Pro Geotech, Inc.

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October 11, 2024

Mr. Bob Beasley, P.E.
ARCADIS
222 S. Main Street, Suite 200
Akron, OH 44308

Reference: **Structure Foundation Exploration Report for Replacement of Bridge Numbers**
HAM-71-1.80 & HAM-71-10.93
Hamilton County, Ohio
PID No.: 102790
PGI's Project No. G23006G

Dear Mr. Beasley:

Enclosed please find our Final Structure Foundation Exploration Report for the above referenced project. Our services included geotechnical field exploration, laboratory testing, engineering analysis, and related design and construction recommendations. These services have been provided in accordance with our proposal dated March 27, 2023. It is important that the items under "Limitations" be precisely followed and complied with.

We appreciate the opportunity of working with you on this project and we invite you to contact us at (440) 717-1415 when we can be of further assistance.

Respectfully,

PRO GEOTECH, INC.

Shan Sivakumaran, P.E.
Project Manager/Geotechnical Engineer

Walid I. Najjar, P.E.
Senior Geotechnical Engineer

Enclosure
G23006G Report/SS/10/11/2024

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FINAL
STRUCTURE FOUNDATION EXPLORATION REPORT
FOR
REPLACEMENT OF BRIDGES HAM-71-1.80 & HAM-22-10.93
HAMILTON COUNTY, OHIO
PROJECT NO. G23006G AND PID NO.: 102790

PREPARED FOR:

ARCADIS

PREPARED BY:

PRO GEOTECH, INC.

OCTOBER 11, 2024

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1.0 EXECUTIVE SUMMARY

This report has been prepared for the proposed design and replacement of Bridge numbers HAM-71-1.80 & HAM-22-10.93 in Cincinnati, Hamilton County, Ohio. The proposed replacement structure will consist of Multistory Ramp/Stairway and Pedestrian Bridge. The Multistory Ramp/Stairway will be reinforced concrete slab on cantilevered concrete beams supported on 13 concrete columns. The Bridge will be three simple spans, prefabricated steel truss with concrete deck supported on three (3) wall-type piers. The total length of the Multistory Ramp will be 675 feet. The total length of the bridge from Pier to existing bridge abutment will be 350 feet.

Exploration: Structure foundation exploration was performed in 1964 under the project designation of HAM-71-0157. A total of five (5) historic test borings identified as B-7-1 (B-007-1-64), B-7-2 (B-007-2-64), B-7-3 (B-007-3-64), R-15-2 (B-015-2-64), and R-16-1 (B-016-1-64) are available for this bridge. A total of six (6) project test borings identified as B-001-0-23 through B-006-0-23 were advanced at the project site. Project test borings B-001-0-23 through B-004-0-23 were advanced at the existing parking lot for Ramp/Stairway foundation design purposes. Project test borings B-005-0-23 and B-006-0-23 were advanced within the ODOT Right of Way on both sides of Interstate-71 for bridge foundation design purposes. These test borings were advanced to approximate depths ranging from 22.0 to 60.0 feet below the existing pavement, riprap, or ground surface.

Findings: The subsurface soils encountered in project test borings consisted of both fill and natural soils above the bedrock. Fill soils were encountered in all project test borings with the exception of B-006-0-23 where bedrock was encountered below topsoil. The fill soils were encountered in project test borings B-001-0-23 through B-005-0-23 to depths ranging from 3.0 feet to 38.5 feet below the existing pavement, riprap stone or concrete surface. The bottom of fill soils layer was encountered at approximate elevations ranging from 496.4 feet to 549.0 feet. Fill soils encountered were both cohesive and non-cohesive and consisted of silty clay (A-6b), silt and clay (A-6a), sandy silt (A-4a), coarse and fine sand (A-3a), gravel and stone fragments with sand (A-1-b), asphalt, cinders, or stone fragments (A-1-a), stone fragments with sand and silt (A-2-4), and multiple combinations of concrete, asphalt, brick and stone fragments with sand. The natural soils encountered below fill soil in the project test borings consisted of silt and clay (A-6a), silty clay (A-6b) and clay (A-7-6). Bedrock consisting of shale interbedded limestone was encountered in all project test borings below approximate depths ranging from 0.25 feet (Elevation 564.5 feet) to 49.5 feet (Elevation 485.4 feet) below the existing surface.

The bedrock core samples consisted of gray shale with interbedded limestone. The shale encountered across the site was thinly laminated, generally highly to moderately fractured, and was calcareous and effervesced freely with dilute hydrochloric acid and ranged from severely to moderately weathered and was very weak to weak. The percent of interbedded limestone encountered in the individual bedrock core runs ranged from 1% to 42% and averaged 13%. The limestone encountered within the shale ranged from gray to black or dark gray to white. White limestone generally indicated the presence of fossils. The limestone ranged from crystalline to clastic and ranged from slightly to highly weathered but was generally moderately weathered and ranged from very weak to moderately strong but was generally moderately strong. The Rock Quality Designation (RQD) obtained for the bedrock core samples varied from 0 to 92% and the recovery ranged from 43 to 100% for the individual rock core runs. Based on the laboratory testing performed on the rock core samples, the point load strengths of the rock core specimens in these project test borings ranged from 33 psi to 1539 psi which characterizes them as “very weak” to “slightly strong”. The compressive strengths of the rock core specimens in these project test borings ranged from 182 psi to 1030 psi which characterizes them as “very weak” to “weak”.

Recommendations:

The unit shaft side and tip resistances were calculated based on the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD) Article 10.8.3.5.4. For the calculation of shaft side resistances, Equation 10.8.3.5.4b-1 was used since the shale bedrock encountered in the borings is not expected to cave during construction. For the calculation of shaft tip resistances, Equation 10.8.3.5.4c-1 was used to determine tip resistance because the rock below the bearing elevation is considered tightly jointed and without seams of compressible material. Since there are a lot of compressive strength testing results and a large variation in the results, OGE performed Bedrock Compressive Strength analyses. Due to the scatter of the Compressive Strength test results (particularly towards the lower elevations), these test results were plotted with respect to elevation (different elevation representing different depositional environments and times) there is a distinct pattern, with compressive strength decreasing with increasing elevation. OGE were able to plot a trendline function to estimate compressive strength (Qu, psi) as a function of elevation (E, ft) as Qu = 0.0226E2 - 30.789E + 10282. They have compared this to the actual testing results and find reasonable agreement. However, due to the scatter of the results (particularly towards the lower elevations), they recommend using the minimum of this function or the average of the strength testing results. Refer to OGE spreadsheet, "HAM-71-1.81 PID 102790 Bedrock Compressive Strength.xlsx," included in Appendix B for full details of this analysis. They recommended their bedrock compressive strength values to be used for design. Table 6.1.1 summarizes the unit tip resistance, unit side resistance, average RQD, and compression strength of bedrock at each test boring location.

Table 6.1.1 – Estimated Design Parameters for Bedrock encountered at Boring Locations

Boring No.	**Top Bedrock Depth (feet)**	**Top Bedrock Elevation (ft.)**	**Average RQD (%)**	**Middle/Lower Core Layers Compressive Strength (psi)**	**Intact Rock Modulus E_i (psi)**	**Unit Side Resistance (ksf)**	**Unit Tip Resistance (ksf)**
Columns C4/C3/C9							
B-001-0-23	49.5	485.4±	78	350/550	31,500	10	198
Columns C2/C10/C13							
B-002-0-23	39.5	495.6±	55	300/550	27,000	9.6	198
Columns C5/C6/C14							
B-003-0-23	27.0	497.3±	59	400/550	36,000	11	198
Columns C7/C8							
B-004-0-23	14.5	509.9±	56	485/250	43,650	12	90
Column C1/C11/C12/C15							
B-007-3-64	30.0	509.2±	20*	485/250	43,650	12	90
Pier 1							
B-004-0-23	14.5	509.9±	56	485/250	43,650	12	90
Pier 2							
B-005-0-23	13.0	539.0±	71	250/250	22,500	8.7	90

*Assumed

The nominal shaft tip resistance was calculated for the selected shaft diameter from the unit tip resistance by multiplying it with the shaft cross-sectional area. The nominal shaft side resistance was calculated for the selected shaft diameter and socket length from the unit side resistance by multiplying it with the shaft length surface area. The tip resistance portion of the factored axial compression resistance is calculated from the nominal shaft tip resistance by multiplying it with a resistance factor of 0.50. The side resistance portion of the factored axial compression resistance is calculated from the nominal shaft side resistance by multiplying it with a resistance factor of 0.55. Table 6.1.2 summarizes factored resistance for the selected diameter and socket length at the columns and pier locations. Calculations performed as per GDM Section 1306.3.2 indicate that in this case, it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance.

Table 6.1.2 – Estimated Design Parameters for Column and Pier Drilled Shafts

Boring No.	Substructure Location	Top Bedrock Elevation (feet)	Shaft Tip Elevation (feet)	Socket Diameter (feet)	Socket Length (feet)	Factored Tip Resistance (kips)
B-001-0-23	C3	487.2±	482.7	4.5	7.0	1575
B-001-0-23	C4	486.0±	481.5	4.5	7.0	1575
B-001-0-23	C9	491.5±	487.0	4.5	7.0	1575
B-002-0-23	C2	494.8±	490.3	4.5	7.0	1575
B-002-0-23	C10	499.0±	494.5	4.5	7.0	1575
B-003-0-23	C5	494.5±	490.0	4.5	7.0	1575
B-003-0-23	C6	500.5±	496.0	4.5	7.0	1575
B-004-0-23	C7	506.5±	500.5	4.5	7.0	716
B-004-0-23	C8	510.0±	504.0	4.5	7.0	716
B-007-3-64	C1	506.5±	501.25	4.5	7.0	716
B-007-3-64	C11	507.0±	501.75	4.5	7.0	715
B-002-0-23	C13	495.0±	490.5	3.5	5.5	952
B-003-0-23	C14	496.0±	491.5	3.5	5.5	952
B-007-3-64	C12	502.0±	497.5	3.5	5.5	433
B-007-3-64	C15	502.5±	498.0	3.5	5.5	433
B-004-0-23	Pier 1	508.9±	502.9	4.0	6.0	565
B-005-0-23	Pier 2	535.0±	529.0	4.0	6.0	565

Bearing resistance for spread footing on weak rock was evaluated in accordance with GDM Section 1303.3.3 for the proposed Pier 3 footing. Table 6.1.4 summarizes the factored bearing resistance for both Pier 3 footings.

Table 6.1.4 – Estimated Design Parameters for Bridge Pier 3 Footing

Boring No.	Substructure Location	Estimated Top of Bedrock Elev. (feet)	Proposed Bearing Elev. (feet)	Width of Footing (feet)	Factored Bearing Resistance (ksf)
B-006-0-23	Pier 3	564.5±	561.0	10.0	28.8

Bearing capacity analysis was performed by using effective stress parameters to estimate the factored bearing resistance for the footing supported on existing fill soils. The Limit Equilibrium bearing resistance analysis performed by ARC personnel and bearing resistance calculation spreadsheet is included in Appendix B. Table 6.1.5 summarizes the factored bearing resistance for the granular fill soils below bearing elevation.

Table 6.1.5 – Estimated Design Parameters for Ramp Abutment/Retaining Walls Footing

Boring No.	Substructure Location	Estimated Top of Bedrock Elev. (feet)	Proposed Bearing Elev. (feet)	Effective Footing Width (feet)	Factored Bearing Resistance (ksf)
B-002-0-23	East/West Abutments	495.6±	534.0	4.58	4.31
B-002-0-23	Retaining Walls	495.6±	534.0	3.0	4.50

The estimated immediate settlements for the Ramp Abutment and Retaining Walls are summarized in Table 6.1.6. Based on the settlement analyses, the anticipated total settlement on the Ramp Abutment and Retaining Walls footings will be in the order of 0.50 inches and 0.25 inches, respectively. Therefore, it is estimated that the maximum total settlement and differential settlement will not exceed one inch and one-half of an inch, respectively.

Table 6.1.6–Summary of Anticipated Settlement for Ramp Abutment/Retaining Walls Footing

Boring No.	Footing Sizes (feet)	Settlement Type	Estimated Settlement (inches)
B-002-0-23	R. Wall - 3.0X23.5	Consolidation	0.0
		Immediate	0.24
B-002-0-23	AB Wall - 5.05X14.33	Consolidation	0.03
		Immediate	0.52

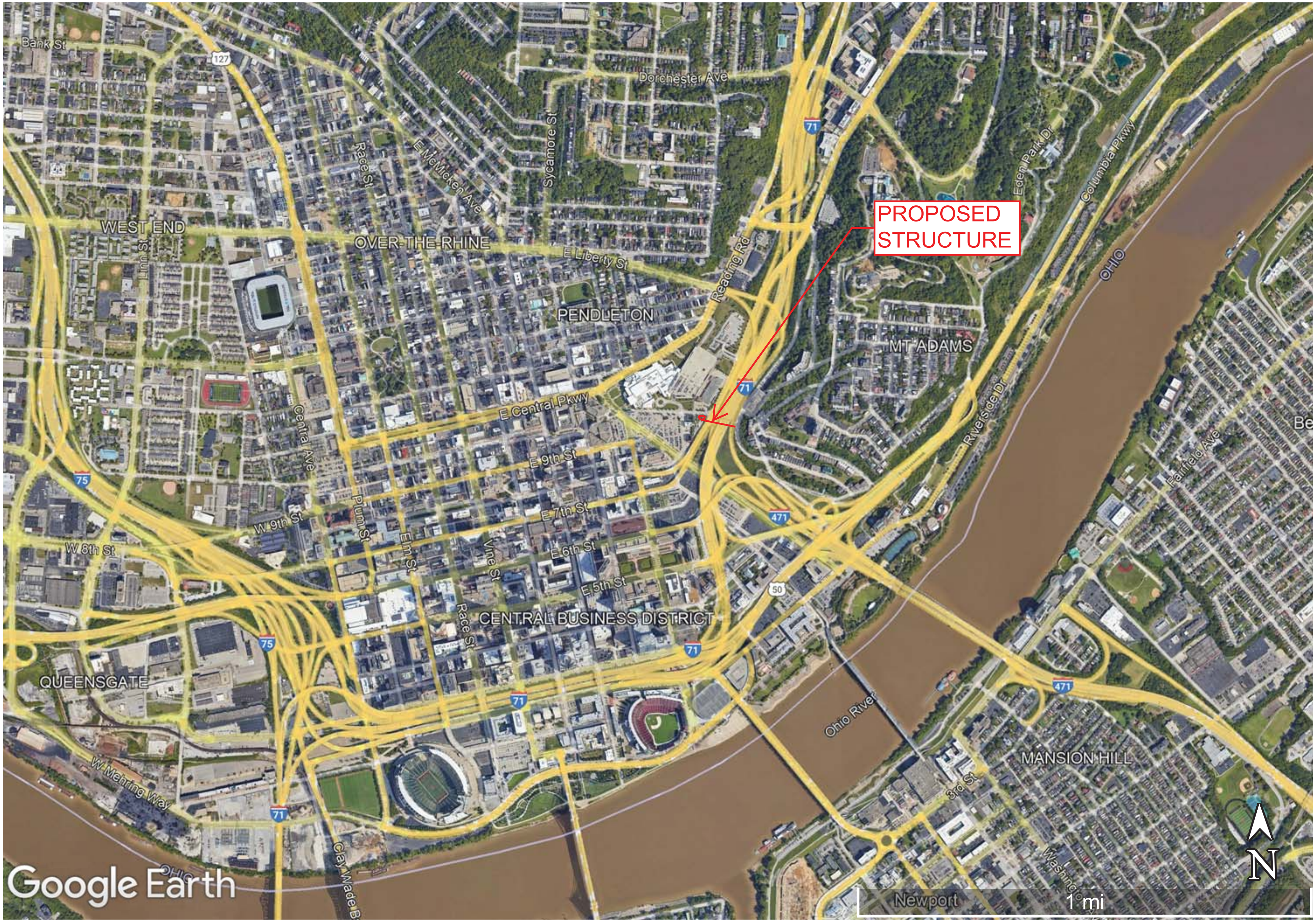
2.0 INTRODUCTION

This report has been prepared for the proposed replacement of Pedestrian Bridge Numbers HAM-71-1.80 & HAM-22-10.93 in Hamilton County, Ohio. It represents the intent of ARCADIS (ARC) the design engineer, and ODOT District 8, the owner, to secure subsurface information at selected locations in accordance with the Ohio Department of Transportation's (ODOT's) *Specifications for Geotechnical Explorations*, and to obtain recommendations regarding geotechnical factors pertaining to the design and construction of this project.

This report has been developed based on the field exploration program, laboratory testing, and information secured for site-specific studies. It must be noted that, as with any exploration program, the site exploration identifies actual subsurface conditions only at those locations where samples were obtained. The data derived through sampling and laboratory testing is reduced by geotechnical engineers and geologists who then render an opinion regarding the overall subsurface conditions and their likely reaction on the site. The actual site conditions may differ from those inferred to exist. Therefore, although a fair amount of subsurface data has been assembled during this exploration, this report may not provide all of the geotechnical data needed for construction of this project. This report was prepared using English units.

2.1 Project Description

Present plans call for the replacement of Pedestrian Bridge Nos. HAM-71-1.80 & HAM-22-10.93 which carries pedestrian traffic from Court Street East, over US 22, I-471 SB, I-71 SB and NB, and I-471 NB to Van Meter Street in Cincinnati Downtown, Ohio. The design information provided by ARC personnel indicates that the existing bridge is a four-span continuous rolled beam with reinforced concrete deck and substructure and single-span plate with reinforced concrete deck and substructure. The total span length of the existing bridges HAM-71-1.80 & HAM-22-10.93 is approximately 318 feet. The proposed replacement structure will consist of Multistory Ramp/Stairway and Pedestrian Bridge. The Multistory Ramp/Stairway will be reinforced concrete slab on cantilevered concrete beams supported on 13 concrete columns. The Bridge will be three simple spans, prefabricated steel truss with concrete deck supported on three (3) wall-type piers. The total length of the Multistory Ramp will be 675 feet. The total length of the bridge from Pier to existing bridge abutment will be 350 feet. The Site Location Map is shown in Figure 2.1.



HAM-71-1.81 & HAM-22-10.93
HIMILTON COUNTY, OHIO
SITE LOCATION MAP (FIG. 2.1)

2.2 Scope of Services

The scope of services for this project was in accordance with Pro Geotech, Inc. (PGI) Proposal No. PG23004 dated March 27, 2023, and was governed by ODOT's *Specifications for Geotechnical Explorations* dated January 14, 2022, ODOT's *Bridge Design Manual*, issued in July 2020 and updated in July 2023 including current AASHTO LRFD specifications, hereafter referred to as ODOT Specifications. Our scope of services consisted of the execution of the following phases:

Phase I – Reconnaissance and Planning, which primarily consisted of planning the field portion of our subsurface exploration, performing the site reconnaissance to evaluate the proposed project site from a geotechnical standpoint, reviewing and compiling all existing geology of the project site obtained from ODOT and ODNR sources, marking the test boring locations, obtaining necessary permits, and notifying the Ohio Utility Protection Services (OUPS) about the proposed drilling operations.

Phase II - Test Boring and Sampling Program, which primarily consisted of field verification of the test boring locations with regards to underground utilities, advancing the test borings at the site, conducting field tests, sampling the subsurface materials, and preparing field drilling logs.

Our scope of services included advancing six (6) test borings at the proposed bridge site for structure foundation design purposes. Two (2) of these test borings were to be advanced within the ODOT Right of Way for the pedestrian bridge design purposes. Four (4) of these test borings were to be advanced at the parking lot for the Ramp/Stairway design purposes. These structural test borings were to be advanced to an approximate depth of 25 feet each below the existing ground surface including obtaining rock core from each boring location. The groundwater conditions were monitored during and upon completion of the drilling operations. PGI provided all the traffic control needed during the fieldwork.

Phase III - Testing Program, which consisted of performing soil classification and engineering properties tests on selected soil and rock samples and classifying the soils in accordance with the ODOT Soil Classification System.

Phase IV - Geotechnical Exploration Report, which included the following:

- A brief description of the project and our exploration methods
- Geology of the site

- Typed drilling logs and laboratory test results
- A description of subsurface soil, rock, and groundwater conditions
- Boring logs showing soil stratigraphy, depths of samples taken, SPT "N" values, SPT (N60) values, existing groundwater conditions, and laboratory test results.
- Recommendations and discussion pertaining to structure foundation design including shallow and deep foundations
- Discussions pertaining to earthwork considerations, groundwater management, and construction monitoring and recommendations for shoring during construction
- Recommendations for shoring during construction
- Preparation of Geotechnical Exploration Plans

The scope of services did not include any environmental assessments for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statement in this report or on the boring logs regarding odors, colors or unusual or suspicious items or conditions is strictly for the client's information.

3.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

3.1 Geology

Based on information obtained from the Physiographic Regions of Ohio map, the bridge site lies within the Outer Bluegrass Region of the Bluegrass Section at approximate elevations ranging from 455 feet to 1120 feet. This Bluegrass Section is located within the Interior Low Plateau Province. The Outer Bluegrass Region is characterized as a dissected plateau of carbonate rocks with moderately high relief (300') with thin, early drift caps and narrow ridges. The geology of the Bluegrass Region generally consists of silt-loam colluvium over Ordovician- and Silurian-age dolomites, limestones, and calcareous shales.

According to Bulletin 44, *Geology of Water in Ohio*, Cincinnati lies on the dissected Lexington peneplain. The area was glaciated by the Illinoian ice sheet and was much modified by outwash from the Wisconsin ice sheet to the north. Based on the *Quaternary Geology of Ohio* and on the natural soils encountered at the site, the main geologic deposit of the project site consists of Illinoian-age lacustrine deposits consisting of massive or laminated clays and silts with potential loess or colluvial cover. According to the *Soil Survey of Hamilton County, Ohio,* the project site is located within the urban area and had incurred cut and fill operations due to construction of existing infrastructure. Thus, the composition of the surface and subsurface soils has changed from natural in most areas. Based on information obtained from the Ohio Geological Survey, top of bedrock in the vicinity of the project site is anticipated to be present at approximate elevations ranging from 485 feet 560 feet. At this elevation, bedrock is expected to consist of Upper Ordovician age Kope Formation shale (75%) and interbedded limestone; gray to bluish gray; contains thin to thick planar bedding. This unit contains sparse fossils, is subject to severe surface weathering and landslides are common where unit crops out.

Based on the Ohio Division of Geological Survey Interactive Map of Ohio Mineral Industries, there are no abandoned underground mines recorded in Hamilton County and there are no active sand and gravel surface mines located within an approximate four (4) mile radius of the site in Ohio. Based on the information obtained from the "Karst Interactive Map of Ohio", there are no suspected karst features within an approximate one (1) mile radius of the project site. Based on the Ohio Division of Geological Survey Interactive Map of Ohio Earthquake Epicenters, three (3) earthquake epicenters are shown in the vicinity of downtown Cincinnati, all with non-instrumental magnitudes ranging from 2.0-2.9. Note that these three (3) earthquakes epicenters are dated 1925, 1936 and 1937.

The above soil and bedrock information has been obtained from the Physiographic Regions of Ohio, printed in April, 1998, *Bedrock Geology and Quaternary Geology online maps from ODNR's Ohio Geology Interactive Map Services*, *Geology of Water in Ohio (Bulletin 44)* issued in 1943 (reprinted in 1968), the *Soil Survey of Hamilton County, Ohio* Web Soil Survey Data Version 22 issued in September 2022 from the *U.S. Department of Agriculture, Natural Resource Conservation Service* website, and the *Covington Quadrangle*, photorevised in 1987.

3.2 Observation of the Project

The reconnaissance of the project site was performed by one of PGI's geotechnical engineers in April 2023. The project site is located in an urban commercial neighborhood with the closest building located within 300 feet from the bridge site. The Ramp/Stairway site area was covered with a concrete slab, asphalt pavement, and riprap stones. Small bushes, shrubs, and a few medium trees were present at the site. The middle and west bridge pier sites will be located in confined areas between Interstate Highways. Buried concrete footings were observed in the vicinity of middle pier site. The concrete pier walls generally appeared to be in fair to good condition. The bottom of the concrete deck generally appeared to be in poor condition. Many areas of spalled concrete and exposed reinforcement were observed. Rust was observed in many places on the exposed steel rebar below the deck.

4.0 EXPLORATION

4.1 Historic and Project Exploration Program

Historical records of geotechnical exploration were available from the ODOT *Transportation Information Mapping System (TIMS)* Website for the existing pedestrian bridge. A Structure foundation exploration was performed in 1964 under the project designation of HAM-71-0157. A total of five (5) historic test borings identified as B-7-1 (B-007-1-64), B-7-2 (B-007-2-64), B-7-3 (B-007-3-64), R-15-2 (B-015-2-64), and R-16-1 (B-016-1-64) are available for this bridge. Locations and ground surface elevations for historic test borings are available. Also, N_{60}-values from SPT tests, soil and rock core descriptions, and rock core recovery were shown on these historic borings. However, RQD information is missing on these historic borings. Still the soil and rock information from historic test borings drilled in the vicinity of proposed bridge will be used to provide recommendations and will be included in Structure foundation exploration sheets. All of the relevant historic information is included in Appendix A.

Current Exploration: In order to explore the subsurface conditions at the project site, drilling, sampling, and field-testing operations were performed in May and June 2023. A total of six (6) project test borings identified as B-001-0-23 through B-006-0-23 were advanced at the project site. Project test borings B-001-0-23 through B-004-0-23 were advanced at the existing parking lot for Ramp/Stairway foundation design purposes. Project test borings B-005-0-23 and B-006-0-23 were advanced within the ODOT Right of Way on both sides of Interstate-71 for bridge foundation design purposes. These test borings were advanced to approximate depths ranging from 22.0 to 60.0 feet below the existing pavement, riprap, or ground surface. All test borings were advanced in accordance with ODOT Specifications for Geotechnical Explorations (SGE). The test boring locations are shown on the "Boring Locations Plan" included in Appendix A.

The test borings were marked in the field by PGI based on boring location plans developed by PGI personnel and approved by ARC personnel. Site geometry, existing structure foundations, utility locations, overhead height, and accessibility were also taken into account when locating the test borings. At the time of test boring location selection, the vertical soil sampling intervals were determined based on the needs for design and construction of the project. A CME 55 ATV mounted drill rig was used to advance the test borings. Borings were advanced using 3.25-inch inside diameter continuous flight hollow stem augers (HSA). Representative disturbed samples of the soils were collected at intervals in accordance with the ODOT Specifications. A standard 2.0-inch outside diameter split-barrel sampler was driven into the soil by means of a 140-lb hammer falling freely through a distance of 30-inches in accordance with the Standard Penetration Test (ASTM D 1586). Where bedrock was encountered, all test borings were advanced and the rock was sampled using a type NQ2 series core barrel, water method. All project test borings were monitored for the presence of groundwater during drilling operations and before rock coring operations. These test borings were backfilled with soil cuttings, mixture of bentonite/soil cuttings, grout seal. Where the pavement encountered, it was capped with 12 inches of asphalt cold patch upon completion of backfilling operations. A certified traffic control company was hired to provide traffic control needed during drilling for project test borings B-005-0-23 and B-006-0-23.

The N-values (N_m) as measured in the field have been corrected to equivalent rod energy ratio of 60% (N_{60}) in accordance with ODOT's *Specifications for Geotechnical Explorations*. Drill Rig hammer system was calibrated by energy testing in accordance with ASTM D4633 and drill rod energy ratio; ER was determined. Automatic Hammer was calibrated on 1/13/2023 for CME 55 ATV (Track) drill Rig with Drill Rod Energy Ratio of 97.1%. The measured N-values (N_m) were corrected to equivalent rod energy ratio of 60 percent, N_{60}, using the equation: $N_{60} = N_m \text{ x } (ER/60)$.

Station, offset and surface elevations at the drilled test boring locations were provided to PGI by ARC personnel. The typed drilling logs are included in Appendix A. These logs show the SPT resistance values (N-values) for each soil sample taken in the test borings and present the classification and description of soils encountered at various depths in the test borings. The sample depth shown on the logs and laboratory test results indicate the top of each sampling or testing interval.

4.2 Laboratory Testing Program

All soil and rock samples obtained during the drilling and sampling operations were returned to PGI's geotechnical soils laboratory in Cleveland, Ohio. Upon arrival, the samples were visually examined and classified by a geotechnical engineer and a geologist to verify the classifications made in the field and to note any additional characteristics which may not have been observed in the field.

Moisture content determination tests were performed on all soil samples as per ODOT specifications. Additional laboratory soil tests were performed on selected soil and rock core samples for the purpose of soil classification and for analysis of engineering characteristics. These tests consisted of Particle Size Analysis and Atterberg Limits. Laboratory rock tests were performed on selected rock core samples. These tests consisted of Compressive Strength of Rock Core and Point Load Strength of Rock Core. All laboratory tests were performed in accordance with the ASTM or other standards listed in "Laboratory Test Standards" located in Appendix B. The results of the laboratory tests are also included in Appendix B. The soils were classified in accordance with the ODOT Soil Classification System, a description of which is also included in Appendix B.

Upon completion of the laboratory testing, all samples were placed in storage at PGI's Cleveland facility. Unless otherwise requested in writing, the soil and bedrock core samples will be retained through completion of ODOT review of Stage 2 plans.

5.0 FINDINGS

5.1 Surficial and Subsurface Soil Conditions

The surficial and subsurface soil conditions in the vicinity of the proposed structures were determined from project test borings B-001-0-23 through B-006-0-23 and historic test borings identified as B-007-1-64, B-007-2-64, B-007-3-64, B-015-2-64, and B-016-1-64.

The surficial soil conditions in the vicinity of the proposed structures were determined from project test borings B-001-0-23 through B-006-0-23. Project test boring B-001-0-23 was advanced through 12 inches of driveway asphalt pavement. Project test boring B-002-0-23 was advanced through riprap stone with the thickness of 12 inches. Project test boring B-003-0-23 and B-005-0-23 were advanced through concrete slab with the thickness 8 inches and 15 inches, respectively. Project test boring B-004-0-23 was advanced through asphalt pavement underlain by aggregate base. The approximate thickness of the asphaltic concrete and aggregate base was 1 inch and 10 inches, respectively. Test boring B-006-0-23 was advanced through 3.0 inches of topsoil.

Project test Borings: The subsurface soils encountered in project test borings consisted of both fill and natural soils above the bedrock. Fill soils were encountered in all project test borings with the exception of B-006-0-23 where bedrock was encountered below topsoil. The fill soils were encountered in project test borings B-001-0-23 through B-005-0-23 to depths ranging from 3.0 feet to 38.5 feet below the existing pavement, riprap stone or concrete surface. The bottom of fill soils layer was encountered at approximate elevations ranging from 496.4 feet to 549.0 feet. Fill soils encountered were both cohesive and non-cohesive and consisted of silty clay (A-6b), silt and clay (A-6a), sandy silt (A-4a), coarse and fine sand (A-3a), gravel and stone fragments with sand (A-1-b), asphalt, cinders, or stone fragments (A-1-a), stone fragments with sand and silt (A-2-4), and multiple combinations of concrete, asphalt, brick and stone fragments with sand. The natural soils encountered below fill soil in the project test borings consisted of silt and clay (A-6a), silty clay (A-6b) and clay (A-7-6). Bedrock consisting of shale interbedded limestone was encountered in all project test borings below approximate depths ranging from 0.25 feet (Elevation 564.5 feet) to 49.5 feet (Elevation 485.4 feet) below the existing surface.

The laboratory test results indicated that the moisture contents of the tested cohesive soil samples obtained from the structure test borings ranged from 5% to 42% and the consistency of these soils ranged from "soft" to "hard" with the majority ranging from "medium stiff" to "hard". The laboratory test results indicated that the moisture contents of the tested non-cohesive soils ranged from 3% to 54% and the relative density of these soils ranged from "very loose" to "dense" with the majority ranging from "loose" to "medium dense". The majority of the cohesive soil samples that were tested for Atterberg limits had natural moisture contents less than their plastic limits, however two samples (B-001-0-23 at 38.5 feet, and B-003-0-23 at 8.5 feet) had moisture contents greater than their plastic limits; and one sample (B-003-0-23 at 16.0 feet) had a moisture content greater than its liquid limit.

Historic Test Borings: The subsurface soil conditions encountered in the vicinity of the proposed structures were determined from the soil information obtained from historic test borings B-007-1-64, B-007-2-64, B-007-3-64, B-015-2-64, and B-016-1-64. Fill soils were encountered in historic test borings B-007-1-64, B-007-2-64, and B-007-3-64 to approximate depths ranging from 1.5 feet to 27.0 feet below the existing concrete or ground surface and no fill soils were encountered in historic test borings B-015-2-64, and B-016-1-64. Fill soils encountered consisted of silty clay (A-6b), silt and clay (A-6a), cohesive sandy silt (A-4a), and combinations of concrete, cinders, and brick with clay. The natural soils encountered below fill soils and above bedrock in the project test borings consisted of silt and clay (A-6a), silty clay (A-6b) and clay (A-7-6). Bedrock consisting of shale interbedded limestone was encountered in all project test borings below approximate depths ranging from 7.5 feet (Elevation 590.6 feet) to 30.0 feet (Elevation 509.2 feet) below the existing ground surface.

The laboratory test results indicated that the moisture contents of the tested cohesive soil samples obtained from the structure test borings ranged from 12% to 25% and the consistency of these soils ranged from "soft" to "hard" with the majority ranging from "medium stiff" to "very stiff". The relative density of these soils ranged from "medium dense" to "very dense". The majority of the cohesive soil samples that were tested for Atterberg limits had natural moisture contents less than their plastic limits, however one sample (B-016-1-64 at 0.5 feet) had a moisture content equal to its plastic limit; one sample (B-007-2-64 at 2.5 feet) had moisture content greater than their plastic limit.

General: For specific conditions of the project and historic test borings at various depths, please refer to the individual test boring logs located in Appendix A of this report. For complete moisture contents and Atterberg limit test results for project test borings, refer to the laboratory test results located in Appendix B.

5.2 Bedrock Conditions

Project Test Borings: Bedrock was encountered in all project test borings to the termination depths. Bedrock was split spoon sampled until little or no penetration or recovery was encountered. Generally, coring was attempted when the split-spoon sampler indicated very little penetration and recovery. Bedrock core samples were then obtained using NQ2 diamond impregnated core barrels. The coring operations were performed in accordance with the procedure for Diamond Core Drilling for Site Investigations (ASTM D 2113). The core samples consisted of gray shale with interbedded limestone. The shale encountered across the site was thinly laminated, generally highly to moderately fractured, and was calcareous and effervesced freely with dilute hydrochloric acid and ranged from severely to moderately weathered and was very weak to weak. The percentage of interbedded limestone encountered in the individual bedrock core runs ranged from 1% to 42% and averaged 13%. The limestone encountered within the shale ranged from gray to black or dark gray to white. White limestone generally indicated the presence of fossils. The limestone ranged from crystalline to clastic and ranged from slightly to highly weathered but was generally moderately weathered and ranged from very weak to moderately strong but was generally moderately strong. No slicken sides were observed, and the fractures were typically tight to narrow and slightly rough to very rough. The Rock Quality Designation (RQD) obtained for the bedrock core samples varied from 0 to 92% and the recovery ranged from 43 to 100% for the individual rock core runs. Based on the laboratory testing performed on the rock core samples, the point load strengths of the rock core specimens in these project test borings ranged from 33 psi to 1539 psi which characterizes them as “very weak” to “slightly strong”. The compressive strengths of the rock core specimens in these project test borings ranged from 182 psi to 1030 psi which characterizes them as “very weak” to “weak”. The Rock Mass Rating for the bedrock core specimens obtained from project test boring B-001-0-23 was 42% and is considered as “fair” rock while the Rock Mass Rating for the bedrock core specimens obtained from project test boring B-002-0-23 was 37% and is considered as “poor” rock. The Rock Mass Rating for the bedrock core specimens obtained from project test borings B-003-0-23 and B-004-0-23 was 42 each and is considered as “fair” rock. The Rock Mass Rating for the bedrock core specimens obtained from project test boring B-005-0-23 was 40% and is considered as “poor” rock while the Rock Mass Rating for the bedrock core specimens obtained from project test boring B-006-0-23 was 31% and is considered as “poor” rock. It appears that the top bedrock surface slopes down from east to west at an approximate angle of 11 degrees. Table 5.2.1 summarizes the elevation, length, recovery, and RQD for each rock core run obtained at the project test borings. Tables 5.2.2 and 5.2.3 summarize the results of compressive strength tests performed at the laboratory on the different rock core specimens at various depths. Refer to the drilling logs, soil profile, and rock core photos in the Appendix for additional bedrock information. Also refer to "Bedrock Descriptions" in Appendix B for general bedrock information.

Table 5.2.1 – Bedrock Core Information for Project Test Borings

Boring Number	Rock Core Run No.	Rock Core Elevations (ft)	Rock Core Depths (ft)	Length of Core Run (ft)	Recovery (%)	RQD (%)
B-001-0-23	NQ2-1	484.9 to 483.9	50.0 to 51.0	1.0	100	0
	NQ2-2	483.9 to 478.9	51.0 to 56.0	5.0	82	72
	NQ2-3	478.9 to 474.9	56.0 to 60.0	4.0	92	92
B-002-0-23	NQ2-1	495.1 to 490.1	40.0 to 45.0	5.0	95	47
	NQ2-2	490.1 to 485.1	45.0 to 50.0	5.0	98	63
B-003-0-22	NQ2-1	496.6 to 493.1	27.5 to 31.0	3.5	100	79
	NQ2-2	493.1 to 488.1	31.0 to 36.0	5.0	85	40
	NQ2-3	488.1 to 486.6	36.0 to 37.5	1.5	100	78
B-004-0-23	NQ2-1	509.7 to 508.9	14.7 to 15.5	0.8	60	0
	NQ2-2	508.9 to 503.9	15.5 to 20.5	5.0	100	62
	NQ2-3	503.9 to 498.9	20.5 to 25.5	5.0	100	63
B-005-0-23	NQ2-1	535.0 to 531.0	17.0 to 21.0	4.0	94	69
	NQ2-2	531.0 to 526.0	21.0 to 26.0	5.0	100	89
	NQ2-3	526.0 to 524.3	26.0 to 27.7	1.7	100	20
B-006-0-23	NQ2-1	557.8 to 552.8	7.0 to 12.0	5.0	43	33
	NQ2-2	552.8 to 547.8	12.0 to 17.0	5.0	100	69
	NQ2-3	547.8 to 542.8	17.0 to 22.0	5.0	100	60

Elevations were provided by ARC Personnel

Table 5.2.2 – Point Load Strength Test Results of Rock Core Specimens

Boring No.	Specimen Depth (ft)	Specimen Elevation (ft)	Point Load Index (psi)	UCS (psi)
B-001-0-23	51.0 - 56.0	483.9 – 478.9	128.21	1539
B-001-0-23	56.0 - 60.0	478.9 – 474.9	21.90	263
B-002-0-23	40.0 - 45.0	495.1 – 490.1	21.63	260
B-002-0-23	45.0 - 50.0	490.1 – 485.1	72.27	867
B-003-0-23	27.5 – 31.0	496.6 – 493.1	52.89	635
B-003-0-23	31.0 – 36.0	493.1 – 488.1	95.68	1148
B-004-0-23	15.5 – 20.5	508.9 – 503.9	58.69	704
B-004-0-23	20.5 – 25.5	503.9 – 498.9	15.39	185
B-005-0-23	17.0 – 21.0	535.0 – 531.0	6.07	73
B-005-0-23	21.0 – 26.0	531.0 – 526.0	20.01	240
B-006-0-23	12.0 - 17.0	552.8 – 547.8	2.76	33
B-006-0-23	17.0 - 22.0	547.8 – 542.8	29.18	350

UCS – Unconfined Compressive Strength

Table 5.2.3 – Compressive Strength Test Results of Rock Core Specimens

Boring No.	Specimen Depth (ft)	Rock Type	Unit Weight (pcf)	CS (psi)
B-001-0-23	53.3	Shale	130.1	356
B-001-0-23	59.0	Shale	145.1	872
B-002-0-23	42.7	Shale	130.4	353
B-002-0-23	46.7	Shale	134.8	220
B-003-0-23	28.5	Shale	130.2	182
B-003-0-23	33.0	Shale	146.6	958
B-004-0-23	19.5	Shale	146.7	1030
B-004-0-23	22.8	Shale	140.5	374
B-005-0-23	21.7	Shale	140.9	439
B-005-0-23	24.2	Shale	139.0	359
B-006-0-23	8.5	Shale	133.3	187
B-006-0-23	14.7	Shale	138.1	261

CS - Compressive Strength

Historic Test Borings: Bedrock was encountered in all historic test borings B-007-1-64, B-007-2-64, B-007-3-64, B-015-2-64, and B-016-1-64. The core samples consisted of gray shale with interbedded limestone. The shale encountered was generally gray, weathered and was calcareous and moderately to slightly tough. The percent of interbedded limestone encountered in the individual bedrock core runs ranged from 5% to 27%. The recovery ranged from 43 to 88% for the individual rock core runs. Table 5.2.4 summarizes the elevation, length, and recovery for each rock core run obtained from historic test borings.

Table 5.2.4 – Bedrock Core Information for Historic Test Borings

Boring Number	Rock Core Run No.	Rock Core Elevations (ft)	Rock Core Depths (ft)	Length of Core Run (ft)	Recovery (%)	RQD (%)
B-007-1-64	NXM-11	513.8 to 508.8	27.0 to 32.0	5.0	73	
	NXM-12	508.8 to 503.8	32.0 to 37.0	5.0	72	
B-007-2-64	NXM-9	521.5 to 516.5	20.0 to 25.0	5.0	67	
	NXM-10	516.5 to 511.5	25.0 to 30.0	5.0	71	
B-007-3-64	NXM-15	505.2 to 500.2	34.0 to 39.0	5.0	60	
B-015-2-64	NXM-5	547.5 to 542.5	9.0 to 14.0	5.0	43	
	NXM-6	542.5 to 537.5	14.0 to 19.0	5.0	75	
B-016-1-64	NXM-11	573.1 to 568.1	25.0 to 30.0	5.0	54	
	NXM-12	568.1 to 563.1	30.0 to 35.0	5.0	73	
	NXM-13	563.1 to 558.1	35.0 to 40.0	5.0	82	
	NXM-14	558.1 to 553.1	40.0 to 45.0	5.0	88	
	NXM-15	553.1 to 548.1	45.0 to 46.5	1.5	83	

5.3 Groundwater Conditions

Groundwater levels were measured at the project test boring locations during and upon completion of drilling operations. In project test borings, no readings were taken upon completion of drilling due to water added to the boreholes during the rock coring operations. The results of these measurements are summarized in Table 5.3.1. It should be noted that groundwater elevations are subject to seasonal fluctuations. All test borings were backfilled immediately upon completion of drilling for safety purposes.

Table 5.3.1 – Groundwater Information

Test Boring	Surface Elevation (ft)	Depth of Groundwater		Groundwater Elevation	
		During Drilling	Upon Completion	During Drilling	Upon Completion
B-001-0-23	534.9	22.5’	NR	512.4	NR
B-002-0-23	535.1	22.0’	NR	513.1	NR
B-003-0-23	524.1	12.5’	NR	511.6	NR
B-004-0-23	524.4	Dry	NR	Dry	NR
B-005-0-23	583.6	Dry	NR	Dry	NR
B-006-0-23	585.2	Dry.	NR	Dry	NR

Elevations were provided by ARC personnel, NR = No Reading

6.0 ANALYSIS AND RECOMMENDATIONS

Based upon the findings of the field exploration program, laboratory testing, and subsequent engineering analysis, the following sections have been prepared to address the geotechnical aspects related to the design and construction of the proposed replacement pedestrian Bridge Nos. HAM-71.1.80 & HAM-22-10.93. Design information provided by the ARC personnel indicates that the proposed replacement structures consisted of Ramp/Stairway and Pedestrian Bridge. Ramp/Stairway consisted of reinforced concrete slab on cantilevered concrete beams on concrete columns and will be used to climb to access the bridge. Pedestrian Bridge will be prefabricated steel truss with concrete deck and will be used to cross over Gilbert Ave., I-471 SB, I-71 SB and NB, and I-471 NB to access the Van Meter Street. Ramp/Stairway structure will be supported on ramp columns identified as C1 through C11 and stair columns identified as C12 through C15 and Pedestrian Bridge will be supported on 3 pier caps and wall identified as Pier 1 through Pier 3. The foundation recommendations are provided in accordance with the ODOT’s Bridge Design Manual, issued in July 2020 and updated in July 2023 and AASHTO LRFD Bridge Design Specifications, current Edition.

6.1 Structure Foundation Systems

Drilled Shafts: Soil and bedrock information obtained from test borings from project test borings B-001-0-23 through B-004-0-23 and historic test borings B-007-1-64 and B-007-3-64 were used to provide foundation recommendations for the proposed Ramp/Stairway and Bridge Pier 1and 2. Project test boring B-001-0-23 was advanced in the vicinity of the proposed Columns C4/C3/C9 while project test boring B-002-0-23 was advanced in the vicinity of the proposed Columns C2/C10/C13. Project test boring B-003-0-

23 was advanced in the vicinity of the proposed Columns C5/C6/C14 while project test boring B-004-0-23 was advanced in the vicinity of the proposed Columns C7/C8/Pier 1. Historic test boring B-007-3-64 was drilled in the vicinity of the proposed Columns C1/C11/C12/C15 while project test boring B-005-0-23 and historic test boring B-015-2-64 were advanced in the vicinity of the proposed Pier 2. As outlined in Section 5.1 - "Subsurface Soil Conditions", overburden soils encountered above the bedrock consisted of both fill soils and natural soils in these project and historic test borings. Fill soils were encountered to depths ranging from 3.0 feet to 38.5 feet below the existing ground surface. Most of the N_{60} values obtained from Standard Penetration Test in fill soil layers were less than 10 and appeared to be uncontrolled fill. These fill materials should not be used to support the column and pier loads. The top of bedrock was encountered at an approximate depth of 49.5 feet (at elevation of 485.4 feet) below the existing asphalt pavement surface in project test boring B-001-0-23, at an approximate depth of 39.5 feet (at elevation of 495.6 feet) below the existing riprap stone surface in project test boring B-002-0-23, at an approximate depth of 27.0 feet (at elevation of 497.3 feet) below the existing concrete surface in project test boring B-003-0-23, at an approximate depth of 14.5 feet (at elevation of 509.9 feet) below the existing asphalt pavement surface in project test boring B-004-0-23, at an approximate depth of 30.0 feet (at elevation of 509.2 feet) below the existing ground surface in Historic test boring B-007-3-64, and at an approximate depth of 13.0 feet (at elevation of 539.0 feet) below the concrete surface in project test boring B-005-0-23. Bedrock consisted of shale interbedded limestone to termination depth in these project and historic test borings. The Rock Quality Designation (RQD) for the core samples in these project test borings ranged from 55% to 86%. Based on the laboratory testing performed on the rock core samples, the point load strengths of the rock core specimens in these project test borings ranged from 73 psi to 1539 psi which characterizes them as “very weak” to “weak”. The compressive strengths of the rock core specimens in these project test borings ranged from 182 psi to 1030 psi which characterizes them as “weak”. The Rock Mass Rating for the bedrock core specimens obtained from project test borings ranges from 37 to 42 and is considered as “poor” rock to “fair” rock.

Since bedrock was encountered in most of these project and historic test borings at relatively deeper depths, deep foundation consisting of drilled shafts may be used to transfer the design loads to the underlying competent bedrock at the proposed column and pier locations. Based on the bridge site plan, the bottoms of the shaft caps of proposed columns will be placed at elevations ranging from 523.0 feet to 536.6 feet. Design information provided by ARC personnel indicate that the maximum compression design loads along a vertical axial direction at the Strength and Service Limits will be 170 kips per shaft and 120 kips, respectively at Columns C1 through C13 locations. The maximum compression design loads along a vertical axial direction at the Strength and Service Limits will be 646 kips per shaft and 628 kips, respectively at Bridge Pier 1 location. The maximum compression design loads along a vertical axial direction at the Strength and Service Limits will be 654 kips per shaft and 641 kips, respectively at Bridge Pier 2 location. The unit shaft side and tip resistances on bedrock were calculated based on the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD) Article 10.8.3.5.4. For the calculation of shaft side resistances, Equation 10.8.3.5.4b-1 was used since the shale bedrock encountered in the borings is not expected to cave during construction. For the calculation of shaft tip resistances, Equation 10.8.3.5.4c-1 was used to determine tip resistance because the rock below the bearing elevation is considered tightly jointed and without seams of compressible material. Based on these equations, unit shaft tip resistance and unit shaft side resistance were calculated for the bedrock encountered at the test boring locations. The rock intact elastic modulus was estimated form equation $E_i = 90\ q_u$, based on correlation with Engineering Properties for Intact Rocks (after Deere, 1968; Peck, 1976; and Horvath and Kenney, 1979).

Since there are a lot of compressive strength testing results and a large variation in the results, OGE performed Bedrock Compressive Strength analyses and compared this to the actual testing results in the report, and find reasonable agreement. However, due to the scatter of the results (particularly towards the lower elevations), OGE recommend using the minimum of this function or the average of the strength testing results. Refer to OGE spreadsheet, "HAM-71-1.81 PID 102790 Bedrock Compressive Strength.xlsx," included in Appendix B for full details of this analysis. They recommended their bedrock compressive strength values to be used for design. These values are shown Table 6.1.1. The bedrock was divided into three layers per boring location: an upper highly weathered layer, and middle cored and lower cored layers (each approximately 5 feet long), in accordance with the augered rock and the typical rock core runs, and rock strength testing performed. The upper highly weathered layer will moderate the sudden change in stiffness from soil to bedrock that results in an unrealistic shear stress concentration. The bedrock strength in the upper highly weathered layer was estimated based on the Stark method, in accordance with GDM Section 404.3. The two rock core layers were applicable to the drilled shaft side and tip resistance, respectively. Table 6.1.1 summarizes the unit tip resistance, unit side resistance, average RQD, and compression strength of middle and lower bedrock layer at each test boring location. The unit shaft side resistance and shaft tip resistance calculation spreadsheets performed by OGE and PGI are included in Appendix B.

Table 6.1.1 – Estimated Design Parameters for Bedrock encountered at Boring Locations

Boring No.	Top Bedrock Depth (feet)	Top Bedrock Elevation (ft.)	Average RQD (%)	Middle/Lower Rock Layers Compressive Strength (psi)	Intact Rock Modulus E_i (psi)	Unit Side Resistance (ksf)	Unit Tip Resistance (ksf)
Columns C4/C3/C9							
B-001-0-23	49.5	485.4±	78	350/550	31,500	10	198
Columns C2/C10/C13							
B-002-0-23	39.5	495.6±	55	300/550	27,000	9.6	198
Columns C5/C6/C14							
B-003-0-23	27.0	497.3±	59	400/550	36,000	11	198
Columns C7/C8							
B-004-0-23	14.5	509.9±	56	485/250	43,650	12	90
Column C1/C11/C12/C15							
B-007-3-64	30.0	509.2±	20*	485/250	43,650	12	90
Pier 1							
B-004-0-23	14.5	509.9±	56	485/250	43,650	12	90
Pier 2							
B-005-0-23	13.0	539.0±	71	250/250	22,500	8.7	90

*Assumed

The nominal shaft tip resistance was calculated for the selected shaft diameter from the unit tip resistance by multiplying it with the shaft cross-sectional area. The nominal shaft side resistance was calculated for the selected shaft diameter and socket length from the unit side resistance by multiplying it with the shaft length surface area. The tip resistance portion of the factored axial compression resistance is calculated from the nominal shaft tip resistance by multiplying it with a resistance factor of 0.50. The side resistance portion of the factored axial compression resistance is calculated from the nominal shaft side resistance by multiplying it with a resistance factor of 0.55. Side resistance from the soil overburden and upper two (2) feet of the bedrock can be ignored. Table 6.1.2 summarizes factored resistance for the selected diameter and socket length at the columns and pier locations. For the Ramp/Stairway Columns and Piers 1 & 2, the factored resistance at the tip was selected for the designing drilled shafts. Based on the factored axial compression resistance for the selected shaft socket length and diameter, the estimated maximum total settlement and differential settlement will not exceed one inch and one-half inch, respectively. The shaft factored resistance calculation spreadsheets are included in Appendix B. Refer to OGE spreadsheet, “HAM-71-1.81 PID 102790 Drilled Shaft Calculation Check” included in Appendix B for a full analysis of the drilled shaft side and tip resistance. Based on the GDM Section 1306.1.2, tip resistance or side resistance must be selected but not both. Calculations performed as per GDM Section

1306.3.2 indicate that in this case, it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. These calculations sheets are included in Appendix B.

Table 6.1.2 – Estimated Design Parameters for Column and Pier Drilled Shafts

Boring No.	Substructure Location	Top Bedrock Elevation (feet)	Shaft Tip Elevation (feet)	Socket Diameter (feet)	Socket Length (feet)	Factored Tip Resistance (kips)
B-001-0-23	C3	487.2±	482.7	4.5	7.0	1575
B-001-0-23	C4	486.0±	481.5	4.5	7.0	1575
B-001-0-23	C9	491.5±	487.0	4.5	7.0	1575
B-002-0-23	C2	494.8±	490.3	4.5	7.0	1575
B-002-0-23	C10	499.0±	494.5	4.5	7.0	1575
B-003-0-23	C5	494.5±	490.0	4.5	7.0	1575
B-003-0-23	C6	500.5±	496.0	4.5	7.0	1575
B-004-0-23	C7	506.5±	500.5	4.5	7.0	716
B-004-0-23	C8	510.0±	504.0	4.5	7.0	716
B-007-3-64	C1	506.5±	501.25	4.5	7.0	716
B-007-3-64	C11	507.0±	501.75	4.5	7.0	715
B-002-0-23	C13	495.0±	490.5	3.5	5.5	952
B-003-0-23	C14	496.0±	491.5	3.5	5.5	952
B-007-3-64	C12	502.0±	497.5	3.5	5.5	433
B-007-3-64	C15	502.5±	498.0	3.5	5.5	433
B-004-0-23	Pier 1	508.9±	502.9	4.0	6.0	565
B-005-0-23	Pier 2	535.0±	529.0	4.0	6.0	565

Drilled shaft socket diameters less than 36 inches are not recommended. The drilled shafts should be spaced at a minimum of 2.5 shaft diameters on center. If drilled shafts are socketed into bedrock, group effect between shafts may be neglected. The diameter of bedrock sockets must be 6 inches less than the diameter of the shaft above bedrock elevation in accordance with Section 305.4.4.2 of the *2020 ODOT Bridge Design Manual*. The drilled shaft supported piers may experience horizontal movement caused by lateral loads and overturning moments. A lateral load analysis should be performed using LPILE computer program by Ensoft or similar computer program for selected shaft diameter and socket length to check whether lateral resistance is adequate to support lateral loads and overturning moments. Table 6.1.3 summarizes the weak rock parameters to perform lateral load analyses by ARC personnel. Refer to OGE spreadsheet, “HAM-71-1.81 PID 102790 Bedrock p-y Properties” included in Appendix B for recommended properties.

Table 6.1.3 - Estimated Weak Rock Parameters for Lateral Load Analyses

Boring No.	Bedrock Layer No.	Top Bedrock Elev.(ft)	Eff. Unit Weight (pcf)	Compressive Strength (psi)	RQD (%)	Joint Condition	E_i Modulus (psi)	E_m Modulus (psi)	K_rm
B-001-0-23	1	485.4±	66.4	190	10	Open	17000	680	0.00050
	2	484.9±	91.8	350	82	Closed	32000	25600	0.00050
	3	479.9±	96.3	550	92	Closed	50000	45000	0.00035
B-002-0-23	1	495.6±	63.7	125	10	Open	11000	440	0.00050
	2	495.1±	76.8	300	47	Closed	27000	3240	0.00050
	3	490.1±	91.7	550	63	Closed	50000	25000	0.00035
B-003-0-23	1	497.3±	62.0	95	10	Open	8550	342	0.00050
	2	496.6±	90.7	400	67	Closed	36000	21600	0.00049
	3	491.6±	82.7	550	51	Closed	50000	7500	0.00035
B-004-0-23	1	509.9±	66.4	190	10	Open	17000	680	0.00050
	2	508.9±	90.7	485	62	Closed	44000	22000	0.00040
	3	503.9±	77.1	250	50	Closed	23000	3450	0.00050
B-005-0-23	1	539.0±	58.6	55	10	Open	4950	198	0.00050
	2	535.0±	88.8	250	75	Closed	23000	17250	0.00050
	3	530.0±	89.3	250	84	Closed	23000	18400	0.00050
B-006-0-23	1	564.5±	55.9	35	10	Open	3150	126	0.00050
	2	561.0±	59.6	64	10	Open	5760	230.4	0.00050
	3	557.8±	71.4	180	33	Closed	16000	1440	0.00050
	4	552.8±	85.4	200	69	Closed	18000	10800	0.00050
	5	547.8±	85.7	250	60	Closed	23000	11500	0.00050
B-007-3-64	1	506.6±	64.4	140	10	Open	13000	520	0.00050
	2	502.6±	90.7	485	60	Closed	44000	22000	0.00040
	3	497.6±	77.1	250	50	Closed	23000	3450	0.00050

Selecting the construction method for installing the drilled shafts is the responsibility of the contractor. Seepage of water into the drilled shaft holes will occur within the soil overburden during installation. If water is encountered at the bottom of the hole due to seepage, care should be taken to remove all water before placing concrete. The successful performance of a drilled shaft depends on the construction method used as well as the quality of workmanship during installation. Therefore, qualified geotechnical personnel should be present during construction for inspection in order to assure the quality of the drilled shafts and to verify that the rock conditions are as per boring logs. Drilled shaft bottoms should be free of all loose material prior to placement of concrete. For detailed drilled shaft construction, refer to Item 524 – "Drilled Shafts" of the ODOT *Construction and Material Specifications* issued on January 2023. For drilled shafts supporting an axial load, BDM plan note 606.8-1 is needed to include for drilled shafts socketed into rock. If only tip resistance or side resistance is used in the rock socket, modify BDM plan note 606.8-1 accordingly.

Spread Footing

Pier 3 Footings**:** Soil and bedrock information obtained from project test boring B-006-0-23 and historic test boring B-016-1-64 was used to provide foundation recommendations for the proposed Bridge Pier 3. Project test boring B-006-0-23 and historic test boring B-016-1-64 were advanced in the vicinity of the proposed Pier 3. As outlined in Section 5.1 - "Subsurface Soil Conditions", the top of bedrock was encountered at an approximate depth of 0.25 feet (at elevation of 564.5 feet) below the existing ground surface in project test boring B-006-0-23 and at an approximate depth of 7.5 feet (at elevation of 590.6 feet) below the existing ground surface in historic test boring B-016-1-64. Bedrock consisted of shale interbedded limestone to termination depth in this project and historic test borings.

Since bedrock was encountered in these project and historic test borings at relatively shallow depth, shallow foundation system consisting of spread footing may be used to transfer the design loads to the underlying competent bedrock at the proposed Pier location. The bottom elevation of spread footing of the proposed Pier will be placed at an elevation 561.0 feet based on the competent bedrock encountered in project test boring B-006-0-23. Design information provided by ARC personnel indicates that the maximum compression design loads along a vertical axial direction at the Service and Strength Limits will be 7.16 ksf and 13.46 ksf, respectively at proposed Pier 3 location. The size of the spread footing will be 9'X23.5'at the proposed Pier 3 location. Bearing resistance for spread footings on rock was evaluated as per GDM Section 1303.3.3. The rock parameters and bearing resistance calculation spreadsheets are included in Appendix B. Table 6.1.4 summarizes the factored bearing resistance on rock below bearing elevation at pier location. A Resistance Factor (ϕ) of 0.45 should be applied to compute the Factored Bearing Resistance at the Strength Limit State. A Resistance Factor (ϕ) of 1.0 should be used to compute the Factored Bearing Resistance at the Service Limit State.

Settlement of the proposed footing at the pier location will be due to elastic compression of bedrock. Based on the AASHTO LRFD Table C10.6.2.5.1-1, the total settlement is limited to one inch for presumptive bearing resistance of 20 ksf at the Service Limit State for weathered or broken bedrock of shale. This means the factored bearing resistance should be limited by the service limit state with presumptive bearing resistance of 20 ksf and the calculated nominal bearing resistance was exceeded the above value. Therefore, it is estimated that the maximum total settlement and differential settlement will not exceed one inch and one-half of an inch, respectively. Since the proposed spread footing will be placed on relatively level ground, and shear failure is not anticipated along the foundation bedrock joints, global stability of the footings is not a concern. The proposed footings supported piers may experience sliding caused by lateral loads. Therefore, pier footings should be keyed into bedrock a minimum of 3 inches in accordance with requirements of Section 204.1, 303.4.1.1, and 606.7 of the *2007 ODOT Bridge Design Manual.*

Table 6.1.4 – Estimated Design Parameters for Bridge Pier 3 Footing

Boring No.	Substructure Location	Estimated Top of Bedrock Elev. (feet)	Proposed Bearing Elev. (feet)	Width of Footing (feet)	Factored Bearing Resistance (ksf)
B-006-0-23	Pier 3	564.5±	561.0	10.0	28.8

<u>Ramp Abutment and Retaining Wall Footings</u>: Design information provided by ARC personnel indicate that the maximum compression design loads of the proposed Ramp east/west Abutment footings along a vertical axial direction at the Service and Strength Limits will be 1.82 ksf and 2.65 ksf, respectively. The proposed Ramp Retaining Wall footings along a vertical axial direction at the Service and Strength Limits will be 0.8 ksf and 1.03 ksf, respectively. The physical footing dimensions of the West/East Abutments and Retaining Wall will be 6.5X14.33 feet and 3.0X23.5 feet and the bottom footings will be placed at bearing elevation of 534 feet. The effective footings size of the West/East Abutments and Retaining Wall will be 5.05X14.33 feet and 3.0X23.5 feet based on the external stability calculations. There is no lateral load on the Retaining Wall. It is just to prevent people from accessing under the ramp. Soil and bedrock information obtained from project test boring B-002-0-23 was used to provide foundation recommendations. As outlined in Section 5.1 - "Subsurface Soil Conditions", soils encountered in project test boring B-002-0-23 consisted of predominantly fill soils above the bedrock. These fill soils were encountered to the depth of 28.5 feet below the existing riprap stone. Fill soils encountered consisted of both cohesive and granular foundation soils including brick fragments, cinders, and slag and appeared to be uncontrolled fill. The consistency of the cohesive soils ranged from “medium stiff” to “stiff” and the relative density of the granular soils were “loose”. However, none of the soils within the bearing zone of these footings is “very loose”. The weakest soil in this profile is a “loose” A-4a sandy silt; however, if they consider the overburden correction for the N_{60} value (conversion to $N1_{60}$), then this material ends up classified as “medium dense”, with a friction angle of around 32 degrees. All of the other soils are more capable. While there are cinders and brick identified in Historic Boring B-007-3-64, these all appear to be well-compacted fill materials, with N_{60} blow counts of 17 or above. None of these could be classified as

"uncontrolled" fill. The bearing resistance of the materials encountered is adequate, and there is no reason for an undercut.

Table 6.1.5 – Estimated Design Parameters for Ramp Abutment/Retaining Walls Footing

Boring No.	Substructure Location	Estimated Top of Bedrock Elev. (feet)	Proposed Bearing Elev. (feet)	Effective Footing Width (feet)	Factored Bearing Resistance (ksf)
B-002-0-23	East/West Abutments	495.6±	534.0	4.58	4.31
B-002-0-23	Retaining Walls	495.6±	534.0	3.0	4.50

Bearing capacity analysis was performed by using effective stress parameters to estimate the factored bearing resistance for the footing supported on existing fill soils. The Ramp Footing and overturning check performed by ARC personnel and bearing resistance calculation spreadsheet are included in Appendix B. Table 6.1.5 summarizes the factored bearing resistance for the existing granular fill soils below bearing elevation. For a maximum bearing pressure of 2.65 ksf at the Strength Limit State from Ramp Footing Bearing check by ARC personnel, this gives us a Capacity-Demand Ratio (CDR) = 4.31/2.65 = 1.63 > 1.00, OK. Settlement analyses were performed on the effective abutment footing size 5.05X14.33 feet and the effective retaining wall footing size 3.0X23.5 feet to estimate the immediate and long-term settlements of the proposed Ramp Abutment and Retaining Walls. The foundation soil profiles below proposed Ramp Abutment and Retaining Walls footings were estimated from project test boring B-002-0-23. The soil parameters for granular soils were estimated from our local experience with similar types of soils. The change in the effective overburden pressure in the foundation soils, which will be caused by the weight of the proposed Ramp Abutment and Retaining Walls, was calculated using the 2(V):1(H) method. The design Factored Load bearing pressure at the Service Limit State will be 1.82 ksf and 0.8 ksf on Abutment and Retaining Wall footings, respectively. Most of the soils within the depth limit of the settlement analyses for the wall footings are granular. The settlement on granular soils will occur during construction. The estimated immediate settlements for the Ramp Abutment and Retaining Walls are summarized in Table 6.1.6. The settlement analyses calculation spreadsheets are included in Appendix B. Based on the settlement analyses, the anticipated total settlement on the Ramp Abutment and Retaining Walls footings will be in the order of 0.50 inches and 0.25 inches, respectively. Therefore, it is estimated that the maximum total settlement and differential settlement will not exceed one inch and one-half of an inch, respectively.

Table 6.1.6–Summary of Anticipated Settlement for Ramp Abutment/Retaining Walls Footing

Boring No.	Footing Sizes (feet)	Settlement Type	Estimated Settlement (inches)
B-002-0-23	R. Wall - 3.0X23.5	Consolidation	0.0
		Immediate	0.24
B-002-0-23	AB Wall - 5.05X14.33	Consolidation	0.03
		Immediate	0.52

All footings must be placed 2.0 feet or greater below the final grade to protect against susceptibility to frost heave. Please note that the top elevation of the shale bedrock may vary with location, and slight adjustments of footing depth may be required in the field. The bedrock footing subgrade should be examined by a competent geotechnical engineer to verify that the maximum factored resistance is being complied with. If any soil or severely weathered bedrock is encountered, it should be removed as directed by an on-site geotechnical engineer and replaced with concrete. The excavated Ramp Abutment and Retaining Walls footing subgrade should be examined by competent geotechnical personnel. If any highly compressible fill materials and/or areas of low bearing capacity with excessive moisture (soft pockets) are encountered, they should be removed as directed by geotechnical personnel. In order to minimize the effects of any slight differential movement that may occur due to variations in the character of the supporting soils and any variations in seasonal moisture contents, it is recommended that all footings be suitably reinforced to make them as rigid as possible.

6.2 Site Seismic Properties

Based on the information obtained from the subsurface soil conditions in the vicinity of the test borings B-001 through B-003, the site class “D” can be assumed and in the vicinity of the test borings B-004 and B-005, the site class “C” can be assumed. These seismic site classes were determined in accordance with BDM Section 305.1.5.

6.3 Groundwater Management

Groundwater was encountered in project test borings B-001-0-23, B-002-0-23, and B-003-0-23 and was measured at approximate depths of 22.5 feet, 22.0 feet, and 12.5 feet below the pavement, riprap stone, or concrete surface during drilling operations. If structure foundation excavations extend below the water level encountered in project test boring locations, water infiltration is anticipated in the proposed excavations. Therefore, low to moderate volume pumping or dewatering may be required during excavation of structure foundations. Please note that the groundwater levels may vary due to seasonal fluctuations and groundwater may appear during excavation where it was not previously encountered.

6.4 Earthwork and Construction Monitoring

Selecting the construction method for installing the drilled shafts is the responsibility of the contractor. During installation of drilled shaft holes, water seepage into the holes will occur below the water level encountered in project test borings. Therefore, the using casing method may be required to support the overburden soils. The successful performance of a drilled shaft depends on the construction method used as well as the quality of workmanship during installation. Therefore, qualified geotechnical personnel should be present during construction for inspection in order to assure the quality of the drilled shafts and to verify that the rock conditions are as per the boring logs. Drilled shaft bottoms should be free of all loose material prior to placement of concrete. For detailed drilled shaft construction, refer to Item 524 – "Drilled Shafts" of the ODOT *Construction and Material Specifications* issued in January 2019.

All excavations should comply with all current and applicable local, state, and federal safety codes, regulations and practices, including the Occupational Safety and Health Administration (OSHA). If proposed cut slopes for the structure foundation are to be exposed for an extended period of time, they must be constructed using a two (2) horizontal to one (1) vertical slope for excavation above the water table and a three (3) horizontal to one (1) vertical slope for excavation below the water table or in granular soils. Soil and rock excavations are expected during construction of the project. It is expected that some harder, less weathered bedrock will be present in the drilled shaft and footer excavation. Therefore special drilling equipment should be required.

All fill material must be approved by a qualified geotechnical engineer prior to placement. The fill materials should be placed in lifts of eight (8) inches in thickness (loose measure) and be compacted to an unyielding condition in accordance with ODOT 203.07 "Compaction and Moisture Requirements" specifications. The top 12 inches of the fill in pavement subgrade areas should be placed in lifts of eight (8) inches in thickness (loose measure) and be compacted to an unyielding condition in accordance with ODOT 204.03 "Compaction of the Subgrade" specifications. All in-place density tests should be performed as per Supplement 1015 "Compaction Testing of Unbound Materials" during earthwork construction. All earthwork operations should be conducted in accordance with ODOT *Construction and Material Specifications*, Item 203, issued 2019.

7.0 LIMITATIONS

This report is subject to the following conditions and limitations:

7.1 The subsurface conditions described are based on an examination of the soil and rock samples at the sampling intervals. Varying soil deposits, including fill material, may exist between the sampling intervals and between or beyond the test boring locations. Variation in subsurface conditions from those indicated in this report may become apparent during the earthwork and/or installation of the foundations. Such variations may require changes and/or modifications in our recommendations. Such changes may cause time delays and/or additional costs. Owners must be made aware of these limitations and must incorporate them in the design budget and scheduling of the project.

7.2 The design of the proposed project does not vary from the technical information provided and specified in this report. All changes in the design must be reviewed by our geotechnical engineers. PGI cannot assume any responsibility for interpretations made by others of the subsurface conditions and their behavior based on this report.

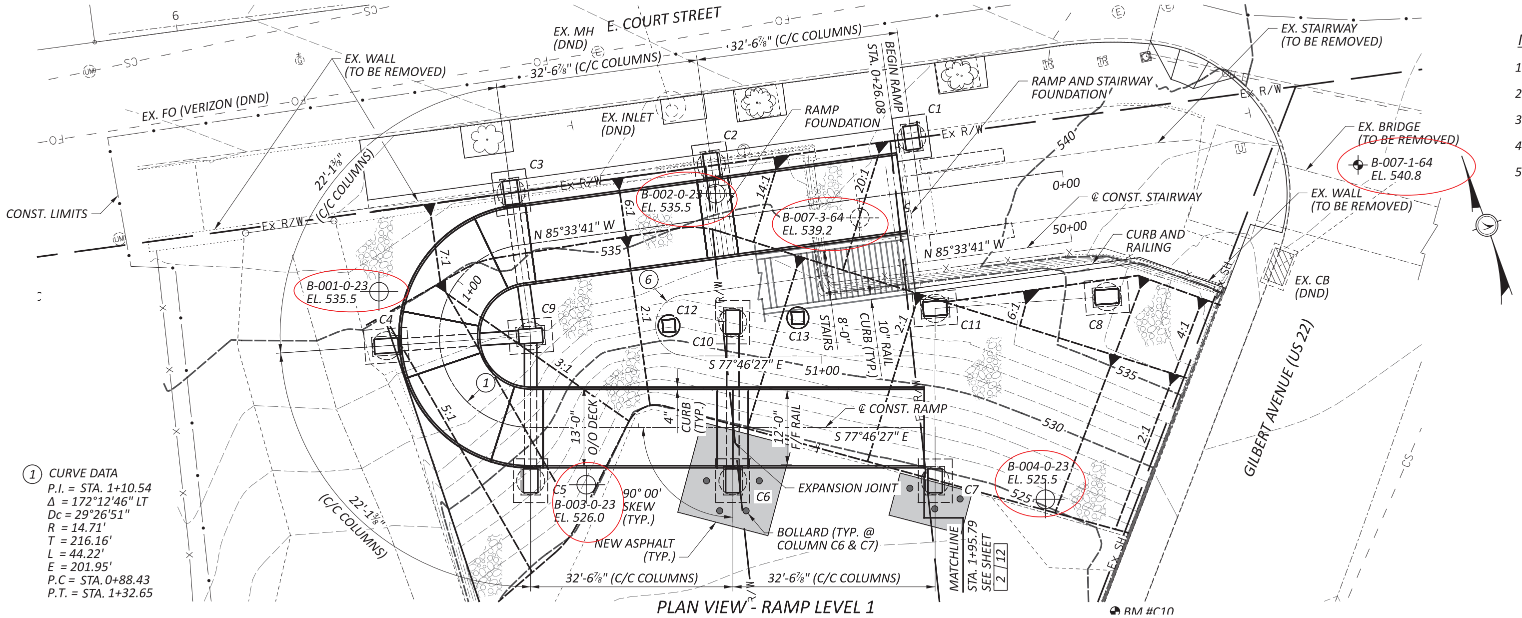
7.3 All earthwork and foundation construction must be performed under the supervision of a Professional Engineer in accordance with ODOT Construction Specifications.

7.4 The subsurface exploration for this project is strictly from a geotechnical standpoint. An environmental site assessment was not included in the scope of these geotechnical services.

7.5 All sheeting, shoring, and bracing of trenches, pits and excavations should be made the responsibility of the contractor and should comply with all current and applicable local, state and federal safety codes, regulations and practices, including the Occupational Safety and Health Administration (OSHA).

APPENDICES

APPENDIX A



① CURVE DATA
P.I. = STA. 1+10.54
Δ = 172°12'46" LT
Dc = 29°26'51"
R = 14.71'
T = 216.16'
L = 44.22'
E = 201.95'
P.C = STA. 0+88.43
P.T. = STA. 1+32.65

PLAN VIEW - RAMP LEVEL 1

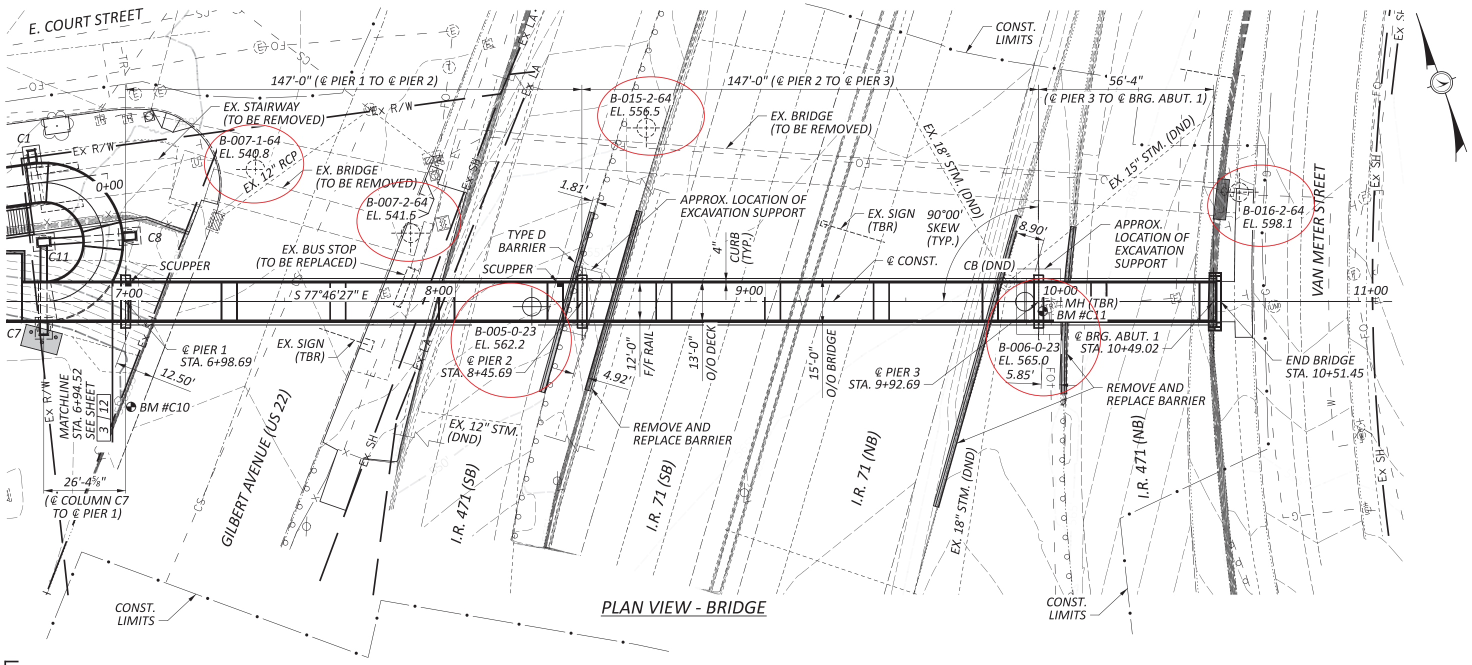
NOTES

1. FOR CENTERLINE CONTROL POINTS, SEE SHEET 4/12 .
2. FOR COLUMN STATION AND OFFSETS, SEE SHEET 11/12 .
3. FOR CURVE DATA ⑥ , SEE SHEET 3/12 .
4. FOR BENCHMARK DATA, SEE SHEET 4/12 .
5. EARTHWORK LIMITS SHOWN ARE APPROXIMATE.

PROJECT AND HISTORIC BORING LOCATION PLAN - PAGE 1

SITE PLAN 1 OF 4
BRIDGE NO. HAM-71-0180
OVER US 22, I.R. 471 & I.R. 71

SFN	3100775
DESIGN AGENCY	ARCADIS 222 SOUTH MAIN STREET SUITE 200 AKRON, OHIO 44308 (330) 434-1995 www.arcadis.com
DESIGNER	RJB
CHECKER	RBB
REVIEWER	CMD 6-15-23
PROJECT ID	102790
SUBSET	1
TOTAL	12
SHEET	P.20
TOTAL	31



PLAN VIEW - BRIDGE

NOTES

1. EARTHWORK LIMITS SHOWN ARE APPROXIMATE.

LEGEND

⊕	PROPOSED SOIL BORING
⊕	HISTORIC SOIL BORING
●	BENCHMARK

(DND) = DO NO DISTURB
(TBR) = TO BE RELOCATED

17.5' REQUIRED MINIMUM VERTICAL CLEARANCE
31.6' ACTUAL MINIMUM VERTICAL CLEARANCE (US 22)
26.8' ACTUAL MINIMUM VERTICAL CLEARANCE (I.R. 471 SB)
17.5' ACTUAL MINIMUM VERTICAL CLEARANCE(I.R. 71 SB)
21.2' ACTUAL MINIMUM VERTICAL CLEARANCE(I.R. 71 NB)
25.6' ACTUAL MINIMUM VERTICAL CLEARANCE(I.R. 471 NB)

PROJECT AND HISTORIC BORING LOCATION PLAN - PAGE 2

SITE PLAN 4 OF 4
BRIDGE NO. HAM-71-0180
OVER US 22, I.R. 471 & I.R. 71

SFN 3100775

DESIGN AGENCY
ARCADIS
222 SOUTH MAIN STREET SUITE 200
AKRON, OHIO 44308
(330) 434-1995
www.arcadis.com

DESIGNER	CHECKER
RJB	RBB

REVIEWER CMD 6-15-23

PROJECT ID 102790

SUBSET	TOTAL
4	12

SHEET	TOTAL
P.23	31

PROJECT: HAM-71-1.80 & HAM-22-10.93
TYPE: BRIDGE REPLACEMENT
PID: 102790 STR ID: HAM-71-1.80
START: 5/12/23 END: 5/12/23

DRILLING FIRM / OPERATOR: TERRACON / K. H.
SAMPLING FIRM / LOGGER: TERRACON / J.H.
DRILLING METHOD: 3.25" HSA
SAMPLING METHOD: SPT / ST / NQ2

DRILL RIG: CME 55/300 ATV/T
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 1/13/23
ENERGY RATIO (%): 90

STATION / OFFSET: 1+05, 11' RT.
ALIGNMENT: RAMP CONST. CETERLINE
ELEVATION: 534.9 (MSL) EOB: 60.0 ft.
COORD: 39.107191, -84.504433

EXPLORATION ID
B-001-0-23

PAGE
1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 534.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
ASPHALT PAVEMENT (12" IN THICKNESS)	533.9																	
MEDIUM DENSE, BLACK, **ASPHALT & STONE FRAGMENTS**, FILL, DAMP	532.9		7 6 4	15	67	SS-1A	-	-	-	-	-	-	-	-	-	3	A-1-a (V)	
MEDIUM DENSE, BLACK, **COARSE AND FINE SAND**, SOME FINES, LITTLE STONE FRAGMENTS, FILL, MOIST	532.4					SS-1B	-	-	-	-	-	-	-	-	-	9	A-3a (V)	
MEDIUM DENSE TO LOOSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE FINES, FILL, DAMP			4 5 5	15	78	SS-2	-	34	25	31	1	9	NP	NP	NP	5	A-1-b (0)	
@6.0'; LOOSE	526.9		6 5 1	9	67	SS-3	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
LOOSE TO VERY LOOSE, BROWN, **GRAVEL AND/OR STONE FRAGMENTS**, "AND" SAND, TRACE FINES, FILL, DAMP			4 4 3	10	22	SS-4	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
@11.0'; VERY LOOSE	521.4		1 1 1	3	33	SS-5	-	52	30	12	3	3	NP	NP	NP	6	A-1-a (0)	
VERY STIFF TO SOFT, BROWN, **SANDY SILT**, LITTLE CLAY, SOME TO NO BRICK, SOME STONE FRAGMENTS, FILL, MOIST TO DAMP			3 3 2	8	61	SS-6	-	-	-	-	-	-	-	-	-	14	A-4a (V)	
@16.0'; SOFT, MOIST			1 1 2	4	56	SS-7	-	-	-	-	-	-	-	-	-	16	A-4a (V)	
@18.5'; STIFF, DAMP			3 4 4	12	44	SS-8	-	-	-	-	-	-	-	-	-	7	A-4a (V)	
@21.0'; MEDIUM STIFF, DAMP	511.9	W	3 3 2	8	11	SS-9	-	-	-	-	-	-	-	-	-	5	A-4a (V)	
LOOSE, BLACK, **NON-PLASTIC SANDY SILT**, LITTLE CINDERS AND COAL/STONE FRAGMENTS, FILL, WET	508.9		2 1 3	6	0	SS-10	-	-	-	-	-	-	-	-	-	37	A-4a (V)	
LOOSE TO DENSE, DARK BROWN, **STONE FRAGMENTS WITH SAND AND SILT**, LITTLE CLAY, FILL, WET			3 3 2	8	89	SS-11	-	36	17	18	14	15	NP	NP	NP	54	A-2-4 (0)	
			3 3 2	8	61	SS-12	-	-	-	-	-	-	-	-	-	38	A-2-4 (V)	
@31.0'; DENSE	502.4		4 16 9	38	100	SS-13	-	-	-	-	-	-	-	-	-	22	A-2-4 (V)	
SOFT TO VERY STIFF, DARK BROWN TO DARK BROWN AND GRAY, **SILTY CLAY**, LITTLE SAND, LITTLE STONE FRAGMENTS, FILL, DAMP TO MOIST			0 0 2	3	100	SS-14	-	-	-	-	-	-	-	-	-	17	A-6b (V)	
@36.0'; VERY STIFF, DARK BROWN AND GRAY, MOIST	496.4		4 5 9	21	89	SS-15	2.50	14	6	8	31	41	37	19	18	25	A-6b (11)	
VERY STIFF, BROWN AND GRAY TO BROWN, **SILTY CLAY**, TRACE SAND, MOIST			0 3 5	12	100	SS-16	2.50	0	1	2	48	49	35	18	17	23	A-6b (11)	

PID: 102790 | STR ID: HAM-71-1.80 | PROJECT: HAM-71-1.80 | STATION / OFFSET: 1+05, 11' RT. | START: 5/12/23 | END: 5/12/23 | PG 2 OF 2 | B-001-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 494.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY STIFF, BROWN AND GRAY TO BROWN, **SILTY CLAY**, TRACE SAND, MOIST *(continued)* @40.0'; BROWN NOTE: SHELBY TUBE WAS PUSHED FROM 40' TO 42'.	491.4	41 42 43			33	ST-17	3.00	-	-	-	-	-	-	-	-	21	A-6b (V)	
VERY STIFF TO HARD, BROWN, **CLAY**, TRACE SAND, TRACE STONE FRAGMENTS, DAMP		44 45	24 13 21	51	56	SS-18	2.00	-	-	-	-	-	-	-	-	18	A-7-6 (V)	
@46.0'; HARD		46 47 48	8 15 18	50	100	SS-19	4.5+	-	-	-	-	-	-	-	-	17	A-7-6 (V)	
@48.5'; HARD	485.4	49 TR	26 48 50/2"	-	14	SS-20A	4.5+	-	-	-	-	-	-	-	-	16	A-7-6 (V)	
SHALE, GRAY, HIGHLY WEATHERED, VERY WEAK TO WEAK.	484.9	50				SS-20B	-	-	-	-	-	-	-	-	-	-	Rock (V)	
INTERBEDDED SHALE (87%) AND LIMESTONE (13%); **SHALE**, GRAY, SEVERELY TO SLIGHTLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY FRACTURED, SLIGHTLY ROUGH TO SLICKENSIDED, OPEN TO TIGHT APERTURE WIDTH;. **LIMESTONE**, GRAY, MODERATELY TO SLIGHTLY WEATHERED, MODERATELY STRONG. NOTE: BEDROCK IS SEVERELY WEATHERED FROM 50' TO 50.8'	483.9	51	0		100	NQ2-1											Rock (V)	
INTERBEDDED SHALE (88%) AND LIMESTONE (12%); **SHALE**, GRAY, HIGHLY TO SLIGHTLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY TO MODERATELY FRACTURED, SLIGHTLY ROUGH TO SLICKENSIDED, TIGHT TO OPEN APERTURE WIDTH;. **LIMESTONE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG. @51.0'- 56.0'; POINT LOAD INDEX STRENGTH = 1539 PSI NOTE: LITTLE IRON STAINING TYPICALLY PRESENT AT LIMESTONE SEAMS WHICH RANGE FROM 1/4" TO 2" IN THICKNESS. @53.3'; COMPRESSIVE STRENGTH OF INTACT ROCK = 356 PSI	478.9	52 53 54 55 56	82		82	NQ2-2											Rock (V)	
INTERBEDDED SHALE (92%) AND LIMESTONE (8%) **SHALE**, GRAY, MODERATELY TO SLIGHTLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY FRACTURED, SLIGHTLY ROUGH, TIGHT APERTURE WIDTH;. **LIMESTONE**, GRAY, MODERATELY TO SLIGHTLY WEATHERED, MODERATELY STRONG. @56.0'- 60.0'; POINT LOAD INDEX STRENGTH = 263 PSI NOTE: LIMESTONE IS GRAY TO WHITE AND GRAY WITH FEW FOSSILIFEROUS LENSES LESS THAN 1/4" IN THICKNESS. @59.0'; COMPRESSIVE STRENGTH OF INTACT ROCK = 872 PSI	474.9	57 58 59 EOB 60	92		92	NQ2-3											Rock (V)	

NOTES: GROUNDWATER WAS ENCOUNTERED AT 22.5' BELOW GROUND SURFACE DURING DRILLING AND NO WATER READING WAS TAKEN UPON COMPLETION BECAUSE WATER WAS USED FOR ROCK CORING OPER

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH SOIL CUTTINGS

COMPANY: PGI DRILLED BY: TERRACON
PROJECT: PEDESTRIAN BRIDGE REPLACEMENT
BRIDGE NO.: HAM-71-1.80
BORING: B-001-0-23 BOX 1/1
DATE of CORING: 5/18/23
RUN-1/NQ2-1: 50.0' - 51.0' REC: 100% RQD: 0
RUN-2/NQ2-2: 51.0' - 56.0' REC: 82% RQD: 72%
RUN-3/NQ2-3: 56.0' - 60.0' REC: 92% RQD: 92%

Pro Geotech, Inc.

Compressive Strength of Rock
ASTM D 7012

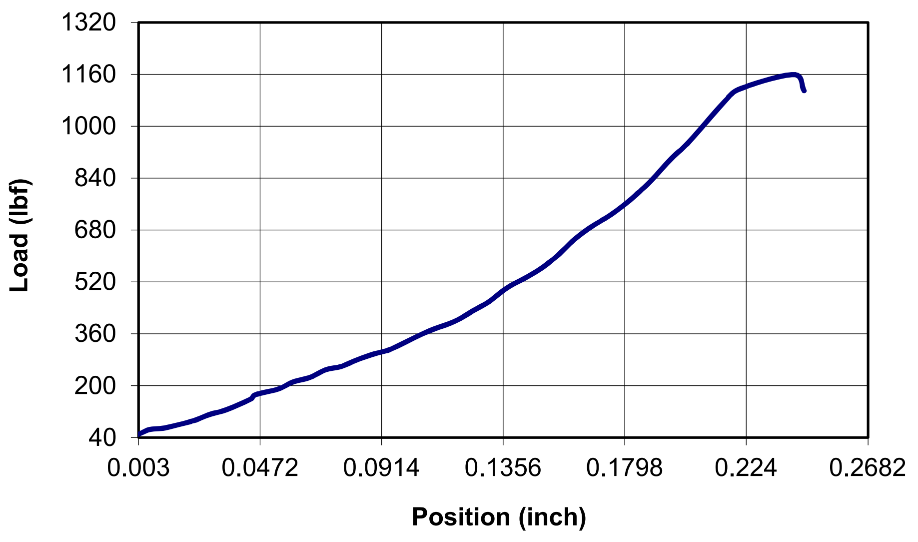
PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	12/19/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-001-0-23	TOP DEPTH (FT)	53.25	BOTTOM DEPTH (FT)	53.59
SAMPLE NUMBER	NQ2-2	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	1+05	OFFSET	11'	OFFSET DIRECTION	RT

FORMATION	SHALE
DESCRIPTION	GRAY, HIGHLY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)
1	4.092	2.036
2	4.120	2.037
3	4.130	2.038
AVERAGE	4.114	2.037

LENGTH/DIAMETER	2.02
CORRECTION FACTOR	1.00
AREA (SQ. INCH)	3.259
MASS (GRAMS)	457.82
UNIT WEIGHT (LBS/FT^3)	130.09

MAXIMUM LOAD (LBS)	1159
COMPRESSIVE STRENGTH (PSI)	356
TIME OF TEST (MINUTES)	12:40
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	NA/DS



BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

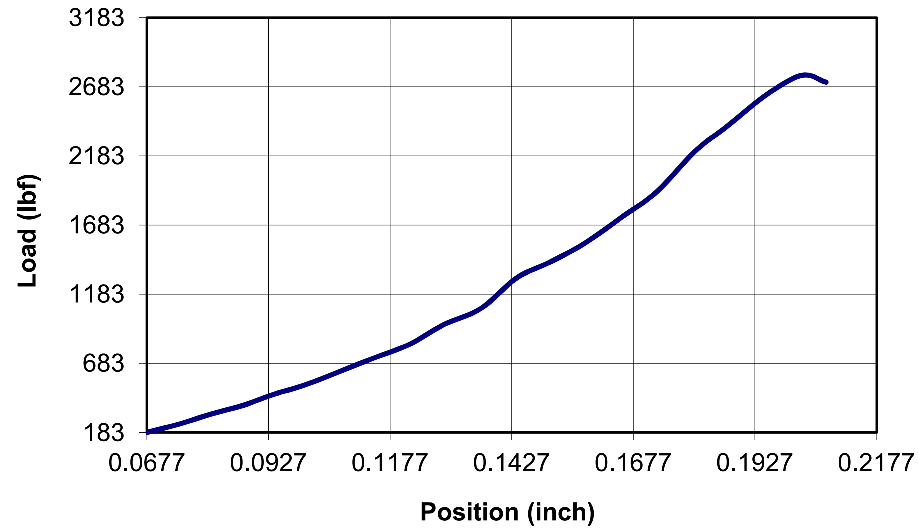
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	7/11/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-001-0-23	TOP DEPTH (FT)	59	BOTTOM DEPTH (FT)	59.35
SAMPLE NUMBER	NQ2-3	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	1+05	OFFSET	11'	OFFSET DIRECTION	RT

FORMATION	SHALE
DESCRIPTION	GRAY, MODERATELY TO SLIGHTLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	4.190	2.011	LENGTH/DIAMETER	2.08
2	4.170	1.990	CORRECTION FACTOR	1.00
3	4.160	2.026	AREA (SQ. INCH)	3.170
			MASS (GRAMS)	503.92
AVERAGE	4.173	2.009	UNIT WEIGHT (LBS/FT^3)	145.11

MAXIMUM LOAD (LBS)	2765
COMPRESSIVE STRENGTH (PSI)	872
TIME OF TEST (MINUTES)	10:00
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	NA/DS



BEFORE TESTING

AFTER FAILURE

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-001-0-23 **Date:** 12/20/2023

Project No.: G23006G **Depth Range:** NQ2-2 - 51.0' - 56.0' **Technician:** NA

Rock Description: SHALE, GRAY, HIGHLY TO SLIGHTLY WEATHERED, VERY WEAK TO WEAK
THINLY LAMINATED, CALCAREOUS, HIGHLY TO MODERARELY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.2	17.0	150	222.75	1087	132.18	0.829	109.60
2	Axial ⊥	49.6	18.0	50	74.25	1136	42.18	0.837	35.32
3	Axial ⊥	49.9	22.0	300	445.50	1397	205.67	0.877	180.44
4	Axial ⊥	50.0	18.0	200	297.00	1146	167.21	0.839	140.30
5	Axial ⊥	50.4	23.0	200	297.00	1475	129.93	0.888	115.38
6	Axial ⊥	47.8	26.0	230	341.55	1582	139.25	0.902	125.64
7	Axial ⊥	49.8	18.0	250	371.25	1141	209.86	0.838	175.92
8	Axial ⊥	49.9	26.0	300	445.50	1652	173.99	0.911	158.50
9	Axial ⊥	49.4	24.0	300	445.50	1509	190.44	0.893	170.00
10	Axial ⊥	49.8	28.0	150	222.75	1775	80.94	0.926	74.95
11	Axial ⊥	50.3	20.0	130	193.05	1281	97.24	0.860	83.65

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	128.21

UCS = $I_{s(50)}$ x 12 (psi)	1539

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-001-0-23 **Date:** 12/21/2023

Project No.: G23006G **Depth Range:** NQ2-3 - 56.0' - 60.0' **Technician:** NA

Rock Description: SHALE, GRAY, MODERATELY TO SLIGHTLY WEATHERED, VERY WEAK TO WEAK THINLY LAMINATED, CALCAREOUS, MODERARELY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.8	25.0	110	163.35	1618	65.15	0.907	59.07
2	Axial ⊥	50.0	38.0	170	252.45	2419	67.33	0.993	66.83
3	Axial ⊥	51.4	24.0	30	44.55	1571	18.30	0.901	16.48
4	Axial ⊥	49.5	18.0	20	29.70	1134	16.89	0.837	14.14
5	Axial ⊥	51.6	17.0	20	29.70	1117	17.16	0.834	14.31
6	Axial ⊥	51.4	28.0	20	29.70	1832	10.46	0.932	9.75
7	Axial ⊥	50.9	19.0	20	29.70	1231	15.56	0.853	13.27
8	Axial ⊥	50.0	17.0	60	89.10	1082	53.11	0.828	44.00
9	Axial ⊥	51.8	25.0	20	29.70	1649	11.62	0.911	10.58
10	Axial ⊥	51.7	19.0	20	29.70	1251	15.32	0.856	13.11
11	Axial ⊥	51.5	21.0	20	29.70	1377	13.92	0.874	12.17

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	21.90
UCS = $I_{s(50)}$ x 12 (psi)	263

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

PROJECT: HAM-71-1.80 & HAM-22-10.93
TYPE: BRIDGE REPLACEMENT
PID: 102790 STR ID: HAM-71-1.80
START: 5/11/23 END: 5/11/23

DRILLING FIRM / OPERATOR: TERRACON / K. H.
SAMPLING FIRM / LOGGER: TERRACON / J.H.
DRILLING METHOD: 3.25" HSA
SAMPLING METHOD: SPT / NQ2

DRILL RIG: CME 55/300 ATV/T
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 1/13/23
ENERGY RATIO (%): 90

STATION / OFFSET: 0+56, 4' RT.
ALIGNMENT: RAMP CONST. CETERLINE
ELEVATION: 535.1 (MSL) EOB: 50.0 ft.
COORD: 39.107205, -84.504235

EXPLORATION ID
B-002-0-23

PAGE
1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 535.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
RIPRAP COVER (5" TO 12" IN DIAMETER WITH GEOTEXTILE BELOW)	534.1	1																
MEDIUM DENSE, BLACK AND GRAY, **CONCRETE AND STONE FRAGMENTS WITH SAND**, LITTLE FINES, FILL, DAMP	532.6	2	3 14 6	30	22	SS-1	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
LOOSE, BROWN, **SANDY SILT**, SOME CLAY, SOME STONE FRAGMENTS, FILL, DAMP	531.1	3				SS-2	-	-	-	-	-	-	-	-	-	5	A-4a (V)	
LOOSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, LITTLE FINES, FILL, DAMP		4 5	5 4 3	10	44	SS-2	-	27	24	32	8	9	NP	NP	NP	5	A-1-b (0)	
	528.1	6	3 2 2	6	28	SS-3A	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
LOOSE, BROWN, **STONE FRAGMENTS WITH SAND AND SILT**, LITTLE CLAY, FILL, DAMP		7				SS-3B	-	-	-	-	-	-	-	-	-	-	A-2-4 (V)	
		8 9 10	3 3 3	9	28	SS-4	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)	
	521.6	11 12 13	3 2 4	9	44	SS-5	-	36	19	15	13	17	21	16	5	10	A-2-4 (0)	
STIFF, BROWN, **SANDY SILT**, SOME CLAY, LITTLE STONE AND BRICK FRAGMENTS, FILL, MOIST		14 15	4 1 4	8	67	SS-6	-	-	-	-	-	-	-	-	-	15	A-4a (V)	
	516.6	16 17 18	1 2 4	9	67	SS-7	1.25	-	-	-	-	-	-	-	-	18	A-4a (V)	
LOOSE TO VERY LOOSE, BLACK, **SANDY SILT**, LITTLE CLAY, LITTLE CINDERS, COAL FRAGS, & GRAVEL, FILL, WET		19 20	1 2 3	8	78	SS-8	-	21	19	23	18	19	NP	NP	NP	30	A-4a (0)	
		21 W 22 23																
@23.5'; VERY LOOSE		24 25	3 2 1	4	67	SS-9	-	-	-	-	-	-	-	-	-	37	A-4a (V)	
	506.6	26 27 28																
MEDIUM STIFF, BROWN AND DARK BROWN, **SILTY CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, MOIST	504.1	29 30	2 1 4	8	78	SS-10	-	6	6	7	41	40	38	22	16	27	A-6b (10)	
HARD, BROWN, **CLAY**, "AND" TO NO LIMESTONE FRAGMENTS, TRACE SAND, DAMP		31 32 33																
		34 35	14 19 27	69	89	SS-11	-	-	-	-	-	-	-	-	-	7	A-7-6 (V)	
		36 37 38																
@38.5'; NO LIMESTONE FRAGMENTS, DAMP	495.6	39	12 46 50/3"	-	100	SS-12A	4.5+	1	1	1	34	63	41	22	19	16	A-7-6 (12)	
SHALE, GRAY, HIGHLY WEATHERED, VERY WEAK TO	495.1	TR				SS-12B	-	-	-	-	-	-	-	-	-	-	Rock (V)	

PID: 102790 | STR ID: HAM-71-1.80 | PROJECT: HAM-71-1.80 | STATION / OFFSET: 0+56, 4' RT. | START: 5/11/23 | END: 5/11/23 | PG 2 OF 2 | B-002-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 495.1	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
WEAK.																		
INTERBEDDED SHALE (96%) AND LIMESTONE (4%); **SHALE**, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY TO MODERATELY FRACTURED, SLIGHTLY ROUGH TO SLICKENSIDED, OPEN TO TIGHT APERTURE WIDTH;. NOTE: 2.5 INCH THICK CLAY/SEVERELY WEATHERED SEAM AT 40 FEET AND 1.5 INCH THICK CLAY SEAM AT 41.9 FEET **LIMESTONE**, GRAY, MODERATELY TO SLIGHTLY WEATHERED, MODERATELY STRONG. NOTE: SHALE IS CALCAREOUS THROUGHOUT RUN & EFFERVESCES FREELY WITH DILUTE HCL. LITTLE IRON STAINING IS PRESENT TYPICALLY AT FEW LIMESTONE SEAMS THAT RANGE IN THICKNESS FROM 1/4" TO 1". NOTE: 0.5 INCH VERTICAL FRACTURE WITH IRON STAINING AT 41.9 FEET NOTE: LIMESTONE IS WHITE & GRAY TO GRAY IN SEAMS RANGINGE IN THICKNESS FROM 0.25 TO 0.5 INCHES; SOME SEAMS FOSSILIFEROUS AND CRYSTALLINE; MODERATELY STRONG TO VERY STRONG; IRON STAINING TYPICALLY PRESENT AT LIMESTONE SEAMS. @40.0'- 45.0'; POINT LOAD INDEX STRENGTH = 260 PSI @42.7'; COMPRESSIVE STRENGTH OF INTACT ROCK = 353 PSI	490.1	41 42 43 44 45	47		95	NQ2-1											Rock (V)	
INTERBEDDED SHALE (83%) AND LIMESTONE (17%); **SHALE**, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY TO MODERATELY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. **LIMESTONE**, GRAY TO GRAY & WHITE, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG. NOTE: SHALE IS CALCAREOUS THROUGHOUT RUN & EFFERVESCES FREELY WITH DILUTE HCL. LITTLE IRON STAINING IS PRESENT TYPICALLY AT FEW LIMESTONE SEAMS THAT RANGE IN THICKNESS FROM 1/4" TO 1". NOTE: 0.75 INCH VERTICAL FRACTURE AT 46.3 FEET. @46.7'; COMPRESSIVE STRENGTH OF INTACT ROCK = 220 PSI NOTE: LIMESTONE IS WHITE & GRAY TO GRAY IN SEAMS RANGINGE IN THICKNESS FROM 0.5 TO 7 INCHES; SOME SEAMS FOSSILIFEROUS AND CRYSTALLINE AND STRONG TO VERY STRONG. @45.0'- 50.0'; POINT LOAD INDEX STRENGTH = 867 PSI	485.1	46 47 48 49 EOB 50	63		98	NQ2-2											Rock (V)	

NOTES: GROUNDWATER WAS ENCOUNTERED AT 22.0' BELOW GROUND SURFACE DURING DRILLING AND NO WATER READING WAS TAKEN UPON COMPLETION BECAUSE WATER WAS USED FOR ROCK CORING OPER

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH SOIL CUTTINGS MIXED WITH BENTONITE

COMPANY: PGI DRILLED BY: TERRACON
PROJECT: PEDESTRIAN BRIDGE REPLACEMENT
BRIDGE NO.: HAM-71-1.80
BORING: B-002-0-23 BOX 1/1
DATE of CORING: 5/18/23
RUN-1/NQ2-1: 40.0' - 45.0' REC: 95% RQD: 47%
RUN-2/NQ2-2: 45.0' - 50.0' REC: 98% RQD: 63%

Pro Geotech, Inc.

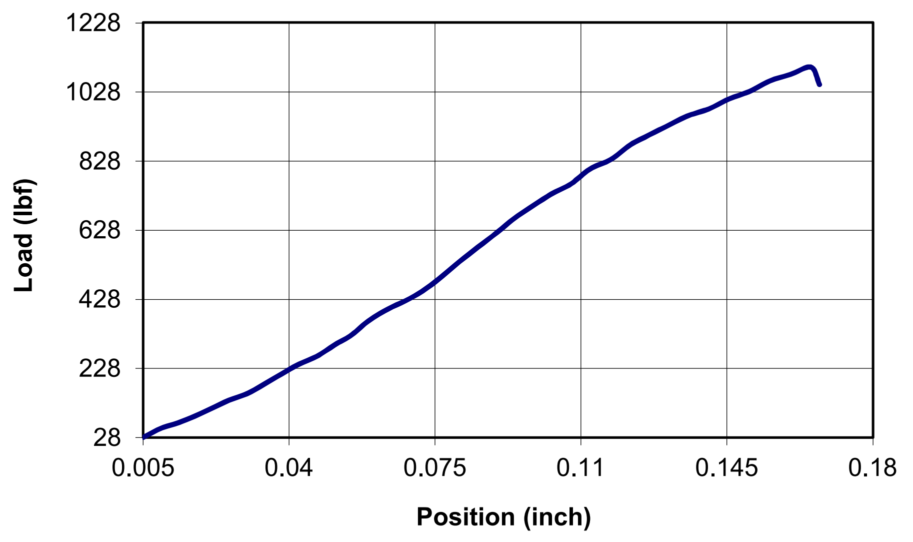
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	12/27/2023
STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71				
BORING NUMBER	B-002-0-23	TOP DEPTH (FT)	42.67	BOTTOM DEPTH (FT)	42.98
SAMPLE NUMBER	NQ2-1	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	0+56	OFFSET	4'	OFFSET DIRECTION	RT

FORMATION	SHALE
DESCRIPTION	GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY TO MODERATELY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	3.790	1.990	LENGTH/DIAMETER	1.90
2	3.760	1.986	CORRECTION FACTOR	1.01
3	3.770	1.980	AREA (SQ. INCH)	3.096
			MASS (GRAMS)	399.88
AVERAGE	3.773	1.985	UNIT WEIGHT (LBS/FT^3)	130.41

MAXIMUM LOAD (LBS)	1100
COMPRESSIVE STRENGTH (PSI)	353
TIME OF TEST (MINUTES)	12:20
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	NA/DS



BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

Compressive Strength of Rock
ASTM D 7012

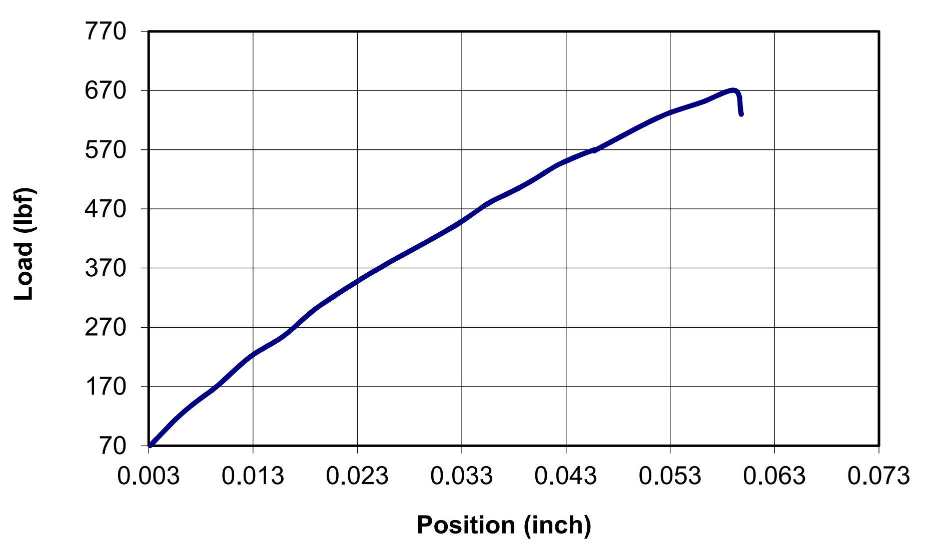
PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	12/19/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-002-0-23	TOP DEPTH (FT)	46.67	BOTTOM DEPTH (FT)	47
SAMPLE NUMBER	NQ2-2	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	0+56	OFFSET	4'	OFFSET DIRECTION	RT

FORMATION	SHALE
DESCRIPTION	GRAY, MODERATELY TO HIGHLYWEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY TO MODERATELY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)
1	3.920	1.970
2	3.880	1.950
3	3.900	1.980
AVERAGE	3.900	1.967

LENGTH/DIAMETER	1.98
CORRECTION FACTOR	1.00
AREA (SQ. INCH)	3.038
MASS (GRAMS)	419.20
UNIT WEIGHT (LBS/FT^3)	134.80

MAXIMUM LOAD (LBS)	670
COMPRESSIVE STRENGTH (PSI)	220
TIME OF TEST (MINUTES)	6:20
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	NA/DS



BEFORE TESTING

AFTER FAILURE

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-002-0-23 **Date:** 7/11/2023

Project No.: G23006G **Depth Range:** NQ2-1 - 40.0' - 45.0' **Technician:** NA

Rock Description: SHALE, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY TO MODERATELY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	49.7	24.0	0	0.00	1519	0.00	0.894	0.00
2	Axial ⊥	50.2	36.0	100	148.50	2301	41.64	0.982	40.87
3	Axial ⊥	49.8	24.0	35	51.98	1522	22.03	0.894	19.71
4	Axial ⊥	49.6	24.0	300	445.50	1516	189.63	0.894	169.44
5	Axial ⊥	49.8	28.0	110	163.35	1775	59.36	0.926	54.96
6	Axial ⊥	49.7	32.0	5	7.43	2025	2.37	0.954	2.26
7	Axial ⊥	49.9	33.0	55	81.68	2097	25.13	0.961	24.16
8	Axial ⊥	50.1	27.0	40	59.40	1722	22.25	0.920	20.46
9	Axial ⊥	50.3	31.0	55	81.68	1985	26.54	0.949	25.20
10	Axial ⊥	50.1	29.5	10	14.85	1882	5.09	0.938	4.78
11	Axial ⊥	49.9	32.0	5	7.43	2033	2.36	0.955	2.25

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	21.63
UCS = $I_{s(50)}$ x 12 (psi)	260

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-002-0-23 **Date:** 12/26/2023

Project No.: G23006G **Depth Range:** NQ2-2 - 45.0' - 50.0' **Technician:** NA

Rock Description: SHALE, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK THINLY LAMINATED, CALCAREOUS, HIGHLY TO MODERARELY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	51.6	18.0	5	7.43	1183	4.05	0.845	3.42
2	Axial ⊥	50.0	20.0	190	282.15	1273	142.97	0.859	122.83
3	Axial ⊥	49.8	21.0	223	331.16	1332	160.45	0.868	139.25
4	Axial ⊥	48.9	19.0	90	133.65	1183	72.89	0.845	61.60
5	Axial ⊥	46.7	23.0	40	59.40	1368	28.02	0.873	24.47
6	Axial ⊥	50.6	17.0	100	148.50	1095	87.48	0.831	72.65
7	Axial ⊥	50.0	17.0	60	89.10	1082	53.11	0.828	44.00
8	Axial ⊥	50.0	20.0	120	178.20	1273	90.30	0.859	77.58
9	Axial ⊥	49.1	20.0	160	237.60	1250	122.60	0.856	104.90
10	Axial ⊥	50.7	17.0	70	103.95	1097	61.11	0.831	50.78
11	Axial ⊥	50.3	29.0	190	282.15	1857	98.01	0.935	91.67

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	72.27

UCS = $I_{s(50)}$ x 12 (psi)	867

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

PROJECT: HAM-71-1.80 & HAM-22-10.93	DRILLING FIRM / OPERATOR: TERRACON / K. H.	DRILL RIG: CME 55/300 ATV/T	STATION / OFFSET: 1+42, 9' RT.	EXPLORATION ID B-003-0-23
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: TERRACON / J.H.	HAMMER: CME AUTOMATIC	ALIGNMENT: RAMP CONST. CETERLINE	
PID: 102790 STR ID: HAM-71-1.80	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 1/13/23	ELEVATION: 524.1 (MSL) EOB: 37.5 ft.	PAGE 1 OF 1
START: 5/11/23 END: 5/11/23	SAMPLING METHOD: SPT / ST / NQ2	ENERGY RATIO (%): 90	COORD: 39.107090, -84.504338	

MATERIAL DESCRIPTION AND NOTES	ELEV. 524.1
CONCRETE SLAB (8" IN THICKNESS)	523.5
DENSE, BLACK, **COARSE AND FINE SAND**, LITTLE FINES, LITTLE BRICK AND STONE FRAGMENTS, FILL, DAMP	521.1
MEDIUM DENSE, BLACK, **CINDERS**, SOME SAND, LITTLE FINES, FILL, MOIST	518.1
VERY STIFF, BROWN AND DARK BROWN TO DARK BROWN, **SANDY SILT**, SOME CLAY, LITTLE STONE FRAGMENTS, FILL, WET @8.5'; SOFT, DARK BROWN NOTE: SHELBY TUBE WAS PUSHED FROM 10' TO 12'.	512.1
SOFT TO VERY STIFF, DARK BROWN TO BLACK, **SILT AND CLAY**, LITTLE SAND, TRACE TO LITTLE STONE FRAGMENTS, FILL, MOIST TO WET @13.5'; WET @16.0'; WET @17.0'; TRACE STONE FRAGMENTS, WET @18.5'; VERY STIFF, LITTLE STONE FRAGMENTS, MOIST	503.1
HARD, BROWN, **CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, DAMP	497.3
SHALE, GRAY, HIGHLY WEATHERED, VERY WEAK TO WEAK.	496.6
INTERBEDDED SHALE (90%) AND LIMESTONE (10%); **SHALE**, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY FRACTURED TO FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. NOTE: FROM 27.5' TO 28.3', SHALE IS SEVERELY WEATHERED @28.5'; COMPRESSIVE STRENGTH OF INTACT ROCK = 182 PSI **LIMESTONE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG. NOTE: SHALE IS CALCAREOUS THROUGHOUT RUN & EFFERVESCES FREELY WITH DILUTE HCL. LITTLE IRON STAINING IS PRESENT TYPICALLY AT FEW LIMESTONE SEAMS THAT RANGE IN THICKNESS FROM 1/4" TO 1". @27.5'- 31.0'; POINT LOAD INDEX STRENGTH = 635 PSI NOTE: FEW THIN LIMESTONE SEAMS ARE GRAY TO WHITE AND GRAY AND FOSSILIFEROUS AND CRYSTALLINE AND ARE MODERATELY STRONG TO VERY STRONG NOTE: 1/2" VERTICAL FRACTURE WITH IRON STAINING	493.1
INTERBEDDED SHALE (93%) AND LIMESTONE (7%); **SHALE**, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. NOTE: SHALE IS CALCAREOUS THROUGHOUT RUN & EFFERVESCES FREELY WITH DILUTE HCL. NO IRON STAINING IS PRESENT. **LIMESTONE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG. @33.0'; COMPRESSIVE STRENGTH OF INTACT ROCK = 958 PSI @31.0'- 36.0'; POINT LOAD INDEX STRENGTH = 1148 PSI NOTE: FEW THIN LIMESTONE SEAMS ARE GRAY TO WHITE AND GRAY AND FOSSILIFEROUS AND CRYSTALLINE AND MODERATELY STRONG TO VERY STRONG FROM 31.3' TO 32.1' AND RANGE IN THICKNESS FROM 1/4" TO 3/4"	488.1
INTERBEDDED SHALE (95%) AND LIMESTONE (5%); **SHALE**, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED, SLIGHTLY ROUGH, TIGHT APERTURE WIDTH;. **LIMESTONE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG. NOTE: SHALE IS CALCAREOUS THROUGHOUT RUN & EFFERVESCES FREELY WITH DILUTE HCL. NO IRON STAINING IS PRESENT. NOTE: WHITE AND GRAY FOSSILIFEROUS AND CRYSTALLINE LIMESTONE LENSES PRESENT FROM 36.6' TO 36.8'	486.6

DEPTHS: W (12.5); TR (27); EOB (37.5)

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
9 12 10	33	78	SS-1	-	-	-	-	-	-	-	-	-	8	A-3a (V)
3 5 9	21	11	SS-2	-	-	-	-	-	-	-	-	-	11	A-1-a (V)
6 6 2	12	78	SS-3	2.50	-	-	-	-	-	-	-	-	24	A-4a (V)
1 1 1	3	78	SS-4	0.25	18	15	18	22	27	30	21	9	28	A-4a (3)
		54	ST-5	-	-	-	-	-	-	-	-	-	22	A-4a (V)
2 1 1	3	56	SS-6	0.25	-	-	-	-	-	-	-	-	42	A-6a (V)
1 1 2	4	56	SS-7	0.25	7	8	9	45	31	38	26	12	39	A-6a (9)
10 7 7	21	67	SS-8	-	-	-	-	-	-	-	-	-	34	A-6a (V)
5 10 14	36	100	SS-9	4.5+	-	-	-	-	-	-	-	-	18	A-7-6 (V)
10 23 24	70	100	SS-10	4.5+	1	8	4	29	58	44	24	20	16	A-7-6 (13)
27 40 50/4"	-	100	SS-11A	-	-	-	-	-	-	-	-	-	13	A-7-6 (V)
			SS-11B	-	-	-	-	-	-	-	-	-	-	Rock (V)
79		100	NQ2-1											Rock (V)
40		85	NQ2-2											Rock (V)
78		100	NQ2-3											Rock (V)

NOTES: GROUNDWATER WAS ENCOUNTERED AT 12.5' BELOW GROUND SURFACE DURING DRILLING AND NO WATER READING WAS TAKEN UPON COMPLETION BECAUSE WATER WAS USED FOR ROCK CORING OPER

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH SOIL CUTTINGS

COMPANY: PGI DRILLED BY: TERRACON
PROJECT: PEDESTRIAN BRIDGE REPLACEMENT
BRIDGE NO.: HAM-71-1.80
BORING: B-003-0-23 BOX 1/1
DATE of CORING: 5/11/23

RUN-1/NQ2-1:	27.5' - 31.0'	REC: 100%	RQD: 79%
RUN-2/NQ2-2:	31.0' - 36.0'	REC: 85%	RQD: 40%
RUN-3/NQ2-3:	36.0' - 37.5'	REC: 100%	RQD: 78%

Pro Geotech, Inc.

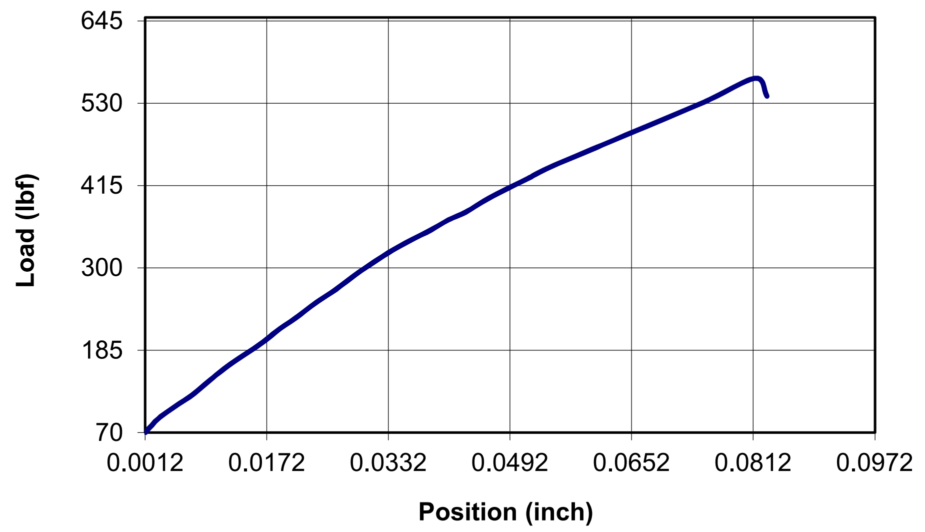
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	12/26/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-003-0-23	TOP DEPTH (FT)	28.5	BOTTOM DEPTH (FT)	28.84
SAMPLE NUMBER	NQ2-1	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	1+42	OFFSET	9'	OFFSET DIRECTION	RT

FORMATION	SHALE
DESCRIPTION	GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY FRACTURED TO FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	4.110	1.978	LENGTH/DIAMETER	2.07
2	4.101	1.988	CORRECTION FACTOR	1.00
3	4.100	1.992	AREA (SQ. INCH)	3.098
			MASS (GRAMS)	434.51
AVERAGE	4.104	1.986	UNIT WEIGHT (LBS/FT^3)	130.21

MAXIMUM LOAD (LBS)	565
COMPRESSIVE STRENGTH (PSI)	182
TIME OF TEST (MINUTES)	8:20
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	NA/DS



BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

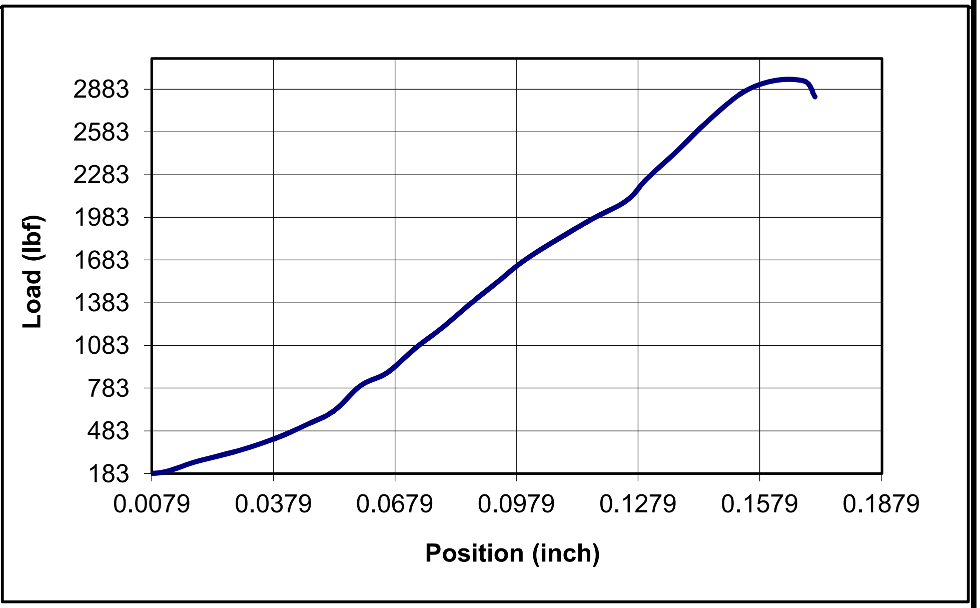
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	7/11/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-003-0-23	TOP DEPTH (FT)	33	BOTTOM DEPTH (FT)	33.34
SAMPLE NUMBER	NQ2-2	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	1+42	OFFSET	9'	OFFSET DIRECTION	RT

FORMATION	SHALE
DESCRIPTION	GRAY, MODERATELY TO HIGHLYWEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	4.100	1.990	LENGTH/DIAMETER	2.08
2	4.120	1.980	CORRECTION FACTOR	1.00
3	4.130	1.960	AREA (SQ. INCH)	3.069
AVERAGE	4.117	1.977	MASS (GRAMS)	486.20
			UNIT WEIGHT (LBS/FT^3)	146.62

MAXIMUM LOAD (LBS)	2940
COMPRESSIVE STRENGTH (PSI)	958
TIME OF TEST (MINUTES)	8:00
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	NA/DS



BEFORE TESTING

AFTER FAILURE

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-003-0-23 **Date:** 12/19/2023

Project No.: G23006G **Depth Range:** NQ2-1 - 27.5' - 31.0' **Technician:** NA

Rock Description: SHALE, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK THINLY LAMINATED, CALCAREOUS, HIGHLY FRACTURED TO FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.2	26.0	50	74.25	1662	28.83	0.912	26.29
2	Axial ⊥	49.5	32.0	5	7.43	2017	2.38	0.953	2.26
3	Axial ⊥	50.0	17.0	90	133.65	1082	79.67	0.828	65.99
4	Axial ⊥	49.7	31.0	5	7.43	1962	2.44	0.947	2.31
5	Axial ⊥	49.5	18.0	140	207.90	1134	118.23	0.837	98.97
6	Axial ⊥	50.0	24.0	5	7.43	1528	3.14	0.895	2.81
7	Axial ⊥	49.6	17.0	200	297.00	1074	178.48	0.827	147.57
8	Axial ⊥	50.0	27.0	120	178.20	1719	66.89	0.919	61.48
9	Axial ⊥	51.5	21.0	5	7.43	1377	3.48	0.874	3.04
10	Axial ⊥	49.9	20.0	190	282.15	1271	143.25	0.859	123.02
11	Axial ⊥	50.1	27.0	180	267.30	1722	100.13	0.920	92.08

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	52.89
UCS = $I_{s(50)}$ x 12 (psi)	635

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-003-0-23 **Date:** 12/25/2023

Project No.: G23006G **Depth Range:** NQ2-2 - 31.0' - 36.0' **Technician:** NA

Rock Description: SHALE, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK THINLY LAMINATED, CALCAREOUS, MODERARELY TO HIGHLY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.0	30.0	360	534.60	1910	180.59	0.941	169.97
2	Axial ⊥	49.8	20.0	190	282.15	1269	143.46	0.858	123.15
3	Axial ⊥	50.2	24.0	90	133.65	1534	56.21	0.896	50.36
4	Axial ⊥	49.2	29.0	150	222.75	1817	79.11	0.931	73.62
5	Axial ⊥	50.1	27.0	100	148.50	1722	55.63	0.920	51.15
6	Axial ⊥	50.2	19.0	60	89.10	1214	47.35	0.850	40.25
7	Axial ⊥	50.2	21.0	140	207.90	1342	99.97	0.869	86.91
8	Axial ⊥	50.6	26.0	200	297.00	1675	114.39	0.914	104.53
9	Axial ⊥	50.5	32.0	180	267.30	2058	83.81	0.957	80.22
10	Axial ⊥	49.9	25.0	380	564.30	1589	229.16	0.903	206.94
11	Axial ⊥	50.0	29.0	250	371.25	1846	129.73	0.934	121.18

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	95.68
UCS = $I_{s(50)}$ x 12 (psi)	1148

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

PROJECT: HAM-71-1.80 & HAM-22-10.93
TYPE: BRIDGE REPLACEMENT
PID: 102790 STR ID: HAM-71-1.80
START: 5/11/23 END: 5/11/23

DRILLING FIRM / OPERATOR: TERRACON / K. H.
SAMPLING FIRM / LOGGER: TERRACON / J.H.
DRILLING METHOD: 3.25" HSA
SAMPLING METHOD: SPT / NQ2

DRILL RIG: CME 55/300 ATV/T
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 1/13/23
ENERGY RATIO (%): 90

STATION / OFFSET: 6+90, 12' RT.
ALIGNMENT: RAMP CONST. CETERLINE
ELEVATION: 524.4 (MSL) EOB: 25.5 ft.
COORD: 39.107045, -84.504084

EXPLORATION ID
B-004-0-23

PAGE
1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 524.4	DEPTHS
ASPHALT PAVEMENT (1" IN THICKNESS)	524.3	
AGGREGATE BASE (10" IN THICKNESS)	523.5	
DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE FINES, FILL, DAMP	518.9	
MEDIUM DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE FINES, LITTLE STONE FRAGMENTS, FILL, MOIST	515.9	
HARD, BROWN, **SILTY CLAY**, TRACE TO "AND" SHALE FRAGMENTS, TRACE SAND, DAMP @13.5'; "AND" SHALE FRAGMENTS	509.9	TR
SHALE, GRAY, HIGHLY WEATHERED, VERY WEAK TO WEAK.	509.7	
INTERBEDDED SHALE (58%) AND LIMESTONE (42%); **SHALE**, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. **LIMESTONE**, GRAY TO GRAY & WHITE, VERY WEAK. NOTE: SHALE IS CALCAREOUS AND EFFERVESCES FREELY WITH DILUTE HCL. GRAY AND WHITE LIMESTONE FROM 15' TO 15.2'.	508.9	
INTERBEDDED SHALE (90%) AND LIMESTONE (10%); **SHALE**, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. NOTE: 1.5" VERTICAL FRACTURE WITH IRON STAINING AT 17.2'. LITTLE IRON STAINING TYPICALLY AT LIMESTONE SEAMS. **LIMESTONE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG. @15.5'- 20.5'; POINT LOAD INDEX STRENGTH = 704 PSI NOTE: SHALE IS CALCAREOUS AND EFFERVESCES FREELY WITH DILUTE HCL. LIMESTONE SEAMS ARE GRAY TO GRAY AND WHITE AND RANGE IN THICKNESS FROM 1/4" TO 2". SOME OF LIMESTONE SEAMS ARE FOSSILIFEROUS AND CRYSTALLINE AND STRONG TO VERY STRONG. @19.5'; COMPRESSIVE STRENGTH OF INTACT ROCK = 1030 PSI	503.9	
INTERBEDDED SHALE (90%) AND LIMESTONE (10%); **SHALE**, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, FRACTURED TO HIGHLY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. **LIMESTONE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG. NOTE: SHALE IS CALCAREOUS AND EFFERVESCES FREELY WITH DILUTE HCL. LIMESTONE IS GRAY AND WHITE WITH SOME SEAMS FOSSILIFEROUS AND CRYSTALLINE AND ARE STRONG TO VERY STRONG. SEAMS RANGE IN THICKNESS FROM 0.5" TO 1.5" WITH OCCASIONAL IRON STAINING. @22.8'; COMPRESSIVE STRENGTH OF INTACT ROCK = 374 PSI @20.5'- 25.5'; POINT LOAD INDEX STRENGTH = 185 PSI	498.9	EOB

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)
					GR	CS	FS	SI	CL	LL	PL	PI		
3 10 14	36	56	SS-1	-	-	-	-	-	-	-	-	-	5	A-1-b (V)
8 15 15	45	67	SS-2	-	21	44	25	3	7	NP	NP	NP	7	A-1-b (0)
8 6 6	18	56	SS-3	-	-	-	-	-	-	-	-	-	13	A-3a (V)
7 20 25	68	89	SS-4	-	-	-	-	-	-	-	-	-	13	A-6b (V)
14 20 24	66	100	SS-5	-	8	5	2	32	53	39	22	17	17	A-6b (11)
19 37 50/2"	-	100	SS-6A	-	-	-	-	-	-	-	-	-	12	A-6b (V)
			SS-6B	-	-	-	-	-	-	-	-	-	-	Rock (V)
0		60	NQ2-1											Rock (V)
62		100	NQ2-2											Rock (V)
50		100	NQ2-3											Rock (V)

NOTES: GROUNDWATER WAS NOT ENCOUNTERED DURING DRILLING AND NO WATER READING WAS TAKEN UPON COMPLETION BECAUSE WATER WAS USED FOR ROCK CORING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH SAND

COMPANY: PGI DRILLED BY: TERRACON
PROJECT: PEDESTRIAN BRIDGE REPLACEMENT
BRIDGE NO.: HAM-71-1.80
BORING: B-004-0-23 BOX 1/1
DATE of CORING: 5/11/23
RUN-1/NQ2-1: 14.7' - 15.5' REC: 60% RQD: 0
RUN-2/NQ2-2: 15.5' - 20.5' REC: 100% RQD: 62%
RUN-3/NQ2-2: 20.5' - 25.5' REC: 100% RQD: 60%

Pro Geotech, Inc.

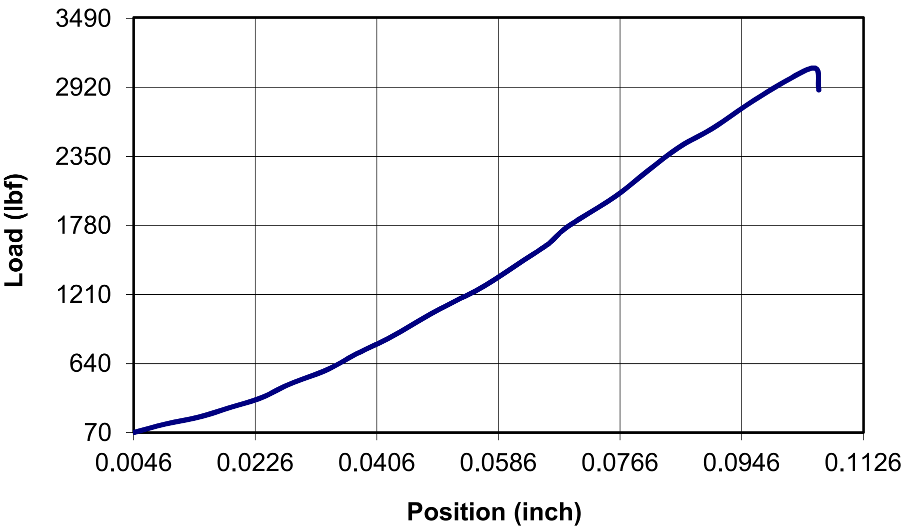
Compressive Strength of Rock
ASTM D 7012

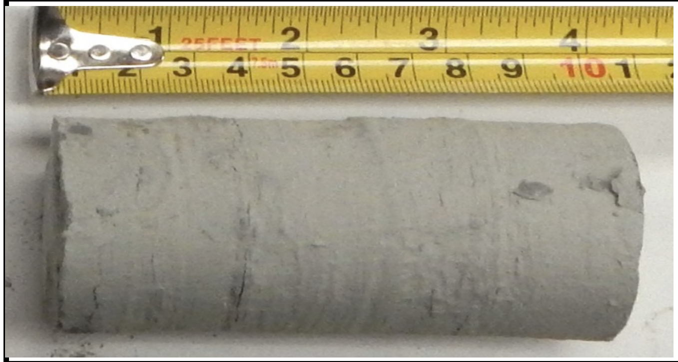
PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	12/27/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-004-0-23	TOP DEPTH (FT)	19.5	BOTTOM DEPTH (FT)	19.82
SAMPLE NUMBER	NQ2-1	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	6+90	OFFSET	12'	OFFSET DIRECTION	RT

FORMATION	SHALE
DESCRIPTION	GRAY, MODERATELY TO HIGHLYWEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		LENGTH/DIAMETER	1.97
1	3.850	1.960		CORRECTION FACTOR	1.00
2	3.840	1.940		AREA (SQ. INCH)	2.986
3	3.840	1.950		MASS (GRAMS)	442.13
AVERAGE	3.843	1.950		UNIT WEIGHT (LBS/FT^3)	146.74

MAXIMUM LOAD (LBS)	3081
COMPRESSIVE STRENGTH (PSI)	1030
TIME OF TEST (MINUTES)	8:00
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	NA/DS




BEFORE TESTING


AFTER FAILURE

Pro Geotech, Inc.

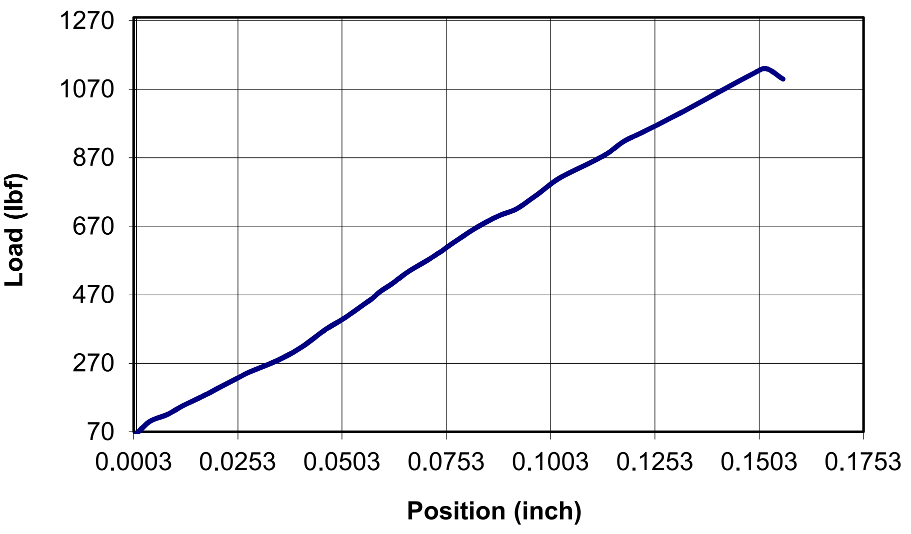
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	12/19/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-004-0-23	TOP DEPTH (FT)	22.83	BOTTOM DEPTH (FT)	23.17
SAMPLE NUMBER	NQ2-2	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	6+90	OFFSET	12'	OFFSET DIRECTION	RT

FORMATION	SHALE
DESCRIPTION	GRAY, MODERATELY TO HIGHLYWEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, FRACTURED TO HIGHLY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		LENGTH/DIAMETER	2.05
1	4.020	1.952		CORRECTION FACTOR	1.00
2	4.026	1.962		AREA (SQ. INCH)	3.025
3	4.021	1.974		MASS (GRAMS)	448.71
AVERAGE	4.022	1.963		UNIT WEIGHT (LBS/FT^3)	140.47

MAXIMUM LOAD (LBS)	1130
COMPRESSIVE STRENGTH (PSI)	374
TIME OF TEST (MINUTES)	10:40
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	NA/DS




BEFORE TESTING


AFTER FAILURE

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-004-0-23 **Date:** 7/11/2023

Project No.: G23006G **Depth Range:** NQ2-2 - 15.5' - 20.5' **Technician:** NA

Rock Description: SHALE, GRAY, MODERATELY HIGHLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.1	18.0	145	215.33	1148	120.99	0.839	101.56
2	Axial ⊥	50.2	21.5	60	89.10	1374	41.83	0.874	36.56
3	Axial ⊥	49.9	30.0	0	0.00	1906	0.00	0.941	0.00
4	Axial ⊥	49.7	36.0	210	311.85	2278	88.32	0.979	86.49
5	Axial ⊥	50.2	32.0	205	304.43	2045	96.02	0.956	91.78
6	Axial ⊥	49.8	23.0	210	311.85	1458	137.96	0.886	122.20
7	Axial ⊥	50.2	25.0	880	1306.80	1598	527.62	0.904	477.07
8	Axial ⊥	50.0	22.0	65	96.53	1401	44.46	0.878	39.03
9	Axial ⊥	49.8	28.0	75	111.38	1775	40.47	0.926	37.47
10	Axial ⊥	49.7	35.0	20	29.70	2215	8.65	0.973	8.42
11	Axial ⊥	50.1	30.0	10	14.85	1914	5.01	0.942	4.71

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	58.69
UCS = $I_{s(50)}$ x 12 (psi)	704

Piston Area = 1.485 sq. Inches

L = Applied Pressure
P = Failure Load
⊥ = Load Applied Perpendicular to Bedding
W = Core Sample Diameter
D = Height of Sample
UCS = Unconfined Compressive Strength

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-004-0-23 **Date:** 12/26/2023

Project No.: G23006G **Depth Range:** NQ2-3 - 20.5' - 25.50' **Technician:** NA

Rock Description: SHALE, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK THINLY LAMINATED, CALCAREOUS, FRACTURED TO HIGHLY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.0	28.0	150	222.75	1783	80.62	0.927	74.71
2	Axial ⊥	50.0	22.0	150	222.75	1401	102.61	0.878	90.07
3	Axial ⊥	48.3	19.0	20	29.70	1169	16.39	0.843	13.81
4	Axial ⊥	50.5	20.0	20	29.70	1286	14.90	0.861	12.83
5	Axial ⊥	50.5	19.0	5	7.43	1222	3.92	0.851	3.34
6	Axial ⊥	52.2	26.0	5	7.43	1728	2.77	0.920	2.55
7	Axial ⊥	50.9	20.0	5	7.43	1296	3.70	0.863	3.19
8	Axial ⊥	50.5	25.0	20	29.70	1607	11.92	0.905	10.79
9	Axial ⊥	50.3	27.0	5	7.43	1729	2.77	0.920	2.55
10	Axial ⊥	50.2	20.0	5	7.43	1278	3.75	0.860	3.22
11	Axial ⊥	50.0	18.0	20	29.70	1146	16.72	0.839	14.03

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	15.39
UCS = $I_{s(50)}$ x 12 (psi)	185

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

PROJECT: HAM-71-1.80 & HAM-22-10.93
TYPE: BRIDGE REPLACEMENT
PID: 102790 STR ID: HAM-71-1.80
START: 6/8/23 END: 6/8/23

DRILLING FIRM / OPERATOR: TERRACON / K. H.
SAMPLING FIRM / LOGGER: TERRACON / J.H.
DRILLING METHOD: 3.25" HSA
SAMPLING METHOD: SPT / NQ2

DRILL RIG: CME 55/300 ATV/T
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 1/13/23
ENERGY RATIO (%): 90

STATION / OFFSET: 8+30, 2' RT.
ALIGNMENT: RAMP CONST. CETERLINE
ELEVATION: 552.0 (MSL) EOB: 27.7 ft.
COORD: 39.106999, -84.503595

EXPLORATION ID
B-005-0-23

PAGE
1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 552.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
CONCRETE SLAB (15" IN THICKNESS)	550.7																	
AGGREGATE BASE (3" IN THICKNESS)	550.5																	
VERY STIFF, BROWN AND GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, MOIST	549.0		2 3 4	10	89	SS-1	4.00	-	-	-	-	-	-	-	-	18	A-6a (V)	
VERY STIFF TO HARD, BROWN, **CLAY**, TRACE STONE FRAGMENTS, TRACE SAND, DAMP			3 5 8	20	100	SS-2	4.00	-	-	-	-	-	-	-	-	19	A-7-6 (V)	
@6.0'; HARD			5 7 11	27	100	SS-3	4.5+	7	2	3	31	57	43	23	20	19	A-7-6 (13)	
@8.5'; HARD			5 11 14	38	56	SS-4	4.5+	-	-	-	-	-	-	-	-	16	A-7-6 (V)	
@11.0'; HARD	539.0		9 13 14	40	100	SS-5	4.5+	2	1	1	34	62	43	23	20	18	A-7-6 (13)	
INTERBEDDED SHALE (94%) AND LIMESTONE (6%); **SHALE**, GRAY, SEVERELY WEATHERED, VERY WEAK;.		TR	32 36 50	129	100	SS-6	-	-	-	-	-	-	-	-	-	9	Rock (V)	
	535.0		44 50/1"	-	100	SS-6	-	-	-	-	-	-	-	-	-	-	Rock (V)	
SHALE, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH. NOTE: SHALE IS SEVERELY WEATHERED FROM 17' TO 17.3' WITH REMAINING ROCK HIGHLY TO MODERATELY WEATHERED NOTE: SHALE IS VERTICALLY FRACTURED AND IRON STAINED FROM 18.1' TO 18.4' NOTE: THIN, GRAY AND WHITE AND GRAY, FOSSILIFEROUS AND CRYSTALLINE LIMESTONE SEAMS RANGING FROM 0.25" TO 1.25" IN THICKNESS FROM 20.2' TO 20.9' @17.0' - 21.0'; POINT LOAD INDEX STRENGTH = 73 PSI NOTE: ALL SHALE AND LIMESTONE EFFERVESCE FREELY WITH DILUTE HCL FROM 17.0' TO 27.7' AND BEDDING IS GENERALLY HORIZONTAL	531.0		69		94	NQ2-1											Rock (V)	
INTERBEDDED SHALE (93%) AND LIMESTONE (7%); **SHALE**, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. @21.7'; COMPRESSIVE STRENGTH OF INTACT ROCK = 439 PSI **LIMESTONE**, GRAY & WHITE, VERY WEAK. NOTE: GRAY AND WHITE AND GRAY, FOSSILIFEROUS AND CRYSTALLINE, LIMESTONE SEAMS WITH BROWN (IRON STAINED) LIME CLAY/SHALE FROM 21.0' TO 21.7' @21.0'- 26.0'; POINT LOAD INDEX STRENGTH = 240 PSI NOTE: VERTICAL FRACTURES WITH IRON STAINING FROM 22.1' TO 22.5' AND AT 23.5' @24.2'; COMPRESSIVE STRENGTH OF INTACT ROCK = 359 PSI	526.0		100		100	NQ2-2											Rock (V)	
INTERBEDDED SHALE (86%) AND LIMESTONE (14%); **SHALE**, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. **LIMESTONE**, GRAY & WHITE, VERY WEAK. NOTE: GRAY TO WHITE AND GRAY, FOSSILIFEROUS AND CRYSTALLINE LIMESTONE SEAMS RANGE FROM 0.75" TO 1.25" IN THICKNESS AND ARE STRONG TO VERY STRONG	524.3	EOB	20		100	NQ2-3											Rock (V)	

NOTES: GROUNDWATER WAS NOT ENCOUNTERED DURING DRILLING AND NO WATER READING WAS TAKEN UPON COMPLETION BECAUSE WATER WAS USED FOR ROCK CORING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH BENTONITE GROUT

COMPANY: PGI DRILLED BY: TERRACON
PROJECT: PEDESTRIAN BRIDGE REPLACEMENT
BRIDGE NO.: HAM-71-1.80
BORING: B-005-0-23 BOX 1/1
DATE of CORING: 6/8/23

RUN-1/NQ2-1:	17.0' - 21.0'	REC: 94%	RQD: 69%
RUN-2/NQ2-2:	21.0' - 26.0'	REC: 100%	RQD: 89%
RUN-3/NQ2-3:	26.0' - 27.7'	REC: 100%	RQD: 20%

Pro Geotech, Inc.

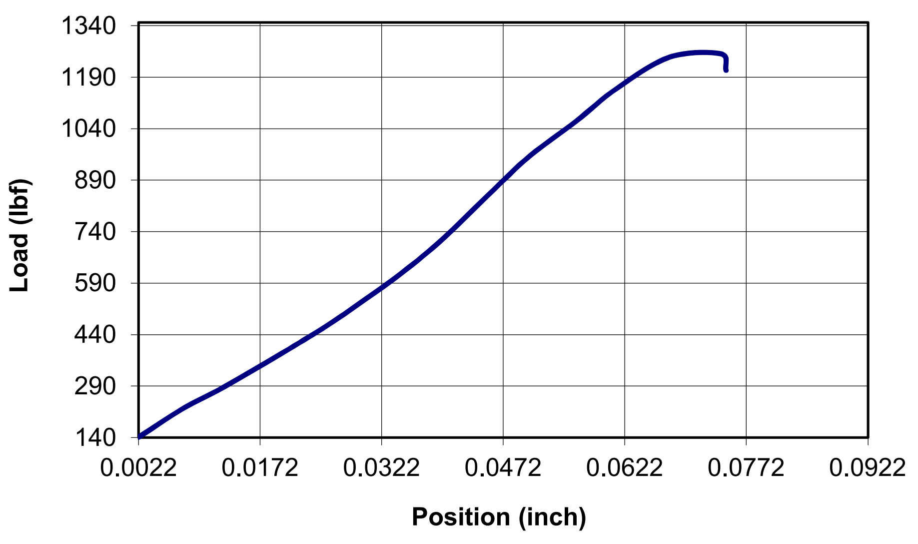
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	12/19/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-005-0-23	TOP DEPTH (FT)	21.667	BOTTOM DEPTH (FT)	21.99
SAMPLE NUMBER	NQ2-1	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	8+30	OFFSET	2'	OFFSET DIRECTION	RT

FORMATION	SHALE
DESCRIPTION	GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		LENGTH/DIAMETER	2.03
1	3.890	1.900		CORRECTION FACTOR	1.00
2	3.870	1.910		AREA (SQ. INCH)	2.865
3	3.880	1.920		MASS (GRAMS)	411.25
AVERAGE	3.880	1.910		UNIT WEIGHT (LBS/FT^3)	140.93

MAXIMUM LOAD (LBS)	1258
COMPRESSIVE STRENGTH (PSI)	439
TIME OF TEST (MINUTES)	5:00
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	NA/DS




BEFORE TESTING


AFTER FAILURE

Pro Geotech, Inc.

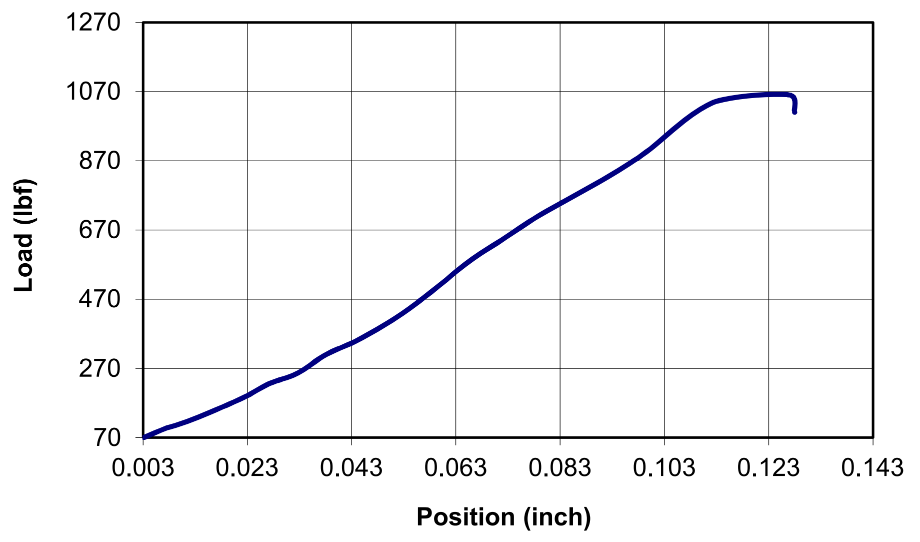
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	12/19/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-005-0-23	TOP DEPTH (FT)	24.167	BOTTOM DEPTH (FT)	24.492
SAMPLE NUMBER	NQ2-2	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	8+30	OFFSET	2'	OFFSET DIRECTION	RT

FORMATION	SHALE
DESCRIPTION	GRAY, MODERATELY TO HIGHLYWEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, MODERATELY TO HIGHLY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)			
				LENGTH/DIAMETER	2.01
1	3.890	1.940		CORRECTION FACTOR	1.00
2	3.900	1.950		AREA (SQ. INCH)	2.956
3	3.910	1.930		MASS (GRAMS)	420.67
AVERAGE	3.900	1.940		UNIT WEIGHT (LBS/FT^3)	139.01

MAXIMUM LOAD (LBS)
1060
COMPRESSIVE STRENGTH (PSI)
359
TIME OF TEST (MINUTES)
7:00
LOADING DIRECTION
PERPENDICULAR TO BEDDING
TECHNICIAN
NA/DS



BEFORE TESTING

AFTER FAILURE

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-005-0-23 **Date:** 7/11/2023

Project No.: G23006G **Depth Range:** NQ2-1 - 17.0' - 21.0' **Technician:** NA

Rock Description: SHALE, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK THINLY LAMINATED, CALCAREOUS, MODERARELY TO HIGHLY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.1	27.5	10	14.85	1754	5.46	0.923	5.04
2	Axial ⊥	49.9	28.5	5	7.43	1809	2.65	0.930	2.46
3	Axial ⊥	50.2	15.0	0	0.00	959	0.00	0.806	0.00
4	Axial ⊥	49.9	26.0	10	14.85	1652	5.80	0.911	5.28
5	Axial ⊥	50.0	22.0	20	29.70	1401	13.68	0.878	12.01
6	Axial ⊥	50.3	28.0	25	37.13	1793	13.36	0.928	12.39
7	Axial ⊥	49.7	24.0	0	0.00	1519	0.00	0.894	0.00
8	Axial ⊥	50.1	24.0	0	0.00	1531	0.00	0.896	0.00
9	Axial ⊥	50.0	28.0	35	51.98	1783	18.81	0.927	17.43
10	Axial ⊥	49.8	26.0	0	0.00	1649	0.00	0.911	0.00
11	Axial ⊥	49.7	20.0	30	44.55	1266	22.71	0.858	19.48

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	6.07
UCS = $I_{s(50)}$ x 12 (psi)	73

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-005-0-23 **Date:** 12/26/2023

Project No.: G23006G **Depth Range:** NQ2-2 - 21.0' - 26.0' **Technician:** NA

Rock Description: SHALE, GRAY, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK THINLY LAMINATED, CALCAREOUS, MODERARELY TO HIGHLY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.1	36.0	150	222.75	2297	62.57	0.981	61.39
2	Axial ⊥	48.6	23.0	100	148.50	1422	67.39	0.881	59.35
3	Axial ⊥	50.7	30.0	5	7.43	1937	2.47	0.944	2.34
4	Axial ⊥	50.9	22.0	60	89.10	1426	40.32	0.881	35.53
5	Axial ⊥	50.0	25.0	5	7.43	1590	3.01	0.903	2.72
6	Axial ⊥	50.7	17.0	60	89.10	1097	52.38	0.831	43.52
7	Axial ⊥	50.8	25.0	50	74.25	1617	29.62	0.907	26.86
8	Axial ⊥	50.4	22.0	5	7.43	1412	3.39	0.879	2.98
9	Axial ⊥	50.1	29.0	5	7.43	1850	2.59	0.934	2.42
10	Axial ⊥	50.1	18.0	5	7.43	1148	4.17	0.839	3.50
11	Axial ⊥	50.0	20.0	5	7.43	1273	3.76	0.859	3.23

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	20.01
UCS = $I_{s(50)}$ x 12 (psi)	240

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

PROJECT: HAM-71-1.80 & HAM-22-10.93	DRILLING FIRM / OPERATOR: TERRACON / K. H.	DRILL RIG: CME 55/300 ATV/T	STATION / OFFSET: 9+88, 0' LT.
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: TERRACON / J.H.	HAMMER: CME AUTOMATIC	ALIGNMENT: RAMP CONST. CETERLINE
PID: 102790 STR ID: HAM-71-1.80	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 1/13/23	ELEVATION: 564.8 (MSL) EOB: 22.0 ft.
START: 6/7/23 END: 6/7/23	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 90	COORD: 39.106921, -84.503044

EXPLORATION ID B-006-0-23

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 564.8	DEPTHS
TOPSOIL (3" IN THICKNESS)	564.5	TR
SHALE, GRAY, SEVERELY WEATHERED, VERY WEAK.	558.8	6
SHALE, GRAY, HIGHLY WEATHERED, VERY WEAK TO WEAK.	557.8	7
INTERBEDDED SHALE (99%) AND LIMESTONE (1%); **SHALE**, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, FRACTURED TO MODERATELY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. NOTE: 1" THICK CLAY SEAM AT 8.0' NOTE: 0.5" THICK GRAY AND WHITE LIMESTONE SEAM AT 8.5' @8.5'; COMPRESSIVE STRENGTH OF INTACT ROCK = 187 PSI NOTE: ALL SHALE AND LIMESTONE EFFERVESCE FREELY WITH DILUTE HCL FROM 7.0' TO 22.0' AND BEDDING IS GENERALLY HORIZONTAL	552.8	12
INTERBEDDED SANDSTONE (82%) AND LIMESTONE (18%); **SANDSTONE**, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY FRACTURED TO MODERATELY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. **LIMESTONE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG. @14.7'; COMPRESSIVE STRENGTH OF INTACT ROCK = 261 PSI @12.0' - 17.0'; POINT LOAD INDEX STRENGTH = 33 PSI NOTE: SOME GRAY AND GRAY AND WHITE, FOSSILIFEROUS, CRYSTALLINE LIMESTONE SEAMS THROUGHOUT RUN RANGING FROM 1" TO 3" IN THICKNESS NOTE: LIMESTONE IS MODERATELY WEATHERED TO UNWEATHERED AND IS SLIGHTLY STRONG TO VERY STRONG NOTE: LITTLE IRON STAINING TYPICAL IN VERTICAL FRACTURES IN LIMESTONE AT 12.3', 14.0', 14.5', 15.0' AND 15.6'	547.8	17
INTERBEDDED SANDSTONE (81%) AND LIMESTONE (19%) ; **SANDSTONE**, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY FRACTURED TO MODERATELY FRACTURED, SLIGHTLY ROUGH, TIGHT TO NARROW APERTURE WIDTH;. **LIMESTONE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG. @17.0' - 22.0'; POINT LOAD INDEX STRENGTH = 350 PSI NOTE: GRAY AND GRAY AND WHITE, FOSSILIFEROUS AND CRYSTALLINE LIMESTONE SEAMS RANGE FROM 0.25" TO 5" IN THICKNESS NOTE: LIMESTONE IS MODERATELY WEATHERED TO UNWEATHERED, FOSSILIFEROUS, CRYSTALLINE AND SLIGHTLY STRONG TO VERY STRONG NOTE: 2.5" THICK IRON STAINED SHALE AND LIMESTONE AT 19.3' WITH 10 DEGREE FROM HORIZONTAL BEDDING AT TOP OF 5" THICK LIMESTONE SEAM	542.8	EOB 22

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
4 10 22	48	100	SS-1	-	-	-	-	-	-	-	-	-	13	Rock (V)
19 24 43	100	100	SS-2	-	-	-	-	-	-	-	-	-	10	Rock (V)
26 50	-	100	SS-3	-	-	-	-	-	-	-	-	-	9	Rock (V)
33		43	NQ2-1											Rock (V)
69		100	NQ2-2											Rock (V)
60		100	NQ2-3											Rock (V)

HOLE SEALED

NOTES: GROUNDWATER WAS NOT ENCOUNTERED DURING DRILLING AND NO WATER READING WAS TAKEN UPON COMPLETION BECAUSE WATER WAS USED FOR ROCK CORING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT

COMPANY: PGI DRILLED BY: TERRACON
PROJECT: PEDESTRIAN BRIDGE REPLACEMENT
BRIDGE NO.: HAM-71-1.80
BORING: B-006-0-23 BOX 1/1
DATE of CORING: 6/7/23
RUN-1/NQ2-1: 7.0' - 12.0' REC: 43% RQD: 33%
RUN-2/NQ2-2: 12.0' - 17.0' REC: 100% RQD: 69%
RUN-3/NQ2-3: 17.0' - 22.0' REC: 100% RQD: 60%

Pro Geotech, Inc.

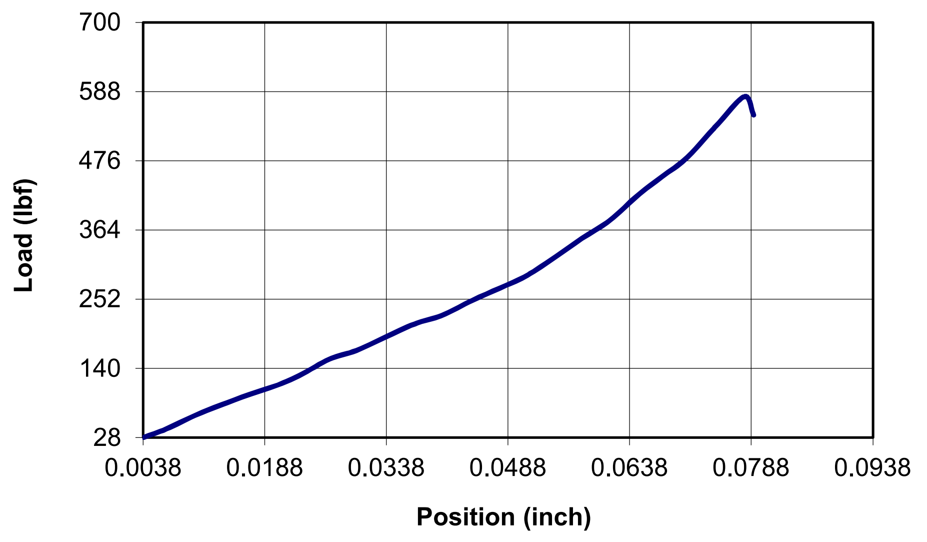
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	12/27/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-006-0-23	TOP DEPTH (FT)	8.5	BOTTOM DEPTH (FT)	8.84
SAMPLE NUMBER	NQ2-1	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	9+88	OFFSET	0	OFFSET DIRECTION	LT

FORMATION	SHALE
DESCRIPTION	GRAY, SEVERELY TO HIGHLYWEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, FRACTURED TO MODERATELY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	4.032	2.000	LENGTH/DIAMETER	2.04
2	4.080	1.970	CORRECTION FACTOR	1.00
3	4.070	1.990	AREA (SQ. INCH)	3.100
			MASS (GRAMS)	440.50
AVERAGE	4.061	1.987	UNIT WEIGHT (LBS/FT^3)	133.32

MAXIMUM LOAD (LBS)
580
COMPRESSIVE STRENGTH (PSI)
187
TIME OF TEST (MINUTES)
8:20
LOADING DIRECTION
PERPENDICULAR TO BEDDING
TECHNICIAN
NA/DS



BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

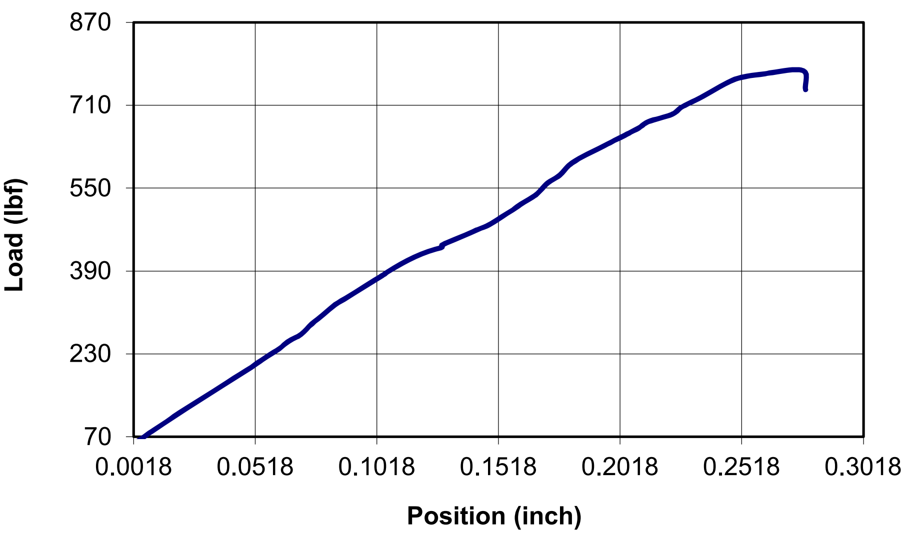
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAM-71-1.80	PGI PROJECT NO.	G23006G	DATE	12/19/2023
	STRUCTURE	PEDESTRIAN BRIDGE OVER GILBERT AVE, I-471, AND 1-71			
BORING NUMBER	B-006-0-23	TOP DEPTH (FT)	14.67	BOTTOM DEPTH (FT)	15
SAMPLE NUMBER	NQ2-2	DISTRICT	8	PID NO.	102790
COUNTY	HAM	ROUTE	71	SECTION	1.81
STATION	9+88	OFFSET	0	OFFSET DIRECTION	LT

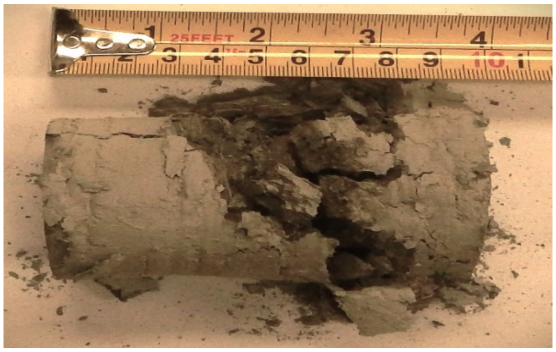
FORMATION	SHALE
DESCRIPTION	GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED, CALCAREOUS, HIGHLY FRACTURED TO MODERATELY FRACTURED

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	3.920	1.980	LENGTH/DIAMETER	2.00
2	3.890	1.990	CORRECTION FACTOR	1.00
3	3.850	1.870	AREA (SQ. INCH)	2.976
			MASS (GRAMS)	419.20
AVERAGE	3.887	1.947	UNIT WEIGHT (LBS/FT^3)	138.05

MAXIMUM LOAD (LBS)
777
COMPRESSIVE STRENGTH (PSI)
261
TIME OF TEST (MINUTES)
15:00
LOADING DIRECTION
PERPENDICULAR TO BEDDING
TECHNICIAN
NA/DS




BEFORE TESTING


AFTER FAILURE

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-006-0-23 **Date:** 12/26/2023

Project No.: G23006G **Depth Range:** NQ2-2 - 12.0' - 17.0' **Technician:** NA

Rock Description: SHALE, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK THINLY LAMINATED, CALCAREOUS, HIGHLY TO MODERARELY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	49.0	35.0	5	7.43	2184	2.19	0.970	2.13
2	Axial ⊥	50.2	24.0	5	7.43	1534	3.12	0.896	2.80
3	Axial ⊥	50.9	25.0	5	7.43	1620	2.96	0.907	2.68
4	Axial ⊥	50.2	20.0	5	7.43	1278	3.75	0.860	3.22
5	Axial ⊥	50.0	21.0	5	7.43	1337	3.58	0.869	3.11
6	Axial ⊥	50.3	31.0	5	7.43	1985	2.41	0.949	2.29
7	Axial ⊥	51.1	25.0	5	7.43	1627	2.95	0.908	2.67
8	Axial ⊥	50.4	22.0	5	7.43	1412	3.39	0.879	2.98
9	Axial ⊥	49.5	25.0	20	29.70	1576	12.16	0.901	10.96
10	Axial ⊥	50.4	27.0	5	7.43	1733	2.76	0.921	2.55
11	Axial ⊥	50.5	27.0	5	7.43	1736	2.76	0.921	2.54

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	2.76
UCS = $I_{s(50)}$ x 12 (psi)	33

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

Point Load Test (ASTM D 5731)

Project: HAM-71-1.80 **Boring No.:** B-006-0-23 **Date:** 7/11/2023

Project No.: G23006G **Depth Range:** NQ2-3 - 17.0' - 22.0' **Technician:** NA

Rock Description: SHALE, GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK THINLY LAMINATED, CALCAREOUS, HIGHLY TO MODERARELY FRACTURED.

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.0	16.5	65	96.53	1050	59.28	0.823	48.78
2	Axial ⊥	49.9	26.5	0	0.00	1684	0.00	0.915	0.00
3	Axial ⊥	50.2	27.0	70	103.95	1726	38.86	0.920	35.75
4	Axial ⊥	50.2	18.5	60	89.10	1182	48.61	0.845	41.08
5	Axial ⊥	49.3	25.0	75	111.38	1569	45.79	0.901	41.23
6	Axial ⊥	49.7	26.0	50	74.25	1645	29.12	0.910	26.50
7	Axial ⊥	50.2	19.0	50	74.25	1214	39.45	0.850	33.53
8	Axial ⊥	50.0	25.0	50	74.25	1592	30.10	0.903	27.19
9	Axial ⊥	49.9	32.0	100	148.50	2033	47.12	0.955	44.98
10	Axial ⊥	50.1	21.5	10	14.85	1371	6.99	0.874	6.10
11	Axial ⊥	50.0	28.0	5	7.43	1783	2.69	0.927	2.49

Note: Bedrock in Dry Condition

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	29.18
UCS = $I_{s(50)}$ x 12 (psi)	350

Piston Area = 1.485 sq. Inches

L = Applied Pressure

P = Failure Load

⊥ = Load Applied Perpendicular to Bedding

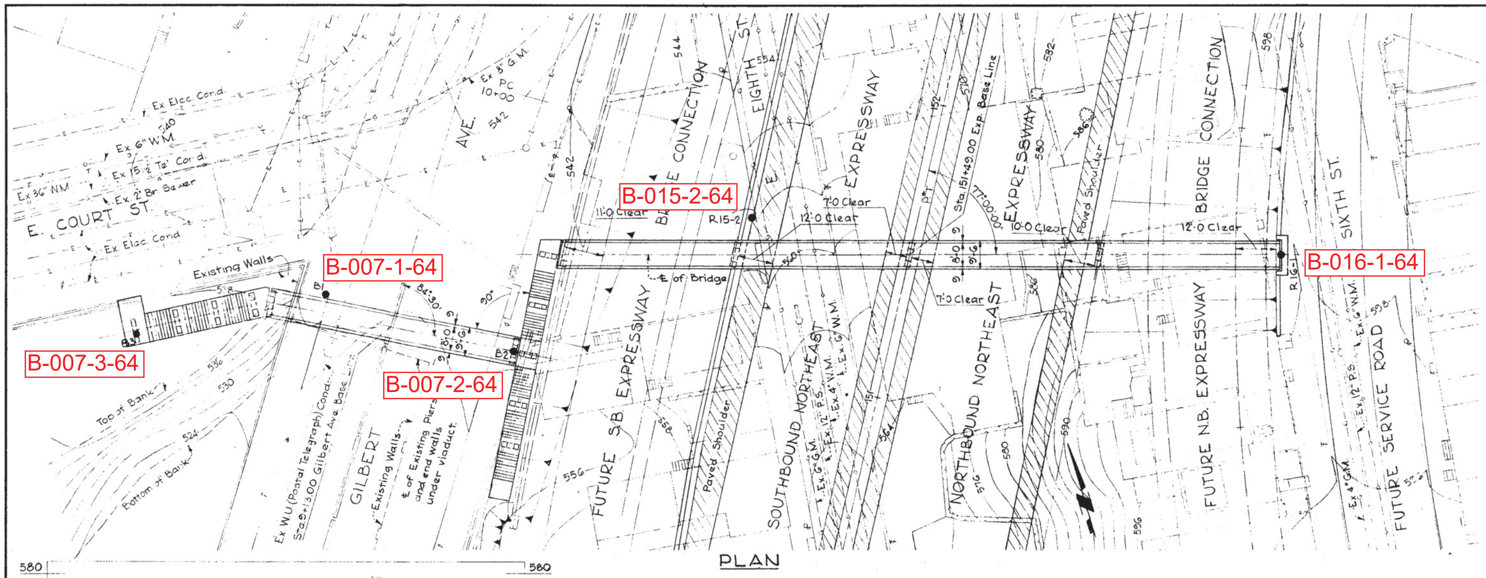
W = Core Sample Diameter

D = Height of Sample

UCS = Unconfined Compressive Strength

FED. RD. DIV.	STATE	PROJECT	FISCAL YEAR
2	OHIO		

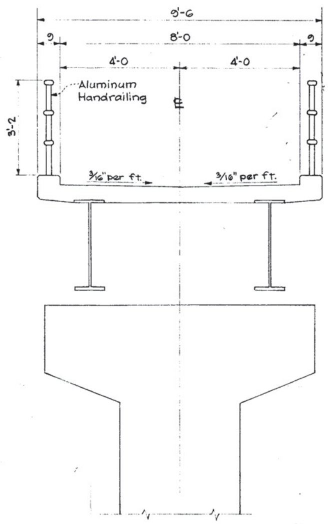
6



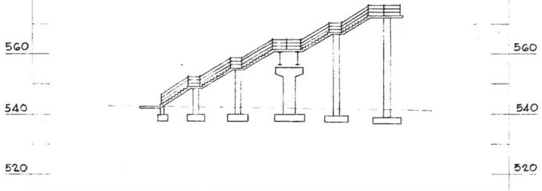
PLAN

HISTORIC BORING LOCATION

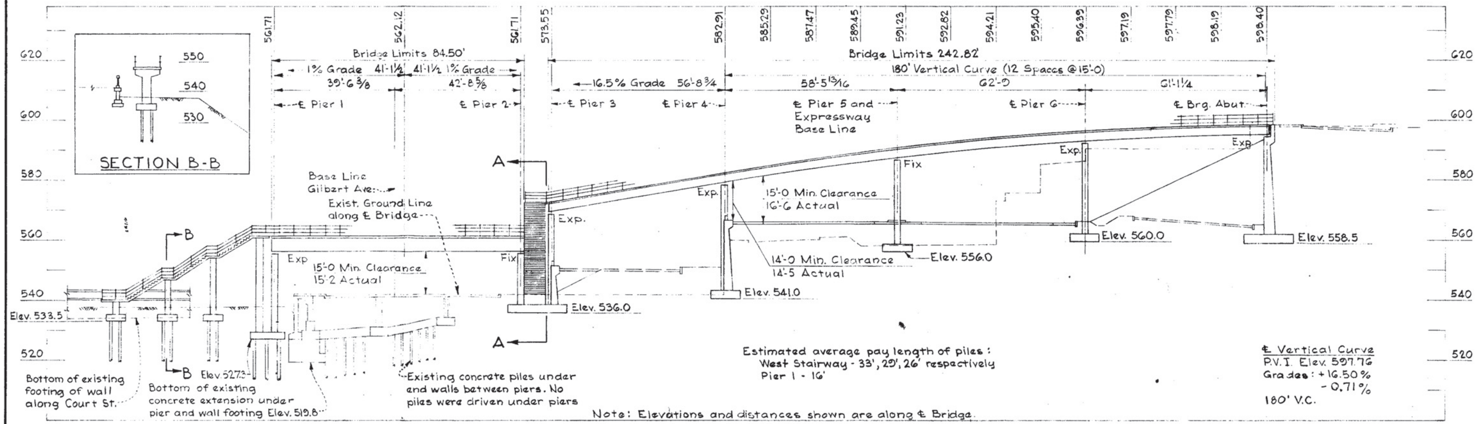
● Symbol denotes Test Boring.



SECTION THRU BRIDGE



ELEVATION A-A



Estimated average pay length of piles:
West Stairway - 33', 29', 26' respectively
Pier 1 - 16'

℄ Vertical Curve
P.V.I. Elev. 597.76
Grades: + 16.50 %
- 0.71 %
180' V.C.

Note: Elevations and distances shown are along ℄ Bridge

ELEVATION

PROPOSED STRUCTURE

TYPE: 4 Span Continuous Rolled Beam with reinforced concrete deck and substructure.
Single Span Plate Girder with reinforced concrete deck and substructure.
SPANS: 56.73', 58.48', 62.75', 61.11' (Rolled Beam Structure)
82.25' (Plate Girder Structure).
WIDTH: 8'-0 Clear between curbs.
LOADING: 85 lb./sq. ft. (uniform)
SKEW: As shown on Plan.

CITY OF CINCINNATI
DEPARTMENT OF PUBLIC WORKS
DIVISION OF ENGINEERING

SITE PLAN

BRIDGE NO. HAM-71-0157

I-71 UNDER PEDESTRIAN CROSSING

CINCY BRIDGE NO. 7

DESIGNED	DRAWN	TRACED	CHECKED	REVIEWED	DATE	REVISED
W.W.	W.W.	J.D.A.				

Form No. 530—16-58

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-007-1-64

TESTING ENGINEERS AND SOILS CONSULTANTS

Page 1 of 2

Sta. & Offset: 7+40.40, 42.3' LT

LOG OF BORING

DATE STARTED 7/27/64 SAMPLER: TYPE Split Spoon & Core Barrel DIA. 2"O.D.& NXM WATER ELEV. IMMEDIATE 524.3 CLIENT: City of Cincinnati, Ohio

DATE COMPLETED 7/27/64 CASING: LENGTH ____ DIA 3.5"I.D.Hollow Stem Augers AFTER ____ HOURS Backfilled PROJECT: Northeast Expressway HAM-71-0.93 Bridge No. HAM-71-0157

BORING No. B-7-1 STATION AND OFFSET 150+86,191'L. BL SURFACE ELEV. 541.8 541.2

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics								
						% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
541.8	0													
541.3					0.5'Concrete									
539.3	2	1	3-2-2	8"	Brown and gray silty clay and cinders (fill), moist - loose	Visual classification-No tests performed								Fill
536.8	4	2	3-3-3	12"	Brown and gray silty clay with rock fragments(fill), moist - medium stiff	Visual classification-No tests performed								Fill
	6	3	2-2-3	15"	Brown and gray silty clay with organic matter and cinders (fill), moist - medium stiff	Visual classification-No tests performed								Fill
	8													
531.8	10	4	3-4-5	16"	do do	Visual classification-No tests performed								Fill
	12	5	5-4-6	8"	Brown and gray silty clay with brick, (fill), moist - stiff	Visual classification-No tests performed								Fill
526.8	14													
	16	6	6-12-40	9"	Brick and limestone fragments with clay (fill), moist - very dense	Visual classification-No tests performed								Fill
	18	7	95	5"	do do	Visual classification-No tests performed								Fill
521.8	20													
519.3	22	8	8-12-5	10"	Brick and concrete with clay (fill), wet - medium dense	Visual classification-No tests performed								Fill
516.8	24	9	2-2-3	11"	Gray silty clay with brick and organic matter (fill), wet - soft	Visual classification-No tests performed								Fill
514.8	26	10	9-12-9	10"	Brick and concrete with brown and gray weathered shale, wet - stiff	Visual classification-No tests performed								Fill
	28	11	NXM	73%	Gray shale, calcareous,moderately tough to tough (1" to 2" pieces,averaging 1½") and gray limestone, crystalline, fossiliferous (1½" to 5½" pieces, averaging 4") 77% shale, 23%limestone									
	30													
	32													
	34													

Form No. 530—16

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-007-1-64

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

Sta. & Offset: 7+40.40, 42.3' LT

DATE STARTED 7/27/64 SAMPLER: TYPE Split Spoon & CoreBarel DIA. 2"O.D.& NXM WATER ELEV. IMMEDIATE 524.3 CLIENT: City of Cincinnati, Ohio

DATE COMPLETED 7/27/64 CASING: LENGTH ____ DIA 3.5"I.D.Hollow Stem Augers AFTER ____ HOURS Backfilled PROJECT: Northeast Expressway HAM-71-0.93

BORING No. B-7-1 STATION AND OFFSET 150+86,191' L. BL SURFACE ELEV. 541.8 541.2 Bridge No. HAM-71-0157

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics								
						% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
504.8	34 36	12	NXM	72%	Gray shale, calcareous, tough(1/2" to 3" pieces averaging 2") and gray limestone, crystalline, fossiliferous, (1/2"to 4"pieces averaging 2") 87% shale, 13% limestone									
	38 40 42 44 46 48				Boring completed									

Form No. 530—16

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-007-2-64

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

Sta. & Offset: 7+90.37, 22.1' LT

DATE STARTED 7/22/64 SAMPLER: TYPE Split Spoon & Core Barrel DIA. 2"O.D.& NXM WATER ELEV. IMMEDIATE None CLIENT: City of Cincinnati, Ohio

DATE COMPLETED 7/23/64 CASING: LENGTH ____ DIA. 3.5"I.D.Hollow Stem Augers AFTER ____ HOURS Backfilled PROJECT: Northeast Expressway HAM-71-0.93 Bridge No. HAM-71-0157

BORING No. B-7-2 STATION AND OFFSET 150+75, 124'L. BL SURFACE ELEV. 542.0 541.4

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics								
						% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
542.0	0													
541.6					Concrete (0.4')									
	2	1	2-9-4	3"	Brown and gray silty clay with gravel and cinders (fill), moist - soft	Visual classification-No tests performed								Fill
539.5														
	4	2	4-9-9	14"	Brown and gray clay, trace sand, trace gravel with shale and limestone fragments, moist - stiff	8	6	3	33	50	43	19	25	A-7-6
	6	3	8-16-25	13"	Brown and gray clay, trace sand, trace gravel with limestone fragments, moist-very stiff	1	2	2	38	57	42	18	13	A-7-6
534.5	8													
		4	19-75	8"	Brown and gray silty clay, trace sand, little gravel with limestone fragments, moist - very stiff	11	5	4	34	46	40	16	16	A-6b
532.0	10													
	12	5	16-41-77	14"	Brown and gray silt and clay, little sand, trace gravel with limestone fragments, moist - very stiff	3	9	4	32	52	39	15	14	A-6a
	14	6	76-45-70	17"	Brown and gray silt and clay, trace sand, trace gravel with limestone fragments, moist - hard	7	3	2	36	52	38	14	12	A-6a
527.0														
	16	7	23-65-80	16"	Brown and gray weathered shale, hard	Visual classification-No tests performed								
	18	8	51-70	9"	do do	Visual classification-No tests performed								
522.0	20													
	22	9	NXM	67%	Gray and brown shale, calcareous, moderately tough (1" to 4½" pieces averaging 2") and gray limestone, crystalline, fossiliferous (two pieces ½" and 4½") 92% shale, 8% limestone									
	24													
	26				Gray shale, calcareous, moderately tough with brown and gray weathered zones in the bottom half of the run (1" to 2½" pieces averaging 1½") and gray limestone, crystalline, fossiliferous, jointed in the bottom 6" of the run (1" to 4½" pieces averaging 3") 79% shale, 21% limestone									
	28	10	NXM	71%										
512.0	30													
	32				Boring completed									
	34													

Form No. 53 6-59

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-007-3-64

TESTING ENGINEERS AND SOILS CONSULTANTS

Page 1 of 2

LOG OF BORING

Sta. & Offset: 0+33.35, 3.1' LT

DATE STARTED 9/9/64 SAMPLER: TYPE Split Spoon & Core Barrel DIA. 2"O.D. & NXM WATER ELEV. IMMEDIATE None CLIENT: City of Cincinnati, Ohio

DATE COMPLETED 9/9/64 CASING: LENGTH ____ DIA. 3.5"I.D.Hollow Stem Augers AFTER 24 HOURS 514.2 PROJECT: Northeast Expressway HAM-71-0.93 Bridge No. HAM-71-0157

BORING No. B-7-3 STATION AND OFFSET 22' S. of S. Curb E. Court Street 78' W. of W. Curb Gilbert Avenue SURFACE ELEV. 537.2 536.6

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics % AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
537.2	0													
		1	18-17-25	10"	Cinders and brick(fill), moist - dense	Visual classification-No tests performed								Fill
	2													
	4	2	17-21-7	9"	do do	Visual classification-No tests performed								Fill
	6	3	18-9-12	10"	do do	Visual classification-No tests performed								Fill
	8	4	10-11-9	16"	do do	Visual classification-No tests performed								Fill
	10	5	6-7-12	15"	do do	Visual classification-No tests performed								Fill
525.2	12													
	14	6	9-8-10	18"	Brown silty clay with gravel and brick fragments (fill), moist - medium stiff	Visual classification-No tests performed								Fill
	16	7	6-7-10	10"	do do	Visual classification-No tests performed								Fill
	18	8	10-14-7	9"	do do	Visual classification-No tests performed								Fill
517.2	20													
	22	9	5-6-9	10"	Dark gray silty clay with brick fragments (fill), moist - stiff	Visual classification-No tests performed								Fill
	24	10	3-4-7	8"	do do	Visual classification-No tests performed								Fill
512.2														
	26	11	4-9-15	14"	Brown and gray clay, trace sand, moist - stiff	0	4	3	38	55	43	18	20	A-7-6
	28	12	15-15-20	17"	Brown and gray clay, trace sand with shale fragments, moist - very stiff	0	1	1	37	61	44	18	16	A-7-6
507.2	30													
	32	13	17-19-27	17"	Brown and gray weathered shale, moist - very stiff to hard	Visual classification-No tests performed								
	34	~~14~~	100	2"	do do	Visual classification-No tests performed								

Form No. 530—16-59

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-007-3-64

TESTING ENGINEERS AND SOILS CONSULTANTS

Page 2 of 2

LOG OF BORING

Sta. & Offset: 0+33.35. 3.1' LT

DATE STARTED 9/9/64 SAMPLER: TYPE Split Spoon & Core Barrel DIA. 2"O.D.& NXM WATER ELEV. IMMEDIATE None CLIENT: City of Cincinnati, Ohio

DATE COMPLETED 9/9/64 CASING: LENGTH ______ DIA. 3.5"I.D.Hollow Stem Augers AFTER 24 HOURS 514.2 PROJECT: Northeast Expressway

BORING No. B-7-3 STATION AND OFFSET 22' S. of S. Curb E. Court Street 78' W. of W. Curb Gilbert Avenue SURFACE ELEV. 537.2 536.6 HAM-71-0.93 Bridge No. HAM-71-0157

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics								
						% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
503.2	34													
498.2	36 38	15	NXM	60%	Brown and gray weathered shale, moderately tough with soft seams (½" to 3½" pieces averaging 2") and gray limestone, crystalline, fossiliferous (three pieces 1" - 1½" and 2") 92% shale, 8% limestone									
	40				Boring completed									
	42													
	44													
	46													
	48													
	50													
	52													
	54													
	56													
	58													
	60													
	62													
	64													
	66													
	68													

13

Form No. 530—16-59

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-015-2-64

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

Sta. & Offset: 8+66.23, 56.1' LT

DATE STARTED 7/31/64 SAMPLER: TYPE Split Spoon & Core Barrel DIA. 2"O.D.& NXM WATER ELEV. IMMEDIATE None CLIENT: City of Cincinnati, Ohio

DATE COMPLETED 7/31/64 CASING: LENGTH ____ DIA. 3.5"I.D.Hollow Stem Augers AFTER ____ HOURS Backfilled PROJECT: Northeast Expressway HAM-71-0.93 Retaining Wall No. 15

BORING No. R-15-2 STATION AND OFFSET 151+38, 39' L., BL SURFACE ELEV. 560.0 559.4

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics % AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
560.0	0													
559.0					Asphaltic concrete									
	2	1	9-8-8	18"	Brown and gray clay, trace sand, trace gravel, moist - very stiff	1	1	2	33	63	45	23	21	A-7-6
	4	2	7-9-10	6"	do do	Visual description-No tests performed								A-7-6
	6	3	6-6-11	16"	Brown and gray clay, trace sand, trace gravel with shale fragments, moist - very stiff	1	4	2	37	56	42	20	20	A-7-6
	8													
551.0		4	16-13-19	18"	Brown clay, trace sand with shale fragments, moist - very stiff	0	3	1	30	66	47	23	21	A-7-6
	10													
	12	5	NXM	43%	Brown and gray weathered shale, slightly tough (½" to 2" pieces averaging 1½") and gray limestone, crystalline, fossiliferous (½" & 1" pieces broken) 95% shale, 5% limestone									
	14													
	16	6	NXM	75%	Brown and gray weathered shale, slightly tough (½" to 2" pieces, averaging 1½") and gray limestone, crystalline, fossiliferous, jointed (1" to 3" pieces averaging 2") 77% shale, 23% limestone									
	18													
541.0														
	20				Boring completed									
	22													
	24													
	26													
	28													
	30													
	32													
	34													

14

Form No. 630—16

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-016-1-64

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

Water used in drilling Page 1 of 2
from elev

Sta. & Offset: 10+57.08, 35.4' LT

DATE STARTED 7/20/64 SAMPLER: TYPE Split Spoon & Core Barrel DIA. 2"O.D.& NXM WATER ELEV. IMMEDIATE None CLIENT: City of Cincinnati, Ohio

DATE COMPLETED 7/22/64 CASING: LENGTH ____ DIA. 3.5"I.D.Hollow Stem Augers AFTER ____ HOURS Backfilled PROJECT: Northeast Expressway HAM-71-0.93 Retaining Wall No. 16

BORING No. R-16-1 STATION AND OFFSET 151+68, 122' Rt. BL SURFACE ELEV. 598.4 597.8

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics								
						% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
598.4	0													
598.0					0.4' Concrete									
	2	1	2-2-3	12"	Brown and gray clay, little sand,trace gravel, moist - stiff	6	8	4	26	56	49	26	23	A-7-6
	4	2	6-9-14	18"	do do	Visual classification-No tests performed								A-7-6
	6	3	6-10-60	16"	Brown and gray clay, trace sand, trace gravel with limestone fragments, moist-very stiff	1	2	2	31	64	45	22	18	A-7-6
590.9	8													
		4	17-17-34	17"	Brown and gray weathered shale, hard	Visual classification-No tests performed								
	10													
	12	5	22-45-75	17"	do do	Visual classification-No tests performed								
	14	6	17-51-68	17"	do do	Visual classification-No tests performed								
	16	7	28-70	13"	do do	Visual classification-No tests performed								
	18	8	33-74	12"	do do	Visual classification-No tests performed								
	20	9	37-90	13"	do do	Visual classification-No tests performed								
	22													
	24	10	41-87	12"	do do	Visual classification-No tests performed								
573.4	26													
	28	11	NXM	54%	Gray and brown shale,calcareous,slightly tough(weathered in the top 2'of the run) (¼"to 3" pieces,averaging 1") and gray limestone, crystalline, fossiliferous, (one 1½"pieces midway in the run)									
	30													
	32	12	NXM	73%	Gray shale,calcareous,moderately tough to tough(½" to 2½" pieces,averaging 1¼") and gray limestone,crystalline, fossiliferous,jointed midway in the run (1"to 4¼"pieces averaging 2½") 80%shale, 20% limestone									
	34													

15

Form No. 530—16

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-016-1-64

TESTING ENGINEERS AND SOILS CONSULTANTS Water used in drilling from

LOG OF BORING

Sta. & Offset: 10+57.08, 35.4' LT

DATE STARTED 7/20/64 SAMPLER: TYPE Split Spoon & Core Barrel DIA. 2"O.D. NXM WATER ELEV. IMMEDIATE None CLIENT: City of Cincinnati, Ohio

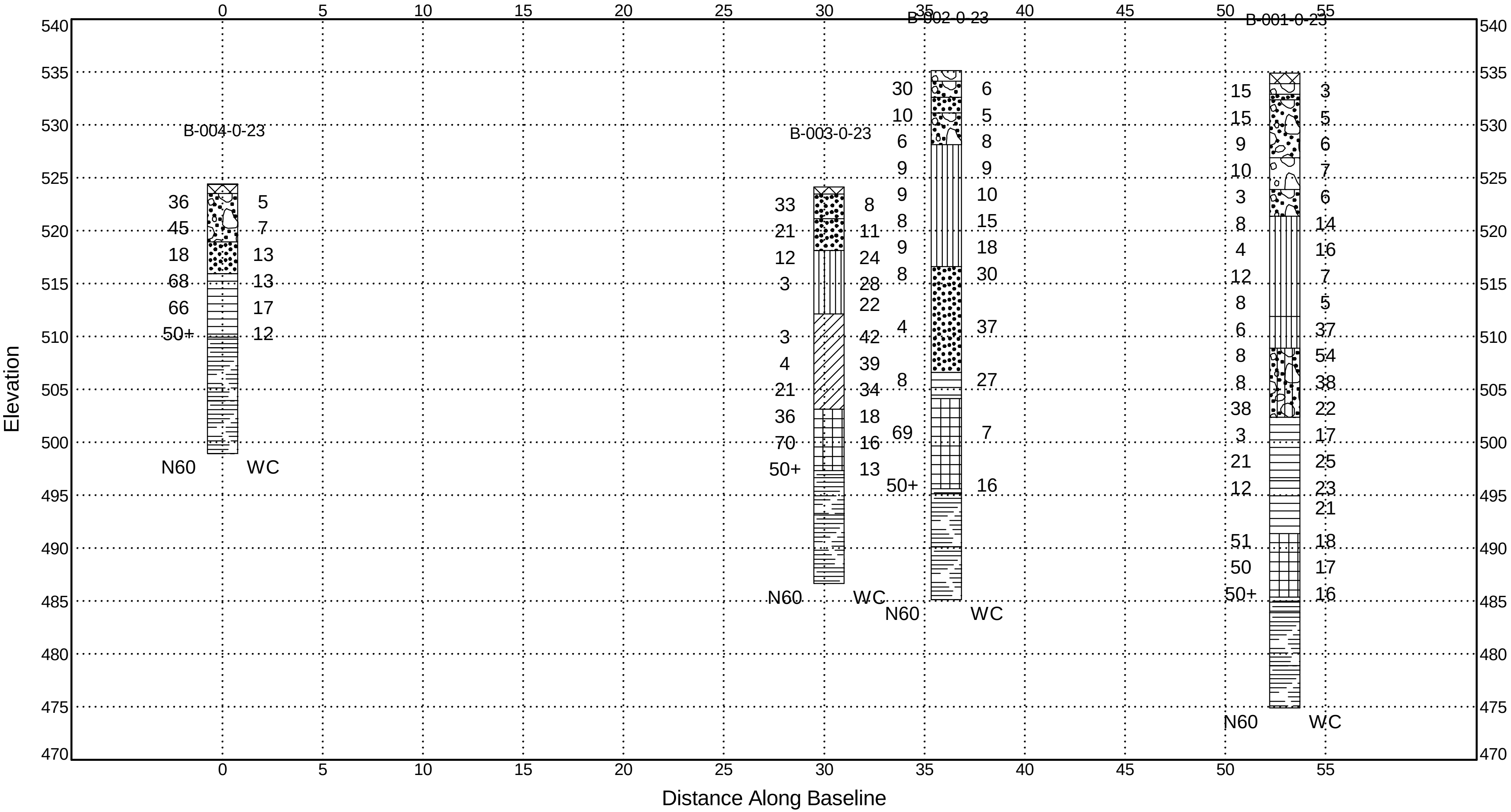
DATE COMPLETED 7/22/64 CASING: LENGTH ____ DIA. 3.5"I.D.Hollow Stem Augers AFTER Backfilled HOURS ____ PROJECT: Northeast Expressway HAM-71-0.93 Retaining Wall No. 16

BORING No. R-16-1 STATION AND OFFSET 151+68,122'Rt.,BL SURFACE ELEV. 598.4 597.8

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics								
						% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
	34													
	36													
	38	13	NXM	82%	Gray shale, calcareous, tough, jointed in top 18" (1" to 3" pieces averaging 2") and gray limestone, crystalline, fossiliferous, jointed and iron oxide stained (one 3" piece) 95% shale, 5% limestone									
	40													
	42													
	44	14	NXM	88%	Gray shale, calcareous, tough (1" to 3" pieces averaging 2") and gray limestone, crystalline, fossiliferous, jointed (1" to 3" pieces averaging 2½") 83% shale, 17% limestone									
	46													
551.9		15	NXM	83%	Gray shale, calcareous, tough (1" to 2½" pieces, averaging 1½") and gray limestone, crystalline, fossiliferous, jointed (two pieces 1½" and 2½") 78% shale, 22% limestone									
	48													
					Boring completed									

16

PROFILE ODOT - PRIMENG.GDT - 7/18/23 19:48 - \\GEOTECHSERVER\SHARED FOLDERS\COMPANY\PUBLIC\PROJECT FILES\23 PROJECTS\G23006G- ACADES HAM-71\LAB DATA SHEETS\G23006G GINT.GP



Borehole	North	East	Elev.	Depth
B-001-0-23	1399764	409569	534.9	60.0
B-002-0-23	1399820	409573	535.1	50.0
B-003-0-23	1399790	409531	524.1	37.5
B-004-0-23	1399862	409514	524.4	25.5

DISTANCES:

Beginning	0
Ending	55

VIEWING ANGLES (degrees):

Horizontal	0.0
Vertical	0.0

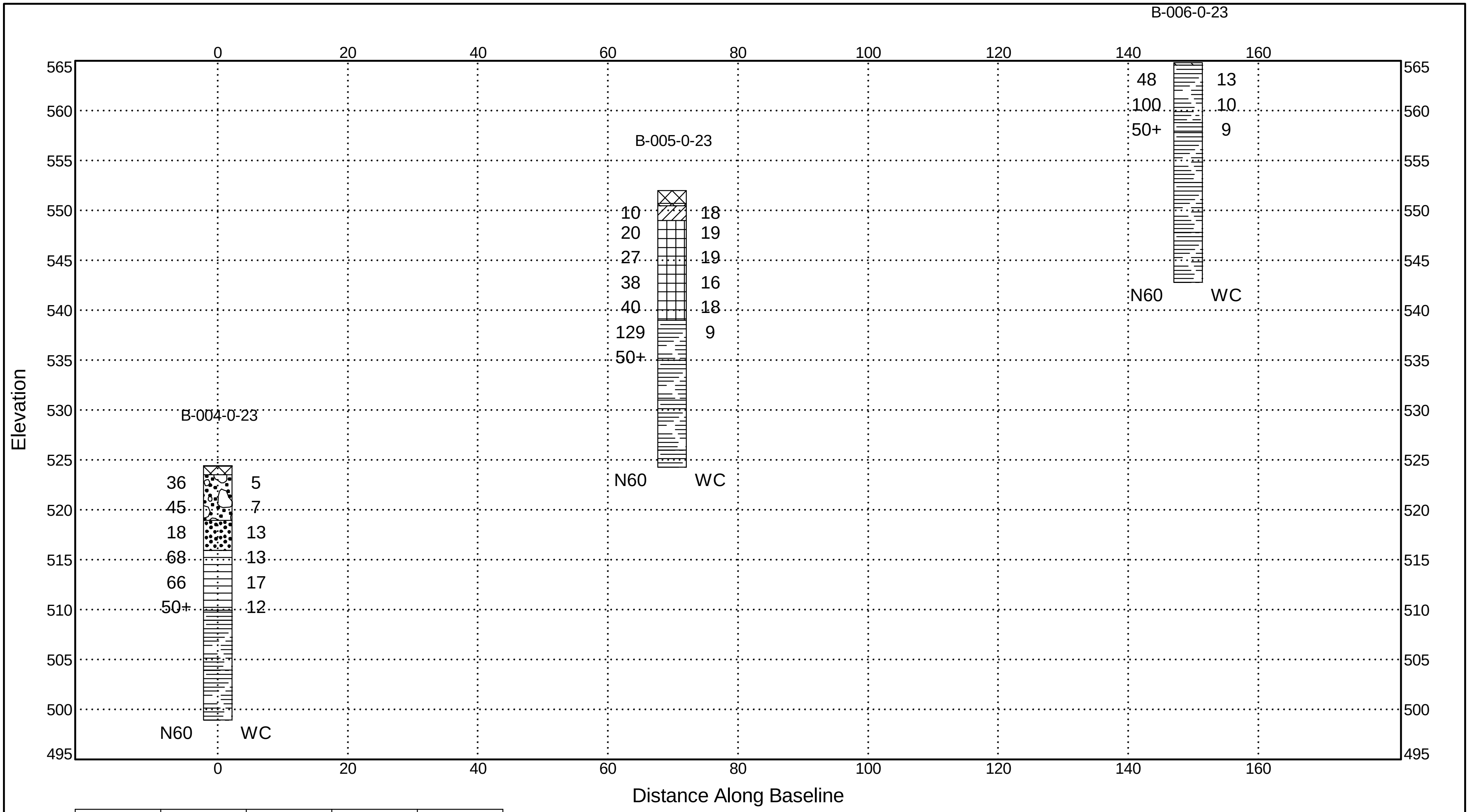
Position	North	East
Left, Front	699924	204751
Right, Front	699888	204792
Left, Back	699924	204751
Right, Back	699888	204792

SOIL/ROCK BORING PROFILE

HAM-71-1.81

HAMILTON COUNTY, OHIO

PID #	DATE	PLATE
102790	Jul 23	1



Borehole	North	East	Elev.	Depth
B-004-0-23	1399862	409514	524.4	25.5
B-005-0-23	1400000	409494	552.0	27.7
B-006-0-23	1400155	409462	564.8	22.0

DISTANCES:

Beginning	0
Ending	160

VIEWING ANGLES (degrees):

Horizontal	0.0
Vertical	0.0

Position	North	East
Left, Front	699931	204758
Right, Front	700088	204730
Left, Back	699931	204758
Right, Back	700088	204730

SOIL/ROCK BORING PROFILE

HAM-71-1.81

HAMILTON COUNTY, OHIO

PID #	DATE	PLATE
102790	Jul 23	2

APPENDIX B

Boring Number	Sample Number	Depth (ft)	Water Content %	Liquid Limit %	Plastic Limit %	Plast. Index	Specific Gravity	Agg. %	Coarse Sand %	Fine Sand %	Silt %	Silt&Clay Comb. %	Clay %	Soil Description	Class. Symbol
B-001-0-23	SS-1A	1.0	3											BLACK ASPHALT AND STONE FRAGMENTS (FILL)	A-1-a (V)
B-001-0-23	SS-1B	2.0	9											BLK COARSE AND FINE SAND SOME FINES, LITTLE STONE FRAGMENTS (FILL)	A-3a (V)
B-001-0-23	SS-2	3.5	5	NP	NP	NP		34	25	31	1	10	9	BROWN GRAVEL & STONE FRAGMENTS WITH SAND, TRACE FINES (FILL)	A-1-b (0)
B-001-0-23	SS-3	6.0	6											BROWN GRAVEL & STONE FRAGMENTS WITH SAND, TRACE FINES (FILL)	A-1-b (V)
B-001-0-23	SS-4	8.5	7											BROWN GRAVEL & STONE FRAGMENTS "AND" SAND, TRACE FINES (FILL)	A-1-a (V)
B-001-0-23	SS-5	11.0	6	NP	NP	NP		52	30	12	3	6	3	BROWN GRAVEL & STONE FRAGMENTS "AND" SAND, TRACE FINES (FILL)	A-1-a (0)
B-001-0-23	SS-6	13.5	14											BROWN SANDY SILT, LITTLE CLAY, SOME BRICK & STONE FRAGS (FILL)	A-4a (V)
B-001-0-23	SS-7	16.0	16											BROWN SANDY SILT, LITTLE CLAY, SOME BRICK & STONE FRAGS (FILL)	A-4a (V)
B-001-0-23	SS-8	18.5	7											BROWN SANDY SILT, LITTLE CLAY, SOME STONE FRAGMENTS (FILL)	A-4a (V)
B-001-0-23	SS-9	21.0	5											BROWN SANDY SILT, LITTLE CLAY, SOME STONE FRAGMENTS (FILL)	A-4a (V)
B-001-0-23	SS-10	23.5	37											BLK NON-PLASTIC SANDY SILT, LITTLE STONE FRAGMENTS (FILL)	A-4a (V)
B-001-0-23	SS-11	26.0	54	NP	NP	NP		36	17	18	14	29	15	DK BROWN STONE FRAGMENTS WITH SAND AND SILT, LITTLE CLAY (FILL)	A-2-4 (0)
B-001-0-23	SS-12	28.5	38											DK BROWN STONE FRAGMENTS WITH SAND AND SILT, LITTLE CLAY (FILL)	A-2-4 (V)
B-001-0-23	SS-13	31.0	22											DK BROWN STONE FRAGMENTS WITH SAND AND SILT, LITTLE CLAY (FILL)	A-2-4 (V)
B-001-0-23	SS-14	33.5	17											DARK BROWN SILTY CLAY, LITTLE SAND, LITTLE STONE FRAGMENTS (FILL)	A-6b (V)
B-001-0-23	SS-15	36.0	25	37	19	18		13	6	8	31	72	41	DK BR & GRAY SILTY CLAY, LITTLE SAND, LITTLE STONE FRAGMENTS (FILL)	A-6b (11)
B-001-0-23	SS-16	38.5	23	35	18	17		0	1	2	48	97	49	BROWN AND GRAY SILTY CLAY, TRACE SAND	A-6b (11)
B-001-0-23	ST-17	40.0	21											BROWN SILTY CLAY, TRACE SAND	A-6b (V)
B-001-0-23	SS-18	43.5	18											BROWN CLAY, TRACE SAND, TRACE STONE FRAGMENTS	A-7-6 (V)
B-001-0-23	SS-19	46.0	17											BROWN CLAY, TRACE SAND, TRACE STONE FRAGMENTS	A-7-6 (V)
B-001-0-23	SS-20A	48.5	16											BROWN CLAY, TRACE SAND, TRACE STONE FRAGMENTS	A-7-6 (V)
B-001-0-23	SS-20B	49.5												GRAY HIGHLY WEATHERED SHALE	Rock (V)
B-002-0-23	SS-1	1.0	6											BLK AND GRAY CONCRETE AND STONE FRAGS WITH SAND, LITTLE FINES (FILL)	A-1-b (V)
B-002-0-23	SS-2	3.5	5											BROWN SANDY SILT, SOME CLAY, SOME STONE FRAGMENTS (FILL)	A-4a (V)
B-002-0-23	SS-2	4.0	5	NP	NP	NP		27	24	32	8	17	9	BROWN GRAVEL & STONE FRAGMENTS WITH SAND, LITTLE FINES (FILL)	A-1-b (0)
B-002-0-23	SS-3A	6.0	8											BROWN GRAVEL & STONE FRAGMENTS WITH SAND, LITTLE FINES (FILL)	A-1-b (V)
B-002-0-23	SS-3B	7.0												BROWN STONE FRAGMENTS WITH SAND AND SILT, LITTLE CLAY (FILL)	A-2-4 (V)
B-002-0-23	SS-4	8.5	9											BROWN STONE FRAGMENTS WITH SAND AND SILT, LITTLE CLAY (FILL)	A-2-4 (V)
B-002-0-23	SS-5	11.0	10	21	16	5		35	19	15	13	30	17	BROWN STONE FRAGMENTS WITH SAND AND SILT, LITTLE CLAY (FILL)	A-2-4 (0)

Pro Geotech, Inc.

TR.-TRACE, BR.-BROWN, LI.-LITTLE, S/F-STONE FRAGMENTS, SO.-SOME, RB-ROADBASE, NP-NON-PLASTIC, POSS-POSSIBLE

Summary of Laboratory Results

Client: ARCADIS U.S., INC
Project: HAM-71-1.80 & HAM-22-10.93
Location: HAMILTON COUNTY, OHIO
Pro. Number: G23006G

Boring Number	Sample Number	Depth (ft)	Water Content %	Liquid Limit %	Plastic Limit %	Plast. Index	Specific Gravity	Agg. %	Coarse Sand %	Fine Sand %	Silt %	Silt&Clay Comb. %	Clay %	Soil Description	Class. Symbol
B-002-0-23	SS-6	13.5	15											BROWN SANDY SILT, SOME CLAY, LITTLE STONE & BRICK FRAGMENTS (FILL)	A-4a (V)
B-002-0-23	SS-7	16.0	18											BROWN SANDY SILT, SOME CLAY, LITTLE STONE & BRICK FRAGMENTS (FILL)	A-4a (V)
B-002-0-23	SS-8	18.5	30	NP	NP	NP		20	19	23	18	37	19	BLACK SANDY SILT, LITTLE CLAY, LI. CINDERS, COAL FRAGS, & GRAVEL (FILL)	A-4a (0)
B-002-0-23	SS-9	23.5	37											BLACK SANDY SILT, LITTLE CLAY, LI. CINDERS, COAL FRAGS, & GRAVEL (FILL)	A-4a (V)
B-002-0-23	SS-10	28.5	27	38	22	16		7	6	7	41	81	40	BROWN & DRK BROWN SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6b (10)
B-002-0-23	SS-11	33.5	7											BROWN CLAY, "AND" LIMESTONE FRAGMENTS, TRACE SAND	A-7-6 (V)
B-002-0-23	SS-12A	38.5	16	41	22	19		0	1	1	34	97	63	BROWN AND GRAY CLAY, TRACE SAND	A-7-6 (12)
B-002-0-23	SS-12B	39.5												GRAY HIGHLY WEATHERED SHALE	Rock (V)
B-003-0-23	SS-1	1.0	8											BLK COARSE & FINE SAND, LITTLE FINES, LITTLE BRICK & STONE FRAGS (FILL)	A-3a (V)
B-003-0-23	SS-2	3.5	11											BLACK CINDERS, SOME SAND, LITTLE FINES (FILL)	A-1-a (V)
B-003-0-23	SS-3	6.0	24											BR. AND DARK BR. SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS (FILL)	A-4a (V)
B-003-0-23	SS-4	8.5	28	30	21	9		18	15	18	22	49	27	DK BR. SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS (FILL)	A-4a (3)
B-003-0-23	ST-5	10.0	22											DK BR. SANDY SILT, SOME CLAY, LITTLE BRICK AND STONE FRAGMENTS (FILL)	A-4a (V)
B-003-0-23	SS-6	13.5	42											DK BR. SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-003-0-23	SS-7	16.0	39	38	26	12		7	8	9	45	76	31	DK BR. SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (9)
B-003-0-23	SS-8	18.5	34											BLACK SILT AND CLAY, LITTLE SAND, LITTLE STONE FRAGMENTS (FILL)	A-6a (V)
B-003-0-23	SS-9	21.0	18											BROWN CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-7-6 (V)
B-003-0-23	SS-10	23.5	16	44	24	20		1	8	4	29	87	58	BROWN CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-7-6 (13)
B-003-0-23	SS-11A	26.0	13											BROWN CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-7-6 (V)
B-003-0-23	SS-11B	27.0												GRAY HIGHLY WEATHERED SHALE	Rock (V)
B-004-0-23	SS-1	1.0	5											BROWN GRAVEL & STONE FRAGMENTS WITH SAND, TRACE FINES (FILL)	A-1-b (V)
B-004-0-23	SS-2	3.5	7	NP	NP	NP		21	44	25	3	10	7	BROWN GRAVEL & STONE FRAGMENTS WITH SAND, TRACE FINES (FILL)	A-1-b (0)
B-004-0-23	SS-3	6.0	13											BROWN COARSE & FINE SAND, LITTLE FINES, LITTLE STONE FRAGS (FILL)	A-3a (V)
B-004-0-23	SS-4	8.5	13											BROWN SILTY CLAY, TRACE SHALE FRAGMENTS, TRACE SAND	A-6b (V)
B-004-0-23	SS-5	11.0	17	39	22	17		8	5	2	31	85	53	BROWN SILTY CLAY, TRACE SHALE FRAGMENTS, TRACE SAND	A-6b (11)
B-004-0-23	SS-6A	13.5	12											BROWN SILTY CLAY, "AND" SHALE FRAGMENTS, TRACE SAND	A-6b (V)
B-004-0-23	SS-6B	14.5												GRAY HIGHLY WEATHERED SHALE	Rock (V)
B-005-0-23	SS-1	1.5	18											BROWN AND GRAY SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGS (FILL)	A-6a (V)
B-005-0-23	SS-2	3.5	19											BROWN CLAY, TRACE STONE FRAGMENTS, TRACE SAND	A-7-6 (V)

Pro Geotech, Inc.

TR.-TRACE, BR.-BROWN, LI.-LITTLE, S/F-STONE FRAGMENTS, SO.-SOME, RB-ROADBASE, NP-NON-PLASTIC, POSS-POSSIBLE

Summary of Laboratory Results

Client: ARCADIS U.S., INC
Project: HAM-71-1.80 & HAM-22-10.93
Location: HAMILTON COUNTY, OHIO
Pro. Number: G23006G

Boring Number	Sample Number	Depth (ft)	Water Content %	Liquid Limit %	Plastic Limit %	Plast. Index	Specific Gravity	Agg. %	Coarse Sand %	Fine Sand %	Silt %	Silt&Clay Comb. %	Clay %	Soil Description	Class. Symbol
B-005-0-23	SS-3	6.0	19	43	23	20		7	2	3	31	88	57	BROWN CLAY, TRACE STONE FRAGMENTS, TRACE SAND	A-7-6 (13)
B-005-0-23	SS-4	8.5	16											BROWN CLAY, TRACE STONE FRAGMENTS, TRACE SAND	A-7-6 (V)
B-005-0-23	SS-5	11.0	18	43	23	20		1	1	1	35	96	62	BROWN CLAY, TRACE SAND, TRACE STONE FRAGMENTS	A-7-6 (13)
B-005-0-23	SS-6	13.5	9											BROWN SEVERELY WEATHERED SHALE	Rock (V)
B-005-0-23	SS-6	16.0												GRAY HIGHLY WEATHERED SHALE	Rock (V)
B-006-0-23	SS-1	1.0	13											GRAY SEVERELY WEATHERED SHALE	Rock (V)
B-006-0-23	SS-2	3.5	10											GRAY SEVERELY WEATHERED SHALE	Rock (V)
B-006-0-23	SS-3	6.0	9											GRAY HIGHLY WEATHERED SHALE	Rock (V)

Pro Geotech, Inc.

TR.-TRACE, BR.-BROWN, LI.-LITTLE, S/F-STONE FRAGMENTS, SO.-SOME, RB-ROADBASE, NP-NON-PLASTIC, POSS-POSSIBLE

Summary of Laboratory Results

Client: ARCADIS U.S., INC
Project: HAM-71-1.80 & HAM-22-10.93
Location: HAMILTON COUNTY, OHIO
Pro. Number: G23006G

ROCK MASS RATING From Table 10.4.6.4-1

Project: HAM-71-1.80 Bridge Replacement **Project No.:** G23006G

Structure: Pedestrian Bridge over I-71, I-471, and Gilbert Ave

Boring No.: B-001-0-23 (NQ2-1, 2, & 3 - 50.0' to 60.0') **Substru. Unit:** Ramp Columns

Strength of Intact Rock Material	
Uniaxial Compressive Strength	122 ksf
Relative Rating	0
Drill Core Quality RQD	
RQD	72%
Relative Rating	12
Joint Conditions	
Spacing of Joints	2.0" - >1.0'
Relative Rating	11
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and soft Joint Wall
Relative Rating	12
Ground water Conditions	
Relative Rating	7
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	42
Class No	III
Description	Fair Rock

Boring No.: B-002-0-23 (NQ2-1 & 2 - 40.0' to 50.0') **Substru. Unit:** Ramp Columns

Strength of Intact Rock Material	
Uniaxial Compressive Strength	72 ksf
Relative Rating	0
Drill Core Quality RQD	
RQD	55%
Relative Rating	9
Joint Conditions	
Spacing of Joints	2" to 1'
Relative Rating	9
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and soft Joint Wall
Relative Rating	12
Ground water Conditions	
Relative Rating	7
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	37
Class No	IV
Description	Poor Rock

ROCK MASS RATING From Table 10.4.6.4-1

Project: HAM-71-1.80 Bridge Replacement **Project No.:** G23006G

Structure: Pedestrian Bridge over I-71, I-471, and Gilbert Ave

Boring No.: B-003-0-23 (NQ2-1, 2, & 3 - 27.5' to 37.5') **Substru. Unit:** Ramp Columns

Strength of Intact Rock Material

Uniaxial Compressive Strength	130 ksf
Relative Rating	1

Drill Core Quality RQD

RQD	59%
Relative Rating	10

Joint Conditions

Spacing of Joints	2.0" - > 1.0'
Relative Rating	12
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and soft Joint Wall
Relative Rating	12

Ground water Conditions

Relative Rating	7

Strike & Dip Orientation of Joint

Relative Rating	0
Total Mass Rating	42
Class No	III
Description	Fair Rock

Boring No.: B-004-0-23 (NQ2-1, 2, & 3 - 14.7' to 25.5') **Substru. Unit:** Ramp Columns & Pier 1

Strength of Intact Rock Material

Uniaxial Compressive Strength	130 ksf
Relative Rating	1

Drill Core Quality RQD

RQD	56%
Relative Rating	10

Joint Conditions

Spacing of Joints	2" to >1'
Relative Rating	12
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and soft Joint Wall
Relative Rating	12

Ground water Conditions

Relative Rating	7

Strike & Dip Orientation of Joint

Relative Rating	0
Total Mass Rating	42
Class No	III
Description	Fair Rock

ROCK MASS RATING From Table 10.4.6.4-1	
Project: HAM-71-1.80 Bridge Replacement	**Project No.:** G23006G
Structure: Pedestrian Bridge over I-71, I-471, and Gilbert Ave	
Boring No.: B-005-0-23 (NQ2-1, 2, & 3 - 17.0' to 27.7')	**Substru. Unit:** Pier 2
Strength of Intact Rock Material	
Uniaxial Compressive Strength	58 ksf
Relative Rating	0
Drill Core Quality RQD	
RQD	71%
Relative Rating	12
Joint Conditions	
Spacing of Joints	2.0" - 1.0'
Relative Rating	9
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and soft Joint Wall
Relative Rating	12
Ground water Conditions	
Relative Rating	7
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	40
Class No	IV
Description	Poor Rock
Boring No.: B-006-0-23 (NQ2-1, 2, & 3 - 7.0' to 22.0')	**Substru. Unit:** Pier 3
Strength of Intact Rock Material	
Uniaxial Compressive Strength	72 ksf
Relative Rating	0
Drill Core Quality RQD	
RQD	54%
Relative Rating	8
Joint Conditions	
Spacing of Joints	2" to 1'
Relative Rating	7
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and soft Joint Wall
Relative Rating	9
Ground water Conditions	
Relative Rating	7
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	31
Class No	IV
Description	Poor Rock

HAM-71-1.81 PID 102790 Bedrock Compressive Strength

Boring	Drilled Shafts	Qu 0-5' (psi)	Qu 5-10' (psi)	PL 0-5' (psi)	PL 5-10' (psi)	Top Elev. (ft)	Bot. Elev (ft)
B-001-0-23	C4/C3/C9	356	872	1539	263	484.9	474.9
B-002-0-23	C2/C10/C13	353	220	260	867	495.1	485.1
B-003-0-23	C5/C6/C14	182	958	635	1148	496.6	486.6
B-004-0-23	C7/C8	1030	374	704	185	508.9	498.9
B-007-3-64	C1/C11/C12/C15					502.6	497.6
B-004-0-23	Pier 1	1030	374	704	185	508.9	498.9
B-005-0-23	Pier 2		439/359	73	240	535.0	525.0
		Qu 0-5' (psi)	Qu 5-10' (psi)	Qu 10-15' (psi)	Qu 15-20' (psi)		
B-006-0-23	Pier 3	31/64	187	261	350	552.8	542.8

Boring	Fx of Qu		AVG of Qu and PL		MIN of Fx and AVG		Selected Value		Rock Socket Side	Rock Socket Tip
	Qu 0-5' (psi)	Qu 5-10' (psi)	Qu 0-5' (psi)	Qu 5-10' (psi)	Qu 0-5' (psi)	Qu 5-10' (psi)	Qu 0-5' (psi)	Qu 5-10' (psi)	Qu 0-5' (psi)	Qu 5-10' (psi)
B-001-0-23	688.627	734.1146	947.5	567.5	688.627	567.5	675	565	350	550
B-002-0-23	599.3362	642.5186	306.5	543.5	306.5	543.5	305	545	300	550
B-003-0-23	586.6018	629.4452	408.5	1053	408.5	629.4452	405	630	400	550
B-004-0-23	486.0161	526.0797	867	279.5	486.0161	279.5	485	280	485	250
B-007-3-64	536.845	578.3369	867	279.5	536.845	279.5	485	280	485	250
B-004-0-23	486.0161	526.0797	867	279.5	486.0161	279.5	485	280	485	250
B-005-0-23	295.2287	329.3937	256	239.5	256	239.5	256	240	250	250
B-006-0-23	182.7733	212.9155	224		182.7733	212.9155	180	200	180	200

	Qu (psi)	Elev. (ft)	PL (psi)	Elev. (ft)	Side Qu 0-5' (psi)	Tip Qu 5-10' (psi)
B-001-0-23	356	481.6	1539	482.4	350	550
	872	475.9	263	477.4		
B-002-0-23	353	492.4	260	492.6	300	550
	220	488.4	867	487.6		
B-003-0-23	182	495.6	635	494.1	400	550
	958	491.1	1148	489.1		
B-004-0-23	1030	504.9	704	506.4	485	250
	374	501.6	185	501.4		
B-005-0-23	439	530.3	73	532.5	250	250
	359	527.8	240	527.5		
B-006-0-23	187	556.3	33	550.3	180	200
	261	550.1	350	545.3		

Average	465.9167	524.75
Std Dev	293.1929	446.2918
Agv - Std Dev	172.7238	78.45822

Sort Elev High to Low

Qu/PL (psi)	Elev. (ft)
187	556.3
33	550.3
261	550.1
350	545.3
73	532.5
439	530.3
359	527.8
240	527.5
704	506.4
1030	504.9
374	501.6
185	501.4
182	495.6
635	494.1
260	492.6
353	492.4
958	491.1
1148	489.1
220	488.4
867	487.6
1539	482.4
356	481.6
263	477.4
872	475.9

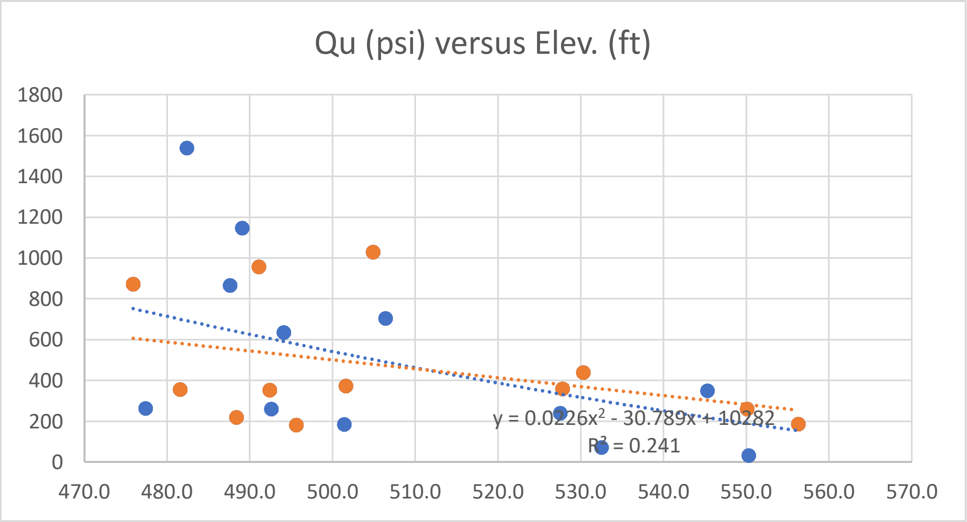
Sort Qu Low to High

Qu/PL (psi)	Elev. (ft)
33	550.3
73	532.5
182	495.6
185	501.4
187	556.3
220	488.4
240	527.5
260	492.6
261	550.1
263	477.4
350	545.3
353	492.4
356	481.6
359	527.8
374	501.6
439	530.3
635	494.1
704	506.4
867	487.6
872	475.9
958	491.1
1030	504.9
1148	489.1
1539	482.4

Average	495.3	495.3
Std Dev	378.7275	378.7275
Agv - Std Dev	116.6058	116.6058

Function Fx of Qu

y= 0.0226x^2-30.789x+10282



HAM-71-1.81 PID 102790 Drilled Shaft Calculation Check

Values from "HAM-71-1.81 PID 102790 Bedrock Compressive Strength.xlsx"

Factored Resistance with Load Transfer from Side to Base

φ_s 0.55 φ_p 0.50

Table 6.1.1 Excerpt		Qu	Qu	qs	qp	Qu side	Qu tip	Qu side	Qu tip	qs	qp	Factored Load	Socket L	Socket D	A_p	Rs	Rp	$\varphi_s R_s$	$\varphi_p R_p$	E_i	RQD	RQD side	RQD tip	E_iRQD	Transfer to Tip (%)	Rp	$\varphi_p R_p$	φ R
		(psi)	(ksf)	(ksf)	(ksf)	(psi)	(psi)	(ksf)	(ksf)	(ksf)	(ksf)	(kips)	(ft)	(ft)	(ft^2)	(kips)	(kips)	(kips)	(kips)	(psi)	(%)	(%)	(%)	(psi)		(kips)	(kips)	(kips)
B-001-0-23	C4/C3/C9	356	51.264	10.42495468	128.16	350	550	50.4	79.2	10.3	198	432	7.00	4.50	15.90431	731	3149	402	1575	31500	82%	82	92	25830	26.85%	268	134	536
B-002-0-23	C2/C10	220	31.68	8.195218118	79.2	300	550	43.2	79.2	9.57	198	379	7.00	4.50	15.90431	676	3149	372	1575	27000	47%	47	63	12690	26.85%	248	124	496
B-002-0-23	C13	220	31.68	8.195218118	79.2	300	550	43.2	79.2	9.57	198	141	5.50	3.50	9.621128	368	1905	203	952	27000	47%	47	63	12690	26.69%	134	67	270
B-003-0-23	C5/C6	958	137.952	17.10141047	344.88	400	550	57.6	79.2	11.1	198	477	7.00	4.50	15.90431	781	3149	430	1575	36000	67%	67.3	51.4	24228	26.85%	287	143	573
B-003-0-23	C14	958	137.952	17.10141047	344.88	400	550	57.6	79.2	11.1	198	141	5.50	3.50	9.621128	425	1905	234	952	36000	67%	67.3	51.4	24228	26.69%	155	77	311
B-004-0-23	C7/C8	374	53.856	10.68525713	134.64	485	250	69.84	36	12.2	90	489	7.00	4.50	15.90431	860	1431	473	716	43650	62%	62	50	27063	26.85%	316	158	631
B-007-3-64	C1/C11	250	36	8.736131867	90	485	250	69.84	36	12.2	90	349	7.00	4.50	15.90431	860	1431	473	716	43650	60%	60	50	26190	26.85%	316	158	631
B-007-3-64	C12/C15	250	36	8.736131867	90	485	250	69.84	36	12.2	90	141	5.50	3.50	9.621128	468	866	258	433	43650	60%	60	50	26190	26.69%	170	85	343
B-004-0-23	Pier 1	374	53.856	10.68525713	134.64	485	250	69.84	36	12.2	90	559	6.00	4.00	12.56637	612	1131	336.4	565	43650	62%	62	50	27063	27.44%	231	116	452
B-005-0-23	Pier 2	359	51.696	10.4687879	129.24	250	250	36	36	8.74	90	539	6.00	4.00	12.56637	439	1131	241.5	565	22500	75%	75.2	84	16920	27.44%	166	83	325
B-006-0-23	Pier 3					180	200																					

In accordance with GDM Section 1306.3.2, it can be seen that in this case, it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance.

HAM-71-1.81 PID 102790 Bedrock p-y Properties

Soil Boring Number	Bedrock Layer Number	Top Elev. (ft)	Bottom Elev. (ft)	Rock Type	RQD (%)	Joint Condition	LPILE p-y Model	Unconfined		Rock Modulus			Unit Weights	
								Q_u (psi)	k_{rm}	E_i (psi)	Ratio E_m/E_i	E_m (psi)	γ_{tot} (pcf)	γ' (eff.) (pcf)
B-001-0-23	1	485.4	484.9	Shale	10	`	Weak Rock	190	0.00050	17000	0.04	680	129	66.4
	2	484.9	479.9	Shale	82	Closed	Weak Rock	350	0.00050	32000	0.8	25600	154	91.8
	3	479.9	474.9	Shale	92	Closed	Weak Rock	550	0.00035	50000	0.9	45000	159	96.3
B-002-0-23	1	495.6	495.1	Shale	10	Open	Weak Rock	125	0.00050	11000	0.04	440	126	63.7
	2	495.1	490.1	Shale	47	Closed	Weak Rock	300	0.00050	27000	0.12	3240	139	76.8
	3	490.1	485.1	Shale	63	Closed	Weak Rock	550	0.00035	50000	0.5	25000	154	91.7
B-003-0-23	1	497.3	496.6	Shale	10	Open	Weak Rock	95	0.00050	8550	0.04	342	124	62.0
	2	496.6	491.6	Shale	67	Closed	Weak Rock	400	0.00049	36000	0.6	21600	153	90.7
	3	491.6	486.6	Shale	51	Closed	Weak Rock	550	0.00035	50000	0.15	7500	145	82.7
B-004-0-23	1	509.9	508.9	Shale	10	Open	Weak Rock	190	0.00050	17000	0.04	680	129	66.4
	2	508.9	503.9	Shale	62	Closed	Weak Rock	485	0.00040	44000	0.5	22000	153	90.7
	3	503.9	498.9	Shale	50	Closed	Weak Rock	250	0.00050	23000	0.15	3450	139	77.1
B-005-0-23	1	539	535	Shale	10	Open	Weak Rock	55	0.00050	4950	0.04	198	121	58.6
	2	535	530	Shale	75	Closed	Weak Rock	250	0.00050	23000	0.75	17250	151	88.8
	3	530	524.3	Shale	84	Closed	Weak Rock	250	0.00050	23000	0.8	18400	152	89.3
B-006-0-23	1	564.5	561	Shale	10	Open	Weak Rock	35	0.00050	3150	0.04	126	118	55.9
	2	561	557.8	Shale	10	Open	Weak Rock	64	0.00050	5760	0.04	230.4	122	59.6
	3	557.8	552.8	Shale	33	Closed	Weak Rock	180	0.00050	16000	0.09	1440	134	71.4
	4	552.8	547.8	Shale	69	Closed	Weak Rock	200	0.00050	18000	0.6	10800	148	85.4
	5	547.8	542.8	Shale	60	Closed	Weak Rock	250	0.00050	23000	0.5	11500	148	85.7
B-007-3-64	1	506.6	502.6	Shale	10	Open	Weak Rock	140	0.00050	13000	0.04	520	127	64.4
	2	502.6	497.6	Shale	60	Closed	Weak Rock	485	0.00040	44000	0.5	22000	153	90.7
	3	497.6	492.6	Shale	50	Closed	Weak Rock	250	0.00050	23000	0.15	3450	139	77.1

Q_u per GDM Section 404.3 for weak, augered bedrock

blows	N	ER	N_{90}	Q_u (ksf)	Q_u (psi)	Average
50/2"	300	90	300	27.60	191.7	
50/3"	200	90	200	18.40	127.8	
50/4"	150	90	150	13.80	95.8	
50/2"	300	90	300	27.60	191.7	
36/50	86	90	86	7.91	54.9	
22/6"	44	90	44	4.05	28.1	34.80976
43/6"	86	90	86	7.91	54.9	
50/6"	100	90	100	9.20	63.9	
27/6"	54	60	36	3.31	23.0	139.3
100/2"	600	60	400	36.80	255.6	

Rock Modulus Ratio (α_E = Em/Ei)

after Table 10.4.6.5-1, AASHTO LRFD BDS 6th Ed. (2012)

RQD (%)	RQD (NUM)	E_m/E_i Closed	E_m/E_i Open
0	0	0.03	0.03
10	0.1	0.04	0.04
20	0.2	0.05	0.05
25	0.25	0.07	0.06
30	0.3	0.09	0.07
35	0.35	0.1	0.08
40	0.4	0.11	0.09
45	0.45	0.12	0.095
50	0.5	0.15	0.1
55	0.55	0.3	0.105
60	0.6	0.5	0.11
65	0.65	0.6	0.115
70	0.7	0.7	0.12
75	0.75	0.75	0.13
80	0.8	0.8	0.15
85	0.85	0.85	0.18
90	0.9	0.9	0.23
95	0.95	0.95	0.4
100	1	1	0.6

Bedrock Unit Weight

Rock Type	Maximum γ (pcf)	Q_u (psi)	Max. Q_u (psi)	Typ. Q_u (psi)
Claystone	175	500	1500	500
Dolomite	181	25000	40000	15000
Limestone	170	15000	40000	15000
Mudstone	175	500	1500	500
Sandstone	175	5000	10000	3500
Shale	175	3500	10000	2500
Siltstone	170	5000	10000	3500

Estimation of Drilled Shaft Resistence and Settlement in Jointed Rock (LRFD 10.8.3.5.4)

Project: HAM-71-1.80 **Project No.:** G23005G

Structure: Pedestrian Bridge over Gilbert Ave., I-471, and I-71

Boring No.: B-001-0-23 **Substructure Unit:** Columns C4/C3/C9

Unit Side Resistence (q_s): $C*P_a*Sqrt(q_u/P_a) <7.8*Pa*Sqrt(f'_c/P_a)$ (Eq. 10.8.3.5.4b-1)

Uniaxial Comp.Strength of Intact Rock, Q_u Side (ksf): **50.4** Atmospheric Pressure P_a(ksf): **2.12**

Regression Constant (C): **1.0** (For Normal Conditions) Concrete Compressive Strength f'_c(ksf): **576**

Unit Side Resistence, qs (ksf): 10.34 From Eq 10.8.3.5.4b-1 using Uniaxial Comp.Strength of Rock

Assumed Unit Side Resistence (ksf): **10.34**

Unit Tip Resistence (q_p): Depth of 2B below base is jointed and have random orientation (Eq. 10.8.3.5.4c-2)

GSI = mi = (T10.4.6.4-1 D=

s = exp[(GSI−100)/(9−3D)] a = {1/2+1/6[(exp((GSI)/(−15))−(exp((20)/(−3))]}

From Eq 10.4.64-2 From Eq 10.4.64-3

mb = mi exp[(GSI−100)/(28−14D)] From Eq 10.4.6.4-4

s= a= mb=

Vertical Effective Stress (ksf) at Tip = A=

Unit Tip Resistence, qp (ksf):

Assumed Unit Tip Resistence, qp (ksf):

Unit Tip Resistence (q_p): 2.5*qu (Eq. 10.8.3.5.4c-1) (Depth of 2B is either intact or tightly jointed)

Uniaxial Comp.Strength of Intact Rock, Q_u Tip (ksf): **79.2**

Unit Tip Resistence, q_p (ksf): 198 Assumed Unit Tip Resistence, qp (ksf): **198**

Calculation of Nominal Resistence of Side and Tip

Shaft Socket Diameter, Br (feet):	**3.00**	**3.50**	**4.00**	**4.50**
Length of Socket, Dr (feet) :	4.5	5.5	6.0	7.0
Perimeter Area of Socket As (Sq. ft)	23.56	38.48	50.27	70.69
Cross-Sectional Area of Socket, Ap (Sq. ft)	7.07	9.62	12.57	15.90
Nominal Shaft Side Resistence, Rs (kips):	243.6	397.9	519.7	730.9
Nominal Shaft Tip Resistence, Rp (kips):	1399.6	1905.0	2488.1	3149.1
Resistence Factor for Side from T. 10.5.5.2.4-1	0.55	0.55	0.55	0.55
Resistence Factor for Tip from T. 10.5.5.2.4-1	0.50	0.50	0.50	0.50
Factored Resistance from Side (kips)	134.0	218.9	285.9	402.0
Factored Resistance from Tip (kips)	699.8	952.5	1244.1	1574.5
Total Factored Resistance (kips)	**833.8**	**1171.4**	**1529.9**	**1976.5**

Butt settlement of drilled Shaft : Q((Dr/Ap*Ec)+(Ips/Br*Em))

Axial Load on Top of Socket, Q (kips) for 1.0" Settlement				
Concrete Young's Modulus, Ec (ksi)	3800	3800	3800	3800
Shortening of Drilled Shaft (Inches)	0.000	0.000	0.000	0.000
Rock Mass Modulus, Em (ksi)	100.0	100.0	100.0	100.0
Ec/Em	38.0	38.0	38.0	38.0
Dr/Br	1.50	1.57	1.50	1.56
Influence Coefficient (Ips) from Fig 4.6.5.5.2A				
(Modified after Pells and Turner (1979))				
Settlement of Base (inches)	0.000	0.000	0.000	0.000
Total Butt Settlement of Shaft (inches)	0.000	0.000	0.000	0.000

Estimation of Drilled Shaft Resistence and Settlement in Jointed Rock (LRFD 10.8.3.5.4)

Project: HAM-71-1.80 **Project No.:** G23005G
Structure: Pedestrian Bridge over Gilbert Ave., I-471, and I-71
Boring No.: B-002-0-23 **Substructure Unit:** Columns C2/C10/C13

Unit Side Resistence (q_s): $C*P_a*Sqrt(q_u/P_a) < 7.8*Pa*Sqrt(f'_c/P_a)$ (Eq. 10.8.3.5.4b-1)

Uniaxial Comp.Strength of Intact Rock, Q_u Side (ksf): **43.2** Atmospheric Pressure P_a(ksf): **2.12**

Regression Constant (C): **1.0** (For Normal Conditions) Concrete Compressive Strength f'_c(ksf): **576**

Unit Side Resistence, qs (ksf): 9.57 From Eq 10.8.3.5.4b-1 using Uniaxial Comp.Strength of Rock

Assumed Unit Side Resistence (ksf): **9.57**

Unit Tip Resistence (q_p): Depth of 2B below base is jointed and have random orientation (Eq. 10.8.3.5.4c-2)

GSI = mi = (T10.4.6.4-1 D=

s = exp[(GSI−100)/(9−3D)] a = {1/2+1/6[(exp((GSI)/(−15))−(exp((20)/(−3))]}
From Eq 10.4.64-2 From Eq 10.4.64-3

mb = mi exp[(GSI−100)/(28−14D)] From Eq 10.4.6.4-4

s= a= mb=

Vertical Effective Stress (ksf) at Tip = A=

Unit Tip Resistence, qp (ksf):

Assumed Unit Tip Resistence, qp (ksf):

Unit Tip Resistence (q_p): 2.5*qu (Eq. 10.8.3.5.4c-1) (Depth of 2B is either intact or tightly jointed)

Uniaxial Comp.Strength of Intact Rock, Q_u Tip (ksf): **79.2**

Unit Tip Resistence, q_p (ksf): 198 Assumed Unit Tip Resistence, qp (ksf): **198**

Calculation of Nominal Resistence of Side and Tip

Shaft Socket Diameter, Br (feet):	**3.00**	**3.50**	**4.00**	**4.50**
Length of Socket, Dr (feet) :	4.5	5.5	6.0	7.0
Perimeter Area of Socket As (Sq. ft)	23.56	38.48	50.27	70.69
Cross-Sectional Area of Socket, Ap (Sq. ft)	7.07	9.62	12.57	15.90
Nominal Shaft Side Resistence, Rs (kips):	225.5	368.3	481.0	676.5
Nominal Shaft Tip Resistence, Rp (kips):	1399.6	1905.0	2488.1	3149.1
Resistence Factor for Side from T. 10.5.5.2.4-1	0.55	0.55	0.55	0.55
Resistence Factor for Tip from T. 10.5.5.2.4-1	0.50	0.50	0.50	0.50
Factored Resistance from Side (kips)	124.0	202.6	264.6	372.1
Factored Resistance from Tip (kips)	699.8	952.5	1244.1	1574.5
Total Factored Resistance (kips)	**823.8**	**1155.1**	**1508.6**	**1946.6**

Butt settlement of drilled Shaft : Q((Dr/Ap*Ec)+(Ips/Br*Em))

Axial Load on Top of Socket, Q (kips) for 1.0" Settlement				
Concrete Young's Modulus, Ec (ksi)	3800	3800	3800	3800
Shortening of Drilled Shaft (Inches)	0.000	0.000	0.000	0.000
Rock Mass Modulus, Em (ksi)	100.0	100.0	100.0	100.0
Ec/Em	38.0	38.0	38.0	38.0
Dr/Br	1.50	1.57	1.50	1.56
Influence Coefficient (Ips) from Fig 4.6.5.5.2A				
(Modified after Pells and Turner (1979))				
Settlement of Base (inches)	0.000	0.000	0.000	0.000
Total Butt Settlement of Shaft (inches)	0.000	0.000	0.000	0.000

Estimation of Drilled Shaft Resistence and Settlement in Jointed Rock (LRFD 10.8.3.5.4)

Project: HAM-71-1.80 **Project No.:** G23005G
Structure: Pedestrian Bridge over Gilbert Ave., I-471, and I-71
Boring No.: B-003-0-23 **Substructure Unit:** Columns C5/C6/C14

Unit Side Resistence (q_s): $C*P_a*Sqrt(q_u/P_a) <7.8*Pa*Sqrt(f'_c/P_a)$ (Eq. 10.8.3.5.4b-1)

Uniaxial Comp.Strength of Intact Rock, Q_u Side (ksf): **57.6** Atmospheric Pressure P_a(ksf): **2.12**

Regression Constant (C): **1.0** (For Normal Conditions) Concrete Compressive Strength f'_c(ksf): **576**

Unit Side Resistence, qs (ksf): 11.05 From Eq 10.8.3.5.4b-1 using Uniaxial Comp.Strength of Rock

Assumed Unit Side Resistence (ksf): **11.1**

Unit Tip Resistence (q_p): Depth of 2B below base is jointed and have random orientation (Eq. 10.8.3.5.4c-2)

GSI = mi = (T10.4.6.4-1 D=

s = exp[(GSI−100)/(9−3D)] a = {1/2+1/6[(exp((GSI)/(−15))−(exp((20)/(−3))]}

From Eq 10.4.64-2 From Eq 10.4.64-3

mb = mi exp[(GSI−100)/(28−14D)] From Eq 10.4.6.4-4

s= a= mb=

Vertical Effective Stress (ksf) at Tip = A=

Unit Tip Resistence, qp (ksf):

Assumed Unit Tip Resistence, qp (ksf):

Unit Tip Resistence (q_p): 2.5*qu (Eq. 10.8.3.5.4c-1) (Depth of 2B is either intact or tightly jointed)

Uniaxial Comp.Strength of Intact Rock, Q_u Tip (ksf): **79.2**

Unit Tip Resistence, q_p (ksf): 198 Assumed Unit Tip Resistence, qp (ksf): **198**

Calculation of Nominal Resistence of Side and Tip

Shaft Socket Diameter, Br (feet):	**3.00**	**3.50**	**4.00**	**4.50**
Length of Socket, Dr (feet) :	4.5	5.5	6.0	7.0
Perimeter Area of Socket As (Sq. ft)	23.56	38.48	50.27	70.69
Cross-Sectional Area of Socket, Ap (Sq. ft)	7.07	9.62	12.57	15.90
Nominal Shaft Side Resistence, Rs (kips):	261.5	427.2	557.9	784.6
Nominal Shaft Tip Resistence, Rp (kips):	1399.6	1905.0	2488.1	3149.1
Resistence Factor for Side from T. 10.5.5.2.4-1	0.55	0.55	0.55	0.55
Resistence Factor for Tip from T. 10.5.5.2.4-1	0.50	0.50	0.50	0.50
Factored Resistance from Side (kips)	143.8	234.9	306.9	431.5
Factored Resistance from Tip (kips)	699.8	952.5	1244.1	1574.5
Total Factored Resistance (kips)	**843.6**	**1187.4**	**1550.9**	**2006.1**

Butt settlement of drilled Shaft : Q((Dr/Ap*Ec)+(Ips/Br*Em))

Axial Load on Top of Socket, Q (kips) for 1.0" Settlement				
Concrete Young's Modulus, Ec (ksi)	3800	3800	3800	3800
Shortening of Drilled Shaft (Inches)	0.000	0.000	0.000	0.000
Rock Mass Modulus, Em (ksi)	100.0	100.0	100.0	100.0
Ec/Em	38.0	38.0	38.0	38.0
Dr/Br	1.50	1.57	1.50	1.56
Influence Coefficient (Ips) from Fig 4.6.5.5.2A				
(Modified after Pells and Turner (1979))				
Settlement of Base (inches)	0.000	0.000	0.000	0.000
Total Butt Settlement of Shaft (inches)	0.000	0.000	0.000	0.000

Estimation of Drilled Shaft Resistence and Settlement in Jointed Rock (LRFD 10.8.3.5.4)

Project: HAM-71-1.80 **Project No.:** G23005G

Structure: Pedestrian Bridge over Gilbert Ave., I-471, and I-71

Boring No.: B-004-0-23 **Substructure Unit:** Columns C7/C8/Pier 1

Unit Side Resistence (q_s): $C*P_a*Sqrt(q_u/P_a) <7.8*Pa*Sqrt(f'_c/P_a)$ (Eq. 10.8.3.5.4b-1)

Uniaxial Comp.Strength of Intact Rock, Q_u Side (ksf): **69.8** Atmospheric Pressure P_a(ksf): **2.12**

Regression Constant (C): **1.0** (For Normal Conditions) Concrete Compressive Strength f'_c(ksf): **576**

Unit Side Resistence, qs (ksf): 12.16 From Eq 10.8.3.5.4b-1 using Uniaxial Comp.Strength of Rock

Assumed Unit Side Resistence (ksf): **12.2**

Unit Tip Resistence (q_p): Depth of 2B below base is jointed and have random orientation (Eq. 10.8.3.5.4c-2)

GSI = mi = (T10.4.6.4-1 D=

s = exp[(GSI−100)/(9−3D)] a = {1/2+1/6[(exp((GSI)/(−15))−(exp((20)/(−3))]}

From Eq 10.4.64-2 From Eq 10.4.64-3

mb = mi exp[(GSI−100)/(28−14D)] From Eq 10.4.6.4-4

s= a= mb=

Vertical Effective Stress (ksf) at Tip = A=

Unit Tip Resistence, qp (ksf):

Assumed Unit Tip Resistence, qp (ksf):

Unit Tip Resistence (q_p): 2.5*qu (Eq. 10.8.3.5.4c-1) (Depth of 2B is either intact or tightly jointed)

Uniaxial Comp.Strength of Intact Rock, Q_u Tip (ksf): **36.0**

Unit Tip Resistence, q_p (ksf): 90 Assumed Unit Tip Resistence, qp (ksf): **90**

Calculation of Nominal Resistence of Side and Tip

Shaft Socket Diameter, Br (feet):	**3.00**	**3.50**	**4.00**	**4.50**
Length of Socket, Dr (feet) :	4.5	5.5	6.0	7.0
Perimeter Area of Socket As (Sq. ft)	23.56	38.48	50.27	70.69
Cross-Sectional Area of Socket, Ap (Sq. ft)	7.07	9.62	12.57	15.90
Nominal Shaft Side Resistence, Rs (kips):	287.5	469.5	613.2	862.4
Nominal Shaft Tip Resistence, Rp (kips):	636.2	865.9	1131.0	1431.4
Resistence Factor for Side from T. 10.5.5.2.4-1	0.55	0.55	0.55	0.55
Resistence Factor for Tip from T. 10.5.5.2.4-1	0.50	0.50	0.50	0.50
Factored Resistance from Side (kips)	158.1	258.2	337.3	474.3
Factored Resistance from Tip (kips)	318.1	433.0	565.5	715.7
Total Factored Resistance (kips)	**476.2**	**691.2**	**902.8**	**1190.0**

Butt settlement of drilled Shaft : Q((Dr/Ap*Ec)+(Ips/Br*Em))

Axial Load on Top of Socket, Q (kips) for 1.0" Settlement				
Concrete Young's Modulus, Ec (ksi)	3800	3800	3800	3800
Shortening of Drilled Shaft (Inches)	0.000	0.000	0.000	0.000
Rock Mass Modulus, Em (ksi)	100.0	100.0	100.0	100.0
Ec/Em	38.0	38.0	38.0	38.0
Dr/Br	1.50	1.57	1.50	1.56
Influence Coefficient (Ips) from Fig 4.6.5.5.2A				
(Modified after Pells and Turner (1979))				
Settlement of Base (inches)	0.000	0.000	0.000	0.000
Total Butt Settlement of Shaft (inches)	0.000	0.000	0.000	0.000

Estimation of Drilled Shaft Resistence and Settlement in Jointed Rock (LRFD 10.8.3.5.4)

Project: HAM-71-1.80 **Project No.:** G23005G

Structure: Pedestrian Bridge over Gilbert Ave., I-471, and I-71

Boring No.: B-007-3-64 **Substructure Unit:** Columns C1/C11/C12/C15

Unit Side Resistence (q_s): C*P_a*Sqrt(q_u/P_a) <7.8*Pa*Sqrt(f'_c/P_a) (Eq. 10.8.3.5.4b-1)

Uniaxial Comp.Strength of Intact Rock, Q_u Side (ksf): **69.8** Atmospheric Pressure P_a(ksf): **2.12**

Regression Constant (C): **1.0** (For Normal Conditions) Concrete Compressive Strength f'_c(ksf): **576**

Unit Side Resistence, qs (ksf): 12.16 From Eq 10.8.3.5.4b-1 using Uniaxial Comp.Strength of Rock

Assumed Unit Side Resistence (ksf): **12.2**

Unit Tip Resistence (q_p): Depth of 2B below base is jointed and have random orientation (Eq. 10.8.3.5.4c-2)

GSI = mi = (T10.4.6.4-1 D=

s = exp[(GSI−100)/(9−3D)] a = {1/2+1/6[(exp((GSI)/(−15))−(exp((20)/(−3))]}

From Eq 10.4.64-2 From Eq 10.4.64-3

mb = mi exp[(GSI−100)/(28−14D)] From Eq 10.4.6.4-4

s= a= mb=

Vertical Effective Stress (ksf) at Tip = A=

Unit Tip Resistence, qp (ksf):

Assumed Unit Tip Resistence, qp (ksf):

Unit Tip Resistence (q_p): 2.5*qu (Eq. 10.8.3.5.4c-1) (Depth of 2B is either intact or tightly jointed)

Uniaxial Comp.Strength of Intact Rock, Q_u Tip (ksf): **36.0**

Unit Tip Resistence, q_p (ksf): 90 Assumed Unit Tip Resistence, qp (ksf): **90**

Calculation of Nominal Resistence of Side and Tip

Shaft Socket Diameter, Br (feet):	**3.00**	**3.50**	**4.00**	**4.50**
Length of Socket, Dr (feet) :	4.5	5.5	6.0	7.0
Perimeter Area of Socket As (Sq. ft)	23.56	38.48	50.27	70.69
Cross-Sectional Area of Socket, Ap (Sq. ft)	7.07	9.62	12.57	15.90
Nominal Shaft Side Resistence, Rs (kips):	287.5	469.5	613.2	862.4
Nominal Shaft Tip Resistence, Rp (kips):	636.2	865.9	1131.0	1431.4
Resistence Factor for Side from T. 10.5.5.2.4-1	0.55	0.55	0.55	0.55
Resistence Factor for Tip from T. 10.5.5.2.4-1	0.50	0.50	0.50	0.50
Factored Resistance from Side (kips)	158.1	258.2	337.3	474.3
Factored Resistance from Tip (kips)	318.1	433.0	565.5	715.7
Total Factored Resistance (kips)	**476.2**	**691.2**	**902.8**	**1190.0**

Butt settlement of drilled Shaft : Q((Dr/Ap*Ec)+(Ips/Br*Em))

Axial Load on Top of Socket, Q (kips) for 1.0" Settlement				
Concrete Young's Modulus, Ec (ksi)	3800	3800	3800	3800
Shortening of Drilled Shaft (Inches)	0.000	0.000	0.000	0.000
Rock Mass Modulus, Em (ksi)	100.0	100.0	100.0	100.0
Ec/Em	38.0	38.0	38.0	38.0
Dr/Br	1.50	1.57	1.50	1.56
Influence Coefficient (Ips) from Fig 4.6.5.5.2A (Modified after Pells and Turner (1979))				
Settlement of Base (inches)	0.000	0.000	0.000	0.000
Total Butt Settlement of Shaft (inches)	0.000	0.000	0.000	0.000

Estimation of Drilled Shaft Resistence and Settlement in Jointed Rock (LRFD 10.8.3.5.4)

Project: HAM-71-1.80 **Project No.:** G23005G
Structure: Pedestrian Bridge over Gilbert Ave., I-471, and I-71
Boring No.: B-005-0-23 **Substructure Unit:** Pier 2

Unit Side Resistence (q_s): $C*P_a*Sqrt(q_u/P_a) <7.8*Pa*Sqrt(f'_c/P_a)$ (Eq. 10.8.3.5.4b-1)

Uniaxial Comp.Strength of Intact Rock, Q_u Side (ksf): **36** Atmospheric Pressure P_a(ksf): **2.12**

Regression Constant (C): **1.0** (For Normal Conditions) Concrete Compressive Strength f'_c(ksf): **576**

Unit Side Resistence, qs (ksf): 8.74 From Eq 10.8.3.5.4b-1 using Uniaxial Comp.Strength of Rock

Assumed Unit Side Resistence (ksf): **8.7**

Unit Tip Resistence (q_p): Depth of 2B below base is jointed and have random orientation (Eq. 10.8.3.5.4c-2)

GSI = mi = (T10.4.6.4-1 D=

s = exp[(GSI−100)/(9−3D)] a = {1/2+1/6[(exp((GSI)/(−15))−(exp((20)/(−3))]}

From Eq 10.4.64-2 From Eq 10.4.64-3

mb = mi exp[(GSI−100)/(28−14D)] From Eq 10.4.6.4-4

s= a= mb=

Vertical Effective Stress (ksf) at Tip = A=

Unit Tip Resistence, qp (ksf):

Assumed Unit Tip Resistence, qp (ksf):

Unit Tip Resistence (q_p): 2.5*qu (Eq. 10.8.3.5.4c-1) (Depth of 2B is either intact or tightly jointed)

Uniaxial Comp.Strength of Intact Rock, Q_u Tip (ksf): **36.0**

Unit Tip Resistence, q_p (ksf): 90 Assumed Unit Tip Resistence, qp (ksf): **90**

Calculation of Nominal Resistence of Side and Tip

Shaft Socket Diameter, Br (feet):	**3.00**	**3.50**	**4.00**	**4.50**
Length of Socket, Dr (feet) :	4.5	5.25	6.0	6.8
Perimeter Area of Socket As (Sq. ft)	23.56	35.74	50.27	67.15
Cross-Sectional Area of Socket, Ap (Sq. ft)	7.07	9.62	12.57	15.90
Nominal Shaft Side Resistence, Rs (kips):	205.9	312.3	439.3	586.9
Nominal Shaft Tip Resistence, Rp (kips):	636.2	865.9	1131.0	1431.4
Resistence Factor for Side from T. 10.5.5.2.4-1	0.55	0.55	0.55	0.55
Resistence Factor for Tip from T. 10.5.5.2.4-1	0.50	0.50	0.50	0.50
Factored Resistance from Side (kips)	113.3	171.8	241.6	322.8
Factored Resistance from Tip (kips)	318.1	433.0	565.5	715.7
Total Factored Resistance (kips)	**431.3**	**604.7**	**807.1**	**1038.5**

Butt settlement of drilled Shaft : Q((Dr/Ap*Ec)+(Ips/Br*Em))

Axial Load on Top of Socket, Q (kips) for 1.0" Settlement				
Concrete Young's Modulus, Ec (ksi)	3800	3800	3800	3800
Shortening of Drilled Shaft (Inches)	0.000	0.000	0.000	0.000
Rock Mass Modulus, Em (ksi)	100.0	100.0	100.0	100.0
Ec/Em	38.0	38.0	38.0	38.0
Dr/Br	1.50	1.50	1.50	1.50
Influence Coefficient (Ips) from Fig 4.6.5.5.2A				
(Modified after Pells and Turner (1979))				
Settlement of Base (inches)	0.000	0.000	0.000	0.000
Total Butt Settlement of Shaft (inches)	0.000	0.000	0.000	0.000

Load Transfer of Factored Resistance from Side to Base in Drilled Shaft
As per GDM Section 1306.3.2

Sub-Structure: C4/C3/C9

Boring: B-001-0-23

Rock socket diameter D = 4.5 feet

Rock socket length is L = 7.0 feet

E_i = 31,500 psi and RQD = 82%

$Q_b/Q_t = 35(L/D^{)-0.6}$, where $E_i \times$ RQD < 50,000 psi

$Q_b/Q_t = 30(L/D^{)-0.8}$, where 50,000 psi ≤ $E_i \times$ RQD < 500,000 psi

$Q_b/Q_t = 25(L/D)^{-1.1}$, where $E_i \times$ RQD ≥ 500,000 psi

$E_i \times$ RQD = 31,500 psi × 82% = 25,830 psi <50,000 psi; therefore,

Select $Q_b/Q_t = 35(L/D^{)-0.6} = 35(7.0/4.5)^{-0.6}$ = 26.85%

The load transfer to the drilled shaft tip = 26.85%

The load transfer to the side = 100% - 26.85% = 73.15%

Total calculated nominal side resistance = 731 kips

Total calculated nominal tip resistance = 3149 kips.

Considering load transfer to base, the nominal tip resistance = 26.85/73.15 × 731 kips = 268 kips;

Total combined factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S$ = 0.50 × 268 kips + 0.55 × 731 kips = 536 kips.

Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. This makes tip factored resistance $R_R = \varphi_{qp}R_P$ = 0.50 × 3149 kips = 1575 kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

Load Transfer of Factored Resistance from Side to Base in Drilled Shaft
As per GDM Section 1306.3.2

Sub-Structure: C2/C10

Boring: B-002-0-23

Rock socket diameter D = 4.5 feet

Rock socket length is L = 7.0 feet

E_i = 27,000 psi and RQD = 47%

$Q_b/Q_t = 35(L/D^{)-0.6}$, where $E_i \times$ RQD < 50,000 psi

$Q_b/Q_t = 30(L/D^{)-0.8}$, where 50,000 psi $\leq E_i \times$ RQD < 500,000 psi

$Q_b/Q_t = 25(L/D)^{-1.1}$, where $E_i \times$ RQD $\geq$ 500,000 psi

$E_i \times$ RQD = 27,000 psi × 47% = 12,690 psi <50,000 psi; therefore,

Select $Q_b/Q_t = 35(L/D^{)-0.6} = 35(7.0/4.5)^{-0.6} = 26.85\%$

The load transfer to the drilled shaft tip = 26.85%

The load transfer to the side = 100% - 26.85% = 73.15%

Total calculated nominal side resistance = 676 kips

Total calculated nominal tip resistance = 3149 kips.

Considering load transfer to base, the nominal tip resistance = 26.85/73.15 × 676 kips = 248 kips;

Total factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S = 0.50 \times 248$ kips $+ 0.55 \times 676$ kips = 496 kips.

Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. This makes tip factored resistance $R_R = \varphi_{qp}R_P = 0.50 \times 3149$ kips = 1574 kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

Load Transfer of Factored Resistance from Side to Base in Drilled Shaft
As per GDM Section 1306.3.2

Sub-Structure: C13

Boring: B-002-0-23

Rock socket diameter D = 3.5 feet

Rock socket length is L = 5.5 feet

E_i = 27,000 psi and RQD = 47%

$Q_b/Q_t = 35(L/D^{)-0.6}$, where $E_i \times RQD < 50{,}000$ psi

$Q_b/Q_t = 30(L/D^{)-0.8}$, where $50{,}000 \text{ psi} \leq E_i \times RQD < 500{,}000$ psi

$Q_b/Q_t = 25(L/D)^{-1.1}$, where $E_i \times RQD \geq 500{,}000$ psi

$E_i \times RQD = 27{,}000 \text{ psi} \times 47\% = 12{,}690 \text{ psi} < 50{,}000$ psi; therefore,

Select $Q_b/Q_t = 35(L/D^{)-0.6} = 35(5.5/3.5)^{-0.6} = 26.69\%$

The load transfer to the drilled shaft tip = 26.69%

The load transfer to the side = 100% - 26.69% = 73.31%

Total calculated nominal side resistance = 368 kips

Total calculated nominal tip resistance = 1905 kips.

Considering load transfer to base, the nominal tip resistance = 26.69/73.31 × 368 kips = 134 kips;

Total factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S = 0.50 \times 134 \text{ kips} + 0.55 \times 368 \text{ kips} = 270$ kips.

Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. This makes tip factored resistance $R_R = \varphi_{qp}R_P = 0.50 \times 1905$ kips = 953 kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

Load Transfer of Factored Resistance from Side to Base in Drilled Shaft
As per GDM Section 1306.3.2

Sub-Structure: C5/C6

Boring: B-003-0-23

Rock socket diameter D = 4.5 feet

Rock socket length is L = 7.0 feet

E_i = 27,000 psi and RQD = 47%

$Q_b/Q_t = 35(L/D^{)-0.6}$, where $E_i \times$ RQD < 50,000 psi

$Q_b/Q_t = 30(L/D^{)-0.8}$, where 50,000 psi ≤ $E_i \times$ RQD < 500,000 psi

$Q_b/Q_t = 25(L/D)^{-1.1}$, where $E_i \times$ RQD ≥ 500,000 psi

$E_i \times$ RQD = 36,000 psi × 67.3% = 24,228 psi <50,000 psi; therefore,

Select $Q_b/Q_t = 35(L/D^{)-0.6} = 35(7.0/4.5)^{-0.6}$ = 26.85%

The load transfer to the drilled shaft tip = 26.85%

The load transfer to the side = 100% - 26.85% = 73.15%

Total calculated nominal side resistance = 785 kips

Total calculated nominal tip resistance = 3149 kips.

Considering load transfer to base, the nominal tip resistance = 26.85/73.15 × 785 kips = 287 kips;

Total factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S$ = 0.50 × 287 kips + 0.55 × 785 kips = 573 kips.

Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. This makes tip factored resistance $R_R = \varphi_{qp}R_P$ = 0.50 × 3149 kips = 1575 kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

Load Transfer of Factored Resistance from Side to Base in Drilled Shaft
As per GDM Section 1306.3.2

Sub-Structure: C14

Boring: B-003-0-23

Rock socket diameter D = 3.5 feet

Rock socket length is L = 5.5 feet

E_i = 27,000 psi and RQD = 47%

$Q_b/Q_t = 35(L/D^{)-0.6}$, where $E_i \times RQD < 50,000$ psi

$Q_b/Q_t = 30(L/D^{)-0.8}$, where $50,000 \text{ psi} \leq E_i \times RQD < 500,000$ psi

$Q_b/Q_t = 25(L/D)^{-1.1}$, where $E_i \times RQD \geq 500,000$ psi

$E_i \times RQD$ = 36,000 psi × 67.3% = 24,228 psi <50,000 psi; therefore,

Select $Q_b/Q_t = 35(L/D^{)-0.6} = 35(5.5/3.5)^{-0.6} = 26.69\%$

The load transfer to the drilled shaft tip = 26.69%

The load transfer to the side = 100% - 26.69% = 73.31%

Total calculated nominal side resistance = 427 kips

Total calculated nominal tip resistance = 1905 kips.

Considering load transfer to base, the nominal tip resistance = 26.69/73.31 × 427 kips = 155 kips;

Total factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S$ = 0.50 ×155 kips + 0.55 × 427 kips = 311 kips.

Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. This makes tip factored resistance $R_R = \varphi_{qp}R_P$ = 0.50 × 1905 kips = 952 kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

Load Transfer of Factored Resistance from Side to Base in Drilled Shaft
As per GDM Section 1306.3.2

Sub-Structure: C7/C8

Boring: B-004-0-23

Rock socket diameter D = 4.5 feet

Rock socket length is L = 7.5 feet

E_i = 43,650 psi and RQD = 62%

$Q_b/Q_t = 35(L/D^{)-0.6}$, where $E_i \times RQD < 50{,}000$ psi

$Q_b/Q_t = 30(L/D^{)-0.8}$, where $50{,}000 \text{ psi} \leq E_i \times RQD < 500{,}000$ psi

$Q_b/Q_t = 25(L/D)^{-1.1}$, where $E_i \times RQD \geq 500{,}000$ psi

$E_i \times RQD = 43{,}650 \text{ psi} \times 62.0\% = 27{,}063 \text{ psi} <50{,}000$ psi; therefore,

Select $Q_b/Q_t = 35(L/D^{)-0.6} = 35(7.5/4.5)^{-0.6} = 26.85\%$

The load transfer to the drilled shaft tip = 26.85%

The load transfer to the side = 100% - 26.85% = 73.15%

Total calculated nominal side resistance = 862 kips

Total calculated nominal tip resistance = 1431 kips.

Considering load transfer to base, the nominal tip resistance = 26.85/73.15 × 862 kips = 316 kips

Total factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S = 0.50 \times 316 \text{ kips} + 0.55 \times 862 \text{ kips} = 631$ kips.

Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. This makes tip factored resistance $R_R = \varphi_{qp}R_P = 0.50 \times 1431 \text{ kips} = 716$ kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

Load Transfer of Factored Resistance from Side to Base in Drilled Shaft
As per GDM Section 1306.3.2

Sub-Structure: C1/C11

Boring: B-007-3-64

Rock socket diameter D = 4.5 feet

Rock socket length is L = 7.00 feet

$E_i = 43{,}650$ psi and RQD = 60%

$Q_b/Q_t = 35(L/D^{)-0.6}$, where $E_i \times \text{RQD} < 50{,}000$ psi

$Q_b/Q_t = 30(L/D^{)-0.8}$, where $50{,}000 \text{ psi} \leq E_i \times \text{RQD} < 500{,}000$ psi

$Q_b/Q_t = 25(L/D)^{-1.1}$, where $E_i \times \text{RQD} \geq 500{,}000$ psi

$E_i \times \text{RQD} = 43{,}650 \text{ psi} \times 60.0\% = 26{,}090 \text{ psi} <50{,}000$ psi; therefore,

Select $Q_b/Q_t = 35(L/D^{)-0.6} = 35(7.00/4.5)^{-0.6} = 26.85\%$

The load transfer to the drilled shaft tip = 26.85%

The load transfer to the side = 100% - 26.85% = 73.15%

Total calculated nominal side resistance = 862 kips

Total calculated nominal tip resistance = 1431 kips

Considering load transfer to base, the nominal tip resistance = 26.85/73.15 × 862 kips = 316 kips

Total factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S = 0.50 \times 316 \text{ kips} + 0.55 \times 862 \text{ kips} = 631$ kips.

Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. This makes tip factored resistance $R_R = \varphi_{qp}R_P = 0.50 \times 1431 \text{ kips} = 716$ kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

Load Transfer of Factored Resistance from Side to Base in Drilled Shaft
As per GDM Section 1306.3.2

Sub-Structure: C12/C15

Boring: B-007-3-64

Rock socket diameter D = 3.5 feet

Rock socket length is L = 5.5 feet

E_i = 43,650 psi and RQD = 60%

$Q_b/Q_t = 35(L/D^{)-0.6}$, where $E_i \times RQD < 50{,}000$ psi

$Q_b/Q_t = 30(L/D^{)-0.8}$, where $50{,}000 \text{ psi} \leq E_i \times RQD < 500{,}000$ psi

$Q_b/Q_t = 25(L/D)^{-1.1}$, where $E_i \times RQD \geq 500{,}000$ psi

$E_i \times RQD$ = 43,650 psi × 60.0% = 26,090 psi <50,000 psi; therefore,

Select $Q_b/Q_t = 35(L/D^{)-0.6} = 35(3.5/5.5)^{-0.6} = 26.69\%$

The load transfer to the drilled shaft tip = 26.69%

The load transfer to the side = 100% - 26.69% = 73.31%

Total calculated nominal side resistance = 470 kips

Total calculated nominal tip resistance = 866 kips.

Considering load transfer to base, the nominal tip resistance = 26.69/73.31 × 470 kips = 172 kips

Total factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S$ = 0.50 × 172 kips + 0.55 × 470 kips = 344 kips.

Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. This makes tip factored resistance $R_R = \varphi_{qp}R_P$ = 0.50 × 866 kips = 433 kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

Load Transfer of Factored Resistance from Side to Base in Drilled Shaft
As per GDM Section 1306.3.2

Sub-Structure: Pier 1

Boring: B-004-0-23

Rock socket diameter D = 4.0 feet

Rock socket length is L = 6.0 feet

E_i = 43,650 psi and RQD = 62%

$Q_b/Q_t = 35(L/D)^{-0.6}$, where $E_i \times RQD < 50,000$ psi

$Q_b/Q_t = 30(L/D)^{-0.8}$, where $50,000 \text{ psi} \leq E_i \times RQD < 500,000$ psi

$Q_b/Q_t = 25(L/D)^{-1.1}$, where $E_i \times RQD \geq 500,000$ psi

$E_i \times RQD = 43,650 \text{ psi} \times 62.0\% = 27,063 \text{ psi} <50,000$ psi; therefore,

Select $Q_b/Q_t = 35(L/D)^{-0.6} = 35(6.0/4.0)^{-0.6} = 27.44\%$

The load transfer to the drilled shaft tip = 27.44%

The load transfer to the side = 100% - 27.44% = 72.56%

Total calculated nominal side resistance = 612 kips

Total calculated nominal tip resistance = 1131 kips.

Considering load transfer to base, the nominal tip resistance = 27.44/72.56 × 612 kips = 231 kips

Total factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S = 0.50 \times 231 \text{ kips} + 0.55 \times 612 \text{ kips} = 452$ kips.

Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. This makes tip factored resistance $R_R = \varphi_{qp}R_P = 0.50 \times 1131$ kips = 565 kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

Load Transfer of Factored Resistance from Side to Base in Drilled Shaft
As per GDM Section 1306.3.2

Sub-Structure: Pier 2

Boring: B-005-0-23

Rock socket diameter D = 4.0 feet

Rock socket length is L = 6.0 feet

E_i = 22,500 psi and RQD = 75%

$Q_b/Q_t = 35(L/D^{)-0.6}$, where $E_i \times RQD < 50,000$ psi

$Q_b/Q_t = 30(L/D^{)-0.8}$, where $50,000 \text{ psi} \leq E_i \times RQD < 500,000$ psi

$Q_b/Q_t = 25(L/D)^{-1.1}$, where $E_i \times RQD \geq 500,000$ psi

$E_i \times RQD$ = 22,500 psi × 75.0% = 16,875 psi <50,000 psi; therefore,

Select $Q_b/Q_t = 35(L/D^{)-0.6} = 35(6.0/4.0)^{-0.6}$ = 27.44%

The load transfer to the drilled shaft tip = 27.44%

The load transfer to the side = 100% - 27.44% = 72.56%

Total calculated nominal side resistance = 439 kips

Total calculated nominal tip resistance = 1131 kips.

Considering load transfer to base, the nominal tip resistance = 27.44/72.56 × 439 kips = 166 kips

Total nominal resistance is $R_P + R_S$ = 166 kips + 439 kips = 605 kips.

Total factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S = 0.50 \times 166 \text{ kips} + 0.55 \times 439 \text{ kips}$ = 325 kips.

Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; it would be better to take the tip resistance alone than to count on the side resistance with limited mobilization of the tip resistance. This makes the tip factored resistance $R_R = \varphi_{qp}R_P = 0.50 \times 1131$ kips = 565 kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

Weak and Augered Rock Unconfined Strength Calculations as per GDM Section 404.5

For weak, augered bedrock, use the SPT N-value to estimate the unconfined compressive strength (UCS = Q_u) per publication FHWA-ICT-17-018 “Modified Standard Penetration Test–based Drilled Shaft Design Method for Weak Rocks” (Stark et.al., 2017), Equation 2.2:
UCS (ksf) = 0.092 × (Nrate)90 (bpf).
There are additional possible modifiers to the equation for borehole diameter, sampler liner, and rod length;
see FHWA-ICT-17-018, Table Q.1 and Skempton (1986) for additional details.

For a bedrock sampled in Boring B-006-0-23,SS-2 by SPT using a hammer with ER = 90.0, resulting in a blow count value of 100/12”.
The following would be the case:
N_{90} = 100/12” × 12” = 100 bpf;
N_{90} = 90/90 × N_{90} = 1.0 × 100 bpf = 100 bpf;
Q_u (ksf) = 0.092 × N_{90} = 0.092 × 100 bpf = 9.2 ksf = 64 psi.

For a bedrock sampled in Boring B-001-0-23,SS-20B by SPT using a hammer with ER = 90.0, resulting in a blow count value of 50/2”.
The following would be the case:
N_{90} = 50/2” × 12” = 300 bpf;
N_{90} = 90/90 × N_{90} = 1.0 × 300 bpf = 300 bpf;
Q_u (ksf) = 0.092 × N_{90} = 0.092 × 300 bpf = 27.6 ksf = 192 psi.

For a bedrock sampled in Boring B-002-0-23,SS-12B by SPT using a hammer with ER = 90.0, resulting in a blow count value of 50/3”.
The following would be the case:
N_{90} = 50/3” × 12” = 200 bpf;
N_{90} = 90/90 × N_{90} = 1.0 × 200 bpf = 200 bpf;
Q_u (ksf) = 0.092 × N_{90} = 0.092 × 200 bpf = 18.4 ksf = 128 psi.

For a bedrock sampled in Boring B-003-0-23,SS-11B by SPT using a hammer with ER = 90.0, resulting in a blow count value of 50/4”.
The following would be the case:
N_{90} = 50/4” × 12” = 150 bpf;

N_{90} = 90/90 × N_{90} = 1.0 × 150 bpf = 150 bpf;

Q_u (ksf) = 0.092 × N_{90} = 0.092 × 150 bpf = 13.8 ksf = 96 psi.

For a bedrock sampled in Boring B-004-0-23,SS-6B by SPT using a hammer with ER = 90.0, resulting in a blow count value of 50/2”. The following would be the case:

N_{90} = 50/2” × 12” = 300 bpf;

N_{90} = 90/90 × N_{90} = 1.0 × 300 bpf = 300 bpf;

Q_u (ksf) = 0.092 × N_{90} = 0.092 × 300 bpf = 27.6 ksf = 192 psi.

For a bedrock sampled in Boring B-005-0-23,SS-6 by SPT using a hammer with ER = 90.0, resulting in a blow count value of 129/12”. The following would be the case:

N_{90} = 129/12” × 12” = 129 bpf;

N_{90} = 90/90 × N_{90} = 1.0 × 129 bpf = 129 bpf;

Q_u (ksf) = 0.092 × N_{90} = 0.092 × 129 bpf = 11.9 ksf = 82 psi.

Bearing Resistance of Bedrock In accordance with GDM Section 1303.3.3

Project: HAM-71-1.80 Project No.: G23005G

Boring No.: B-006-0-23 Substructure Unit: Pier 3

ALL of the following three conditions were met:

- the bedrock surface under the footing is not steeply sloping such that discontinuities would control the bearing resistance (a bedrock slope of 2H:1V or less),
- the foundation bedrock has a Rock Mass Rating (RMR) ≤ 70, and OHIO DEPARTMENT OF TRANSPORTATION January 2024 Geotechnical Design Manual Page 13-15
- the foundation bedrock is moderately strong or less in strength ($q_u \leq 7500$ psi),

Drained shear strength properties (c′ and ϕ′) were calculated in accordance with Bieniawski (1989) and use the Terzaghi/Vesic/Munfakh method to calculate nominal bearing resistance of the bedrock in accordance with AASHTO LRFD Article 10.6.3.1.2a.

The Bieniawski (1989) drained shear strength equations are as follows:

$c' = [0.104 \times RMR]$ (ksf); $\phi' = [(RMR/2) + 5°]$ (degrees)

Rock Mass Rating of 31 and considered as “Poor Rock” was obtained as per AASHTO LRFD Table 10.4.6.4-1.

$c' = 0.104 \times 31 = 3.224$ ksf = 3224 psf $\phi' = (31/2) + 5° = 21$ Degrees

In determination of the RMR, keep in mind the following.

The Spacing of Joints component of the RMR is not related to bedding, but is related to discontinuities.

Furthermore, similarly to calculation of Rock Quality Designation (RQD), ignore mechanical breaks. For example, a 10-foot run of shale, with bedding of less than an inch could have only 3 or 4 natural, non-mechanical, discontinuities, such that the Spacing of Joints would fall in the 1-to-3-foot range, and not in the < 2-inch range.

If the natural discontinuities do not contain gouge or slickensides, do not rate them within the lowest two categories for Condition of Joints.

Do not use the cases of “Water under moderate pressure” or “Severe water problems.” The original intent of the RMR is for stability of the rock in roof-support for tunneling excavations, where moderate to severe water pressures could be encountered in the rock at some depth, and interstitial water pressure can make the rock significantly less stable. The Ground Water component of the RMR is not applicable to considerations of rock stability in shallow bearing considerations for spread footings (or even drilled shafts). In fact, it is recommended in the literature to ignore the Ground Water factor, and always assume “Completely Dry, 10” for surface excavations of the rock; however, we prefer to default to “Moist Only, 7” out of conservatism, unless the rock is indeed completely dry.

AASHTO LRFD Article 10.6.3.1.2c states, “Limit analysis, or limit equilibrium analysis, should be considered to estimate the nominal bearing resistance of footings on or adjacent to slopes composed of soils and/or site conditions that are not consistent with the parameters and conditions described in the reference documents (i.e., embedment >0, layered soils, steeper slopes).” However, there is no reason that bearing resistance calculation by limit equilibrium (LE) analysis need be limited to footings on or adjacent to slopes.

This method particularly lends itself to layered soil systems, which are relatively difficult to analyze by other methods, such as those which are recommended in AASHTO LRFD Articles 10.6.3.1.2d through 10.6.3.1.2f.

BEARING CAPACITY ANALYSIS

AASHTO Article 10.6.3.2 and Munfakh, et al. (2001)

Project	**HAM-71-1.80**
Project#	**G23006G**
Bore#	**B-006-0-23 (Pier 3)**
Method	**AASHTO 10.6.3.1.2**
Foundation Dimension	
Width of Footing (B) (feet)	10.00
Length of Footing (L) (feet)	23.50
Length (L_f)/Width (B_f) (>5 is continous footing)	2.4
Type of Footing	Spread
Footing Bearing Elevation (feet)	561.00
Depth of Footing (D_f) Feet below Proposed Grade	3.0
Depth of groundwater Table (D_w) below Footing (ft)	0.0
Height of Slope (Hs) (feet)	Flat Ground
Soil Parameters	
Undrained Shear Strength/Cohesion (psf)	3224
Angle of internal friction (Phi) Degrees	21
Unit Weight of soil above base of footing (pcf)	125
Unit Weight of soil below base of footing (pcf)	140
Bearing Capacity Factors (from AASHTO LRFD Table 10.6.3.1.2a-1)	
N_c	15.80
N_q	7.10
$N\gamma$	6.20
Shape Correction Factors	
s_c	1.19
s_q	1.16
s_γ	0.83
Load Inclination Factors	
ic	1.0
iq	1.0
i_γ	1.0
Correction for Water Table	
D_f+1.5B_f	18.0
C_{wq}	0.500
$C_{w\gamma}$	0.500
Embedment Depth Correction Factor	
Df/Bf	0.3
d_q	1.0
Bearing Capacity Terms	
Cohesion Term	60680
Surcharge Term	1549
Unit Weight Term	1801
Nominal Bearing Resistence (psf)	64029
Resistance Factor for bearing (per AASHTO Table 10.5.5.2.2-1)	0.45
Factored Bearing Resistence (psf)	28813

AASHTO Eqn 10.6.3.1.2a

qn = c*Nc*Sc*ic + (Gamma)*Df*Nq*sq*dq*iq*Cwq+0.5*(Gamma)*Bf*Nγ*sγ*iγ*Cwγ

HAM-71 - Ramp Footing Bearing and Overturning Check

Stage 3

Designer: N. Swank **Date:** 12/27/2023

Checker: B. Beasley **Date:** 4/1/2024

Final

Designer: N. Swank **Date:** 8/8/2024

Checker: B. Beasley **Date:** 9/4/2024

This sheet is used to calculate ABLRFD inputs.

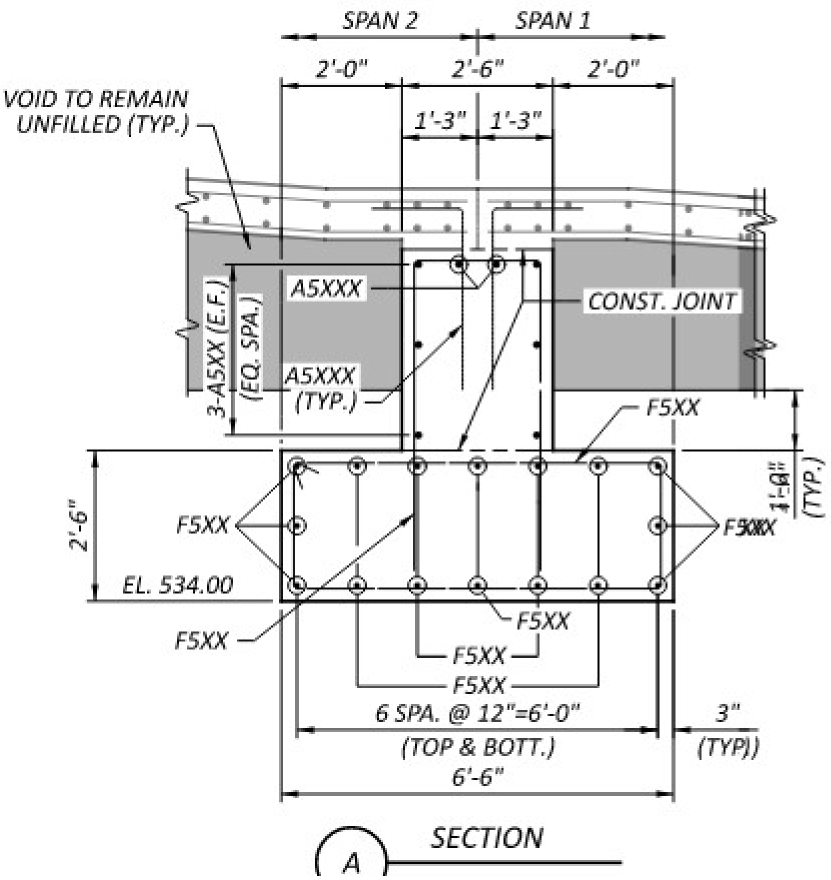
User inputs are highlighted in - Yellow

Sheet assumption are highlighted in - *User may change if necessary* Blue

Sheet checks are highlighted in - Green

Inputs from other files are highlighted in purple - Purple

$\gamma_{conc} := 150 \cdot pcf$



$Footing\_Width := 6 \cdot ft + 6 \cdot in$ Iterate

Bearing Check

Substructure Deadload

$Stem\_ht := mean(539.75 - 536.50, 539.94 - 536.50)\cdot ft = 3.345\,ft$

$Stem_{wt} := Stem\_ht\cdot[2\cdot(1\cdot ft + 3\cdot in)]\cdot(13\cdot ft + 10\cdot in)\cdot\gamma_{conc} = 17.352\cdot kip$

$Footing_{area} := (Footing\_Width)\cdot[15\cdot ft + 10\cdot in - (1\cdot ft + 0\cdot in + 6\cdot in)] = 93.167\,ft^2$

$Footing_{wt} := (2\cdot ft + 6\cdot in)\cdot(Footing\_Width)\cdot(14\cdot ft + 10\cdot in)\cdot\gamma_{conc} = 36.156\cdot kip$

$$Substructure_{bearing\_pressure\_ser} := \frac{Stem_{wt} + Footing_{wt}}{Footing_{area}} = 0.574\cdot ksf$$

$Substructure_{bearing\_pressure\_str} := 1.25\cdot Substructure_{bearing\_pressure\_ser} = 0.718\cdot ksf$

'Superstructure' Deadload

$$Span_1 := (1\cdot ft + 6\cdot in) + (1\cdot ft + 9\cdot in) + \left(25\cdot ft + 0\frac{3}{8}\cdot in\right) + 2\cdot(1\cdot ft + 3\cdot in) = 30.781\,ft$$

$$Span_2 := 32\cdot ft + 6\frac{7}{8}\cdot in = 32.573\,ft$$

$Ramp\_Width := (6\cdot ft + 8\cdot in) + (7\cdot ft + 2\cdot in) = 13.833\,ft$

$Ramp_{thick} := 10\cdot in$

$Curb_{area} := mean(8\cdot in, 1\cdot ft + 2\cdot in)\cdot 4\cdot in = 0.306\,ft^2$

$Rail_{wt} := 50\cdot plf$

$$Superstructure_{wt} := \left(\frac{Span_1}{2} + \frac{Span_2}{2}\right)\cdot\left[\left(Ramp\_Width\cdot Ramp_{thick} + 2\cdot Curb_{area}\right)\cdot\gamma_{conc} + 2\cdot Rail_{wt}\right] = 60.846\cdot kip$$

$$Superstructure_{bearing\_pressure\_ser} := \frac{Superstructure_{wt}}{Footing_{area}} = 0.653\cdot ksf$$

$Superstructure_{bearing\_pressure\_str} := 1.25\cdot Superstructure_{bearing\_pressure\_ser} = 0.816\cdot ksf$

Pedestrian Live Load - Both Spans Loaded

$$\text{Ped\_LL} := 90 \cdot \text{psf}$$

$$\text{Walkway\_Area}_{\text{Ped\_Load}} := \left(\frac{\text{Span}_1}{2} + \frac{\text{Span}_2}{2} \right) \cdot (12 \cdot \text{ft} + 0 \cdot \text{in}) \cdot \text{Ped\_LL} = 34.211 \cdot \text{kip}$$

$$\text{Pedestrian}_{\text{bearing\_pressure\_ser\_even}} := \frac{\text{Walkway\_Area}_{\text{Ped\_Load}}}{\text{Footing}_{\text{area}}} = 0.367 \cdot \text{ksf}$$

$$\text{Pedestrian}_{\text{bearing\_pressure\_str\_even}} := 1.75 \cdot \text{Pedestrian}_{\text{bearing\_pressure\_ser\_even}} = 0.643 \cdot \text{ksf}$$

Maximum Bearing Pressure- Vertical Only - Zero Eccentricity

$$\begin{aligned} \text{Max\_Bearing}_{\text{ser\_even}} := \; & \text{Substructure}_{\text{bearing\_pressure\_ser}} \cdots \quad = 1.595 \cdot \text{ksf} \\ & + \text{Superstructure}_{\text{bearing\_pressure\_ser}} \cdots \\ & + \text{Pedestrian}_{\text{bearing\_pressure\_ser\_even}} \end{aligned}$$

$$\begin{aligned} \text{Max\_Bearing}_{\text{str\_even}} := \; & \text{Substructure}_{\text{bearing\_pressure\_str}} \cdots \quad = 2.177 \cdot \text{ksf} \\ & + \text{Superstructure}_{\text{bearing\_pressure\_str}} \cdots \\ & + \text{Pedestrian}_{\text{bearing\_pressure\_str\_even}} \end{aligned}$$

Pedestrian Live Load - Single Span Loaded with Slab Fixed to Supports

Service Case

$$\text{Walkway\_Area}_{\text{Ped\_Load\_uneven\_ser}} := (12\cdot\text{ft} + 0\cdot\text{in})\cdot\text{Ped\_LL} = 1.08\cdot\text{klf}$$

$$V_{\text{Ped\_Loa\_uneven\_ser}} := \frac{\text{Walkway\_Area}_{\text{Ped\_Load\_uneven\_ser}}\cdot\text{Span}_2}{2} = 17.589\cdot\text{kip}$$

$$M_{\text{Ped\_Loa\_uneven\_ser}} := \frac{\text{Walkway\_Area}_{\text{Ped\_Load\_uneven\_ser}}\cdot{\text{Span}_2}^2}{12} = 95.49\,\text{ft}\cdot\text{kip}$$

$$\Sigma_{\text{vert\_loads\_ser}} := \text{Stem}_{\text{wt}} + \text{Footing}_{\text{wt}} + \text{Superstructure}_{\text{wt}} + V_{\text{Ped\_Loa\_uneven\_ser}} = 131.944\cdot\text{kip}$$

$$e_{\text{ser}} := \frac{M_{\text{Ped\_Loa\_uneven\_ser}}}{\Sigma_{\text{vert\_loads\_ser}}} = 8.685\cdot\text{in}$$

$$\frac{\text{Footing\_Width}}{6} = 1.083\,\text{ft} \qquad \text{if}\left(e_{\text{ser}} < \frac{\text{Footing\_Width}}{6}, \text{"Trapezoidal Bearing"}, \text{"Triangular Bearing"}\right) = \text{"Trapezoidal Bearing"}$$

Strength Case

$$V_{\text{Ped\_Loa\_uneven\_str}} := \frac{1.75\cdot\left(\text{Walkway\_Area}_{\text{Ped\_Load\_uneven\_ser}}\right)\cdot\text{Span}_2}{2} = 30.781\cdot\text{kip}$$

$$M_{\text{Ped\_Loa\_uneven\_str}} := \frac{1.75\cdot\left(\text{Walkway\_Area}_{\text{Ped\_Load\_uneven\_ser}}\right)\cdot{\text{Span}_2}^2}{12} = 167.107\,\text{ft}\cdot\text{kip}$$

$$\Sigma_{\text{vert\_loads\_str}} := (.9)\cdot\text{Stem}_{\text{wt}} + (.9)\cdot\text{Footing}_{\text{wt}} + (.9)\cdot\text{Superstructure}_{\text{wt}} + V_{\text{Ped\_Loa\_uneven\_str}} = 133.701\cdot\text{kip}$$

$$e_{\text{str}} := \frac{M_{\text{Ped\_Loa\_uneven\_str}}}{\Sigma_{\text{vert\_loads\_str}}} = 1.25\,\text{ft}$$

$$\frac{\text{Footing\_Width}}{6} = 1.083\,\text{ft} \qquad \text{if}\left(e_{\text{str}} < \frac{\text{Footing\_Width}}{6}, \text{"Trapezoidal Bearing"}, \text{"Triangular Bearing"}\right) = \text{"Triangular Bearing"}$$

Actual / Effective Bearing Dimensions

$$\text{Abut\_Width}_{\text{act}} := 6\cdot\text{ft} + 6\cdot\text{in}$$

$$\text{Abut\_Width}_{\text{ser\_eff}} := \text{Abut\_Width}_{\text{act}} - 2\cdot e_{\text{ser}} = 5.053\,\text{ft}$$

$$\text{Abut\_Width}_{\text{str\_eff}} := \text{Abut\_Width}_{\text{act}} - 2\cdot e_{\text{str}} = 4\,\text{ft}$$

$$\text{Abut\_Lgth}_{\text{act}} := 14\cdot\text{ft} + 4\cdot\text{in}$$

$$\text{Abut\_Lgth}_{\text{ser\_eff}} := \text{Abut\_Lgth}_{\text{act}} = 14.333\,\text{ft}$$

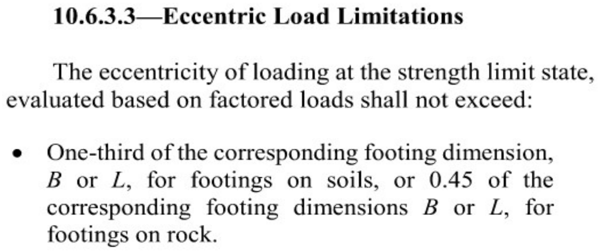
$$\text{Abut\_Lgth}_{\text{str\_eff}} := \text{Abut\_Lgth}_{\text{act}} = 14.333\,\text{ft}$$

Maximum Effective Bearing Pressure

$$\text{Maximum\_Bearing}_{\text{ser\_effective}} := \frac{\Sigma_{\text{vert\_loads\_ser}}}{\left(\text{Abut\_Width}_{\text{ser\_eff}}\cdot\text{Abut\_Lgth}_{\text{ser\_eff}}\right)} = 1.822\cdot\text{ksf}$$

$$\text{Maximum\_Bearing}_{\text{str\_effective}} := \frac{\Sigma_{\text{vert\_loads\_str}}}{\left(\text{Abut\_Width}_{\text{str\_eff}}\cdot\text{Abut\_Lgth}_{\text{str\_eff}}\right)} = 2.332\cdot\text{ksf}$$

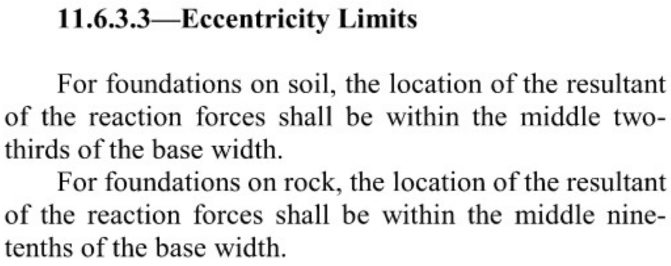
Overturning Check



10.6.3.3—Eccentric Load Limitations

The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:

- One-third of the corresponding footing dimension, *B* or *L*, for footings on soils, or 0.45 of the corresponding footing dimensions *B* or *L*, for footings on rock.



11.6.3.3—Eccentricity Limits

For foundations on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds of the base width.

For foundations on rock, the location of the resultant of the reaction forces shall be within the middle nine-tenths of the base width.

Eccentricity at Strength Limit State: $e_{str} = 14.998 \cdot in$

Width of Footing: $B := Abut\_Width_{act} = 6.5\,ft$

$$if\left(e_{str} < \frac{B}{3}, \text{"Eccentricity OKAY"}, \text{"Revise Footing"}\right) = \text{"Eccentricity OKAY"}$$

Bearing Check

$Maximum\_Bearing_{str\_effective} = 2.332 \cdot ksf$

$Factored\_Bearing\_Resistance := 4.31 \cdot ksf$ *Final Geotechnical Exploration Rpt for HAM-71-1.81, Table 6.1.5, Dated Sept 3, 2024*

$$if\left(Maximum\_Bearing_{str\_effective} < Factored\_Bearing\_Resistance, \text{"Bearing OK"}, \text{"Bearing NG"}\right) = \text{"Bearing OK"}$$

HAM-71 - Retaining Wall Footing Bearing and Overturning Check

Stage 3

Designer: N. Swank **Date:** 12/28/2023

Checker: B. Beasley **Date:** 1/4/2023

Final

Designer: N. Swank **Date:** 8/8/2024

Checker: B. Beasley **Date:** 9/4/2024

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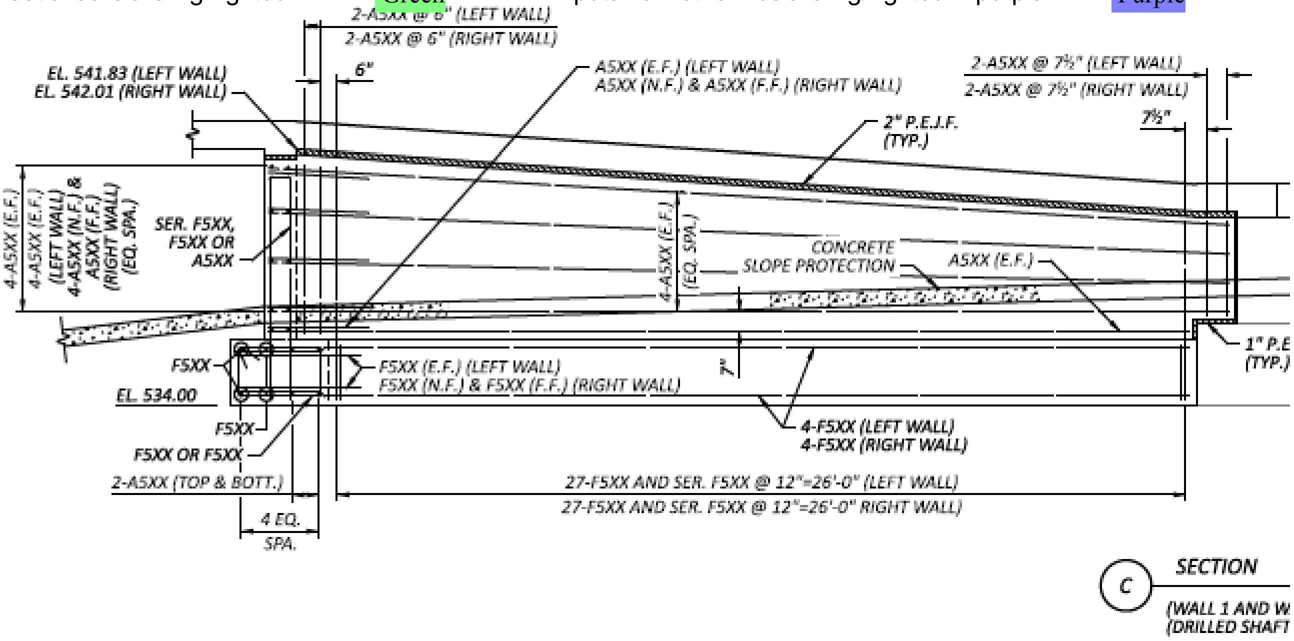
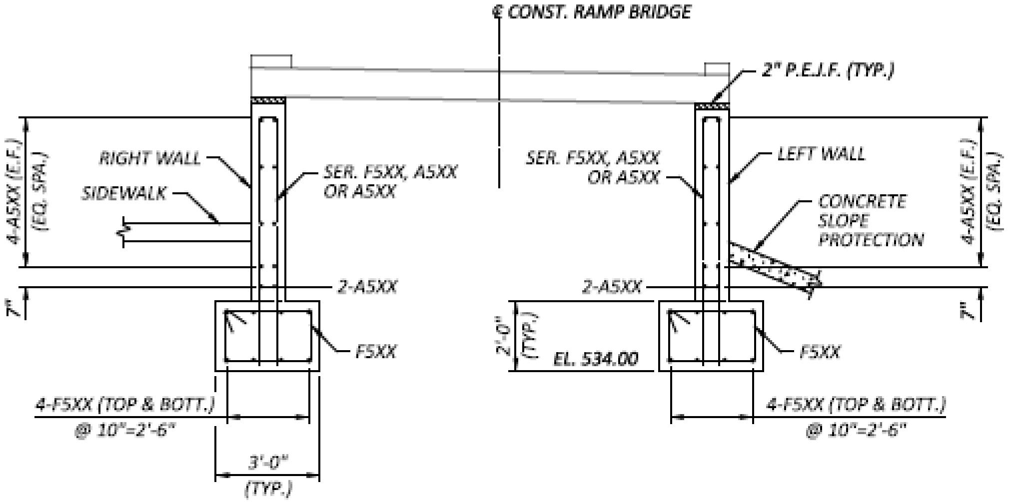


$Footing\_Width := 3 \cdot ft + 0 \cdot in$

$\gamma_{conc} := 150 \cdot pcf$



Substructure Deadload

$\text{Stem\_ht} := \text{mean}(541.83 \cdot \text{ft} - 536.00 \cdot \text{ft} - 2 \cdot \text{in}, 540.10 \cdot \text{ft} - 536.00 \cdot \text{ft} - 2 \cdot \text{in}) = 4.798\,\text{ft}$

$\text{Stem}_{\text{wt}} := \text{Stem\_ht} \cdot (1 \cdot \text{ft} + 0 \cdot \text{in}) \cdot \gamma_{\text{conc}} = 0.72 \cdot \text{klf}$

$\text{Footing}_{\text{area}} := \text{Footing\_Width} = 3 \cdot \frac{\text{ft}^2}{\text{ft}}$

$\text{Footing}_{\text{wt}} := (2 \cdot \text{ft} + 0 \cdot \text{in}) \cdot (\text{Footing\_Width}) \cdot \gamma_{\text{conc}} = 0.9 \cdot \text{klf}$

$\text{Substructure}_{\text{bearing\_pressure\_ser}} := \frac{\text{Stem}_{\text{wt}} + \text{Footing}_{\text{wt}}}{\text{Footing}_{\text{area}}} = 0.54 \cdot \text{ksf}$

$\text{Substructure}_{\text{bearing\_pressure\_str}} := 1.25 \cdot \text{Substructure}_{\text{bearing\_pressure\_ser}} = 0.675 \cdot \text{ksf}$

'Superstructure' Deadload

$\text{Ramp\_Width} := (6 \cdot \text{ft} + 8 \cdot \text{in}) + (7 \cdot \text{ft} + 2 \cdot \text{in}) = 13.833\,\text{ft}$

$\text{Ramp}_{\text{thick}} := 10 \cdot \text{in}$

$\text{Curb}_{\text{area}} := \text{mean}(8 \cdot \text{in}, 1 \cdot \text{ft} + 2 \cdot \text{in}) \cdot 4 \cdot \text{in} = 0.306\,\text{ft}^2$

$\text{Rail}_{\text{wt}} := 50 \cdot \text{plf}$

$\text{Superstructure}_{\text{wt}} := \frac{\left[\text{Ramp}_{\text{thick}} \cdot (\text{Ramp\_Width}) + 2 \cdot \text{Curb}_{\text{area}}\right] \cdot \gamma_{\text{conc}} + 2 \cdot \text{Rail}_{\text{wt}}}{4} = 0.48 \cdot \text{klf}$

$\text{Superstructure}_{\text{bearing\_pressure\_ser}} := \frac{\text{Superstructure}_{\text{wt}}}{\text{Footing\_Width}} = 0.16 \cdot \text{ksf}$

$\text{Superstructure}_{\text{bearing\_pressure\_str}} := 1.25 \cdot \text{Superstructure}_{\text{bearing\_pressure\_ser}} = 0.2 \cdot \text{ksf}$

Pedestrian Live Load

$\text{Ped\_LL} := 90 \cdot \text{psf}$

$\text{Walkway\_Area}_{\text{Ped\_Load}} := \frac{(12 \cdot \text{ft} + 0 \cdot \text{in}) \cdot \text{Ped\_LL}}{4} = 0.27 \cdot \text{klf}$

$\text{Pedestrian}_{\text{bearing\_pressure\_ser}} := \frac{\text{Walkway\_Area}_{\text{Ped\_Load}}}{\text{Footing\_Width}} = 0.09 \cdot \text{ksf}$

$\text{Pedestrian}_{\text{bearing\_pressure\_str}} := 1.75 \cdot \text{Pedestrian}_{\text{bearing\_pressure\_ser}} = 0.158 \cdot \text{ksf}$

Maximum Actual Bearing Pressure

$$\mathrm{Max\_Bearing}_{\mathrm{ser\_even}} := \mathrm{Substructure}_{\mathrm{bearing\_pressure\_ser}} \cdots + \mathrm{Superstructure}_{\mathrm{bearing\_pressure\_ser}} \cdots + \mathrm{Pedestrian}_{\mathrm{bearing\_pressure\_ser}} = 0.79 \cdot \mathrm{ksf}$$

$$\mathrm{Max\_Bearing}_{\mathrm{str\_even}} := \mathrm{Substructure}_{\mathrm{bearing\_pressure\_str}} \cdots + \mathrm{Superstructure}_{\mathrm{bearing\_pressure\_str}} \cdots + \mathrm{Pedestrian}_{\mathrm{bearing\_pressure\_str}} = 1.032 \cdot \mathrm{ksf}$$

Actual / Effective Bearing Dimensions

$e_{\mathrm{ser\_rw}} := 0 \cdot \mathrm{in}$

$e_{\mathrm{str\_rw}} := 0 \cdot \mathrm{ft}$

$\mathrm{RW\_Width}_{\mathrm{act}} := 3 \cdot \mathrm{ft} + 0 \cdot \mathrm{in}$

$\mathrm{RW\_Lgth}_{\mathrm{act}} := 23 \cdot \mathrm{ft} + 6 \cdot \mathrm{in}$

$\mathrm{RW\_Width}_{\mathrm{ser\_eff}} := \mathrm{RW\_Width}_{\mathrm{act}} - 2 \cdot e_{\mathrm{ser\_rw}} = 3\ \mathrm{ft}$

$\mathrm{RW\_Lgth}_{\mathrm{ser\_eff}} := \mathrm{RW\_Lgth}_{\mathrm{act}} = 23.5\ \mathrm{ft}$

$\mathrm{RW\_Width}_{\mathrm{str\_eff}} := \mathrm{RW\_Width}_{\mathrm{act}} - 2 \cdot e_{\mathrm{str\_rw}} = 3\ \mathrm{ft}$

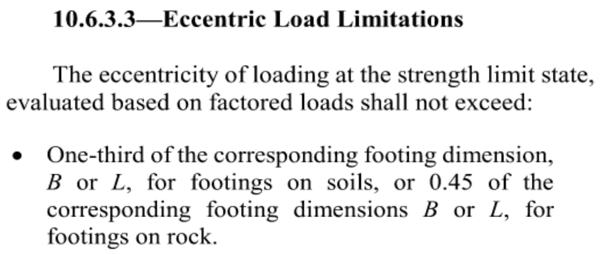
$\mathrm{RW\_Lgth}_{\mathrm{str\_eff}} := \mathrm{RW\_Lgth}_{\mathrm{act}} = 23.5\ \mathrm{ft}$

Maximum Effective Bearing Pressure

$$\mathrm{Max\_Bearing}_{\mathrm{ser\_even\_effective}} := \frac{\mathrm{Max\_Bearing}_{\mathrm{ser\_even}} \cdot \left(\mathrm{RW\_Width}_{\mathrm{act}} \cdot \mathrm{RW\_Lgth}_{\mathrm{act}}\right)}{\left(\mathrm{RW\_Width}_{\mathrm{ser\_eff}} \cdot \mathrm{RW\_Lgth}_{\mathrm{ser\_eff}}\right)} = 0.79 \cdot \mathrm{ksf}$$

$$\mathrm{Max\_Bearing}_{\mathrm{str\_even\_Effective}} := \frac{\mathrm{Max\_Bearing}_{\mathrm{str\_even}} \cdot \left(\mathrm{RW\_Width}_{\mathrm{act}} \cdot \mathrm{RW\_Lgth}_{\mathrm{act}}\right)}{\left(\mathrm{RW\_Width}_{\mathrm{str\_eff}} \cdot \mathrm{RW\_Lgth}_{\mathrm{str\_eff}}\right)} = 1.032 \cdot \mathrm{ksf}$$

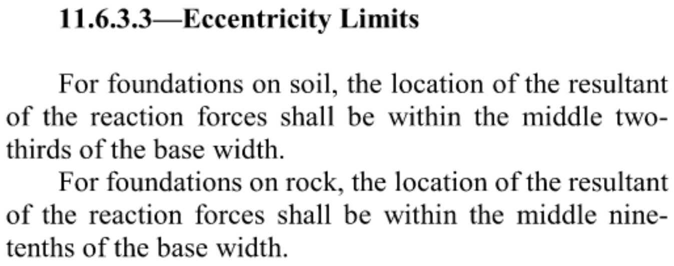
Overturning Check



10.6.3.3—Eccentric Load Limitations

The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:

- One-third of the corresponding footing dimension, *B* or *L*, for footings on soils, or 0.45 of the corresponding footing dimensions *B* or *L*, for footings on rock.



11.6.3.3—Eccentricity Limits

For foundations on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds of the base width.

For foundations on rock, the location of the resultant of the reaction forces shall be within the middle nine-tenths of the base width.

Eccentricity at Strength Limit State: $e_{str\_rw} = 0 \cdot in$

Width of Footing: $B := RW\_Width_{act} = 3\ ft$

$$\text{if}\left(e_{str\_rw} < \frac{B}{6}, \text{"Eccentricity OKAY"}, \text{"Revise Footing"}\right) = \text{"Eccentricity OKAY"}$$

Bearing Check

$Max\_Bearing_{str\_even\_Effective} = 1.032 \cdot ksf$

$Factored\_Bearing\_Resistance := 4.5 \cdot ksf$

Final Geotechnical Exploration Rpt for HAM-71-1.81, Table 6.1.5, Dated Sept 3, 2024

$$\text{if}\left(Max\_Bearing_{str\_even\_Effective} < Factored\_Bearing\_Resistance, \text{"Bearing OK"}, \text{"Bearing NG"}\right) = \text{"Bearing OK"}$$

AASHTO Article 11.6.3: ABUTMENT & RETAINING WALL

Project	**HAM-71-1.80 (Ramp Abutment)**
Project#	**G23006G**
Bore#	**B-002-0-23**
Method	**AASHTO Eqn 10.6.3.1.2a**
Foundation Dimension	
Width of Footing (B') (feet)	4.58
Length of Footing (L') (feet)	14.33
Length (L')/Width (B') (>5 is continous footing)	3.1
Type of Footing	Spread
Footing Bearing Elevation (feet)	534.0
Depth of Footing (D_f) Feet below Proposed Grade	3.0
Depth of Groundwater Table above Footing (ft)	0.0
Height of Slope (Hs) (feet)	Flat Ground
Soil Parameters	
Ave. Undrained Shear Strength/Cohesion (psf)	0
Angle of internal friction (Phi) Degrees	32
Unit Weight of soil above base of footing (pcf)	120
Unit Weight of soil below base of footing (pcf)	125
Bearing Capacity Factors per LRFD Table 10.6.3.1.2a-1	
N_c	35.50
N_q	23.20
$N\gamma$	30.20
Shape Correction Factors	
s_c	1.209
s_q	1.200
s_γ	0.872
Load Inclination Factors	
ic	1.0
iq	1.0
i_γ	1.0
Correction for Water Table	
D_f+1.5B'	9.9
C_{wq}	0.5
$C_{w\gamma}$	0.5
Embedment Depth Correction Factor	
Df/B'	0.7
d_q	1.16
Bearing Capacity Terms	
Cohesion Term	0
Surcharge Term	5811
Unit Weight Term	3767
Nominal Bearing Resistance (psf)	9578
Resistance Factor for bearing (per AASHTO Table 11.5.7-1)	0.45
Factored Bearing Resistence (psf)	4310

AASHTO Eqn 10.6.3.1.2a

qn = c*Nc*Sc*ic + (Gamma)*Df*Nq*sq*dq*iq*Cwq+0.5*(Gamma)*Bf*Nγ*sγ*iγ*Cwγ

AASHTO Article 11.6.3: ABUTMENT & RETAINING WALL

Project	**HAM-71-1.80 (Ramp Retaining Walls)**
Project#	**G23006G**
Bore#	**B-002-0-23**
Method	**AASHTO Eqn 10.6.3.1.2a**
Foundation Dimension	
Width of Footing (B') (feet)	3.0
Length of Footing (L') (feet)	23.5
Length (L')/Width (B') (>5 is continous footing)	7.8
Type of Footing	Strip
Footing Bearing Elevation (feet)	534.0
Depth of Footing (D_f) Feet below Proposed Grade	3.0
Depth of Groundwater Table above Footing (ft)	0.0
Height of Slope (Hs) (feet)	Flat Ground
Soil Parameters	
Ave. Undrained Shear Strength/Cohesion (psf)	0
Angle of internal friction (Phi) Degrees	32
Unit Weight of soil above base of footing (pcf)	120
Unit Weight of soil below base of footing (pcf)	125
Bearing Capacity Factors per LRFD Table 10.6.3.1.2a-1	
N_c	35.50
N_q	23.20
$N\gamma$	30.20
Shape Correction Factors	
s_c	1.083
s_q	1.080
s_γ	0.949
Load Inclination Factors	
ic	1.0
iq	1.0
i_γ	1.0
Correction for Water Table	
D_f+1.5B'	7.5
C_{wq}	0.5
$C_{w\gamma}$	0.5
Embedment Depth Correction Factor	
Df/B'	1.0
d_q	1.22
Bearing Capacity Terms	
Cohesion Term	0
Surcharge Term	5488
Unit Weight Term	2687
Nominal Bearing Resistance (psf)	8174
Resistance Factor for bearing (per AASHTO Table 11.5.7-1)	0.55
Factored Bearing Resistence (psf)	4496

AASHTO Eqn 10.6.3.1.2a

qn = c*Nc*Sc*ic + (Gamma)*Df*Nq*sq*dq*iq*Cwq+0.5*(Gamma)*Bf*Nγ*sγ*iγ*Cwγ

SETTLEMENT ANALYSIS FOR RAMP RETAINING WALL AT STA. 0+56

Project:	HAM-71-1.80	**Project #**	G23006G	**Test Boring #**	B-002-0-23
Type of Foundation	Existing Grade Elevation (feet)	535.1	Groundwater Table below existing ground (feet)		18.5
Shallow Foundation (Strip Footing)	Bottom Elev. of Footing (feet)	534.0	Unit Weight of Water (pcf)		62.4
Length = 23.5'	Select Granular Fill Height (feet)		Pre-consolidation Pressure (psf)		
Width = 3.0'	Applied Pressure Top of Foundation Soil (psf)	800	Unit Weight of Fill above the Footing (pcf)		128

Depth Below Leveling Pad (Z)	**AVERAGE PROPERTIES**		**CALCULATIONS**		**Total**
D_f=3.0' & Z=0.0'	Thickness of Layer (feet)	1.4	OB Pressure at the top Layer(psf)	384	**Setlement**
D_f = Depth below Prop. Grade	Corrected SPT Value (N)	20	OB Pressure at the center Layer (psf)	474	**(inches)**
Conc & Stone frags w/sand (A-1-b)	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	630	
(Above Water Table)	Moisture content (%)	6	Bearing Capacity Index (C)	95	
Z=0.70' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.06	**0.06**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.56	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	128			
D_f=4.4' & Z=1.4'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	563	
D_f=4.4' & Z=1.4'	Thickness of Layer (feet)	1.5	OB Pressure at the top Layer(psf)	563	**Setlement**
	Corrected SPT Value (N)	20	OB Pressure at the center Layer (psf)	655	**(inches)**
Sandy Silt (A-4a)	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	427	
(Above Water Table)	Moisture content (%)	5	Bearing Capacity Index (C)	95	
Z=2.15' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.04	**0.04**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.75	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	122			
D_f=5.9' & Z=2.9'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	746	
D_f=5.9' & Z=2.9'	Thickness of Layer (feet)	3.0	OB Pressure at the top Layer(psf)	746	**Setlement**
Gravel & Ston Frags w/sand (A-1-b)	Corrected SPT Value (N)	8	OB Pressure at the center Layer (psf)	926	**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	273	
(Above Water Table)	Moisture content (%)	5	Bearing Capacity Index (C)	52	
Z=4.4' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.08	**0.08**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.85	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	120			
D_f=8.9' & Z=5.9'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	1106	
D_f=8.9' & Z=5.9'	Thickness of Layer (feet)	6.5	OB Pressure at the top Layer(psf)	1106	**Setlement**
SF with sand and silt (A-2-4)	Corrected SPT Value (N)	9	OB Pressure at the center Layer (psf)	1300	**(inches)**
	Specific Gravity of Soil Solids (G)	2.67	Excess Pressure At Center Due to appliedLoad	142	

(Below Water Table)	Moisture content (%)	10	Bearing Capacity Index (C)	59	
Z=9.15' (At Centre of Layer)	Liquid Limit (%)	21	Immediate Settlement in Foundation Soil (inches)	0.06	**0.06**
	Plastic Limit (%)	16	Initial Void Ratio (e_0)	0.96	
	Plasticity Index (%)	5			
	Unit Weight of soil (pcf)	122			
D_f=15.4' & Z=12.4'	Submerged Unit Weight of Soil (pcf)	59.6	OB Pressure at the bottom Layer (psf)	1494	

Total Settlement:	0.24
Consoilidation Settlement:	
Immediate Settlement:	0.24

Test Boring B-002-0-23 (Sta. 0+56)
Stress Distribution using 2 V : 1 H Slope Method for Strip footing

Width of the footing B (feet)	**3**	**Length of the footing B (feet)**	**23.5**	**App. Design Pressure (psf)**	**800**

Depth (Z) below the Existing Ground (ft)	0.7	2.15	4.4	9.15					

Vertical Stress Intensity (psf) at Z	630	427	273	142					

SETTLEMENT ANALYSIS FOR RAMP ABUTMENT WALL AT STA. 0+56

Project:	HAM-71-1.80	**Project #**	G23006G	**Test Boring #**	B-002-0-23
Type of Foundation	Existing Grade Elevation (feet)	535.1	Groundwater Table below existing ground (feet)		18.5
Shallow Foundation (Spread Footing)	Bottom Elev. of Footing (feet)	534.0	Unit Weight of Water (pcf)		62.4
Effective Footing Length = 14.33'	Select Granular Fill Height (feet)		Pre-consolidation Pressure (psf)		
Effective Footing Width = 5.05'	Applied Pressure Top of Foundation Soil (psf)	1820	Unit Weight of Fill above the Footing (pcf)		128
Depth Below Leveling Pad (Z)	**AVERAGE PROPERTIES**		**CALCULATIONS**		**Total**
D_f=3.0' & Z=0.0'	Thickness of Layer (feet)	1.4	OB Pressure at the top Layer(psf)	384	**Settlement**
D_f = Depth below Prop. Grade	Corrected SPT Value (N)	20	OB Pressure at the center Layer (psf)	474	**(inches)**
Conc & Stone frags w/sand (A-1-b)	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to applied Load	1526	
(Above Water Table)	Moisture content (%)	6	Bearing Capacity Index (C)	95	
Z=0.70' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.11	**0.11**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.56	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	128			
D_f=4.4' & Z=1.4'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	563	
D_f=4.4' & Z=1.4'	Thickness of Layer (feet)	1.5	OB Pressure at the top Layer(psf)	563	**Settlement**
	Corrected SPT Value (N)	20	OB Pressure at the center Layer (psf)	655	**(inches)**
Sandy Silt (A-4a)	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to applied Load	1111	
(Above Water Table)	Moisture content (%)	5	Bearing Capacity Index (C)	95	
Z=2.15' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.08	**0.08**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.75	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	122			
D_f=5.9' & Z=2.9'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	746	
D_f=5.9' & Z=2.9'	Thickness of Layer (feet)	3.0	OB Pressure at the top Layer(psf)	746	**Settlement**
Gravel & Ston Frags w/sand (A-1-b)	Corrected SPT Value (N)	8	OB Pressure at the center Layer (psf)	926	**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to applied Load	745	
(Above Water Table)	Moisture content (%)	5	Bearing Capacity Index (C)	52	
Z=4.4' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.18	**0.18**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.85	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	120			
D_f=8.9' & Z=5.9'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	1106	
D_f=8.9' & Z=5.9'	Thickness of Layer (feet)	6.5	OB Pressure at the top Layer(psf)	1106	**Settlement**
SF with sand and silt (A-2-4)	Corrected SPT Value (N)	9	OB Pressure at the center Layer (psf)	1300	**(inches)**
	Specific Gravity of Soil Solids (G)	2.67	Excess Pressure At Center Due to applied Load	395	

(Below Water Table)	Moisture content (%)	10	Bearing Capacity Index (C)	59	
Z=9.15' (At Centre of Layer)	Liquid Limit (%)	21	Immediate Settlement in Foundation Soil (inches)	0.15	**0.15**
	Plastic Limit (%)	16	Initial Void Ratio (e_0)	0.96	
	Plasticity Index (%)	5			
	Unit Weight of soil (pcf)	122			
D_f=15.4' & Z=12.4'	Submerged Unit Weight of Soil (pcf)	59.6	OB Pressure at the bottom Layer (psf)	1494	
D_f=15.4' & Z=12.4'	Thickness of Layer (feet)	5.0	OB Pressure at the top Layer(psf)	1494	**Settlement**
Sandy silt (A-4a Plastic)	Corrected SPT Value (N)	9	OB Pressure at the center Layer (psf)	1633	**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to applied Load	226	
(Below Water Table)	Moisture content (%)	17	Compression Index (C_c)	0.17	
Z=14.9' (At Centre of Layer)	Liquid Limit (%)	30	Recompression Index (C_r)	0.017	
	Plastic Limit (%)	21	Initial Void Ratio (e_0)	0.67	
	Plasticity Index (%)	9	Settlement due to compression (inches)		
	Unit Weight of soil (pcf)	118	Settlement due to recompression (inches)	0.03	**0.03**
D_f=20.4' & Z=17.4'	Submerged Unit Weight of Soil (pcf)	55.6	OB Pressure at the bottom Layer (psf)	1772	

Total Settlement: 0.55

Consolidation Settlement: 0.03

Immediate Settlement: 0.52

Test Boring B-002-0-23 (Sta. 0+56)

Stress Distribution using 2 V : 1 H Slope Method for Spread footing (Square / Rectangular)

Width of the footing B (feet)	**5.05**	**Length of the footing B (feet)**	**14.33**	**App. Design Pressure (psf)**	**1822**

Depth (Z) below the footing (feet)	0.7	2.15	4.4	9.15	14.9				

Vertical Stress Intensity (psf) at Z	1526	1111	745	395	226				

IV.A Foundations of Structures Checklist

C-R-S: HAM-71-1.80
PID: 102790
Reviewer: SSHAN
Date: 7/18/23

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	Y	
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	Y	
	other (describe other methods)		
	Spread Footings	(Y/N/X)	Notes:
4	Are there spread footings on the project? If no, go to Question 11	Y	Bridge Pier 3
5	Have the recommended bottom of footing elevation and reason for this recommendation been provided?	Y	
a.	Has the recommended bottom of footing elevation taken scour from streams or other water flow into account?	X	
6	Were representative sections analyzed for the entire length of the structure for the following:		
a.	factored bearing resistance?	Y	
b.	factored sliding resistance?	X	
c.	eccentric load limitations (overturning)?	X	
d.	predicted settlement?	Y	
e.	overall (global) stability?	X	
7	Has the need for a shear key been evaluated?	X	
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?	X	
9	Have the Service I and Maximum Strength Limit States for bearing pressure on soil or rock been provided?	Y	

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

IV.A Foundations of Structures Checklist

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

IV.A Foundations of Structures Checklist

Drilled Shafts		(Y/N/X)	Notes:
20	Are there drilled shafts on the project? If no, go to the next checklist.	Y	
21	Have the drilled shaft diameter and embedment length been specified?	Y	
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:	N	
a.	total factored lateral shear?		
b.	total factored bending moment?		
c.	maximum deflection?		
d.	reinforcement design?		
24	If a bedrock socket is required, has a minimum rock socket length equal to 1.5 times the rock socket diameter been used, as per BDM 305.5.2?	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?	X	
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?	N	
29	If special construction features (e.g., slurry, casing, load tests) are required, have all the proper items been included in the plans?	N	
30	If necessary, have wet construction methods been specified?	X	
General		(Y/N/X)	Notes:
31	Has the need for load testing of the foundations been evaluated?	N	
a.	If needed, have details and plan notes for load testing been included in the plans?		

VI.B. Geotechnical Reports

C-R-S: HAM-71-1.81	**PID:** 102790	**Reviewer:** SShan	**Date:** 4/11/2024

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

VI.B. Geotechnical Reports

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

VI.A. Soil Profile Checklist

C-R-S: HAM-71-1.81	**PID:** 102790	**Reviewer:** SShan	**Date:** 4/11/2024

General Presentation		(Y/N/X)	Notes:
1	Has an electronic copy of all geotechnical submissions been provided to the District Geotechnical Engineer (DGE)?	N	Will be provided by ARC
2	Have the cadd files been prepared using the appropriate version of the ODOT CADD standards?	Y	
3	Has the geotechnical specification (title and date) under which the work was performed been clearly identified on every submission (reports, plans, etc.)?	Y	
4	Has the first complete version of all documents being submitted been labeled as 'Draft'?	Y	
5	Subsequent to ODOT's review and approval, has the complete version of the revised documents being submitted been labeled as 'Final'?	Y	
a.	Have the C-R-S, PID number, and product title been included in the folder name?	Y	
6	If the project includes structures, have all structure explorations been presented together under the same cover sheet? (Do not create separate Structure Foundation Exploration Sheets)	Y	
7	Has a scale of 1"=1' been used for cover sheets, laboratory test data sheets, and boring log sheets, if applicable?	Y	
8	Based on the project length, has the correct horizontal scale been used to plot the project data?		
	Check scale used:		
	1" = 5', 10', 20', 25', 40', or 50' for projects 1500' or less (use largest scale appropriate to present entire plan on one sheet)	Y	
	1" = 50' projects greater than 1500'		
9	Has a scale of 1" = 10' been utilized for the vertical scale of the project data?	Y	
10	If the project includes structures, has the plan and profile view been shown at the same scale as the Site Plan for the proposed structure(s), when possible?	Y	

VI.A. Soil Profile Checklist

General Presentation		(Y/N/X)	Notes:
11	If the project includes culverts, have the plan and profile been presented along the flowline of the culvert?	X	
12	Have the cross-sections been plotted at a scale of 1” = 10’ (preferred) or 1” = 20’ (for higher or wider slopes)?	X	
Cover Sheet		(Y/N/X)	Notes:
13	Has the following general information been provided on the cover sheet:	Y	
a.	Brief description of the project, including the bridge number of each bridge involved in the plan set, if any?	Y	
b.	Brief description of historic geotechnical explorations referenced in this exploration? State if no historic records are available.	Y	
c.	Generalized information about the geology of the project area, including terrain, soil origin, bedrock types, and age?	Y	
d.	Brief presentation of geological and topographical information derived from the field reconnaissance? Include comments on structure and pavement conditions.	Y	
e.	Brief presentation of test boring and sampling methods? Include date of last calibration and drill rod energy ratio as a percent for the hammer systems used.	Y	
f.	Summary of general soil, bedrock, and groundwater conditions, including a generalized interpretation of findings?	Y	
g.	A statement of which version (date) of the SGE specification the exploration was performed in accordance with?	Y	
h.	Statement of where geotechnical reports are available for review?	Y	
i.	Initials of personnel and dates they performed field reconnaissance, subsurface exploration and preparation of the soil profile?	Y	

VI.A. Soil Profile Checklist

Cover Sheet		(Y/N/X)	Notes:
14	Has a Legend been provided?	Y	
15	Have the following items been included in the Legend:		
a.	Symbols and usual descriptions for only the soil and bedrock types presented in the Soil Profile, as per the Soil and Rock Symbology Chart in Appendix D of the SGE?	Y	
b.	All miscellaneous symbols and acronyms, used on any of the sheets, defined?	Y	
c.	The number of soil samples for each classification that were mechanically classified and visually described in the current exploration?	Y	
16	Has a Location Map, showing the beginning and end stations for the project, been shown on the cover sheet, sized per the L&D3 Manual?	Y	
17	Have the station limits for each plan and profile sheet for projects with multiple alignments, or greater than 1500’, been identified in a table?	X	
18	Have the station limits for any cross section sheets been identified in the same table?	X	
19	Has a list of any structures for which structure foundation explorations been performed been identified in the same table?	X	
20	If sampling and testing for a scour analysis was performed, has this data been shown in tabular form?	X	
21	Has a summary table of test data for all roadway and subgrade boring samples been shown?	X	
22	If borings from previous subsurface explorations are being used, has that data been shown in a separate table?	X	
23	In the summary table, has the data been displayed by roadway and subgrade boring in ascending stationing order for each roadway?	X	
24	Have the centerline or baseline station, offset, and exploration identification number been provided for each boring presented in the table?	X	

VI.A. Soil Profile Checklist

Cover Sheet	(Y/N/X)	Notes:
25 For each sample, has the following information been provided in the summary table:	X	
a. Sample depth interval?		
b. Sample number and type?		
c. N_{60}?		
d. Percent recovery?		
e. Hand Penetrometer?		
f. Percentage of aggregate, coarse sand, fine sand, silt, and clay size particles?		
g. Liquid limit, plastic limit, plasticity index, and water content, all rounded to the nearest percent or whole number?		
h. ODOT classification and Group Index?		
i. Visual description of samples not mechanically classified, including water content, and estimated ODOT classification with 'Visual' in parentheses?		
j. Sulfate Content test results?		
26 Have all undisturbed test results been displayed in graphical format on the sheet prior to the plan and profile sheets?		
Surface Data	(Y/N/X)	Notes:
27 Has the following information been shown on each roadway plan drawing:		
a. Existing surface features described in Section 702.5.1?	Y	
b. Proposed construction items, as described in Section 702.5.2?	Y	
c. Project and historic boring locations, with appropriate exploration targets and exploration identification numbers?	Y	
d. Notes regarding observations not readily shown by drawings?	X	
28 Have the existing ground surface contours been presented?	Y	
29 If cross sections are to be developed for stationing covered on a plan sheet, has an index for the appropriate cross section sheets been included on the plan sheet?	X	

VI.A. Soil Profile Checklist

Subsurface Data	(Y/N/X)	Notes:
30 Has all the subsurface data been presented in the form of a profile along the centerline or baseline, and on cross sections where applicable?	Y	
31 Have the graphical boring logs been correctly shown, as follows:		
a. Location and depth of boring indicated by a heavy dashed vertical line?	Y	
b. Exploration identification number above the boring?	Y	
c. Logs indicate soil and bedrock layers with symbols 0.4" wide and centered on the heavy dashed vertical line where possible?	Y	
d. Bedrock exposures with 0.4" wide symbols, but without a heavy dashed vertical line?	Y	
e. Soil and bedrock symbols as per ODOT Soil and Rock Symbology chart (SGE - Appendix D)?	Y	
f. Historical borings shown in same manner with the exploration identification number above the boring?	X	
32 Have the proposed groundline and existing groundline been shown on the profile view, according to ODOT CADD standards?	Y	
33 Have the locations of the proposed structure foundation elements been shown on the profile view?	Y	
34 Have the offsets from centerline or baseline been indicated above the borings in the profile view?	Y	
35 Have borings located immediately adjacent to the centerline or baseline and considered representative of centerline or baseline subsurface conditions been referenced directly to the centerline or baseline?	X	
36 Have offset borings in or near the same elevation interval of a centerline or baseline boring been plotted either on a cross section or immediately above or below the centerline boring in a box containing an elevation scale?	Y	
37 Have cross-sections been developed to show subsurface conditions disclosed by a series of borings drilled transverse to centerline or baseline?	X	

VI.A. Soil Profile Checklist

Subsurface Data		(Y/N/X)	Notes:
38	Have the existing and proposed groundlines been displayed on cross section sheets according to ODOT CADD standards?	X	
39	Have bedrock exposures shown on the cross sections been plotted along the contour of the cross section?	X	
40	Has the following information been provided adjacent to the graphical logs or bedrock exposure:		
a.	Thickness, to the nearest inch, of sod/topsoil or other shallow surface material written above the boring (with corresponding symbology at top of log)?	Y	
b.	Moisture content, to nearest whole percent, with the bottom of the text aligned with the bottom of the sample? Label this column as 'WC' at bottom of the boring.	Y	
c.	N_{60}, aligned with the bottom of sample? Label column as 'N_{60}' at bottom of boring.	Y	
d.	Free water indicated by a horizontal line with a 'w' attached, and water level at the end of drilling indicated by an open equilateral triangle, point down?	Y	
e.	Complete geologic description of each bedrock unit, including unit core loss, unit RQD, SDI, and compressive strength test results? (Do not present geologic descriptions for structure borings for which this information is presented on the boring logs as described in 703.3)	Y	
f.	Visual description of any uncontrolled fill or interval not adequately defined by a graphical symbol?	Y	
g.	Organic content with modifiers, per 603.5?	N	
h.	Designate a plastic soil with moisture content equal to or greater than the liquid limit minus three with a 1/8" solid black circle adjacent to the moisture content?	Y	
i.	Designate a non-plastic soil with moisture content exceeding 25% or exceeding 19% but appearing wet initially, with a 1/8" open circle with a horizontal line through it adjacent to the moisture content?	Y	
j.	The reason for discontinuing a boring prior to reaching the planned depth indicated immediately below the boring?	Y	

VI.A. Soil Profile Checklist

Boring Logs	(Y/N/X)	Notes:
41 Have the boring logs of all structure borings, all geohazard borings, and any roadway borings drilled in the vicinity of the structures or geohazard been shown on the boring log sheets following the plan and profile sheets? (Create the logs in accordance with 703.3)	Y	
42 Have the boring logs been developed by integrating the driller's field logs, laboratory test data, and visual descriptions?	Y	
43 Has the following boring information been included in the heading of each boring log:		
a. Exploration identification number?	Y	
b. Project designation (C-R-S) and PID?	Y	
c. Structure File Number (if applicable) and project type.	Y	
d. Centerline or baseline name, station, offset, and surface elevation?	Y	
e. Coordinates?	Y	
f. Method of drilling?	Y	
g. Date started and date completed?	Y	
h. Method and material (including quantity) used for backfilling or sealing, including type of instrumentation, if any?	Y	
i. Date of last calibration and drill rod energy ratio (ER) in percent for the hammer system(s) used?	Y	
44 Has the following boring information been included in each boring log:		
a. A depth and elevation scale?	Y	
b. Indication of stratum change?	Y	
c. Description of material in each stratum?	Y	
d. Depth of bottom of boring?	Y	
e. Depth of boulders or cobbles, if encountered?	X	
f. Caving depth?	X	
g. Water level observations?	Y	
h. Artesian water level and height of rise?	X	
i. Heaving sand?	X	
j. Cavities or other unusual conditions?	X	
k. Depth interval represented by sample?	Y	
l. Sample number and type?	Y	
m. Percent recovery for each sample?	Y	
n. Measured blow counts for each 6 inches of drive for split spoon samples?	Y	
o. N_{60} to the nearest whole number?	Y	
p. Hand penetrometer?	Y	

VI.A. Soil Profile Checklist

Boring Logs		(Y/N/X)	Notes:
q.	Particle-size analysis?	Y	
r.	Liquid limit, plastic limit, plasticity index?	Y	
s.	Water content?	Y	
t.	ODOT soil classifications, with "V" in parentheses for those samples that are not mechanically classified?	Y	
u.	Top of bedrock and bedrock descriptions?	Y	
v.	Run rock core percent recovery?	Y	
w.	Run RQD?	Y	
x.	Unit rock core percent recovery?		
y.	Unit RQD?	Y	
z.	SDI, if applicable?	Y	
aa.	Rock compressive strength test results, if applicable?	Y	

LABORATORY TEST STANDARDS

STANDARD	REFERENCE NUMBER
I. Soil/Rock Testing	
Description and Identification of Soils (Visual-Manual Procedures)	ASTM D 2488
Classification of Soils for Engineering Purposes (USCS).	ASTM D 2487
Laboratory Determination of Water (Moisture) Content of Soil and Rock	ASTM D 2216
Classification for Sizes of Aggregate for Road and Bridge Construction	ASTM D 488
Liquid Limit, Plastic Limit, and Plasticity Index of Soils	ASTM D 4318
Shrinkage Factors of Soils by Mercury Method	ASTM D 427
Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	ASTM D 2974
Specific gravity of Soils	ASTM D 854
Direct Shear Test of Soils under Consolidated Drained Conditions	ASTM D 3080
Particle-Size Analysis of Soils	ASTM D 422
Unconfined Compressive Strength of Cohesive Soils	ASTM D 2166
Compressive Strength of Intact Rock Core Specimens	ASTM D 7012
Slake Durability Index of Shale/Similar Weak Rock Test	ASTM D 4644
Point Load Test of Rock Core Specimens	ISRM* / ASTM D5731
CBR (California Bearing Ration) of Laboratory-Compacted Soils	ASTM D 1883
Laboratory Compaction Characteristics of Soil using Standard Effort	ASTM D 698
Laboratory Compaction Characteristics of Soil using Modified Effort	ASTM D 1557
One-Dimensional Consolidation Properties of Soils	ASTM D 2435
One-Dimensional Swell or Settlement Potential of Cohesive Soils	ASTM D 4546
Ph of Soil	ASTM D 4972

*ISRM – International Society for Rock Mechanics

II. Concrete Testing

Compressive Strength for Cylindrical Concrete Specimens	ASTM C-39
Acid-Soluble Chloride in Mortar and Concrete	ASTM C 1152

CLASSIFICATION OF SOILS

Ohio Department of Transportation

(The classification of a soil is found by proceeding from top to bottom of the chart. The first classification that the test data fits is the correct classification.)

SYMBOL	DESCRIPTION	Classifcation		LL_O/LL x 100*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
		AASHTO	OHIO							
○	Gravel and/or Stone Fragments	A-1-a			30 Max.	15 Max.		6 Max.	0	Min. of 50% combined gravel, cobble and boulder sizes
○∴	Gravel and/or Stone Fragments with Sand	A-1-b			50 Max.	25 Max.		6 Max.	0	
F S	Fine Sand	A-3			51 Min.	10 Max.	NON-PLASTIC		0	
⁙	Coarse and Fine Sand	--	A-3a			35 Max.		6 Max.	0	Min. of 50% combined coarse and fine sand sizes
▥○	Gravel and/or Stone Fragments with Sand and Silt	A-2-4				35 Max.	40 Max.	10 Max.	0	
		A-2-5					41 Min.			
▤○	Gravel and/or Stone Fragments with Sand, Silt and Clay	A-2-6				35 Max.	40 Max.	11 Min.	4	
		A-2-7					41 Min.			
▥	Sandy Silt	A-4	A-4a	76 Min.		36 Min.	40 Max.	10 Max.	8	Less than 50% silt sizes
+ + + +	Silt	A-4	A-4b	76 Min.		50 Min.	40 Max.	10 Max.	8	50% or more silt sizes
▧	Elastic Silt and Clay	A-5		76 Min.		36 Min.	41 Min.	10 Max.	12	
▨	Silt and Clay	A-6	A-6a	76 Min.		36 Min.	40 Max.	11 - 15	10	
▤	Silty Clay	A-6	A-6b	76 Min.		36 Min.	40 Max.	16 Min.	16	
▩	Elastic Clay	A-7-5		76 Min.		36 Min.	41 Min.	≤LL-30	20	
▦	Clay	A-7-6		76 Min.		36 Min.	41 Min.	>LL-30	20	
■+■	Organic Silt	A-8	A-8a	75 Max.		36 Min.				W/o organics would classify as A-4a or A-4b
■▦■	Organic Clay	A-8	A-8b	75 Max.		36 Min.				W/o organics would classify as A-5, A-6a, A-6b, A-7-5 or A-7-6

MATERIAL CLASSIFIED BY VISUAL INSPECTION

▧	Sod and Topsoil	^ ˅ >	Uncontrolled Fill (Describe)	⁙	Bouldery Zone	◙	Peat, S-Sedimentary W-Woody F-Fibrous L-Loamy & etc
▩	Pavement or Base						

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

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

1) STRENGTH OF SOIL:

Non-Cohesive (granular) Soils - Compactness

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

2) COLOR :

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

3) PRIMARY COMPONENT

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

Cohesive (fine grained) Soils - Consistency

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

4) COMPONENT MODIFIERS:

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

5) Soil Organic Content

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

6) Relative Visual Moisture

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

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

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

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

3) WEATHERING

Description	Field Parameter
Unweathered	No evidence of any chemical or mechanical alternation of the rock mass. Mineral crystals have a bright appearance with no discoloration. Fractures show little or no staining on surfaces.
Slightly weathered	Slight discoloration of the rock surface with minor alterations along discontinuities. Less than 10% of the rock volume presents alteration.
Moderately weathered	Portions of the rock mass are discolored as evident by a dull appearance. Surfaces may have a pitted appearance with weathering “halos” evident. Isolated zones of varying rock strengths due to alteration may be present. 10 to 15% of the rock volume presents alterations.
Highly weathered	Entire rock mass appears discolored and dull. Some pockets of slightly to moderately weathered rock may be present and some areas of severely weathered materials may be present.
Severely weathered	Majority of the rock mass reduced to a soil-like state with relic rock structure discernable. Zones of more resistant rock may be present, but the material can generally be molded and crumbled by hand pressures.

5) TEXTURE

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

4) RELATIVE STRENGTH

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

6) BEDDING

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

7) DESCRIPTORS

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

8) DISCONTINUITIES

a) Discontinuity Types

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

b) Degree of Fracturing

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

c) Aperture Width

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

d) Surface Roughness

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

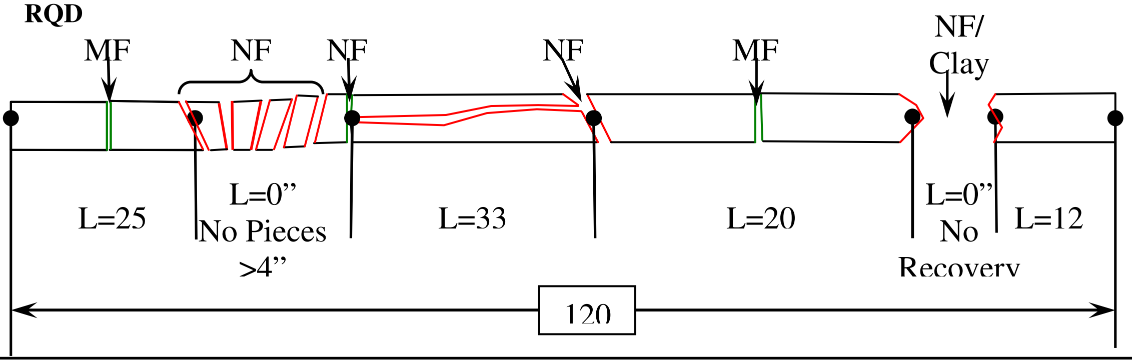
10) LOSS

$$Run\ Loss = \left(\frac{L_R - R_R}{L_R}\right) * 100 \quad Unit\ Loss = \left(\frac{L_U - R_U}{L_U}\right) * 100$$

L_R=Run Length R_R=Run Recovery

L_U=Rock Unit Length R_U=Rock Unit Recovery

9) RQD



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

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