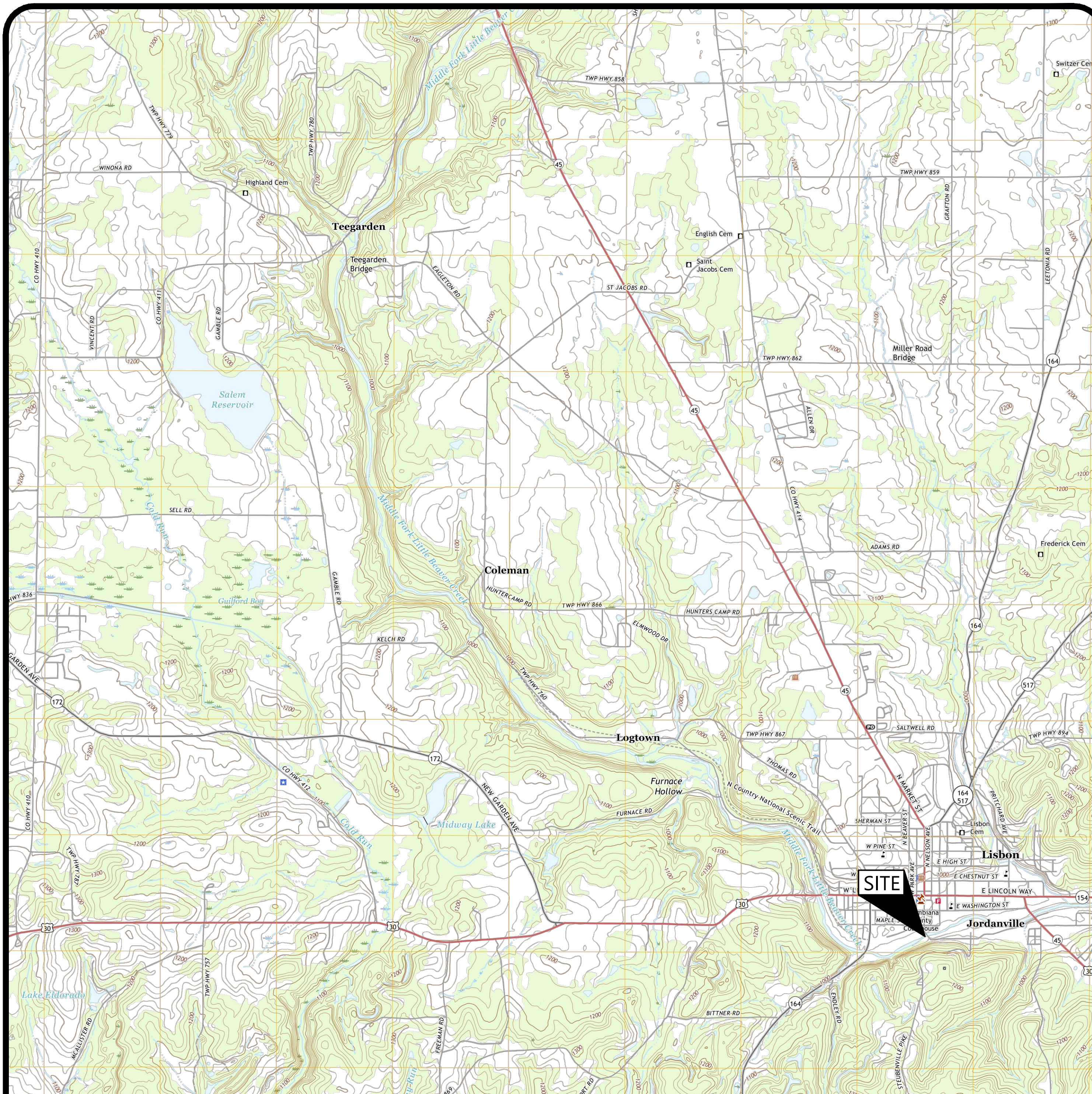
Our conclusions and recommendations are based on limited data from a field exploration program. Subsurface conditions can vary widely between explored areas. Some variations may not become evident until construction. If conditions are encountered that appear different than those described in our report, 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 the 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 – General Project Information



N

USGS Mapping:
Lisbon USGS Quad

0 4000 8000

GRAPHIC SCALE (IN FEET)

Drawing Path: T:\GEO\Projects\2021\216853A_IBI_COL Market St Bridge\CAD\Construction\Plan of Borings & Vicinity Map.dwg

Vicinity Map

Structure Foundation Exploration
COL-Market Street Bridge Replacement
Lisbon, Columbiana County, Ohio

SCALE:	FIGURE NO.
GRAPHIC	1
DATE:	
06-16-2022	
PROJECT NUMBER	
216853A	

NOTE: LOCATIONS X-001-1-21 AND X-002-1-21 ARE PAVEMENT AUGER PROBES ONLY, WITH NO SAMPLING.

LEGEND

X-001-1-21 PAVEMENT AUGER PROBE NUMBER AND LOCATION

B-001-0-21 BORING NUMBER AND LOCATION

0 40 80
GRAPHIC SCALE (IN FEET)

Drawing Path: T:\GEO\Projects\2021\216853A_IBI_COL Market St Bridge\CAD\Construction\Plan of Borings & Vicinity Map.dwg

Plan of Borings	SCALE: GRAPHIC DATE: 06-16-2022	FIGURE NO.
Structure Foundation Exploration COL-Market Street Bridge Replacement Lisbon, Columbiana County, Ohio	PROJECT NUMBER 216853A	**2**

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
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

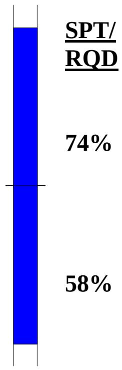
The following terms are used to describe density and consistency of soils:

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 2007 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: 216853A

PROJECT: COL-MARKET ST. BRIDGE	DRILLING FIRM / OPERATOR: OTB / C. SVITAK	DRILL RIG: OTB ATV B-57	STATION / OFFSET: 9+04, 14' LT
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: S&ME / M. KHAN	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. MARKET ST
PID: 114501 BR ID:COL-MARKET-0170	DRILLING METHOD: 3-1/4" HSA / NQ	CALIBRATION DATE: 11/25/20	ELEVATION: 944.2 (MSL) EOB: 37.0 ft.
START: 4/5/22 END: 4/5/22	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 90*	COORD: 40.768865 N, 80.767431 W

EXPLORATION ID **B-001-0-21**

PAGE 1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 944.2	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 - 7 INCHES	943.6																	
GRANULAR BASE - 8 INCHES	942.9	1																
FILL: Very-stiff to hard brown **SILT AND CLAY**, some fine to coarse sand, little fine gravel, few coal fragments, few pockets of organic matter, damp.		2	2 5 6	17	67	SS-1	4.5	-	-	-	-	-	-	-	-	16	A-6a (V)	
		4	2 3 5	12	78	SS-2	2.75	18	10	17	24	31	31	18	13	18	A-6a (5)	
- @ 6.0'; brown mottled with gray.		7	2 5 6	17	67	SS-3	3.0	-	-	-	-	-	-	-	-	22	A-6a (V)	
		9	3 4 5	14	67	SS-4	2.5	-	-	-	-	-	-	-	-	16	A-6a (V)	
	931.2	12	3 4 6	15	100	SS-5	3.75	-	-	-	-	-	-	-	-	21	A-6a (V)	
FILL: Very-stiff to hard gray and brown **SILT AND CLAY**, some fine to coarse sand, little fine gravel, few brick and sandstone fragments, damp.		14	3 3 5	12	78	SS-6	4.5	18	17	16	22	27	36	21	15	17	A-6a (5)	
	926.2	17	4 6 6	18	100	SS-7	3.5	-	-	-	-	-	-	-	-	15	A-6a (V)	
FILL: Stiff to very-stiff dark gray to black **SILT AND CLAY**, little fine to coarse sand, few brick fragments, damp.	923.7	19	2 2 2	6	100	SS-8	2.0	0	2	17	36	45	36	22	14	28	A-6a (10)	
FILL: Loose dark gray to black **GRAVEL WITH SAND**, little clay, trace silt, few coal fragments, wet.	921.2	W 922.2 / 22	4 3 2	8	100	SS-9	-	34	19	30	5	12	-	-	-	17	A-1-b (V)	
Very-stiff to hard dark gray **SILT**, some clay, little fine to coarse sand, little fine to coarse gravel, damp.	919.2	24	2 2 8	15	72	SS-10	4.0	8	2	11	53	26	31	21	10	17	A-4b (8)	
SHALE, gray, highly weathered, weak.		TR 25	18 50	-	100	SS-11	-	-	-	-	-	-	-	-	-	8	Rock (V)	
	917.2	27	50-5"	-	100	SS-12	-	-	-	-	-	-	-	-	-	8	Rock (V)	
SANDSTONE, gray, moderately weathered, fine to medium grained, strong, thick to very thick bedded, fractured to highly fractured, very blocky, good condition; RQD = 24%, REC = 100%. - @ 27.0' to 29.0' SDI = 97.9%.		28–29	29		99	NQ-13											CORE	

PID: 114501	BR ID: COL-MARKET-0170	PROJECT: COL-MARKET ST. BRIDGE	STATION / OFFSET: 9+04, 14' LT	START: 4/5/22	END: 4/5/22	PG 2 OF 2	B-001-0-21

MATERIAL DESCRIPTION AND NOTES	ELEV. 914.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
- @ 31.0' to 31.4' Qu = 9,048 psi.	912.6	31																
SANDSTONE, gray to dark gray, moderately weathered, fine to medium grained, strong, very thick bedded, slightly fractured, blocky, good condition; RQD = 88%, REC = 95%.	907.5	32 – 36	79		94	NQ-14											CORE	
SHALE, gray, moderately to highly weathered, weak, very thinly bedded, arenaceous, highly fractured, disintegrated, poor condition; RQD = 0%, REC = 33%.	907.2	EOB 37																

NOTES:
- Groundwater noted at 22.0' during drilling.
- Water measured at 14.3' upon completion after rock coring.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SOIL CUTTINGS

PROJECT: COL-MARKET ST. BRIDGE	DRILLING FIRM / OPERATOR: OTB / C. SVITAK	DRILL RIG: OTB ATV B-57	STATION / OFFSET: 11+03, 14' LT
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: S&ME / M. KHAN	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. MARKET ST
PID: 114501 BR ID: COL-MARKET-0170	DRILLING METHOD: 3-1/4" HSA / NQ	CALIBRATION DATE: 11/25/20	ELEVATION: 944.1 (MSL) EOB: 36.5 ft.
START: 4/4/22 END: 4/4/22	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 90*	COORD: 40.769279 N, 80.767905 W

EXPLORATION ID B-002-0-21

MATERIAL DESCRIPTION AND NOTES	ELEV. 944.1
ASPHALT - 7 INCHES	943.5
GRANULAR BASE - 11 INCHES	942.6
FILL: Stiff to very-stiff **SILT AND CLAY**, some fine to coarse gravel, some fine to coarse sand, few brick, tile, coal and shale fragments, damp. - @ 11.0' iron staining	928.6
FILL: Stiff gray **SILTY CLAY**, some fine to coarse sand, little fine to coarse gravel, few coal, wood and brick fragments, moist.	923.6
FILL: Stiff black **ELASTIC CLAY**, "and" fine sand, trace coarse sand, trace fine gravel, moderately organic, few wood and bark fragments, few roots, wet. - @ SS-9: LOI = 6.6%; ODLL/LL = 75.5%	921.1
Soft to medium-stiff gray **SILT AND CLAY**, "and" fine to coarse sand, little fine gravel, moist.	920.1
Medium-dense gray **GRAVEL WITH SAND**, little silt, little clay, moist to wet.	919.1
SANDSTONE, light brown, highly weathered, weak to slightly strong.	917.6
SANDSTONE, brown and gray, moderately weathered, medium to coarse grained, moderately strong to strong, thin bedded, highly fractured to fractured, narrow, slightly rough, blocky/disturbed/seamy, fair condition, 1/4" clay seam at 27.2', RQD = 0%, REC = 100%.	915.4

DEPTHS: W 920.6; TR (25)

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
3 2 3	8	67	SS-1	3.75	-	-	-	-	-	-	-	-	17	A-6a (V)
2 3 4	11	44	SS-2	3.0	-	-	-	-	-	-	-	-	16	A-6a (V)
2 2 4	9	67	SS-3	3.25	34	15	10	18	23	37	23	14	16	A-6a (2)
2 3 8	17	67	SS-4	2.5	-	-	-	-	-	-	-	-	17	A-6a (V)
3 4 4	12	100	SS-5	1.5	-	-	-	-	-	-	-	-	13	A-6a (V)
2 2 4	9	56	SS-6	4.0	30	13	20	15	22	32	20	12	17	A-6a (1)
2 2 2	6	100	SS-7	2.0	-	-	-	-	-	-	-	-	23	A-6b (V)
2 2 2	6	100	SS-8	1.0	14	3	26	32	25	39	23	16	32	A-6b (7)
2 1 2	5	100	SS-9	1.25	1	2	41	31	25	49	30	19	54	A-7-5 (9)
1 2 10	18	100	SS-10A	0.5	15	13	34	19	19	-	-	-	25	A-6a (V)
			SS-10B	-	52	10	15	12	11	-	-	-	15	A-1-b (V)
50-2"	-	100	SS-11	-	-	-	-	-	-	-	-	-	15	Rock (V)
13		100	NQ-12											CORE

PID: 114501	BR ID: COL-MARKET-0170	PROJECT: COL-MARKET ST. BRIDGE	STATION / OFFSET: 11+03, 14' LT	START: 4/4/22	END: 4/4/22	PG 2 OF 2	B-002-0-21

MATERIAL DESCRIPTION AND NOTES	ELEV. 914.1	DEPTHS	SPT/RQD	N60	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
- @ 26.5' to 28.5' SDI = 98.3%.		31																
SANDSTONE, gray, slightly to moderately weathered, medium to coarse grained, strong, thin to medium bedded, highly fractured to fractured, narrow, slightly rough, blocky, good condition, clay smearing at 32.5', RQD = 32%, REC = 100%. *(continued)*		32 33 34	37		100	NQ-13											CORE	
- @ 34.5' to 34.8' Qu = 8,921 psi.	907.6	35 36 EOB																

NOTES:
- Groundwater noted at 23.5' during drilling.
- Water measured at 12.0' upon completion after coring.

NOTES: SEE ABOVE.

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

B-001-0-21

Run #:	Depth		Recovery		RQD	
NQ-13	27.0'	32.0'	59½ / 60	99%	17¼ / 60	29%
NQ-14	32.0'	37.0'	56¼ / 60	94%	47¼ / 60	79%
COL-Market Street Bridge Replacement PID 114501						

S&ME, Inc.

B-002-0-21

Run #:	Depth		Recovery		RQD	
NQ-12	26.5’	31.5’	60 / 60	100%	8 / 60	13%
NQ-13	31.5’	36.5’	60 / 60	100%	22 / 60	37%
COL-Market Street Bridge Replacement PID 114501						

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.:	216853A	Report Date:	05/16/22
Project Name:	COL-Market St. Bridge Replacement	Test Date(s):	05/11/22
Client Name:	IBI Group		
Client Address:	4150 Belden Village Street, Suite 104, Canton, OH 43235	Received Date:	04/28/22
Location:	B-001-0-21	Depth/Elev., ft:	31.0 - 31.4
Sample Description:	Gray Sandstone		

Angle of load relative to lithology: Approximately perpendicular

Test Results

Moisture Content	***4.5*** %	***Dry Unit Weight***	***139.5*** pcf
Compressive Strength	***9,048*** psi		

Strain rate: 0.015 in/min.

Notes / Deviations / References:

J. Folsom	Jacob Folsom	Laboratory Services Manager	5/16/2022
Technical Responsibility	*Signature*	*Position*	*Date*

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: COL-Market St. Bridge Replacement Location: B-001-0-21 Depth, feet: 31.0 - 31.4

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."

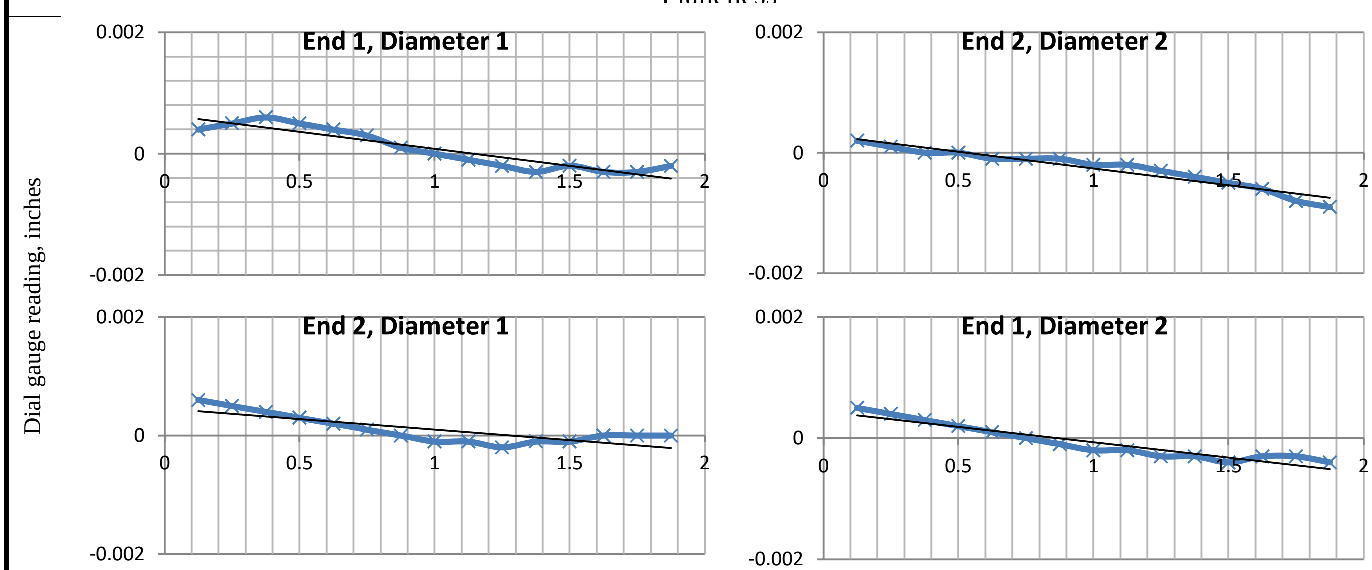
Length to Diameter Ratio

Length, inches: 4.35 Diameter, inches: 1.986
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:	0.01
Slope difference, Diameter 2, degrees:	0.00

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:	-0.02
Slope of End 1, Diameter 2, degrees:	-0.03
Slope of End 2, Diameter 2, degrees:	-0.03

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:	3.35
Temperature:	room temperature

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

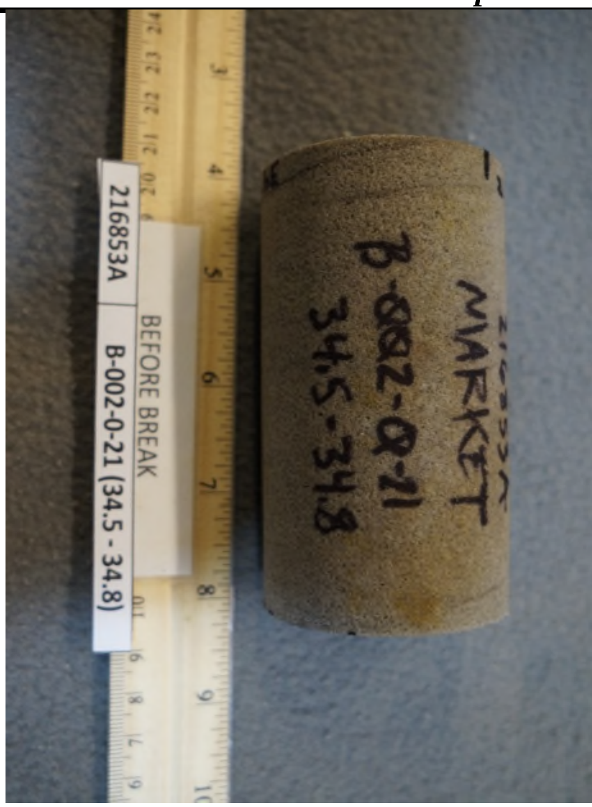
Project No.:	216853A	Report Date:	05/16/22
Project Name:	COL-Market St. Bridge Replacement	Test Date(s):	05/11/22
Client Name:	IBI Group		
Client Address:	4150 Belden Village Street, Suite 104, Canton, OH 43235	Received Date:	04/28/22
Location:	B-002-0-21	Depth/Elev., ft:	34.5 - 34.8
Sample Description:	Gray Sandstone		

Angle of load relative to lithology: Approximately perpendicular

Test Results

Moisture Content ***0.4*** **%** ***Dry Unit Weight*** ***142.0*** **pcf**

Compressive Strength ***8,921*** **psi**



Strain rate: 0.015 in/min.

Notes / Deviations / References: **Test specimen did not meet the ASTM D7012 specification for a height to diameter ratio of 2:1. Test results for specimens not meeting this requirement may differ from test results obtained from specimens meeting this requirement.**

J. Folsom	Jacob Folsom	Laboratory Services Manager	5/16/2022
Technical Responsibility	*Signature*	*Position*	*Date*

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:	COL-Market St. Bridge Replacement	Location:	B-002-0-21	Depth, feet:	34.5 - 34.8

Summary of Specimen Tolerances

Length/diameter target:	**BEST AVAILABLE**	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: 3.93 Diameter, inches: 1.987

Ratio: 1.98 length to 1 diameter

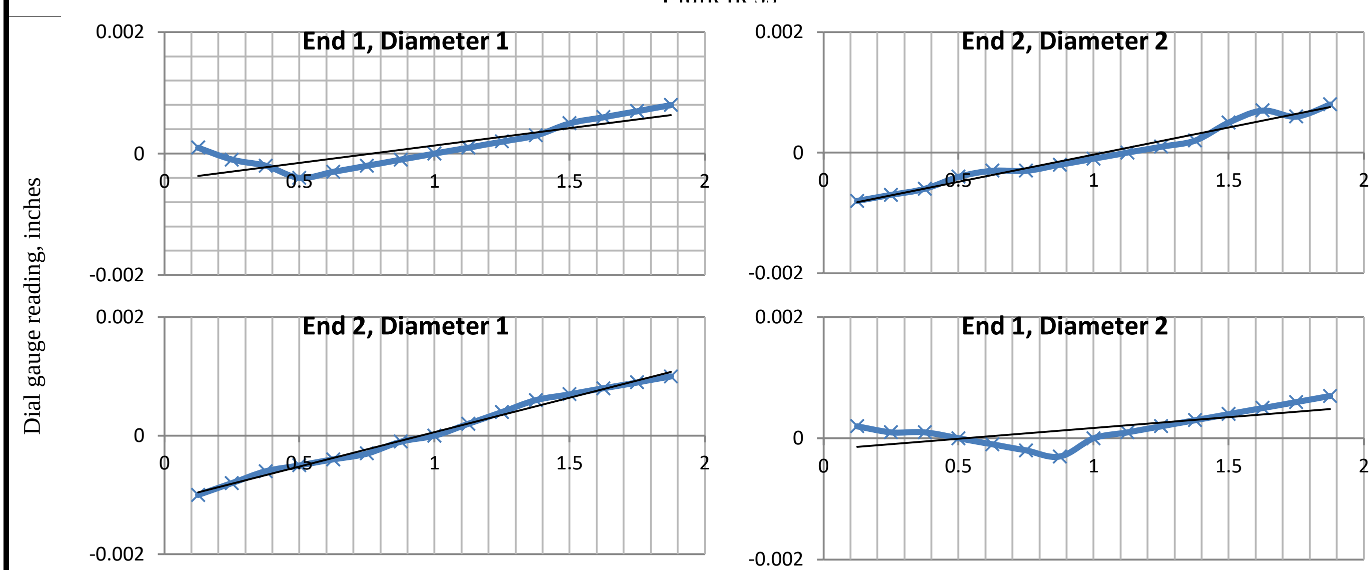
Target tolerance: L:D ratio between 2 to 1 and 2.5 to 1

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:	0.03
Slope difference, Diameter 2, degrees:	0.03

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:	0.07
Slope of End 1, Diameter 2, degrees:	0.02
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:	3.35
Temperature:	room temperature

Form No. 2370-LEX-SDI2CYCLE
Revision No. : 2
Revision Date: 11/09/20

Slake Durability Index Test

ASTM D4644

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

Project No: 216853A Project Name: COL-Market St. Bridge Replacement Report: 05/16/22

Core ID	**Slake Durability Index, %**	Desc. Of Fragments	Avg. Water Temp., °C	Natural Moisture Content, %	Before 1st Wash	After 2nd Wash
B-001-0-21 27.0 - 29.0	**97.9**	I	23 [23 - 24]	5.0		
B-002-0-21 26.5 - 28.5	**98.3**	I	23 [23 - 24]	3.5		

References / Comments / Deviations:

Reviewed by: J. Folsom 05/16/22 Jacob Folsom

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 – Calculations

Project Number:	216853A	Calculated By:	BKS
Project Name:	COL-Market Street Bridge	Date:	9/8/2023
Project Location:	Lisbon, Ohio	Checked By:	RSW
Client Name:	Arcadis	Date:	9/8/2023

Version 2.0 (8/31/16)

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		Market Street Bridge over Little Beaver Creek	
Boring ID	B-001-0-21	Foundation Element Description	Rear Abut.
Surface Elev.	944.2	Top of Shaft / Base of Shaft Cap Elevation	931.3

Analysis Desc.	Term/Info Description	Unit	Layer 1	Layer 2	Layer 3	Layer 4
Boring/Layer Information	Bedrock Type/Description		SS	SS		
	Layer Top Depth (from G.S.)	ft	27	31.6		
	Layer Top Elevation	MSL	917.2	912.6		
	Layer Bottom Depth (from G.S.)	ft	31.6	36.7		
	Layer Bottom Elevation	MSL	912.6	907.5		
	Layer Thickness	ft	4.6	5.1		
GSI Index Calculation (AASHTO LRFD, 9th Edition; Hoek, et al., 2013; Bieniawski, Z.T. 1989)	RQD	%	24	88		
	Discontinuity Length Rating		D	D		
	Separation Rating		D	C		
	Roughness Rating		A	A		
	Infilling Rating		B	A		
	Weathering Rating		C	C		
	Estimated JCond89 Value		15	20		
	Estimated GSI Value (quan.)		34.5	74		
	Estimated GSI Value (qual.)		45	60		
	Design GSI Value		45	65		
Unit Side Resistance Calculations (AASHTO LRFD, 9th Edition)	Compressive Strength, q_u	psi	9048	7500		
	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	34.94		
	Reduction Factor, α_E		0.47	0.94		
	q_s (Eqn. 10.8.3.5.4b-2)	ksf	10.68	21.35		
	q_s (Design)	**ksf**	**20**	**30**		
	q_s (Design)	**tsf**	**10**	**15**		

Definition of Bedrock Type Abbreviations:

SS = Sandstone | *SH = Shale* | *in/b = interbedded with*

SLTS = Siltstone | *CLST = Claystone*

PLATE 1

Project Number:	216853A	Calculated By:	BKS
Project Name:	COL-Market Street Bridge	Date:	9/8/2023
Project Location:	Lisbon, Ohio	Checked By:	RSW
Client Name:	Arcadis	Date:	9/8/2023

Version 2.0 (8/31/16)

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

		Term/Info Description	Unit	Layer 1	Layer 2	Layer 3	Layer 4
Unit End Bearing Resistance Calculations	Intermediate Parameters Required for GSI Empirical Approach, AASHTO LRFD Eqn. 10.8.3.5.4c-2	Compressive Strength, q_u	ksf	576.00	576.00		
		Disturbance Factor, D		0.5	0.2		
		Empirical Parameter, s		0.0006534	0.0155039		
		Empirical Parameter, a		0.5081	0.502		
		Constant, m_i (Table 10.4.6.4-1)		17	17		
		Empirical Parameter, m_b		1.2388	4.239		
		Depth of Soil Cover	ft	25	25		
		Average γ_m of Soil Cover	pcf	125	125		
		Average γ_m of Bedrock	pcf	140	140		
		Depth to Water Table	ft	20	20		
		Estimated Shaft Tip Depth (BGS)	ft	35	35		
		Vertical Effective Stress, σ'_{vb}	ksf	3.589	3.589		
		Intermediate Parameter, A		54.29	120.77		
	Intermediate Parameters Required for Canadian Geotechnical Society Solution (Eqn. 13-21 from FHWA-NHI-10-016, GEC 10)	Rock Socket Diameter, B	ft	3	3		
		Rock Socket Embedment, D_s	ft	5	5		
		s_v Selection ID		8	7		
		s_v	ft	0.5	0.67		
		t_d Selection ID		3	4		
		t_d	in	0.1	0.05		
		Check 1		YES	YES		
		Check 2		YES	YES		
		USE t_d/s_v		0.017	0.006		
		NEW s_v		N/A	N/A		
		Check 3		YES	YES		
		USE s_v/B		0.167	0.223		
		K_{sp}		0.128	0.193		
		d		1.7	1.7		
	Solutions & Design Strength Selection	q_p (Eqn. 10.8.3.5.4c-1)	ksf	1440	1440		
		q_p (Eqn. 10.8.3.5.4c-2)	ksf	248.27	668.41		
		q_p (FHWA-IF-99-025, Eqn. 11.6)	ksf	LOW RQD	547.62		
		q_p (FHWA-NHI-10-016, Eqn. 13-21)	ksf	369.38	556.95		
		q_p (Design)	**ksf**	**600**	**1440**		
		q_p (Design)	**tsf**	**300**	**720**		

Project Number:	216853A	Boring(s):	B-001-0-21	Calculated By:	BKS
Project Name:	COL-Market Street Bridge	Layer Depth Range:	27' - 31.6'	Date:	9/8/2023
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.2' - 912.6'	Checked By:	RSW
Client Name:	Arcadis	Foundation Element:	Rear Abut.	Date:	9/8/2023

Version 2.0 (8/31/16)

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	D	1	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	D	1	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	A	6	A	B	C	D	E
			Very Rough	Rough	Slightly Rough	Smooth	Slickensided
			RELATIVE RATING				
			6	5	3	1	0
Infilling (Gouge) Rating	B	4	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 $JCond_{89}$	References:
15	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:	216853A	Boring(s):	B-001-0-21	Calculated By:	BKS
Project Name:	COL-Market Street Bridge	Layer Depth Range:	31.6' - 36.7'	Date:	9/8/2023
Project Location:	Lisbon, Ohio	Layer Elevation Range:	912.6' - 907.5'	Checked By:	RSW
Client Name:	Arcadis	Foundation Element:	Rear Abut.	Date:	9/8/2023

Version 2.0 (8/31/16)

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	D	1	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	A	6	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 $JCond_{89}$	References:
20	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:	216853A	Boring(s):	B-001-0-21			
Project Name:	COL-Market Street Bridge	Layer Depth Range:	27' - 31.6'			
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.2' - 912.6'	Calc / Check By:	BKS	RSW
Client Name:	Arcadis	Analysis Purpose:	Rear Abut.	Date:	09/08/23	09/08/23

Version 2.0 (8/31/16)

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 =	9048	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, <u>Equation [10.8.3.5.4b-1] with C = 1.0 is recommended for routine design of rock sockets.</u> 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)

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 = 9048 psi

f'_c = 4000 psi

RQD = 24 %

Fractured Rock = No *(i.e. susceptible to caving)*

Joint Type = Closed

α_E = 0.47 (Table 10.8.3.5.4b-1)

q_s = 10.68 ksf

q_s = 5.34 tsf

SUMMARY

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

q_s (fractured rock) = 10.68 ksf *(eqn. 10.8.3.5.4b-2)*

q_s (design) = 20 ksf

q_s (design) = 10 ksf

Project Number:	216853A	Boring(s):	B-001-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	27' - 31.6'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.2' - 912.6'
Client Name:	Arcadis	Analysis Purpose:	Rear Abut.

Version 2.0 (8/31/16)

	Calc	Check
Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

Driiled Shafts in Rock - Example Calculations - Determine End Bearing Resistance, q_p (Utilizing 4 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 = 9048 psi

f'_c = 4000 psi

q_p = 1440 ksf

q_p = 720 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 1: AASHTO LRFD Equation 10.8.3.5.4c-2

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

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.

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.

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:	216853A	Boring(s):	B-001-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	27' - 31.6'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.2' - 912.6'
Client Name:	Arcadis	Analysis Purpose:	Rear Abut.

Version 2.0 (8/31/16)

Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 Methods) - Continued*

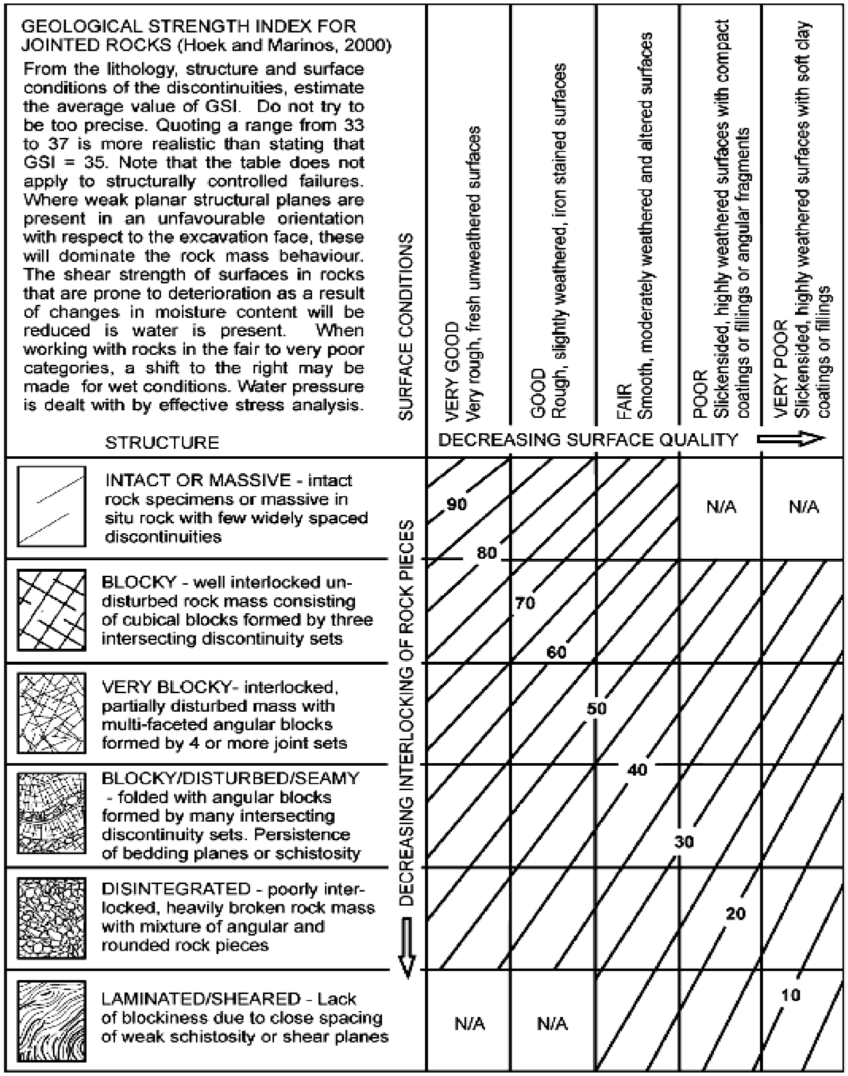


Figure 10.4.6.4-1—Determination of *GSI* for Jointed Rock Mass (Hoek and Marinos, 2000)

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	Texture: Medium	Texture: Fine	Texture: 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:	216853A	Boring(s):	B-001-0-21			
Project Name:	COL-Market Street Bridge	Layer Depth Range:	27' - 31.6'			
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.2' - 912.6'	Calc / Check By:	BKS	RSW
Client Name:	Arcadis	Analysis Purpose:	Rear Abut.	Date:	09/08/23	09/08/23

Version 2.0 (8/31/16)

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 Methods) - Continued*

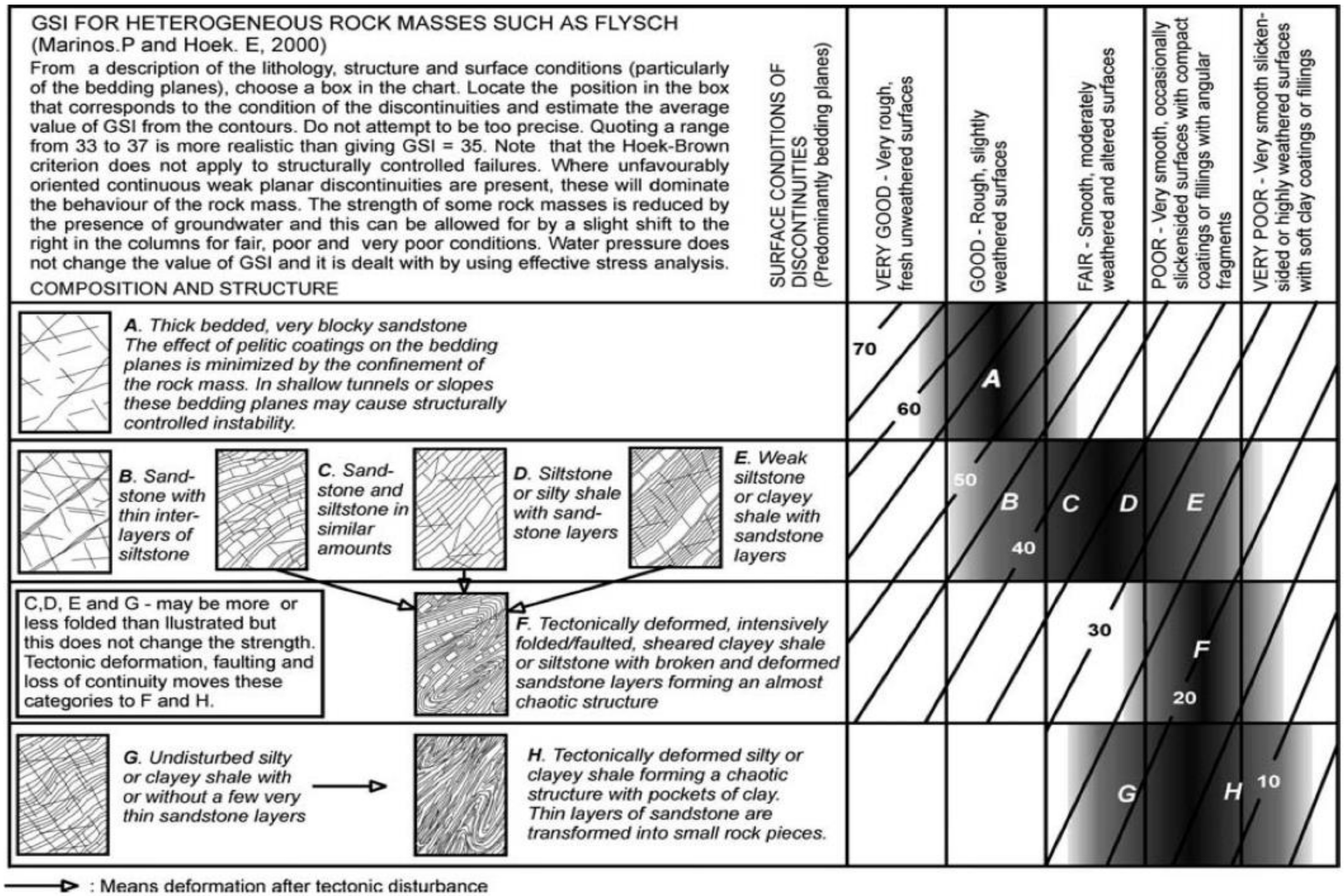


Figure 10.4.6.4-2—Determination of *GSI* for Tectonically Deformed Heterogeneous Rock Masses (Marinos and Hoek 2000)

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:	216853A	Boring(s):	B-001-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	27' - 31.6'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.2' - 912.6'
Client Name:	Arcadis	Analysis Purpose:	Rear Abut.

Version 2.0 (8/31/16)

	Calc	Check
Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 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 = 24

$JCond_{89}$ = 15

GSI (Quan.) = 34.5 $GSI = 1.5JCond_{89} + RQD/2$

GSI (Qual.) = 45 from Figures 10.4.6.4-1 & 10.4.6.4-2

GSI (Design) = 45

D = 0.5

m_i = 17

s = 0.0006534

a = 0.5081

m_b = 1.2388

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) =	25	ft
Average Unit Weight of Soil Cover ($\gamma_{m,soil}$) =	125	pcf
Average Unit Weight of Bedrock ($\gamma_{m,rock}$) =	140	pcf
Depth to Water Table (D_w) =	20	ft
Estimated Shaft Tip Depth Below Ground Surface (D_t) =	35	ft

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

where:

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

σ'_{vb} = 3.589 ksf

A = 54.29

Step 3: Determine estimated tip resistance

q_p = 248.27 ksf

q_p = 124.14 tsf

Project Number:	216853A	Boring(s):	B-001-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	27' - 31.6'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.2' - 912.6'
Client Name:	Arcadis	Analysis Purpose:	Rear Abut.

Version 2.0 (8/31/16)

	Calc	Check
Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 Methods) - Continued*

Method 3: FHWA-IF-99-025 Equation 11-6

$$q_p = 4.83(q_u)^{0.51}$$

RQD = 24

q_u = 4000 psi

q_u = 27.58 MPa

q_p = LOW RQD MPa

q_p = LOW RQD ksf

q_p = LOW RQD tsf

where:

q_p = unit end bearing resistance (MPa)

q_u = compressive strength of rock (MPa) (1 psi = 0.00689475728 MPa)

NOTE: Equation 11-6 should only be used when the following are true:

1) Rock mass has an RQD value between 70% and 100%;
2) Closed joints are approximately horizontal; and
3) q_u > 0.5 MPa (5.2 tsf or 72.5 psi)

Method 4: FHWA-NHI-10-016 Equations 13-21 thru 13-23

Equation 13-21:

$$q_p = q_{BN} = 3q_u K_{sp} d$$

Equation 13-22:

$$K_{sp} = \frac{3 + \frac{s_v}{B}}{10\sqrt{1 + 300\frac{t_d}{s_v}}}$$

Equation 13-23:

$$d = 1 + 0.4\frac{D_s}{B} \le 3.4$$

where:

q_p = unit end bearing resistance (ksf)

q_u = compressive strength of rock (ksf)

s_v = vertical spacing between discontinuities

t_d = aperature (thickness) of discontinuities

B = socket diameter (ft)

D_s = depth of socket (rock) embedment (ft)

Project Number:	216853A	Boring(s):	B-001-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	27' - 31.6'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.2' - 912.6'
Client Name:	Arcadis	Analysis Purpose:	Rear Abut.

Version 2.0 (8/31/16)

Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 Methods) - Continued*

Method 4: FHWA-NHI-10-016 Equations 13-21 thru 13-23 (continued)

B =	3	ft
D_s =	5	ft
s_v Selection ID =	8	
s_v =	0.5	ft
t_d Selection ID =	3	
t_d =	0.1	in

Check 1:	Is B > 1 ft	
B =	3	
PASS CHECK?	YES	
Check 2:	Is $0 < t_d/s_v < 0.02$	
t_d/s_v =	0.017	
PASS CHECK?	YES	If no, adjust s_v
USE t_d/s_v =	0.017	
NEW s_v =	N/A	ft
Check 3:	Is $0.05 < s_v/B < 2.0$	
s_v/B =	0.167	
PASS CHECK?	YES	
USE s_v/B =	0.167	

Adapted from Table 600-14 in 2007 ODOT SGE, July 2014 Update

Selection ID	Degree of Fracturing	Spacing, s_v	Design Value, s_v	
		(ft)	(ft)	(mm)
1	Unfractured	> 10.0	10	3048
2	Intact to Unfractured	$3.0 < s_v$	8	2438
3	Intact	$3.0 < s_v < 10.0$	6	1829
4	Slightly Fractured to Intact	$1.0 < s_v < 10.0$	4	1219
5	Slightly Fractured	$1.0 < s_v < 3.0$	2	610
6	Moderately to Slightly Fractured	$0.33 < s_v < 3.0$	1	305
7	Moderately Fractured	$0.33 < s_v < 1.0$	0.67	204
8	Fractured to Moderately Fractured	$0.16 < s_v < 1.0$	0.5	152
9	Fractured	$0.16 < s_v < 0.33$	0.25	76
10	Highly Fractured to Fractured	$s_v < 0.33$	0.16	49
11	Highly Fractured	$s_v < 0.16$	0.1	30

Adapted from Table 600-15 in 2007 ODOT SGE, July 2014 Update

Selection ID	Condition of Fractures	Aperture, t_d	Design Value, t_d	
		(in)	(in)	(mm)
1	Open	$0.2 < t_d$	0.5	13
2	Narrow to Open	$0.05 < t_d$	0.15	3.8
3	Narrow	$0.05 < t_d < 0.2$	0.1	2.5
4	Tight to Narrow	$t_d < 0.2$	0.05	1.3
5	Tight	$t_d < 0.05$	0.02	0.5

*Selections 2, 4, 6, 8 & 10 represents cross overs between two descriptions

Project Number:	216853A	Boring(s):	B-001-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	27' - 31.6'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.2' - 912.6'
Client Name:	Arcadis	Analysis Purpose:	Rear Abut.

Version 2.0 (8/31/16)

	Calc	Check
Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 Methods) - Continued*

Method 4: FHWA-NHI-10-016 Equations 13-21 thru 13-23 (continued)

q_u = 4000 psi

q_u = 576 ksf

K_{sp} = 0.128

d = 1.7

q_p = 369.38 ksf

q_p = 184.69 tsf

End Bearing Resistance, q_p Summary

Method	Reference	q_p Value	Unit
1	AASHTO LRFD Eqn. 10.8.3.5.4c-1	1440	ksf
2	AASHTO LRFD Eqn. 10.8.3.5.4c-2	248.27	ksf
3	FHWA-IF-99-025 Eqn. 11-6	N/A	ksf
4	FHWA-NHI-10-016 Eqn. 13-21	369.38	ksf

q_p (Design) = 600 ksf

q_p (Design) = 300 tsf

Project Number:	216853A	Calculated By:	BKS
Project Name:	COL-Market Street Bridge	Date:	9/8/2023
Project Location:	Lisbon, Ohio	Checked By:	RSW
Client Name:	Arcadis	Date:	9/8/2023

Version 2.0 (8/31/16)

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		Market Street Bridge over Little Beaver Creek	
Boring ID	B-002-0-21	Foundation Element Description	Fwd. Abut.
Surface Elev.	944.1	Top of Shaft / Base of Shaft Cap Elevation	930.55

Analysis Desc.	Term/Info Description	Unit	Layer 1	Layer 2	Layer 3	Layer 4
Boring/Layer Information	Bedrock Type/Description		SS	SS		
	Layer Top Depth (from G.S.)	ft	26.5	28.7		
	Layer Top Elevation	MSL	917.6	915.4		
	Layer Bottom Depth (from G.S.)	ft	28.7	36.5		
	Layer Bottom Elevation	MSL	915.4	907.6		
	Layer Thickness	ft	2.2	7.8		
GSI Index Calculation (AASHTO LRFD, 9th Edition; Hoek, et al., 2013; Bieniawski, Z.T. 1989)	RQD	%	0	32		
	Discontinuity Length Rating		D	D		
	Separation Rating		D	C		
	Roughness Rating		A	A		
	Infilling Rating		B	A		
	Weathering Rating		C	C		
	Estimated JCond89 Value		15	20		
	Estimated GSI Value (quan.)		22.5	46		
	Estimated GSI Value (qual.)		35	55		
	Design GSI Value		30	50		
Unit Side Resistance Calculations (AASHTO LRFD, 9th Edition)	Compressive Strength, q_u	psi	6000	8921		
	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	34.94		
	Reduction Factor, α_E		0.45	0.51		
	q_s (Eqn. 10.8.3.5.4b-2)	ksf	10.22	11.58		
	q_s (Design)	**ksf**	**20**	**30**		
	q_s (Design)	**tsf**	**10**	**15**		

Definition of Bedrock Type Abbreviations:

SS = Sandstone *SH = Shale* *in/b = interbedded with*

SLTS = Siltstone *CLST = Claystone*

Project Number:	216853A	Calculated By:	BKS
Project Name:	COL-Market Street Bridge	Date:	9/8/2023
Project Location:	Lisbon, Ohio	Checked By:	RSW
Client Name:	Arcadis	Date:	9/8/2023

Version 2.0 (8/31/16)

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

		Term/Info Description	Unit	Layer 1	Layer 2	Layer 3	Layer 4
Unit End Bearing Resistance Calculations	Intermediate Parameters Required for GSI Empirical Approach, AASHTO LRFD Eqn. 10.8.3.5.4c-2	Compressive Strength, q_u	ksf	576.00	576.00		
		Disturbance Factor, D		0.5	0.2		
		Empirical Parameter, s		0.0000884	0.0025996		
		Empirical Parameter, a		0.5223	0.5057		
		Constant, m_i (Table 10.4.6.4-1)		17	17		
		Empirical Parameter, m_b		0.6065	2.3375		
		Depth of Soil Cover	ft	25	25		
		Average γ_m of Soil Cover	pcf	125	125		
		Average γ_m of Bedrock	pcf	140	140		
		Depth to Water Table	ft	20	20		
		Estimated Shaft Tip Depth (BGS)	ft	35	35		
		Vertical Effective Stress, σ'_{vb}	ksf	3.589	3.589		
		Intermediate Parameter, A		35.24	77.32		
	Intermediate Parameters Required for Canadian Geotechnical Society Solution (Eqn. 13-21 from FHWA-NHI-10-016, GEC 10)	Rock Socket Diameter, B	ft	3	3		
		Rock Socket Embedment, D_s	ft	5	5		
		s_v Selection ID		10	10		
		s_v	ft	0.16	0.16		
		t_d Selection ID		3	4		
		t_d	in	0.1	0.05		
		Check 1		YES	YES		
		Check 2		NO	NO		
		USE t_d/s_v		0.02	0.02		
		NEW s_v		0.41667	0.20833		
		Check 3		YES	YES		
		USE s_v/B		0.139	0.069		
		K_{sp}		0.119	0.116		
		d		1.7	1.7		
	Solutions & Design Strength Selection	q_p (Eqn. 10.8.3.5.4c-1)	ksf	1440	1440		
		q_p (Eqn. 10.8.3.5.4c-2)	ksf	138.46	399.2		
		q_p (FHWA-IF-99-025, Eqn. 11.6)	ksf	LOW RQD	LOW RQD		
		q_p (FHWA-NHI-10-016, Eqn. 13-21)	ksf	343.41	334.75		
		q_p (Design)	**ksf**	**600**	**1440**		
		q_p (Design)	**tsf**	**300**	**720**		

Project Number:	216853A	Boring(s):	B-002-0-21	Calculated By:	BKS
Project Name:	COL-Market Street Bridge	Layer Depth Range:	26.5' - 28.7'	Date:	9/8/2023
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.6' - 915.4'	Checked By:	RSW
Client Name:	Arcadis	Foundation Element:	Fwd. Abut.	Date:	9/8/2023

Version 2.0 (8/31/16)

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	D	1	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	D	1	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	A	6	A	B	C	D	E
			Very Rough	Rough	Slightly Rough	Smooth	Slickensided
			RELATIVE RATING				
			6	5	3	1	0
Infilling (Gouge) Rating	B	4	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 $JCond_{89}$	
15	

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:	216853A	Boring(s):	B-002-0-21	Calculated By:	BKS
Project Name:	COL-Market Street Bridge	Layer Depth Range:	28.7' - 36.5'	Date:	9/8/2023
Project Location:	Lisbon, Ohio	Layer Elevation Range:	915.4' - 907.6'	Checked By:	RSW
Client Name:	Arcadis	Foundation Element:	Fwd. Abut.	Date:	9/8/2023

Version 2.0 (8/31/16)

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	D	1	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	A	6	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 $JCond_{89}$	References:
20	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:	216853A	Boring(s):	B-002-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	26.5' - 28.7'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.6' - 915.4'
Client Name:	Arcadis	Analysis Purpose:	Fwd. Abut.

Version 2.0 (8/31/16)

	Calc	Check
Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

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 =	6000	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:	216853A	Boring(s):	B-002-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	26.5' - 28.7'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.6' - 915.4'
Client Name:	Arcadis	Analysis Purpose:	Fwd. Abut.

Version 2.0 (8/31/16)

	Calc	Check
Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

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 =	6000	psi
f'_c =	4000	psi
RQD =	0	%
Fractured Rock =	No	*(i.e. susceptible to caving)*
Joint Type =	Closed	

α_E = 0.45 (Table 10.8.3.5.4b-1)

q_s = 10.22 ksf

q_s = 5.11 tsf

SUMMARY

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

q_s (fractured rock) = 10.22 ksf *(eqn. 10.8.3.5.4b-2)*

q_s (design) = 20 ksf

q_s (design) = 10 ksf

Project Number:	216853A	Boring(s):	B-002-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	26.5' - 28.7'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.6' - 915.4'
Client Name:	Arcadis	Analysis Purpose:	Fwd. Abut.

Version 2.0 (8/31/16)

	BKS	RSW
Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 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 = 6000 psi

f'_c = 4000 psi

q_p = 1440 ksf

q_p = 720 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 1: AASHTO LRFD Equation 10.8.3.5.4c-2

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

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.

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.

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: 216853A

Project Name: COL-Market Street Bridge

Project Location: Lisbon, Ohio

Client Name: Arcadis

Boring(s): B-002-0-21

Layer Depth Range: 26.5' - 28.7'

Layer Elevation Range: 917.6' - 915.4'

Analysis Purpose: Fwd. Abut.

Version 2.0 (8/31/16)

Calc / Check By: BKS RSW

Date: 09/08/23 09/08/23

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 Methods) - Continued*

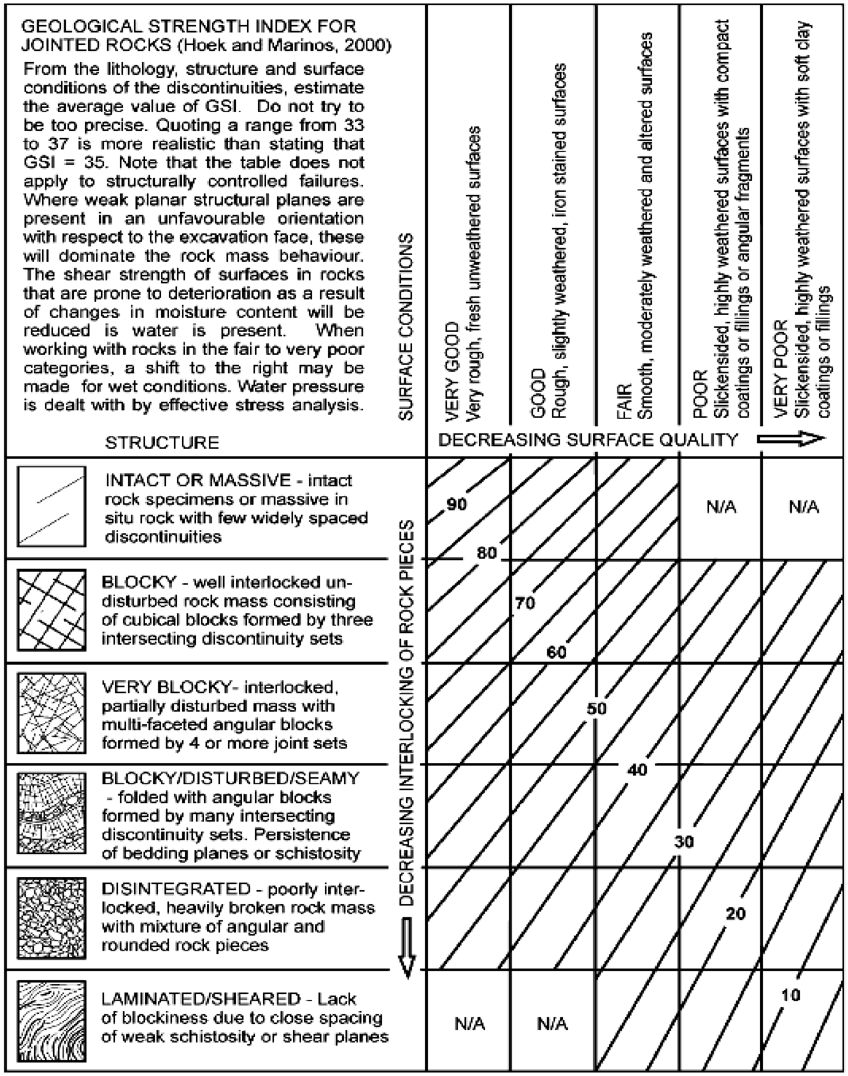


Figure 10.4.6.4-1—Determination of *GSI* for Jointed Rock Mass (Hoek and Marinos, 2000)

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:	216853A	Boring(s):	B-002-0-21			Version 2.0 (8/31/16)
Project Name:	COL-Market Street Bridge	Layer Depth Range:	26.5' - 28.7'			
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.6' - 915.4'	Calc / Check By:	BKS	RSW
Client Name:	Arcadis	Analysis Purpose:	Fwd. Abut.	Date:	09/08/23	09/08/23

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 Methods) - Continued*

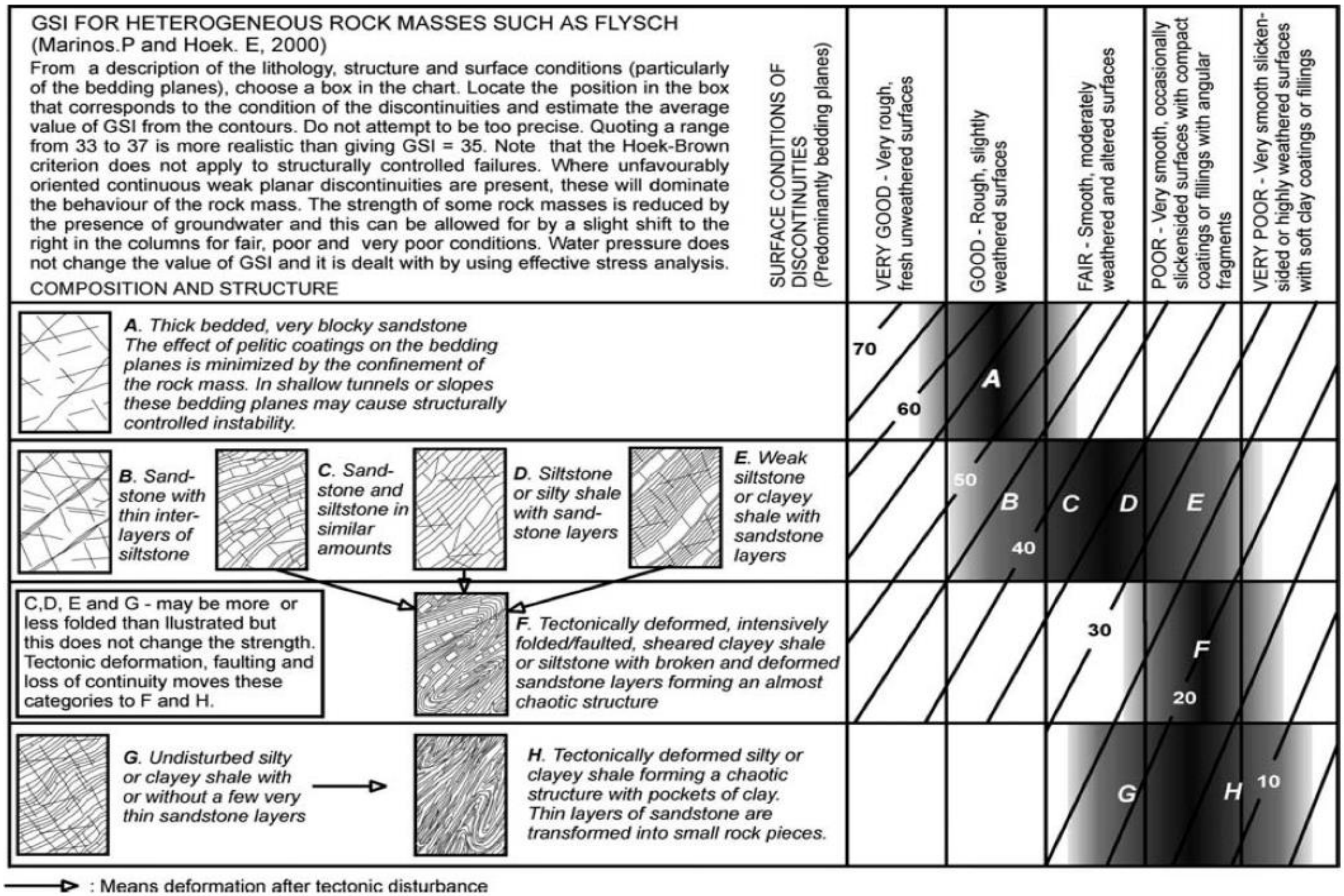


Figure 10.4.6.4-2—Determination of *GSI* for Tectonically Deformed Heterogeneous Rock Masses (Marinos and Hoek 2000)

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).

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 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 = 0

$JCond_{89}$ = 15

D = 0.5

m_i = 17

GSI (Quan.) = 22.5 $GSI = 1.5JCond_{89} + RQD/2$

GSI (Qual.) = 35 from Figures 10.4.6.4-1 & 10.4.6.4-2

GSI (Design) = 30

s = 0.0000884

a = 0.5223

m_b = 0.6065

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) = 25 ft

Average Unit Weight of Soil Cover ($\gamma_{m,soil}$) = 125 pcf

Average Unit Weight of Bedrock ($\gamma_{m,rock}$) = 140 pcf

Depth to Water Table (D_w) = 20 ft

Estimated Shaft Tip Depth Below Ground Surface (D_t) = 35 ft

σ'_{vb} = 3.589 ksf

A = 35.24

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

where:

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

Step 3: Determine estimated tip resistance

q_p = 138.46 ksf

q_p = 69.23 tsf

Project Number:	216853A	Boring(s):	B-002-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	26.5' - 28.7'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.6' - 915.4'
Client Name:	Arcadis	Analysis Purpose:	Fwd. Abut.

Version 2.0 (8/31/16)

Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 Methods) - Continued*

Method 3: FHWA-IF-99-025 Equation 11-6

$$q_p = 4.83(q_u)^{0.51}$$

RQD = 0

q_u = 4000 psi

q_u = 27.58 MPa

q_p = LOW RQD MPa

q_p = LOW RQD ksf

q_p = LOW RQD tsf

where:

q_p = unit end bearing resistance (MPa)

q_u = compressive strength of rock (MPa) (1 psi = 0.00689475728 MPa)

NOTE: Equation 11-6 should only be used when the following are true:

1) Rock mass has an RQD value between 70% and 100%;
2) Closed joints are approximately horizontal; and
3) q_u > 0.5 MPa (5.2 tsf or 72.5 psi)

Method 4: FHWA-NHI-10-016 Equations 13-21 thru 13-23

Equation 13-21: $$q_p = q_{BN} = 3q_u K_{sp} d$$

Equation 13-22: $$K_{sp} = \frac{3 + \frac{s_v}{B}}{10\sqrt{1 + 300\frac{t_d}{s_v}}}$$

Equation 13-23: $$d = 1 + 0.4\frac{D_s}{B} \leq 3.4$$

where:

q_p = unit end bearing resistance (ksf)

q_u = compressive strength of rock (ksf)

s_v = vertical spacing between discontinuities

t_d = aperature (thickness) of discontinuities

B = socket diameter (ft)

D_s = depth of socket (rock) embedment (ft)

Project Number:	216853A	Boring(s):	B-002-0-21		
Project Name:	COL-Market Street Bridge	Layer Depth Range:	26.5' - 28.7'		
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.6' - 915.4'	Calc / Check By:	BKS / RSW
Client Name:	Arcadis	Analysis Purpose:	Fwd. Abut.	Date:	09/08/23 / 09/08/23

Version 2.0 (8/31/16)

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 Methods) - Continued*

Method 4: FHWA-NHI-10-016 Equations 13-21 thru 13-23 (continued)

B =	3	ft
D_s =	5	ft
s_v Selection ID =	10	
s_v =	0.16	ft
t_d Selection ID =	3	
t_d =	0.1	in

Check 1: Is B > 1 ft

B = 3

PASS CHECK? YES

Check 2: Is $0 < t_d/s_v < 0.02$

t_d/s_v = 0.052

PASS CHECK? NO If no, adjust s_v

USE t_d/s_v = 0.02

NEW s_v = 0.41667 ft

Check 3: Is $0.05 < s_v/B < 2.0$

s_v/B = 0.139

PASS CHECK? YES

USE s_v/B = 0.139

Adapted from Table 600-14 in 2007 ODOT SGE, July 2014 Update

Selection ID	Degree of Fracturing	Spacing, s_v (ft)	Design Value, s_v (ft)	Design Value, s_v (mm)
1	Unfractured	> 10.0	10	3048
2	Intact to Unfractured	$3.0 < s_v$	8	2438
3	Intact	$3.0 < s_v < 10.0$	6	1829
4	Slightly Fractured to Intact	$1.0 < s_v < 10.0$	4	1219
5	Slightly Fractured	$1.0 < s_v < 3.0$	2	610
6	Moderately to Slightly Fractured	$0.33 < s_v < 3.0$	1	305
7	Moderately Fractured	$0.33 < s_v < 1.0$	0.67	204
8	Fractured to Moderately Fractured	$0.16 < s_v < 1.0$	0.5	152
9	Fractured	$0.16 < s_v < 0.33$	0.25	76
10	Highly Fractured to Fractured	$s_v < 0.33$	0.16	49
11	Highly Fractured	$s_v < 0.16$	0.1	30

Adapted from Table 600-15 in 2007 ODOT SGE, July 2014 Update

Selection ID	Condition of Fractures	Aperture, t_d (in)	Design Value, t_d (in)	Design Value, t_d (mm)
1	Open	$0.2 < t_d$	0.5	13
2	Narrow to Open	$0.05 < t_d$	0.15	3.8
3	Narrow	$0.05 < t_d < 0.2$	0.1	2.5
4	Tight to Narrow	$t_d < 0.2$	0.05	1.3
5	Tight	$t_d < 0.05$	0.02	0.5

*Selections 2, 4, 6, 8 & 10 represents cross overs between two descriptions

Project Number:	216853A	Boring(s):	B-002-0-21
Project Name:	COL-Market Street Bridge	Layer Depth Range:	26.5' - 28.7'
Project Location:	Lisbon, Ohio	Layer Elevation Range:	917.6' - 915.4'
Client Name:	Arcadis	Analysis Purpose:	Fwd. Abut.

Version 2.0 (8/31/16)

	BKS	RSW
Calc / Check By:	BKS	RSW
Date:	09/08/23	09/08/23

Driiled Shafts in Rock - Example Calculations - *Determine End Bearing Resistance, q_p (Utilizing 4 Methods) - Continued*

Method 4: FHWA-NHI-10-016 Equations 13-21 thru 13-23 (continued)

q_u = 4000 psi

q_u = 576 ksf

K_{sp} = 0.119

d = 1.7

q_p = 343.41 ksf

q_p = 171.71 tsf

End Bearing Resistance, q_p Summary

Method	Reference	q_p Value	Unit
1	AASHTO LRFD Eqn. 10.8.3.5.4c-1	1440	ksf
2	AASHTO LRFD Eqn. 10.8.3.5.4c-2	138.46	ksf
3	FHWA-IF-99-025 Eqn. 11-6	N/A	ksf
4	FHWA-NHI-10-016 Eqn. 13-21	343.41	ksf

q_p (Design) = 600 ksf

q_p (Design) = 300 tsf

Summary of Bedrock Properties
COL-Market Street Bridge Replacement

Proposed Single Span Bridge

Borings Performed:

Boring No.	Substructure Unit	Method	Notes
B-001-0-21	Rear Abut.	Truck Rig	~22' S of Rear Abutment
B-002-0-21	Fwd. Abut.	Truck Rig	~40' N of Forward Abutment

Bedrock Description: **SANDSTONE**, gray moderately weathered, fine to medium grained, strong, thick to very thick bedded, fractured to highly fractured.

Bedrock Information:

Boring No.	Ground Surface Elevation	Top of Bedrock Elevation	Depth to Bedrock	Total Core Length	Lithologic Recovery (%)*	Lithologic RQD (%)*	Top of Core Run	Bottom of Core Run	Length of Core Run	Recovery (%) by Core Run	RQD (%) by Core Run
B-001-0-21	944.2	919.2	25	10	100	29					
							27.0'	32.0'	5.0'	99	29
							32.0'	37.0'	5.0'	94	79
B-003-0-21	944.1	919.1	25	10	100	13					
							26.5'	31.5'	5.0'	100	13
							31.5'	36.5'	5.0'	100	37

*Applies to upper layer of bedrock only.

Summary of Bedrock Properties (continued)
COL-Market Street Bridge Replacement

Bedrock Properties & Test Results:

Boring No.	Core No.	Top of Core Run	Bottom of Core Run	Test Sample Depth	Unit Weight (pcf)	Unconfined Compressive Strength (psi)	Sample Depth	Slake Durability Index (%)
B-001-0-21	NQ-13	27.0'	32.0'	31.0' - 31.4'	139.5	9,048	27.0' - 29.0'	97.9
B-002-0-21	NQ-12	26.5'	31.5'				26.5' - 28.5'	98.3
B-002-0-21	NQ-13	31.5'	36.5'	34.5' - 34.8'	142	8,921		
					140.8 (Average pcf)	Ave = 8,985		98.1 (Average %)

Assessment for Scour Resistance

July 2020 BDM Section 305.4.1.1, Scour

- Assess scour resistant rock according to BDM Section 305.2.1.2.b

Per BDM 305.2.1.2.b.B:	Scour resistant Rock will have the following:	Meets?	Average	
	- Qu $\geq$ 2500 psi	Yes	8,985	psi
	- SDI $\geq$ 90%	Yes	98.1	%
	- RQD $\geq$ 65% (Lithologic)	No	21	%
	- Unit Weight $\geq$ 150 pcf	No	140.8	pcf
	- Rock Mass Rating (RMR) $\geq$ 75	No	(Estimated 42 to 68)	
	- Geologic Strength Index (GSI) $\geq$ 75	No	(Estimated 20 to 65)	
	- Erodability Index (K) $\geq$ 100	Yes		

Erodability Index

K = (Ms)(Kb)(Kd)(Js)

K = 214.3

Qu (ave) =	8,985	psi
RQD (ave) =	21	%
Kb = (RQD/Jn) =	6.29	(must be greater than 0.1)
Ms =	61.962069	Mpa
Kd = (Jr/Ja) =	0.5	
Js =	1.1	(select from Table 4-26 of HEC-18, based on rx core observation)
Jn =	3.34	(select from Table 4-23 of HEC-18, based on rx core observation)
Jr =	1.5	(select from Table 4-24 of HEC-18, based on rx core observation)
Ja =	3	(select from Table 4-26 of HEC-18, based on rx core observation)

Summary of Bedrock Properties (continued)
COL-Market Street Bridge Replacement

Calculate Critical Shear Stress

From HEC-18: Re-arrange Eqn 7.38 ('Critical Stream Power') and Eqn. 7.39 ('Approach Flow Stream Power') to calculate the critical shear stress for non-scour resistant bedrock as follows:

Critical Shear Stress: $\tau_c\ (\text{Pa}) = \rho\ (1000\ K^{0.75} / 7.853\ \rho)^{2/3}$

ρ = Mass density of water = 1000 kg/m3
K = Erodability Index = 214.3

τ_c (Pa) = 3705 Pa

Project #: 216853A
Project: Market St, Lisbon, Columbiana County, OH

Calculated By: BKS
Date: 9/8/2023

Boring B-001-0-21

Layer Top Elevation	Layer Bottom Elevation	Layer Thickness	Average Blow Counts, Raw N	Layer/Bpf
944.2	931.2	13	10	1.3
931.2	926.2	5	10	0.5
926.2	923.7	2.5	4	0.625
923.7	921.2	2.5	5	0.5
921.2	919.2	2	10	0.2
919.2	844.2	75	100	0.75
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
			Total, Denominator	3.875
			$= 100/\sum_{i=0}^{n} Layer/N$	25.8

Site Class A
Site Class B
Site Class C $\overline{N} > 50$
Site Class D $15 \leq \overline{N} \leq 50$
Site Class E $\overline{N} < 15$

Boring B-002-0-21

Layer Top Elevation	Layer Bottom Elevation	Layer Thickness	Average Blow Counts, Raw N	Layer/Bpf
944.1	928.6	15.5	7	2.21428571
928.6	923.6	5	4	1.25
923.6	921.1	2.5	3	0.83333333
921.1	920.1	1	3	0.33333333
920.1	919.1	1	12	0.08333333
919.1	844.1	75	100	0.75
844.1		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
0		0		0
			Total, Denominator	5.464
			$= 100/\sum_{i=0}^{n} Layer/N$	18.3

Site Class A
Site Class B
Site Class C $\overline{N} > 50$
Site Class D $15 \leq \overline{N} \leq 50$
Site Class E $\overline{N} < 15$

Appendix III – OGE Geotechnical Checklists

I. Geotechnical Design Checklists

Project:	**COL-Market Street Bridge**	**PDP Path:**	
PID:	**114501**	**Review Stage:**	**Final**

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. Geotechnical Profile VI. D. Geotechnical Reports	 ✓

II. Reconnaissance and Planning Checklist

C-R-S: COL-Market Street Bridge	**PID:** 114501	**Reviewer:** BKS	**Date:** 11/30/2023

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:		Preliminary plan information was available at time of exploration.
	Roadway plans		
	Structures plans	✓	
	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	No historic boring information was available.
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	X	Station/offset not available at time of drilling, but was provided to S&ME by Arcadis.
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, soundings, 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)		
	Rock Slope (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 and b)		
	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: COL-Market Street Bridge	PID: 114501	Reviewer: BKS	Date: 11/30/2023

Use this Checklist in conjunction with the bridge foundation design guidance in GDM Section 1300
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/N/X)	Notes:
1	Has the shear strength of the foundation soils been determined?	Y	
	Check method used:		
	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?	Y	
3	Has the shear strength of the foundation bedrock been determined?	Y	
	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 305.3.1.2 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 305.3.2.2?		
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 305.3.5.6?		
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 305.3.5.7?		

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?	X	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?	Y	
23	For shafts undergoing lateral loading, have the following been determined:	X	Lateral loading analyses to be performed by others. S&ME provided input parameters.
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 305.4.2?	Y	
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?	Y	
26	If scour is predicted, has shaft resistance in the scour zone been neglected?	✓	No bedrock scour calculated. Shafts being supported by tip resistance.
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?	Y	See Sections 6.4.3 through 6.4.5.
29	If special construction features (e.g., slurry, casing, load tests) are required, have all the proper items been included in the plans?	Y	See Sections 6.4.3 through 6.4.5.
30	If necessary, have wet construction methods been specified?	Y	See Sections 6.4.3 through 6.4.5.
General		(Y/N/X)	Notes:
31	Has the need for load testing of the foundations been evaluated?	Y	See Sections 6.4.3 through 6.4.5.
a.	If needed, have details and plan notes for load testing been included in the plans?	Y	See Sections 6.4.3 through 6.4.5.

VI.B. Geotechnical Reports

C-R-S: COL-Market Street Bridge	PID: 114501	Reviewer: BKS	Date: 11/30/2023

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 meet this demand?	Y	
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 706.1 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 706.2 of the SGE?	Y	
b.	an Introduction as described in Section 706.3 of the SGE?	Y	
c.	a section titled "Geology and Observations of the Project," as described in Section 706.4 of the SGE?	Y	
d.	a section titled "Exploration," as described in Section 706.5 of the SGE?	Y	
e.	a section titled "Findings," as described in Section 706.6 of the SGE?	Y	
f.	a section titled "Analyses and Recommendations," as described in Section 706.7 of the SGE?	Y	
Appendices		(Y/N/X)	Notes:
8	Do all geotechnical reports being submitted contain all applicable Appendices as described in Section 706.8 of the SGE?	Y	
9	Do the Appendices present a site Boring Plan showing all boring locations as described in Section 706.8.1 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 706.8.2 of the SGE?	Y	
11	Do the Appendices include reports of undisturbed test data as described in Section 706.8.3 of the SGE?	Y	
12	Do the Appendices include calculations in a logical format to support recommendations as described in Section 706.8.4 of the SGE?	Y	